EVALUATION OF THE IMPACTS OF HARBOUR ENGINEERING

ANIBARE BAY
REPUBLIC OF NAURU (RON)

May 2001 *SOPAC Technical Report 316*

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EXECUTIVE SUMMARY

This report presents the preliminary results of a coastal engineering, and environmental evaluation, of design parameters, for a fishing boat harbour facility at Anibare Bay, Republic of Nauru.

The Government of the Republic of Nauru (RON) requested the South Pacific Applied Geoscience Commission (SOPAC) Secretariat, Fiji Islands, to evaluated the development, and comment on the impacts on the harbour facility on sedimentation and erosion of the adjacent coastline, and also, advise on maintenance of the constructed facility at the development site.An in-country site visit was made between 28th February 2000 - 7th March 2000. A follow-up visit was also made between 5th - 7th September 2000, during which time project **NR 2000.007 (Assessment of Coastal Engineering Design of a Rip-Rap Facility at Yaren District)** was executed.

This follow-up visit was done, so as to gain further information and insight on response of the beach-coastal system to the engineered facility. It also provides additional and new time-series observation data. This new information has been incorporated into the present report for the benefit of RON.

This project is executed on behalf of the Nauru Fisheries and Marine Resources Authority (NFMA), Government of the Republic of Nauru (RON). The purpose of this study can be defined as follows:

1. *assess the current coastal development;*
2. *evaluate any present coastal problems;*
3. *assess any future shoreline management problems and*
4. *recommend and/or propose appropriate monitoring and maintenance strategies and system/s for the development site.*

Following the visit, and after briefing of officials of Nauru Fisheries and Marine Resources Authority, SOPAC Secretariat indicated that it would provide appropriate guidelines, for shoreline management in the form of technical reports. This report is the final document. The report presents information regarding the impacts of the harbour development. and contains recommendations for appropriate shorefront development, in particular guidelines for preparing EA and EIA for the coastal development. Key engineering and environmental issues are also discussed and highlighted in the text as well as guidelines for preparation of a full EIA. This project was funded and executed under **SOPAC Task NR 99.045** (RON - Appendix I).

Anibare District is located in the Eastern part of Nauru. The development site is located in Anibare Channel, on the southern half of the Anibare Bay. The developed coastline is just about 700 m North of the Mennen Hotel facility. RON received bi-lateral funding from the Government of Japan (GOJ), for the development of a port and harbour facility at Anibare Bay. The development site chosen is a former cut and excavated channel in Anibare Bay, which was also used for fishing activities on the East coast of the Island. The purpose of this harbour facility is to provide RON with additional docking, launch and berthing facilities to support their offshore tuna fishing fleet.

Coastal infrastructure and development, like this one, on a small island state, like RON, can impact adversely on coastal processes. In addition, deleterious impacts can also result along the natural and built shorelines at the site, and in the vicinity of the development. Consequently, it is important to monitor development activities and their impacts, both positive and negative, on the coast. In this way, negative and deleterious impacts of the development can be captured immediately and remedied, where possible, in a timely and appropriate fashion. Further, information gained from such monitoring can be useful for future coastal development and engineering design of similar and related facilities on the island. Since it is the intention of RON to develop another harbour facility on the western side of the island, information gained from this monitoring study can be crucial to optimum planning and design of that new facility The present study was set-up for this very purpose, with the objective of advising RON with timely and appropriate advice on the impacts of the harbour facility and strategies for addressing these impacts. Since RON is a small island, with limited land area, the loss of any land or coastal property represents a significant one. In addition, developing economies, like RON, can be seriously affected by damage to or loss of civil infrastructure and residential facilities from "silent" natural hazards, like coastal erosion. Therefore, addressing these types of coastal development problems is of paramount importance to Nauru's coastal communities and for the livelihood and well-being of residents and the nation as a whole. In the context of sustainable natural resource management, coastal development and it's impacts on coastal erosion is also an important process that need to be addressed, if future generations are to derive benefits from coastal resources, like beaches and reefs.

The harbour development plans were consulted, for assessment of dimensions and quantities of the various components of the Anibare Harbour facility (Tetra, 1999). Environmental (including geological and geotechnical) information and data were collected during site visits to the eroding coast. This included wave and littoral information, beach sediment characteristics, erosion characteristics, and documentation of damage to any critical facilities and infrastructure. Beach sediments were described according to American Society for Testing Material (ASTM, 2000) guidelines, which are in accordance with the Unified Soil Engineering Classification System. Rock classification and descriptions are based on ASTM (2000) guidelines and conforms to the international criteria set out by the International Society for Rock Mechanics (ISRM, 1981) and the Engineering Group of the Geological Society of London guidelines (Smith and Collins, 1993). Beach profiles were measure with a Sokkia Automatic level, using a Sokkia survey staff, a Brunton compass and a hand-held, Garmin Global Positioning System (GPS). Erosion scarps were measured with these surveying instruments. Steel lamp poles along the main coastal road were used as benchmarks for surveying beach profiles. Beach profiles elevations were corrected to the Nauru's Chart Datum, by precise levelling loops, using the Sokkia Automatic Level, from surveyed benchmarks at the development site. Several benchmarks, which were established for construction of the harbour, were used in this assessment. Wind speeds and direction was measured with a digital, hand-held anemometer.

Positions in the field were determined with a Garmin hand-held GPS. Information on waves was also obtained from Tetra (1998) and vertical, colour, 1992 stereopairs/aerial photographs. The dolomite limestone rock strength was measured in-situ, with an ELE Schmidt L-Type Hammer, while concrete strength was measured with an ELE Concrete Testing Hammer. Rip-rap/armourstone description and revetment evaluation are based design on criteria of CIRIA (1991) and Pilarczyk (1996), Latham (1998), Geological Society of London (GSL, 1999). Design analysis has been produced after Numerical Analysis with International Institute for Hydraulic Engineering, Delft University of Technology (IHE-Delft, 1999), Coastal and River Engineering Support System (CRESS) and the U. S. Army Corps of Engineers, Automatic Coastal Engineering System (ACES). Computations of rip-rap dimensions, armourstone stability and wave run-up are based on numerical equations of van de Meer (1987 and 1998) and are modifications and improvements of the typical Hudson (1958) formula. Computations are therefore for random wave attack, more typical in the natural environments.

In summary, the coast is part of an emergent, Holocene reef-carbonate system, with the beach being comprised entirely of carbonate sediments developed on phosphate-rich, cavernous, dolomite limestone bedrock. The coastline is partly rocky with classic karst limestone pinnacles found throughout the bay. The reef is a coral dominated system and is narrow and well-flushed, with many closely-spaced reef channels. The coastline at Anibare Bay is an active and dynamic one. The relatively coarse admixture of abraded sand and gravel and highly abraded karts pinnacles testify to this. The beach at the development site is moderately steep and has experienced erosion in the recent past and show sign of recent and current erosion, with fresh erosion scarps. The harbour development appears to have increased this erosion at the localized level, at the adjacent, undeveloped coastal segments. The topographic elevation of the coastal land areas are relatively low with respect to CDL and MSL and under EHWST or during windy and low pressure systems, when large (3 m +) waves approach shore, from the East, the beach, coastal road and adjacent areas can be easily overtopped. The relatively narrow and almost flat backreef and reef flat, together with the numerous closely-spaced reef channels make it almost impossible to dissipitate significant wave energy and prevent overtopping during these conditions. In addition, the almost featureless backreef cannot trap sediments entrained in longshore currents and therefore, sediments removed from the local areas can be completely lost from the coastal system in Anibare Bay.

The harbour development acts as a large groin, which breaks the continuity and smoothness of the concave Anibare Bay. As a result it interrupts Southerly longshore currents and will cause erosion of downdrift areas (to the South). By its very nature, the harbour also acts as a headland, protruding into the bay. As a result of this "headland-like" morphology, wave diffract around it, and cause much agitation and disturbance of beach sediments immediately North and South of the harbour. Therefore, the facility can cause erosion on both the North and South aspects of the harbour.

With respect to the concrete breakwaters built to protect the harbour and mooring basin, these structures are already overtopped by 3 m high spring tide waves. While the design and construction firm (Tetra, 1999) indicated that a 50-year, of 5.34 m design wave was used to design the harbour, the fact that a 3 m high offshore wave, after undergoing decay over the reef crest, can overtop the main breakwater, raises some concern as to what acceptable risks were allowed/selected for this facility.

To that end some numerical analysis was performed for the facility. Numerical computation were done for only the design (5.34 m) and average wave heights (3 m) chosen for the harbour facility and under EHWST conditions, as these represent extreme and common wave height respectivley. The reef crest was modeled as a dynamically stable, submerged breakwater, situated below MSL and 1.5 m below CDL. In addition it has a relatively straight offshore slope of 25°-40° and the backreef is unusually flat and smooth for a reef environment, with a constant seaward grade of less than 1° (see Section 3.10). Wave transformation was done using numerical equations proposed for a dynamically stable submerged breakwater (IHE-Delft, 1999).

For a 5.34 m, 1-in-50-year wave height proposed by Tetra (2000), wave period of 10 sec, a reef crest of about 12 m wide, a forereef slope of 1: 1.2, in fore reef water depth of about 10 m (seaward of the reef crest), under EHWST (freeboard level of -4.14 m; with the reef crest at –1.5 m below CDL) and under Easterly (the modal) wave approach, the transformed wave computed was 3.7 m.

This wave transformation corresponds to 30 % decay in wave height across the reef crest. Interestingly enough, this is consistent with a wave that will run-up and overtop the coastal road under EHWST, and also that, which has been observed by residents, at the site, for more than a decade.

For such a wave height, the size and density of boulders required to maintain stable structural conditions under the design/ and transformed wave, in the backreef, and on parts of the facility (e.g. on the groin and spending beach rip-rap), may be larger or more dense than those specified.

For the local dolomite limestone used at the harbour site, which is dense, but porous, and with an estimated unit weight of 2650 kg/m^3, and under a 3.7 m transformed wave, during EHWST, the nominal stone diameter required would be about 1 m. This is about 3-5 times the diameter specified by Tetra (2000). Tetra's (2000) diameter is 500-1000 kg/pc or about 18-35 % of that computed by the author (assuming a rock density of 2650 kg/m^3).

If a 3.7 m high transformed wave impacts on the vertical seaward face of the main breakwater, under an EHWST, overtopping of the structure will be about 0.457 m^3/sec. At mean water level (1.57 m above CDL) overtopping will be 0.13 m^3/sec.

In addition, the navigational channel will not cause significant wave decay, as would the adjacent intact reef crest. This is because it was dredged to –2.5 m below CDL (1 m deeper than the level of the existing reef crest), with a 30 m wide funnel-like entrance that narrows to 20 m. The freeboard height above EHWST level is therefore –5.14 m, estimated from Tetra (2000) designs, while the seaward channel slope is 1: 16.

If a 5.34 m high wave break over the navigational access channel, the transformed wave will be about 4 m high or about 0.6 m higher than the transformed wave over the intact reef crest (3.4 m). This will then run-up on the spending beach rip-rap, and enter the mooring area, causing choppy conditions to develop within the harbour. Despite the fact that there is a spending beach of rip-rap, some reflection and refraction will occur on the landward side of the access channel.

It is therefore important and necessary to cater for routine and regular maintenance of the spending beach rip-rap so as to ensure that any rip-rap dislodgement, erosion or damage is repaired.

Some numerical analysis was also performed for average wave climate for the same reef architecture and harbour design.

For an average offshore non-broken wave height of 3 m (within a 1-year return interval computed by Tetra, 2000), with a 6 sec period, the transformed wave on EHWST, over the same reef morphology will be much smaller, at 2.4 m or 20 % decay.

For such a transformed wave, the required rip-rap for the groin, under EHWST, should be at least 0.65 m diameter or 741 kg (assuming a rock density of 2650 kg/m^3). The rip-rap required for stability at the spending beach should be 0.3 m or 60 kg, also assuming a rock density of 2650 kg/m^3. Overtopping of the main breakwater by a 2.4 m transformed wave will be about 0.07 m^3/sec, smaller, but nevertheless, noticeable.

Management of built shorelines, like those in Anibare Bay, is a dynamic process based on assessments and re-assessments and therefore, cannot be pursued by targeting specific activities, continuously, through time. Shoreline management strategies must be developed to reflect the current and future/forecasted needs. Forecast should also be for the short-term, medium-term and long-term, and therefore, strategies must be developed which reflect these changing needs through time. However, management strategies must reflect site dynamics, which may be summarized as follows:

1. The development site is part of a dynamic open-ocean coast;
2. The beach is protected by a narrow (12 m wide) fringing coral reef, with many closely spaced reef channels;
3. The beach is narrow and of carbonate sand, with a moderate slope;
4. The beach and shoreline has been subject to natural erosion in the past and recent years;
5. Surf zone hydraulics show that on breaking waves run-up the backreef, unto the beach, without any further decay;
6. Backwash is strong across the backreef and on mean or low tide drains the backreef;
7. The entire harbour acts as a groin along Anibare bay and interrupts Southerly longshore currents;
8. The facility also diffract Easterly approaching waves to the North and South;
9. The harbour construction has therefore already caused alteration of surf zone hydraulics and caused local eddys to develop;
10. Eddys already cause local scouring at the toe of various harbour structures (breakwaters, rip-rap, groin and spending beach);
11. The beach and shoreline has been affected by harbour construction and erosion has exacerbated;
12. Scouring and erosion will continue in the immediate future;
13. Fine sediments (sand and silt) will become easily suspended and eroded from adjacent beaches, in response to scouring and eddying under wave attack;
14. Erosion is on both the updrift and downdrift aspect;
15. Updrift erosion has resulted from wave diffraction around the structure from Easterly waves;
16. Downdrift erosion is due to interruption of longshore currents and sediment transport and sediment starvation;
17. The downdrift aspect is more eroded than the updrift side;
18. The beach on the updrift aspect has completely disappeared and the scoured underlying bedrock is exposed;
19. The main breakwater is overtopped by present EHWST and will continue to be overtopped;
20. It is expected that any waves generated by low-pressure systems and which affects the harbour site, will also overtop the harbour breakwater and run-up on land across the road;
21. Based on analysis of Tetra's (2000) design wave, their estimated transformed wave (1.9 m) over the reef crest is smaller than those wave heights observed by mariners in Nauru (3 m +);
22. Estimates of a transformed wave over the reef crest (modeled as a submerged breakwater) show a higher wave height (3.4 m) than that predicted by Tetra (2000), and is consistent with those observed by maritime personnel in Nauru;
23. The above suggests that there will be greater overtopping and run-up for the harbour site than those predicted by Tetra (2000);
24. The navigational channel access is deeper than the water over the reef crest and will therefore facilitate large waves to enter the harbour and mooring basin, especially under EHWST;

25. Waves entering the access channel, in the backreef, will be larger than those North or South of the habour facility; and
26. Larger waves entering the mooring basin will create some choppy conditions within the harbour, even though some dissipitation will occur on the spending beach (rip-rap) slope.

In relation to the above, and to prevents any further erosion or exacerbation of shoreline retreat, the following programmes will be needed at the harbour development site. Failure to implement these will allow erosion or damage to coastal facilities and infrastructure to go un-noticed, which may cause further negative impacts along the shorefront. Recommendation include:

1. Eroding sections of the coastline need to be protected immediately if erosion is to be arrested or stopped;
2. If eroding coasts at the harbour site is left un-protected, it will cause further loss of coastal soils, beach sediments and damage to the adjacent road or other infrastructure;
3. Protection strategies should be for the immediate and medium-long term;
4. A bio-engineering system of coastal protection should be employed;
5. To prevent overtopping of coastal areas, a rip-rap system could be utilized along the edge of the coastline, at the land-sea interface;
6. Biological protection can mean planting of locally adaptable coastal tree and shrub species;
7. It would be best to select and plant species of flora which are adaptable to the existing coastal conditions, especially those which grow along Nauru's coastline (see SPREP, 1998);
8. I would recommend the use of geotextile or erosion mats for erosion control, if available, as these can trap sediments within their structure;
9. Any geotextile selected should be appropriate for the hydraulic and tropical UV conditions;
10. Dolomite limestone rip-rap can also be used for coastal protection;
11. Limestone rip-rap is also recommended because it is cheap, available locally and easy to use;
12. The local limestone is also of good/suitable density for coastal engineering protection structures;
13. Attempts should be made to select boulders without phosphate residue (to prevent eutrophication in coastal waters);
14. Rip-rap should be sized based on wave and surf-zone hydraulics;
15. A coastal monitoring programme should be put in place to periodically assess any changes to the coastline at the harbour site;
16. Monitoring will facilitate rapid repair to damaged infrastructure, facilities and eroding coasts;
17. Monitoring should be beach profiling, beach sediment sampling and coastal littoral hydraulic assessment;
18. Visual inspection of failure characteristics and wave run-up/overtopping should be documented and photographed when they occur;
19. This monitoring should be at least quarterly in the first 3 years, which can be decreased to twice yearly thereafter;
20. The coastal monitoring data should be reviewed as soon as possible and coastal management strategies modified/revised to reflect any changes or new developments at the site;
21. All beach profiles and surveys should be levelled from Nauru's suryeying benchmarks;
22. The data collected should be archived in a database for easy access and retrieval;
23. *SOPAC Training Reports 84* should be consulted for details of set-up of a beach monitoring programme;
24. Future coastal developments should be preceded by an environmental impact assessment (EIA);
25. An EIA, if properly done, would identify problems and positive attributes of the development and would assist developers and project managers in planning for and decreasing any deleterious impacts at the project site;

26. The report also provides some general guidelines for conducting an EIA and the EIA process.

KEYWORDS: Anibare District, Republic of Nauru, port and harbour engineering, coastal development, coastal protection, erosion assessment, coastal monitoring.

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1.0 INTRODUCTION

1.1 Background

This report presents the preliminary results of a coastal engineering, and environmental evaluation, of design parameters, for a fishing boat harbour facility at Anibare Bay, Republic of Nauru (Figures 1-3).

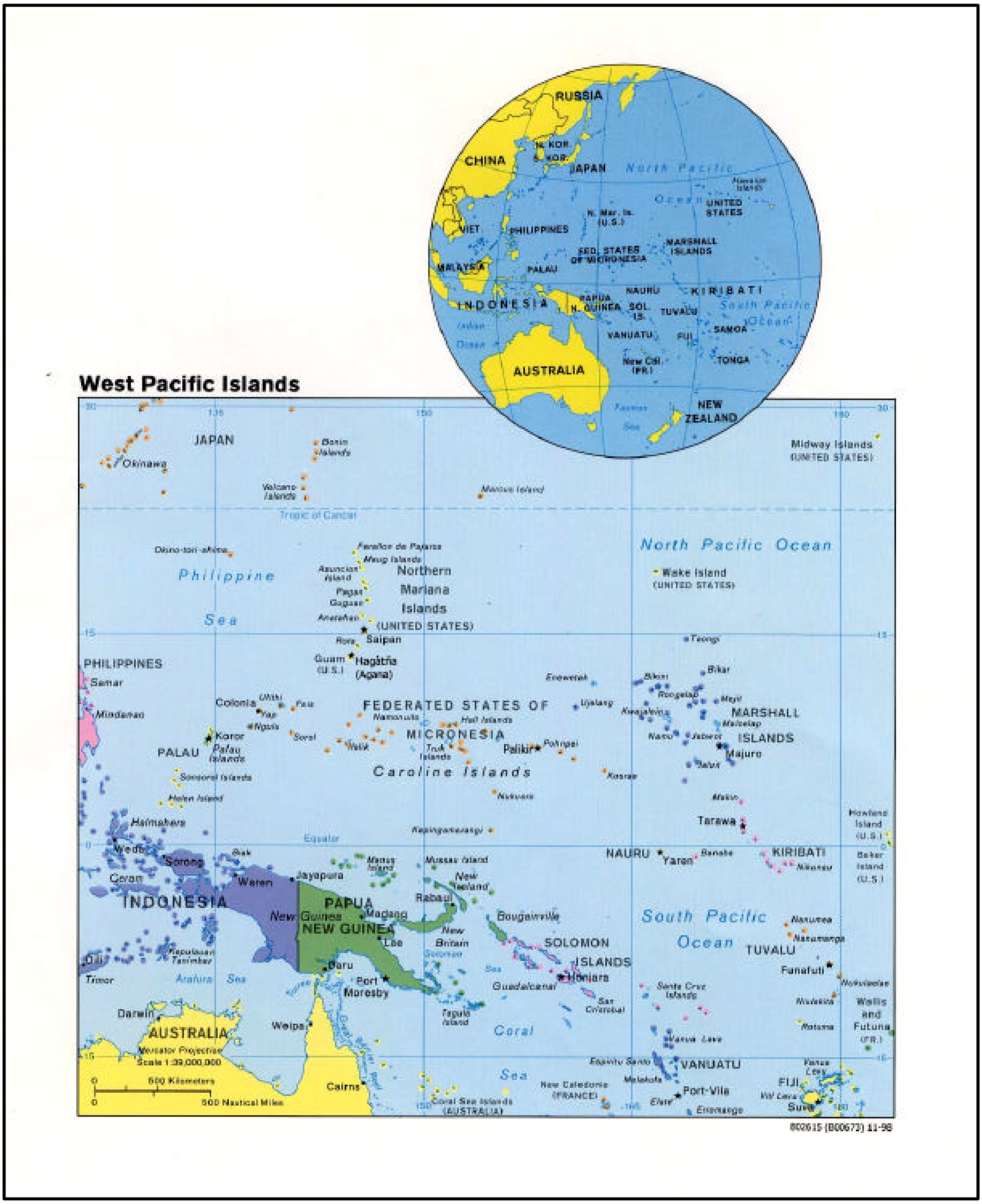

Figure 1. Map of the South Pacific region and the Republic of Nauru (RON).

The Government of the Republic of Nauru (RON) requested the South Pacific Applied Geoscience Commission (SOPAC) Secretariat, Fiji Islands, to evaluated the development, and comment on the impacts on the harbour facility on sedimentation and erosion of the adjacent coastline, and also, advise on maintenance of the constructed facility at the development site.

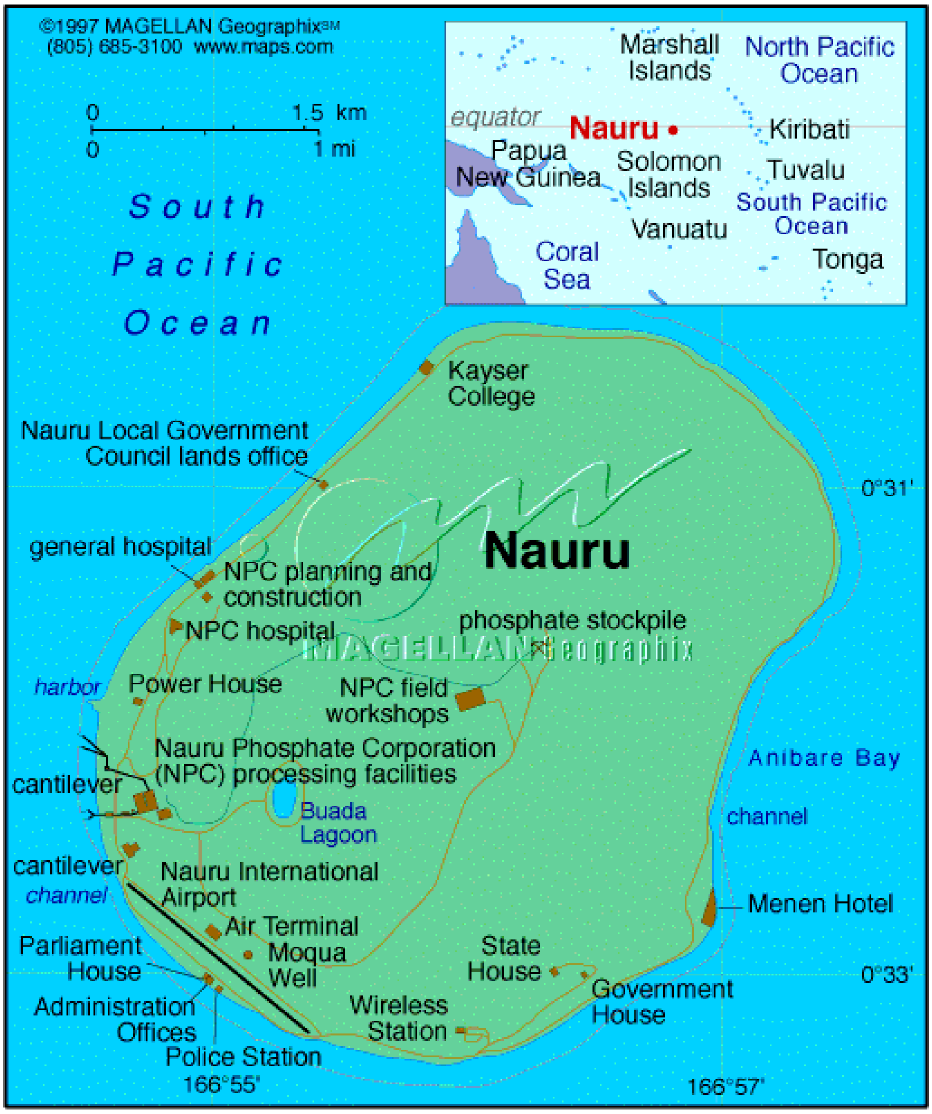

Figure 2. Location map of Nauru (Magellan Geographix, 2000).

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This project is executed on behalf of the Nauru Fisheries and Marine Resources Authority (NFMA), Government of the Republic of Nauru (RON).

1.2 Purpose of the Study

The purpose of this study can be defined as follows:

- ✓ assess the current coastal development;
- ✓ evaluate any present coastal problems;
- ✓ assess any future shoreline management problems and
- ✓ recommend and/or propose appropriate monitoring and maintenance strategies and system/s for the development site (Figures 2 and 3).

Following the visit, and after briefing of officials of Nauru Fisheries and Marine Resources Authority, SOPAC Secretariat indicated that it would provide appropriate guidelines, for shoreline management in the form of technical reports.

This report is the final document, *SOPAC Technical Report 316*. The report presents information regarding coastal impacts of the harbour development and a full analysis of the shorefront development, for the said coastline.

In addition, this report contains recommendations for appropriate shorefront management, and guidelines for preparing EA and EIA for future coastal development. Key engineering and environmental issues are discussed with regards to the harbour design, construction and maintenance and are highlighted in the text. Implications for short and long-term management are also presented.

This project was funded and executed under **SOPAC Task NR 99.045** (RON - Appendix I).

1.3 Location of the Study Area

Anibare District is located in the Eastern part of Nauru. The development site is located in Anibare Channel, on the southern half of the Anibare Bay (Figures 2 and 3). The developed coastline is just about 700 m North of the Mennen Hotel facility (Figures 3 and 4).

1.4 Needs and Rationale

RON received bi-lateral funding from the Government of Japan (GOJ), for the development of a port and harbour facility at Anibare Bay.

The development site chosen is a former cut and excavated channel in Anibare Bay (Figures 3 and 4), which was also used for fishing activities on the East coast of the Island.

The purpose of this harbour facility is to provide RON with additional docking, launch and berthing facilities to support their offshore tuna fishing fleet.

Coastal infrastructure and development, like this one, on a small island state, like RON, can impact adversely on coastal processes. In addition, deleterious impacts can also result along the natural and built shorelines at the site, and in the vicinity of the development. Consequently, it is important to monitor development activities and their impacts, both positive and negative, on the coast.

In this way, negative and deleterious impacts of the development can be captured immediately and remedied, where possible, in a timely and appropriate fashion. Further, information gained from such monitoring can be useful for future coastal development and engineering design of similar and related facilities on the island.

Since it is the intention of RON to develop another harbour facility on the western side of the island, information gained from this monitoring study can be crucial to optimum planning and design of that new facility The present study was set-up for this very purpose, with the objective of advising RON with timely and appropriate advice on the impacts of the harbour facility and strategies for addressing these impacts.

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Therefore, addressing these types of coastal development problems is of paramount importance to Nauru’s coastal communities and for the livelihood and well-being of residents and the nation as a whole.

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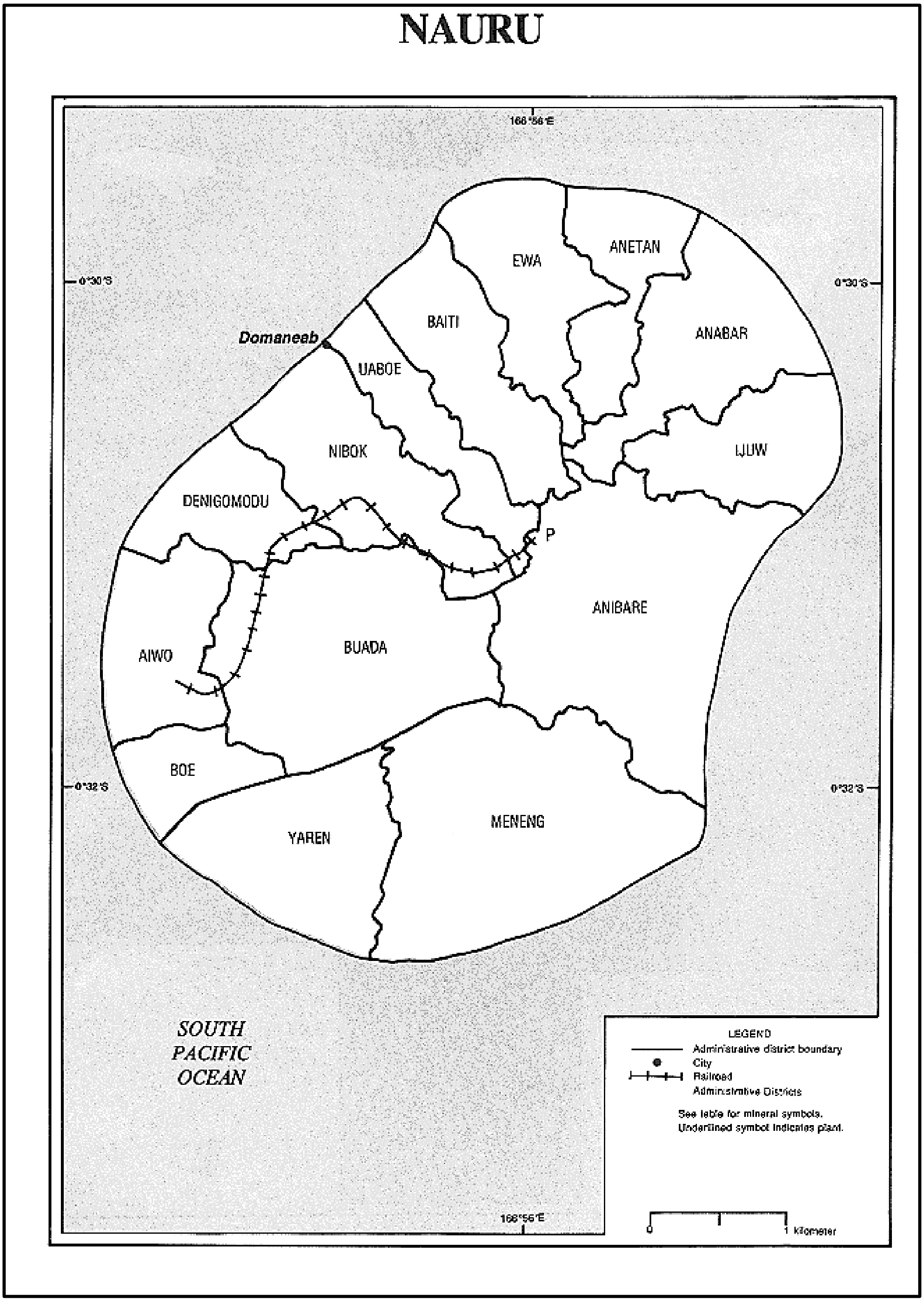

Figure 3. Nauru, showing districts and boundaries including Anibare District (Commonwealth Secretariat, 2000).

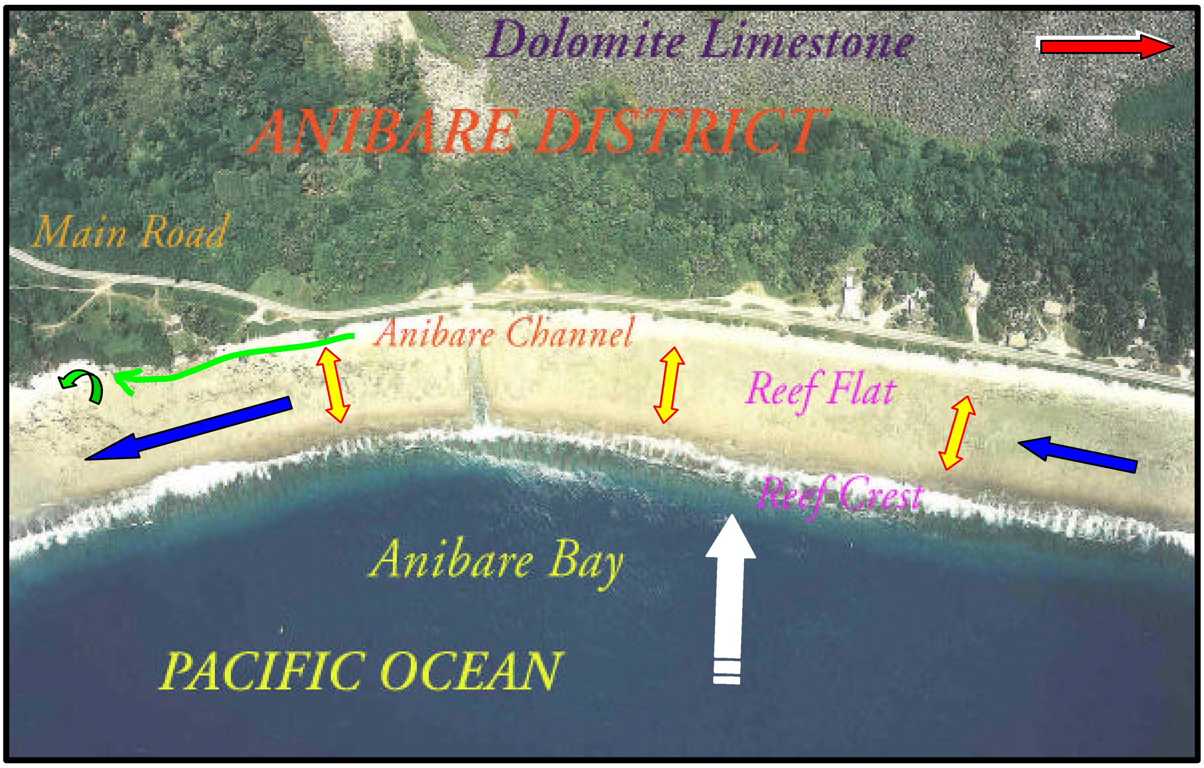

Figure 4. Anibare Bay and the Anibare Channel. This image is part of a 1992, 1:10,000, vertical stereopair collection, taken for topographic mapping of Nauru. This photo is supplied courtesy of Nauru Phosphate Co-operation. Green arrows show shoreline scouring/erosion path created by longshore currents; blue depict the dominant Southerly longshore currents; yellow arrows show swash/wave approach in backreef and gravity-induced backwash; and the white arrow show modal offshore wave approach. The red arrow at the top right points to North.

2.0 METHODOLOGY

2.1 The Harbour Plan and Design

The harbour development plans were consulted, for assessment of dimensions and quantities of the various components of the Anibare Harbour facility (Tetra, 1999; Figure 5).

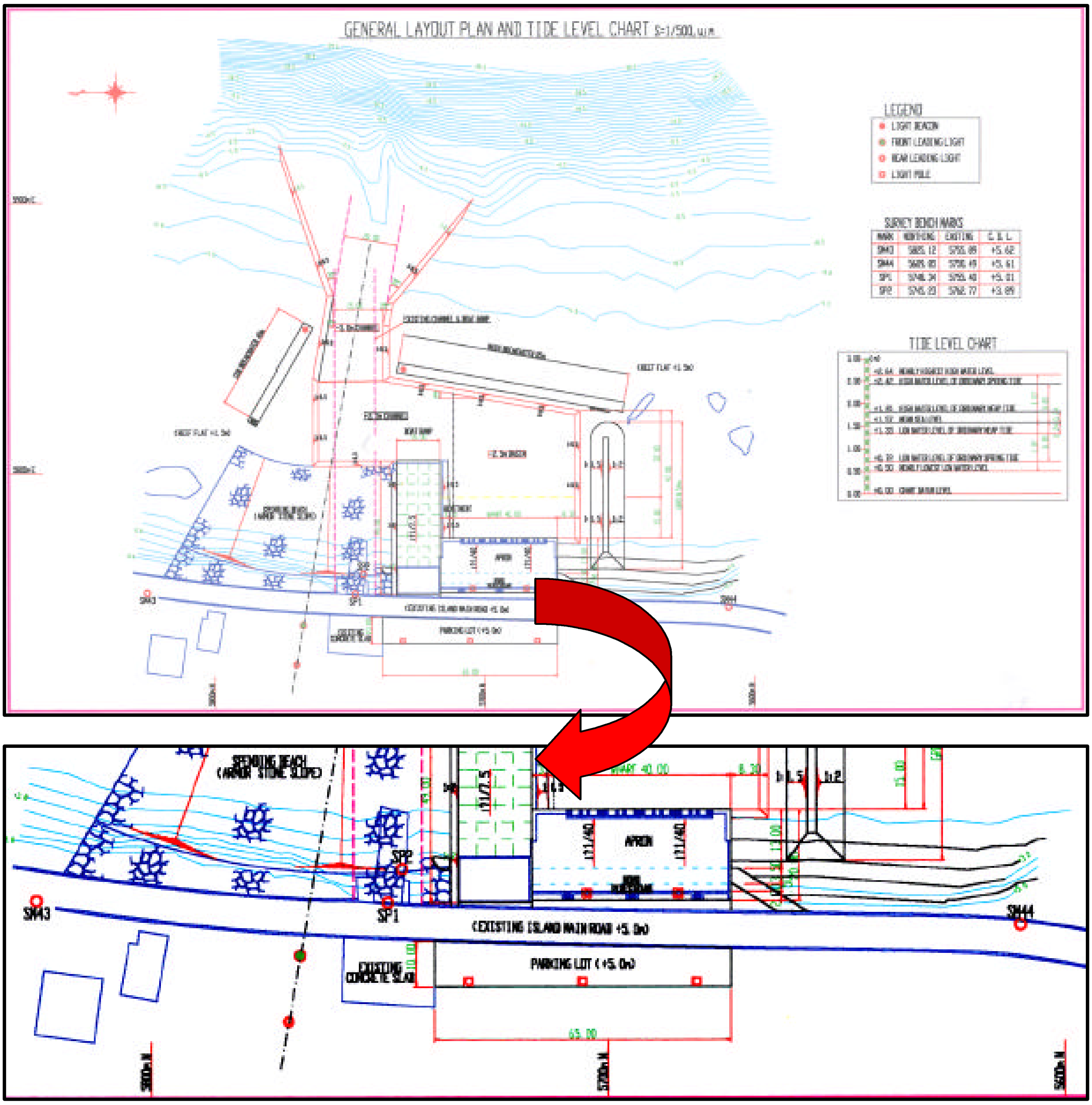

Figure 5. Top - engineering plan and design of the harbour facility; and Bottom - location of benchmarks at the site. SM 43, SM 44, SP 1 and SP 2 corresponds to beach profiles AB-1, AB-3, AB-2, AB-4 respectively).

Environmental (including geological and geotechnical) information and data were collected during site visits to the eroding coast.

This included wave and littoral information, beach sediment characteristics, erosion characteristics, and documentation of damage to any critical facilities and infrastructure.

Beach sediments were described according to American Society for Testing Material (ASTM, 2000) guidelines, which are in accordance with the Unified Soil Engineering Classification System.

Rock classification and descriptions are based on ASTM (2000) guidelines and conforms to the international criteria set out by the International Society for Rock Mechanics (ISRM, 1981) and the Engineering Group of the Geological Society of London guidelines (Smith and Collins, 1993).

Seven beach profiles were measured (Figure 5, Table 1 and Appendix II) with a Sokkia™ Automatic level, using a Sokkia™ survey staff, a Brunton™ compass and a hand-held, Garmin™ Global Positioning System (GPS). Erosion scarps were measured with these surveying instruments.

Steel lamp poles along the main coastal road (Figure 4) were used as benchmarks for surveying beach profiles (Figure 6). Beach profiles elevations were corrected to the Nauru's Chart Datum, by precise levelling loops, using the Sokkia™ Automatic Level, from surveyed benchmarks at the development site. Several benchmarks, which were established for construction of the harbour, were used in this assessment.

Figure 6. A steel lamp pole used as benchmark.

Table 1 presents a list of benchmarks for the site, and their elevations with respect to various water levels and chart datum. Table 2 presents the tidal levels for Nauru and their elevations with respect to Chart Datum (CDL). Figure 5 show the location of these benchmarks which were established for construction of the harbour facility. These are all located along the coastal roadway, at the site.

Table 1. Benchmarks and their elevation at Anibare Bay harbour. These benchmarks are shown in Figure 5 and the beach profiles corresponding to these are also identified in Figure 5.

Benchmark Number and Coordinates	*Height Above Chart Datum Level (CDL), m*
SM 43 (5825.12 N and 5755.89 E)	5.62
SM 44 (5609.85 N and 5750.49 E)	5.61
SP 1 (5748.34 N and 5755.40 E)	5.01
SP 2 (5745.23 N and 5762.77 E)	3.89

** Note - Coordinates refer to the local grid on RON, Universal Transverse Mercator and Government Metric Zone 58*

Wind speeds and direction was measured with a digital, hand-held anemometer (Figure 6).

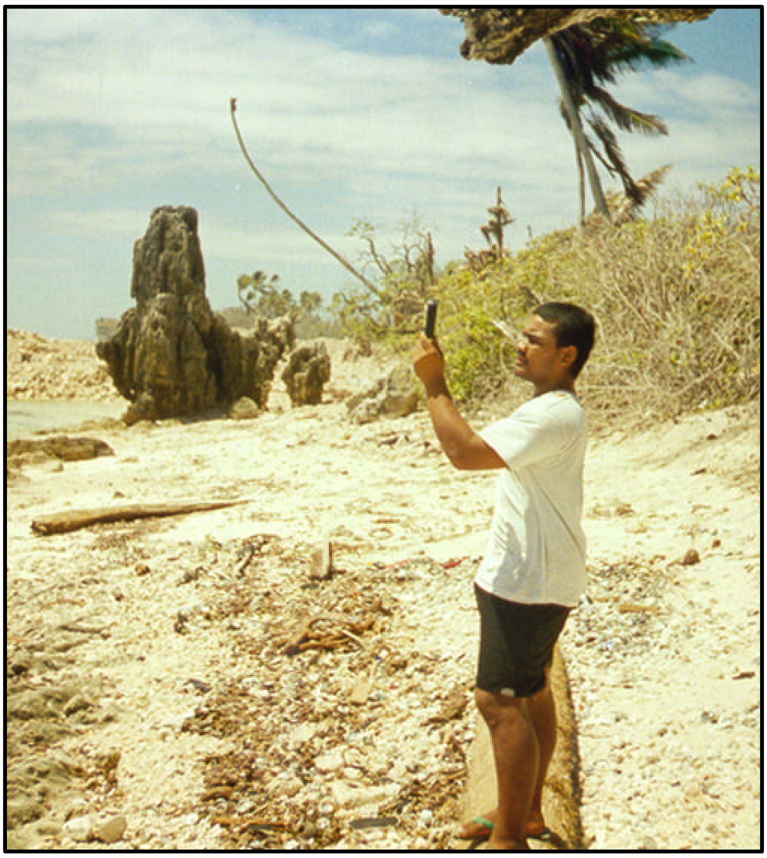

Figure 7. Measuring wind speed, Anibare Bay (Fisheries Department Assistant using an anenometer).

Table 2. Tidal ranges and their elevations with respect to Chart Datum Level (CDL) or Lowest Astronomical Tides (LAT). Information from Tetra (1999).

Tidal Levels	*Elevation With Respect to Chart Datum Level (CDL), m*
Highest High Water Level (HHWL)	2.64
High Spring Tide Water Level (HWST)	2.42
High Neap Tide Water Level (HWNT)	1.81
Mean Water Level (MWL)	1.57
Low Neap Tide Water Level (LWNT)	1.33
Low Spring Tide Water Level (LWST)	0.73
Chart Datum Level (CDL)	0

SOPAC

Positions in the field were determined with a Garmin™ hand-held GPS. Information on waves was also obtained from Tetra (1998) and vertical, colour, 1992 stereopairs/aerial photographs.

The dolomite limestone rock strength was measured in-situ, with an ELE® Schmidt™ L-Type Hammer, while concrete strength was measured with an ELE® Concrete Testing Hammer.

Rip-rap/armourstone description and revetment evaluation are based design on criteria of CIRIA (1991) and Pilarczyk (1996), Latham (1998), Geological Society of London (GSL, 1999). Design analysis has been produced after Numerical Analysis with International Institute for Hydraulic Engineering, Delft University of Technology (IHE-Delft, 1999), Coastal and River Engineering Support System (CRESS) and the U. S. Army Corps of Engineers, Automatic Coastal Engineering System (ACES).

Computations of rip-rap dimensions, armourstone stability and wave run-up are based on numerical equations of van de Meer (1987 and 1998) and are modifications and improvements of the typical Hudson (1958) formula. Computations are therefore for random wave attack, more typical in the natural environments.

3.0 RESULTS

3.1 Geologic and Tectonic Setting

Nauru is located at the Southeast end of the Nauru Basin. Nauru Basin is a 4-5 km deep ocean basin extending from the Southwest of the Marshall/Gilbert Islands, at approximately 6^0 N, to the Ontong Java Plateau in the Southwest (Figure 7).

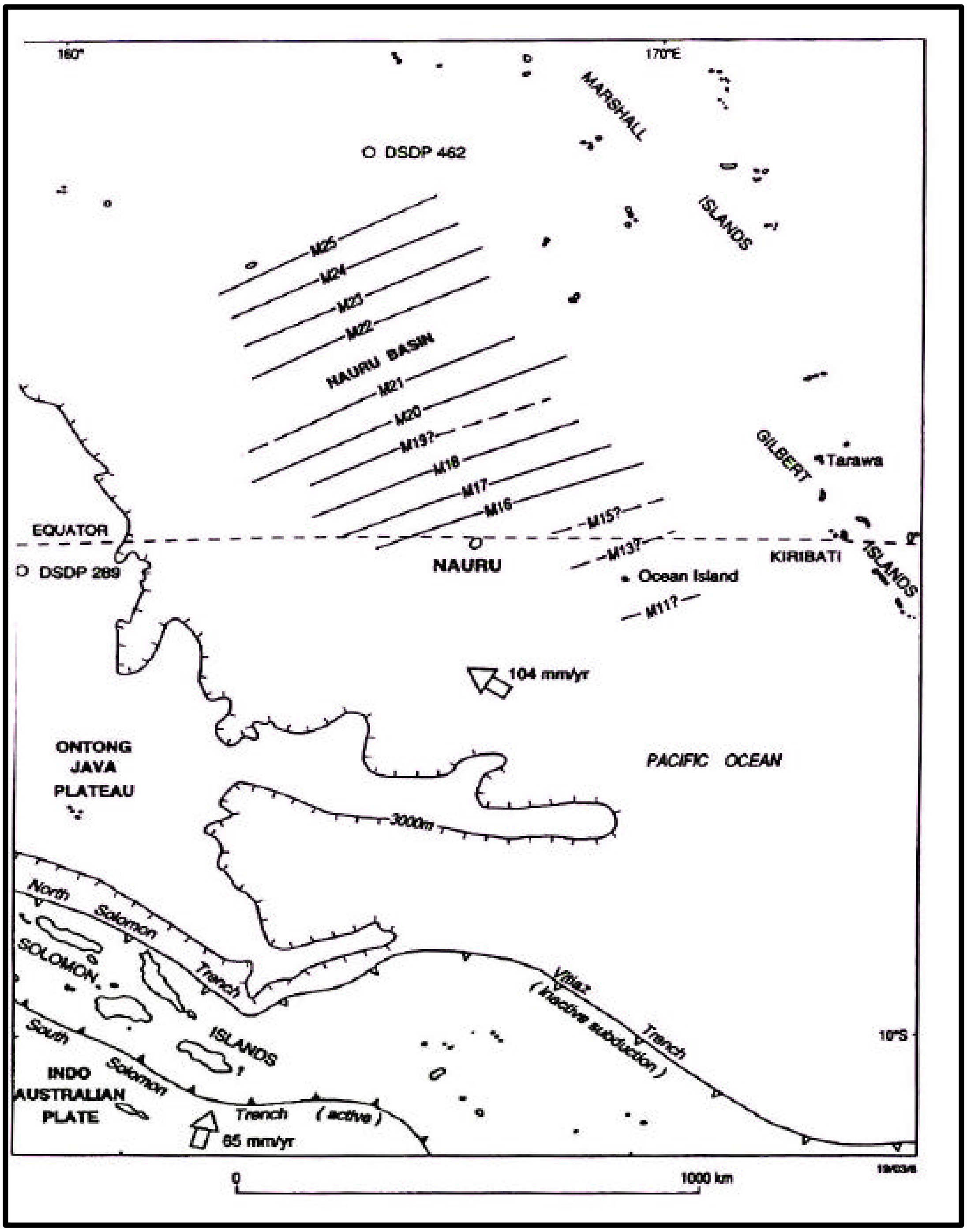

Figure 8. Geological setting of Nauru (Minster and Jordan, 1978).

The Nauru Basin is an ocean trough which transects this part of the North central Pacific Ocean. The Ontong Java Plateau is a crustal high, which extends from the Solomon Trench, just Northeast of the Solomon Islands, to about 10^0 N latitude, towards the Equator (Figure 7).

Nauru evolved between the mid-Eocene to Oligocene times (Jacobson et al, 1997), and represents an emergent part of a submarine basaltic seamount, created by hotspot volcanism, which coincided with major plate re-organization in the central Pacific region. Hill and Jacobson (1989) also note that it is probable that Nauru overlies oceanic lithosphere accreted at a mid-oceanic spreading ridge at about 132 Ma.

A coral atoll subsequently developed on this raised volcanic seamount, which after final emergence, in the Holocene, to its present day location, produced the present day island. The volcanic seamount, which underlies this emergent atoll, rises to more than 4000 m above the ocean floor. Nauru also has a small land, 22 km^2, and sub-sea circumference (Hill and Jacobson, 1989; Figures 8 and 9).

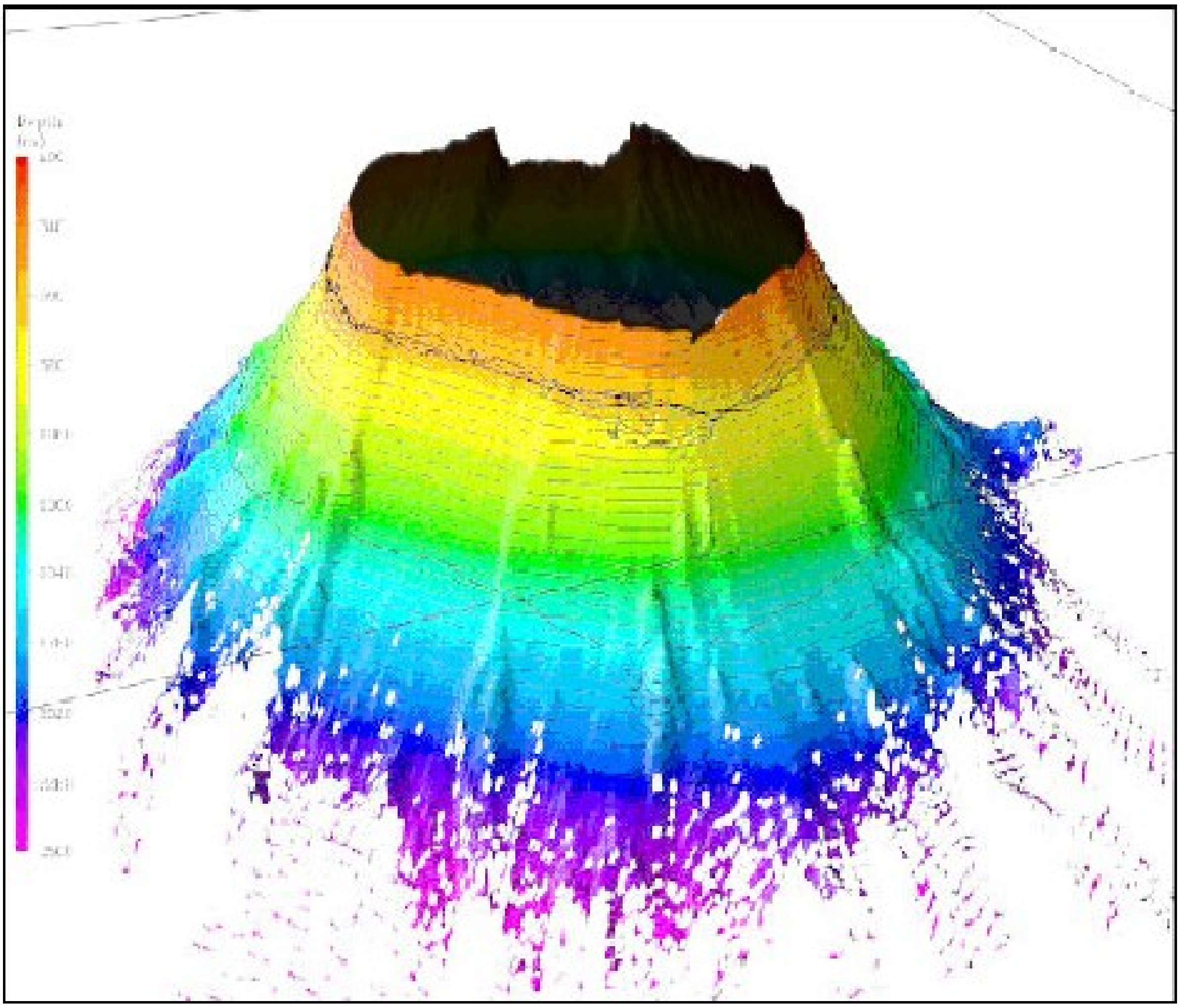

Figure 9. 3-D image of Nauru Island from bathymetric data. Note the step sharp offshore slopes. Data obtained from Government of the Republic of Nauru.

As a result of its small circumference and deep nearshore waters, Nauru therefore has a sharp and steep offshore slope gradient (Figures 8 and 9). This steep ocean slope, combined with a narrow reef crest and back reef (of less than 250 m wide) cause the island to be affected by true ocean processes, with almost no alteration to and transformation of wave processes between the open Pacific Ocean, the reef and the land-sea interface.

The Pacific Plate, at Nauru, is also presently moving at about 104 mm/yr, towards the Northwest (Minster and Jordan, 1978).

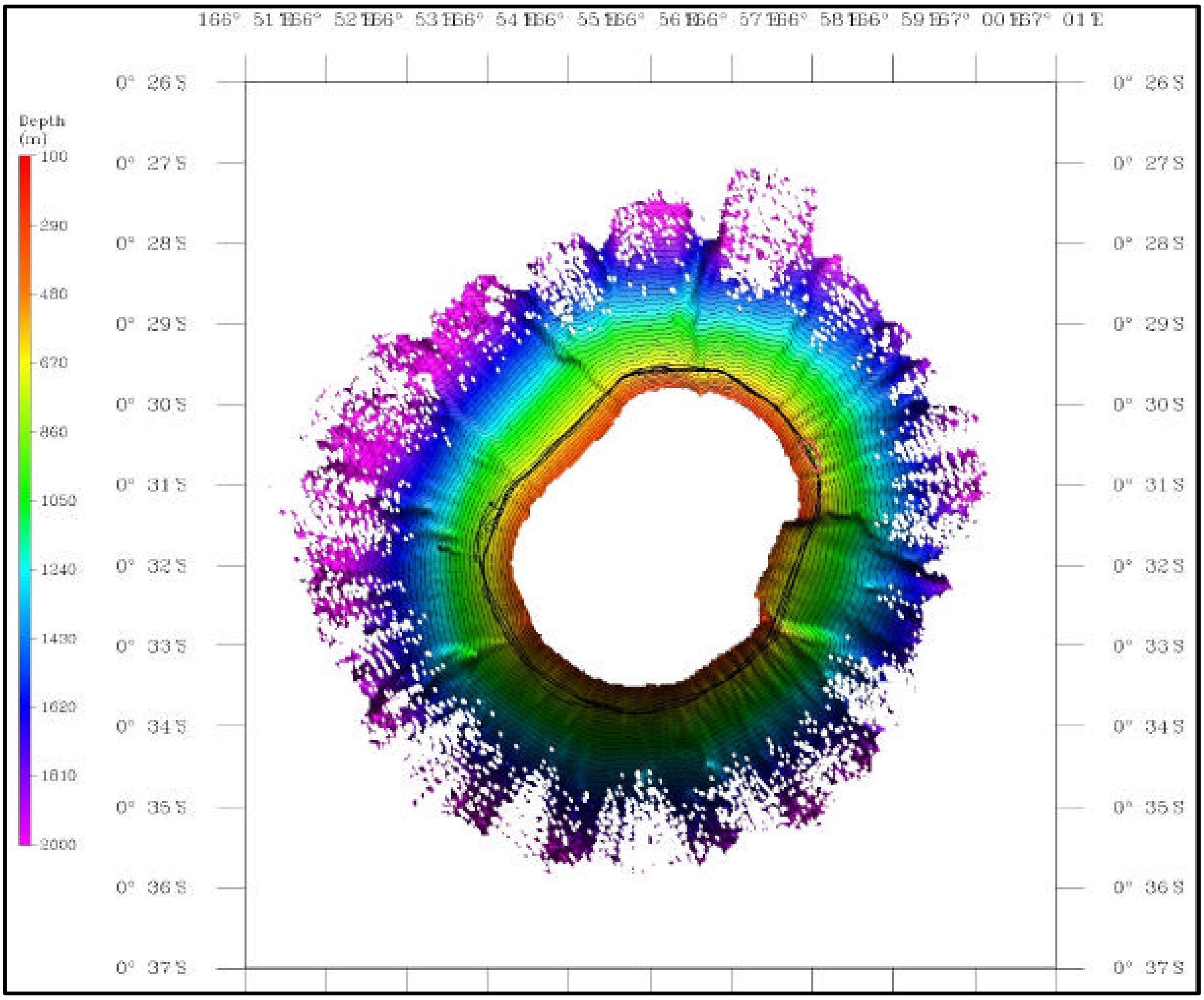

Figure 10. Plan view of Nauru Island from bathymetric data obtained depicted in Figure 9. Data obtained from Government of the Republic of Nauru.

3.2 The Study Area-General Features

Nauru Island is an emergent coral atoll, lying just South of the Equator, between 0° 30” – 0° 34’ S latitudes and 166° 54’’ – 166° 58” E longitudes.

Anibare Bay is located in the Anibare District, and is one of the larger districts on the Eastern part of Nauru (Figures 3 and 4). Anibare Bay is about 2.5 km long, extending from Mennen Hotel in the South, into Ijuw District in the North (Figures 2 and 4).

Figure 11. Beach morphology. Top left – arcuate, coconut palm-fringed sandy beach, north Anibare Bay; middle photo – upper beach showing erosion, central Anibare Bay and lower left – dolomite limestone pinnacles in the surf zone, central Anibare Bay.

The bay is arcuate and largely asymmetrical (Figures 4 and 10), with an actively eroding coastline (Figure 10). It is the most indented part of the coastline of Nauru (Figures 8 and 9). However, this indentation is tectonically controlled.

Analysis of August 1992, vertical, colour aerial photography (1: 5,000) and Figures 8 and 9, show a well developed asymmetrical coastline, with a developing headland to the South, at the border of Meneng and Yaren Districts and a smaller, but still well-defined "point" to the North, on the Southern part of the Ijuw District (Figure 3). This well-defined bay, is the result of a submarine failure of the rock mass which forms the basement of Nauru Island.

Analysis of bathymetric data for the Island, show a well-defined, concave and arcuate failed slope, which forms an amphitheatre on the underlying seamount which supports Nauru Island (Figures 8 and 9). This feature has not been subject to analysis before, but has been mentioned briefly in other reports (Jacobson and Hill, 1993, Jacobson et al, 1997).

Analysis of the failed slope show that the upper part of the slope, near the top of the main scarp, has rotated backwards and reflects a tilting "into" the land (Figure 11). In addition, the lower part of the failed slope, from which the mass was detached, has a sub-sea elevation of about 900-1100 m. Sub-sea elevations below 1100 m show the characteristic and typical bulge of a landslide deposit (Figure 11). This deposit extends to approximately 2000 m below mean sea level.

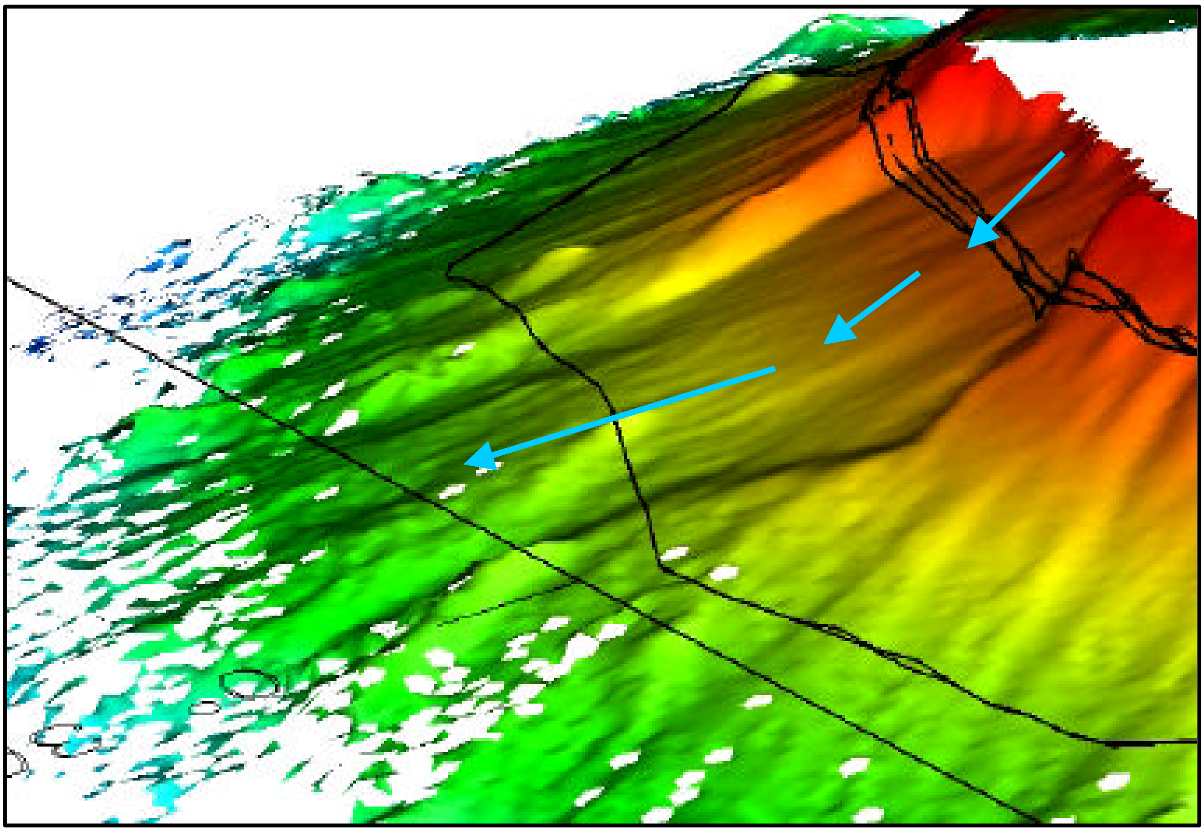

Figure 12. Physiography of the sub-sea landslide which produced Anibare Bay.

The scarp of the landslide has produced the geomorphologic feature, which is now Anibare Bay (Figure 11). The width of the bay corresponds to the maximum width of the main scarp of the submarine landslide. This main scarp is also about 2.5 km wide.

Analysis of August 1992, vertical aerial photographs reveals an arcuate lineament, which defines the landslide scarp (Figure 4). This lineament is a fracture produced by failure of the slope. This lineament is also more heavily vegetated than surrounding areas and with a distinct curvilinear grove of several tree and plant species (Figure 4).
This development of the tree grove may have been possible, due to the exposure of the freshwater table (produced as a result of downslope displacement of the formerly elevated limestone terrain and development of the landslide scarp in the area).

Northeast-trending fault lineaments are also found to the North of the Anibare Bay, in Ijuw District (Jacobson and Hill, 1993 and Jacobson et al, 1997). This lineament extends for several tens of metres inland. Barrett (1988) has also documented the arcuate lineament and others in the vicinity of Anibare Bay on Nauru Island.

Elevations in Anibare Bay varies between 3 m in the central part of the bay, to 12 m in the Northern part of the bay. At the harbour site, elevations range from 2.32 – 4.05 m above mean sea level (see Tables 1 and 2 for reference to Chart Datum Level and Mean Sea Level).

3.3 The Coastal/Shorefront Areas

The coastal areas of Anibare Bay has several shorefront settlements, and is connected by the major coastal, two-lane/dual carriageway, sealed, flexible pavement asphalt roadway (Figure 4). This coastal highway is the main transportation artery on the island and is situated almost along the edge of the shoreline in Anibare Bay (Figure 4).

The coastal land area is vegetated with several shrub and hardy trees and a few coconut palms. The shrub line is at the roadway's edge (Figure 4).

Coastal soils are almost non-existent. There are sandy coral soils in some shallow depressions and pockets. These soils are found along the Holocene coastal terrace deposits (Jacobson et al, 1997) which fringes Anibare Bay. This terrace is very narrow, about 15 m wide for most of the length of the bay. These terrace soils are granular, light to medium brown, are free draining and are non-plastic or cohesionless (Figures 11 and 12).

The underlying bedrock in the Bay area is dolomite coral and reef limestone (Figure 13) of Pleistocene to Pliocene age (Jacobson et al, 1997). This limestone were elevated in the late Tertiary to Holocene period, due to uplift of the Pacific seafloor in the area and formed what is the present day island of Nauru.

The limestone exposed in Anibare Bay consists primarily of re-crystallised Mollusc and coral skeleton, with other varieties of reef skeletal materials, like calareous algae and other invertebrate fauna (Figure 13).

Jacobson and Hill (1988) note that the limestone in Nauru are packestone and grainstone, and were deposited in a saline lagoon/backreef environment. As mentioned, Molluscs were identified in specimens from the Bay (Figure 13).

Figure 13. Above – Exposed coastal soils at an erosion scarp near the development site.

Figure 14. Right – Details of the brown coastal soils and vertical erosions scarp from Figure 13.

Field analysis of limestone outcrops show that there has been considerable re-crystallisation of these limestones and infilling on primary porosity, which has produced a very compact, fine-grained structure (Figure 13). This characteristic has resulted in the formation of a much heavier and denser material, than typical/modern coral skeleton on living reefs. It is possible that this re-crystallisation was associated with dolomite formation.

Dolomite is a magnesium-enriched carbonate, with higher specific gravity than calcium carbonate. Dolomite can be precipitated at the freshwater and seawater interface in the ground, within the inter-tidal zone, or at the Ghyben-Herzberg Interface.

Land (1971)) conducted extensive studies on the precipitation of dolomite within reef carbonates in similar saline, lagoon and backreef environments.

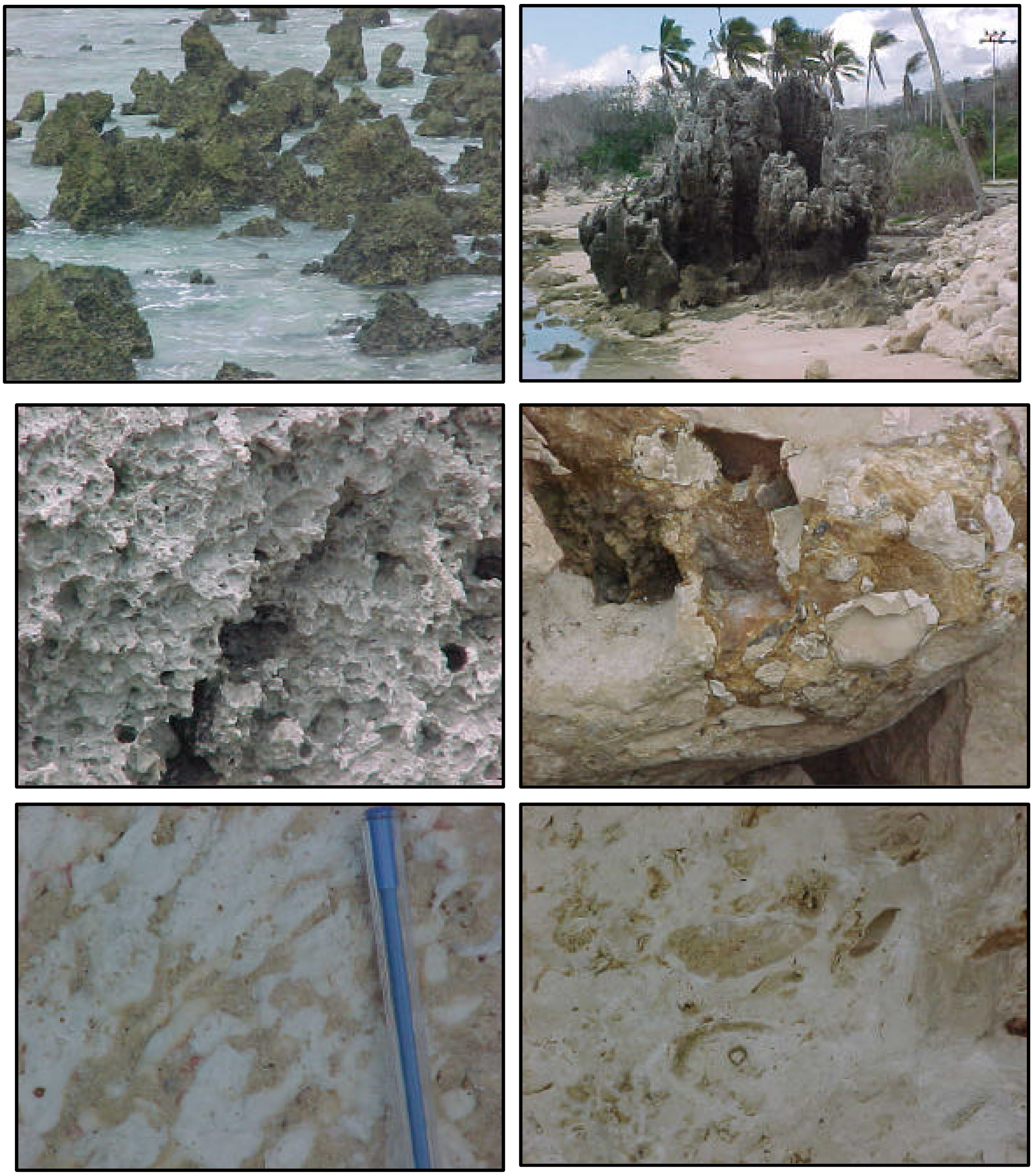

Figure 15. Characteristics of coastal dolomite limestone exposures. The top two photos show typical pinnacle sin the surf zone. The middle left photo shows typical hone-combed structure due to solution weathering, while the middle right photo shows dark brown phosphate on a limestone boulder. The lower photos show recrystallised coral skeleton (left) and recrystallised Mollusc skeleton (right).

Land (1971), based on work on emergent, Pleistocene Jamaican reefs, concluded that dolomitisation takes place where there is abundant freshwater mixing with seawater in coastal reef-carbonate aquifers. In addition, he noted that where the total dissolved solids (TDS) in the mixing zone is equal to or exceeds 800 mg/L, the water becomes supersaturated with respect to dolomite, and dolomite is precipitated within the rock fabric. Where dolomite limestone is precipitated, the carbonate host rock tends to become more dense and attains a higher specific gravity, due to re-crystallisation and filling of primary porosity. Consequently, the rock becomes less porous, with a lower void ratio and with a corresponding increase in unit weight.

The limestone in Anibare Bay is porous, with many pitted solution cavities (Figure 13), some up to 5 cm long and several centimeters wide. However, these are due to secondary chemical solution and fracturing due to Pleistocene to Holocene tectonic activity. The fine grained and re-crystallized nature of this limestone, has also caused the development of low inter-granular porosity. The micro-crystalline nature of this limestone can cause fracture and splintering, producing highly angular, jagged fragments, with high aspect ratio. Outcrops along this part of the coast and on the adjacent reef flat occur as pinnacles (Figure 13), some rising up to 7.5 m high, e.g. on the Southern side of the Bay and the harbour site near the Mennen Hotel.

3.4 The Anibare Channel

Anibare Channel in Anibare Bay was used as a fishing facility for many years, and was the site of a reef-blasting exercise, which was done to construct a small fishing boat access channel in the early 1970's. At the time, there was no other boat facility in that part of the island. The channel, which was created by blasting the reef, was about 10 – 12 m wide and varied from 0.25 – 4.0 m deep (at high tide), with an almost 90° side slope. The estimated seaward slope of this channel was about 8° (Figure 14). The channel was about 80 m long, with a 6 m wide a d 20 m long boat ramp.

Figure 16. Anibare Channel before harbour construction. Note the concrete boat ramp in the foreground and the proximity of surf on the reef crest in the background.

Fishing boats were usually lauched from and anchored at the site, from a 20 m long and 5 m wide, concrete boat ramp, which was also built at the site (Figure 14).

The 4.0 m deep channel segment was cut in the reef crest, with live coral and other benthic reef biota removed. The shallower end of the blasted channel was primarily backreef areas or reef flat.

3.5 The Anibare Bay Beach

The backreef in Anibare Bay is relatively flat and almost featureless, and is affected by drying during low and falling tides. This is common during extreme low water spring tides (ELWST), when it is almost completely dry, except for small water-filed erosion pools (Figure 12). ELWST for Nauru is about 0.84 m below mean water level (Table 1). This causes a much wider beach area to become exposed.

The beach to the North of the bay is relatively gentle to moderate sloping, about 5° (Figure 15). Those to the south of the bay are similar in morphology.

Figure 17. Gentle to moderately sloping beach, North Anibare Bay.

The beach at the development site and in Anibare Bay is relatively narrow, with an average width of 5 m. Maximum width varies up to 8 m, with some areas having only a 1 m wide beach (Figure 16). Beach slope varies from those areas in the North and South parts of the Bay.

The beach in the bay has a generally concave morphology (Figure 10), with an almost constant grade to the water's edge. The site has the following characteristics:

- ✓ a medium to fine, carbonate sand beach, with medium to coarse gravel on the lower beach areas (Figure 16);
- ✓ the beach is about 6 m wide with a 12-15° slope towards the sea (Figure 16);
- ✓ sands are clean, with less than 5 % fines;

- ✓ all sand grains are reef detritus, with more than 75 % coral detritus;

Figure 18. Beach physiography (top photos) and sedimentology (middle and lower photos) at the development site. Note the coarse sub-rounded and well washed Mollusca and coral gravels on the lower beach in the middle left and top right photos; and the sub-angular boulders with sand on the mid-beach, in the middle right photo. The lower left photo shows filamentous green algae on angular boulders; while the lower right photo shows sub-rounded, medium gravel deposited between rip-rap boulders pockets on the lower beach at the site.

- ✓ Molluscs, foraminifera tests, and Echinoderm tests comprise most of the remaining 25 % of the sand fragments;
- ✓ the upper beach has a thicker sand accumulation, about 0.75 m, which is usually thicker during the early half of the year (Figure 16);
- ✓ small coconut trees and coastal shrubs line the coastal areas (Figure 15);
- ✓ shorefront vegetation are largely small shrubs and are currently being affected by severe erosion (Figures 15);
- ✓ the erosion scarp at the problem coastline is between 0.80 m and 1.75 m high (Figures 11-12 and 17);

Figure 19. Erosion scarps and exposure of coastal soils and tree roots at the development site.

- ✓ the scarp is vertical, with some overhanging sections where coconut tree roots still bind shallow and surface soils (Figures 12 and 17);
- ✓ the eroded scarp has exposed in-situ coastal soils (Figures 17);
- ✓ the lower beach extends to the reef crest for about 90-110 m;
- ✓ the backreef area is a typical Holocene, elevated, almost featureless platform, with few pocket or depressions (Figure 12);
- ✓ the backreef is covered with angular limestone pinnacles (Figure 13);
- ✓ pinnacles are cavernous, dolomite limestone (Figure 13);
- ✓ the interface of the lower beach and backreef area is a depositional site for medium to coarse, well-washed gravel (Figure 16);
- ✓ these gravels are generally sub-rounded and are 100 % carbonate grains (Figure 16);

- ✓ gravels in this areas are well sorted and free from fines (Figure 16);
- ✓ about 90 % of all gravel fragments are of coral debris, with less than 10 % comprising invertebrate skeletal Molluscs fragments(Figure 16); and
- ✓ all gravel fragments have a high aspect ratio (Figure 16).

3.6 The Harbour Facility: Quantity Surveys

Nauru new harbour involved dredging 29,579 m^3, building 434.8 m^3 of boat ramp, 1,667.1 m^3 of steel-reinforced concrete breakwater, 2,085.2 m^3 of wharf and apron, 1,139 m^3 of sand barriers, 70.2 m^3 of access road, 1 mooring basin and steering area, 2 navigational aids and lighting fixtures and 650 m^2 of boat parking facilities (Figures 18 and 19).

Figure 20. Characteristics of the new harbour facility. Top left – mooring basin/wharf, with quay wall and apron on left of the photo; top right –sub-breakwater and main concrete breakwaters (left and fight respectively); lower left – rubble groin, with rip-rap and main concrete breakwater; and lower right – concrete boat ramp, with rip-rap pitched spending slope on the rear right.

The dredging basin was excavated down to 2.5 m below CDL, while the channel and mooring basin were dredged to 2.5-3.0 m below CDL. The shallower end of the mooring basin is closer to land. Excavation was achieved by drilling of boreholes and dynamite blasting within the boreholes to fracture the hard dolomite limestone. Many closely spaced boreholes were drilled on low tide, since the limestone is very competent. The blasted debris was subsequently removed using a hydraulic excavator.

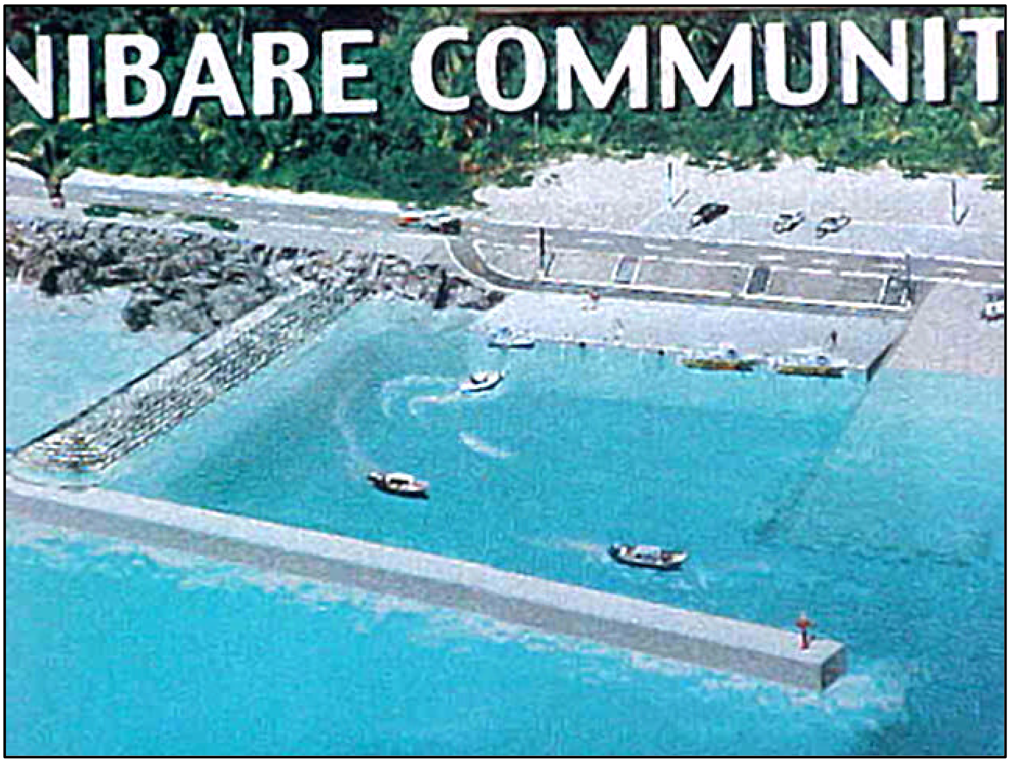

Figure 21. An artist's impression of the harbour facility when in operation.

The boat ramp include a concrete pavement, an underwater, concrete retaining wall and pre-cast concrete slab blocks.

The concrete breakwaters were constructed in-situ and consist of a 85 m long, 3.7 m wide, North-South trending main unit (1,094.5 m^3); and a 40 m long, 4 m wide, Northwest-Southeast trending sub-unit (571.7 m^3).

Both breakwaters are vertical gravity structures, anchored to the underlying dolomite limestone by 0.5 m deep and 1.0 m wide, concrete-filled interlocking keys. The main unit is leveled at 4.8 m above CDL and the sub-unit is leveled at 4.6 m above CDL.

The wharf and apron consist of a retaining wall, a concrete pavement, rubble stone rip-rap, curbing and an in-situ concrete quay wall. The access road consists of a concrete pavement and stone works, while the boat parking facility is a gravel-paved pavement.

An East-West trending rubble groin was built on the Southern side of the harbour facility and is 54 m long and 2 m wide at its top. It was leveled at 4.5 m above CDL and has 1: 1.5 slopes facing the wharf (North) and 1: 1.2 side slopes facing the South.

Dolomite limestone sourced from phosphate mining sites on Nauru was used to engineer the groin. Rip-rap for this structure were 500-1,000 kg/pc[1] The groin consists of an inner core of 50-100 kg/pc dolomite limestone boulders and a 0.7 m thick, single outer layer of 500-1,000 kg/pc. It is built on the backreef substrate, which has an original elevation of 1.5 m above CDL.

Figure 22. Rip-rap characteristics at the harbour site. Top left – a single-layered rip-rap, protecting an eroded segment of road, on the Southern side of the harbour; top left – rip-rap at the toe of the groin; lower left – algae-covered rip rap at the toe of the groin, alongside the mooring basin; and lower right – note rip-rap boulder shape, angularity and pits.

This harbour facility was constructed in the backreef environment, in a maximum water depth of 3.0 m below chart datum level (CDL) or 1.57 m below mean sea level (MSL). The maximum elevations on the facility are on the concrete breakwaters,

[1] *Designers' specification. These are not to be converted. The author assume that pc implies, per cubic metre, as most specifications were given in metres.*

which were levelled at 4.80 m above CDL. On an EHWST, the breakwaters are only 2.16 m above water level.

3.7 Erosion Characteristics

Figures 11-12 and 15-17 show some characteristics of the beach erosion at the development site. In addition, Figure 20 shows an example of a rip-rap protection for a segment of eroding roadway immediately South of the development site. This segment of eroding roadway developed during the lifetime of the harbour development project.

From Figures 12 and 17, one can see a clear erosion scarp, which has exposed plant roots at the upper beach/land interface. Coastal soils are also exposed here, as the scarp is a new one. Several shrubs fell on the beach at the site. Figure 21 shows typical scouring of the upper beach and beach-land interface, exposing the underlying in-situ limestone bedrock. This is common during high tides, especially EHWST, and are associated with large, Easterly approaching wind waves and plunging breakers. One can also observe the typical scoured surface of the exposed limestone bedrock, created by wave action (swash and backwash; Figure 21). This scouring became more pronounced after construction started at the site. To date, the sand which was eroded here has never recovered at this site.

Figure 23. Scouring and removal of beach sand on the mid and upper beach, exposing the underlying scoured and eroded dolomite limestone. Note the coastal shrubs which are have now collapsed on the upper beach. This site is just North of the harbour facility.

Close examination of the scarp here revealed that the erosion was recent. At this site, one can see the exposed, in-situ, dark brown coastal soils. This is best illustrated in Figures 11 and 12.

At the development site, larger tree species are also affected by wave erosion (Figure 17). While these trees show evidence of long-term stress, due to wave erosion (seaward-curved tree trunk), the present erosion has exposed the tree roots, which bind the coastal soils, and is now threatening to erode the coastal road, behind the vegetation line.

Eroded areas are also common along continuous segments of the beach, immediately North and South of the harbour site. A typical 1.15 m high erosion scarp is shown in Figures 11-12. One can observe large clumps of surficial soil, which toppled onto the upper beach. Details of the scarp show the exposed in-situ, coastal terrace soils, and the fresh erosion of the coastal land area. The beach fronting this scarp is typically narrow, with a thin sediment cover and is extensively eroded, also exposing the underlying limestone bedrock.

Recent visits to the development site in October 2000, has also show that erosion is already prevalent and a problem, immediately South of the harbour facility. This has resulted in damage to segments of the coastal road foundation and flexible bituminous pavement. To arrest this, a rip-rap facility was put in place at the site by the harbour construction engineers, using the local dolomite limestone boulders (Figure 20, top left). However, this has not stopped erosion of this local segment of coast, but has only transferred the erosion from that point, to areas just South of it. It is significant to note that the entire harbour facility acts as a large groin, perpendicular to the natural shoreline. With dominant longshore currents to the South, any facility built perpendicular to the shore, like this one, will result in aggravated erosion of coastal areas to the South or on the lee aspect of the facility. This is exactly what has already happened, and resulted in the erosion previously described. In addition, waves approaching from the East will also cause some diversion/diffraction of surf around the facility and therefore, to the North and to the South of the facility. Consequently, areas immediately North of the harbour will also be affected by aggravated erosion and may loose beach sediments. This has already happened and is seen in Figures 16, 17 and 21. In addition, such diffraction will also add to the erosion already being experienced to the South of the harbour.

For harbour facilities like this one, which are built on a narrow backreef, and fringed by a narrow reef crest and adjacent to very deep offshore areas, it is important to realize that coastal engineering pre-disposes the facility to harsh, almost open-ocean wave conditions. This is because there is hardly any significant buffer zone to reduce the deleterious effects of large oceanic waves and breakers. While there is a reef and backreef, these are extremely narrow, and already, do not reduce significant wave energy which impact Nauru's coast. Consequently, this facility will suffer considerable impacts from wave forces, wave run-up ad overtopping during its lifespan. This will inevitable lead to the development of eddies and significant turbulence around the facility, and individual structures (breakwaters, groin and rip-rap), leading to scouring and sediment erosion. Sediments removed from these areas, are more likely be lost from the local ecosystem, as the backreef is too narrow and has almost no sediment sinks (bathymetric lows/depressions) to trap sediment in motion, before they reach the reef crest and reef channels. While some sediments may be transported and accumulate downdrift/South, a significant amount will be lost from the reef environments, to the open ocean, via the reef channels.

3.8 Some Macrobenthic Biota

The following are some common macrobenthic biota found on the beach and in the surf zone.

- ✓ the lower beach and backreef areas are 50 % covered with green, filamentous algae (Figure 22);
- ✓ abundant green algae indicates that there is eutrophication (nutrient enrichment) in the coastal waters;
- ✓ macrobenthic species are common on the lower, rock part of the beach area and backreef; including green and brown algae, red algae, sea cucumbers; crabs; sea urchins and a variety of Molluscs (Figure 22) attached to rock substrate;
- ✓ as mentioned previously, the bay is fringed by a coral reef, which contain several species of coral; and
- ✓ the reef is narrow and averages 15 m in width.

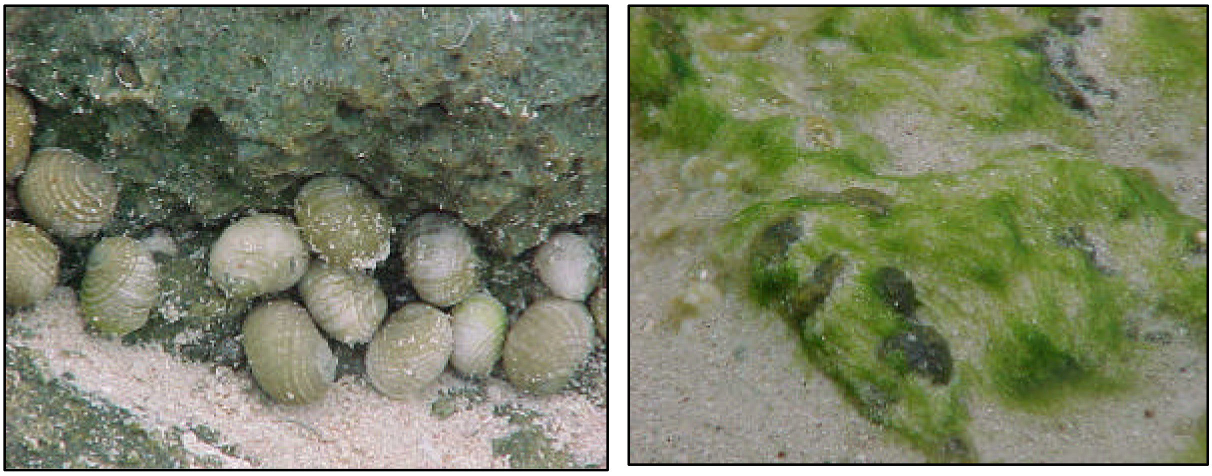

Figure 24. Examples of benthic biota. Left - mollusc fauna on the dolomite limestone substrate. These feed on green algae. Right – filamentous green algae attached to limestone rock on the lower beach at Anibare Bay.

3.9 Waves and Littoral Hydraulics

Figure 23 show a typical plunging breaker sequence for the East coast of Nauru, including the Anibare Harbour site.

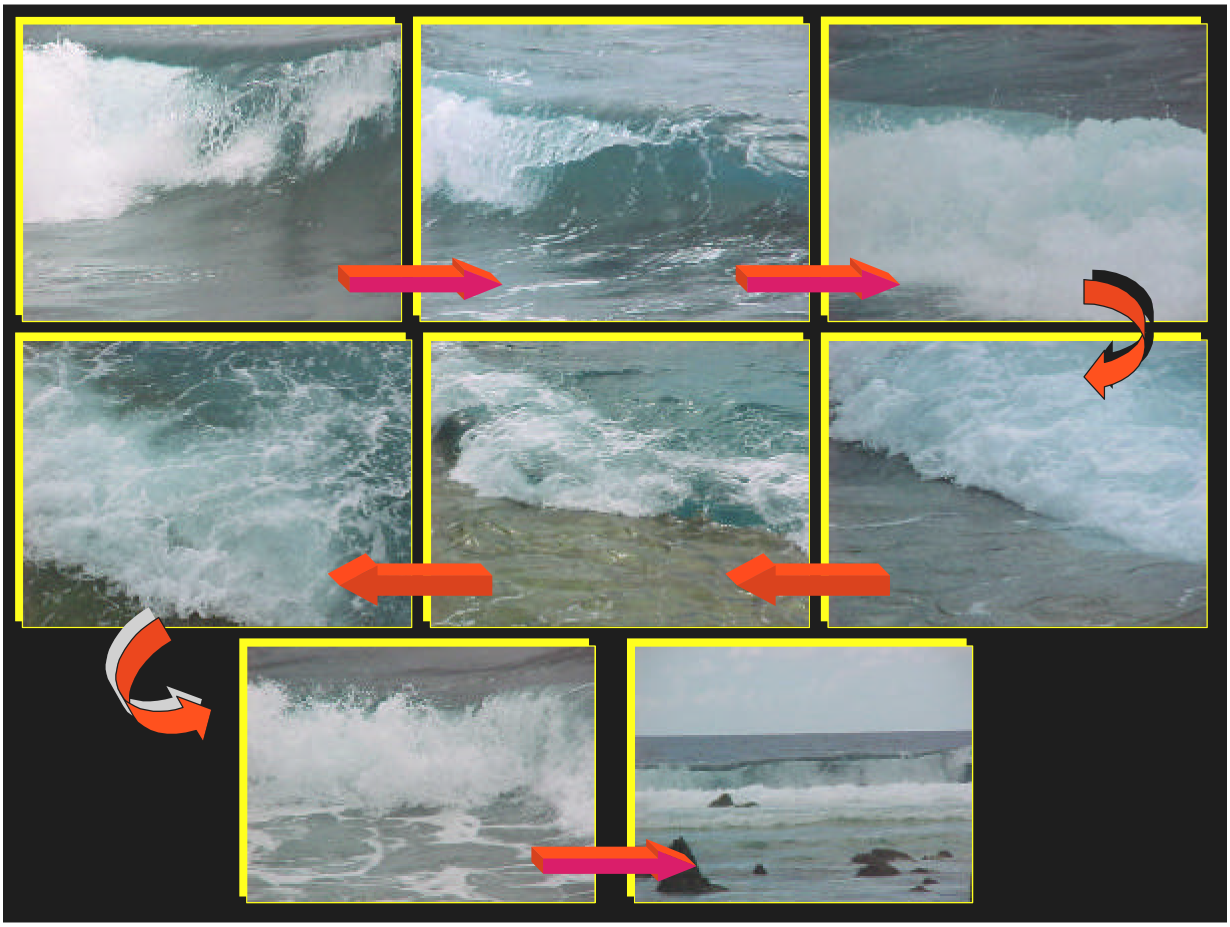

Figure 25. A typical plunging breaker sequence photographed on the East coast of Nauru. The waves shown here were 1.5 m high and broke as plunging breakers, typical for this area. The first photo inclusion show the start of the sequence, while the last shows the beginning of another sequence. When the swash retreats as backwash, almost all the water drains off the backreef platform (middle right inclusion), creating a strong drag effect on the limestone and sediments substrate. This facilitates and leads to amplification of the impact forces of the breaker when it subsequently crashes on the reef crest.

The hydrodynamics of the site is diagrammatically illustrated in Figures 24 and 25.

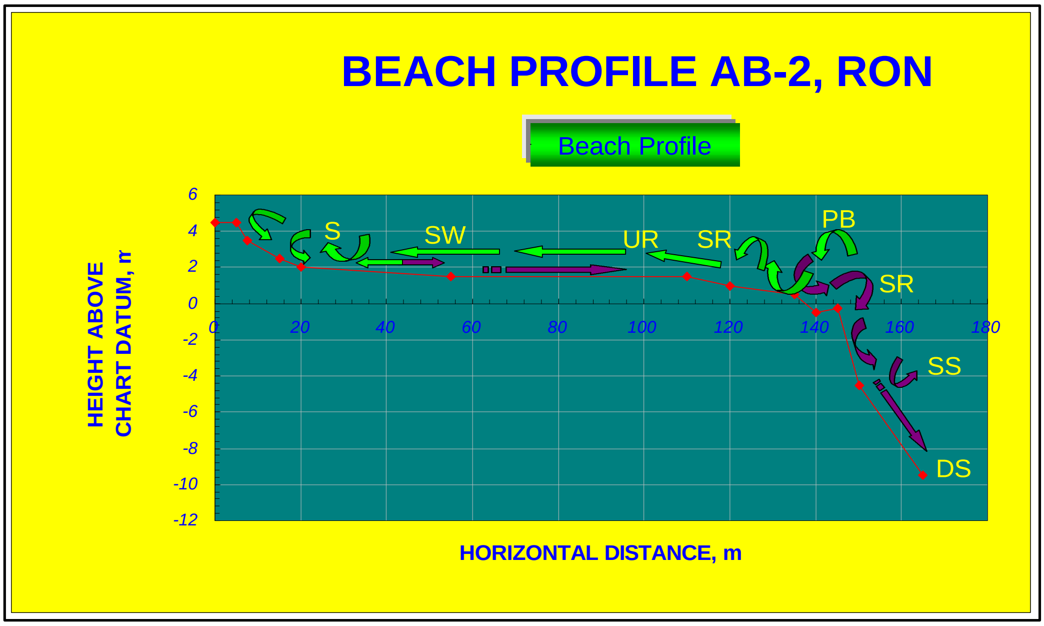

Figure 26. Hydrodynamics of beaches at the harbour site. This is based on an actual beach profile surveyed perpendicular to the coast and along the central part of the harbour development site. Green arrows show the swash sequence, while purple arrows show the backwash sequence. SR – surging (green) and sediment re-suspension (purple); PB - plunging breaker; SW - swash; S-scouring; SS - sediment suspension and DS - deep sea deposition.

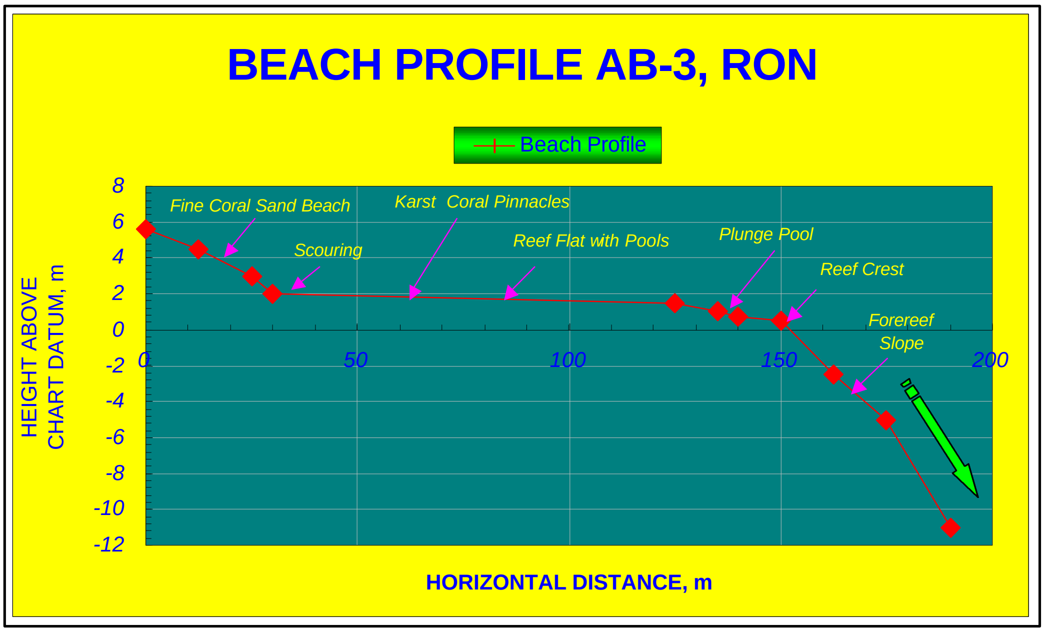

Figure 27. Beach profile morphology from a representative profile at SM 44 (see Table 1 for location co-ordinates).

The development site has the following wave characteristics.

- ✓ waves are typically plunging, with sometimes surging breakers (Figure 30);
- ✓ wave heights measured at the site based on this visit and previous visits, between 1998-2000, average 2.5 m, but can be 3 m high;
- ✓ significant wave heights, based on field observations, and discussions with the Nauru's Harbor Master and the Fisheries Department is 3.0 m;
- ✓ a typical example of a plunging breaker wave sequence, and photographed at the site, is illustrated in Figure 23 (starting top left and following the arrows);
- ✓ as with typical plunging breakers, waves come crashing on the beach and run-up with significant speed, causing significant removal of loose/cohesionless beach sediments;
- ✓ with a narrow and almost flat, backreef at the site, and in the vicinity of the development area, any disturbance of beach sediments and their removal, can lead to cross-shore sediment transport and removal from the backreef, across the reef crest, and into the deep water environment;
- ✓ most waves approach the shore at the study area almost perpendicular, to the shoreline (at 015°);
- ✓ longshore transport is largely to the South, from Easterly approaching waves;
- ✓ with perpendicular approaching waves, no longshore is generated;
- ✓ when Westerly winds affect the area, longshore is to the North;
- ✓ winds are generally from the East;
- ✓ swells are also common in the area and were mapped from aerial photography and approach from East-Northeast and
- ✓ Easterly winds generate Easterly approaching waves.

3.10 Beach Profiles and Littoral Dynamics

Seven beach profiles were surveyed in the Anibare Bay. These are AB-1 to AB-7. All sites are referenced with respect to the coastal road markers (AB-5, AB-6 and AB-7, steel electricity poles; see Appendix II for position), construction survey benchmarks (AB-1/SM 43, AB-2/SP-1 and AB-3/SM 44; see Table 1 for position) or the location of the previous Anibare Channel (site AB-4).

Four beach profiles were surveyed at the harbour site (AB1, AB-2, AB-3 and AB-4), and two to the North (AB-5 and AB-6) and one to the South of Anibare Bay (AB-7).

The graphical plots of representative beach profile data, beach sediments and their changes in response to the harbour development, will now be presented. In addition, the impacts of the development of the natural and built coastline will be presented and discussed.

Figures 26-29 presents some details of the beach profile survey sites and littoral conditions at the time of the survey.

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

Profile Number:	AB-1, RON
Location of Profile:	North side of the Harbour Development, SM 43
Profile Direction:	090/270
Beach Orientation:	360/180
Time of Survey:	2nd. March, 2000
Date:	2nd. March, 2000
Surveyor/s:	Russell J. Maharaj, Geologist & Engineer
Tide:	Rising Neap
Wind Speed, m/s:	13.5-15.0. Sometimes up to 18.0.
Wind Approach/Direction:	East/090
Wave Height, m:	1.5-2
Wave Approach:	065-090
Breaker Height, m:	1.5
Breaher Approach:	065-090
Breaker Depth, m:	NA
Longshore Current Direction:	South/180
Longshore Current Speed, m/s:	To compute using CRESS/ACES

SOPAC

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

Profile Number:	AB-2, RON
Location of Profile:	Central Part of the Harbour Development, SP 1
Profile Direction:	090/270
Beach Orientation:	360/180
Time of Survey:	2nd. March, 2000
Date:	2nd. March, 2000
Surveyor/s:	Russell J. Maharaj, Geologist & Engineer
Tide:	Rising Neap
Wind Speed, m/s:	13.5-15.0. Sometimes up to 18.0.
Wind Approach/Direction:	East/090
Wave Height, m:	2
Wave Approach:	065-090
Breaker Height, m:	1.5-2
Breaher Approach:	065-090
Breaker Depth, m:	NA
Longshore Current Direction:	South/180
Longshore Current Speed, m/s:	To compute using CRESS/ACES

SOPAC

Figure 28. Details of beach profile AB-1 (SM 43) and AB-2 (SP-1).

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

Profile Number:
Location of Profile: Southern Part of the Harbour Development, SM 44
090/270
Beach Orientation:
Time of Survey: 2nd. March, 2000
2nd. March, 2000
Surveyor/s:

Tide: Rising Neap
13.5-15.0. Sometimes up to 18.0.
Wind Approach/Direction:
Wave Height, m: 2
065-090
Breaker Height, m:
Breaher Approach: 065-090
NA
Longshore Current Direction:
Longshore Current Speed, m/s: To compute using CRESS/ACES

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

AB-4, RON
Location of Profile:
Profile Direction: 090/270
360/180
Time of Survey:
Date: 2nd. March, 2000
Russell J. Maharaj, Geologist & Engineer

Tide:
Wind Speed, m/s: 13.5-15.0. Sometimes up to 18.0.
East/090
Wave Height, m:
Wave Approach: 065-090
1.5-2
Breaher Approach:
Breaker Depth, m: NA
South/180
Longshore Current Speed, m/s:

Figure 29. Details of beach profile AB-3 (SM 44) and AB-4 (along the old Anibare Channel).

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru

Profile Number:	AB-5, RON
Location of Profile:	Northern part of Anibare Bay @ Steel Eelectricity Pole No. 189
Profile Direction:	105/285
Beach Orientation:	015/195
Time of Survey:	2nd. March, 2000 @ 1100 hours
Date:	2nd. March, 2000
Surveyor/s:	Russell J. Maharaj, Geologist & Engineer
Tide:	Rising Neap, 1.33 m
Wind Speed, m/s:	4.6-12.2
Wind Approach/Direction:	East/090
Wave Height, m:	2
Wave Approach:	065-090
Breaker Height, m:	1.5-2
Breaher Approach:	065-090
Breaker Depth, m:	NA
Longshore Current Direction:	South/180
Longshore Current Speed, m/s:	To compute using CRESS/ACES

SOPAC

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

Profile Number:	AB-6, RON
Location of Profile:	Northern part of Anibare Bay @ Steel Eelectricity Pole No. 204
Profile Direction:	090/270
Beach Orientation:	005/185
Time of Survey:	2nd. March, 2000 @ 1130 hours
Date:	2nd. March, 2000
Surveyor/s:	Russell J. Maharaj, Geologist & Engineer
Tide:	Rising Neap, 1.45 m
Wind Speed, m/s:	13.5-15.0. Sometimes up to 18.0.
Wind Approach/Direction:	East/090
Wave Height, m:	2
Wave Approach:	065-090
Breaker Height, m:	1.5-2
Breaher Approach:	065-090
Breaker Depth, m:	NA
Longshore Current Direction:	South/180
Longshore Current Speed, m/s:	To compute using CRESS/ACES

SOPAC

Figure 30. Details of beach profile AB-5 and AB-6 (roadway steel electricity poles).

BEACH PROFILE AND LITTORAL FIELD DATA SHEET

Project: Anibare Bay Fishing Harbour Development, Republic of Nauru
Objectives: Coastal Monitoring and Engineering Design Assessment

Profile Number:	AB-7, RON
Location of Profile:	Southern part of Anibare Bay @ Steel Eelectricity Pole No. 219
Profile Direction:	090/270
Beach Orientation:	355/175
Time of Survey:	2nd. March, 2000 @ 1200 hours
Date:	2nd. March, 2000
Surveyor/s:	Russell J. Maharaj, Geologist & Engineer
Tide:	Rising, 1.50 above MSL
Wind Speed, m/s:	13.5-15.0. Sometimes up to 18.0.
Wind Approach/Direction:	East/090
Wave Height, m:	2
Wave Approach:	065-090
Breaker Height, m:	1.5-2
Breaher Approach:	065-090
Breaker Depth, m:	NA
Longshore Current Direction:	South/180
Longshore Current Speed, m/s:	To compute using CRESS/ACES

SOPAC

Figure 31. Details of beach profile AB-7 (roadway steel electricity poles).

From the littoral information collected, the development site experiences wind between 4.5-18 m/sec, generally from the East. This is common during the month of the survey. However Westerly winds are strong during November-February (van Loon 1984 and Tetra, 1999). Mean wind speed for Nauru based on Tetra (1999) for 1989-1997, was 4.34 m/sec, with a maximum of 13.27 m/sec. Maximum speeds reported by Tetra (1999) was for the month of February, with the minimum during July. Further, from November to March, Nauru experiences the strongest winds, usually above 4 m/sec, from the East, East-Southeast and East-Northeast. These Easterly winds represents 60 % of the wind estimates presented by Tetra (1999). Therefore, in terms of wind-wave exposure and wave hydrodynamics, the Eastern coast and Anibare Bay is located in the most dynamic part of the island.

Average wave heights at the time of surveys were between 1.50-2.0 m high. However, during the October field survey, 3.0 m high waves were observed. Breaker heights were comparable, while plunging breakers were the norm. Waves approach from 065-090° and are similar to wind approach. Tetra (1999), based on wave hindcasting (using wind data) notes that waves predominate from North-Northeast to Southeast, with 0.5-1.0 m wave heights being the most common. It is the author’s opinion that Westerly waves will not be significant along the East coast and Anibare Bay, due to decay in height as they are propagated along the coast, towards the East. The effect of typhoons and cyclones, at the latitudes where Nauru is located is not significant (van Loon, 1984 and Maharaj, 2001b).

Tetra (1999) did wave hindcasting using atmospheric pressure data for ten major low-pressure systems which affected Nauru between 1990-1996. Their computations show that 1, 2, 5, 10, 25, 50 and 100-year return period wave heights were 3.90 m, 4.15 m, 4.48 m, 4.74 m, 5.08 m, 5.34 m and 5.60 m respectively. Of the pressure systems analyzed, four of these were from the East (60-120°), with wind speeds between 6-17 m/sec and waves of 16-18 sec, and 3.9-4.2 m high. For the design life of the harbour, Tetra (1999) used a 5.34 m or 50-year design wave.

Longshore currents were generally observed flowing South, computed (using CRESS and ACES) to be between 20-25 cm/sec. Significant sea spray was observed during gusty winds. On high tide, wave overtopping was observed at the main concrete breakwater element, with (millimeters thick) sea salt precipitation on the concrete surface.

A representative set of beach profiles are presented as Figures 30-32, for the actual development site. Beach slopes examined from beach profiles had the following characteristics.

The coastal land areas at the harbour site and South of the harbour site in Anibare Bay varies in height between 4.5-5.6 m above CDL (Figures 30-32). Areas to the North of the bay have much higher elevations, up to 7 m above CDL.

The beach slopes to the North of the bay measure 5°-7°, with gentler slopes near the water line and 4.5° slopes on the upper beach. The backreef in these areas can be about 5°.

Beaches at the development site average 6.8°, with gentler, 4° upper beach slopes.

Beaches to the South of the bay are steeper than those at the harbour site, but comparable to those to the North of the bay, averaging 8°, with 4° upper beach slopes.

The middle section of the bay therefore has the gentlest beaches.

In the vicinity of the development site, the backreef has an average elevation of 1.5 m above CDL and is almost flat, with a very small gradient (< 1°) towards the ocean/reef. The beach has an average slope of 6.8°, with a thin carbonate sand cover. It has an average elevation of 2 m above CDL.

The plunge point (where waves break on the reef crest) has an average seaward slope of 11°. This slope is gentler, shoreward of the plunge point, measuring 2°-2.3°. The plunge point has an average elevation of 0.5 m below CDL.

The reef crest is at about 0.25-0.50 m below CDL. The fore-reef slope drops off at about 40°, for the first 5-6 m beyond the reef crest, then gentles out beyond that distance to 25° for the subsequent 10 m (e.g. in Figure 31). Therefore, any nearshore sediments, transported from the beach to the reef crest, can be easily transported offshore, by bottom currents and gravity flows once they enter the steep reef channels passages.

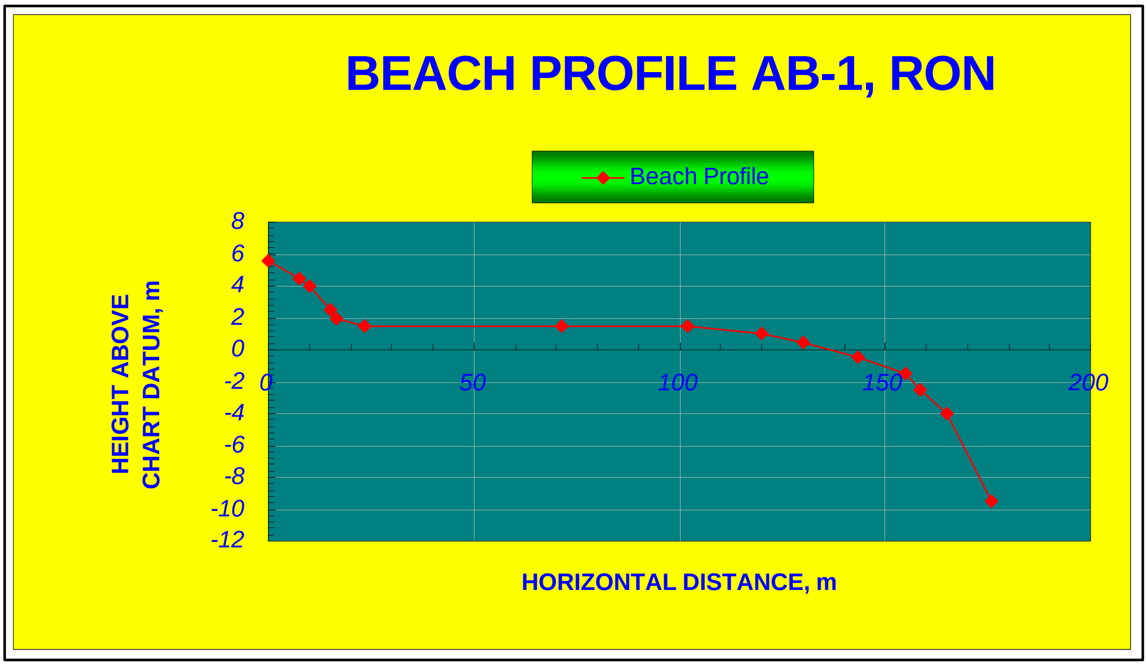

Figure 32. Beach profile for site AB-1.

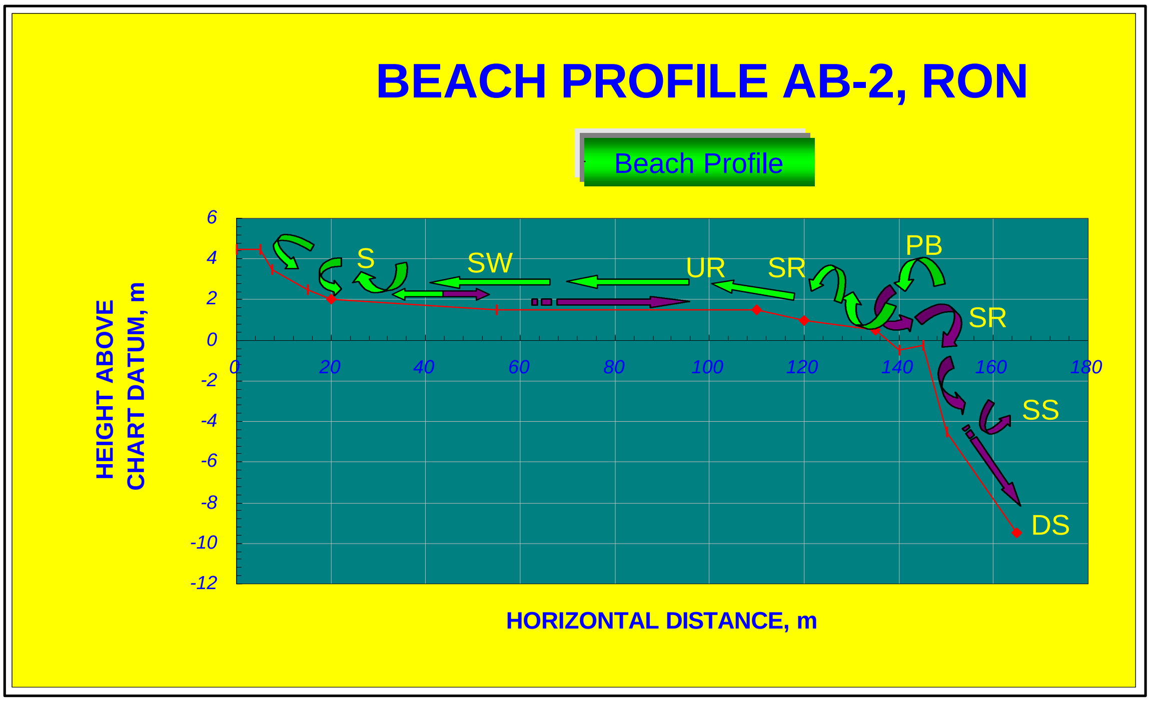

Figure 33. Beach profile for site AB-2.

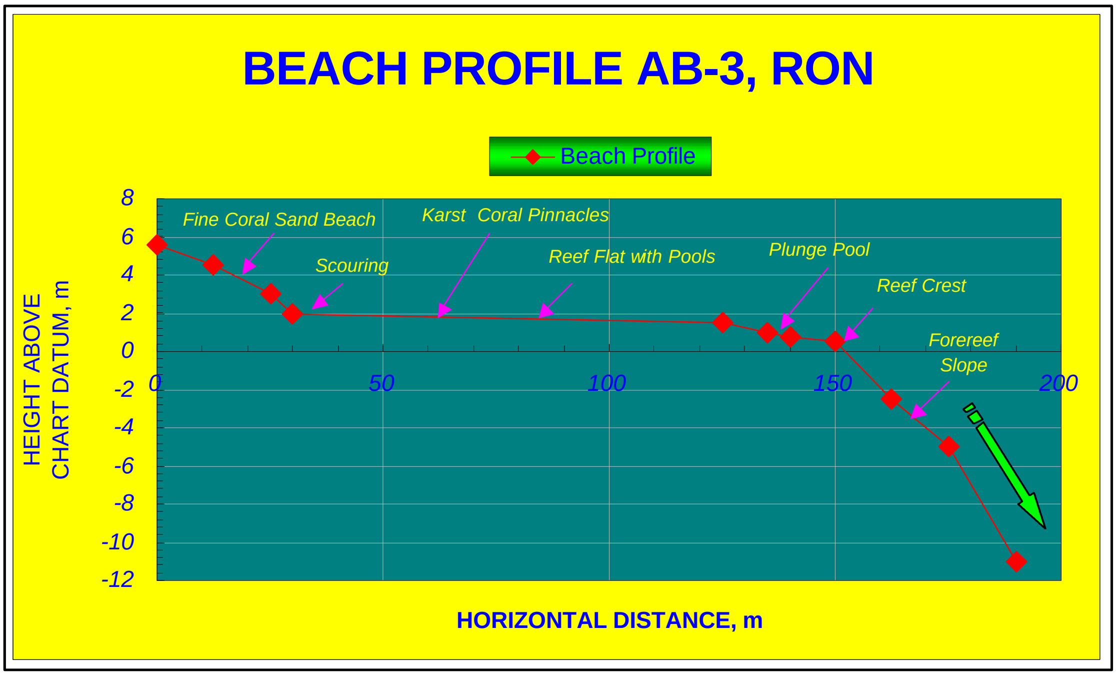

Figure 34. Beach profile for site AB-3.

From a portion of a vertical aerial photography presented as Figure 4, one can see many reef channels along the reef crest, at the harbour site. The horizontal spacing of these channels is les than 15 m apart. In addition, construction of the navigational access channel at the harbour site has emphasized one of these channels, making it steeper, deeper and wider at the reef crest. This navigational channel can facilitate more rapid removal of sediments from the harbour site and environs. Since longshore currents is generally to the South, this navigational channel can also intercept Southerly flowing currents, causing sedimentation with the channel and/or diversion of longshore currents. Diversions can be either into the harbour basin or offshore. Inner diversion can be expected when tides are high and rising and with large breaking breaking waves, under strong onshore winds. Offshore diversion may occur on falling tides and on calmer seas during less windy periods.

4.0 DISCUSSION

4.1 Impacts of the Harbour Facility on Coastal Evolution

The harbour facility already cause diversion and interruption of the dominant, Southerly flowing longshore currents in Anibare Bay. In addition the incoming Easterly waves are diffracted, both to the North and South of the harbour. Both these alterations have already resulted in local eddys, scouring and erosion of loose carbonate beach sands and their removal from the area. The sediment, which has remained on the beaches, is primarily coarse gravels and boulders. In some cases, all the sediments have been eroded from the adjacent beaches, e.g. on the Northern aspect of the harbour.

The breakwaters, both the main and sub-unit, cause considerable reflection of incoming transformed waves, between the reef crest and the breakwater elements. This is considerable, as the seaward aspect of both these structures are vertical and smooth. Already, there is considerable run-up and overtopping of these structures under EHWST, when offshore waves of 2-2.5 m high break on the reef crest. This reflected energy will not facilitate sediment accretion in the vicinity of any of these structures and the harbour, especially when the width of the backreef between the reef crest and the breakwater is less than 10 m and gravity flow under backwash is strong. In addition, reflected waves combine with subsequent incoming waves causing the growth of these subsequent waves. From the site visit conducted on 2nd March 2000, every third wave was found to be larger by at least 15 %. This is a significant increase in wave heights, and can become more noticeable on an EHWST and during low-pressure systems or choppy seas.

To the immediate South of the harbour facility, souring and sediment removal has resulted in damage to the roadway and road sub-grade. This required immediate protection, which to date, employed the use of dolomite rip-rap boulders. Attempts were also made to arrest the scouring which affected on beaches and the beach-land interface to the North of the harbour facility. It is significant to note that this scouring and sediment removal continues, even though rip-rap have been placed at these sites.

As discussed in the Sections 3.70-3.10 of this report, since the harbour acts as a groin, diverting longshore currents towards offshore, sediment eroded and removed from the site, is sediment lost from the coastal system, as most of this material is transported offshore via reef channels. This effect will eventually cause sediment starvation of downdrift Southerly coastal segments, and exacerbate erosion of these areas. The long-term evolution of this coast will therefore be affected by this change in sediment supply and alteration in nearshore hydraulics.

4.2 Performance of Coastal Protection Structures and Breakwaters

The peformance of the coastal protection structures and harbour at the development site will also affect the long-term evolution of the beaches in Anibare Bay and environs. In particular, if failure occur in sections of the structure, this can cause further alternation of nearshore hydraulics and the sediment budget.

One of the factors which can influence the performance of the facility is the hydraulic design conditions chosen for the development.

Tetra (2000), the design and construction company, chose a 10 sec, 5.34 m high, 1–in-50-year return interval design wave. They performed numerical analysis of wave transformation for the harbour site, under HWST (2.6 m above CDL) and deduced that a 5.34 m wave can decay by about 65 % over the reef crest, to a 1.9 m wave in the backreef/reef flat (Tetra, 2000). This decay is considerable for such a large wave during HWST conditions.

The Harbour Master in Nauru, divers, fishing boat operators and Nauru's Fisheries Department personnel report 4-5 m waves from the East, at the development site, under annual gusty winds, especially between December to February. These have caused run-up on coastal land (averaging 5 m above CDL), across the roadway in all instances.

Numerical calculation of run-up of a 1.9 m transformed wave (Tetra, 2000) under EHWST will not be significant and will be much less than 1 m. For the existing coastal area, which is levelled at about 5 m above CDL, this should not run-up beyond the roadway. However, the fact that 4-5 m waves under EHWST have been observed to run-up beyond the road, raises some doubt regarding Tetra's (2000) wave transformation/numerical analysis. Further, since these run-up and overwash have been observed for several decades on Nauru's East coast, it is the author's opinion that this is worth considering and valuable in the absence of instrumentation data. Tetra's (2000) wave transformation analysis therefore should be treated with some caution.

It is the author's opinion that wave transformation over irregular bottoms or those with arbitrary shape/s due to shoaling, breaking and reduction renders numerical equations unsolvable analytically. This opinion has also been expressed by Pilarczyk (1996) on his discussion of shallow-water wave transformation.

In coral reef environments, one can find the best examples of these types of irregular bottom, which includes hydrographic features like sandy shoals, depressions, lagoons, reef pinnacles, barrier reefs and patch reefs. This complicates the analysis of wave transformation, from open ocean, over the reef crest, and into the backreef.

For the harbour study, Tetra (2000) did not provide the source of their transformation equations. However, from analysis of their equation constituents, it is apparent that the Goda (1985) formulae, for random wave height were applied. In that analysis, Tetra (2000) computed the significant transformed waves where the ratio of wave height/wave length (5.34 m/162.3 m) < 0.20. As mentioned before, their computation indicated that the transformed wave was 1.90 m.

However, as also previously mentioned, this height is less than those observed at the site over many years. This again raises the question of the Goda (1985) method for wave transformation on reefs, and as applied by Tetra (2000), and further highlights the complicated nature of reef hydraulics associated with irregular hydrography. Goda (1985) himself also notes that there are problems associated with random wave transformation in the natural nearshore zone.

Despite the complications associated with wave transformation in reef environments and discussed above, it may be possible to assess wave transformation using another technique.

In the following analysis, numerical computation were done for only the design (5.34 m) and average wave heights (3 m) chosen for the harbour facility and under

EHWST conditions, as these represent extreme and common wave height respectivley.

If one considers the architecture of the reef environment at the harbour site, it is possible to compute a transformed wave. The reef crest is the best analog of a dynamically stable, submerged breakwater, situated below MSL and 1.5 m below CDL. In addition it has a relatively straight offshore slope of 25°-40° and the backreef is unusually flat and smooth for a reef environment, with a constant seaward grade of less than 1° (see Section 3.10).

The seaward slope angle influences overtopping and wave transformation. In addition, wave energy dissipitation is at the reef crest where there is the greatest wave attack.

To that end one may analyze wave transformation at the harbour site, using numerical equations proposed for a dynamically stable submerged breakwater (van der Meer; in Thorne et al, 1995).

For a 5.34 m, 1-in-50-year wave height proposed by Tetra (2000), wave period of 10 sec, a reef crest of about 12 m wide, a forereef slope of 1: 1.2, in fore reef water depth of about 10 m (seaward of the reef crest), under EHWST (freeboard level of -4.14 m; with the reef crest at –1.5 m below CDL) and under Easterly (the modal) wave approach, the transformed wave computed was 3.7 m.

This wave transformation corresponds to 30 % decay in wave height across the reef crest. Interestingly enough, this is consistent with a wave that will run-up and overtop the coastal road under EHWST, and also that, which has been observed by residents, at the site, for more than a decade.

For such a wave height, the size and density of boulders required to maintain stable structural conditions under the design/ and transformed wave, in the backreef, and on parts of the facility (e.g. on the groin and spending beach rip-rap), may be larger or more dense than those specified.

For the local dolomite limestone used at the harbour site, which is dense, but porous, and with an estimated unit weight of 2650 kg/m^3, and under a 3.7 m transformed wave, during EHWST, the nominal stone diameter required would be about 1 m. This is about 2-3 times the diameter specified by Tetra (2000). Tetra's (2000) diameter is 500-1000 kg/pc or about 18-35 % of that computed by the author (assuming a rock density of 2650 kg/m^3).

If a 3.7 m high transformed wave impacts on the vertical seaward face of the main breakwater, under an EHWST, overtopping of the structure will be about 0.457 m^3/sec. At mean water level (1.57 m above CDL) overtopping will be 0.13 m^3/sec.

In addition, the navigational channel will not cause as much wave decay, as over the adjacent intact reef crest. This is because it was dredged to –2.5 m below CDL (1 m deeper than the level of the existing reef crest), with a 30 m wide funnel-like entrance that narrows to 20 m. The freeboard height above EHWST level is therefore –5.14 m, estimated from Tetra (2000) designs, while the seaward channel slope is 1: 16.

If a 5.34 m high wave break over the navigational access channel, the transformed wave will be about 4 m high or about 0.6 m higher than the transformed wave over the intact reef crest (3.4 m). This will then run-up on the spending beach rip-rap, and enter the mooring area, causing choppy conditions to develop within the harbour.

Despite the fact that there is a spending beach of rip-rap, some reflection and refraction will occur on the landward side of the access channel.

It is therefore important and necessary to cater for routine and regular maintenance of the spending beach rip-rap so as to ensure that any rip-rap dislodgement, erosion or damage is repaired.

Some numerical analysis was also performed for average wave climate for the same reef architecture and harbour design.

For an average offshore non-broken wave height of 3 m (within a 1-year return interval computed by Tetra, 2000), with a 6 sec period, the transformed wave on EHWST, over the same reef morphology will be much smaller, at 2.4 m or 20 % decay.

For such a transformed wave, the required rip-rap for the groin, under EHWST, should be at least 0.65 m diameter or 741 kg (assuming a rock density of 2650 kg/m^3). The rip-rap required for stability at the spending beach should be 0.3 m or 60 kg, also assuming a rock density of 2650 kg/m^3. Overtopping of the main breakwater by a 2.4 m transformed wave will be about 0.07 m^3/sec, smaller, but nevertheless, noticeable.

5.0 PREPARATION OF AN ENVIRONMENTAL IMPACT ASSESSMENT

Environmental Impact Assessment (EIA) is a decision-making process, which is based on comprehensive analysis, that evaluates all planned projects or programs which might seriously affect the total (living and non-living) environment (Maharaj, 2000). A more general and less comprehensive analysis, usually done at the conceptual stage of a project is the Environmental Assessment (EA).

Such an evaluation should be prepared for any major development, for example, plans to build various facilities, including harbours, dams, highways, and power plants. It should be noted that EA or EIA is not a report, but a *process* of exploration, analysis and verification, leading to determination.

A Government agency also may issue a report for any large state, local, or private project that it will regulate or support financially.

The purpose of an EIA or an EA is to make a Government agency consider the possible environmental damage that might result from a project under its jurisdiction.

The appropriate Government agency, such as Pohnpei State EPA, must also consider alternatives to the project under consideration, so as to facilitate sound development, in an environmentally friendly manner.

The agency itself then decides whether to allow the project to proceed or whether to alter or abandon it. It bases its decision on certain Government and technical guidelines. For example, before building a highway, an agency must study how pollution from vehicles using it will affect the area.

The agency prepares a preliminary environmental impact statement (EA) and submits it to federal, state, and local agencies, as well as the general public. After reviewing their reactions, the agency issues a final report and makes its decision.

The EA and the EIA processes thus describe both governmental and analytical methods. The final result of the EIA is the Environmental Management Plan (EMP). This describes and schedules the countermeasures necessary for mitigating the negative environmental impacts and optimising the development initiatives. EA and EIA are therefore, a central part of the process of development review and control.

There are three main components for the preparation of an EIA. These are as follows:

2. Identifying the characteristics of the project site;
3. Describing the nature of the project; and
4. Evaluating the impacts for different dimensions.

This will then lead to the development of mitigation guidelines and the development of an EMP. For coastal development, such as boat harbour facilities, a full EIA should be done by the developer. This process will result in the preparation of a documents with guidelines for optimum development with minimum deleterious environmental impacts and it should present environmental data and discuss issues regarding the proposed development and include the following aspects, e.g. in UNEP (1997).

5.1 General Site Characteristics

This subsection briefly describes the character of the site in terms of significant on-site features and proximity to nearby features such as roads, towns, rivers, and other natural amenities. A site area map should be provided and its scale should be sufficient to clearly show site boundaries and elevation contours.

5.2 Plan of Sub-division

In this subsection, the plan of subdivision is depicted and generally described. Information provided in written and mapped form should address the following:

- ✓ the size and location of all sub-divisions;
- ✓ the size and distribution of transportation and utility networks;
- ✓ the proposed land use pattern;
- ✓ the distribution of major buildings and facilities such as administrative offices, waste handling facilities, settling ponds;
- ✓ the location of other facilities of note such as major berms, fences, utility corridors; and.
- ✓ a description of development phases.

5.3 Major Components

This subsection provides detailed information on the component facilities that will be installed in the estate to meet the needs of tenant industries. Such facilities might include but are not limited to:

- ✓ roads;
- ✓ docking facilities;
- ✓ electric transmission lines;
- ✓ water supply lines and pumping stations;
- ✓ sewage lines and treatment facilities;
- ✓ solid and hazardous waste disposal facilities;
- ✓ administration buildings and laboratories;
- ✓ fire-fighting facilities;
- ✓ ponds;
- ✓ commercial area and recreation facilities;
- ✓ permanent housing;

- ✓ construction camps;
- ✓ material laydown and storage areas;
- ✓ maintenance yards;
- ✓ coastal protection structures; and
- ✓ dikes, berms, and ditches.

Text, maps, drawings and cross sections should be used as appropriate to convey the following types of facility information:

- ✓ dimensions (area, height, depth);
- ✓ production capacities (number, volume);
- ✓ waste-handling capabilities (treatment levels, suitability for handling different materials and fluids, etc.);
- ✓ construction methods (cut and fill, excavation, piling); and
- ✓ construction materials (types, quantities, sources, storage areas, etc.).

5.4 Construction Management Plan

The construction management plan should consist of three parts. The first part should set out the proposed construction schedule for the overall project and each of the major project stages. The second part should discuss general construction practices and procedures that are being proposed either to expedite the building of the project or to control negative side effects arising from construction activities. Such measures might include:

- ✓ hours of activity;
- ✓ noise and dust control practices;
- ✓ procedures for controlling debris;
- ✓ procedures for the disposal of oils and grease from construction equipment;
- ✓ building temporary service roads around communities; and
- ✓ regulating traffic flows on local roads, etc.

The third part of this subsection should set out labor force projections for each of the major construction phases. A labor force curve showing men per month is a useful graphic. In addition to labor force this subsection might also indicate the number of construction vehicles that will travel to and from the site on a daily basis during the different construction stages.

5.5 Operation and Maintenance

A number of subjects should be addressed in this subsection. It ought to cover the overall operation of the facility and generally touch on the construction and operation of generic types of potential tenant industry.

Matters that should be addressed include:

- ✓ a description of the type and scale of the development;
- ✓ a description of the land-use regulations and construction and operating covenants that will be applied to industries locating on the site;
- ✓ a description of the general services that will be provided to tenants and an indication of how these services will be paid for, operated, and maintained;
- ✓ a description of how the facility will be monitored to determine whether they comply with prescribed construction and operating plans; and
- ✓ a description of emergency planning measures, such as spill containment, fires protection, and evacuation.

In discussing the various services in this section, care should be taken to address in comprehensive detail the operating procedures and practices that apply to:

- ✓ water supply, sewage, and effluent treatment;
- ✓ solid and hazardous waste handling and disposal; and
- ✓ air emission controls.

5.6 Key Issues

The purpose of this section is to scope environmental issues to determine which are most significant. Scoping helps to ensure that the ensuing environmental studies, analyses, and mitigation measures will be focused on environmental matters, which are judged to be important. In turn, it helps to ensure that effort and money are not wasted in dealing with issues that are of little importance or that are not relevant.

5.7 Method of Scoping

This subsection describes the various methods used by the study team to scope environmental issues relevant to the project undertaking.

Such methods could include:

- ✓ site visits and preliminary field work;
- ✓ discussions with local officials and community leaders;
- ✓ review of available literature dealing with projects of similar nature and environmental conditions in the local area; and
- ✓ experience of the proponent and study team.

5.8 Identification and Description

Based on the activities set out in the previous subsection the key issues and potential impacts should now be identified and described. Discussions should detail:

- ✓ identification of the environmental components that may be affected;
- ✓ an indication of the extent to which existing conditions will be altered;
- ✓ a determination of the location and area of potential impacts;
- ✓ an indication of whether the impacts are short or long term;
- ✓ an assessment of whether the impacts are reversible and
- ✓ determination of the ease with which potential impacts can be mitigated.

5.9 Environmental Conditions

This section describes in both qualitative and quantitative terms the existing conditions for those environmental factors determined in the preceding chapter to be important.

For each of the factors addressed, information should be provided with one or more of the following purposes in mind:

- ✓ assist in determining the environmental changes that will occur because of the project;
- ✓ assist with project design so that potential negative project effects are minimized; and

- ✓ assist with project design so that potential positive project effects are optimized.

5.10 Natural Environment

This section describes the natural environmental setting of the study area. It includes both living and non-living attributes. Factors and subjects that could be discussed include, but are not limited to the following:

Topography:

- ✓ general and characteristic landforms;
- ✓ elevation and datum;
- ✓ digital terrain model/DTM;
- ✓ slope architecture;
- ✓ slope hydraulics and runoff;
- ✓ river and channel hydrology; and
- ✓ grading of the site

Soils:

- ✓ general soil characteristics (pedology and engineering properties);
- ✓ soil chemistry;
- ✓ load-bearing capacity;
- ✓ permeability;
- ✓ shrink-swell characteristics
- ✓ existing and potential erosion;
- ✓ soil/rock and slope failures;
- ✓ problematic foundation; and
- ✓ soil hydraulics

Geology:

- ✓ geology;
- ✓ geotechnics;
- ✓ tectonic faulting; and
- ✓ seismic conditions.

Groundwater conditions:

- ✓ aquifers, aquicludes and aquitards (confined and unconfined);
- ✓ nature and extent of aquifer and recharge areas;
- ✓ groundwater wells;
- ✓ well hydraulics and pumping statistics;
- ✓ pollution and point-source pollutants; and
- ✓ groundwater water quality (physical and chemical characteristics).

Nearshore marine conditions:

- ✓ hydrography/depth of nearshore coastal waters;
- ✓ coastal and surf-zone hydrodynamics;
- ✓ beach water table;
- ✓ wave climate;
- ✓ tidal regime;
- ✓ current conditions and circulation;
- ✓ coastal oceanographic hazards and
- ✓ water quality (physical and chemical characteristics).

Climate:

- ✓ annual and seasonal rainfall distributions;
- ✓ annual temperature and temperature ranges;
- ✓ local and regional air quality;
- ✓ wind speeds, directions, frequencies and seasonal variations;
- ✓ meteorological hazards;
- ✓ potential for floods, hurricanes, other natural hazards; and
- ✓ frequency of inversions.

Noise levels:

- ✓ ambient noise levels.

Vegetation and wildlife:

- ✓ terrestrial and aquatic vegetation (species composition, abundance, and habitat significance);

- ✓ terrestrial and aquatic animal life (species composition, abundance);
- ✓ rare and/or endangered species;
- ✓ unique natural systems (stream systems, wildlife breeding areas, etc.);
- ✓ resource areas (fisheries, plantations, forests, etc.); and
- ✓ the degree of susceptibility of any of the above have to pollutants that could be discharged from the project area.

For all of the parameters, maps should be used whenever practical to show the location and distribution of the phenomena in the study area. Where sampling and monitoring were carried out to generate information, such as might be the case for water- and air-quality data, the sampling location should be mapped and the frequency or duration of sampling described.

5.11 Socio-Economic and Cultural Environment

The socio-economic and cultural environment within the study area is described in this subsection. It encompasses those aspects of the environment, which relate to man, his activities, his perceptions, and his beliefs. Factors and subjects that could be discussed include:

Social and community conditions:

- ✓ population characteristics (size,) demographics, distribution, etc.);
- ✓ community characteristics (attitudes, behavior, cohesion, lifestyles, health, etc.);
- ✓ community facilities and services (schools, hospitals, recreation, etc.); and
- ✓ housing characteristics (types of housing, occupancy levels, age, conditions etc.)

Land use and resource areas:

- ✓ land use and resource areas (types, patterns, density, intensity of usage, areas, and lot sizes);
- ✓ infrastructure (water, sewer, solid waste, energy and transportation facilities);
- ✓ zoning controls and regulations in effect; and
- ✓ regional plans or resource strategies in effect.

Aesthetic and cultural resources:

- ✓ historical, archaeological or architectural sites;
- ✓ scenic areas, views, and natural landscapes; and
- ✓ other areas or features of unique cultural importance.

Economic conditions:

- ✓ employment and unemployment patterns, including occupation distribution and availability of workforce;
- ✓ income levels and trends;
- ✓ economic base of area;
- ✓ land ownership patterns; and
- ✓ land value.

It is strongly recommended that maps be used wherever possible to show the location and distribution of socio-economic and cultural features within the study area.

5.12 Environmental Impacts

This section clearly sets out the possible positive and negative impacts that the project could impose on the environment. For each factor the possible impacts and subsequent mitigative actions for the construction and operation phases should be addressed separately. A format similar to the following is suggested for each component:

Environmental effects

- ✓ the potential direct and indirect effects associated with the project;
- ✓ the description should include the benefits and adverse environmental effects; and
- ✓ location, area, importance, duration, and reversibility should be discussed for each impact.

Mitigation/corrective measures

- ✓ ways of mitigating environmental effects should be described;
- ✓ commitments to mitigation should be specified;

- ✓ if the level of information available at this stage is insufficient to permit a determination of mitigation measures, an indication should be made of what further work is required;
- ✓ any commitments to undertaking further investigations should be specified, and the schedule and manner in which this work will be undertaken should be described;
- ✓ special contingency plans on procedures to be used in case of emergencies should be described; and
- ✓ procedures to optimize positive impacts should be set out.

5.13 Conclusions

This final section briefly reviews findings of the study across the various environmental factors. It notes the potential impacts and indicates whether mitigation measures can alleviate all concerns, and if they cannot, it identifies the residual impacts.

This section follows the sequence of environmental factors presented in the last two chapters. In essence, it is an overall summation of the environmental soundness of the proposed project.

5.14 Environmental Management Plan (EMP)

One of the important first steps in establishing an Environmental Management System (EMS) is to understand the range and diversity of environmental issues to be addressed.

The list of issues is no longer than many managers at first believe. The relationship between issues is also an important factor, for action on one issue can easily affect the estate's performance on another. The preparation of a comprehensive environmental assessment report is thus an important first step. Some of the specific management elements, which contribute to improving environmental performance, are described below.

Elements of an environmental program:

1. Sound policies and clear objectives, which define environmental issues and identify the estate's approach, such as emphasis on prevention rather than treatment.
2. Well-defined operating standards and realistic targets for discharges and site safety.

3. Visible and effective management commitment to environmental protection.
4. Clearly defined line management responsibility and accountability.
5. Adequate resources for the program.
6. Regular review of environmental performance e.g. audits.
7. Programs on training and awareness on environmental risks.
8. Effective incident reporting and investigation.
9. Effective contingency planning for accidents spills and fires.
10. Reporting systems within the estate, and with the public.

The individual management elements may already be included in formal standards for environmental management systems, such as ***ISO-14001***, on Environmental Management Systems.

A clear statement of overall environmental policy greatly helps the various initiatives to function in a coherent manner. Such policy is often framed in a simple way to allow easy communication to employees and the public. A more comprehensive implementation document should be available to provide detailed guidance for operational managers.

A policy statement may include principles, objectives, definition of responsibilities, and an outline of the means to accomplish the goals. In addition to reaffirming regulatory compliance, quantitative environmental targets should be given wherever possible.

Open reporting and communications is often included as a policy component. The policy and systems will not function without a sound corporate structure that facilitates environmental action on the site, provides comments to top management, and responds to issues as they develop. Every industry has a different structure and no one reporting and responsibility model is universally applicable.

Nevertheless some of the common elements are:

1. Responsibility for environmental performance at a high level, preferably the chief executive.
2. Review of environmental performance by the Board;
3. A director with responsibility for environmental co-ordination;
4. An environmental committee to enable input from all levels to be considered;
5. Independence at the working level to see that the most appropriate environmental action is taken;
6. Good communications;

7. Use of environmental specialists for certain tasks; and
8. Technical back up for environmental services.

5.15 Guidelines for Preparation and Site Grading

An acceptable plan for these activities usually consists of two parts:

1. A narrative report describing the development (including the scheduling or phasing of major construction activities), and explaining the methods, techniques, and procedures (including maintenance of control measures) to be followed.
2. A map (or several maps of the same scale) or a base map with overlays, depicting the topography and natural features of the area, the limits for clearing and grading, existing and anticipated erosion problems, and the location of suitable control measures. The map should be an integral part of any site plan, grading plan or construction drawings.

Conservation practices for erosion and sediment control should meet or exceed guidelines and specifications established by government.

Conservation practices needed to control accelerated erosion and sedimentation. The degree of slope, nature and types of soil, drainage characteristics, proximity to property boundaries and watercourses, area disturbed, amount of cut and fill, and other factors all have a direct bearing on what combination of conservation practices will result in an adequate erosion and sedimentation control plan.

Great care must be taken in selecting the right control measure for each erosion site. Although erosion problems often share similar symptoms, their causes may differ significantly. For this reason, it is wise to undertake a thorough site investigation. This will help to determine the exact nature of the problem and how to correct it. Unless the actual causes of a problem are adequately determined, the applied remedial measure may fail to correct it, and may even aggravate it.

The selection, design and implementation of effective erosion and sediment control measures require a clear identification of efforts. It is important to avoid an indiscriminate choice of measures, but rather to select those that appropriately meet the specific problems for financial as well as ecological reasons.

A broad classification of erosion and sediment problems such as those presented below provides a basis for considering categories of problems and control strategies.

1. An erosion problem exists where damage attributable to erosion involves the direct loss of soil, which in turn, can mean the loss of roadways, the

undermining of structures, and other damage necessitating costly repair.

2. A sediment problem exists where there is damage associated with the deposition of eroded materials at a down location for example, clogging of culverts, filling of drainage ditches and stream channels, silting of ponds and reservoirs, and contamination of downstream waters by sediment-borne pollutants.
3. Problem Type 1 involves an erosion problem but no sediment problems. Such a situation may occur where locally-eroded sediments, even in substantial quantities, are transported and deposited relatively short distances downslope or within the construction boundaries, but do not move into a waterway system.
4. Problem Type II involves both an erosion problem and a sediment problem. This type of situation can result from substantial material being eroded and transported into downstream ditches and stream channels.
5. Problem Type III involves a sediment problem only. This type of situation may occur when the direct loss of soil is insufficient to create local damage at the erosion sources, but the accumulated sediment transported downstream creates depositional or water quality problems.

Recognizing the wide variations from one site to another, the following elements are to be considered in the development of the site grading and erosion and sedimentation control plan.

A general statement of the development must be included in the narrative section of the plan include:

1. Description of the overall development.
2. Date that the development is to begin and expected date that final stabilization will be completed.
3. Description of erosion control program.
4. Description of stormwater management program.

The plan is to include cross sections showing approximate elevation relationships between buildings, parking yards, streets, and adjacent properties at key locations. Elevations, slopes, and gradients of major installations are to be identified, and the proposed alterations to the existing topography illustrated.

The topographic features are to be shown on a topographical map, which is also to include map scale and north arrow.

Also, the map is to show:

1. The location of the development relative to highways, property boundaries, buildings, water supplies, and other identifiable landmarks or significant features.
2. Contours at an interval and scale that will adequately describe the area before, and following construction.
3. Critical environmental areas located within, or in proximity of, the development areas, such as streams, lakes, ponds, wetland areas, drainage ditches, flood plains and wells.
4. Nature and extent of existing vegetation.

Information on the soils presented in the narrative and shown on the map is to be provided. This information should include:

1. Adequate description of each soil, including type, texture, slope, depth, drainage and structure.
2. Surface area of each soil (soils data is readily available in those areas for which modern soil surveys are either completed or in progress. In the absence of a soil survey, a mechanical analysis of the soil should be made to the depth of the planned disturbance. Alternatively, a qualified engineer should make an on-site evaluation.)

The stormwater management program is to be described in the narrative and the location of facilities shown on the map.

The description of the stormwater management program is to include:

1. The anticipated amount of runoff from the area and the upstream watershed; runoff-producing factors and methods of calculation.
2. Analysis of problems posed by storm runoff on downstream areas.
3. Analysis of local drainage factors which may contribute to on-site or off-site problems.
4. Description of the permanent measures and facilities designed to cope with the problem/s.

The proposed alterations of the area are to be shown on the map and are to include:

1. Boundary limits and acreage of the development.

2. Limits of clearing and grading
3. Areas of cuts and fills and proposed side slopes.
4. Location for roads (including stream crossings), buildings, storm sewers, and other structures.
5. Location and protection of stockpiles of excess fill or topsoil.

The temporary erosion and sedimentation control measures (vegetative and mechanical) to be used during active construction are to be included in the narrative and shown on the map and are to include:

1. Purpose of control measures.
2. Types of measures, facilities, and expected length of service.
3. Location of measures and facilities.
4. Dimensional details of the facilities.
5. Design considerations and calculations (if applicable).

The permanent erosion and sedimentation control measures for long-term protection are to be included in the narrative and shown on the map including:

1. Purpose of control measures.
2. Types of measure and facilities.
3. Location of measures and facilities.
4. Dimensional details of facilities.
5. Design considerations and calculations.
6. Landscaping or vegetative details such as seeding, sodding or mulching.

The maintenance program for the control facilities is to be described in the narrative and is to include:

1. Inspection program, including frequency and schedule.
2. Re-sodding or re-seeding vegetated areas.
3. Repair or reconstruction measures.
4. Method and frequency of removal.

6.0 CONCLUSIONS

This report presents an analysis and assessment of the impacts of harbour development at Anibare Bay, Nauru.

It presents and describes the scope of the study, the data set collected, the methods and standards used, and some results. In addition, it presents discussions on the dynamics of the site, the present coastal erosion problem and implications of harbour engineering along the coast in Anibare Bay. Details of beach profile data; beach sediments, the harbour design and construction activities and the impacts of development on the natural and built coastline are presented.

In summary, the coast is part of an emergent, Holocene reef-carbonate system, with the beach being comprised entirely of carbonate sediments developed on phosphate-rich, cavernous, dolomite limestone bedrock. The coastline is partly rocky with classic karst limestone pinnacles found throughout the bay. The reef is a coral dominated system and is narrow and well-flushed, with many closely-spaced reef channels. The coastline at Anibare Bay is an active and dynamic one. The relatively coarse admixture of abraded sand and gravel and highly abraded karts pinnacles testify to this. The beach at the development site is moderately steep and has experienced erosion in the recent past and show sign of recent and current erosion, with fresh erosion scarps. The harbour development appears to have increased this erosion at the localized level, at the adjacent, undeveloped coastal segments. The topographic elevation of the coastal land areas are relatively low with respect to CDL and MSL and under EHWST or during windy and low pressure systems, when large (3 m +) waves approach shore, from the East, the beach, coastal road and adjacent areas can be easily overtopped. The relatively narrow and almost flat backreef and reef flat, together with the numerous closely-spaced reef channels make it almost impossible to dissipitate significant wave energy and prevent overtopping during these conditions. In addition, the almost featureless backreef cannot trap sediments entrained in longshore currents and therefore, sediments removed from the local areas can be completely lost from the coastal system in Anibare Bay.

The harbour development acts as a large groin, which breaks the continuity and smoothness of the concave Anibare Bay. As a result it interrupts Southerly longshore currents and will cause erosion of downdrift areas (to the South). By its very nature, the harbour also acts as a headland, protruding into the bay. As a result of this "headland-like" morphology, wave diffract around it, and cause much agitation and disturbance of beach sediments immediately North and South of the harbour. Therefore, the facility can cause erosion on both the North and South aspects of the harbour.

With respect to the concrete breakwaters built to protect the harbour and mooring basin, these structures are already overtopped by 3 m high spring tide waves. While the design and construction firm (Tetra, 1999) indicated that a 50-year, of 5.34 m design wave was used to design the harbour, the fact that a 3 m high offshore wave, after undergoing decay over the reef crest, can overtop the main breakwater, raises some concern as to what acceptable risks were allowed/selected for this facility.

To that end some numerical analysis was performed for the facility. Numerical computation were done for only the design (5.34 m) and average wave heights (3 m) chosen for the harbour facility and under EHWST conditions, as these represent extreme and common wave height respectivley. The reef crest was modeled as a dynamically stable, submerged breakwater, situated below MSL and 1.5 m below CDL. In addition it has a relatively straight offshore slope of 25°-40° and the backreef is unusually flat and smooth for a reef environment, with a constant seaward grade of less than 1° (see Section 3.10). Wave transformation was done using numerical equations proposed for a dynamically stable submerged breakwater (IHE-Delft, 1999).

For a 5.34 m, 1-in-50-year wave height proposed by Tetra (2000), wave period of 10 sec, a reef crest of about 12 m wide, a forereef slope of 1: 1.2, in fore reef water depth of about 10 m (seaward of the reef crest), under EHWST (freeboard level of -4.14 m; with the reef crest at –1.5 m below CDL) and under Easterly (the modal) wave approach, the transformed wave computed was 3.7 m.

This wave transformation corresponds to 30 % decay in wave height across the reef crest. Interestingly enough, this is consistent with a wave that will run-up and overtop the coastal road under EHWST, and also that, which has been observed by residents, at the site, for more than a decade.

For such a wave height, the size and density of boulders required to maintain stable structural conditions under the design/ and transformed wave, in the backreef, and on parts of the facility (e.g. on the groin and spending beach rip-rap), may be larger or more dense than those specified.

For the local dolomite limestone used at the harbour site, which is dense, but porous, and with an estimated unit weight of 2650 kg/m^3, and under a 3.7 m transformed wave, during EHWST, the nominal stone diameter required would be about 1 m. This is about 3-5 times the diameter specified by Tetra (2000). Tetra's (2000) diameter is 500-1000 kg/pc or about 18-35 % of that computed by the author (assuming a rock density of 2650 kg/m^3).

If a 3.7 m high transformed wave impacts on the vertical seaward face of the main breakwater, under an EHWST, overtopping of the structure will be about 0.457 m^3/sec. At mean water level (1.57 m above CDL) overtopping will be 0.13 m^3/sec.

In addition, the navigational channel will not cause as much wave decay, as over the adjacent intact reef crest. This is because it was dredged to –2.5 m below CDL (1 m deeper than the level of the existing reef crest), with a 30 m wide funnel-like entrance that narrows to 20 m. The freeboard height above EHWST level is therefore –5.14 m, estimated from Tetra (2000) designs, while the seaward channel slope is 1: 16.

If a 5.34 m high wave break over the navigational access channel, the transformed wave will be about 4 m high or about 0.6 m higher than the transformed wave over the intact reef crest (3.4 m). This will then run-up on the spending beach rip-rap, and enter the mooring area, causing choppy conditions to develop within the harbour. Despite the fact that there is a spending beach of rip-rap, some reflection and refraction will occur on the landward side of the access channel.

It is therefore important and necessary to cater for routine and regular maintenance of the spending beach rip-rap so as to ensure that any rip-rap dislodgement, erosion or damage is repaired.

Some numerical analysis was also performed for average wave climate for the same reef architecture and harbour design.

For an average offshore non-broken wave height of 3 m (within a 1-year return interval computed by Tetra, 2000), with a 6 sec period, the transformed wave on EHWST, over the same reef morphology will be much smaller, at 2.4 m or 20 % decay.

For such a transformed wave, the required rip-rap for the groin, under EHWST, should be at least 0.65 m diameter or 741 kg (assuming a rock density of 2650 kg/m^3). The rip-rap required for stability at the spending beach should be 0.3 m or 60 kg, also assuming a rock density of 2650 kg/m^3. Overtopping of the main breakwater by a 2.4 m transformed wave will be about 0.07 m^3/sec, smaller, but nevertheless, noticeable.

7.0 RECOMMENDATIONS

Management of built shorelines, like those in Anibare Bay, is a dynamic process based on assessments and re-assessments and therefore, cannot be pursued by targeting specific activities, continuously, through time. Shoreline management strategies must be developed to reflect the current and future/forecasted needs. Forecast should also be for the short-term, medium-term and long-term, and therefore, strategies must be developed which reflect these changing needs through time. However, management strategies must reflect site dynamics, which may be summarized as follows:

1. The development site is part of a dynamic open-ocean coast;
2. The beach is protected by a narrow (12 m wide) fringing coral reef, with many closely spaced reef channels;
3. The beach is narrow and of carbonate sand, with a moderate slope;
4. The beach and shoreline has been subject to natural erosion in the past and recent years;
5. Surf zone hydraulics show that on breaking waves run-up the backreef, unto the beach, without any further decay;
6. Backwash is strong across the backreef and on mean or low tide drains the backreef;
7. The entire harbour acts as a groin along Anibare bay and interrupts Southerly longshore currents;
8. The facility also diffract Easterly approaching waves to the North and South;
9. The harbour construction has therefore already caused alteration of surf zone hydraulics and caused local eddys to develop;
10. Eddys already cause local scouring at the toe of various harbour structures (breakwaters, rip-rap, groin and spending beach);
11. The beach and shoreline has been affected by harbour construction and erosion has exacerbated;
12. Scouring and erosion will continue in the immediate future;
13. Fine sediments (sand and silt) will become easily suspended and eroded from adjacent beaches, in response to scouring and eddying under wave attack;
14. Erosion is on both the updrift and downdrift aspect;
15. Updrift erosion has resulted from wave diffraction around the structure from Easterly waves;
16. Downdrift erosion is due to interruption of longshore currents and sediment transport and sediment starvation;
17. The downdrift aspect is more eroded than the updrift side;
18. The beach on the updrift aspect has completely disappeared and the scoured underlying bedrock is exposed;
19. The main breakwater is overtopped by present EHWST and will continue to be overtopped;
20. It is expected that any waves generated by low-pressure systems and which affects the harbour site, will also overtop the harbour breakwater and run-up on land across the road;

21. Based on analysis of Tetra's (2000) design wave, their estimated transformed wave (1.9 m) over the reef crest is smaller than those wave heights observed by mariners in Nauru (3 m +);
22. Estimates of a transformed wave over the reef crest (modeled as a submerged breakwater) show a higher wave height (3.4 m) than that predicted by Tetra (2000), and is consistent with those observed by maritime personnel in Nauru;
23. The above suggests that there will be greater overtopping and run-up for the harbour site than those predicted by Tetra (2000);
24. The navigational channel access is deeper than the water over the reef crest and will therefore facilitate large waves to enter the harbour and mooring basin, especially under EHWST;
25. Waves entering the access channel, in the backreef, will be larger than those North or South of the habour facility; and
26. Larger waves entering the mooring basin will create some choppy conditions within the harbour, even though some dissipitation will occur on the spending beach (rip-rap) slope.

In relation to the above, and to prevent any further erosion or exacerbation of shoreline retreat, the following programmes will be needed at the harbour development site. Failure to implement these will allow erosion or damage to coastal facilities and infrastructure to go un-noticed, which may cause further negative impacts along the shorefront. Recommendation include:

1. Eroding sections of the coastline need to be protected immediately, if erosion is to be arrested or stopped;
2. If eroding coasts at the harbour site is left un-protected, it will cause further loss of coastal soils, beach sediments and damage to the adjacent road or other infrastructure;
3. Protection strategies should be for the immediate and medium-long term;
4. A bio-engineering system of coastal protection should be employed;
5. To prevent overtopping of coastal areas, a rip-rap system could be utilized along the edge of the coastline, at the land-sea interface;
6. Biological protection can mean planting of locally adaptable coastal tree and shrub species;
7. It would be best to select and plant species of flora which are adaptable to the existing coastal conditions, especially those which grow along Nauru's coastline (see SPREP, 1998);
8. I would recommend the use of geotextile or erosion mats for erosion control, if available, as these can trap sediments within their structure;
9. Any geotextile selected should be appropriate for the hydraulic and tropical UV conditions;
10. Dolomite limestone rip-rap can also be used for coastal protection;
11. Limestone rip-rap is also recommended because it is cheap, available locally and easy to use;
12. The local limestone is also of good/suitable density for coastal engineering protection structures;

13. Attempts should be made to select boulders without phosphate residue (to prevent eutrophication in coastal waters);
14. Rip-rap should be sized based on wave and surf-zone hydraulics;
15. All design should be performed by a competent professional;
16. A coastal monitoring programme should be put in place to periodically assess any changes to the coastline at the harbour site;
17. Monitoring will facilitate rapid repair to damaged infrastructure, facilities and eroding coasts;
18. Monitoring should be beach profiling, beach sediment sampling and coastal littoral hydraulic assessment;
19. Visual inspection of failure characteristics and wave run-up/overtopping should be documented and photographed when they occur;
20. This monitoring should be at least quarterly in the first 3 years, which can be decreased to twice yearly thereafter;
21. The coastal monitoring data should be reviewed as soon as possible and coastal management strategies modified/revised to reflect any changes or new developments at the site;
22. All beach profiles and surveys should be levelled from Nauru's national suryeying benchmarks;
23. The data collected should be archived in a database for easy access and retrieval;
24. *SOPAC Training Reports 84* should be consulted for details of set-up of a beach monitoring programme;
25. Future coastal developments should be preceded by an environmental impact assessment (EIA);
26. An EIA, if properly done, would identify problems and positive attributes of the development and would assist developers and project managers in planning for and decreasing any deleterious impacts at the project site;
27. This report provides some general guidelines for conducting an EIA and the EIA process.

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APPENDIX I

NR 1999.045

MONITORING AND ASSESSMENT OF ANIBARE SMALL BOAT HARBOUR
ANIBARE BAY, REPUBLIC OF NAURU (RON)

Country: Nauru *Task: NR 1999.045* *SOPAC: Coastal Unit*
Proposed: October 1999 *Started: February 2000*

Objectives: 1. Evaluate coastal processes in the vicinity of Anibare small boat harbour.
2. Evaluate the stability of the shoreline in the harbour areas.
3. Identify impacts (positive and negative) of harbour construction.
4. Propose management and engineering plans/options for the coastal area.

Proposed Output: Technical reports detailing the results of the field studies, data analysis and outlining recommendations for management of the harbour area and coast.

Background: The Government of Nauru is in the process of constructing a small boat harbour in Anibare. The harbour is located in a dynamic coastal area, which is subject to erosion and sediment transport and also the possible threat to adjacent coastal infrastructure. In response to needs expressed by the Government of Nauru, SOPAC proposes to assess and evaluate the stability and dynamics of the shorelines in the vicinity of the harbour and comment on possible solution/s to any foreseen problems associated with construction of the said facility. In addition, management plans for the said coastal areas will be prepared for the project.

Equipment: Field GPS unit, survey equipment, laptop and desktop computer resources, camera, sediment/soil sampling equipment and laboratory testing facilities. Coastal engineering software (CRESS and ACES).

Work Plan: Analyse the harbour project scope of works.
Analyse any pre-construction field data and reports for the said area.
Conduct beach profile surveys in the area.
Collect beach sediments and coastal soils.
Described coastal geology and soil conditions in the field.
Test soil and sediments for engineering geological and hydraulic properties.
Analyse historical aerial photo or remote sensing data.
Evaluate coastal stability.
Identify management and engineering options.
Identify recommendations.

Information: Project scope of works.
Clients: Government of Nauru (RON).
Personnel: Russell Maharaj.
Other: Nauru Government Personnel.

Reports: *SOPAC Preliminary Report 127* and *SOPAC Technical Report 316.*

Funding: CFTC and SOPAC.

APPENDIX II – BEACH PROFILES AB-5, AB-6 AND AB-7