EVALUATION OF IMPACTS OF HARBOUR ENGINEERING

ANIBARE BAY, REPUBLIC OF NAURU

May 2003 *SOPAC Miscellaneous Report 506*

Russell J. Maharaj,
Training Co-ordinator
SOPAC South Pacific Applied Geoscience Commission
Postal Address: Private Mail Bag, GPO, Suva, Fiji Islands
Street Address: Mead Road, Nabua, Fiji Islands
Tel: +679 338 1377 Fax: +679 337 0040
E-mail: russellm@sopac.org Web site: http://www.sopac.org

Asian and Pacific Coasts 2003

EVALUATION OF THE IMPACTS OF HARBOUR ENGINEERING IN ANIBARE BAY, REPUBLIC OF NAURU[1]

Russell J. Maharaj

This paper presents the evaluation of impacts of harbour engineering in Anibare Bay, Republic of Nauru. The bay and beach is of carbonate sand and gravel, behind a narrow reef and backreef developed on dolomite a substrate. The bay is dynamic and open to the ocean, with active erosion at the development site. The beach is moderately sloping and shows sign of current erosion, with fresh scarps. Harbour development has increased this erosion and erosion has exacerbated. Updrift and downdrift erosion has resulted from wave diffraction around the structure by Easterly modal waves. Structures are frequently overtopped, with noticeable run-up. Rip-rap specified for construction is too small, while breakwater heights are conservative for the prevailing hydraulic regime, while flanking is an immediate problem. Monitoring and post-construction analysis is recommended, while maintenance is needed for already eroded beaches. These will facilitate timely repair to damaged infrastructure and eroding coasts. It is recommended that future coastal developments should be preceded by an environmental impact assessment.

Keywords: Nauru; harbour; coastal development; coastal protection; erosion; impacts.

1. Introduction

This paper presents results of assessment of coastal impacts of harbour construction at Anibare Bay, Republic of Nauru (RON; Figures 1-3). A site visit was made, and environmental data collected to document the response of the beach system to the development. The purpose of this study was to assess the coastal development; evaluate any coastal problems; and assess future shoreline management strategies. Key engineering and environmental issues are discussed with regards to the harbour design, construction and maintenance.

Anibare District is located in the Eastern part of RON. The development site is Anibare Channel, South Anibare Bay. The development site is a former excavated channel which was used for fishing. The purpose of this harbour facility is to provide RON with launch and berth to support their tuna fishery. Coastal development on a small island state, like RON, can impact

---

[1] *Refereed paper published in: W. Kioka et. al. 2003. Proceedings of the Second International Conference on Asian and Pacific Coasts (APAC2003), 7-11 September 2003, Chiba, Japan. CD-ROM, Version 1.0.*

adversely on the coast. Since RON already has a small area of only 21 km^2, the erosion of any land will be a significant loss. In addition, developing economies, like RON, can be seriously affected by damage to or loss of civil infrastructure from “silent” natural hazards like coastal erosion. Therefore, addressing these coastal problems is of paramount importance to Nauru’s coastal communities. 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.

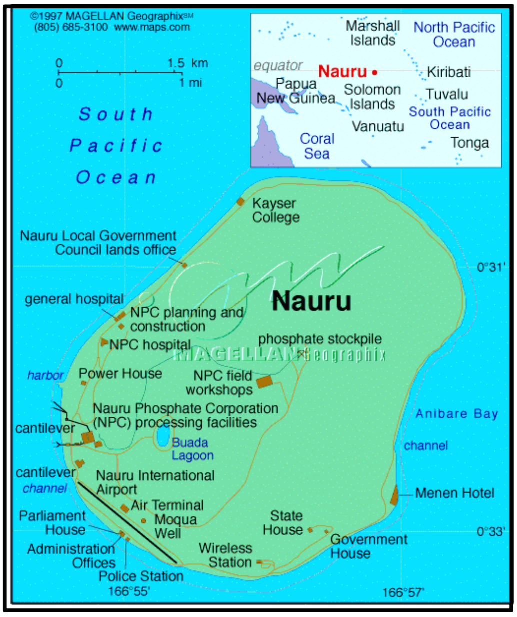

Figure 1. Nauru, Anibare Bay and channel (Magellan Geographix 2000).

2. Methodology

The harbour development plans were consulted for quantity estimates. Geological and geotechnical data were collected, including wave and littoral information, beach sediment and erosion characteristics, and documentation of damage to infrastructure. Sediments and rock were characterised according to the American Society for Testing Material (ASTM 2000). Seven beach profiles were measured with a Sokkia and Garmin, Global Positioning System (GPS) instruments and corrected to Nauru’s Chart Datum (CDL). Erosion scarps were measured. Wind speeds and direction were measured with a digital anemometer. Positions in the field were determined with a Garmin hand-held GPS. Information on waves was also obtained from Tetra (1999) and 1992 vertical, aerial photographs. Dolomite strength was measured in-situ, with an ELE Schmidt L-Type Hammer. Concrete strength was measured with an ELE Concrete Testing Hammer. Rip-rap and revetment evaluation are based on design criteria of CIRIA (1991) and Geological Society of London (GSL 1999). Numerical analysis has been produced with IHE-Delft (1999) and the U. S. Army Corps of Engineers, Automatic Coastal Engineering System. Computations of rip-rap dimensions, armourstone stability and wave run-up are based on van de Meer’s (1987 and 1998) modifications of Hudson (1958) formula.

Figure 2. The harbour. Left – mooring basin/wharf, with quay wall and apron; top right – sub-breakwater; top center – rubble groin with rip-rap spending slope, lower center – rubble groin.

Figure 3. Beach morphology and erosion. Note exposed dolomite substrate on the lower beach and erosion at beach-land interface.

3. Results

3.1 Geology

Nauru Island is an emergent coral atoll, just South of the Equator, between 0° 30" – 0° 34' S latitudes and 166° 54'' – 166° 58" E longitudes. Nauru is located at the Southeast end of the Nauru Basin, a 4-5 km deep ocean basin extending from the Southwest of the Marshall Islands, at approximately 6? N, to the Ontong Java Plateau in the Southwest. 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, extending from the Solomon Trench, just Northeast of the Solomon Islands, to about

10? N latitude. Nauru evolved between the mid-Eocene to Oligocene (Jacobson et al., 1997), and is an emergent part of a submarine basalt seamount, created by hotspot volcanism, coinciding with major plate re-organization in the central Pacific. Jacobson and Hill (1993) 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 Holocene emergence, produced the present 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, and sub-sea circumference (Jacobson and Hill 1993). As a result of its small circumference and deep nearshore waters, Nauru has sharp, steep offshore slopes. These steep slopes, combined with a narrow reef crest and backreef (less than 250 m wide) cause the island to be affected by ocean processes, with very little wave transformation over the reef. The Pacific Plate, at Nauru, is also presently moving towards the Northwest at about 104 mm/yr (Minster and Jordan 1978).

Anibare Bay is located in the Anibare District, one of the large districts in Eastern Nauru. Anibare Bay is about 2.5 km long, extending from Mennen Hotel in the South, into Ijuw District in the North. The bay is arcuate and largely asymmetrical, with an active eroding coastline. It is the most indented part of the coastline of Nauru. However, this indentation is tectonically controlled. Vertical aerial photographs 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. 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 . This feature has not been subject to analysis before, but has been mentioned briefly in other reports (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. 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. This deposit extends to approximately 2000 m below mean sea level. The scarp of the landslide has produced Anibare Bay. The width of the bay is the width of the main scarp of the landslide, 2.5 km. Vertical aerial photographs reveal an arcuate lineament, which defines the landslide scarp, produced by slope failure. This lineament is also more heavily vegetated, 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). Northeast-trending fault lineaments are also found to the North of Anibare Bay, in Ijuw District (Jacobson et al., 1997). These lineament extend 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 North. At the harbour site, elevations range from 2.32 – 4.05 m above mean sea level.

3.2 The shorefront

The coastal areas of Anibare Bay has several shorefront settlements, and is connected by the major coastal, dual carriageway, sealed, flexible pavement asphalt road (Figure 1). This coastal road is the main transportation artery on the island, and is situated almost along the edge of the shoreline in Anibare Bay. The coastal land is vegetated with shrubs and coconut palms. The shrub line is at the roadway’s edge. Coastal soils are found only in shallow depressions and pockets and are sandy. 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 cohesionless. The underlying bedrock in the Bay is Pleistocene to Pliocene dolomite (Jacobson et al., 1997). This dolomite was elevated in the late Tertiary to Holocene, due to uplift of the Pacific seafloor. The dolomite exposed in Anibare Bay consists primarily of re-crystallised Mollusca and coral skeleton, calareous algae and other invertebrate reef fauna. Jacobson and Hill (1993) note that the limestone in Nauru are packestone and grainstone, and were deposited in a saline lagoon/backreef environment. As mentioned, Mollusca were identified in specimens from the Bay. Dolomite outcrops show considerable re-crystallisation and infilling of primary porosity, which has produced a fine-grained, dense dolomite. The limestone in Anibare Bay is porous, with many pitted solution cavities, some up to 5 cm long, created by secondary, chemical solution and Pleistocene to Holocene tectonic fracturing. The re-crystallized nature of this dolomite has also caused the development of low, inter-granular porosity. The micro-crystalline nature of this limestone cause splintering, producing highly angular fragments. Outcrops along this part of the coast occur as pinnacles, some up to 7.5 m high.

3.3 *Anibare Bay*

Anibare Channel, Anibare Bay, was used as a fishing facility for many years, and was the site of a reef-blasting in the early 1970's. The channel which was created was 10 – 12 m wide and 0.25 – 4.0 m deep (at high tide), with 90° side slopes and an 8° seaward slope. The channel was 80 m long, 6 m wide and 20 m long. Fishing boats were launched from and anchored at the site, from a 20 m long and 5 m wide, concrete ramp. The 4.0 m deep channel segment was cut in the reef crest removing live coral and other reef biota. The shallow end of the channel was backreef. The backreef in Anibare Bay is flat and featureless, dries during extreme low water spring tides (ELWST), when it is almost completely dry, except for small water-filed pools. ELWST for Nauru is about 0.84 m below mean water level. This causes a wider beach area to become exposed. The bay beach slopes 5-7° seaward.

The beach at the development site and in Anibare Bay is relatively narrow, with an average width of 5 m, maximum width of 8 m, and minimum width of 1 m. Beach slope varies. The beach is concave, with an almost constant grade. It is a medium to fine, carbonate sand, with medium to coarse gravel on the lower beach. The beach is about 6 m wide with a 12-15° seaward slope. Sands are clean, with less than 5 % fines. All sand is reef detritus, with more than 75 % coral. Molluscs, foraminifera tests, and Echinoderm tests comprise most of the remaining 25 %. The upper beach has a thicker sands, about 0.75 m, usually thicker during the first half of the year. Small coconut palm and coastal shrubs line the shorefront, which are affected by severe erosion. Erosion scarp are 0.80-1.75 m high; vertical, with some overhanging sections, exposing in-situ soil or rock. The backreef area varies between 90-110 m wide. The backreef is an elevated, Holocene, featureless platform, with few depressions, covered with cavernous, angular dolomite pinnacles. The interface of the lower beach and backreef area is a depositional site for medium to coarse, well-sorted, sub-rounded gravel of 100 % carbonate grains. About 90 % of all gravel fragments are coral, with less than 10 % comprising Molluscs fragments.

3.4 *The harbour*

The 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 (Figure 2). The dredging basin was excavated down to 2.5 m below CDL, while the channel and mooring basins 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. The debris was removed using a hydraulic excavator. 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 an 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 breakwater 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, 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. Local dolomite was used to engineer the groin. Rip-rap for this structure were 500-1,000 kg/m^3. The groin consists of an inner core of 50-100 kg/m^3 dolomite boulders and a 0.7 m thick, single outer layer of 500-1,000 kg/m^3. It is built on the backreef, which has an elevation of 1.5 m above CDL. The harbour was constructed in the backreef, in a maximum water depth of 3.0 m below CDL or 1.57 m below mean sea level (MSL). The maximum elevations on the facility are on the concrete breakwaters, levelled at 4.80 m above CDL. On EHWST, the breakwaters are only 2.16 m above water level.

3.5 *Erosion characteristics*

Figure 3 show some erosion characteristics of the beach. Rip-rap protection is being used for segments of eroding coast like this, South of the harbour site. This segment of eroding roadway developed during harbour development. Erosion scarps have exposed plant roots. Several shrubs fell on the beach at the site. Scouring of the upper beach and beach-land interface has exposed the underlying dolomite. This is common during EHWST, and is associated with Easterly wind waves and plunging breakers. One can see the typical scoured surface of the exposed limestone bedrock. Scouring became pronounced after construction. To date, the sand which was eroded has never recovered. The present erosion which has exposed plant roots, and is now threatening to erode the coastal road, behind the vegetation line. Eroded areas are also common along continuous segments of the beach, North and South of the harbour site. A typical 1.15 m high erosion scarp is present. One can observe large clumps of surficial soil, which toppled onto the upper beach. The beach fronting these scarp is typically narrow, with a thin sediment cover exposing the underlying dolomite. To arrest erosion, dolomite rip-rap was put in place. 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. 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. This is exactly what has already happened, and resulted in the erosion previously described. In addition, waves approaching from the East also cause some diffraction of surf around the facility and therefore, to the North and to the South. Consequently, areas immediately North of the harbour has be affected by erosion and has lost beach sediments. Such diffraction will also add to the erosion already being experienced to the South of the harbour.

For harbour facilities like this one, built on narrow reefs, adjacent to very deep ocean, it is important to realize that coastal engineering pre-disposes the facility to harsh, open-ocean waves, as there is a very narrow buffer zone to reduce the deleterious impact of large ocean waves. While there is a reef, this is extremely narrow, and do not reduce significant wave energy which impact Nauru's coast. Consequently, this facility will suffer considerable impact 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. 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 to trap sediment in motion, before they reach the reef crest and reef channels. While some sediment may accumulate downdrift, South, a significant quantity will be lost from the reef environments, to the open ocean, via reef channels.

3.6 *Wave hydrodynamics*

Waves are typically plunging. Wave height average 2.5 m. Significant wave height is 3.0 m. Typical plunging breakers crash on the beach, with rapid run-up causing significant scouring. With a narrow and almost flat backreef, any disturbance of beach sediments and their removal, can lead to cross-shore sediment transport and loss to deep-water environments. Most waves approach the shore at 015°. Longshore transport is to the South, from Easterly waves. Perpendicular waves do not generate longshore, but cause beach erosion. Westerly winds cause a Northerly longshore. Modal winds are from the East. Swells are common and approach from East-Northeast. Beach profiles are concave with gentle to moderate slope.

The site experiences Easterly wind between 4.5-18 m/sec. Westerly winds are strong during November-February (van Loon 1984 and Tetra 1999). Mean wind speed was 4.34 m/sec (Tetra 1999), with a 13.27 m/sec maximum. Maximum speed is observed in February, with a minimum in July. Nauru experiences the strongest wind from East, East-Southeast and East-Northeast between November to March, and usually above 4 m/sec. These Easterly winds represent 60 % of the wind estimates presented by Tetra (1999). Therefore, in terms of wind-wave exposure and wave hydrodynamics, Anibare Bay is located in the most dynamic part of the island.

Wave heights at the time of surveys varied between 1.50-2.0 m. However, during October field surveys, 3.0 m waves were observed. Breaker heights were comparable, while plunging breakers were the norm. Waves approached from 065-090°, similar to wind. From wave hindcasting, Tetra (1999) notes that waves predominate from North-Northeast to Southeast, with 0.5-1.0 m wave heights being 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, as they are propagated towards the East. Cyclones at Nauru's latitudes are not significant (van Loon 1984).

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, with 16-18 sec and 3.9-4.2 m high waves. For the design life of the harbour, Tetra (1999) used a 5.34 m or 50-year design wave.

Longshore current flows South, between 20-25 cm/sec. Significant sea spray was observed during gusty winds. On high tide, overtopping was observed at the main breakwater, with (millimeters thick) sea salt precipitation on the concrete surface.

Beach profiles have the following characteristics. The land at the harbour site and South of the harbour in Anibare Bay varies in height between 4.5-5.6 m above CDL. 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°, but can be up to 15°, with 4° upper beach slopes. Beaches to the South of the bay are steeper averaging 8°, with 4° upper beach slopes. The middle section of the bay 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 < 1° gradient towards the 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 has an average seaward slope of 11°. Shoreward of the plunge point, slopes measure 2°-2.3°. The plunge point has an average elevation of 0.5 m below CDL. The reef crest varies between 0.25-0.50 m below CDL. The fore-reef slope is 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. Therefore, any nearshore sediments, transported from the beach to the reef crest, can be easily transported offshore, by gravity flows once they enter the steep reef channel passages. The horizontal spacing of these channels is less than 15 m. 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 facilitates more rapid removal of sediment from the harbour site. Since longshore current is generally to the South, this navigational channel can also intercept Southerly flowing currents, causing sedimentation within the channel and diversion of longshore current. Diversion can be either into the harbour basin or offshore. Inner diversion can be expected when tides are high and rising and with large breaking waves, under strong onshore winds. Offshore diversion may occur on falling tides and on calmer seas.

4. Discussion

The harbour cause diversion and interruption of the Southerly longshore current in Anibare Bay. In addition, the incoming Easterly waves are diffracted, both to the North and South of the harbour. These cause local eddys, and erosion of loose carbonate sand and their removal. The sediment which has remained on the beaches, is primarily coarse gravel and boulder. In some cases, all the sediment has 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, by 2-3 m waves. This reflected energy does not facilitate sediment accretion in the vicinity of structures and the harbour, especially when the backreef is less than 10 m wide and backwash is strong. In addition, reflected waves combine with subsequent incoming waves causing the growth of these subsequent waves. Every third wave was found to be larger by at least 15 %. This increase in wave height is noticeable on EHWST and during choppy seas.

To the immediate South of the harbour, souring 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. Attempts were also made to arrest beach scouring. It is significant to note that this scouring and sediment removal continues, even though rip-rap have been placed at these sites. Since the harbour acts as a groin, sediment eroded from the site, is sediment lost from the coastal system, as most of this material is transported offshore. This eventually causes sediment starvation of downdrift Southern coastal segments, exacerbating erosion. The long-term evolution of this coast will therefore, be affected by this change in sediment supply and alteration in nearshore hydraulics.

The peformance of the harbour will also affect the long-term evolution of Anibare Bay and environs. If failure occurs, 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 condition chosen for the development. Tetra (2000) chose a 10 sec, 5.34 m high, 1-in-50-year 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 to a 1.9 m wave in the backreef. This decay is considerable. The Harbour Master in Nauru, divers, fishing boat operators and Nauru’s Fisheries Department personnel report 4-5 m waves from the East under gusts, especially from December to February. These cause run-up averaging 5 m above CDL.

Numerical calculation show that run-up of a 1.9 m transformed wave, under EHWST, will be less than 1 m. For the existing coastal area, 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) numerical analysis. Further, since this 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 in the absence of instrumentation data. 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 been expressed by Pilarczyk (1996). In coral reefs, one can find the best examples of irregular bottom, which include sand shoals, depressions, lagoons, reef pinnacles, barrier reefs and patch reefs. This complicates the analysis of open ocean wave transformation 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 show 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, applied by Tetra (2000). 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. Numerical computation were done for only design (5.34 m) and average wave heights (3 m) chosen for the harbour under EHWST conditions, as these represent extreme and common wave heights respectivley.

If one considers the architecture of the reef environment at the harbour, 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 flat and smooth for a reef environment, with a constant seaward grade of less than 1°. 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 (Thorne et al.

1995). For a 5.34 m, 10 sec, 1-in-50-year wave height, a 12 m wide reef crest, with 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 (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 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 used at the harbour site, 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/ m^3 or 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, because it was dredged to –2.5 m below CDL (1 m deeper than the level of the 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, 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 (3.4 m). This will run-up on the spending beach rip-rap, and enter the mooring area, causing choppy conditions to develop. 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 regular maintenance of the spending beach rip-rap, so as to ensure timely repair.

Numerical analysis performed for an average 3 m, 6 sec wave (1-year return interval), show the transformed wave on EHWST, over the reef will be 2.4 m or a 20 % decay. For such a transformed wave, the required dolomite rip-rap for the groin, under EHWST, should be at least 0.65 m 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 will be about 0.07 m^3/sec, smaller, but nevertheless, noticeable.

5. Conclusions and Recommendations

The coast is part of an emergent, Holocene reef-carbonate system, with a carbonate sand and gravel beach developed on phosphate-rich, cavernous, dolomite bedrock. The coastline is partly rocky, with karst pinnacles. The reef is a coral dominated, narrow and well-flushed, with many closely-spaced reef channels. The coastline at Anibare Bay is dynamic, moderately sloping, and shows sign of recent and current erosion. The harbour appears to have increased this erosion. The harbour development acts as a large groin, interrupting Southerly longshore current, causing 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.

The concrete breakwaters are overtopped by 3 m waves. This raises some concern as to what acceptable risks were allowed for this facility. Numerical analysis for a 5.34 m, 10 sec, wave show a 30 % wave decay over the reef crest. This is consistent with run-up of the coastal road, observed by residents under EHWST. For such a waves, the size and density of boulders required to maintain a stable structure on the facility need to be 3-5 times the diameter specified by Tetra (2000). In addition, the navigational channel will not cause as much wave decay, as over the adjacent intact reef crest. If a 5.34 m wave breaks 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. It is therefore, important and necessary to cater for routine and regular maintenance.

Management of 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. To prevent any further erosion or exacerbation of shoreline retreat, the following programmes will be needed at the harbour site.

Eroding sections of the coastline need to be protected immediately, if erosion is to be arrested or reduced. 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. Protection strategies should be for the immediate and medium-long term.

A bio-engineering system should be employed. To prevent overtopping of coastal areas, a rip-rap system could be utilized along the edge of the coastline. Biological protection by planting locally coastal tree and shrub species would be best, to select flora which are adaptable to the existing micro-climate. The use of geotextile or bio-erosion mats, like coconut fibre mats, for scour control, is recommended and should be appropriate for the hydraulic and tropical UV conditions. The local dolomite is suitable for coastal protection, because of high density. Rip-rap should be sized based on actual wave hydraulics. All design should be performed by a competent professional. A coastal monitoring programme should be put in place to periodically assess any changes to the coastline at the harbour site. Monitoring will facilitate rapid repair to damaged infrastructure and eroding coasts. Monitoring should be beach profiling, beach sediment sampling and coastal littoral hydraulic assessment. Visual inspection of failure characteristics and wave run-up should be documented and photographed when they occur. This monitoring should be at least quarterly in the first 3 years, which can be decreased to twice yearly thereafter. The coastal monitoring data should be reviewed as soon after collection, and coastal management strategies modified/revised to reflect any changes or new developments Beach profiles and surveys should be levelled from Nauru’s national suryeying benchmarks. The data collected should be archived in a database for easy access and retrieval. Future coastal developments should be preceded by an environmental impact assessment (EIA), to minimize similar negative impacts.

Acknowledgements

This project was funded by the Commonwealth Secretariat. The Departments of Industry and Economic Development, Fisheries; Tourism; the Ministry of Industry and Economic Development; and the Harbour Master's Office, RON and Tetra Ltd provided logistic support.

References

ASTM. (2000). *Annual Book of ASTM Standards. Soil, Rock, Building Stones and Geotextiles,* ASTM, 978p.

Barrett, P. J. (1988). Report on Phosphate, other Minerals and Groundwater Resources and on Aspects of Rehabilitation Planning and Methodology, Nauru, Pacific Ocean, *Commission of Inquiry into the Rehabilitation of the Worked-Out Phosphate Lands of Nauru,* **9**, D946-D999.

CIRIA. (1991). *Manual of the Use of Rock in Coastal and Shoreline Engineering, CIRIA Special Publication 83 & CUR Report 154,* A. A. Balkema, 607p.

GSL. (1999). *Stone: Building Stone, Rock, Fill and Armourstone in Construction,* Geological Society of London, 480p.

Hudson, R. N. (1958). *Design of quarry stone cover layers for rubble mound breakwaters,* U. S. Corps of Engineers, Waterways Experiments Station Research Report, 2-2p.

IHE-Delft. (1999). *CRESS: Coastal and River Engineering Support System, Version 7.00*, IHE.

Jacobson, G., and Hill, P. J. (1993). *Groundwater and the rehabilitation of Nauru. Collected Case Studies in Engineering Geology, Hydrogeology and Environmental Geology,* Geological Society of Australia/ Butterfly Books, 119p.

Jacobson, G., Hill, P. J., and Ghassemi, F. (1997). *Geology and Hydrogeology of Nauru Island, Geology and Hydrogeology of Carbonate Islands,* Elsevier, 707-742.

Magellan Geographix (2000). www.maps.com.

Minster, J. B., and Jordan, T. H. (1978). Present-date plate motions, *Jour. Geophy. Res*. **83,** 5331-5354.

Pilsarczyk, K. W. (1996). *Offshore Breakwaters and Shore Evolution Control,* A. A. Balkema, 560p.

Tetra. (1999). *The Project for Improvement of the Community Boat Harbour at Anibare,* Tetra Co. Ltd.

Tetra. (2000). *Basic Design Study Report on the Project for Improvement of Community Boast Harbour in the Republic of Nauru,* JICA and Tetra Co. Ltd.

Thorne, C. R., Abt, S. R., Barends, F. B. J., Maynord, S. T., and Pilarczyk, K. Y. 1995. *River, Coastal and Shoreline Protection. Erosion Control Using Riprap and Armourstone,* John Wiley, 784p.

Van de Meer, J. W. (1987). Stability of breakwater armour layers. *Coastal Eng.* **11,** 219-239.

Van de Meer, J. W. (1998). A code for dike height design and examination. *Proc. ICE Conf. Coastlines, Structures and Breakwaters,* Thomas Telford.

Van Loon, H. (1984). *Climates of the Oceans, Volume 15*, Elsevier, 716p.