AGAT BAY REGIONAL SHORELINE ASSESSMENT

Planning Assistance to States Program
Draft Assessment Report

March 2020
US ARMY CORPS OF ENGINEERS
HONOLULU DISTRICT

EXECUTIVE SUMMARY

The Honolulu District of the U.S. Army Corps of Engineers (USACE), at the request of Government of Guam Bureau of Statistics and Plans (Gov. Guam), has conducted a regional study of the Agat shoreline located on the Island of Guam to identify areas of significant shoreline erosion, determine the causes of the erosion, develop conceptual plans for shoreline stabilization, and investigate various modifications to Agat Small Boat Harbor to address issues experienced by harbor users. This study is being accomplished in accordance with the Planning Assistance to States agreement executed between the Commonwealth of the Northern Mariana Islands and the U.S. Army Corps of Engineers, Honolulu District and Section 22 of the Water Resourced Development Act of 1974 (Public Law 93-251, as amended).

The study area is along the western shoreline of Guam, within Agat Bay just south of Apra Harbor. It extends from Apaca Point (just north of the Namo River) south to Facpi Point, a distance of approximately five miles. Areas of particular interest within the region are the Inn on the Bay, the Agat Mayor's Office, Apra Small Boat Harbor (SBH), and Nimitz Beach Park. Erosion has been observed in these locations, with various types of infrastructure being impacted.

Historical shoreline change was evaluated for the study area to quantify regional shoreline changes and to assess how the Agat SBH may have impacted these natural processes. Shoreline photos prior to and following harbor construction, including the most recent images available, have been examined to identify any changes that may be related to harbor construction. The reaches in the middle of the region between Namo River/Inn on the Bay south through Agat SBH are relatively stable over the long-term, with a few erosion "hot spots", but overall a slight gain of sediment. The southernmost area between Nimitz Beach and Facpi Point appears slightly erosive over the historical period of analysis.

Overall, the variability in erosion and accretion along the shoreline shows that sediment movement within the region is complex, and not strongly dominant in one direction or the other alongshore, but rather influenced by small circulation cells controlled by bathymetry and coastal morphology. In addition, variability in shoreline change trends over both pre- and post-harbor construction periods does not strongly implicate the construction of the harbor in being responsible for a strong influence on these trends. Historical hydrosurveys indicate that the harbor is acting as a sediment sink, trapping sediment that would otherwise move throughout the littoral zone.

The effect of wave and current interaction is important for evaluating regional circulation and wave patterns in the Agat Bay region. The Coastal Modeling System (CMS) models (CMS-Wave and CMS-Flow) are were used to evaluate this interaction because of their capability for inline steering (coupling) of results from one model to the other. Several discrete three-day time series of this historic information were selected in order to represent typical wave and current conditions occurring over several tide cycles. Use of these representative oceanographic conditions provides a general understanding of current conditions that would occur frequently, and therefore, dominate circulation patterns and sediment transport patterns. The Particle Tracking Model (PTM) was utilized as a qualitative tool; intended to identify the potential sediment pathways for a particular wave and current condition, and therefore provide insight to the dominant directions of sediment transport over the long-term.

Based on the combined results of both the historical shoreline change analysis as well as the wave, circulation, and PTM modeling, it is evident that the dominant direction of sediment transport along the shoreline north of Agat SBH is from north to south, both prior to and after construction of the harbor. At Nimitz Beach Park, the dominant direction of transport is now from south to north, differing from pre-harbor conditions. Therefore, construction of the harbor has likely altered dominant sediment transport direction in this location.
Erosion at Inn on the Bay and the Agat Mayor's Office is due in part to a trend of offshore transport during typical and extreme wave events, caused by wave-generated currents. This may also have been exacerbated in recent years by higher than normal water levels in the western Pacific. The analysis shows that overall, there is a deficit of sediment in the region, and that exploration of beneficial use of dredged sediment, as well as offshore and upland sand deposits is warranted in order to replace some of the sand that has been lost.

Concepts for modifications to the existing harbor were developed in the following to address two issues being experienced: 1) Strong currents through the harbor that affect navigation and berthing of vessels, and have caused damages to harbor infrastructure (i.e. – boat slips), and 2) shoaling caused by sediment transported into the harbor, which reduces authorized depths in the channels and results in increased maintenance requirements. The results of each proposed modification on waves and currents was evaluated with wave and circulation modeling to determine, at a level of detail commensurate with the concept designs, whether adverse effects would result. Concepts are based on modifications of the existing layout of the harbor, and are focused on modification of navigation structures.

Two of the concepts evaluated are recommended for further evaluation: 1) enclosing the harbor by connecting the existing revetted mole to the existing main breakwater, and 2) constructing a north breakwater just to the north of the harbor, but leaving a gap between the new structure and the main breakwater. Both concepts were shown to reduce current velocities in the harbor, and would also reduce shoaling rates. The rough order of magnitude costs of these concepts are between $7 and 10 million.

Conceptual plans for shoreline stabilization were developed for Inn on the Bay, Agat Mayor's Office, and Nimitz Beach Park, all of which were experiencing adverse impacts due to shoreline erosion. At Inn on the Bay, replacing the current seawall with a new seawall that includes a deeper toe foundation secured into hard material for stability and a crest that is raised above the current elevation is recommended to provide better long-term protection from continued erosion. At the Agat Mayor's Office, a similar, strengthened seawall concept is recommended to protect the current infrastructure, and the addition of a sand placement in front of the repaired seawall is recommended to provide further protection as well as to restore a portion of the eroded beach. At Nimitz Beach Park, sidewalk infrastructure should be relocated farther inland away from the eroding coastline, and a living shoreline should be implemented as a natural shoreline stabilization option. Rough order of magnitude construction costs for these shoreline stabilization concepts are approximately: $345,000 (Inn on the Bay), $465,000 (Agat Mayor's Office), and $260,000 (Nimitz Beach Park).

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1. Introduction and Study Authority

The United States (U.S.) territory of Guam, located in the western Pacific Ocean, is the largest and southernmost of the Mariana Islands. Its location approximately 4,120 miles to the west of Hawaii makes it the westernmost limit of the U.S., and qualifies it as a culturally, strategically, and economically important territory. The importance of protecting Guam's coastal infrastructure while preserving its existing shoreline resources such as beaches and reefs, particularly in the face of its exposure to an extreme tropical storm climate and rising sea levels, requires close attention to natural and human-induced shoreline changes, coastal processes, and shoreline management methods available. This study is being accomplished in accordance with the Planning Assistance to States (PAS) Program agreement executed between the Commonwealth of the Northern Mariana Islands and the U.S. Army Corps of Engineers (USACE) Honolulu District under the authority of Section 22 of the Water Resourced Development Act of 1974 (Public Law 93-251, as amended).

2. Purpose

The Honolulu District, at the request of Government of Guam Bureau of Statistics and Plans (Gov. Guam), has conducted a regional study of the Agat Bay shoreline to identify areas of significant shoreline erosion, determine the causes of the erosion, develop conceptual plans for shoreline stabilization, and investigate various modifications to the federally authorized Agat Small Boat Harbor to address navigation issues experienced by harbor users. Results of this completed PAS study will be used by Gov. Guam for identification of necessary harbor modifications in the Port Authority of Guam's Harbor Master Plan as well as for planning and prioritizing shoreline mitigation projects. Imperial units (feet, miles, etc.) will be used to the greatest extent possible in this report, though hindcast information, modeling and geospatial data are typically generated in metric units, and will be converted to imperial units where possible.

3. Study Area Overview

The island of Guam is the result of submarine volcanic ridges, and is comprised of both volcanic rock and coralline limestone. Much of the island is surrounded by fringing coral reefs, resulting in the majority of beaches being composed of calcium carbonate sands. The shoreline is characterized by embayments surrounded by headlands and fronted by shallow, flat limestone reefs varying in width from a few hundred to several thousand feet.

3.1. Study Area Description

The study area is along the western shoreline of Guam, within Agat Bay just south of Apra Harbor. It extends from Apaca Point (just north of the Namo River) south to Facpi Point, a distance of approximately five miles (Figure 1). Reef widths along the shoreline vary from approximately 100 to 2,600 feet, with natural and manmade channels transecting the reef at

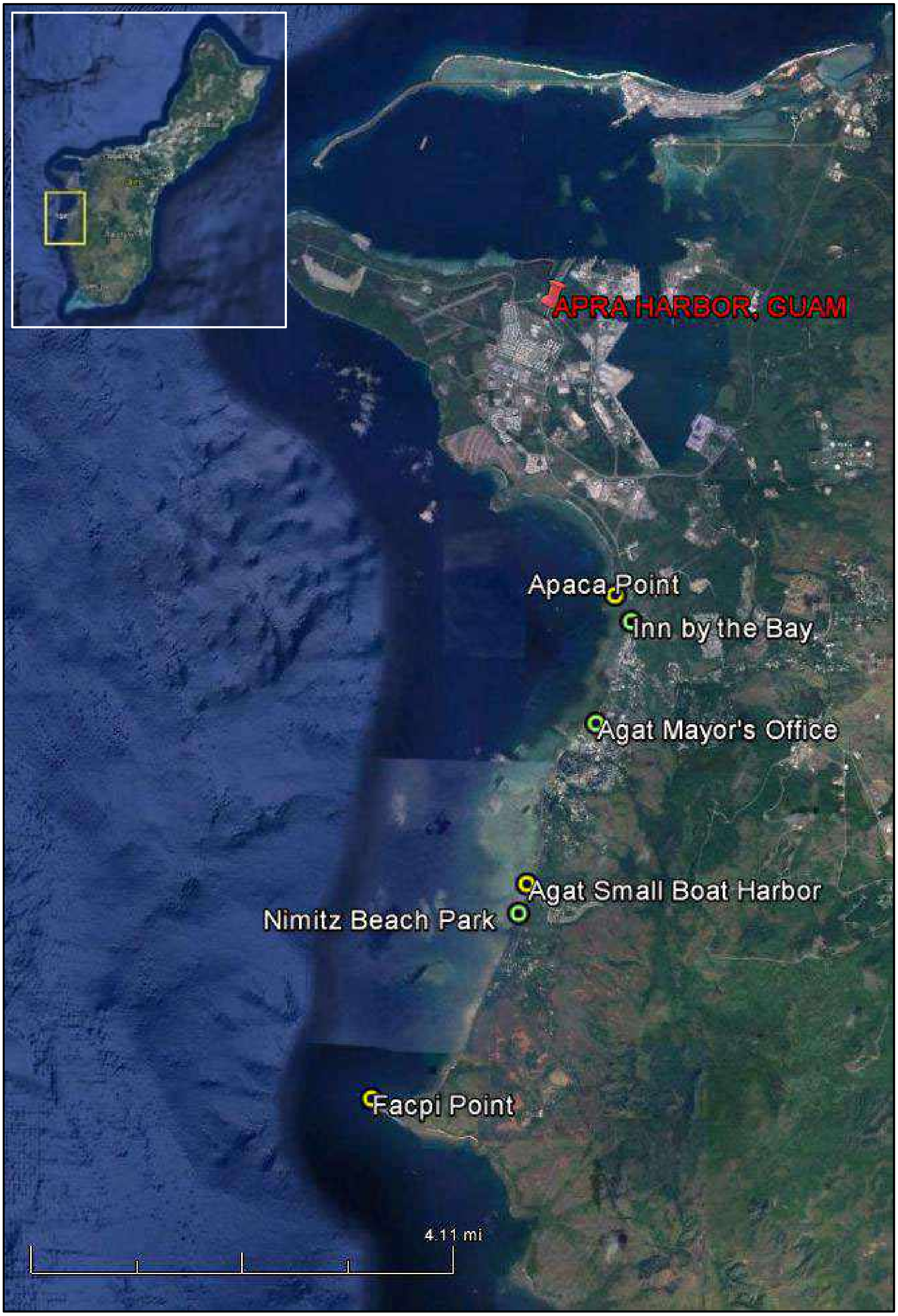

Figure 1. Map of study area extents

various locations. Beaches are generally narrow, consist of calcareous and volcanic sediment, and are backed by a low coastal plain with developed areas around the village of Agat. Route 2 highway runs parallel to and is within 500 feet of the coastline for the majority of the study area. Areas of particular interest within the region are the Inn on the Bay, the Agat Mayor's Office, Agat Small Boat Harbor (SBH), and Nimitz Beach Park, which are shown in Figure 1. A detailed description of these areas and their present condition follows in later sections of this report.

3.2. Study Area Climate

3.2.1. Winds

The predominant winds in the Guam are the trade winds, which occur approximately 70 percent of the time, and arrive from north-northeast (22.5 degrees True North [TN]) through east-southeast (112.5 degrees TN). The trade winds are most consistent between January and June, averaging between 15 to 25 miles per hour (mph) [13 to 21 knots or 6.5 to 11 meters/second]. Hindcast wind and wave information offshore of the study area is available from the USACE Wave Information Study (WIS) Program (http://wis.usace.army.mil). WIS information is generated using the numerical hindcast model WISWAVE, driven by wind fields overlaying a bathymetric grid. Model output includes significant wave height, peak and mean wave period, peak and mean wave direction, wind speed, and wind direction. In the Pacific, the WIS hindcast database covers a 32-year period of record extending from 1980 to 2011. WIS Station 81099 is located off the west coast of Guam (Figure 2). The Station 81099 wind rose provides frequency, wind speed, and direction of average annual winds (Figure 3).

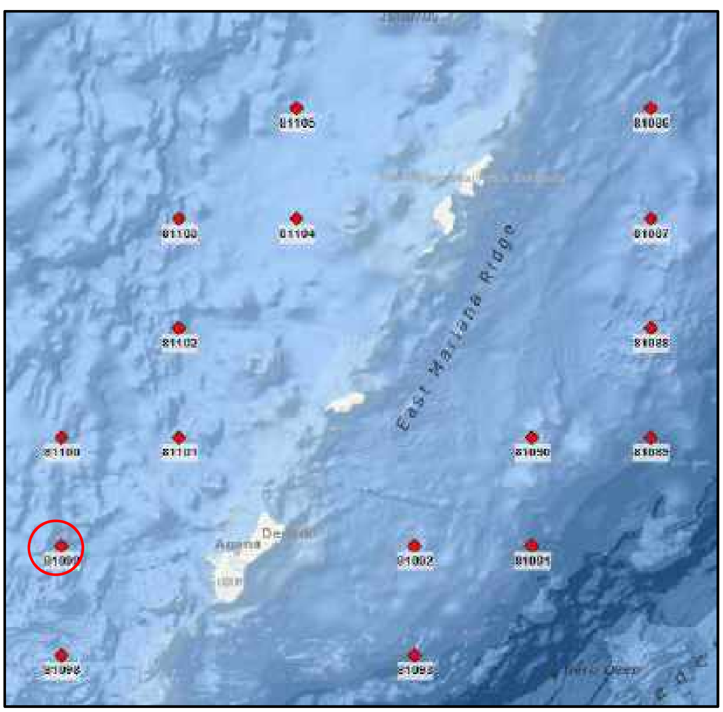

Figure 2. Guam and CNMI WIS hindcast stations (Station 81099 west of Guam circled in red).

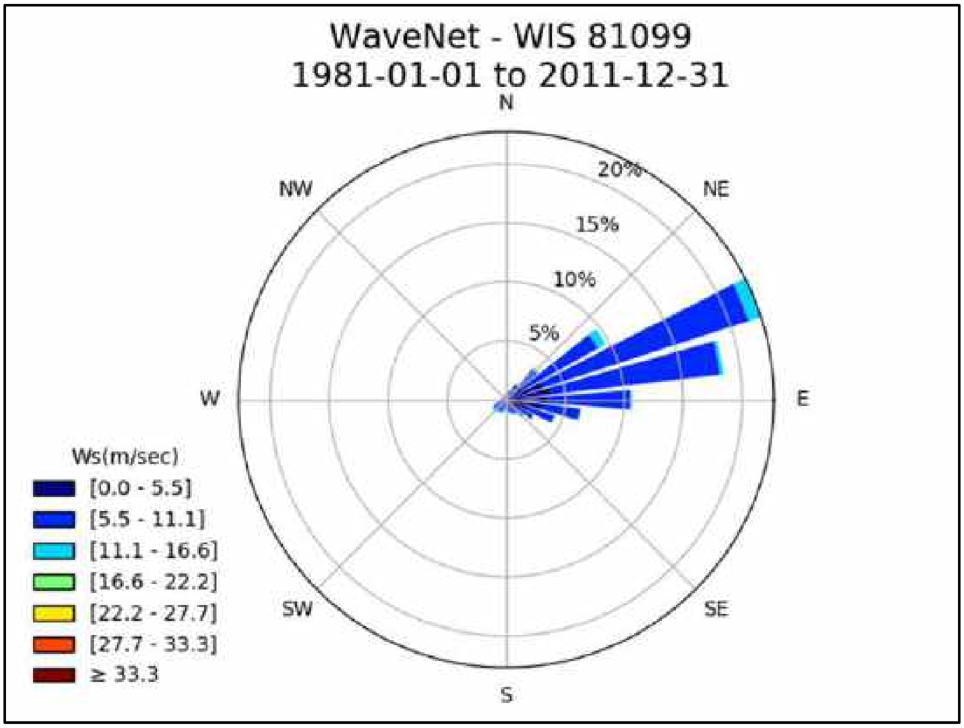

Figure 3. Wind Rose from WIS hindcast station 81099 near Agat Bay region of Guam.

During July – December, winds often become light and variable. This is also considered the wetter season, with about 70 percent of annual rainfall occurring during this period. The latter half of the year is also the active tropical cyclone season. Typhoons and tropical storms can develop quickly and bring high, damaging winds (120 mph or more) and intense rainfall. An average of three tropical storms and one typhoon pass within 180 miles of Guam each year (Pacific Islands Climate Education Partnership, 2014). Winds are also affected in the region by the climate pattern known as the El Niño Southern Oscillation (ENSO), and large-scale changes in sea level pressure across Indonesia and the tropical Pacific linked to El Niño (https://www.climate.gov). During an El Niño year, trade winds are usually weaker, there is less rainfall on Guam, and the ocean surface is warmer with above-average sea surface temperatures.

3.2.2. Tides

Tides in Guam are semi-diurnal and the tide range is 1.6 feet [ft] (0.5 meters [m]), with two high and two low tides each day. The closest tide station to the study area, maintained by the National Oceanographic and Atmospheric Administration (NOAA), is Apra Harbor, Guam (Station 1630000). The mean tide range at this station is 1.62 feet, and the spring tide range is 2.35 feet. The station has been recording water levels since 1948 (over 70 years), and shows the maximum water level occurring on August 28, 1992 at an elevation of 2.92 feet Mean Sea Level (MSL), during Typhoon Omar which made landfall on Guam. Tidal datums relative to MSL at Apra Harbor for the tidal epoch spanning 1983-2001 are shown in Table 1.

Table 1. Tidal Datums at NOAA Station 1630000: Apra Harbor, Guam

Tidal Datum	Feet (above MSL)
Mean Higher High Water (MHHW)	0.97
Mean High Water (MHW)	0.85
Mean Tide Level (MTL)	0.04
Mean Sea Level (MSL)	0.00
Guam Vertical Datum of 2004 (GUVD04)	0.00
Mean Low Water (MLW)	-0.77
Mean Lower Low Water (MLLW)	-1.37

3.2.3. Waves

The island of Guam is exposed to three distinct wave types: waves generated by the prevailing local winds, swell waves generated by distant storms, and waves from tropical cyclones passing near Guam. Trade wind waves are typically from northeast through east-southeast, with wave heights in the range of 1 to 6 feet (0.3 to 2m) and wave periods between 5 to 10 seconds. Swell waves from distant storms (usually in the north Pacific) can range from 6 to 18 feet (2 to 6 m) in height and have wave periods from 10 to 16 seconds. Tropical storm and typhoon waves can approach from almost any direction (though the storms typically track east to west or southeast to northwest), resulting in waves to 40+ feet (13+ m) in deep water and wave periods in the 8 to 14 second range.

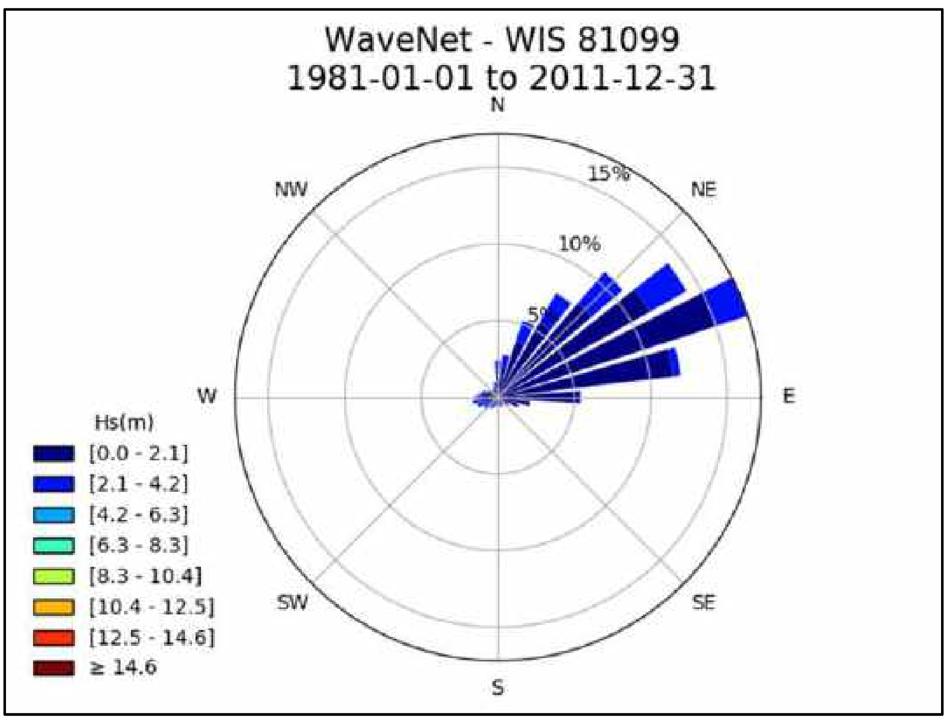

Figure 4. Wave Rose from WIS hindcast station 81099 near Agat Bay region of Guam.

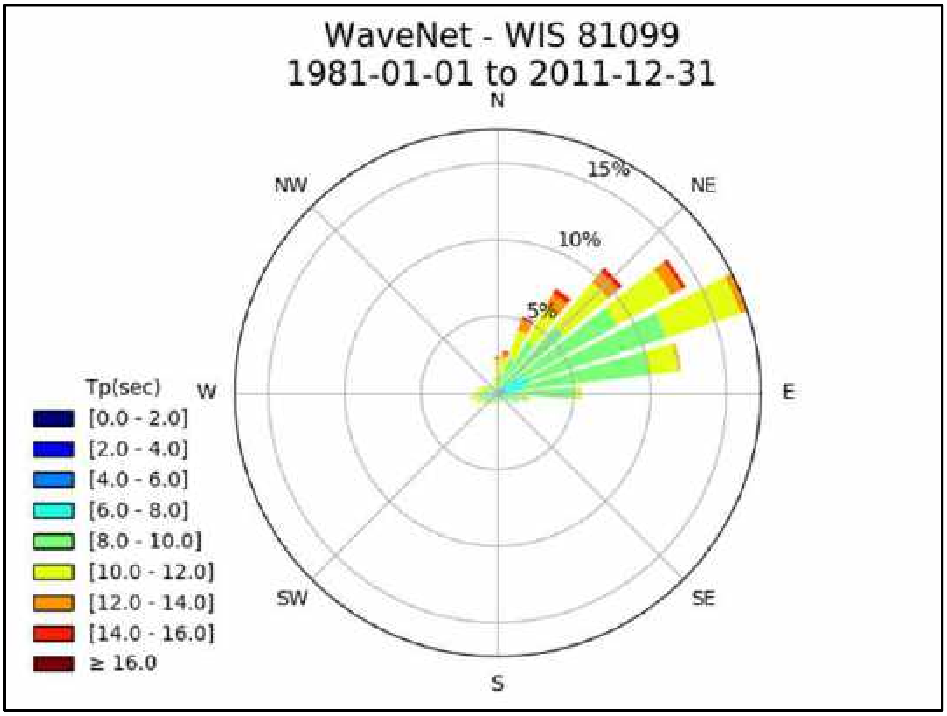

Figure 5. Wave Period Rose from WIS hindcast station 81099 near Agat Bay region of Guam.

As shown in the wave rose of unfiltered WIS wave parameter information in Figure 4, trade wind waves less than 4m in height from the northeast sector prevail at this deep water station. Corresponding peak wave periods (shown in Figure 5) are between 6 and 16 seconds.

The study area is well sheltered from much of the trade winds and associated waves due to its location along the west coast of Guam. The headlands that bound the region also provide some sheltering, such that the window of wave exposure is approximately northwest (335 degrees from north) counter-clockwise through south-southwest (215 degrees from north), as shown in Figure 6. Filtering of the hindcast information to include only approaching waves within this window reveals a significantly different wave climate for the study area.

Frequency analysis of wave parameters from the hourly hindcast information after filtering shows that a large percent (82.0%) of waves affecting the study area are between 1.0 and 2.5 meters in significant wave height (Table 2). Wave direction frequency is spread across the wave window, with the highest concentration of waves (16%) coming from the direction bin centered at 260 degrees (west-southwest) and the majority of total waves coming from the southwest quadrant. Peak spectral wave period analysis (Table 3) shows that almost half of deep water waves have between 8 and 10 second wave periods, and over 92% are between 6 and 12 seconds. These parametric values will govern analysis of operational conditions at Agat SBH and determination of typical sediment transport pathways.

Figure 6. Wave exposure window for Agat shoreline

Table 2. Frequency Table of Significant Wave Height vs. Wave Direction

Significant Wave Ht (m)	Wave Direction (10 degree bins)										
	235-244	245-254	255-264	265-274	275-284	285-294	295-304	305-314	315-324	325-335	Total
<=1m	0.15%	0.10%	0.22%	0.28%	0.30%	0.21%	0.27%	0.26%	0.24%	0.25%	2.29%
1-1.49m	2.51%	3.01%	4.12%	3.94%	4.59%	3.73%	2.76%	2.84%	2.52%	2.49%	32.50%
1.5-1.99m	3.14%	4.24%	5.34%	5.56%	4.21%	4.16%	2.74%	1.82%	1.97%	1.55%	34.73%
2-2.49m	2.04%	2.43%	2.42%	2.51%	1.41%	1.12%	1.17%	0.74%	0.54%	0.44%	14.81%
2.5-2.99m	1.44%	1.61%	1.48%	0.91%	0.54%	0.24%	0.24%	0.22%	0.17%	0.23%	7.07%
3-3.49m	0.50%	1.06%	1.31%	0.61%	0.21%	0.10%	0.06%	0.07%	0.05%	0.11%	4.09%
3.5-3.99m	0.44%	0.38%	0.48%	0.24%	0.04%	0.01%	0.05%	0.02%	0.01%	0.06%	1.73%
4-4.49m	0.52%	0.19%	0.38%	0.25%	0.02%	0.03%	0.04%	0.03%	0.07%	0.02%	1.55%
4.5-4.99m	0.25%	0.15%	0.11%	0.03%	0.00%	0.02%	0.02%	0.06%	0.01%	0.01%	0.66%
5-5.49m	0.11%	0.02%	0.16%	0.02%	0.01%	0.01%	0.00%	0.00%	0.00%	0.00%	0.33%
5.5-5.99m	0.05%	0.05%	0.01%	0.02%	0.02%	0.00%	0.00%	0.00%	0.00%	0.00%	0.14%
6-6.49m	0.00%	0.00%	0.00%	0.03%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.03%
6.5-6.99m	0.00%	0.00%	0.02%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.03%
7-7.49m	0.00%	0.00%	0.03%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.03%
11-11.5m	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.01%
Total	11.14%	13.23%	16.09%	14.40%	11.36%	9.62%	7.34%	6.06%	5.59%	5.17%	100.00%

Table 3. Frequency Table of Peak Wave Period vs. Wave Direction

Peak Wave Period (s)	Wave Direction (10 degree bins)										
	235-244	245-254	255-264	265-274	275-284	285-294	295-304	305-314	315-324	325-335	Total
4-5.9s	0.38%	0.18%	0.27%	0.19%	0.14%	0.12%	0.14%	0.04%	0.05%	0.02%	1.52%
6-7.9s	3.43%	4.06%	3.70%	2.49%	1.83%	1.53%	0.81%	0.37%	0.46%	0.31%	18.98%
8-9.9s	5.19%	6.08%	8.06%	7.02%	6.00%	4.52%	3.16%	2.51%	2.24%	2.19%	46.99%
10-11.9s	1.94%	2.50%	3.31%	4.38%	2.88%	2.44%	2.28%	2.34%	2.05%	2.09%	26.22%
12-13.9s	0.19%	0.38%	0.73%	0.31%	0.35%	0.73%	0.84%	0.75%	0.74%	0.51%	5.53%
14-16s	0.01%	0.02%	0.02%	0.00%	0.15%	0.28%	0.11%	0.05%	0.06%	0.04%	0.76%
Total	11.14%	13.23%	16.09%	14.40%	11.36%	9.62%	7.34%	6.06%	5.59%	5.17%	100.00%

The extreme wave climate is driven by typhoon and tropical storms, and is important when considering design of coastal structures in this region. The WIS hindcast provides the most appropriate available database to analyze extreme waves, due to its long period of record (32 years from 1980 to 2011). The largest wave in the hindcast information record occurred in November 1991 during Typhoon Yuri, with a deep water wave height of 60.0 feet (18.3m) and wave period of 16.8 seconds, approaching from 101 degrees (east-southeast). The next largest wave on record during this period is significantly less at 13.0m (42.6 feet), approaching from 89 degrees (east). As noted, the study area coastline is sheltered from waves approaching from the east (which is the most typical approach angle of tropical storm generated waves). For this reason, the filtered wave parameter record was used to conduct return period analysis. The results of this analysis are shown in Table 4, with the wave heights and corresponding return periods and probability of annual exceedance shown. The theoretical return period is the inverse of the probability that the event will be exceeded in any one year, so as an example the 50-year return period wave would have a 2% chance of being exceeded in any given year.

Table 4. Return Period/Annual Exceedance Probability for filtered WIS wave parameters

Return Period (years)	Annual Probability of Exceedance	Wave Height (ft)	Wave Height (m)
2	50%	14.14	4.31
5	20%	18.24	5.56
10	10%	21.71	6.62
25	4%	26.73	8.15
50	2%	30.77	9.38
100	1%	35.03	10.68

3.3. Currents

Currents in the area are highly dependent on reef flat bathymetry and wave-induced mass transport of water over the shallow reef flat, which itself is dependent on water level (including tide, storm surge, and other components). Winds can also influence surface currents. Longshore currents are generated in areas of slightly deeper water along the shoreline, and flow is onshore/offshore in areas of natural or man-made channels, depending on a flooding or ebbing tide.

3.4. Sediment Inputs

Sediments in the Agat Bay region include both calcareous and terrestrial components, supplied by erosion of offshore reefs and upland volcanic rock, respectively. Terrestrial sediments are supplied by several rivers that discharge into the area, the Namo River, Togcha River, Finile Creek, Gaan River and Auau Creek.

4. Existing Conditions and Problems Experienced

4.1. Study Area Shoreline Description

The coastline north of the study area, on the Apra peninsula, is primarily cliffs with a few pocket beaches. The majority of the study area is characterized by a narrow, sandy shoreline backed by heavy vegetation and infrastructure. A coastal highway (Route 2) runs through this region, and in some areas, homes or other structures were built in the narrow stretch between the road and the shoreline. A few places have hardened the shoreline to protect threatened infrastructure, including the Inn on the Bay and the Agat Mayor's Office.

The photos in Figure 7 represent typical conditions at Apaca Point, which is just north of Namo River. Small pocket beaches have formed between rocky headlands. Palm trees with an exposed root structure indicate the impact of erosion in this area.

Figure 7. Conditions at Apaca Point. The photo on the left is facing south, and on the right is facing north.

Just to the south of Apaca Point is a hotel named Inn on the Bay. A cemented rock wall fronts the property to protect the facilities from the retreating shoreline (Figure 8, right). To the south of the Inn (Figure 8, left), the sandy shoreline extends for about 2 miles to the Agat mayor's office (Figure 9). The photo on the right in Figure 9 shows the typical narrow beach, facing north from the mayor's office. In the photo on the left, the mayor's office building is in the foreground, protected by a cemented rock seawall. The shoreline protection runs along the entire property, but there is still a small sandy beach fronting the seawall structure.

The photos in Figure 10 show the shoreline condition just north of Agat Small Boat Harbor. Grassy vegetation comes down to the water's edge in some areas. Several utility poles along this region have become undermined, as seen in the photo on the right.

Figure 8. Inn on the Bay, with a hardened shoreline, is shown in the photo on the right. Typical shoreline conditions just south of the Inn are shown in the photo on the left.

Figure 9. At the Agat mayor's office, the photo on the right shows typical conditions to the north, and the photo on the left shows the property at the mayor's office.

Figure 10. The shoreline just north of Agat Small Boat Harbor, with undermined utility poles seen on the right.

Agat SBH was constructed by USACE in 1989. The federal project includes an entrance channel maintained to depth of 14 feet below MLLW, a turning basin maintained to 11 feet below MLLW, a main access channel maintained to 9 feet below MLLW, an offshore breakwater, a stub breakwater, and two revetted moles (Figure 11). The revetted moles protect the shoreside facilities, while the offshore breakwater protects the access channel and harbor basin. The harbor has not been maintenance dredged during the period since initial construction.

Historic hydrographic surveys between 1991 and 2017 indicate an average annual shoaling rate of approximately 188 cubic yards [yd^3] (144 cubic meters [m^3]) per year. The most recent survey conducted in 2017 shows 6,400 yd^3 (4898 m^3) of material above project depth (Figure 12).

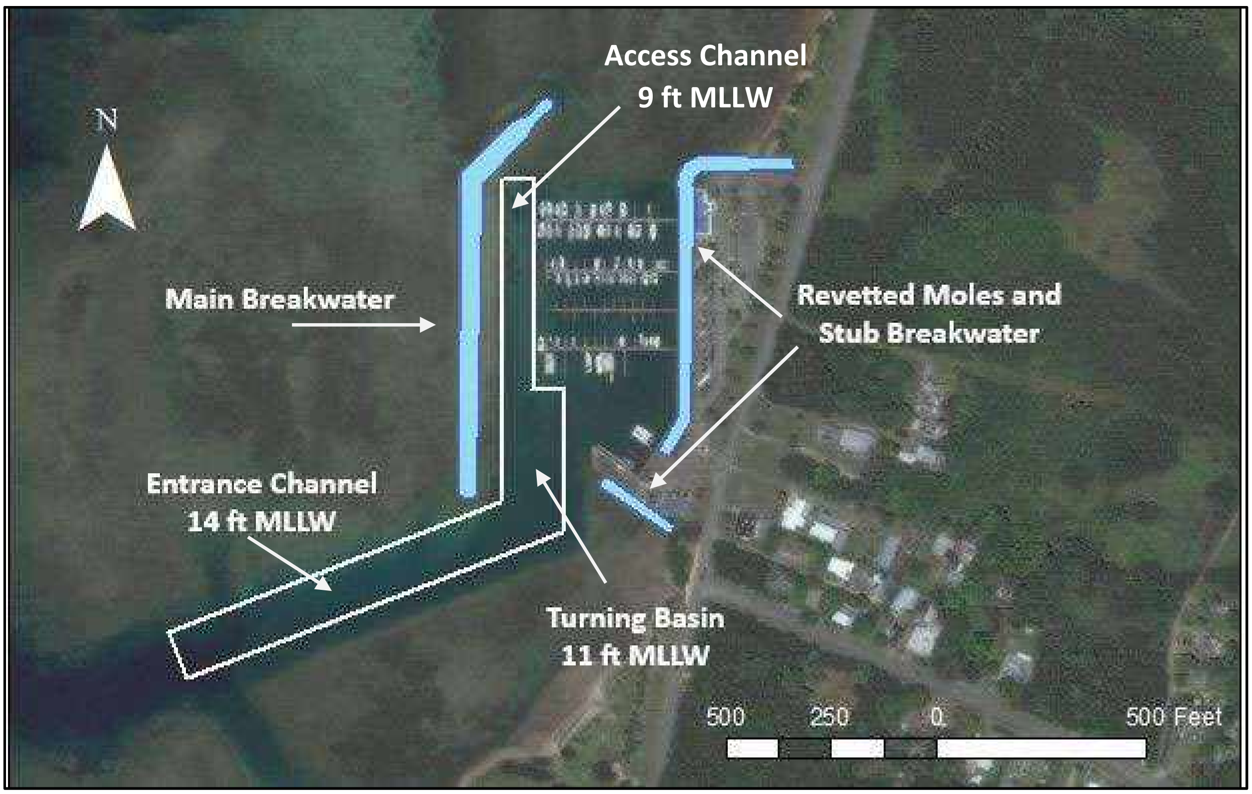

Figure 11. Agat Small Boat Harbor Map

Examination of hydrographic survey plots from 2001, 2012 and 2017 (Figure 12) show that the first areas to shoal following construction were the northern side of the access channel, and the northern side of the entrance channel. This shoaling has continued and has been followed by shoaling of the northern side of the turning basin. This shoaling pattern indicates that sediment infilling the federal channel is coming from the north, both through the gap between the main breakwater and revetted mole, as well as around the outside of the main breakwater. This sediment transport direction is consistent with circulation modeling results shown later in this report. Figure 13 shows ground photos of the harbor.

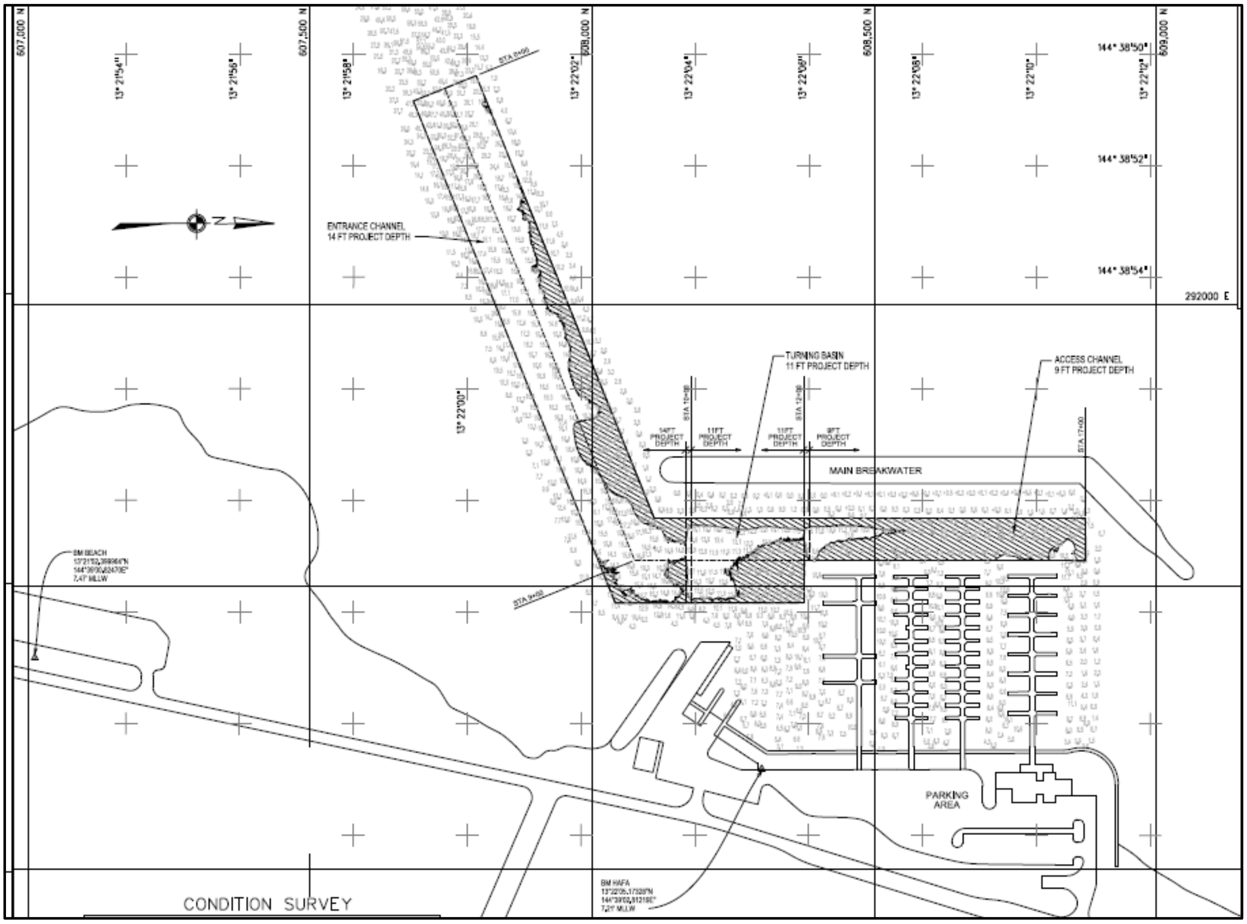

Figure 12. 2017 channel hydrographic survey indicating shoaling in crosshatched areas of the federal channel.

Figure 13. Conditions at Agat Small Boat Harbor.

To the south of Agat Small Boat Harbor is a public park called Nimitz Beach (Figure 14). Broken slabs of cement are seen strewn along the beach in some sections. In other sections, park infrastructure, such as a walking path, may soon be impacted by the retreating shoreline.

Figure 14. Nimitz Beach Park, just south of Agat Small Boat Harbor.

4.2. Harbor Conditions based on User Observations

Presently, the harbor experiences significant currents under certain wave and tide conditions, due to the longshore currents that exist along the shoreline. These currents pass through the harbor berthing areas and interior channels between the detached breakwater and the shoreline. Currents also flow through the entrance channel, with the direction of flow dependent on the tidal phase and wave conditions. These strong currents have an effect on navigation through channels and stability of berthing area infrastructure. It is also thought that this circulation pattern contributes to sediment being transported into the harbor and settling out in deeper areas, resulting in the harbor shoaling shown in Figure 12.

Waves are not known to cause significant problems to navigation or harbor operations at the present time. The majority of waves (except for those during relatively infrequent extreme events) break on the outer edge of the reef flat, and are dissipated greatly before reaching the harbor area. Long period or infragravity waves (waves with period greater than 30 seconds), are known to exist in this region, and can travel across the reef flat as bore-like waves. These waves can cause harbor oscillation (seiching) and high currents. No direct observation of these phenomena have been reported in the study area; however, this will be investigated as part of the study process through numerical wave modeling.

5. Historical Shoreline Change Analysis

5.1. Analysis Period and Methods

Historical shoreline change was evaluated for the study area to quantify regional shoreline changes and to assess if Agat SBH has impacted these natural processes. Shoreline change analysis evaluates the advance or retreat of a shoreline over time. Shorelines are typically classified as being stable (not moving), receding (moving landward), or advancing (moving seaward). Understanding historical shoreline change allows coastal engineers and coastal managers to make more informed decisions regarding shorelines, including whether engineering measures are needed in areas of chronic erosion.

Shoreline change for the study area was evaluated using the Digital Shoreline Analysis System (DSAS) in ArcGIS. Aerial imagery from various sources was downloaded for the following years: 1944, 1993, 2006, and 2018. This ensured that shorelines prior to and following harbor construction, including the most recent images available, were examined to quantify potential impacts of federally authorized general navigation features on adjacent shorelines. The imagery was georectified and used to digitize shoreline positions in ArcGIS, by visual estimation of the beach toe. Using DSAS, an onshore baseline and 1,000 meter-long transects were laid out at 20 meter (65.6 feet) intervals along the entire region. DSAS then calculated the distance between each shoreline pair and divided by the time difference in years to determine the shoreline change rate at each transect in feet per year. Because inter-annual historical aerial imagery was unavailable, each set of imagery was assumed as representative of the entire year, regardless of the time of year it was obtained. Volume change was estimated by applying a conversion value of 3.3 cubic meters (m^3) of sand per square meter of beach (0.4 cubic yards per square foot) based on guidance provided by the Coastal Engineering Manual (USACE 2002).

5.2. Historical Shoreline Change Rates and Volume Change

An overall assessment of the region was done by comparing changes from 1944 to 2018, a span of 75 years. In Figure 15, negative values shown in red indicate regions where the shoreline has retreated and positive values shown in green indicate regions of advance. Shoreline change rates between -0.5 and 0.5 m/year (m/yr) were determined to be within the estimated error inherent in georeferencing and digitization, and therefore theses shorelines are categorized as stable. The majority of the region falls into this category. One area that has seen extensive shoreline retreat is the area just north of Apaca Point. Other areas that have experienced moderate shoreline retreat include the Inn on the Bay and the Agat mayor’s office. Additionally, at Nimitz Beach Park was an area of notable shoreline advancement. These areas will be further evaluated below.

5.2.1. Shoreline Change at Apaca Point

Figure 16 shows the region around Apaca Point and the shoreline positions for all years evaluated. The shoreline change calculation in this area was split over two segments as indicated. Erosion in this area, with a rate up to -1.5 m/yr has led to the largest shoreline changes in the region. However, the rate of erosion appears to have decreased in recent years, based on the changes observed from 2006 to 2018. It can be clearly seen in Figure 16 how the shape of the shoreline has drastically changed since 1944. It should be noted that the Namo River Flood Control Project was built by USACE in 1979 under the authority of Section 205 of the 1948 Flood Control Act, as amended. The project includes an inline weir, designed to trap sediment in the upper portion of the project and prevent sediment from reaching the coast. This is a potential factor in the observed sediment deficit in this area.

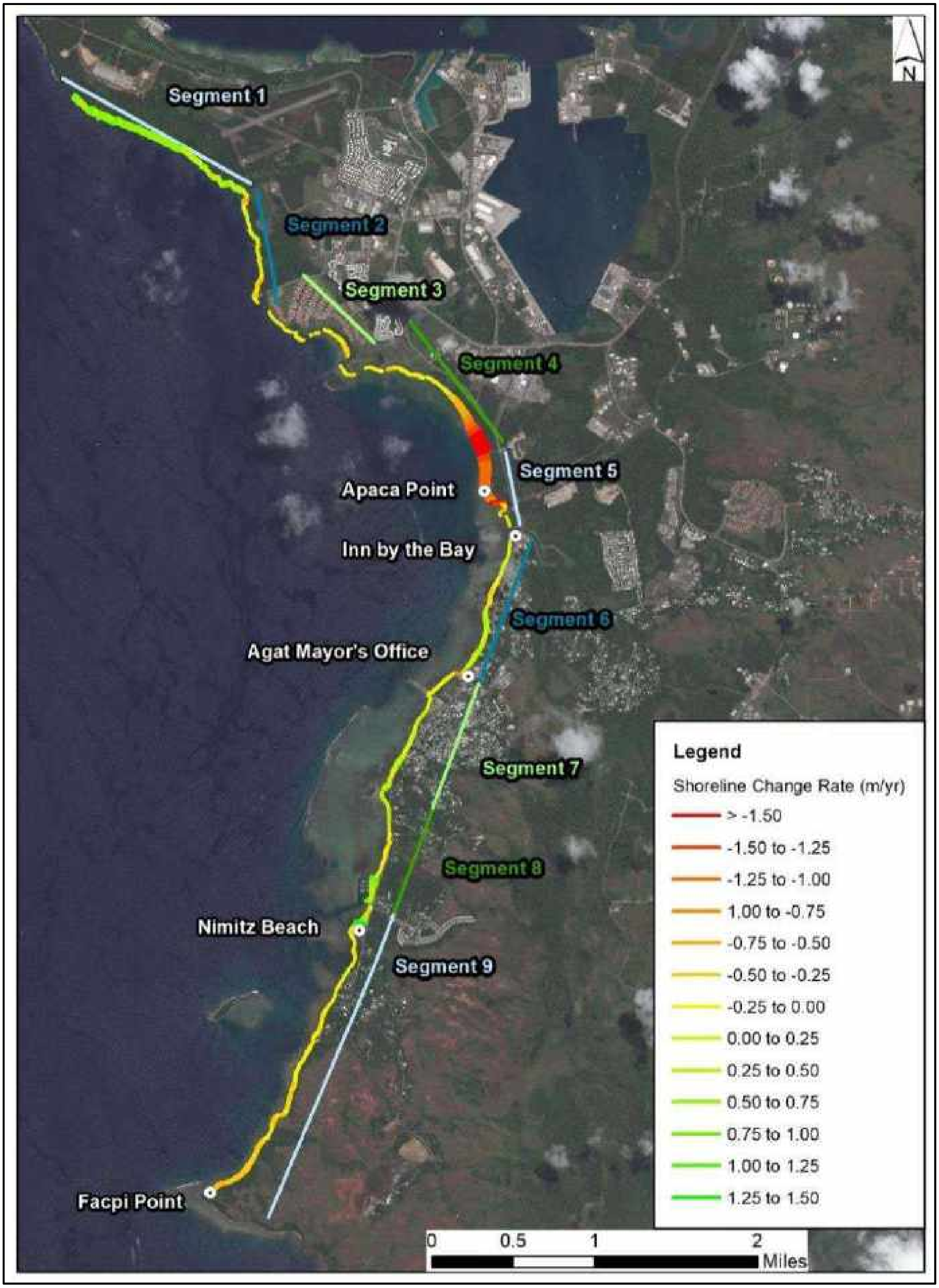

Figure 15. Shoreline change rate (m/yr) calculated between 1944 and 2018.

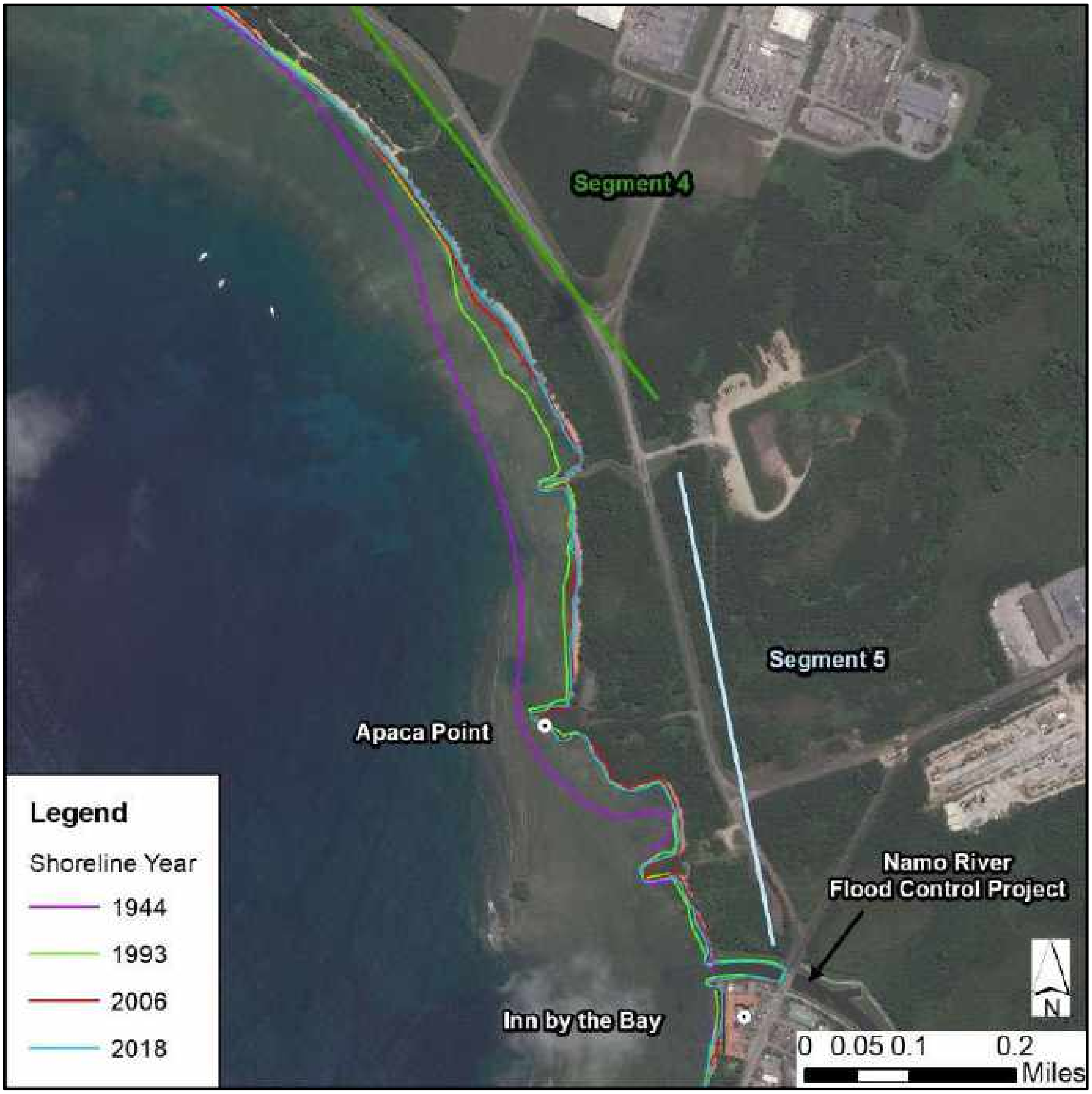

Figure 16. Shoreline changes in the area around Apaca Point.

Figure 17 shows the shoreline change rates at individual transects along this reach, and corresponds to the larger shoreline change rate overview shown in Figure 15. This figure details the specific areas where erosion rates of -1.0 to -1.5 m/year exist north of Apaca Point and areas of slightly less erosion to the south of the point. This shoreline does not appear to have any infrastructrure immediately upland, but considering the significant historical erosion rates observed, should be monitored for potential future impacts.

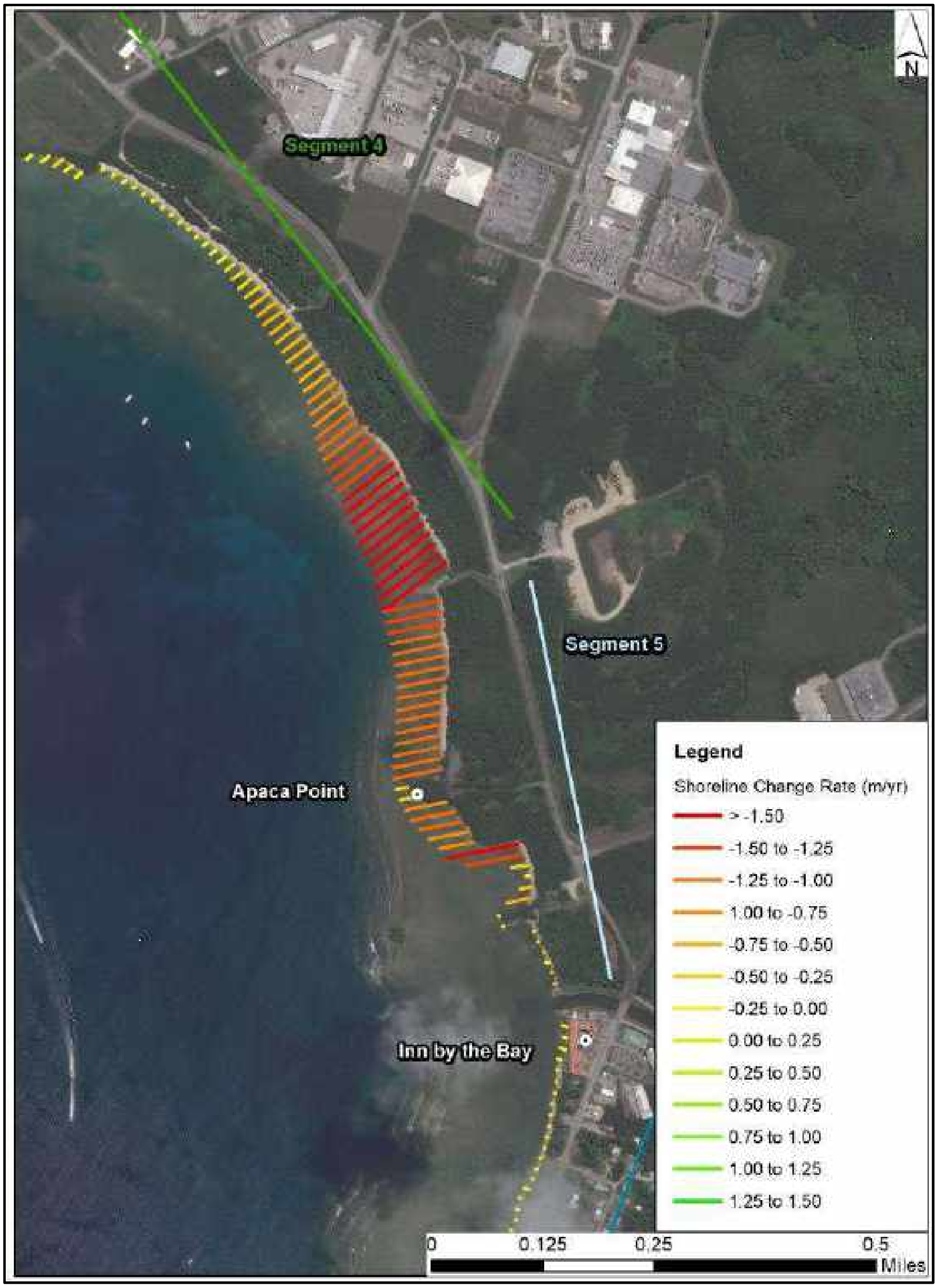

Figure 17. Shoreline change rates at Apaca Point (1944-2018)

5.2.2. Shoreline Change at Inn on the Bay and Agat Mayor's Office

Figure 18 shows the historical shoreline changes in the area immediately in front of Inn on the Bay. The shoreline just to the south of the Namo River entrance has noticeably retreated throughout the period of record. Though the calculated change rates are relatively small (0 to -0.25 m/yr) these can be significant impacts when buildings or other infrastructure are located close to the shoreline. The shoreline further to the south appears to have advanced significantly between 2006 and 2018, which may be an indication of recent southerly directed sediment transport in this area. This will be investigated further by circulation modeling later in this report.

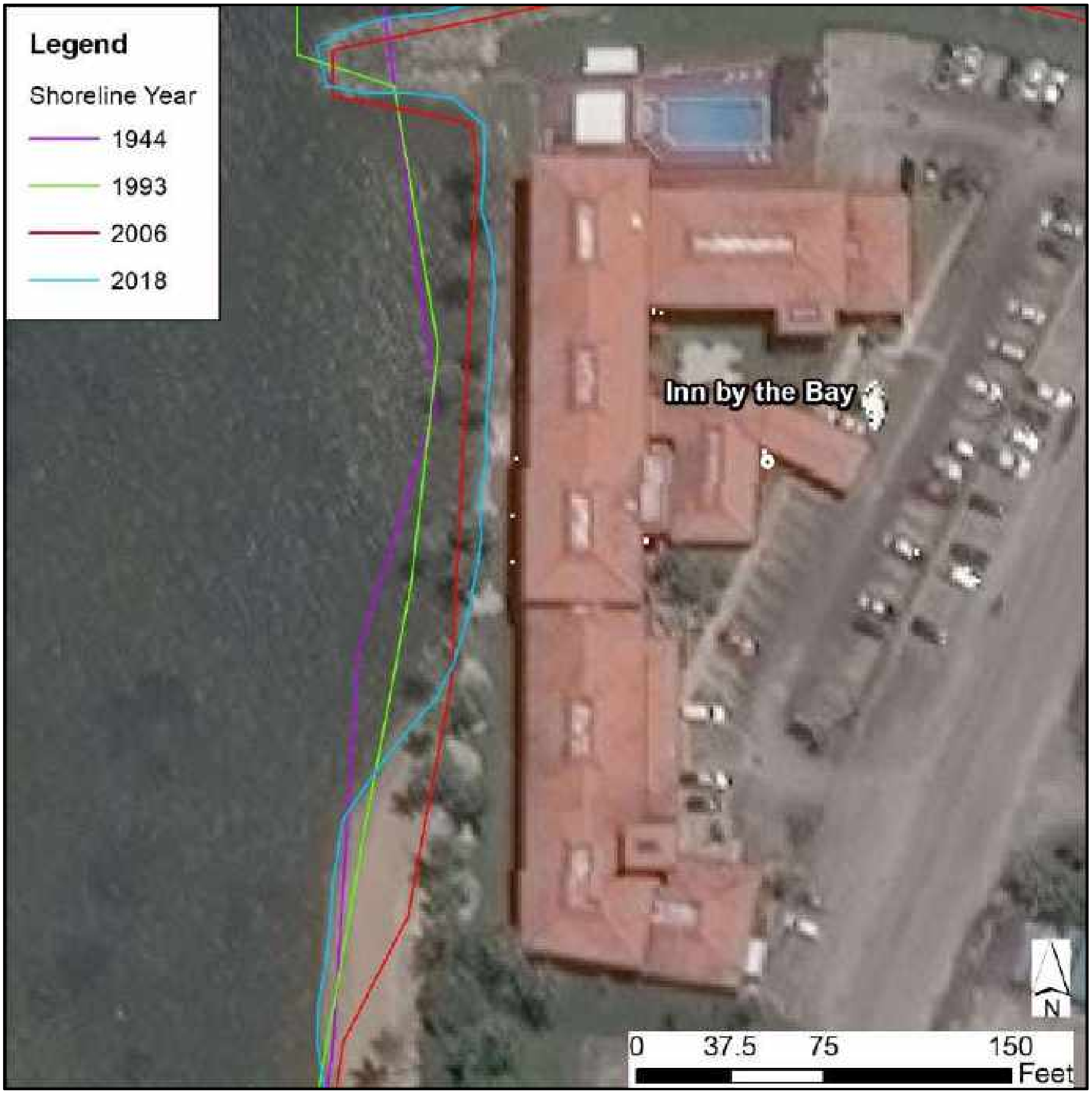

Figure 18. Shoreline changes in the area in front of Inn on the Bay.

Shoreline changes in the area in front of the Agat mayor's office are shown in Figure 19. In the area immediately fronting the mayor's office property, the shoreline has noticeably retreated between 1993 and 2006. This area appears relatively stable between 2006 and 2018. However, immediately to the north the trend switches and the shoreline appears to be accreting from 1944 to 2018, and particularly in recent years. This trend may be indicative of a northerly directed transport in this segment of the shoreline. This will also be investigated further by circulation modeling presented later in this report.

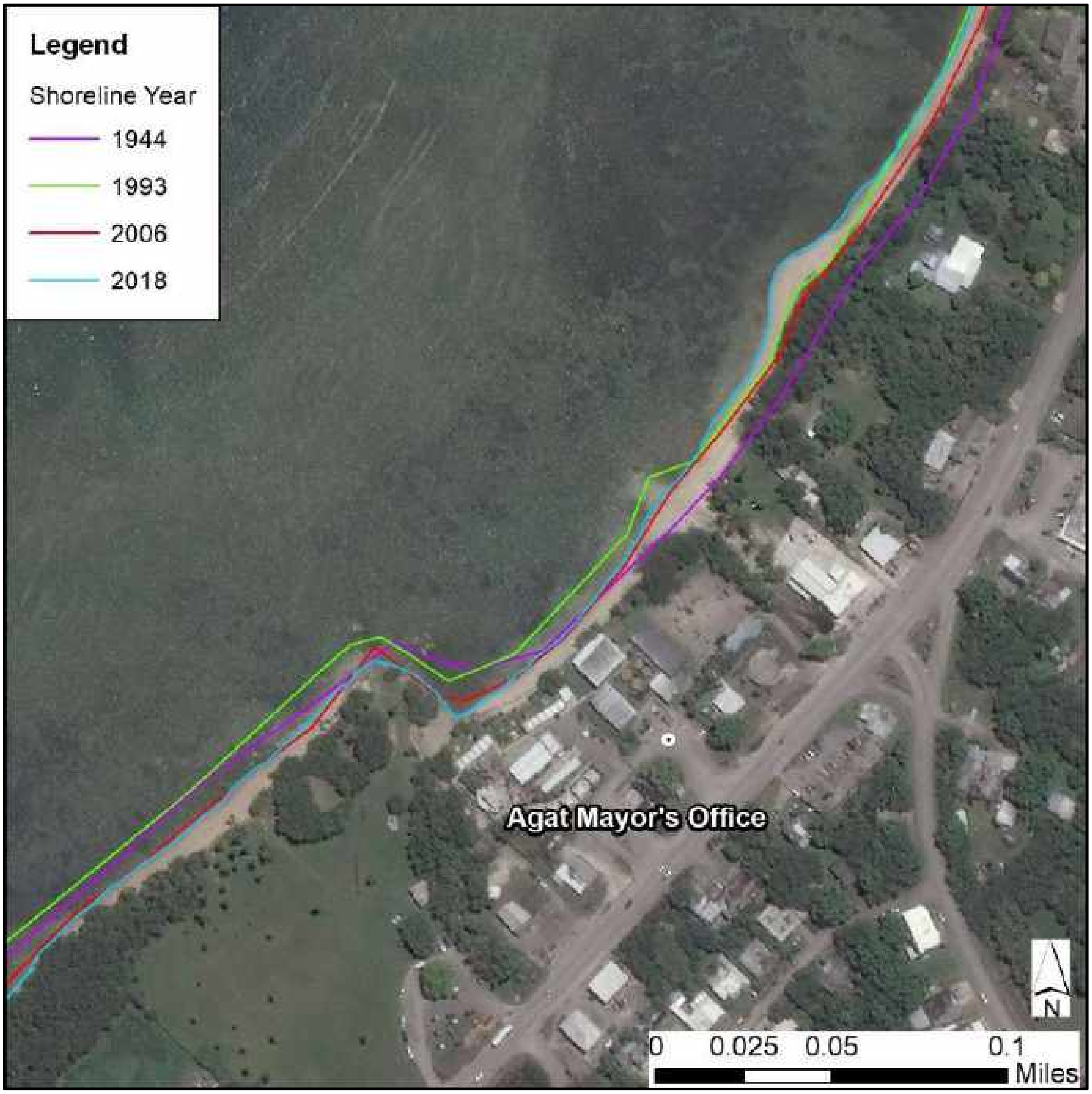

Figure 19. Shoreline changes in the area in front of the Agat mayor's office.

Figure 20 shows the shoreline change rates at individual transects along this reach, and corresponds to the larger shoreline change rate overview shown in Figure 14. This figure details the specific areas where an erosion rate of -0.75 to -1.0 m/year exists along one transect fronting the Agat Mayor's Office, accretion rates in the range of +0.25 to+ 0.75 m/year exist along the shoreline north of the mayor's office, and the trend changes back to slightly erosive (0.0 to -0.5 m/year) in the northernmost half of this reach through the shoreline fronting the Inn on the Bay.

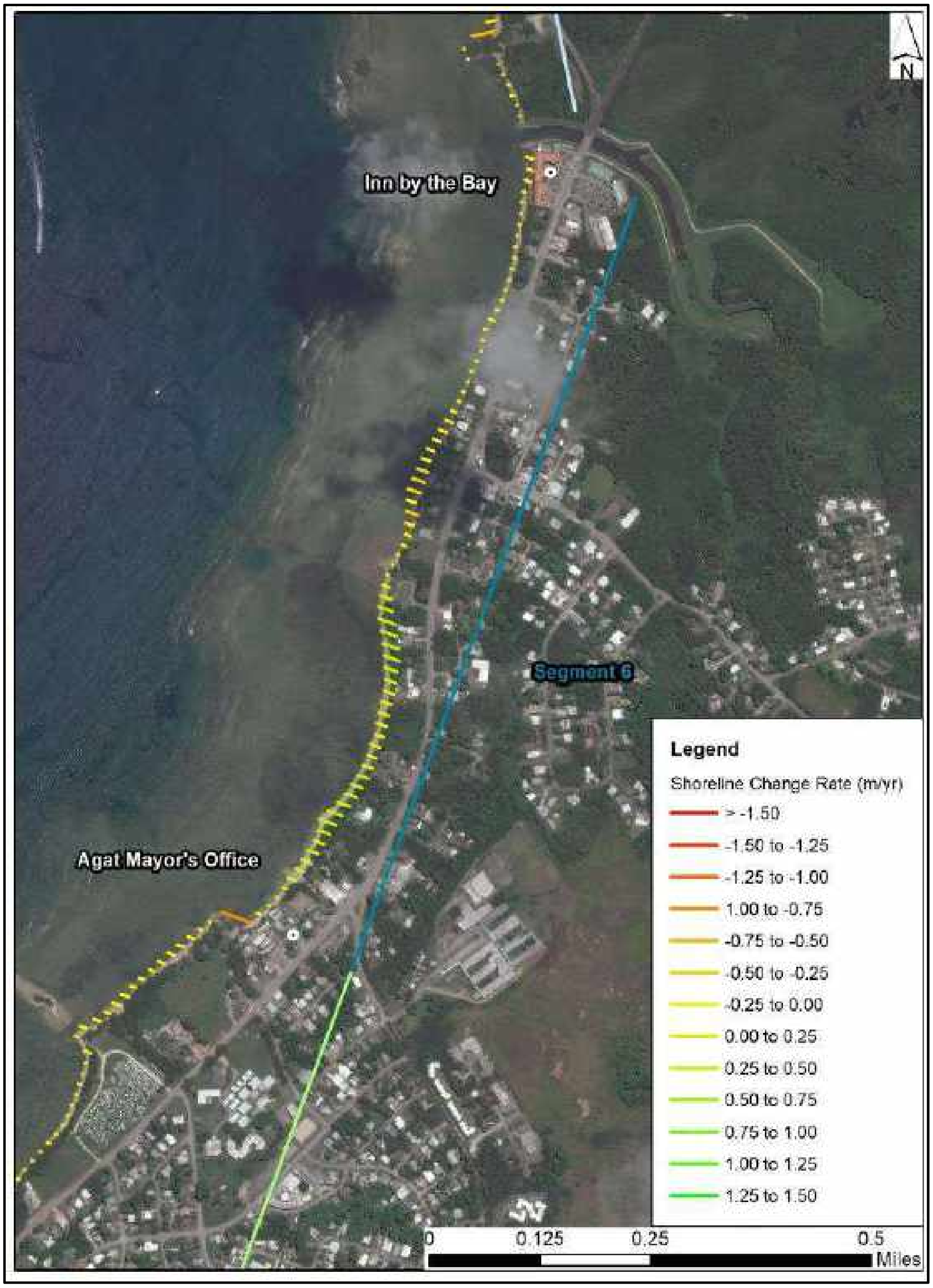

Figure 20. Shoreline Change Rates at Inn on the Bay to Agat Mayor's Office

5.2.3. Shoreline Change at Agat SBH and Nimitz Beach Park

The final area of concern is the area immediately surrounding Agat Small Boat Harbor, including Nimitz Beach Park. The historical shoreline changes in this region are shown in Figure 20. The construction of the harbor is clearly seen between the 1944 shoreline and the post-1989 shorelines. The area just to the north of the harbor where utility poles are being undermined (refer back to Figure 10) appears slightly erosive, primarily in recent years, from 2006 to 2018. This may correspond to sediment moving south toward the harbor, as evidence by the harbor shoaling previously shown. The shape of Nimitz Beach Park has also significantly changed, primarily accreting but also generally shifting to the north. The movement of material along the coast to the north could account for the observed erosion in some areas at the southern end of the park (refer back to Figure 14). Historical imagery does not show a clear trend of observable shoreline impacts in either direction that can be attributed to construction of Agat SBH. Littoral transport does not appear to have changed dramatically based on the pre-construction aerial photo (other than possible shoaling of the harbor as indicated by hydrographic surveys).

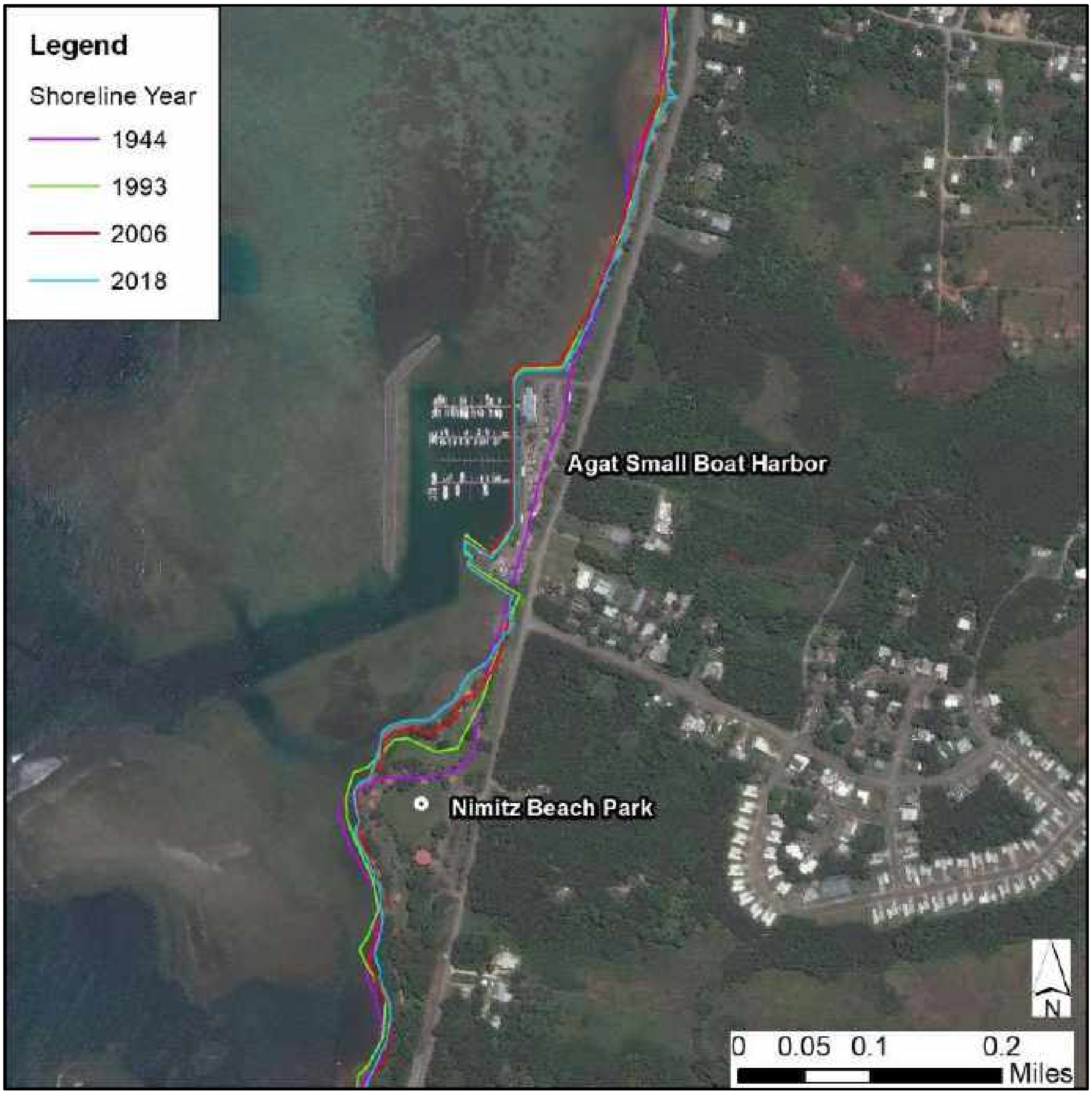

Figure 20. Shoreline changes in the area around Agat Small Boat Harbor and Nimitz Beach Park.

Figure 21 shows the shoreline change rates at individual transects along this reach, and corresponds to the larger shoreline change rate overview shown in Figure 15. This figure shows that erosion rates to the north of the harbor are within 0.0 to -0.5 m/yr, and that north Nimitz Beach Park is accreting at a rate of 0.0 to +1.0 m/yr, while the south side of the park is slightly erosive at rates of approximately 0.0 to -0.5 m/yr.

Figure 21. Shoreline Change Rates at Agat SBH and Nimitz Beach Park

5.2.4. Volume Change Rates in the Agat Bay region

As noted above, volume change rates for the various segments of the shoreline (loosely corresponding to estimated littoral cells and shown in Figure 14) were calculated using a typical beach profile shoreline for a reef-lined beach to convert linear erosion rates to rates of volume per year, per length unit along the shoreline. Table 5 relates the average volume change rates so that rates can be compared between segments despite differing baseline lengths.

Table 5. Average Volume Change Rates (erosion = red, accretion = green)

Seg. #	Segment	Average Volume Change Rate (m3/yr/m)			
		1944-2018	1944-1993	1993-2006	2006-2018
4	Apra Peninsula	-1.87	-2.26	-1.73	-0.44
5	Apaca Point	-2.54	-3.65	-1.49	0.83
6	Inn by the Bay to Agat Mayor's Office	0.22	0.49	-1.68	1.17
7	Agat Mayor's Office to North of Agat SBH	0.37	1.11	-1.94	-0.18
8	Agat SBH	0.44	0.86	0.34	-1.16
9	Nimitz Beach to Facpi Point	-0.49	-0.30	-0.66	-1.07

This view of the data outlines a few important trends about the region as a whole. In the long term (1944-2018), the areas at the northern end of the region, between the Apra peninsula and the Namo River, are experiencing the most loss of sediment. The shoreline segments in the middle of the region between Namo River/Inn on the Bay south through Agat SBH are relatively stable over the long-term, with a few erosion “hot spots”, but overall a slight gain of sediment. The southernmost shoreline segment between Nimitz Beach and Facpi Point appears slightly erosive over the historical period of analysis.

Shorter-term trends indicate that in the period between 1993 – 2006, most of the region was losing sediment, without an obvious location of accretion to balance the loss. This may be due to one or more of many causes such as increased tropical storm activity, upland construction reducing sediment supply, or the documented increased water levels due to intensification of Pacific trade winds since the early 1990s through about 2010. Investigation of these potential causes of potential accelerated erosion are outside the scope of this study. It does appear that in the more recent years from 2006 to 2018, some of these areas near Apaca Point and the Inn on the Bay to the Agat Mayor’s Office may be recovering somewhat. Overall, the variability in erosion and accretion along the shoreline shows that sediment movement within the region is complex, and not strongly dominant in one direction or the other alongshore, but rather influenced by small circulation cells controlled by bathymetry and coastal morphology. In addition, variability in shoreline change trends over both pre- and post-harbor construction periods does not imply that construction of the harbor influenced these trends.

6. Wave, Circulation and Sediment Transport Investigations

6.1. Regional Wave/Current Modeling

6.1.1. CMS Model Descriptions and Steering Process

The effect of wave and current interaction is important for evaluating regional circulation and wave patterns in the Agat Bay region. Wave conditions affect currents through wave setup, and currents may also affect the waves themselves, affecting wave steepness and wave breaking, particularly in shallow water. The Coastal Modeling System (CMS) models (CMS-Wave and CMS-Flow) are well-suited to evaluate this interaction because of their capability for inline steering (coupling) of results from one model to the other. This interaction means that for every time step (or iteration) in the simulation, the wave model will pass steady-state calculated wave parameters (height, period, direction) and wave radiation stresses and other parameters to the flow model for its calculations. The flow model in turn will pass back water level and current data to the wave model, enabling a direct solution for a seemingly difficult iterative process.

CMS-Wave, a two dimensional (2D), phase-averaged spectral wave model, can be applied to large domains, covering deep-water offshore areas up to the shoreline. CMS-Wave is a spectral wave transformation model and solves the steady-state wave-action balance equation on a non-uniform Cartesian grid. It considers wind wave generation and growth, diffraction, reflection, dissipation due to bottom friction, white capping and breaking, wave-wave and wave-current interactions, wave runup, wave setup, and wave transmission through structures.

CMS-Flow is a 2D shallow-water wave equations based flow model that can be used for hydrodynamic modeling (calculation of water level and depth-averaged current). Both the explicit and implicit versions of flow (circulation) model are available to provide estimates of water level and current given tides, winds, and river flows (where applicable) as boundary conditions. CMS-Flow solves the conservative form of the shallow-water equations that includes terms for the Coriolis force, wind stress, wave stress, bottom stress, vegetation flow drag, bottom friction, wave roller, and turbulent diffusion. Governing equations are solved using the finite volume method on a non-uniform Cartesian grid.

6.1.2. CMS Model Domains and Forcing

The details of the study area, including the shoreline, headlands, offshore islands, Agat Harbor navigation channel, turning basin, harbor structures, and adjacent coasts were included in the CMS-WAVE wave modeling grids. Bathymetry data was obtained from the UH-SOEST Pacific Islands Benthic Habitat Mapping Center (PIBHMC) (http://www.soest.hawaii.edu/pibhmc/cms/) and includes deep water multibeam survey data collected by NOAA Ship Hiialaka'i and R/V Ahi in 2010, as well as nearshore airborne Light Detection and Ranging (LiDAR) data collected by the Naval Oceanographic Office (NAVO) and the USACE Joint Airborne LiDAR Technical Center of Expertise as shown in Figure 22. Harbor bathymetry data was collected by USACE as part of annual project condition surveys in 2017.

A nested grid setup (multiple grids with varying resolution) was utilized for CMS-WAVE to transform deep water incident waves to the nearshore (Figure 23). The deep water regional wave grid (with its boundary located at WIS Station 81099) was used in half-plane mode and oriented in alignment with the approaching wave window, and is shown in red in Figure 23. A smaller, local wave grid focused on the study area is shown in yellow. CMS-Flow was applied using a domain identical in size, resolution and bathymetry to the local CMS-Wave grid, both for efficiency and compatibility between the two models during steering simulations. The model was forced using wave conditions (wave height, wave period, wave direction, wave dissipation, radiation stress gradient) from CMS-Wave to calculate water levels and current velocities within the study region.

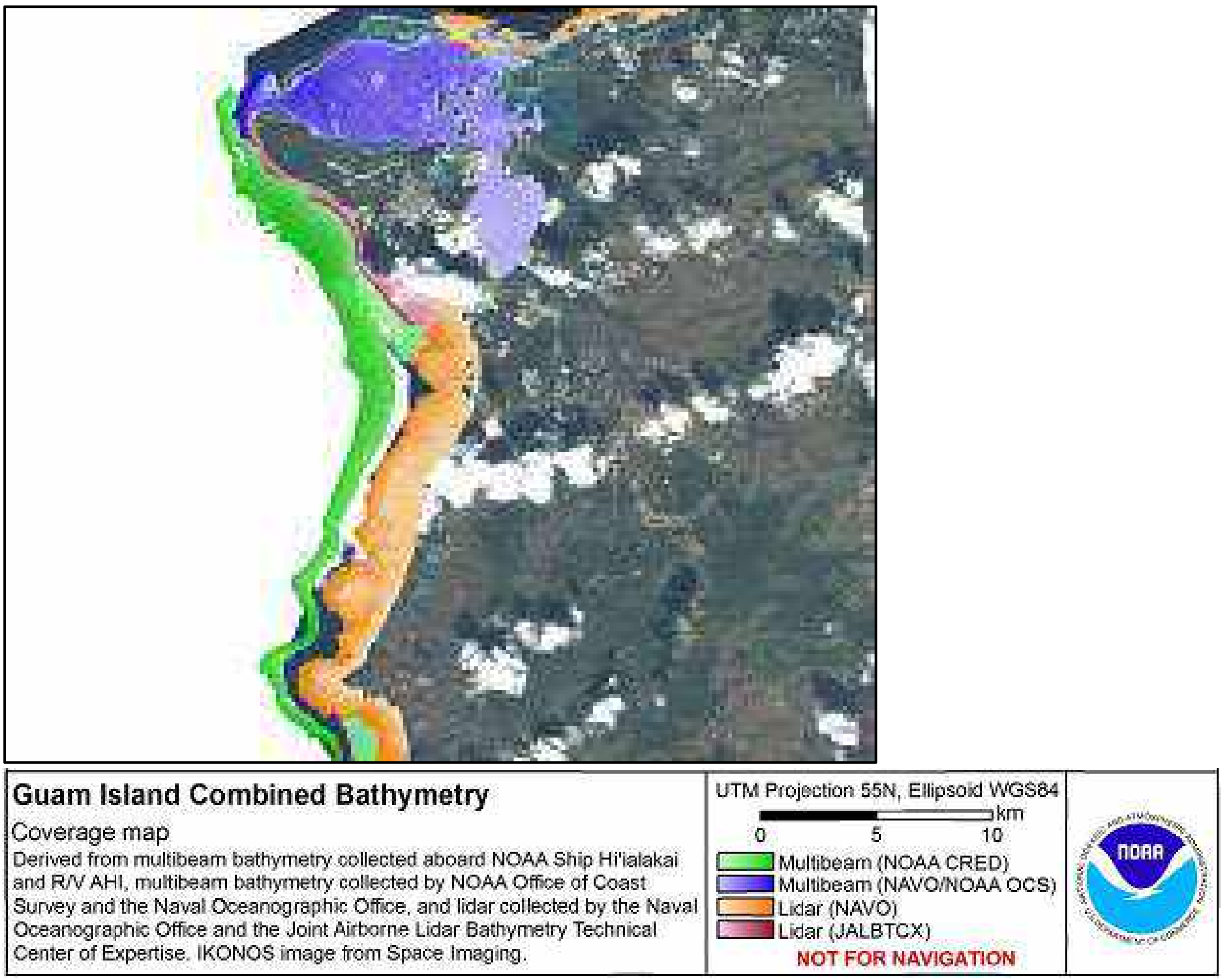

Figure 22. PIBHMC bathymetry data from multiple sources for the study area

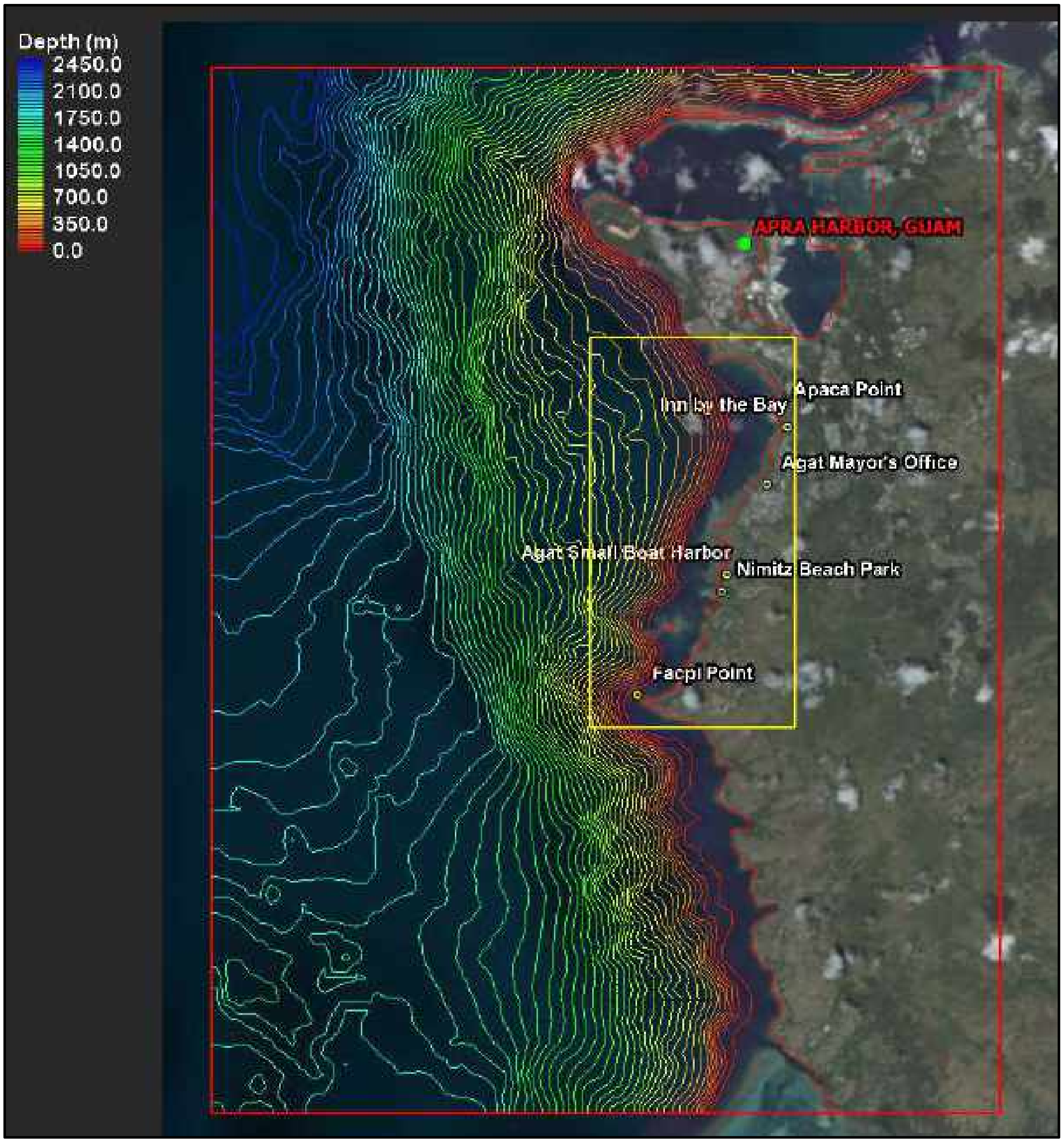

Figure 23. Nested CMS Wave/Flow grid boundaries and bathymetry for regional modeling

CMS-Wave was supplied offshore wind and wave boundary conditions from WIS time series information, and CMS-Flow was forced with measured water level time series data from the NOAA tide gage at Apra Harbor (refer back to Figure 1). A spatially varying bottom friction coefficient was also used in both models to incorporate the effects of the shallow and irregularly-shaped reef bathymetry on wave transformation and circulation. Several discrete three-day time series of this historic information were selected in order to represent typical wave and current conditions occurring over several tide cycles. Use of these representative oceanographic conditions provides a general understanding of current conditions that would occur frequently, and therefore, dominate circulation patterns and sediment transport patterns. The selected time series used for coupled wave and circulation modeling including relevant parameters, are shown in Table 6.

Table 6. Wave and Current Modeling Simulations

Description	Time Series Date	Typical Wave Parameters			Typical Wind Parameters	
		Hs (m)	Tp (s)	Dir (TN)	Wind Speed (m/s)	Wind Dir (TN)
Prevailing Wave Condition	8/26/2011-8/28/2011	1.4-1.6	9.0-9.7	255-264 (WSW)	3.9-5.8	140-225 (SE/SW)
Calm Wave Condition	8/24/2008-8/26/2008	0.7-0.9	6.8-9.8	239-282 (SW/NW)	2.1-4.2	162-214 (SE/SW)
Annual Wave Condition	7/29/2009-8/1/2009	3.0-3.3	10.2-11.5	235-255 (SW)	8.9-10.6	221-241 (SW)

6.1.3. Wave and Currents in the Agat Bay Region

Results of the CMS-Wave and CMS-Flow linked models provide an overview of wave transformation from deep water to the shoreline, as well as circulation patterns within the Agat Bay study region, both in the offshore and nearshore reef areas. Wave transformation offshore is controlled by deep bathymetry that allows waves to propagate toward the shoreline with little to no shoaling or refraction until they reach the shallow reef shelf along the shoreline. Wave shoaling and breaking occurs at the fore reef and reef crest, and much of the wave energy is dissipated during this process. Remaining energy may propagate as a reformed wave that travels across the reef flat toward the shoreline, with its wave height controlled by the water depth over the reef. This depth-limited breaking environment makes the components of dynamic still water level (DSWL), including tide, wave setup, oceanographic oscillations, etc., critical to correctly predicting the wave energy that affects nearshore processes along the shoreline. Figure 24 illustrates a simplified profile of the wave breaking process across a fringing reef.

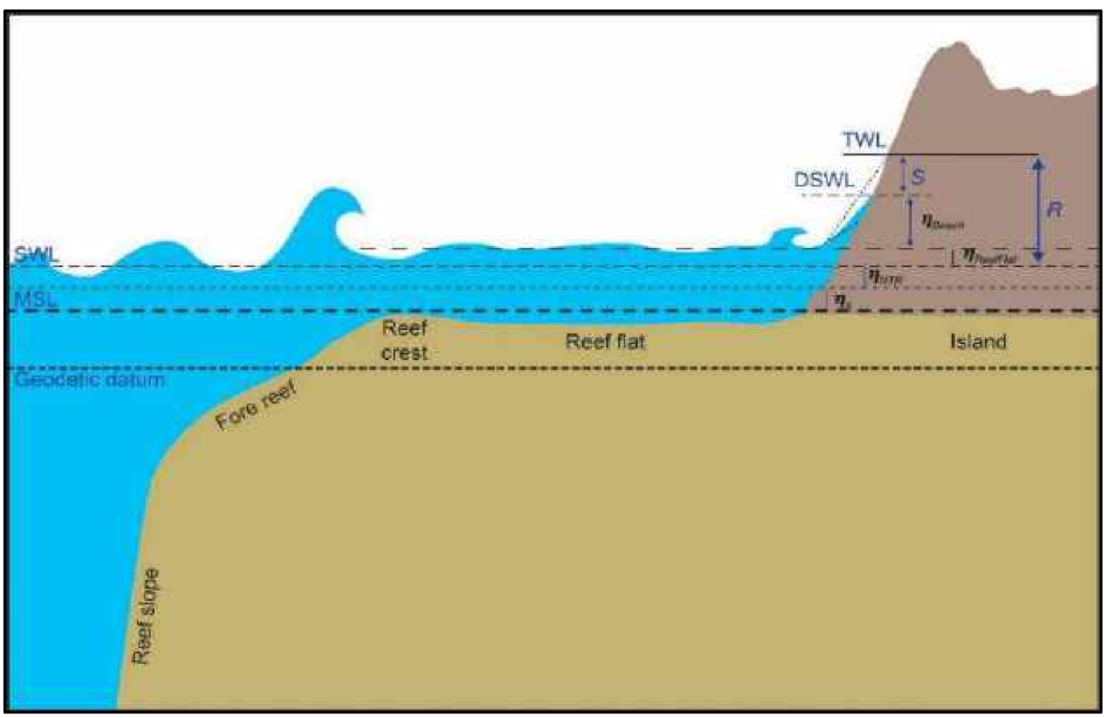

Figure 24. Schematic of wave breaking over a typical reef profile.

Similarly, circulation patterns are strongly influenced by both water level changes and nearshore bathymetry. In deeper water far offshore of the fringing reef, currents are typically small (<0.3 m/s) and controlled by changes in tide and to some extent, wind direction and speed. Circulation is generally spatially homogeneous in these areas, with some variation due to the formation of eddies and other oceanographic phenomenon occurring in intermediate and deep water. Current patterns in shallow water are affected by the fore reef and reef flat bathymetry, and are significantly more variable in both magnitude and direction than those offshore. A representative overview of a typical current pattern in the region illustrates these spatial variations in Figure 25 below, a color contour plot of depth averaged current velocity magnitudes in meters/second, and vector arrows representing current direction.

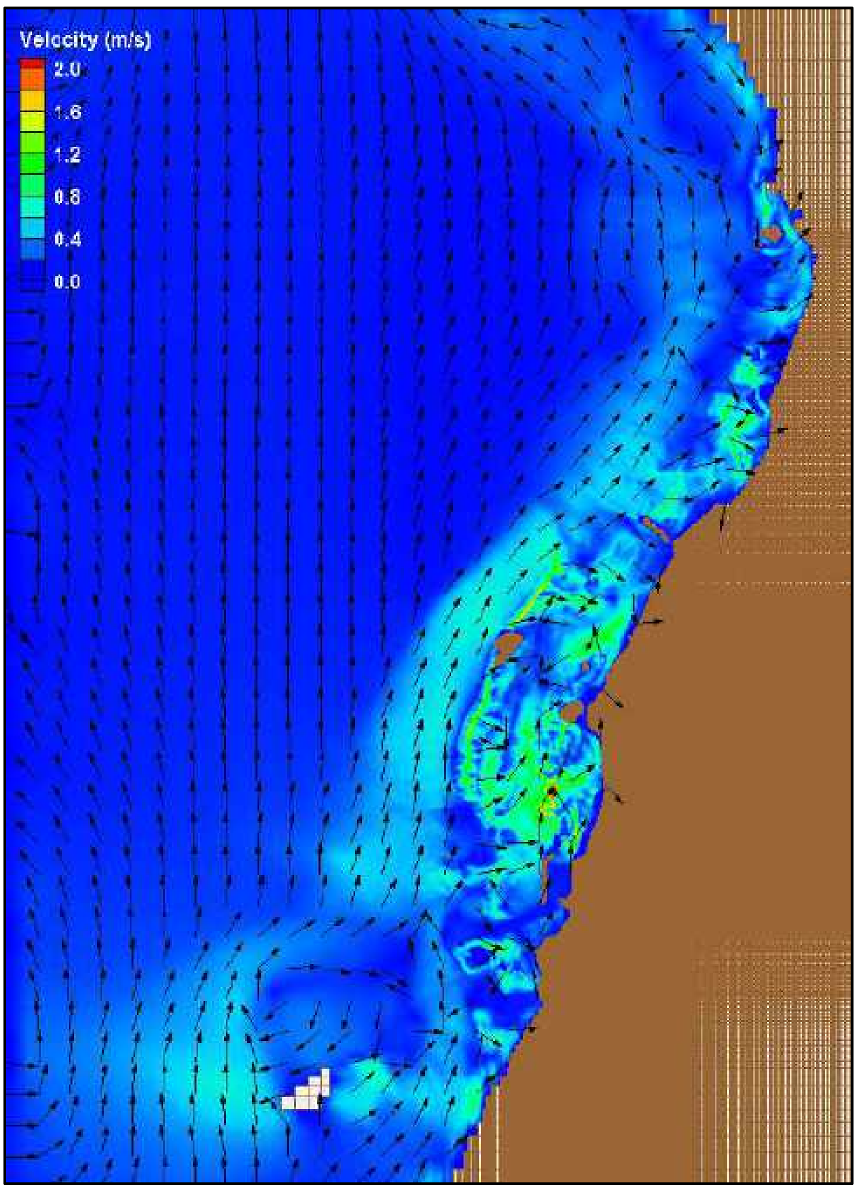

Figure 25. Typical current patterns offshore and nearshore in study area.

Wave breaking at the reef edge may result in higher currents in these areas due to radiation stress gradients generated during this process. The wave breaking process also generates wave setup (an increase in nearshore water level, thereby increasing the depth of water on the reef flat). This water level increase in combination with the tidal stage can dictate the direction and magnitude of current flow in the nearshore. In other words, an area along the shoreline with substantial wave breaking and wave setup will experience a 'build up' of water surface elevation in the nearshore, while an area down the coast that may have less wave setup due to sheltering or deeper water will have a slightly lower water elevation. This gradient will cause water to flow from areas of high water surface elevation to areas of lower water surface elevation, and can result in strong currents. These nearshore currents are also affected by sharp changes in bathymetry, such as natural or man-made channels existing on the reef flat. All of these factors (wave breaking, tides and bathymetry) can create a complex and dynamic circulation field in a fringing reef environment such as the Agat Bay region, and are observed in the CMS model results.

The results of the wave-circulation modeling can be examined with plots of the wave and current fields in areas of interest within the region. The conditions shown in these plots, however, are 'snapshots', and therefore, highly dependent on the tidal stage of the time series from which they are taken. While wave conditions are relatively consistent over the 3-day simulations, tidal elevations vary by as much as 2.5 feet (0.76 m) on a roughly 24-hour cycle. During very low tide, the most shallow areas of the reef flat are dry, and therefore of little interest for circulation driven sediment transport. The following analyses focus on times of high slack (calm) tide, ebb (outgoing) tide, or flood (incoming) tide. It should also be noted that CMS-Wave includes a limited approximation of wave setup, as do most phase-averaged wave models. Since typical wave conditions (rather than extreme conditions) are being examined for the purposes of circulation and sediment transport, wave setup values will likely be small relative to total water level and are not considered to be highly influential on the results shown.

6.1.3.1. Prevailing Wave Conditions

The wave hindcast analysis presented in section 3.2.3 indicated that the prevailing offshore wave conditions that could affect the study region had the following parameters:

- Significant wave height (Hs) of 1 – 2 meters (occurring 67% of the time)
- Spectral peak wave period (Tp) of 8 -10 seconds (occurring 47% of the time)
- Spectral peak wave direction (Dp) of 255 -265 degrees (occurring 16% of the time and correlating closely with the above wave height and periods)

In order to represent this condition, the WIS hindcast was searched for events that had these approximate parameters, sustained over several days. A condition that occurred in late August of 2011 (wind and wave parameters shown in Table 6) was selected for simulation. Six-minute interval recorded water level elevations during this period from the Apra Harbor water level gage were downloaded and provided as the circulation model boundary condition. The water level station time series from NOAA/CO-OPS Station 1630000 at Apra Harbor during August 26-28, 2001 is shown in Figure 26. The plot indicates that recorded water levels (green line) are approximately 0.15 to 0.20m (0.6-0.65 ft) above predicted tidal elevations (blue line). This is likely due to oceanographic processes (e.g. – Pacific Decadal Oscillation, El Nino, etc.) that have been mentioned previously and have been documented in research. The recorded (not predicted) water levels from this station were used for this and all other simulations.

Deep water wave parameters were used to generate wave spectra at a 3 hour intervals, and in combination with WIS wind speed and direction at the same save point, were used to provide the boundary condition for the wave model. The parent grid simulations were completed with the linked models, and time series output from selected locations along the inshore boundary of both models was used to provide boundary conditions for the child grid simulations over the same time period.

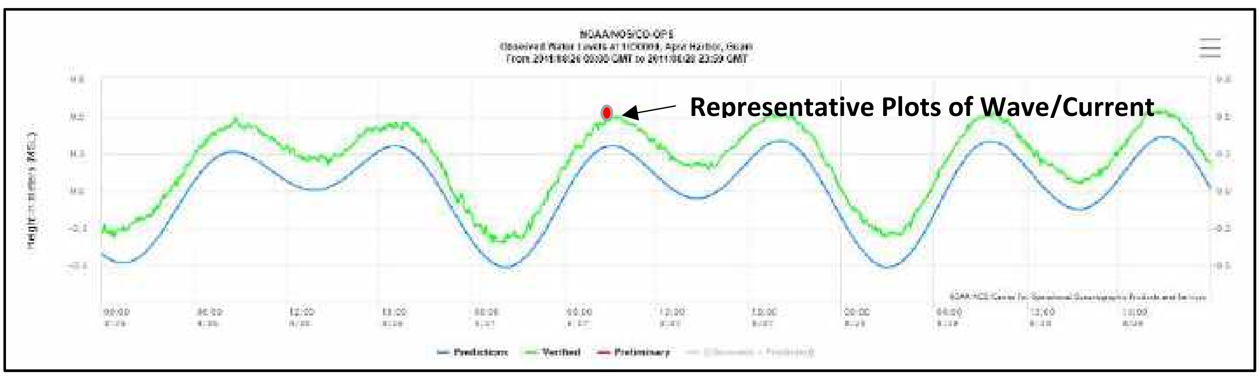

Figure 26. Predicted and recorded water levels at NOAA/CO-OPS Station at Apra Harbor, Guam for 26-28 August 2011.

Representative wave and current field snapshots from August 27, 2011 at 9:30 am Greenwich Mean Time (GMT) are shown as an example of conditions in the nearshore during a high tide water level (0.495m MSL or 1.62 ft MSL), annotated in Figure 26. Figure 27 shows a color contour plot of wave height in the northern half of the study region at this time. Areas of interest including Inn on the Bay and Agat Mayor's Office are annotated for reference. The plot indicates wave heights outside the nearshore in the range of 1.25 to 1.75 meters as shown in green, comparable with the offshore wave height of ~1.5 meters. Over the nearshore reef flat, wave heights are reduced to the range of 0.1 to 0.25 m as shown in blue, due to the wave breaking process described earlier. Examination of other wave field plots in this area at lower water levels during the simulation indicate similar to smaller wave height magnitudes, as would be expected due to the relationship between wave height and water level.

Figure 28 shows a similar wave contour plot for the southern end of the study region, near Agat Small Boat Harbor and Nimitz Beach Park. Wave heights in intermediate water depth are similar to those in the previous figure at 1.25 to 1.75 meters, with some areas approaching 2.0 meters significant wave height in likely areas of wave breaking due to shallow bathymetric features. Nearshore wave heights over the reef flat are small at 0.1 to 0.25 meters, with some nearshore channels showing larger wave heights in the range of 0.5 to 1.0 meters approaching the shoreline in the vicinity of the harbor and beach park. Again, review of other wave field plots in this area at lower water levels during the simulation indicate similar to smaller wave height magnitudes, as would be expected due to the relationship between wave height and water level.

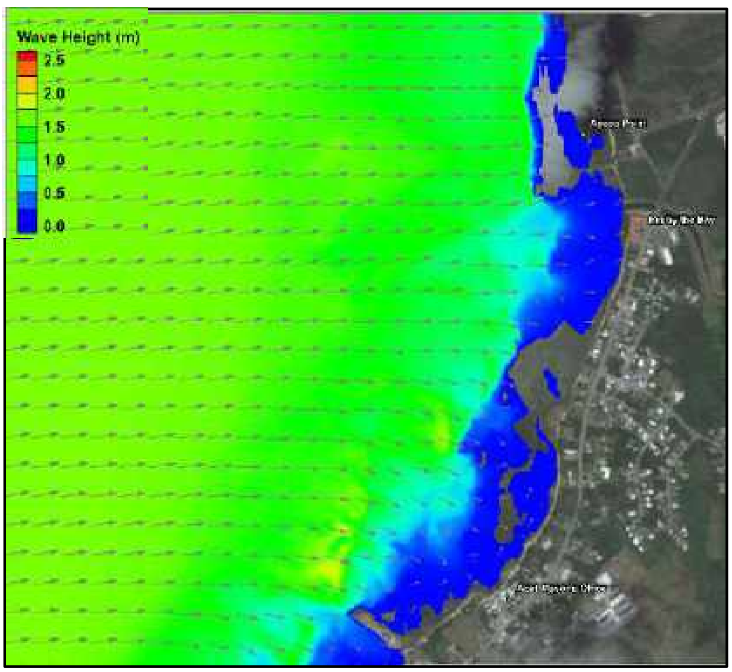

Figure 27. Wave field contour plot at high tide in northern end of study region. (Arrows indicate wave direction, colors indicate wave height)

Examination of water surface elevation (WSE) illustrates the process of wave setup causing gradients in water level along the shoreline discussed previously. Figure 29 shows the WSE near Agat SBH at the same time in the simulation (high tide). In this plot, higher elevations (up to 1.5 m above MSL) are shown in blue, while low elevations (not visible at this high tide stage) would be shown in red. The relative increase in water level north of the harbor is caused by wave breaking across the widest reef area in the region (refer back to aerial photo in Figure 14). The gradient in WSE between this location and the harbor is on the order of 0.5 to 0.75m, and causes a southward directed current that is evident in upcoming plots.

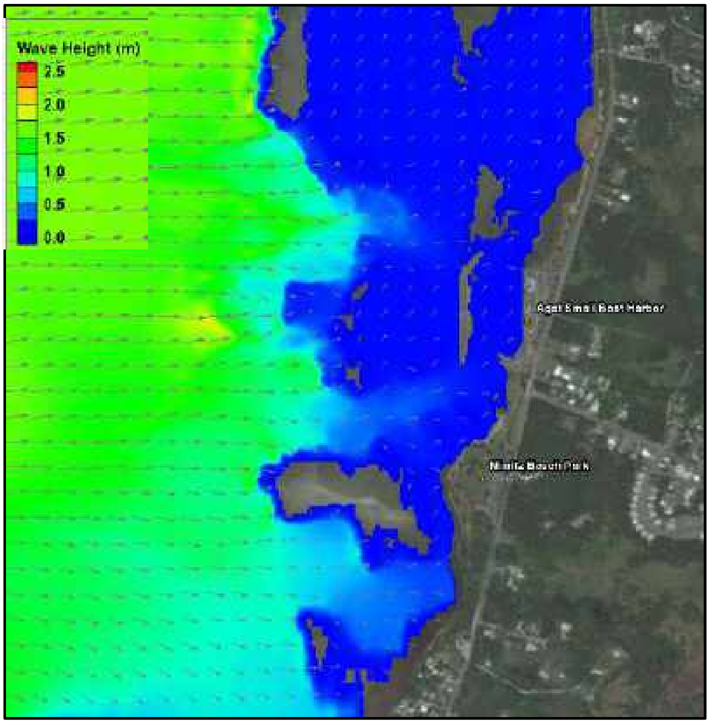

Figure 28. Wave field contour plot at high tide in southern end of study region. (Arrows indicate wave direction, colors indicate wave height)

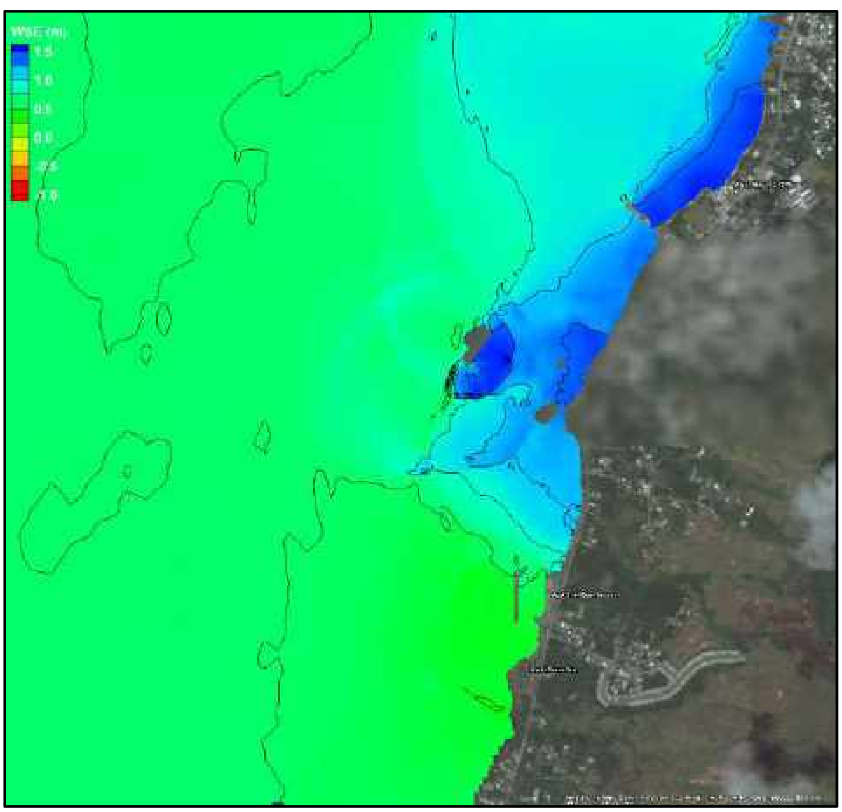
Figure 29. Water surface elevation (WSE) plot at high tide near Agat SBH.

Detailed current velocity plots during this time in the simulation provide insight into circulation patterns in and around the areas of interest within the region. Figure 30 is a plot of the current field near Inn on the Bay, where current velocities are plotted with higher velocities indicated by the green to red colors, and lower velocities in blue. Vector arrows represent current direction at this time in the simulation. The plot shows velocities along the shoreline in the 0.0 to 1.0 m/s range, directed southward from Namo River to the Inn on the Bay, and converging with northward directed currents from Inn on the Bay on. At this convergence of currents, there is an offshore directed flow directed toward an area of higher current magnitudes (2.0 to 3.0 m/s) near the island offshore (indicated by blank simulation cells).

A similar current velocity plot is shown near the Agat Mayor's Office in Figure 31. This plot indicates a northward directed nearshore flow in front of the Mayor's Office that converges with a southward directed flow further up the shoreline, with the combination of flow directions moving offshore. This is consistent with the area of erosion adjacent to a littoral barrier just in front of the mayor's office, and a small salient that has formed in the area of convergence and is visible in aerial photography (refer back to Figure 19). Similar to the previous plot, velocities just off the shoreline are larger in magnitude as indicated by green shading, and are directed offshore to the northwest.

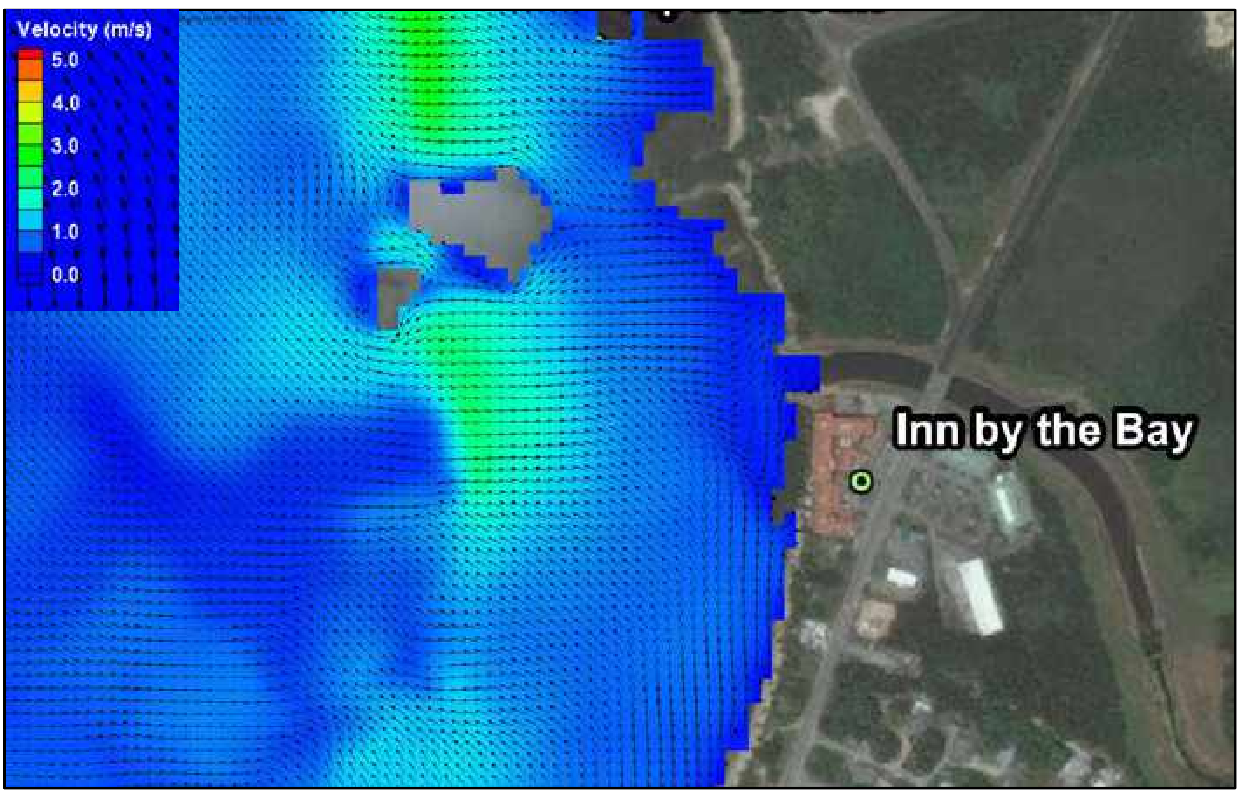

Figure 30. Current velocity field at Inn on the Bay on 8/27/11 at 9:30 am (GMT)

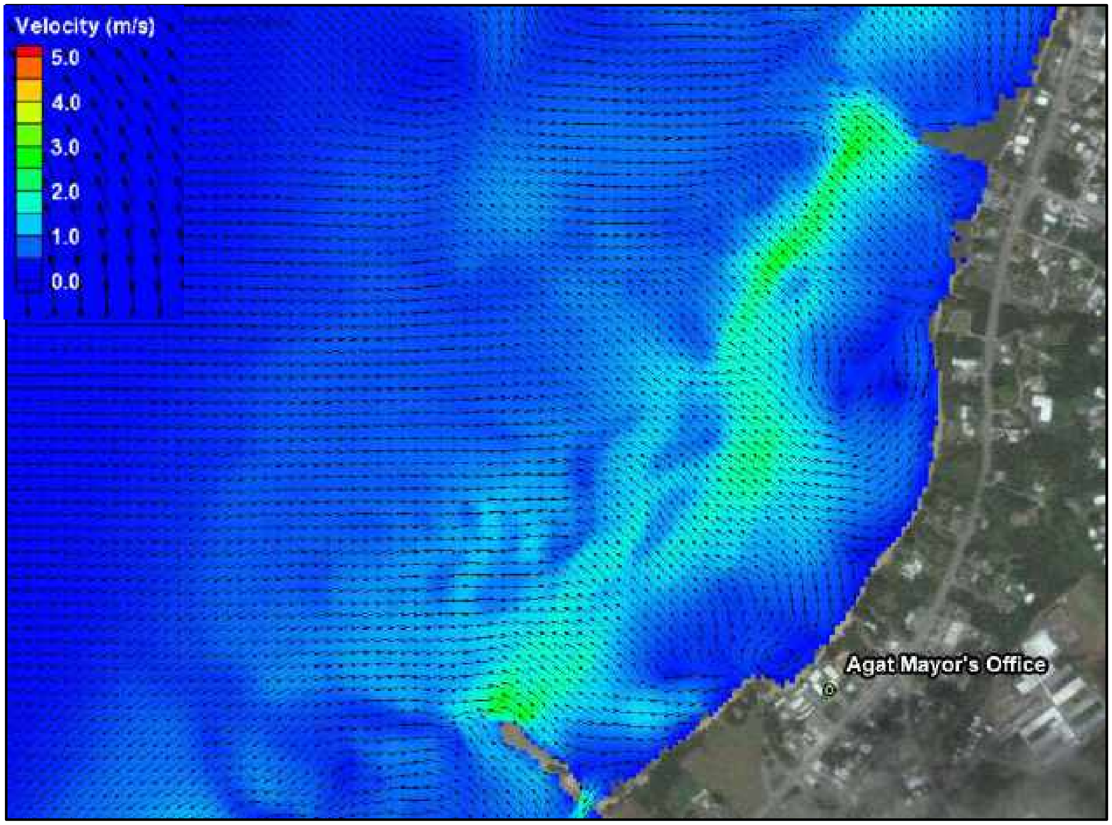

Figure 31. Current velocity field at Agat Mayor's Office on 8/27/11 at 9:30 am (GMT)

Finally, a third current velocity plot is shown for the area surrounding Agat SBH and Nimitz Beach Park in Figure 32a. Currents along the shoreline just north of the harbor are directed toward the south, with a relatively strong current (2.0+ m/s) entering the harbor and continuing along the interior of the main breakwater. A strong current here is in agreement with the gradient in WSE mentioned previously, and is consistent with the observed shoaling in the harbor coming from the north as mentioned in section 4.1, as well as harbor user observations of strong currents in the harbor. Currents to the south of the harbor at Nimitz Beach Park are directed toward the south and offshore. Further discussion of how these circulation patterns may be affecting sediment transport in the regions will be presented in the following section.

For comparison to pre-project conditions, an assumed bathymetry was used (with federal navigation features removed) for a simulation with the same prevailing wave and water level boundary conditions. Results at the same time and extents are shown in Figure 32b. This comparison shows that the southward directed current flow in the Agat SBH vicinity exists without the harbor, though with slightly lower velocity magnitude. Another noteworthy difference is that longshore current in the Nimitz Beach Park region without the harbor is more dominant toward the south. This indicates that, without the harbor, the dominant sediment transport direction would also be from north to south under prevailing wave conditions.

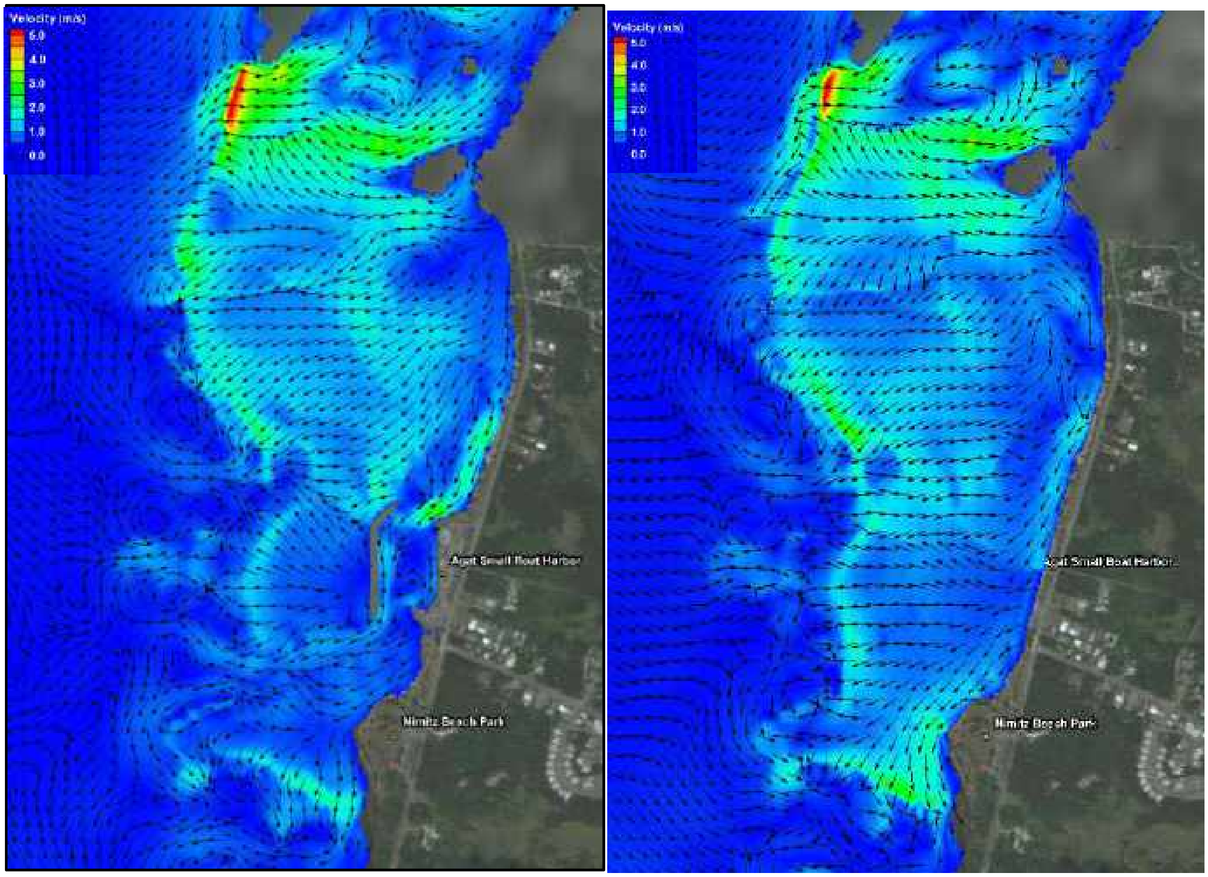

Figure 32a and 32b. Current velocity field at Agat SBH and Nimitz Beach Park on 8/27/11 at 9:30 am (GMT) for existing conditions (left) and pre-project conditions (right)

6.1.3.2. Calm Wave Condition Simulation

A low wave condition was also simulated in order to evaluate nearshore processes that occur during calmer times and compare these to prevailing conditions. The wave hindcast analysis was searched for this condition which occurred in August 2008:

- Significant wave height (Hs) of < 1.0 meter
- Spectral peak wave period (Tp) of 7 -10 seconds
- Spectral peak wave direction (Dp) varying from SW to NW
- Winds variable from SE through SW at 2-4 m/sec

This condition was modeled using the same boundary forcing (wave spectra and wind from WIS, and water levels from Apra Harbor NOAA gage) sources as the more prevailing condition, and at the same frequency, but for the different time period. This calm condition showed similar characteristics to the prevailing condition, including wave breaking at the reef crest and a homogeneous current pattern offshore of the reef, with a more complex circulation pattern over the reef flat and in areas of sharp changes in bathymetry.

Current magnitudes in the nearshore were weaker than those for the higher wave condition (i.e. – less than 1.0 m/s), as would be expected. There were also some areas where reversals of the predominant current direction were observed, as shown in the plot in Figure 33 at Agat SBH and Nimitz Beach Park. In contrast with the prevailing condition, flow through and offshore of the harbor goes from south to north, as does the current along the Nimitz Beach Park shoreline. This simulation shows that although these conditions do not occur often, there can be times when circulation changes from the dominant direction, and may cause weak reversals in sediment transport direction and changes to navigation practices.

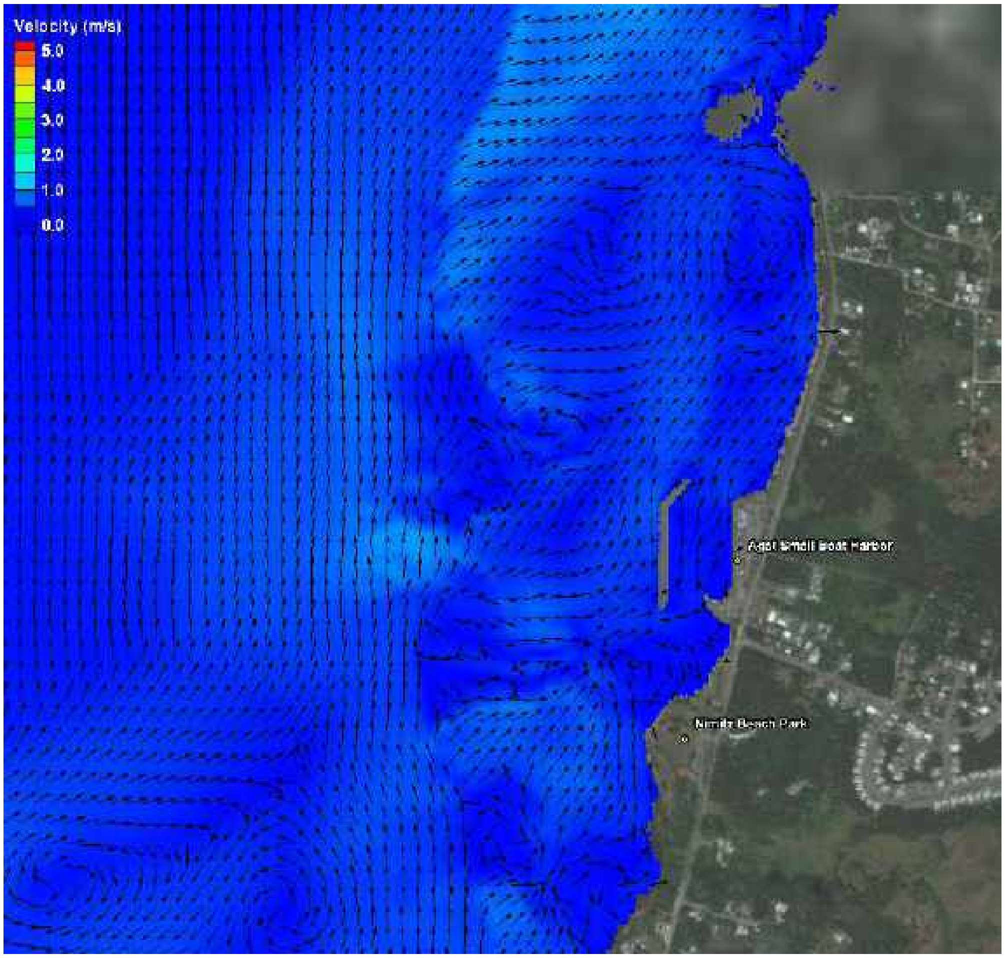

Figure 33. Representative current velocity field at Agat SBH and Nimitz Beach Park for low wave condition (at highest tide)

6.1.3.3. Annual Return Period Wave Condition Simulation

In order to generate an upper bound for 'typical' annual wave conditions an event corresponding to the 1-year return period (99% probability of exceedance) was simulated. The wave hindcast analysis was searched for this offshore condition which occurred in July/August 2009:

- Significant wave height (Hs) of ~ 3.0 meter
- Spectral peak wave period (Tp) of 10-12 seconds
- Spectral peak wave direction (Dp) from SW (245 deg TN)

Tables 2 and 3 (frequency analysis of offshore wave parameters) show that this wave height occurs about 4% of the time, and the wave period range occurs about 26% of the time. The tables also indicate that larger wave events such as this are more likely to come from the southwest, as opposed to the prevailing condition which can come from southwest or northwest. This condition was again modeled using the boundary forcing (wave spectra and wind from WIS, and water levels from Apra Harbor NOAA gage) sources as the previous simulations, and at the same frequency, but for the different time period. This annual event showed similar characteristics to the prevailing condition, including wave breaking at the reef crest and small waves (<0.5m) across the reef flat, and a homogeneous current pattern offshore of the reef, with a more complex circulation pattern over the reef flat and in areas of sharp changes in bathymetry.

The current fields and WSE plots did show less wave setup across the wide reef just to the north of Agat SBH, which may be due to the differing angle of the incoming waves. At several times in the simulation, this caused currents to flow in opposite directions of the prevailing condition, as shown in the current field plot at Agat SBH during a high tide condition in Figure 34, where currents flow through the harbor from south to north.

Several discrete three-day time series of this historic information were selected in order to represent typical wave and current conditions occurring over several tide cycles. Use of these representative oceanographic conditions provides a general understanding of current conditions that would occur frequently, and therefore, dominate circulation patterns and sediment transport patterns. Correlation between these currents and potential sediment transport pathways will be evaluated in the next section of this report.

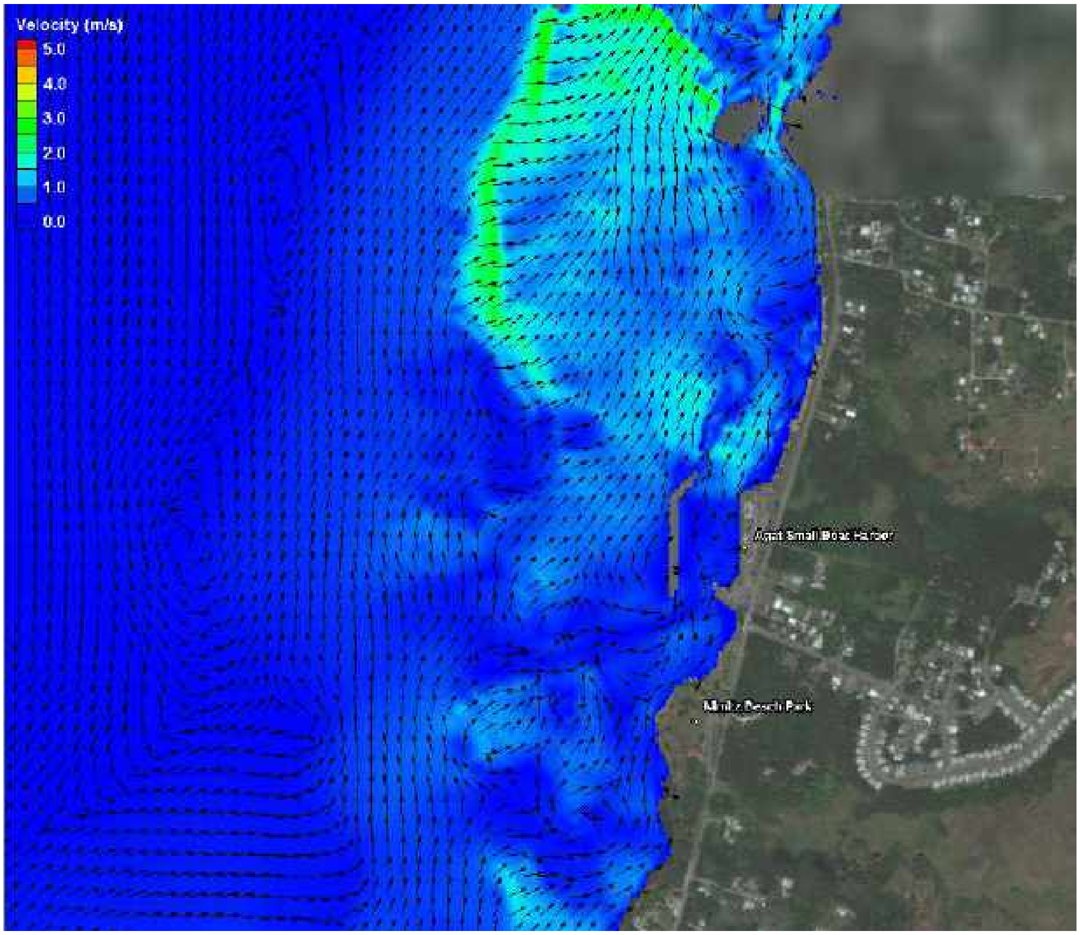

Figure 34. Representative current velocity field at Agat SBH and Nimitz Beach Park for annual wave condition (at highest tide)

6.2. Sediment Transport Pathways in the Region

The Particle Tracking Model (PTM) is a Lagrangian-based particle transport model that allows the user to simulate particle transport processes to determine particle fate and pathways. The model uses waves and currents from CMS as forcing functions, and provides powerful visualizations of sediment transport pathways for a given simulation. In the application of PTM for this study, particles were color coded according to the location where they originated. Particles were estimated to be 0.5mm (1.0Φ, medium to coarse sand) in median diameter (d_{50}), based on *Marine Geology of Guam* (Emery, 1962), and were released at a continuous rate over 3 hours within the 3-day simulations. Sources were placed at four locations within the region, and as close to the shoreline as possible: Inn on the Bay, Agat Mayor's Office, north of Agat SBH, and Nimitz Beach Park. PTM is utilized here as a purely qualitative tool; it is not intended to quantify actual volumes that are mobilized during a wave event, nor the amounts that are deposited in specific locations. Rather, it is intended to identify the potential sediment pathways for a particular wave and current condition, and therefore provide insight to the dominant directions of sediment transport over the long-term.

6.2.1.1. PTM for Prevailing Wave Conditions

Sediment transport pathways identified by the PTM simulations for the prevailing wave condition (Hs = 1.5m, Tp = 9.0s, Direction (Dir) = WSW) are likely to illustrate the primary sediment pathways driving transport in the region, and associated shoreline changes observed in the shoreline change analysis previously presented. Figures 35a and 35b present an overview of the PTM simulation for the Agat Bay region, at two distinct points in time, in order to visualize the movement of sediment over the simulation period. Figure 35a on the left shows the PTM results 3 hours following the initial release of particles, and Figure 35b on the right shows PTM results 18 hours later (21 hours following initial release). This later image is intended to show areas where particles that were initially transported out of the release area may have become inactive (deposited on the sea floor), either by going offshore into deep water, or along the shoreline in an area where currents will no longer mobilize them for transport. Release points are indicated by black triangles, and particles are colored by source location.

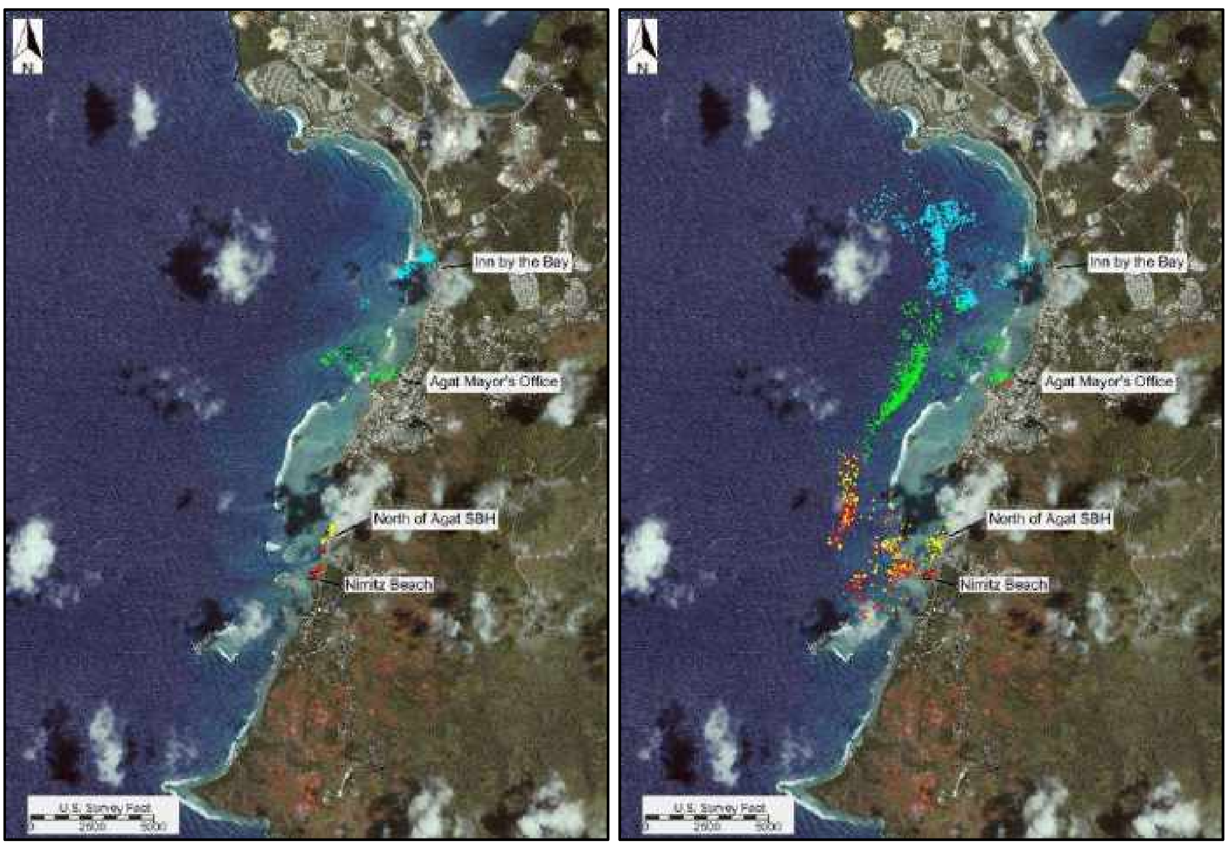

Figure 35a and 35b: PTM Results for the prevailing wave condition at 3 hours following release (left) and 21 hours following release (right), over the entire Agat Bay region.

Several general observations about sediment pathways for this condition can be made using this overview. First, blue and green particles (released from Inn on the Bay and Agat Mayor's Office shorelines, respectively) appear to be transported offshore relatively quickly in the simulation, indicating a significant westward directed current in this part of the region. Particles near Agat SBH and Nimitz Beach Park (shown in yellow and red, respectively) remain close to the shoreline in the 3 hours following release, but appear to be moving more alongshore in this part of the region. In Figure 35b, 18 hours following the release of particles, a significant portion of particles from all sources has moved offshore, likely into deeper water where they are no longer active and will no longer be available to the littoral system. This indicates that some of the overall volume losses in the region are due to particles moving outside the active littoral zone through offshore directed channel flow. There are also particles that remain along the shoreline; the fate and pathways of this sediment will be examined in the following figures with a closer look at specific points of interest.

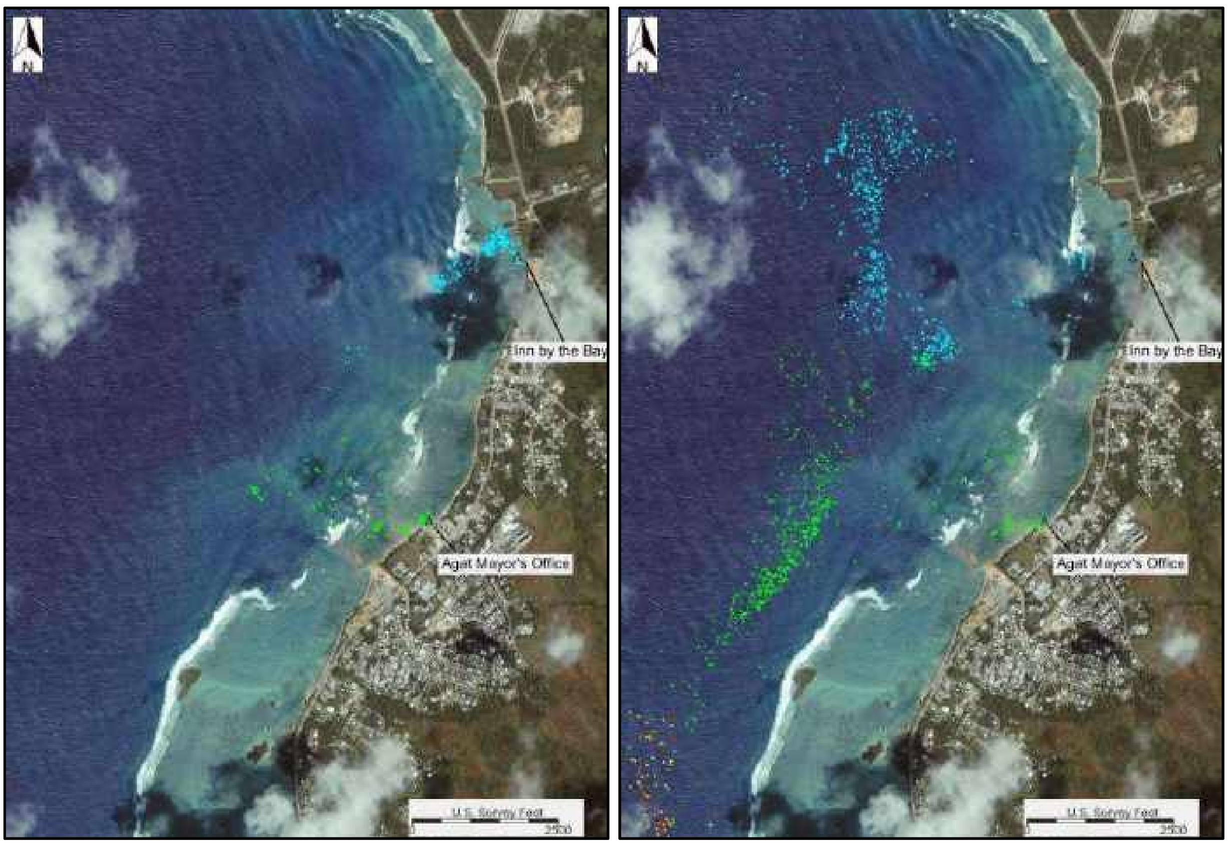

Figure 36a and 36b: PTM Results for prevailing wave condition at 3 hours following release (left) and 21 hours following release (right), at Inn on the Bay and Agat Mayor's Office.

Figures 36a and 36b show a magnified version of the previous PTM results at the northern end of the region, displaying results from the sources near Inn on the Bay and Agat Mayor's Office. From these plots it can be seen that the initial transport direction at Inn on the Bay appears to be to the north, and later some of the material moves toward the offshore. The sediment release at the Agat Mayor's Office appears to be moving toward the south, with some transport offshore as well, later in the simulation. At both locations, the PTM results indicate that before sediment can travel very far along the coast, at least some portion of it is moved toward the offshore under these circulation conditions, resulting in a sediment deficit for these areas.

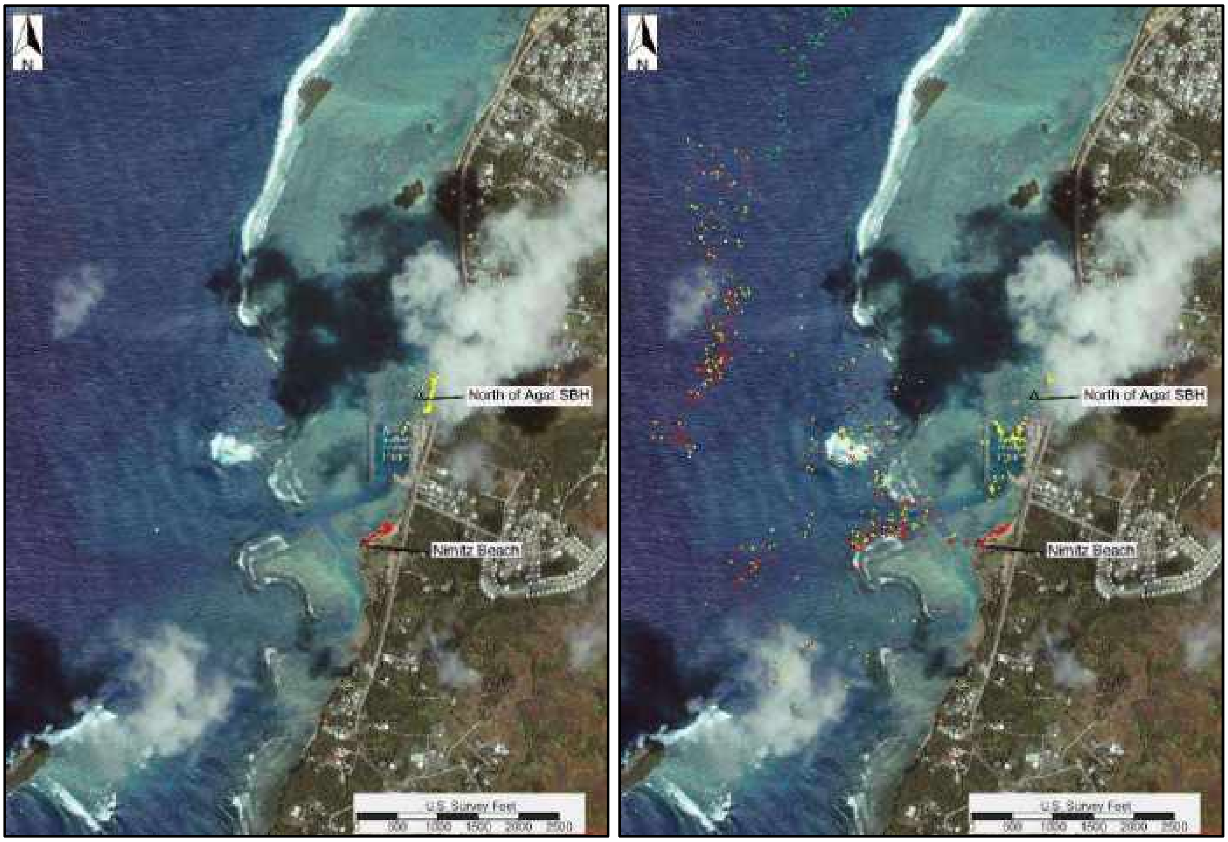

Figure 37a and 37b: PTM Results for prevailing wave condition at 3 hours following release (left) and 21 hours following release (right), at Agat SBH and Nimitz Beach Park.

Figures 37a and 37b show a magnified version of the PTM results at the southern end of the region, displaying results from the sources north of Agat SBH and Nimitz Beach Park. Figure 37a shows that in the first three hours after release, transport at the shoreline area north of Agat SBH is in both the north and south directions, while transport at Nimitz Beach Park is consistently toward the north. Later in the simulation (Figure 37b), particles from both sources have moved offshore (likely through the harbor entrance channel and adjacent channels), but the majority of particles from the area north of Agat SBH have moved into the harbor, and particles from Nimitz Beach Park are still consistently moving north along the shoreline. These results indicate that: 1) closing off Agat SBH to flow from the north could reduce harbor shoaling, 2) a littoral barrier along north of the harbor would reduce sediment transport out of the shoreline area to the north and 3) erosion at Nimitz Beach Park on the southern side is supplying sediment to the accreting areas on the north side of the park for these conditions.

6.2.1.2. PTM for Calm Wave Conditions

Sediment transport pathways identified by the PTM simulations for the calm wave condition (Hs = 0.8m, Tp = 8.0s, Dir = SW/NW) are less likely to represent dominant pathways, because of their relative infrequency. However, examination of these results is still informative because they may better represent transport in lower tide conditions, where minimal wave energy reaches the shoreline. Figures 38a and 38b present an overview of the PTM simulation for the entire Agat Bay region, again Figure 38a displaying 3 hours following the initial release of particles, and Figure 38b showing PTM results 18 hours later (21 hours following initial release). Release points are in the same location as the prevailing wave condition, and particles are again colored by source location.

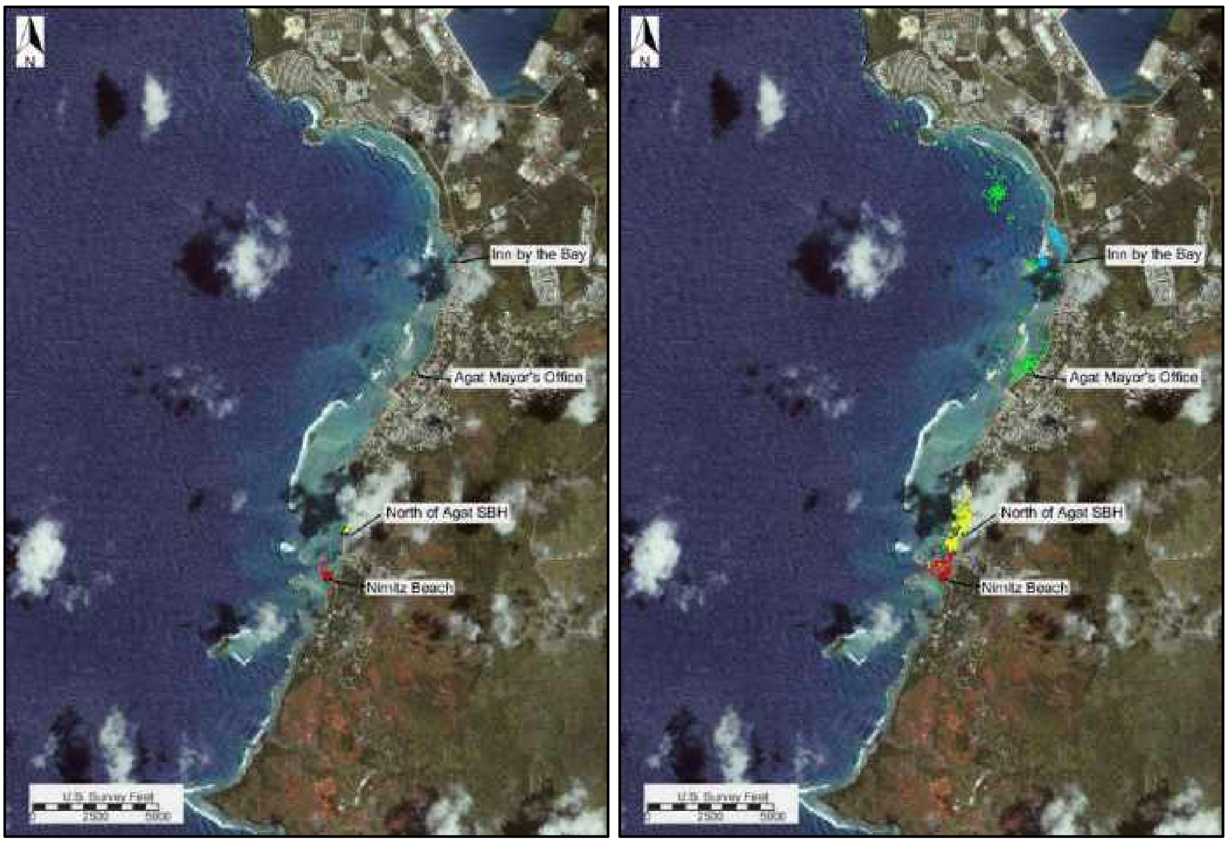

Figure 38a and 38b: PTM Results for the calm wave condition at 3 hours following release (left) and 21 hours following release (right), over the entire Agat Bay region.

The initial movement of particles shown in Figure 38a is relatively minimal for all source points, as would be expected under smaller wave conditions which generate weaker circulation. There is minimal transport out of the source area in the northern region at Inn on the Bay and Agat Mayor's Office, and slight northward transport in the southern region at the areas north of Agat SBH and Nimitz Beach Park. Later in the simulation, it is evident that there is much less offshore transport of material at all locations than for the prevailing wave condition, which would be expected in a condition with weaker circulation.

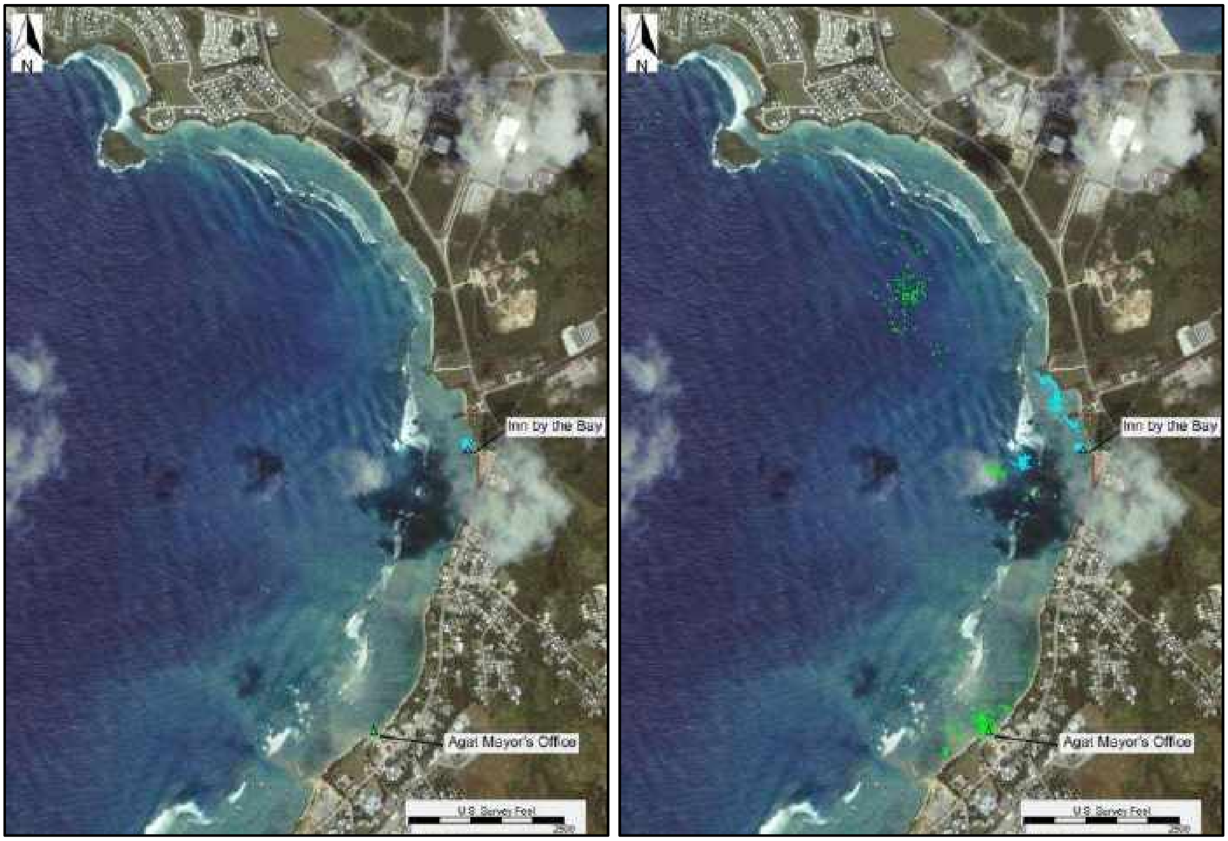

Figure 39a and 39b: PTM Results for prevailing wave condition at 3 hours following release (left) and 21 hours following release (right), at Inn on the Bay and Agat Mayor's Office.

Figures 39a and 39b show a magnified version of the calm wave condition PTM results at the northern end of the region, displaying results from the sources near Inn on the Bay and Agat Mayor's Office. As mentioned, minimal movement of particles is seen in the first three hours after particle release (Figure 39a). Later in the simulation, shown in Figure 39b, transport at Inn on the Bay is consistently to the north (in agreement with transport direction in the prevailing wave condition), and particles at Agat Mayors' office spreading evenly between north and south along the nearshore, with some particles moving offshore and to the north beyond the reef flat.

Figures 40a and 40b show a magnified version of the PTM results at the southern end of the region, displaying results from the sources near the north of Agat SBH and Nimitz Beach Park. Early in the simulation, small particle movement to the north is apparent at both locations. Later in the simulation, particles have spread to both the north and south of the initial release point north of Agat SBH, and some particles have settled in the harbor. Points from Nimitz Beach Park have moved to the north, with some moving into the harbor entrance channel via a small connection channel between the two areas. Other particles have remained along the shoreline to the north of the release point, where accretion was noted in aerial photos. These results show that under calm conditions: 1) closing off Agat SBH to flow from the north would reduce harbor shoaling, 2) a littoral barrier along north of the harbor would reduce sediment transport out of the shoreline area to the north and 3) erosion at Nimitz Beach Park on the southern side is supplying sediment to the accreting areas on the north side of the park, as well as causing some of the shoaling observed in the entrance channel.

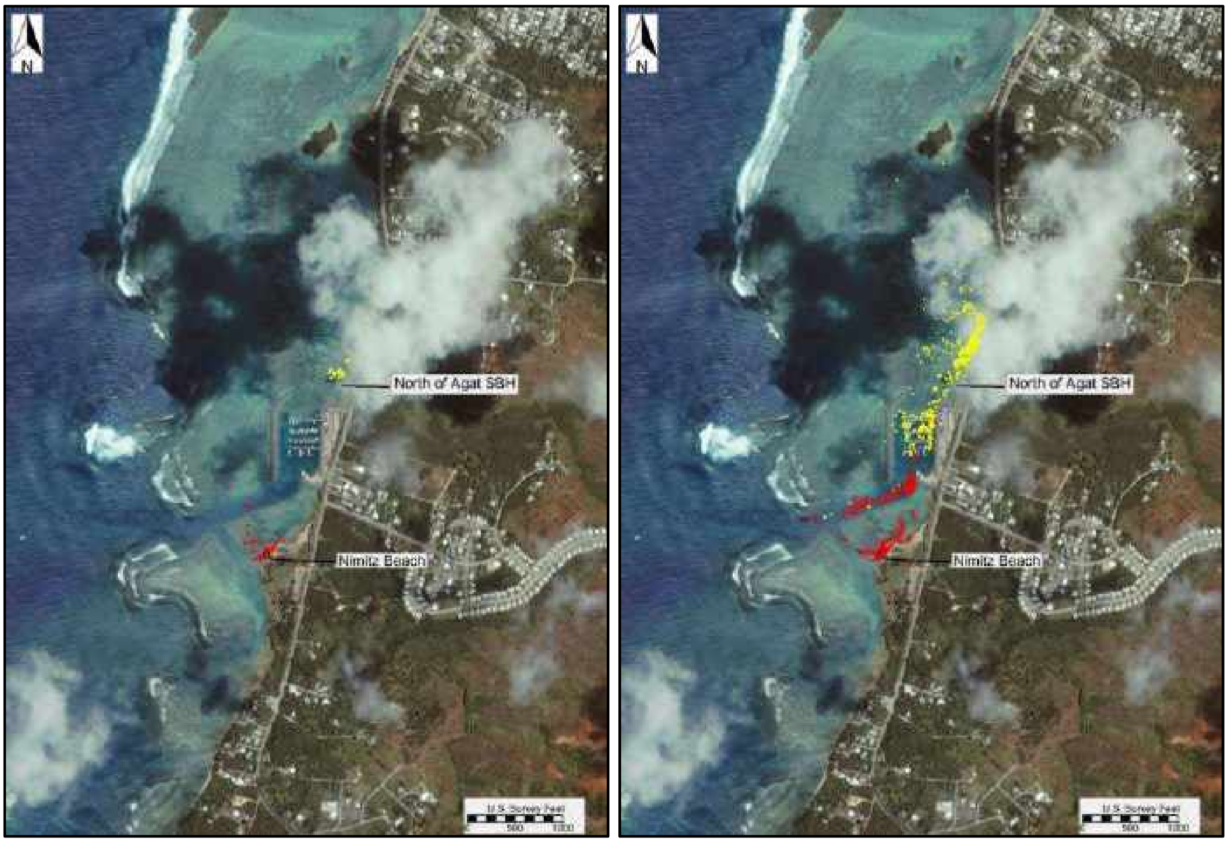

Figure 40a and 40b: PTM Results for Calm Wave Condition at 3 hours following release (left) and 21 hours following release (right), at Agat SBH and Nimitz Beach Park.

6.2.1.3. PTM for Annual Event Wave Conditions

Sediment transport pathways identified by the PTM simulations for the annual extreme wave condition (Hs = 3.0m, Tp = 11.0s, Dir = SW) are less frequent, and less likely to dominate sediment transport pathways. This condition is useful in showing an annual extreme event which has the capability to transport a significant amount of sediment, even if it is only experienced once per year on average. Figures 41a and 41b show the PTM results 3 hours following the initial release of particles, and 18 hours later (21 hours following initial release). Release points are in the same location as the previous wave conditions, and particles are again colored by source location.

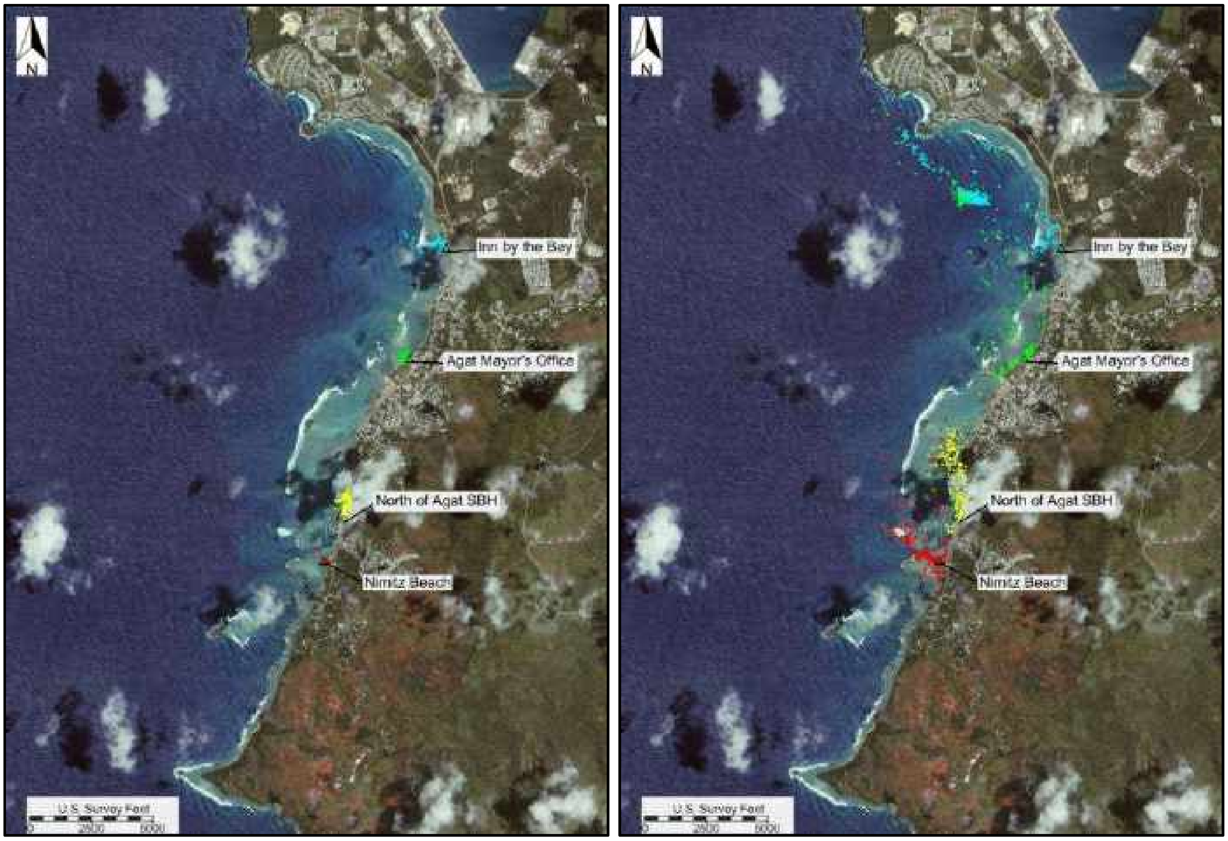

Figure 41a and 41b: PTM Results for the annual extreme wave condition at 3 hours following release (left) and 21 hours following release (right), over the entire Agat Bay region.

The initial movement of particles shown in Figure 41a is moderate for all source points. There is slight northward transport in all locations, possibly due to the southwesterly direction of incoming waves and associated currents. Later in the simulation, the northerly transport of particles continues at Inn on the Bay, while the other three locations of interest show some variability in transport direction.

Figures 41a and 42b show a magnified version of the annual event wave condition PTM results at the northern end of the region, displaying results from the sources near Inn on the Bay and Agat Mayor's Office. Particles originating at the Inn on the Bay appear to be moving to the north immediately, with continued northerly transport and some offshore transport later in the simulation. The source at the Agat Mayor's Office shows offshore and somewhat northerly transport at the beginning of the simulation, but later particles have spread to both the north and the south of the original source point, with some material drifting to the offshore and to the north toward Apra peninsula.

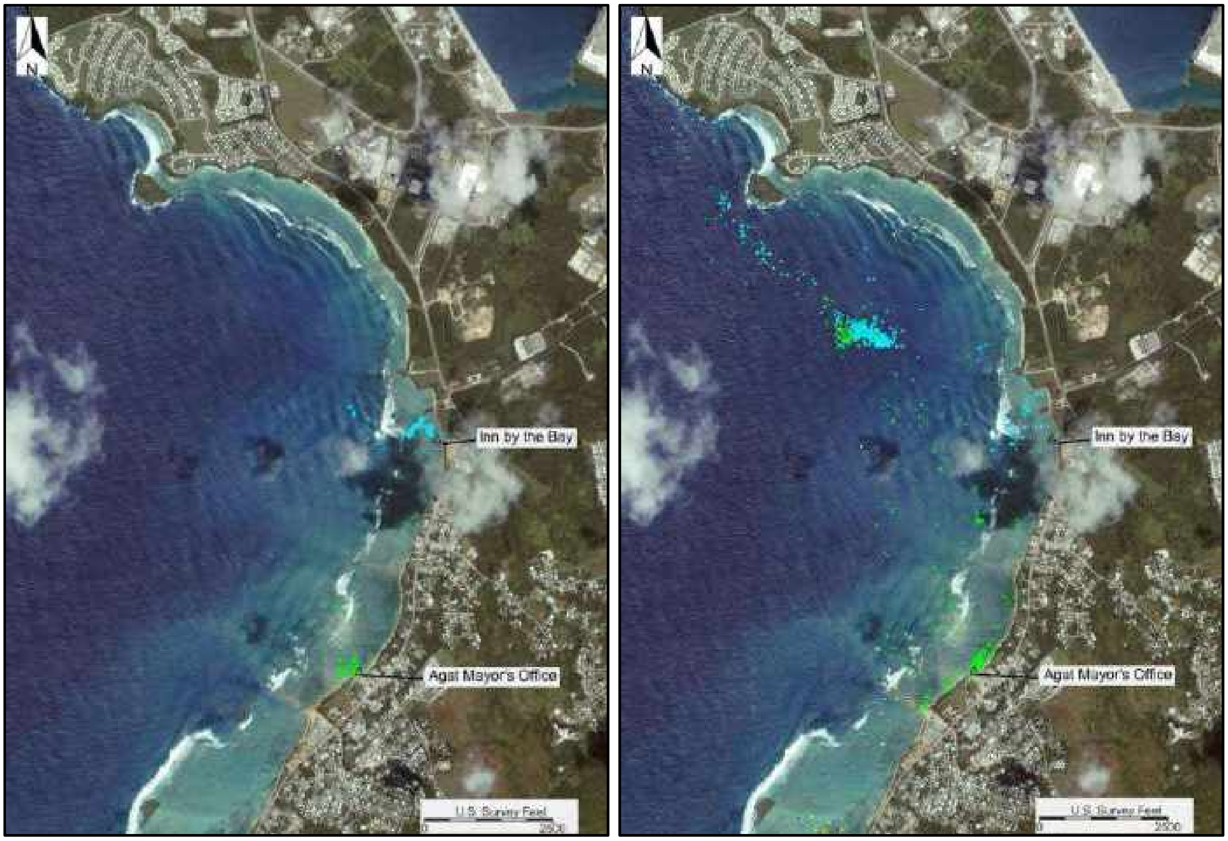

Figure 42a and 42b: PTM Results for annual extreme wave condition at 3 hours following release (left) and 21 hours following release (right), at Inn on the Bay and Agat Mayor's Office.

Figures 43a and 43b show a magnified version of the PTM results at the southern end of the region, displaying results from the sources near the north of Agat SBH and Nimitz Beach Park. Early in the simulation (Figure 43a), transport to the north is seen at the source adjacent to Agat SBH, while minimal transport out of the source area is seen at Nimitz Beach Park. Later in the simulation as shown in Figure 43b, particles are continuing to move toward the north from Agat SBH, with only a few particles going into the harbor. Some particles have attached to the shoreline, while others continue to the north around the small island in the nearshore. Particles from Nimitz Beach Park are again traveling through a small channel toward the Agat SBH entrance channel, similar to the other two simulations except that the particles are moving offshore after they reach the entrance channel. In contrast to the other simulations, some of the particles have also traveled toward the south side of Nimitz Beach Park in this simulation.

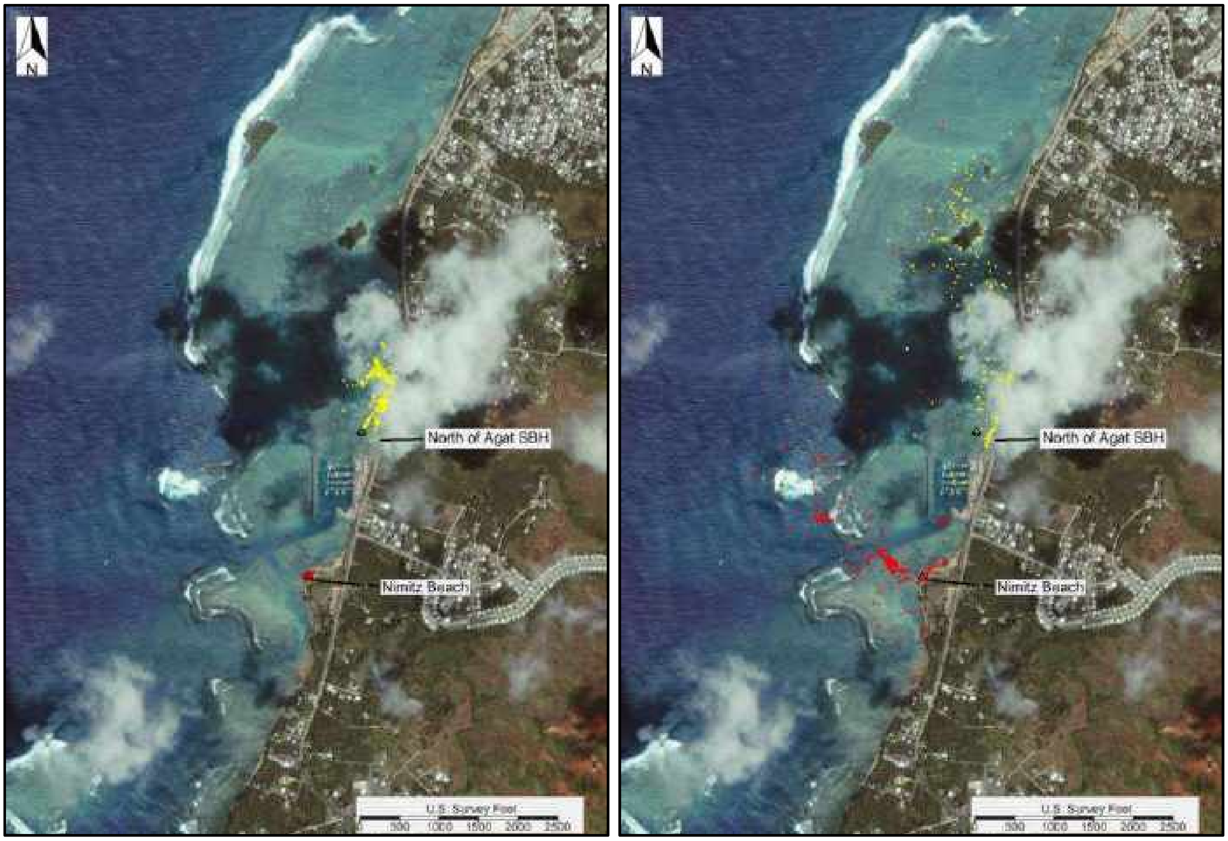

Figure 43a and 43b: PTM Results for annual extreme wave condition at 3 hours following release (left) and 21 hours following release (right), at Agat SBH and Nimitz Beach Park.

6.2.1.4. Summary of PTM Results for Three Wave Conditions

Table 7 summarizes the observed PTM transport directions discussed above, for each location of interest, and for each of the three modeled conditions. These general descriptions are made in order to determine if each location shows a dominant sediment pathway. The trends in transport at Inn on the Bay are fairly consistent throughout all three simulations, showing transport of particles offshore and sometimes to the north. At the Agat Mayor’s Office, transport is observed in both the north and south directions in the same condition, indicating that this may be an area where circulation diverges. Sediment transport along the shoreline to the north of Agat SBH is not as clear cut; particle movement is observed in both the northerly and southerly directions (and sometimes both) dependent on which simulation is examined. This may indicate some dependence on wave direction; when waves have a more westerly or northwesterly component (during prevailing and calm conditions) sediment moves toward the south and settles in the harbor. During an energetic southwesterly event (such as the annual event), transport is reversed toward the north. Finally, at Nimitz Beach Park, there is a consistent sediment pathway through a small channel joining the park shoreline to the outer entrance channel of Agat SBH. The direction of transport once particles reach the channel is more offshore during larger wave events and into the harbor during calm conditions, somewhat correlated to the transport north of Agat SBH. Transport along the shoreline is mostly consistent toward the north, which is in agreement with shoreline change analysis showing erosion at the southern end of the park and accretion at the northern end of the park. However, this differs with evidence of dominant southerly directed flow shown for the pre-project condition in Figure 32b. This indicates that construction of the harbor potentially reversed dominant transport direction in the Nimitz Beach Park area.

Table 7. Summary of PTM observations at areas of interest

	Simulation and Wave Direction			
Area of Interest	**Prevalent Wave Condition (WSW)**	**Calm Wave Condition (SW/NW)**	**Annual Wave Event (SW)**	**Dominant Direction?**
Inn by the Bay	strong offshore	north/moderate offhsore	north/strong offshore	north/offshore
Agat Mayors Office	south/offshore	north/south	north/south/offshore	north/south and offshore for bigger waves
Agat SBH	south into harbor	north/south into harbor	north	no - wave direction dependent?
Nimitz Beach Park	north and into entrance/offshore	north and into entrance/harbor	north and into entrance onshore/offshore, some to south	north through channel into entrance, offshore in bigger waves

6.3. Summary of Regional Modeling Results

Based on the combined results of both the historical shoreline change analysis as well as the wave, circulation, and PTM modeling presented above, it is evident that the dominant direction of sediment transport along the shoreline north of Agat SBH is from north to south, both prior to and after construction of the harbor. The harbor is acting as a sediment sink trapping around 188 yd^3 (144 m^3) each year. Transport at Nimitz Beach Park is now from south to north, and some of the sediment is moving offshore through the entrance channel. Prior to the construction of the harbor, the dominant direction of transport was in the opposite direction under prevailing conditions. Therefore, construction of the harbor has likely altered dominant sediment transport direction in this location.

Erosion at Inn on the Bay and the Agat Mayor's Office is due in part to a trend of offshore transport during typical and extreme wave events, caused by wave-generated currents. This may also have been exacerbated in recent years by higher than normal water levels in the western Pacific. The analysis shows that overall, there is a deficit of sediment in the region, and that exploration of offshore and upland sand deposits is warranted in order to replace some of the sand that has been lost.

7. Conceptual Harbor Modifications

7.1. Problems to be Addressed

Concepts for modifications to the existing harbor are developed in the following to address two issues being experienced: 1) Strong currents through the harbor that affect navigation and berthing of vessels, and have caused damages to harbor infrastructure (i.e. – boat slips), and 2) shoaling caused by sediment transported into the harbor, which reduces depths in the channels and results in increased maintenance requirements. In development and evaluation of these modification concepts, it is important to ensure that modifications will not impact navigation in existing channels by causing unacceptable currents at points of constriction, strong wave reflection, increased exposure to incoming waves, or increased harbor resonance (seiching). The results of each proposed modification on currents was evaluated to determine, at a level of detail commensurate with the concept designs, whether or not adverse effects would result.

7.2. Proposed Conceptual Modifications

Based on information from Gov. Guam and harbor users, the federally authorized depths at Agat SBH are adequate. Therefore, modifications (widening and deepening) to the existing entrance channel, access channel or turning basin were not considered in the alternatives. Alternatives considered consisted of additional navigation structures and modifications to the existing main breakwater.

7.2.1. Alternative 1 – No Changes to Existing Conditions

Evaluation of the 'as is' condition was conducted to develop a baseline for comparison of other conceptual modifications. This will be referred to as Alternative 1 – Existing Conditions, and consists of the existing main breakwater, stub breakwater, and revetted mole remaining in place as they currently exist. These structures are illustrated in Figure 44.

Figure 44. Agat SBH Alternative 1 – Existing Conditions

7.2.2. Alternative 2 – Enclose Agat SBH Basin

This concept includes construction of a new breakwater, approximately 480 feet in length, which connects the existing main breakwater to the existing revetted mole (Figure 45). The 225 ft-long spur currently at the north end of the main breakwater would be removed, and the stone reused as part of the new breakwater segment. The purpose of this modification would be to divert current flow entering the harbor from the north, as well as reducing the amount of sediment shoaling.

Figure 45. Agat SBH Alternative 2 – Enclosed Basin

7.2.3. Alternative 3 – Enclose Agat SBH Basin with a Gap

This concept includes construction of a new breakwater, approximately 330 feet in length, which extends from the existing revetted mole toward the main breakwater, but leaves a 150 ft-long gap (Figure 46). The 225 ft-long spur currently at the north end of the main breakwater would be removed, and the stone reused as part of the new breakwater segment. The purpose of this modification would be to divert current flow entering the harbor from the north, as well as reducing the amount of sediment shoaling from this direction, while leaving a small gap to allow for some circulation through the harbor in both directions.

Figure 46. Agat SBH Alternative 3 – Enclosed Basin with Gap

7.2.4. Alternative 4a – Extend Breakwater (300 feet)

This concept includes construction of a new breakwater, approximately 300 feet in length, which extends from the existing main breakwater spur toward the shoreline, but leaves a gap between the structure and the revetted mole (Figure 47). The 225 ft-long spur currently at the north end of the main breakwater would be remain in place. The purpose of this modification would be to divert current flow entering the harbor from the north, as well as reducing the amount of harbor shoaling, while leaving a small gap to allow for some circulation through the harbor in both directions. This concept would also provide additional harbor area that could be used for expansion of the access channel and addition of more boat slips in the future if desired.

7.2.5. Alternative 4b – Extend Breakwater (400 feet)

This concept is similar to Alternative 4a in that it includes construction of a new breakwater, but approximately 400 feet in length, which extends from the existing main breakwater spur toward the shoreline, but leaves a gap between the structure and the revetted mole (Figure 48). The 225 ft-long spur currently at the north end of the main breakwater would remain in place. The purpose of this modification would be to divert current flow entering the harbor from the north, as well as reducing the amount of harbor shoaling, while leaving a small gap to allow for some circulation through the harbor in both directions. The increased length of the breakwater would provide additional wave protection and diversion of

Figure 47. Agat SBH Alternative 4a – Extend Breakwater (300 ft)

Figure 48. Agat SBH Alternative 4b – Extend Breakwater (400 ft)

flow in comparison to Alternative 4a. This concept would also provide additional harbor area that could be used for expansion of the access channel and addition of more boat slips in the future if desired.

7.2.6. Alternative 5 –North Breakwater

This concept includes construction of a north breakwater, approximately 675 feet in length, which extends from the shoreline area north of the harbor, but does not connect to the existing breakwater (Figure 49). The 225 ft-long spur currently at the north end of the main breakwater would be remain in place. The purpose of this modification would be to divert current flow entering the harbor from the north, as well as reducing the amount of sediment shoaling, while leaving a small gap along the western edge of the harbor to allow for some circulation through the harbor in both directions. This concept would also provide additional harbor area that could be used for expansion of the access channel and addition of more boat slips in the future if desired.

Figure 48. Agat SBH Alternative 5 – Construct New Breakwater (675 ft)

7.2.7. Typical Structure Cross-Section

The cross-section of the breakwater modifications is the same for all alternatives, because the exposure to waves and the foundation conditions are the same (or very similar) in all of the alternatives. The as-built cross-section of the existing main breakwater was replicated with one exception – the main breakwater currently consists of a structure with a 1V (vertical):2H (horizontal) side slope on the oceanside of the structure, which is directly exposed to wave attack. The harborside side slope of the existing structure is 1V:1.5H since it is on the backside of the structure and subject to less impact from overtopping waves. Both sides of the proposed structure cross-section contain side slopes of 1V:1.5H because this structure will not be exposed to direct wave attack, and a 1V:1.5H side slope is considered stable in non-breaking wave conditions. This slightly steeper slope will result in a smaller footprint and reduced stone costs. Figure 49 shows the conceptual design cross-section used for evaluating alternatives. Other design elements are the same as the existing breakwater and are as follows.

- Armor Layer: 2 stones thick, 2,000-4,000 pound (lb) stone
- Under Layer: 2 stones thick, 200 -500 lb stone
- Core Material: 5" – 6" stone
- Crest Elevation: 13 ft (MLLW)
- Crest Width: 9 feet
- Shallow foundation built on coral limestone (no toe trench)

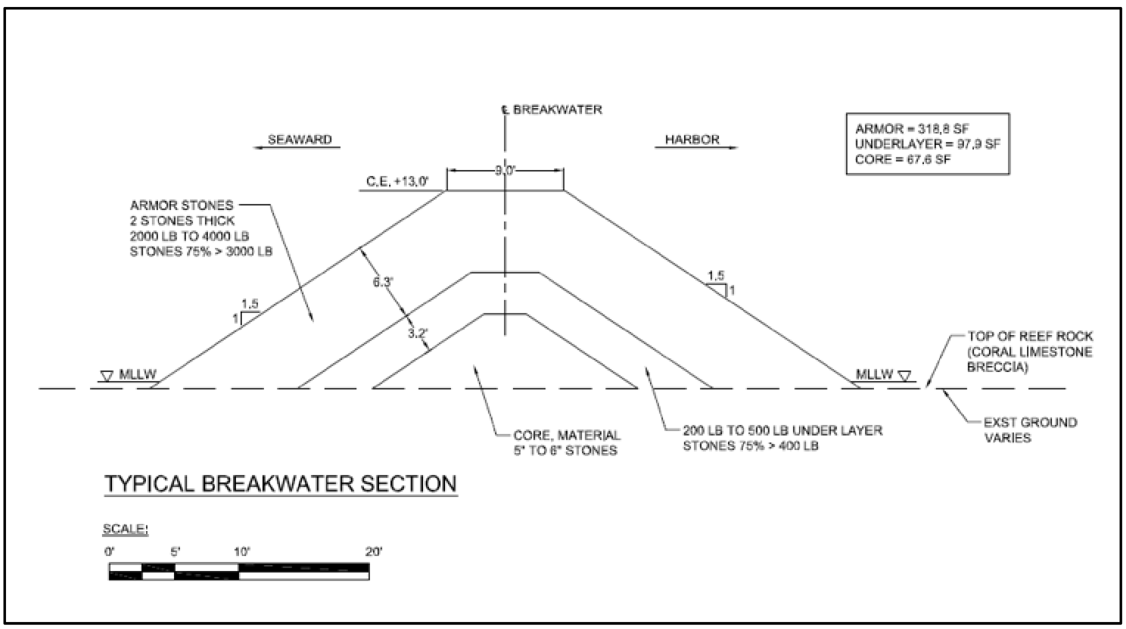

Figure 49. Conceptual Design Cross-Section for All Alternatives

The original breakwater cross-section and armor stone size was based on a depth limited breaking wave height of 6.4 feet on the reef flat fronting the structure, occurring under a tropical storm condition and determined using analytical methods (e.g. - diffraction diagrams and constant wave height to water depth ratio). This structure has been stable, requiring little to no repair since its construction 30 years ago, indicating that this stone size is adequate for the majority of wave conditions encountered. However, to validate that the stone size will be stable, wave modeling was completed using CMS-Wave to transform the 50 year offshore wave (9.4 m/30.8 ft) at the 2% annual exceedance probability water level to the harbor area. This cursory analysis showed a wave of less than 1.0m (3.28 ft) occurring at the structure, which would require less than the original stone size to remain stable. For this stage of concept development, it was decided to maintain the existing stone size for modifications in order to be conservative. This and other design elements should be further evaluated in the detailed design phase, but the existing cross-section serves as a satisfactory estimate for evaluation of costs at the conceptual design stage.

7.3. Comparison of Alternatives

Comparison of conceptual alternatives to evaluate resulting currents through the harbor was completed using the previously developed CMS-Flow and CMS-Wave models in steering mode. Parent and child grids utilized for the regional modeling discussed previously were used for these steering simulations, with child grids modified to reflect bathymetry changes at the harbor for each alternative. Use of these models ensures that both tide and wave-generated currents are included in the comparison of alternatives to the existing condition. Both the prevailing and annual wave conditions were simulated, in order to include varying wave direction (WNW and SW, respectively), and varying flow directions observed in the harbor (as noted from simulations of the existing condition and shown in Figures 32 and 34). The calm wave condition was assumed to have similar but lower magnitude results as these conditions. The current fields shown, as with those shown previously, are 'snapshots' in time (same time for all alternatives) and are selected to illustrate the most energetic conditions over the three day simulation for comparison.

7.3.1. Harbor Currents for Alternative 2: Enclose Agat SBH Basin

Enclosing the north side of the breakwater by connecting the existing detached breakwater to land would be the most obvious alternative to reduce flow through the harbor. This would divert nearshore currents coming from north of the harbor toward the offshore (under the prevailing wave condition), and would drastically reduce the currents inside the harbor to almost zero, as shown in Figure 50. By comparison, in the existing condition for the prevailing wave case (H_s= 1.5m in deep water), the maximum velocity that occurs in the gap between the breakwater and the revetted mole is about 2.9 m/s. The diverted flow in this wave condition under Alternative 2 increases velocities on the oceanside of the existing breakwater, moving parallel to the structure toward the entrance channel. This could create a cross-current at the entrance channel under prevailing wave conditions. In addition, this modification would reduce harbor shoaling by interrupting any sediment transport from the north, but this may also lead to impoundment of sediment along the new breakwater root. It also appears that this modification increases the magnitude of current and its direction toward the shoreline to the north of the harbor, when compared to Figure 32.

This alternative concept was also modeled for the annual wave condition (waves from SW), since a reversal of current direction (toward north) was observed in the modeling of this condition with the existing harbor configuration (refer back to Figure 34). A representative current velocity field is shown in Figure 51. For this condition, the modification to the harbor interrupts the northward flow, and currents north of the harbor are directed offshore.

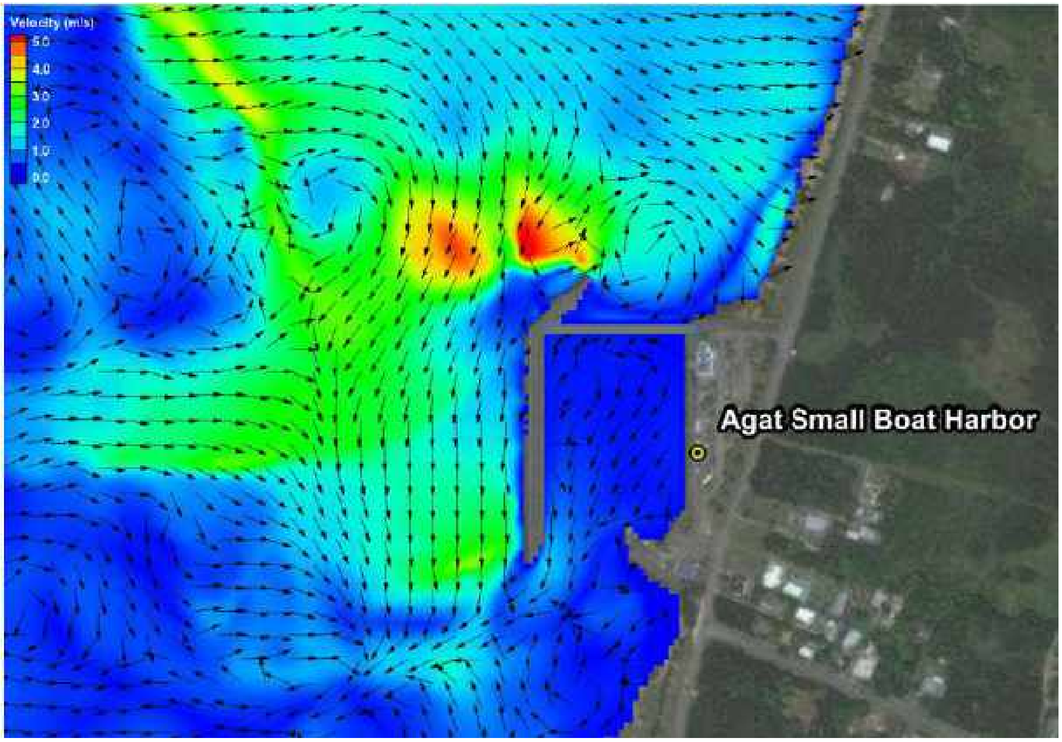

Figure 50. Alt 2 Current velocity field for prevailing wave condition - 8/27/11 at 9:30 am (GMT)

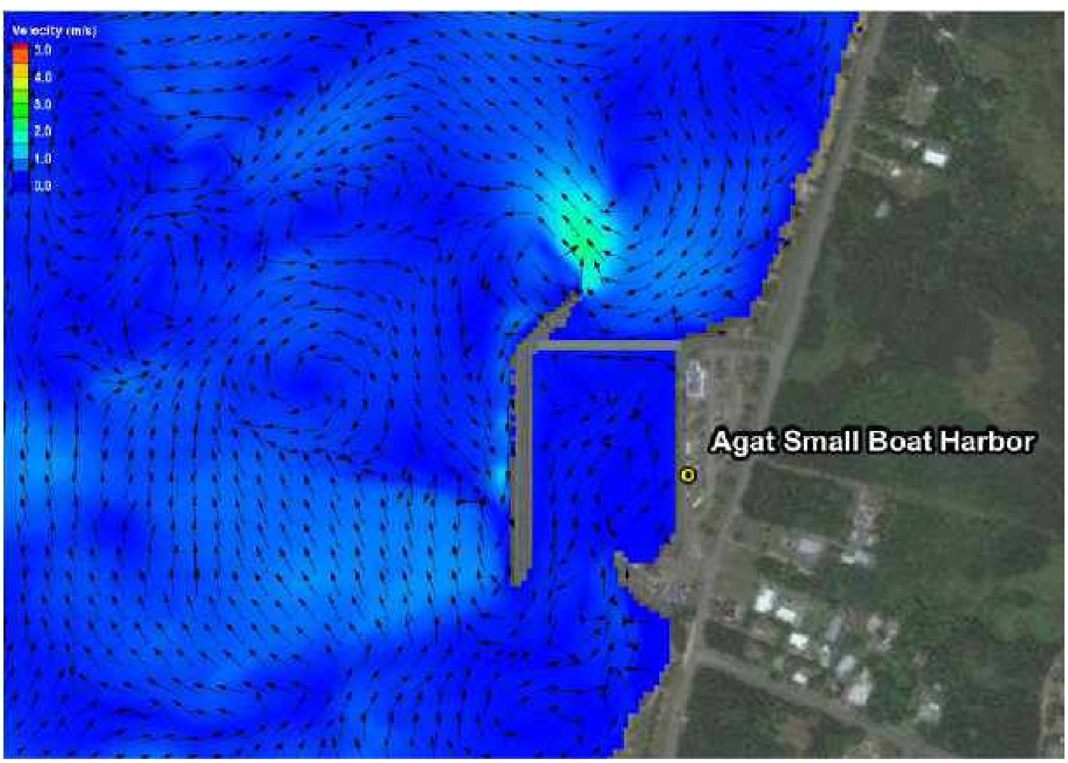

Figure 51. Alt 2 Current velocity field for annual wave condition - 7/30/09 at 12:00pm (GMT)

7.3.2. Harbor Currents for Alternative 3: Enclose Agat SBH Basin with Gap

Alternative 3 (enclosing the harbor basin but leaving a small gap between the existing and new structures) was simulated under the prevailing condition as shown in Figure 52. Similar to Alternative 2, this would divert nearshore currents coming from the north of the harbor toward the offshore. However, flow through the gap appears to focus flow through the harbor, increasing the velocity in the harbor compared to the existing condition, up to a range of 3 to 5 m/s, which is not acceptable for safe navigation or operations. The diverted flow in this wave condition again increases velocities on the oceanside of the existing breakwater, moving parallel with the structure toward the entrance channel. This could create a cross-current at the entrance channel under prevailing wave conditions. In addition, this modification would reduce harbor shoaling by interrupting any sediment transport from the north, but this may also lead to impoundment of sediment along the new breakwater root.

This alternative concept was also modeled for the annual wave condition (waves from SW). A representative current velocity field is shown in Figure 53. With the new structure blocking the majority of the northward flow out of the harbor, flow direction reverses. There is a moderate constriction of flow through the gap that causes higher velocities to develop, similar in direction to the prevailing wave condition.

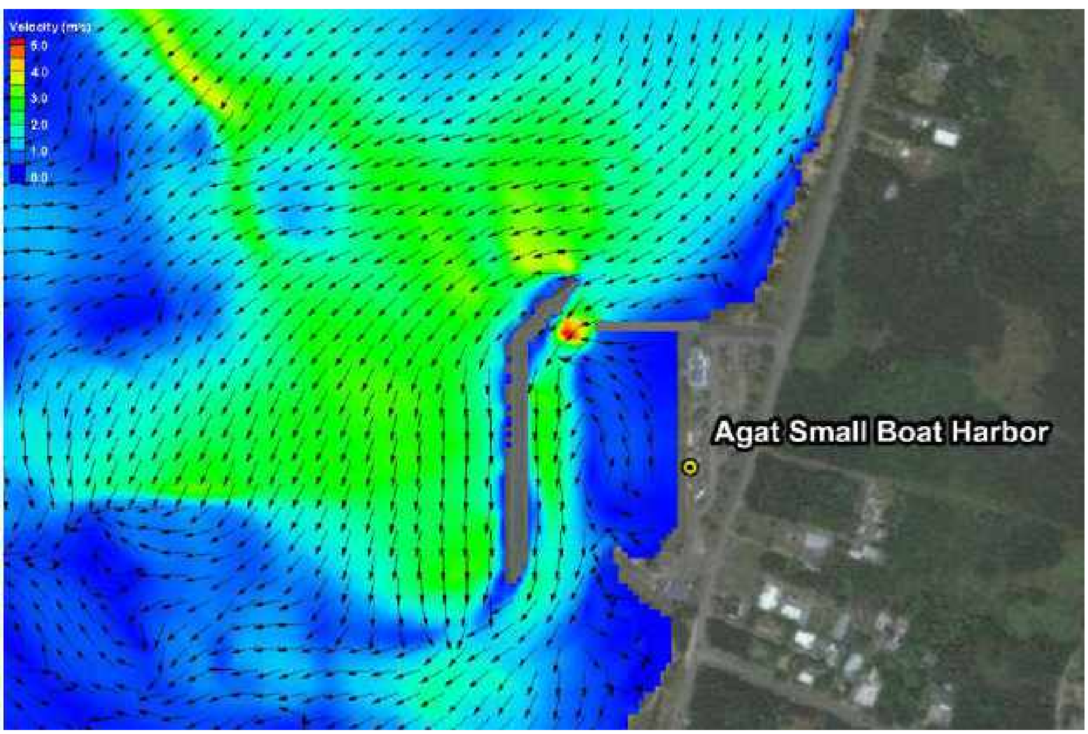

Figure 52. Alt 3 Current velocity field for prevailing wave condition - 8/27/11 at 9:30 am (GMT)

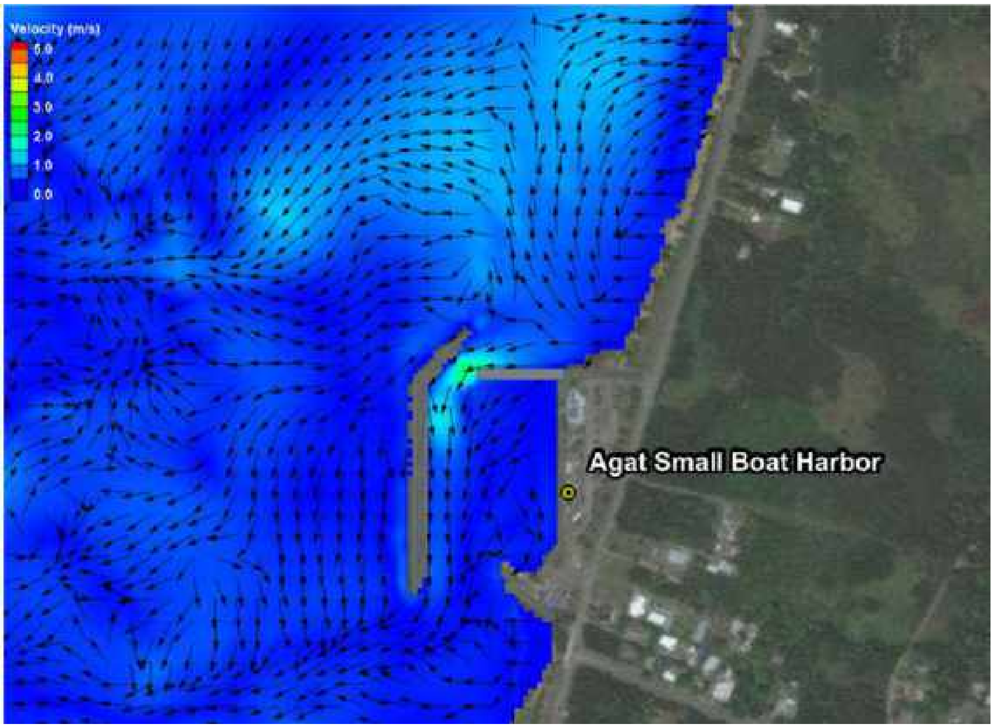

Figure 53. Alt 3 Current velocity field for annual wave condition - 7/30/09 at 12:00pm (GMT)

7.3.3. Harbor Currents for Alternative 4a: Extend Breakwater (300 feet)

Alternative 4a (extending existing breakwater landward 300 feet but leaving a gap between the existing revetted mole and new structures) was simulated under the prevailing condition as shown in Figure 54. Similar to Alternatives 2 and 3, this would divert nearshore currents coming from the north of the harbor toward the offshore. However, flow through the gap reaches a maximum velocity 4.8 m/s under this condition due to the constriction of flow, which is not acceptable for safe navigation or operations. The diverted flow in this wave condition again increases velocities on the oceanside of the existing breakwater, moving parallel with the structure toward the entrance channel. This could create a cross-current at the entrance channel under prevailing wave conditions. This modification may reduce harbor shoaling by interrupting any sediment transport from the north, but there is still the possibility of sediment coming through the gap near the shoreline.

This alternative concept was also modeled for the annual wave condition (waves from SW). A representative current velocity field is shown in Figure 55. With the new structure blocking the majority of the potential northward flow out of the harbor, flow direction to the north of the harbor reverses toward the south. There is a moderate constriction of flow through the gap that causes higher velocities to develop, similar in direction to the prevailing wave condition.

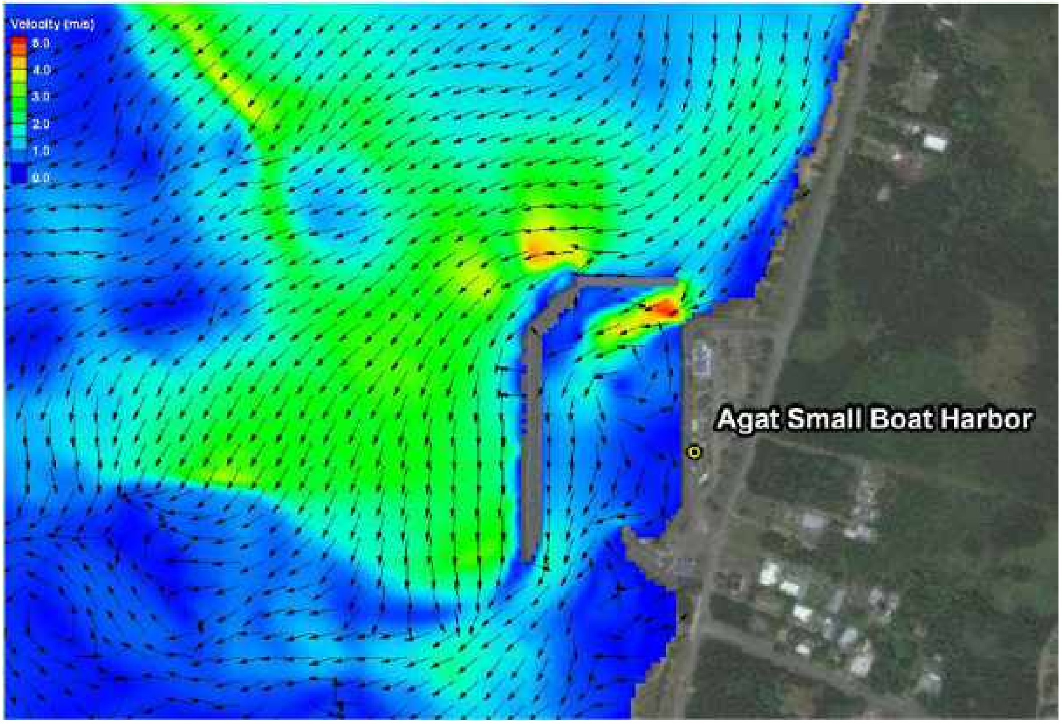

Figure 54. Alt 4a Current velocity field for prevailing wave condition - 8/27/11 at 9:30 am (GMT)

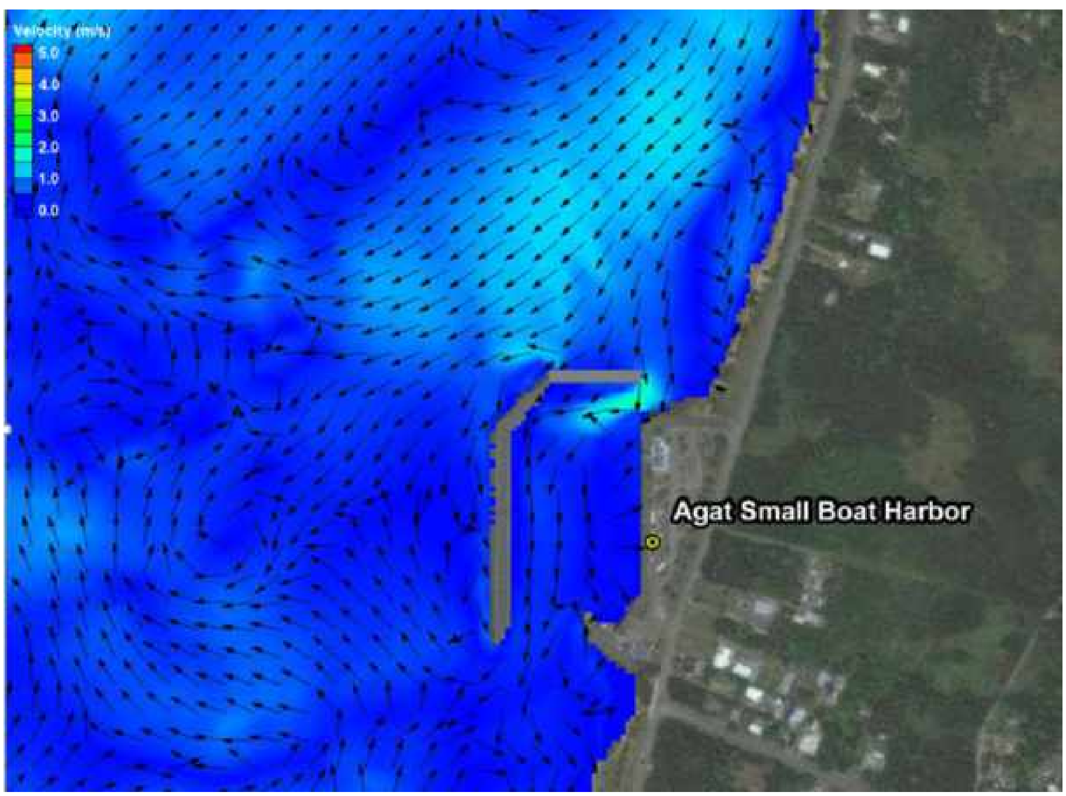

Figure 55. Alt 4a Current velocity field for annual wave condition - 7/30/09 at 12:00pm (GMT)

7.3.4. Harbor Currents for Alternative 4b: Extend Breakwater (400 feet)

Alternative 4b (extending existing breakwater landward 400 feet but leaving a gap between the existing revetted mole and new structures) was simulated under the prevailing condition as shown in Figure 56. Similar to previous alternatives, this would divert nearshore currents coming from the north of the harbor toward the offshore. However, similar to Alternative 4a, flow through the gap reaches a maximum velocity 4.8 m/s under this condition due to the constriction of flow, which is not acceptable for safe navigation or operations. The diverted flow in this wave condition again increases velocities on the oceanside of the existing breakwater, moving parallel with the structure toward the entrance channel. This could potentially create a cross-current at the entrance channel under prevailing wave conditions. This modification may reduce harbor shoaling by interrupting any sediment transport from the north, but there is still the possibility of sediment coming through the gap near the shoreline.

This alternative concept was also modeled for the annual wave condition (waves from SW). A representative current velocity field is shown in Figure 57. With the new structure blocking the majority of the potential northward flow out of the harbor, flow direction to the north of the harbor reverses toward the south. There is a moderate constriction of flow through the gap that causes higher velocities to develop, similar in direction to the prevailing wave condition.

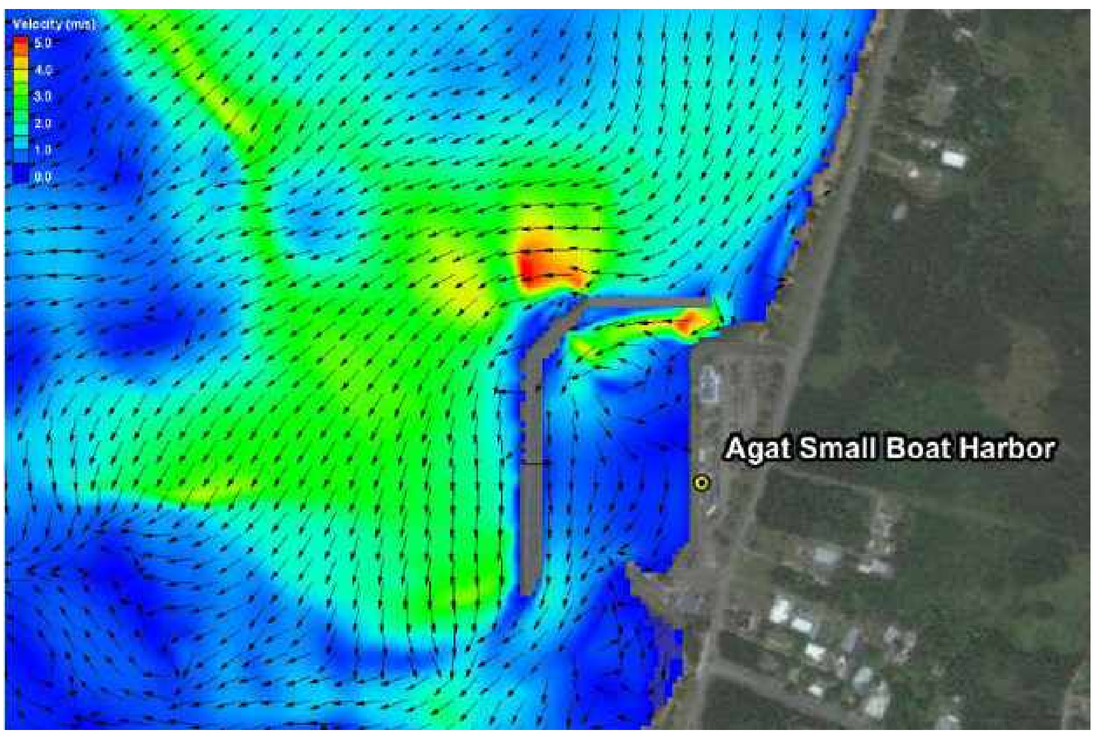

Figure 56. Alt 4b Current velocity field for prevailing wave condition - 8/27/11 at 9:30 am (GMT)

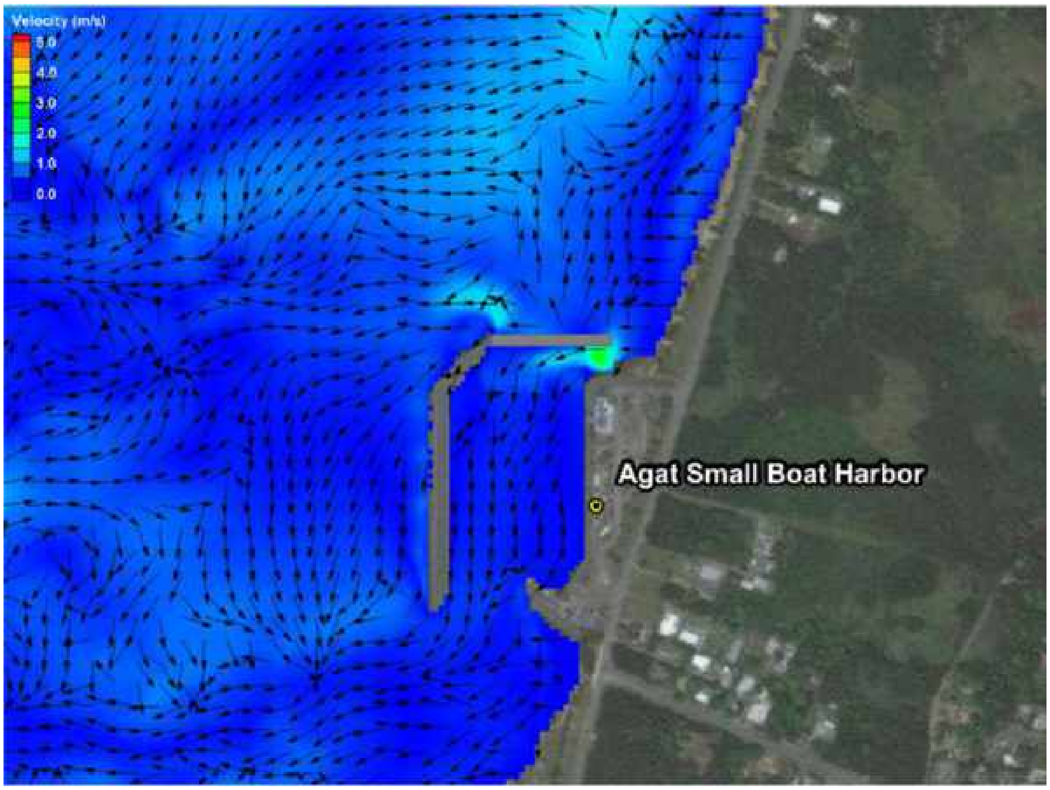

Figure 57. Alt 4b Current velocity field for annual wave condition - 7/30/09 at 12:00pm (GMT)

7.3.5. Harbor Currents for Alternative 5: New North Breakwater

Alternative 5 (new breakwater north of harbor leaving a gap between existing breakwater) was simulated under the prevailing condition as shown in Figure 58. Similar to previous alternatives, this would divert nearshore currents coming from the north of the harbor, though in this case an eddy of significant velocity is formed to the north of the new structure. The flow through the gap between new and existing structures is comparatively minimal at about 0.9 m/s. The diverted flow in this wave condition does not increase velocities on the oceanside of the existing breakwater. There is still flow moving out the harbor entrance at the constriction between the stub breakwater and the main breakwater. This modification should reduce harbor shoaling by interrupting any sediment transport from the north, but this may also lead to impoundment of sediment along the new breakwater root.

A representative current velocity field for the annual condition is shown in Figure 59. With the new structure blocking the majority of the northward flow out of the harbor, current direction to the north of the harbor is reversed (now coming from north) in comparison with the existing condition. There is a small constriction of flow exiting the harbor due to the gap between the new breakwater and the existing main breakwater, but because in general, flow velocities are small for this condition, it is about 1.35 m/s, which may be acceptable.

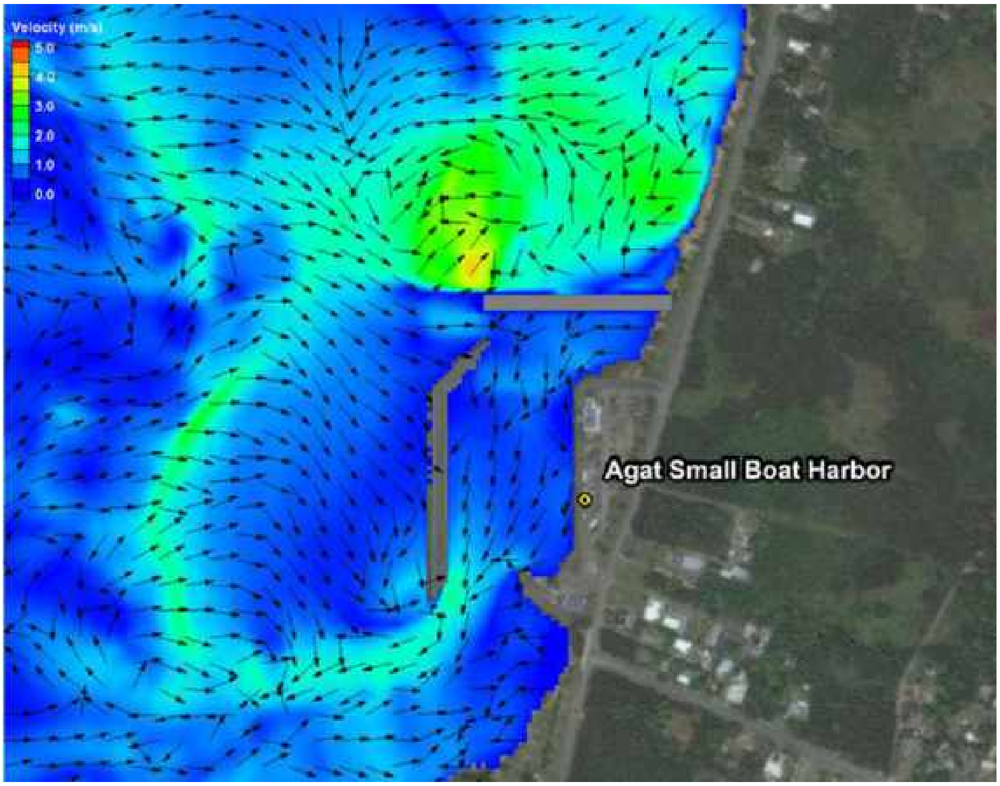

Figure 58. Alt 5 Current velocity field for prevailing wave condition - 8/27/11 at 9:30 am (GMT)

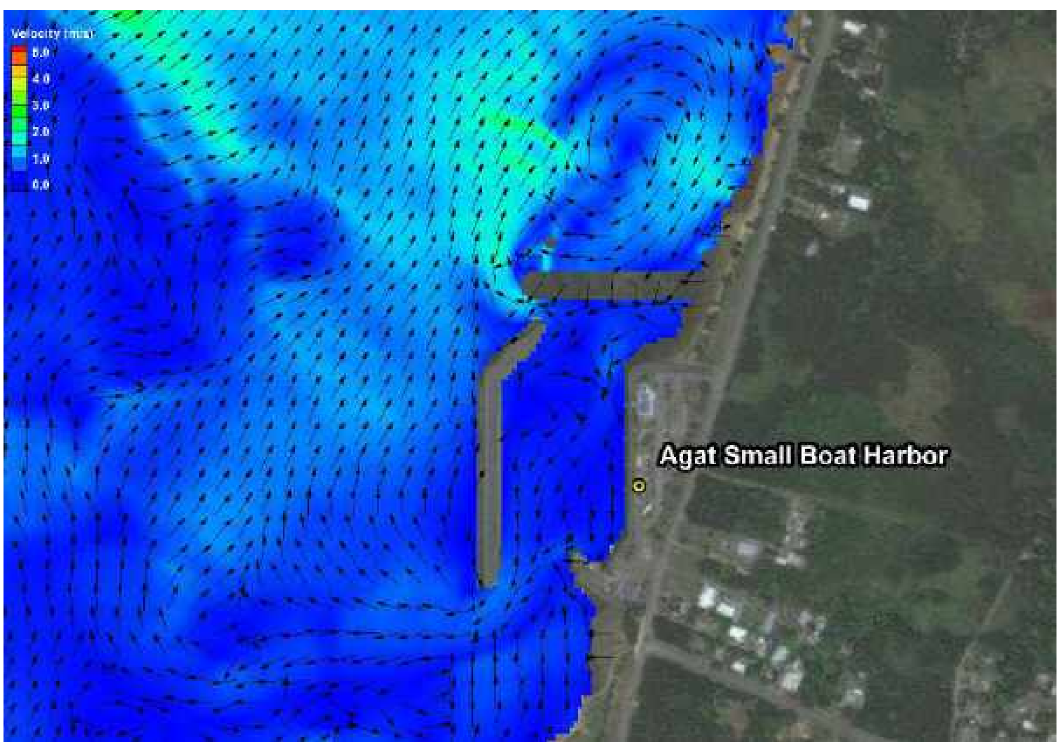

Figure 59. Alt 5 Current velocity field for annual wave condition - 7/30/09 at 12:00pm (GMT)

7.3.6. Summary of Alternatives Comparison for Currents

Analysis of CMS-Flow current velocity fields for the various alternatives under both prevailing and annual wave event conditions provides a general understanding of how each concept will alter current velocity at and adjacent to Agat SBH. Examination of these results indicate that several of the alternatives may result in velocities within the harbor that are unacceptably high during prevailing wave conditions, and to a lesser extent during the annual wave condition. For this reason, alternatives that include a gap along the northern edge of the harbor basin (Alternatives 3, 4a, and 4b) have been removed from further consideration. Alternative 2, enclosing the harbor basin, is a viable option, but does increase currents outside the existing main breakwater, and has the potential to impact navigation in the entrance channel. Alternative 5, building a new breakwater to the north of the harbor that extends slightly beyond the main breakwater, is also a concept that should be brought forward for further consideration. Both of these alternatives successfully reduce currents through the harbor, and in turn will reduce harbor shoaling, but do not have significant detrimental effects to navigation under the conditions simulated.

7.4. Comparison of Alternatives – Wave Height

The phase-averaged wave transformation model used to evaluate the study area shows that minimal wave energy reaches the harbor, due to dissipation over the wide and shallow reef. This is true even for large offshore waves, and associated elevated water levels. Based on this information, it is not likely that harbor modifications will have significant effect in increasing wave heights within the harbor, or cause harbor seiching due to resonance. However, evaluation of changes to wave height in the harbor due to new structures, as well as potential harbor resonance, should be evaluated with a phase-resolving wave model that includes physics-based simulation of processes such as wave reflection and diffraction. This should be conducted for Alternative 2 and Alternative 5 in the detailed design phase, and modifications can be made to these alternatives if model results warrant.

7.5. ROM Cost Estimates for Agat Harbor Modification Concepts

A Rough Order of Magnitude (ROM) construction cost estimate was developed for each alternative (other than Alternative 1), based on the following assumptions. ROM cost estimates are shown in Table 8 below.

<u>ROM Cost Assumptions:</u>

- Work to be performed by Guam contractors and labor force. Therefore, mobilization and demobilization were based on local conditions. Guam labor force was used, based labor rates provided by Bureau of Labor Statistics (May 2017) adjusted to 2019.
  - <u>Note</u>: Mobilization and demobilization costs would increase significantly for a contractor outside of Guam. Bidding climate should be evaluated closely prior to contract solicitation.
- Assumes all materials are furnished from Guam (assumes 2% sales tax).

- Assumes no UXO (Unexploded Ordinance) since small footprint area and no excavation is scoped.
- Does not include environmental mitigation.
- Material is assumed to be similar in features to limestone characteristics
- Assumes 5-10% voids in neat line area for placement material and 10% waste and overbuild.
- Field Office Overhead = 25%, Home Office Overhead = 10%, Profit = 10%, Bond = 2%, Price Level = 2019
- Pre-Construction Engineering/Design costs not included
- Construction Supervision and Administration costs not included
- Site Access: Assume staging area at Agat Harbor.
- Borrow Areas: Assumed to be available on Guam for all material including core, under layer and armor stone. Assumes reuse of armor stone from demolished spur.
- Unusual Conditions (Soil, Water, and Weather): Open water excavation and placement.
- Weather Days: Not specifically considered for this level of estimate.
- Equipment and Labor Availability and Distance Traveled: Equipment and labor availability is a concern based upon the remote location of Guam. The assumption being that contractors with the equipment and labor resources necessary can be found on Guam or nearest port Saipan.
- No environmental coordination or permitting is included.
- No real estate costs are included.
- **<u>Contingency of 35% is included</u>** at this conceptual stage based on the lack of detailed technical information and scope clarity resulting in major estimate assumptions in technical information and quantities, heavy reliance on cost engineering judgment, cost book, parametric, historical, and little specific crew-based costs.

Table 8. ROM Cost Estimates for Agat Harbor Modification Concepts

Summary
Agat Small Harbor Alternative Level Costs
(Contractor within Guam)

	Description	Quantity	UOM	Unit Cost**	Project Cost
2	**Enclosed Basin**				$ 7,703,000
	Mob, Demob & Preparatory Work (Assume 60 days)	1	LS	$ 2,353,500	$ 2,353,500
	Demolition of Existing Breakwater Spur	3,701	CY	$ 90	$ 332,200
	Core Stone (5" to 6")	1,256	CY	$ 382	$ 479,900
	Underlayer Stone (200-500 lb)	1,819	CY	$ 416	$ 757,300
	Armor Stones (2000 - 4000 lb)	9,062	TONS	$ 330	$ 2,993,700
	Associated General Items	1	LS	$ 786,400	$ 786,400
3	**Enclosed Basin with Gap**				$ 6,099,300
	Mob, Demob & Preparatory Work (Assume 60 days)	1	LS	$ 2,353,500	$ 2,353,500
	Demolition of Existing Breakwater Spur	3,701	CY	$ 90	$ 332,200
	Core Stone (5" to 6")	863	CY	$ 382	$ 329,800
	Underlayer Stone (200-500 lb)	1,250	CY	$ 416	$ 520,600
	Armor Stones (2000 - 4000 lb)	6,230	TONS	$ 285 *	$ 1,776,800
	Associated General Items	1	LS	$ 786,400	$ 786,400
4a	**Extend Breakwater (short)**				$ 6,347,100
	Mob, Demob & Preparatory Work (Assume 60 days)	1	LS	$ 2,353,500	$ 2,353,500
	Core Stone (5" to 6")	785	CY	$ 382	$ 300,000
	Underlayer Stone (200-500 lb)	1,137	CY	$ 416	$ 473,400
	Armor Stones (2000 - 4000 lb)	5,663	TONS	$ 430	$ 2,433,800
	Associated General Items	1	LS	$ 786,400	$ 786,400
4b	**Extend Breakwater (long)**				$ 7,416,236
	Mob, Demob & Preparatory Work (Assume 60 days)	1	LS	$ 2,353,500	$ 2,353,500
	Core Stone (5" to 6")	1,047	CY	$ 382	$ 399,900
	Underlayer Stone (200-500 lb)	1,516	CY	$ 416	$ 631,236
	Armor Stones (2000 - 4000 lb)	7,551	TONS	$ 430	$ 3,245,200
	Associated General Items	1	LS	$ 786,400	$ 786,400
5	**New North Breakwater**				$ 10,637,600
	Mob, Demob & Preparatory Work (Assume 60 days)	1	LS	$ 2,353,500	$ 2,353,500
	Core Stone (5" to 6")	1,766	CY	$ 382	$ 674,800
	Underlayer Stone (200-500 lb)	2,558	CY	$ 416	$ 1,065,200
	Armor Stones (2000 - 4000 lb)	12,743	TONS	$ 430	$ 5,476,500
	Associated General Items	1	LS	$ 1,067,600	$ 1,067,600
	* Unit Cost based on reuse of demolished Spur armor stone (approx. 3,476 Tons).				
	** Costs include contingency (35%). Does not include mitigation, lands and damages, E&D, or S&A costs.				

8. Areas of Concern Due to Erosion

As identified previously, there are three priority areas of concern that have been impacted by ongoing erosion: Inn on the Bay, Agat Mayor's Office, and Nimitz Beach Park. The following sections will review of the erosion issues at each site, conclusions from the analysis discussed above, an overview of shoreline stabilization measures, and conceptual plans to address the concerns of each location including ROM cost estimates.

At the Inn on the Bay, the numerical modeling results were generally in agreement showing that currents are moving sediment offshore and to the north. This offshore directed sediment transport is likely causing the erosion observed at this location. It is also evident from analysis of historical imagery that the shoreline has notably eroded directly in front of this property, particularly in the past 25 years. A cemented rock wall has been constructed along the shore-facing side of the property in an attempt to stop the erosion and protect the inn (Figure 8). There is no dry beach at any tide level in the northern section of the property, but a narrow beach fronts the wall to the south. A combination of the continued sediment erosion and wave impacts (e.g. overtopping), in addition to the shallow toe foundation of the wall, have led to some undermining and erosion of backfill that has been filled with grout. As these processes continue the wall may ultimately fail, which would lead to damages to the property, including loss of palm trees and undermining of the foundation of the building structure.

At the Agat Mayor's Office, the prevailing conditions indicate that a northward directed nearshore flow in front of the Mayor's Office converges with a southward directed flow further up the shoreline, with the combination of flow directions moving offshore. This north/south/offshore flow is also observed in the PTM modeling and correlates to the trends observed in the historical shoreline change analysis. The location of convergence is also consistent with a small salient that has formed just to the north of the Mayor's Office. This north and offshore flow, in combination with a littoral barrier formed by the rocky headland to the south of the Mayor's Office, is likely transporting some of the sand out of the littoral system and contributing to the erosion issues at this location. The Agat Mayor's Office and the adjacent property of the Sagan Bisita are protected by a vertical concrete rock masonry (CRM) seawall. Though a narrow beach still exists here, continued erosion could lead to undermining of the wall and damage to the buildings and facilities behind it.

The third priority area is at Nimitz Beach Park, directly to the south of Agat SBH. Numerical modeling results indicate that transport along the park shoreline is mostly consistent toward the north, which is in agreement with shoreline change analysis showing erosion at the southern end of the park and accretion at the northern end of the park. The PTM results showed that there is a consistent sediment pathway through a small channel joining the park shoreline to the outer entrance channel of Agat SBH. Sediment may either be transported offshore or into the harbor depending on the wave conditions. In the southern portion of the park, broken cement slabs are evidence of the impacts of the erosion. As this erosion due to northerly sediment transport continues, other park infrastructure may soon be threatened also.

9. Shoreline Stabilization Measures Overview

There are many engineering measures that can help to stabilize shorelines that are impacted by erosion. These measures can be represented on a scale of “green” to “gray”, where green indicates softer and more natural approaches (e.g. vegetation and living shorelines) and gray measures are hard structures (e.g. revetments and seawalls) (Figure 60). Different measures can also be combined based on the needs of each situation. There are many factors to consider when determining which measure(s) would be best for a given situation, including rate of erosion, typical wave conditions, and existing backshore features (e.g. building vs. open space).

In Guam, prior to the implementation of any shoreline stabilization measure, the policies and guidelines of the Territorial Seashore Protection Act and the Guam Comprehensive Development Plan should be taken into consideration, including the following:

> Shore Area Development: Only those uses shall be located within the Seashore Reserve which: (1) enhance, are compatible with or do not generally detract from the surrounding coastal area's aesthetic and environmental quality and beach accessibility; or (2) can demonstrate dependence on such a location and the lack of feasible alternative sites.
>
> Visual Quality: Preservation and enhancement of, and respect for the island's scenic resources shall be encouraged through increased enforcement of and compliance with sign, litter, zoning, subdivision, building and related land-use laws; visually objectionable uses shall be located to the maximum extent practicable, so as not to degrade significantly views from scenic overlooks, highways, and trails.

Adapted from the guidelines developed by a Systems Approach to Geomorphic Engineering (SAGE), the subsections below summarize several green to gray measures, including advantages, disadvantages, relative cost, level of maintenance required, and environmental considerations (SAGE 2015).

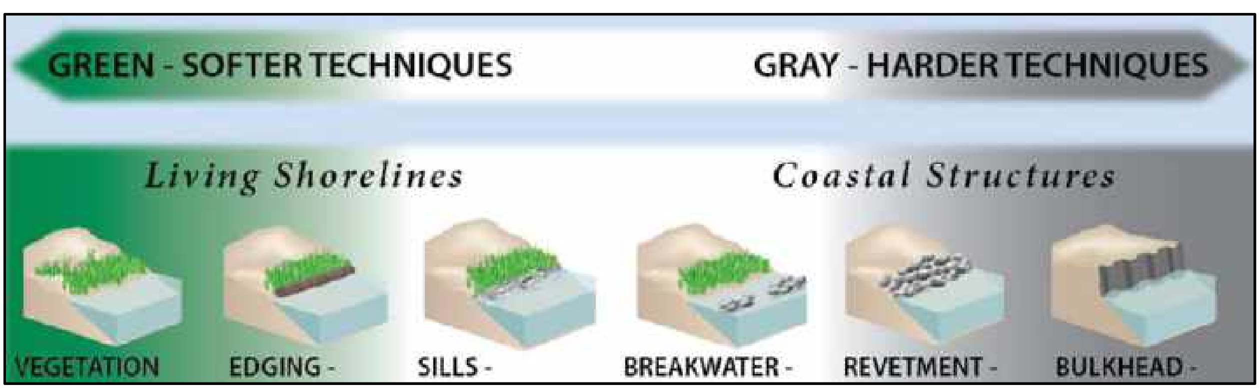

Figure 60. Categories of shore protection ranging from “green” softer to “gray” harder measures.

9.1. Vegetation Only

This measure includes only vegetation (native plants), with the intent that roots of the plants will hold soil in place to reduce erosion due to wave action (Figure 61). It provides a buffer to upland areas, breaks small waves and is suitable for low wave energy environments (UH & OCCL 2004). A wave decay study conducted in the laboratory simulating the effect of vegetation density on wave dissipation showed a decrease in wave energy with increasing stem density (Figure 62) (Anderson et al. 2013).

The benefits of vegetation include the fact that it dissipates wave energy, slows inland water transfer, increases natural stormwater infiltration, provides habitat and ecosystem services, exerts minimal impact to natural community and ecosystem processes, maintains aquatic/terrestrial interface and connectivity, and assists in flood water storage. This measure is also relatively low in cost, and can be implemented with minimal equipment and labor. The disadvantages include no storm surge reduction ability, no high water protection, it is only appropriate in limited situations (not effective in high energy environments), there is uncertainty of successful vegetation growth, and competition with invasive plant species. Environmental considerations include continual maintenance of vegetation, minor environmental impacts, and permits may or may not be required.

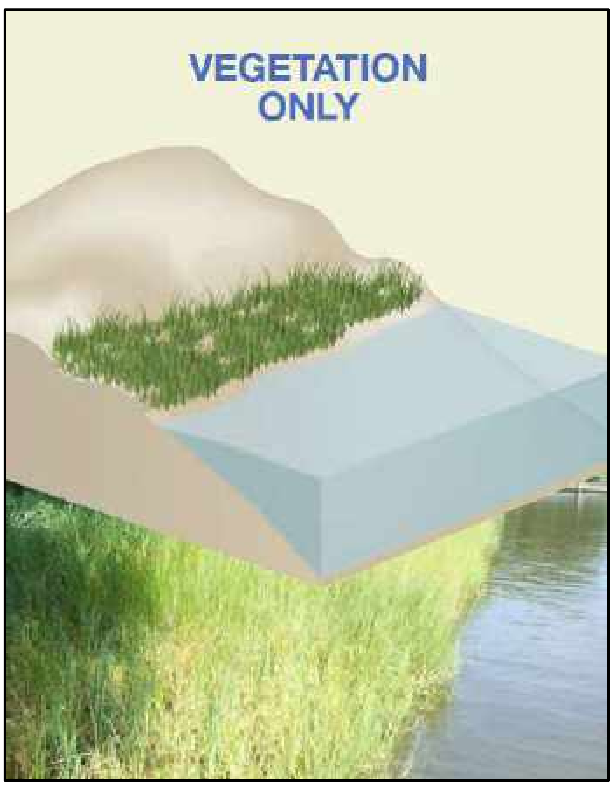

Figure 61. Vegetation only measure.

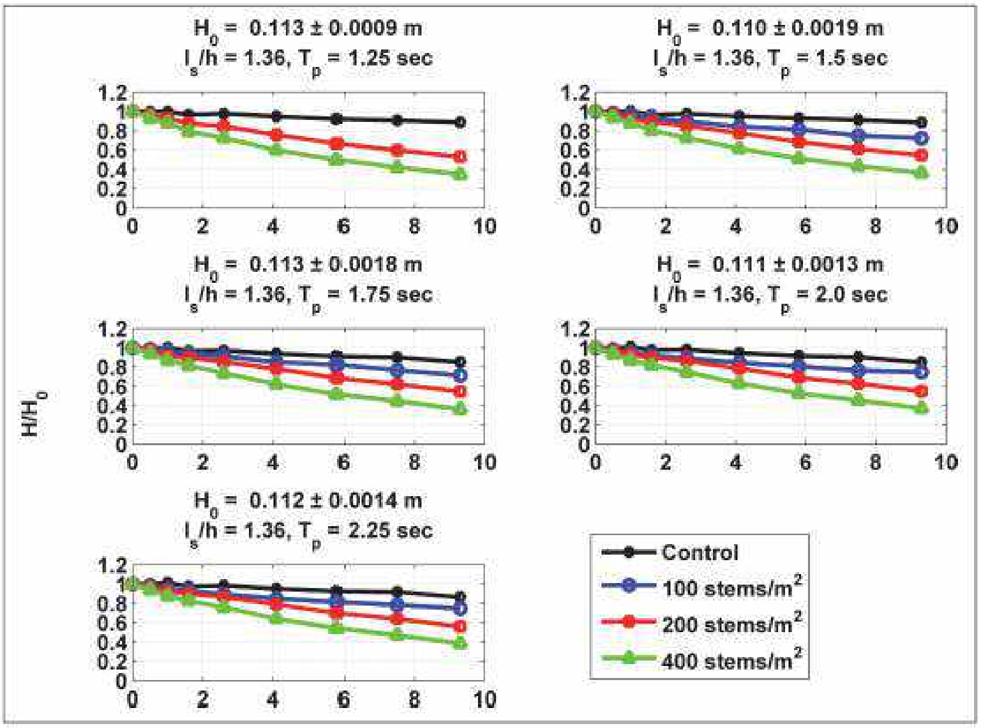

Figure 62. Effect of stem density on wave decay using artificial vegetation where H_o is the initial wave height and H is the wave height at various locations along the wave tank (horizontal axis).

9.2. Edging

The edging measure is the same as the vegetation only alternative, except a structure is added to hold the toe of the existing or vegetated slope in place (Figure 62). It can reduce shoreline erosion and is suitable in low wave energy environments. Edging materials include snow fencing, erosion control blankets, geotextile tubes, living reef (oyster/mussel) and rock gabion baskets. Native plants and materials must be appropriate for salinity levels and site conditions. Similar to vegetation only, the advantages include dissipation of wave energy, slowing down of inland water transfer, provision of habitat and ecosystem services, increase in natural stormwater infiltration, and toe protection helps prevent wetland edge loss. It is also relatively low in cost. The disadvantages are that there is no high water protection, it is a larger physical footprint, there is uncertainty of successful vegetation growth, and competition with invasive plant species. Environmental considerations include continual maintenance of vegetation, minor environmental impacts, and permits may or may not be required.

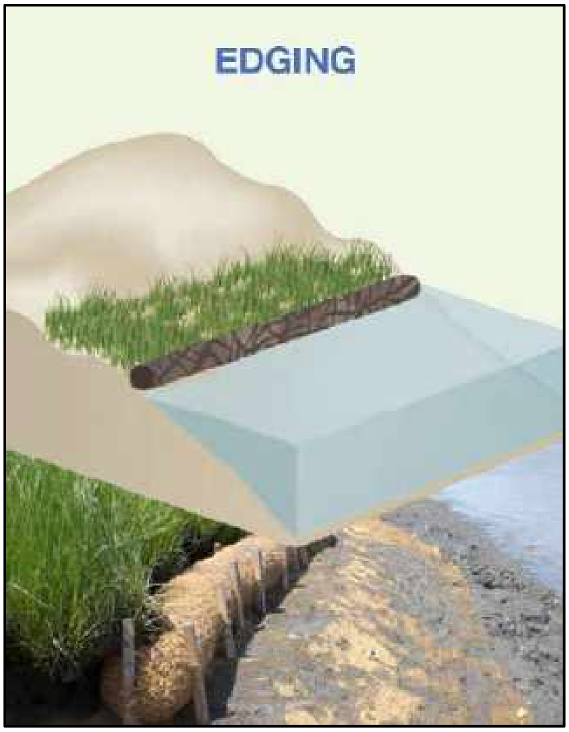

Figure 63. Vegetation with edging measure.

9.3. Sill

A sill is constructed parallel to existing or vegetated shoreline, reduces wave energy and prevents erosion (Figure 63). A gapped sill approach allows habitat connectivity, greater tidal exchange, and better waterfront access. It can reduce shoreline erosion and is suitable in low wave energy environments. Sill materials include stone, sand bars, living reef (oyster/mussel) and rock gabion baskets. Advantages of sills include provision of habitat and ecosystem services, dissipation of wave energy, slowing down of inland water transfer, an increase in natural stormwater infiltration, and toe protection that helps to prevent wetland edge loss. It is relatively low in cost, but does require some construction equipment. Disadvantages include increased land area requirement, no high water protection, uncertainty of successful vegetation growth, and competition with invasive plant species. Environmental considerations include continual maintenance of vegetation, a larger physical footprint, limited environmental impacts, and permitting is required.

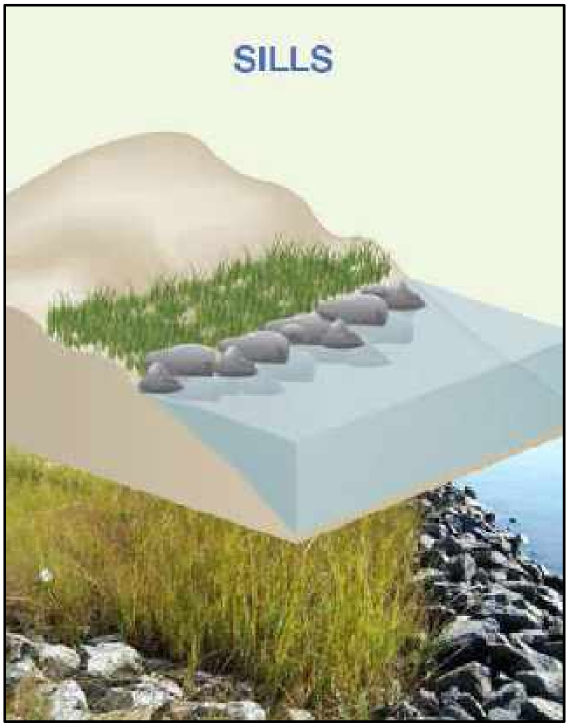

Figure 63. Vegetation with sill measure.

9.4. Beach Nourishment with and without Vegetation on Dune

For this measure, beach quality sand is added from an adjacent or outside source to nourish an eroding beach (Figure 64). Such nourishment widens the beach and extends the shoreline seaward. Beach nourishment is an effective means of erosion protection in low-lying oceanfront areas with available sources of beach quality sand or other native sediments. Vegetated dunes help anchor sand and provide a buffer to protect inland areas from waves, flooding and erosion. Dunes can be strengthened by inclusion of a geotextile tube or rock core. Advantages include the expansion of usable beach area, lower environmental impact than hard structures, flexibility, and ease of redesign along with provision of habitat and ecosystem services. Vegetation can be planted on the dune to increase its resilience to storm events. The cost of this measure is moderate, and depends on quantity and supply of sand. Disadvantages include continual sand renourishment required in the absence of terminal structures, limited high water protection, application is limited to locations with a sustainable sand source, and there are possible impacts to regional sediment transport. Environmental considerations include large physical footprint requirement, moderate environmental impact, impacts may be reversible, and permitting is required.

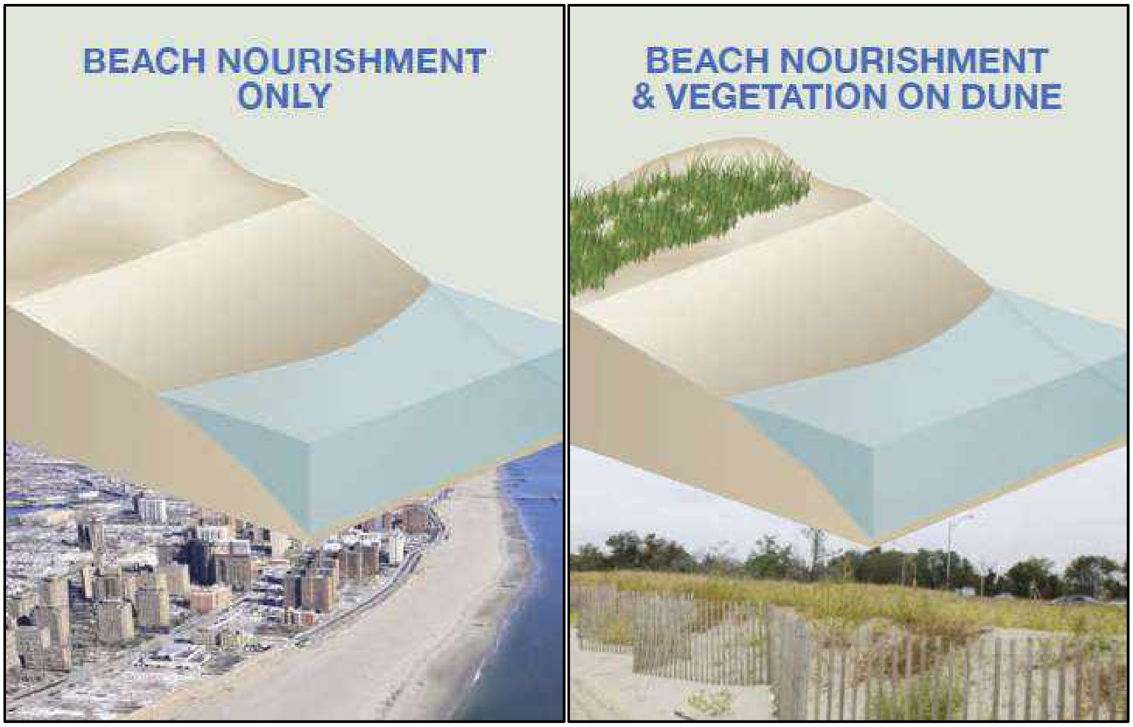

Figure 64. Beach nourishment with and without dune vegetation measure.

9.5. Breakwater

A breakwater is an offshore structure intended to break waves, reduce the force of wave action and encourage sediment accretion (Figure 65). A breakwater can be floating or placed on the ocean floor, attached to shore or not, and continuous or segmented. A segmented breakwater allows habitat connectivity, greater tidal exchange, and better waterfront access. Breakwaters can be designed for high wave energy environments and are often utilized in conjunction with ports, harbors and marinas. A breakwater can be made of rock, grout-filled bags, wood, concrete armor units or living reef (oyster/mussel) in low wave environments. Artificial reefs can also provide a similar wave breaking function afforded by breakwaters. Advantages include reduction of wave force and height, stabilization of a wetland, ability to function like reefs in some cases, economically feasible in shallow areas, and moderate storm surge flood level reduction ability. The cost of this measure is typically moderate to high, depending on site conditions and availability of materials. Disadvantages include the fact that they are expensive to construct in deep water, can reduce water circulation (minimal for floating breakwater), potential navigational hazards (if detached), and may require a large footprint. There can be significant environmental impact in and out of water, the impacts are not reversible, there is minimal maintenance, and permits are required.

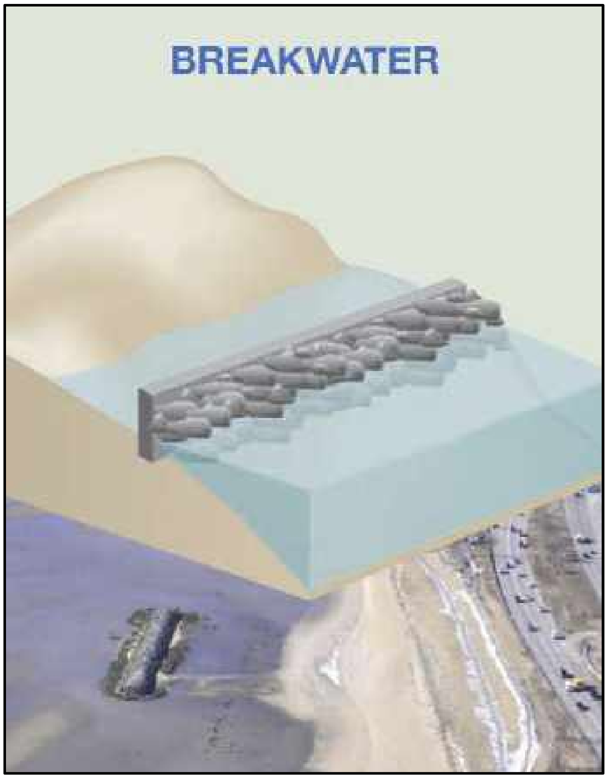

Figure 65. Breakwater measure.

9.6. Revetment

A revetment consists of armoring a shoreline slope with the intent of maintaining shoreline position and protecting the area landward from wave impacts and erosion (Figure 66). A revetment is suitable in areas of pre-existing hardened shorelines and in some cases along chronically eroding shorelines with limited sediment supply. It may also be appropriate where shoreline recession threatens infrastructure that is not able to be relocated. Materials that are commonly used in revetment construction include stone, concrete armor units, concrete slabs, sand/concrete filled geotextile bags, geo-tubes, and rock-filled gabion baskets. Revetments dissipate wave action, require minimal maintenance if well-constructed, and have an indefinite lifespan. The cost of this measure is typically moderate, depending on site conditions and availability of materials. Disadvantages however include significant land area requirement, potential loss of intertidal habitat, potential erosion of adjacent unreinforced shoreline, and prevention of the upland from being a sediment source to the system. Environmental considerations include potential impact in and out of water, impacts are not reversible, minimal maintenance required, and permits are required.

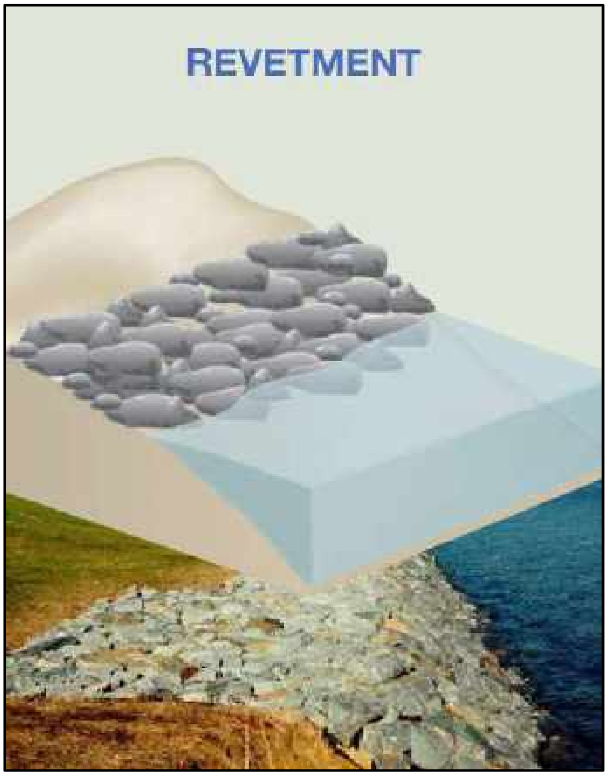

Figure 66. Revetment measure.

9.7. Bulkhead

A bulkhead is constructed parallel to the shoreline and is similar to a vertical retaining wall (Figure 67). It is intended to hold soil in place, survive the impacts of waves/currents and provide for a stable shoreline. Suitable applications are in high energy settings and sites with pre-existing hardened shoreline structures. These types of structures are commonly used along working waterfronts in ports, harbors and marinas. Bulkhead material options include various types of sheet pile, timber, reinforced concrete and rock-filled gabions. Bulkheads are suitable in moderate wave action, functional in tide level fluctuations, have a long lifespan and require minimal footprint. The cost of this measure is relatively high dependent on materials uses. Disadvantages of bulkheads are that they don't provide major flood protection, may induce erosion of seabed and adjacent unreinforced shoreline, may result in loss of intertidal habitat, prevent upland from being a sediment source to the system, and may reflect a significant amount of the incident wave energy. They can cause relatively large environmental impact in and out of water, impacts may not be reversible, there is minimal maintenance, and permits are required.

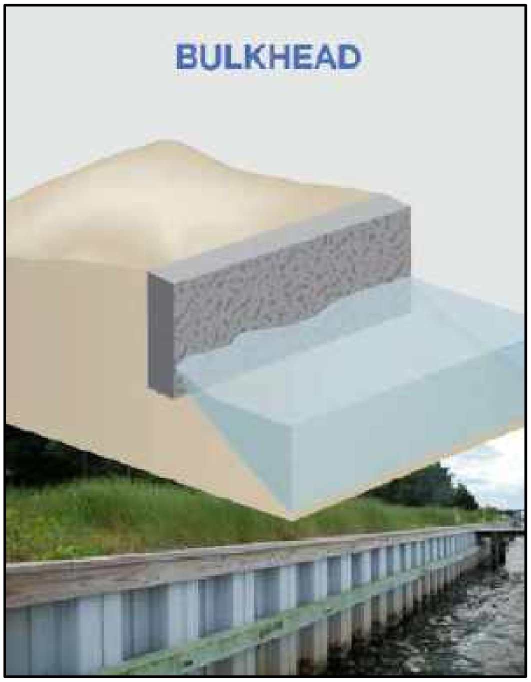

Figure 67. Bulkhead measure.

9.8. Seawall

A seawall is similar to a bulkhead in that it is constructed parallel to the shoreline and is basically a vertical retaining wall (Figure 68). It is intended to hold soil in place, survive the impacts of waves/currents and provide for a stable shoreline. Suitable applications are in high energy settings and sites with pre-existing hardened shoreline structures. These types of structures are commonly used along bay and ocean shorelines. Seawall material options include various types of sheet pile, masonry, and pre-fabricated concrete elements. Advantages of seawalls include reduction of storm surge flooding, resistance to strong wave forces, shoreline stabilization behind the structure, low maintenance costs, and a limited footprint. This measure can range from moderate to high in cost. Disadvantages include localized toe scour, disruption of sediment transport leading to beach erosion, potential loss of intertidal zone, prevention of upland from being a sediment source to the system, and may be damaged from overtopping. They can cause environmental impacts in and out of the water, impacts may not be reversible, there is minimal maintenance, and permits are required.

9.9. Groin

Groins are built perpendicular to the shoreline and intercept littoral sediment transport on their updrift side (Figure 69). They are designed to reduce longshore currents and hold sand on the adjacent shoreline. A field of properly placed T-groins can result in shoreline planforms that are stable under even severe conditions. They are most effective in combination with a properly sized beach nourishment project. Material options include stone, concrete armor units, timber and sheet pile. Groin design must provide appropriate sized stone for the site specific wave climate. Advantages include dissipation of wave energy, methods and materials are adaptable, and can be combined with beach nourishment projects to reduce renourishment costs. Costs for this measure would be moderate to high. Disadvantages include potential erosion of adjacent sites, and no storm surge protection. The environmental considerations are the same as for bulkheads and seawalls.

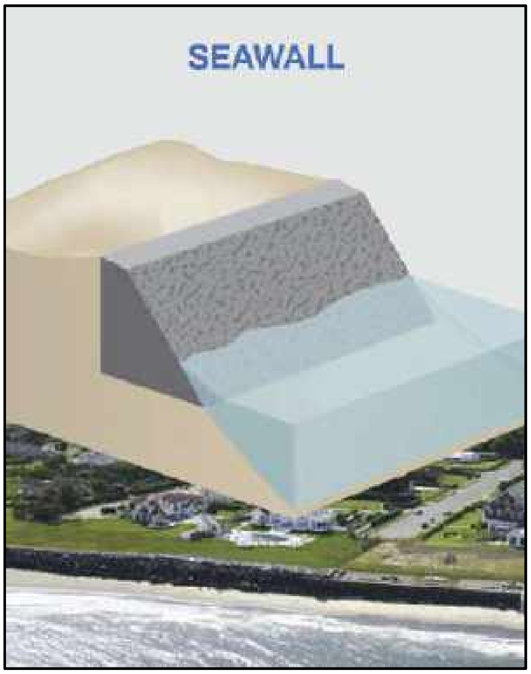

Figure 68. Seawall measure.

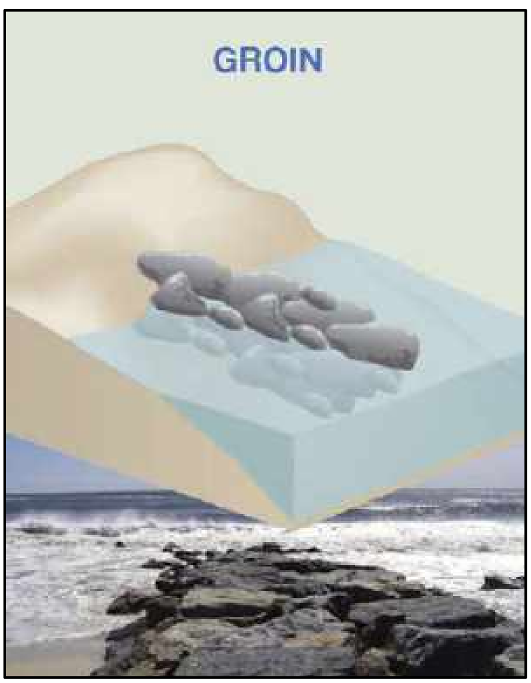

Figure 69. Groin measure.

10. Conceptual Plan – Inn on the Bay

10.1. Existing Conditions at Inn on the Bay

Inn on the Bay is a multi-unit apartment building, located between Route 2 and the shoreline, immediately south and adjacent to the Namo River Federal Flood Control Project (Figure 70). On the side of the property facing the water, a grouted concrete rock masonry (CRM) wall protects the base of the building; a grassy area with coconut trees extends toward the shoreline, varying between 15-40 ft wide; this area of fill is held in place by another grouted CRM wall. The northern portion of the property has no dry beach in front of the seawall, but at the southern end sand still remains in front of the wall providing a beach about 30-50 ft wide that continues to the south (Figures 71-73). The grouted seawall appears to be poorly designed and constructed, with varying slopes and dimensions, cracks forming in the CRM, and other damages. It is likely that the wall foundation was built on or just below the existing ground surface and is vulnerable to scour and undermining, evidenced by poured grout in areas of fill washout behind the structure. These areas also potentially indicate that overtopping is another ongoing issue.

Figure 70. Location of Inn on the Bay and alignment of existing grouted CRM wall.

Figure 71. Inn on the Bay property, looking north. Typical of northern section on the left, with no beach fronting the wall. Typical southern section on the right, where a sandy beach starts to form.

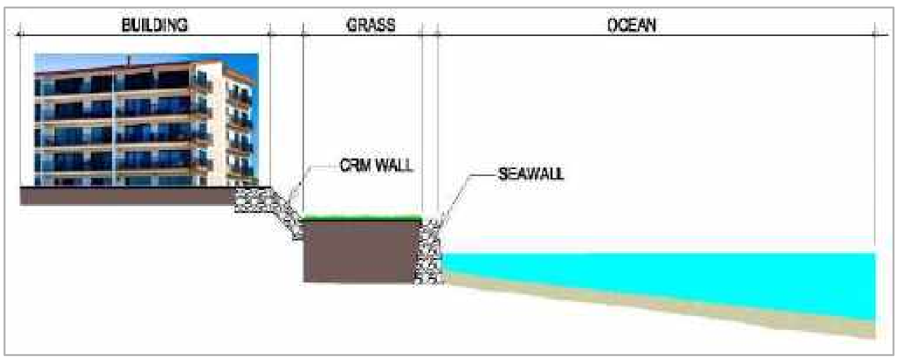

Figure 72. Sketch of existing profile at the northern section of Inn on the Bay (not to scale).

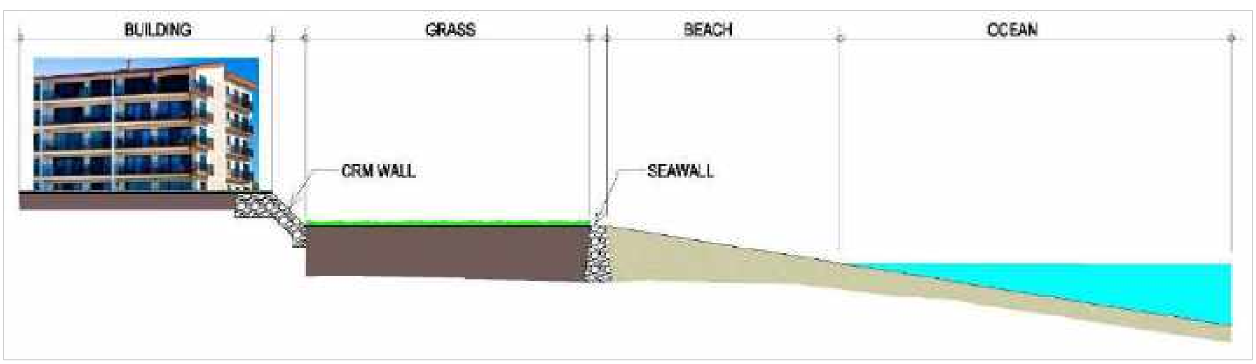

Figure 73. Sketch of existing profile at the southern section of Inn on the Bay (not to scale).

10.2. Conceptual Plan at Inn on the Bay

Since the existing apartment building and associated facilities are so close to the shoreline here, with no space to retreat, the primary purpose of any shoreline stabilization measure implemented here should be to protect the existing facilities. Therefore, gray measures are suitable for this situation, also considering that the shoreline is already hardened here in an effort to protect the property.

The proposed conceptual plan consists of removing the existing grouted CRM seawall (355 ft long) and replacing it with an engineered grouted CRM wall approximately 370 ft long. The wall should be constructed parallel to the existing building, with a proposed alignment shown in Figure 74. This would require that the wall be built seaward of the existing wall in the northern section of the property, but some excavation may occur with the wall removal in the southern portion. In the southern section where the new wall would be set back slightly from the existing alignment, existing beach sand should be backfilled in front of the new wall. The wall would be constructed with rock sized 1 to 2 feet and at a 1:1 slope to minimize the footprint and quantity of construction materials. The toe of the structure should be notched 3 ft into hard material for stability and to prevent undermining. Hard substrate is assumed to be at the same depth as the reef flat in this area, approximately 1.5 ft below mean sea level. To provide additional protection from wave overtopping and future sea level rise, the top of the wall is elevated 2 ft above the existing structure, to an elevation of approximately 6 feet above MSL. Conceptual sketches of the proposed design are shown in Figures 75 & 76.

Figure 74. Alignment of the proposed grouted wall for Inn on the Bay.

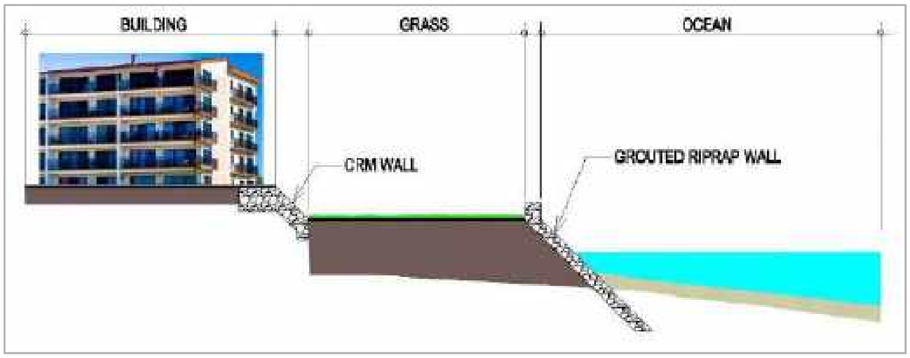

Figure 75. Schematic at the northern section of Inn on the Bay for proposed concept (not to scale).

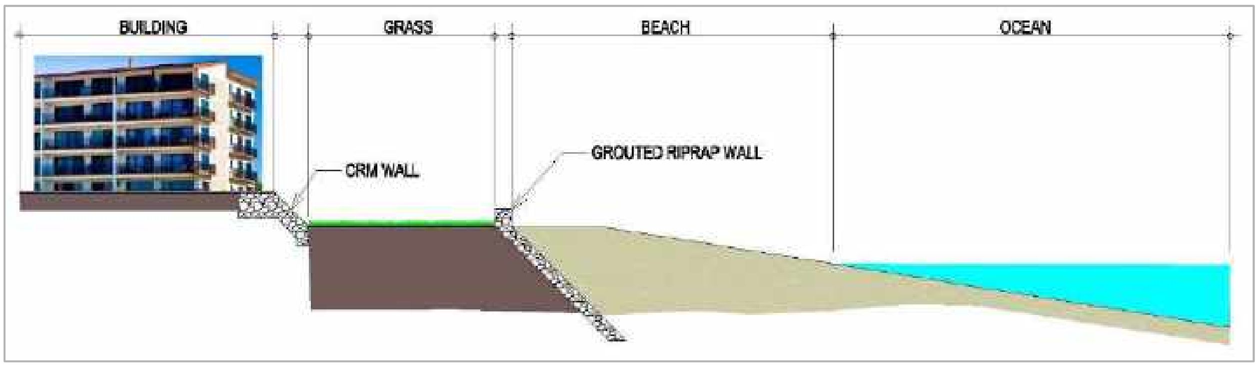

Figure 76. Schematic at the southern section of Inn on the Bay for proposed concept (not to scale).

11. Conceptual Plan – Agat Mayor's Office

11.1. Existing Conditions at Agat Mayor's Office

The Agat Mayor's Office is a compound of buildings located on Route 2 just north of the Agat Beach Unit of the War in the Pacific Nation Historic Park. The furthest ocean ward building is just a few feet from a CRM seawall that protects it from the eroding shoreline. Adjacent to the mayor's office is another community facility, Agat Sagan Bisita, with pavilions along the shoreline and an adjoining seawall (Figure 77). The proximity of these buildings and facilities to the seawall make them vulnerable to wave overtopping during high wave events. The seawall itself is vulnerable to undermining due to continued erosion of the beach.

The wall in front of the mayor's office is about 3 ft high and 80 ft long. The beach in front of it is about 15-20 ft wide (Figure 78). The south end of the wall makes a 90 degree turn landward at the junction of the two properties. The wall in front of Agat Sagan Bisita is more elevated, about 4 ft high, and setback approximately 20 ft from the Mayor's Office wall. At the southern end of the Sagan Bisita property, the wall turns inland again (Figure 79).

Figure 77. Grouted CRM wall protecting Agat Mayor's Office in the foreground. Teal pavilions in the background are part of Agat Sagan Bisita.

Figure 78. Profile of existing conditions at Agat Mayor's Office (not to scale).

11.2. Conceptual Plan at Agat Mayor's Office

Similar to Inn on the Bay, the buildings and facilities at this location are built close to the shoreline with no space to retreat and need to be protected from continued erosion and wave impacts. The existing seawall is providing minimal protection and is likely at risk of undermining. Also, since a beach does exist in front of this property, it is desirable to keep a beach here to the extent possible.

Figure 79. Alignment of existing seawall fronting Agat Mayor's Office and Agat Sagan Bisita.

The proposed conceptual plan consists of removing the existing seawall fronting the Agat Mayor's Office and Agat Sagan Bisita (365 ft long) and replacing it with an engineered grouted CRM wall along the same alignment. The wall would be constructed with rock sized 1 to 2 feet and at a 1:1 slope to minimize the footprint and quantity of construction materials. A smaller footprint will help to preserve the existing beach. The toe of the structure should be notched 3 ft into hard material for stability and to prevent undermining. Hard substrate is assumed to be at the same depth as the reef flat in this area, approximately 2.5 ft below mean sea level. To provide additional protection from wave overtopping and future sea level rise, the top of the wall is elevated 2 ft above the existing structure, or approximately 5 ft above MSL. A conceptual sketch of the proposed design is shown in Figures 80 & 81.

This location is a good candidate for a beach nourishment project using dredged material from Agat Small Boat Harbor due to its relative proximity to the harbor (approximately 1.5 miles). A nourishment project would not only widen the existing beach, but also provide additional protection to the seawall and backshore areas. At the time of this report, USACE is planning to dredge Agat SBH in 2022. Although sediment samples have not yet been taken in the harbor, based on the sediment transport analyses from this study, it is assumed to be beach quality sand. Based on recent hydrographic surveys of the harbor, there may be as much as 6,500 cubic yards (cy) of material to be dredged. If this material is dewatered and stockpiled, rather than being disposed of at the Guam Deep Ocean Disposal Site, it would be available for beneficial use projects such as this.

Figures 80 and 81 show the conceptual layout and cross-section for a beach nourishment in front of the Agat Mayor's Office. With 1,600 cy of material, a beach fill can be constructed with a 3 ft high and 20 ft wide berm along 300 ft in front of the seawall. Extending the beach out at a 1:10 slope would widen the beach by approximately 50 ft (as shown in Figure 80). This would help to stabilize the shoreline in the area that needs it most. However, if more suitable material is available, the same beach fill profile could be extended along the coast to the north to fill in more of the littoral cell. An additional 300 ft (600 ft total) of beach could be constructed with a total of 3,500 cy of material, or a total of 1,150 ft of beach could be constructed with 6,500 cy of material. As noted above, there are other important considerations for a beach fill project including regular renourishment to maintain the beach, possible impacts to regional sediment transport, possible environmental impacts, and required environmental permitting and coordination. These factors have not be evaluated in depth here and should be further examined if this measure is pursued by the Government of Guam.

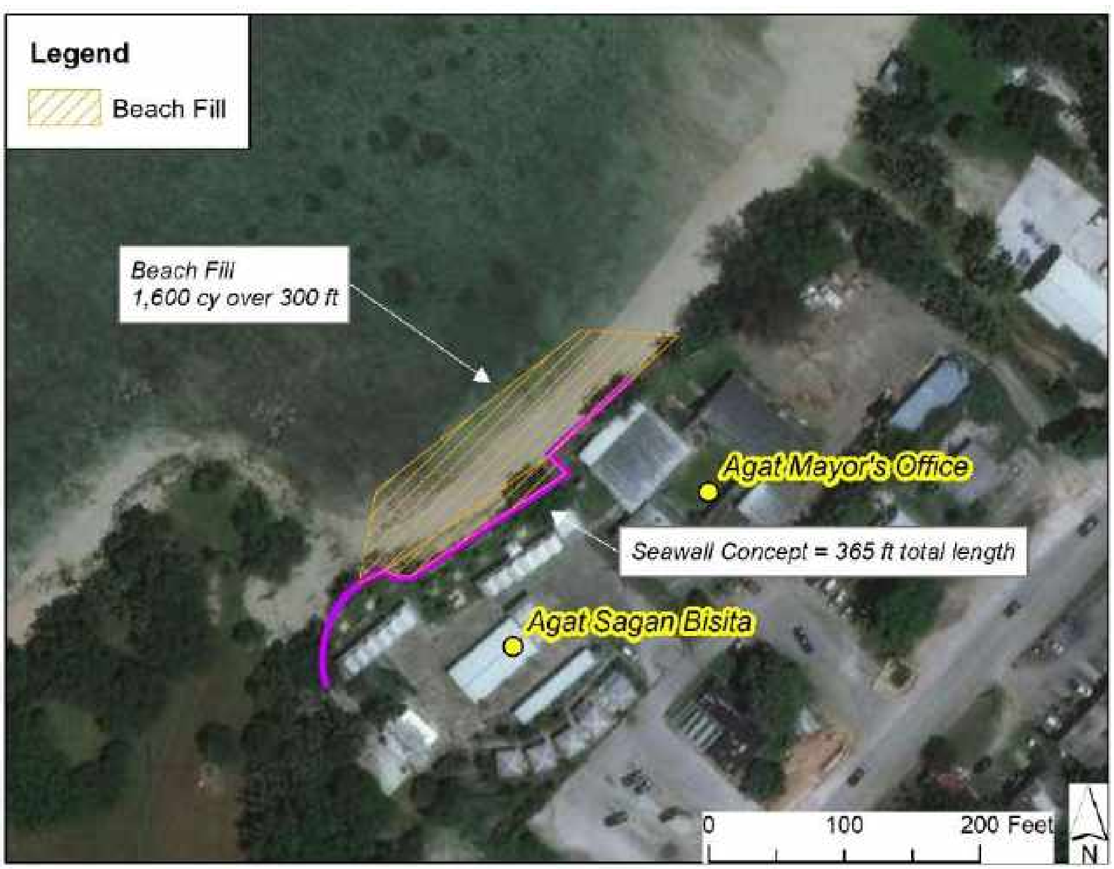

Figure 80. Conceptual layout of a new seawall and a beach nourishment.

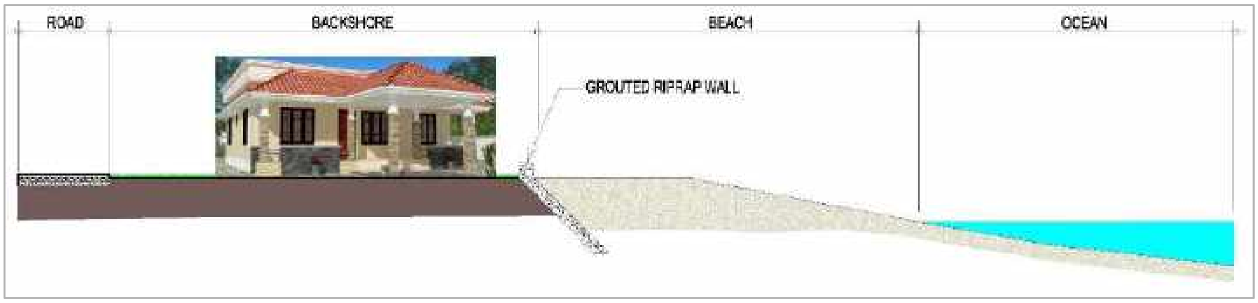

Figure 81. Conceptual sketch of CRM wall and beach fill at the Agat Mayor's Office (not to scale).

12. Conceptual Plan – Nimitz Beach Park

12.1. Existing Conditions at Nimitz Beach Park

Nimitz Beach Park is approximately 11 acres of public park situated immediately south of Agat SBH, between Route 2 and the ocean. It has 2,380 ft of existing shoreline. The majority of the park is open green space or covered with large shade trees. A sidewalk loops through the central section of the park, providing access to 6 pavilions and a restroom facility. A second restroom facility is accessed by an additional length of sidewalk (Figure 82). Some of the park shoreline has eroded so that sections of the sidewalk are now at the waters' edge and will be undermined and damaged if the shoreline is not stabilized (Figure 83).

12.1. Conceptual Plan at Nimitz Beach Park

To address the erosion issues at Nimitz Beach Park, it is recommended that the two sections of sidewalk closest to the eroding shoreline be removed for public safety. This would consist of approximately 230 ft of sidewalk from the central loop and 270 ft of sidewalk that accesses the second restroom, as indicated in Figure 82. At the central loop, new sidewalk would not be necessary since a pathway already exists to provide access to the pavilions. Construction of approximately 245 ft of new sidewalk would provide access to the lower restroom along the inner part of the park, away from the eroding shoreline (Figure 82).

Additionally, establishment of a living shoreline would help to stabilize the park shoreline edge while maintaining a natural setting. A living shoreline is a form of shoreline erosion control feature that incorporates native vegetation and preserves native habitat (Davis et al. 2015). They maintain continuity of the natural land–water interface and reduce erosion while providing habitat value and enhancing coastal resilience. Living shorelines are suitable in low wave energy environments, such as that at Nimitz Beach Park. The concept incorporates the use of plant species along the water's edge, and may be combined with other natural materials such as coir logs, geotextile tubes, stone sills, nearshore berms, or erosion control blankets. Vegetation helps to stabilize a shoreline as roots anchor soil in place and the plants serve as an upland buffer and wave breaker in low wave energy environments.

Figure 82. Nimitz Beach Park, showing the existing conditions and proposed measures.

Figure 83. Sketch of existing conditions at Nimitz Beach Park (not to scale).

The proposed concept is to plant a range of vegetation including seagrasses at the water's edge, shrubs, and then intertidal and wooded trees (Figure 84). Seagrass fields maintain a higher bed elevation that will help to attenuate waves. Shrubs should be selected that are salt and wind tolerant, and can thrive in various types of soils, including sand. Erosion control matting would be used as an aid to control erosion during the establishment period of protective vegetation. A living shoreline may require periodic maintenance as natural materials degrade over time.

The first priority for the establishment of a living shoreline are the sections of higher erosion that are near the existing sidewalks, a total of 310 ft (210 ft near the pavilions and 100 ft near the second restroom), shown in Figure 82. However, this living shoreline concept can be easily extended to the remaining shoreline along Nimitz Beach Park to help stabilize the entire area.

Figure 84. Sketch of conceptual plan for Nimitz Beach Park, with a living shoreline and relocation of the sidewalk (not to scale).

13. ROM Cost Estimates For Conceptual Shoreline Stabilization

A Rough Order of Magnitude (ROM) construction cost estimate was developed for each concept based on the following assumptions. ROM cost estimates are shown in Table 9 below.

ROM Cost Assumptions:

- Work to be performed by Guam contractors and labor force. Therefore, mobilization and demobilization were based on local conditions. Guam labor force was used, based labor rates provided by Bureau of Labor Statistics (May 2017) adjusted to 2020.
    - Note: Mobilization and demobilization costs would increase significantly for a contractor outside of Guam. Bidding climate should be evaluated closely prior to contract solicitation.
- Assumes all materials are furnished from Guam (assumes 2% sales tax).
- Material for CRM walls is assumed to be similar in features to limestone characteristics
- Assumes voids and bulking in neat line area for placement material and 10% waste and overbuild.
- Field Office Overhead = 25%, Home Office Overhead = 10%, Profit = 10%, Bond = 2%, Price Level = 2020
- Pre-Construction Engineering/Design costs not included
- Construction Supervision and Administration costs not included
- Environmental Permitting costs not included
- Borrow Areas: Assumed to be available on Guam for all material.
- Material Disposal: Concrete from sidewalk removal is assumed to be taken to local crushing site for reuse.
- Site Conditions: Disposal areas are assumed to be in good working condition and no restoration or dike construction has be assumed to be necessary.
- Unusual Conditions (Soil, Water, and Weather): Tight working areas around existing facilities will require smaller plant.
- Weather Days: None anticipated.
- Equipment and Labor Availability and Distance Traveled: Equipment and labor availability is a concern based upon the remote location of Guam. The assumption being that contractors with the equipment and labor resources necessary can be found on Guam.
- No real estate costs are included.
- **Contingency of 35% is included** at this conceptual stage based on the lack of detailed technical information and scope clarity resulting in major estimate assumptions in technical information and quantities, heavy reliance on cost engineering judgment, cost book, parametric, historical, and little specific crew-based costs.

Table 9. ROM Cost Estimates for Shoreline Stabilization Concepts

Summary Shoreline Stabilization Concepts ROM Costs			
Description	**Quantity**	**Unit**	**Project Cost***
Inn on the Bay	**1**	**EA**	**$345,000**
Mobilization and Demobilization	1	LS	$10,800
Remove existing grouted CRM wall	355	LF	$18,600
Construct Rock with Grout Wall	370	LF	$286,600
Associated Cost	1	EA	$29,000
Agat Mayor’s Office with Beach Fill Option 1	**1**	**EA**	**$464,800**
Mobilization and Demobilization	1	LS	$5,400
Remove existing grouted CRM wall	365	LF	$27,600
Construct Rock with Grout Wall and 300 LF of Beach Fill	365	LF	$402,800
Associated Cost	1	EA	$29,000
Beach Fill Option 2, Added Cost *Additional 300 LF of Beach Fill (600 LF total)*	*1*	*EA*	*$137,700*
Beach Fill Option 3, Added Cost *Additional 850 LF of Beach Fill (1,150 LF total)*	*1*	*EA*	*$429,800*
Nimitz Beach Park	**1**	**EA**	**$259,800**
Mobilization and Demobilization	1	LS	$8,100
Remove Sidewalks	500	LF	$150,900
Add Sidewalks	245	LF	$28,000
Stabilize the Shoreline with Vegetation (Priority 1)	310	LF	$53,400
Associated Cost	1	EA	$19,400
Stabilize the Shoreline with Vegetation (Priority 2), Added Cost	*2,070*	*LF*	*$310,700*

*Project Cost includes 35% contingency. Does not include mitigation, lands and damages, E&D, or S&A costs.

14. Summary

A regional analysis of Agat Bay, including Agat SBH and areas of interest due to recent shoreline changes, has been conducted using various tools and historical information. This has provided information regarding historical trends in sediment transport, and associated coastal processes influencing the sediment pathways in the region. In addition, modifications to Agat Small Boat Harbor have been investigated for the purpose of reducing harbor currents and shoaling. Conceptual plans have also been developed for shoreline stabilization at three locations of concern within the study region.

Overall, the analysis showed that sediment movement within the region is complex, and not strongly dominant in one direction or the other alongshore, but rather influenced by small circulation cells controlled by bathymetry and coastal morphology. Since this region is characterized by narrow beaches and a limited sediment supply, even relatively small rates of erosion can have noticeable impacts to shoreline infrastructure. These issues will be further exacerbated by future sea level rise, but can be better managed holistically with an understanding of regional sediment transport. Future adaptation planning should be considered for vulnerable coastal infrastructure, with measures that employ adaptation strategies including elevation, protection, or retreat where feasible.