Re: *SAJ-2004-12518 (SP-JCM)*
St. John Marina Yacht Club

The Summer's End Group, LLC, Applicant
Response to the Corps request for additional information
by letter dated October 26, 2017

Exhibit "B"
Wind & Wave Analysis

Marina Site Suitability Analysis

Summer's End Marina

Prepared for:

Summer's End LLC

St. John, U.S. Virgin Islands

Prepared by:

Water Environment Consultants
Mount Pleasant, SC

December 9, 2017

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Executive Summary

Water Environment Consultants, LLC (WEC) was initially contracted by Technomarine USA to complete a study to define the extreme wind, wave and water level conditions at the Summer's End project site on St. John, USVI for the purposes of their marina works design. Subsequently, the Owner (Summer's End LLC) requested WEC to use readily available information and expand upon the study to evaluate the suitability of the site to serve as a marina location, from a site exposure perspective. The primary results of the expanded analysis are summarized below and presented in more detail in the following document.

Table ES-1. Extreme stillwater and wave setup elevations

Return Period (yr)	3-sec wind speed (mph)	1-min wind speed (mph)	Hourly wind speed (mph)
1	45[1]	33	27
10	74	55	44
25	112	83	67
50	130	96	78
100	143	106	85

1. Extrapolated from dataset

Table ES-2. Extreme stillwater and wave setup elevations

Event return period (yrs)	Stillwater elevation[1] (ft MSL)	Wave setup[2] (ft)	Stillwater + setup (ft MSL)
1	1[3]	0.8[3]	1.8
10	3.5	1.1	4.6
25	4.5	1.2	5.7
50	5.2	1.3	6.5
100	6	3.2	9.2

Notes:
1. 25-yr stillwater interpolated based on logarithmic trend of other data points.
2.10, 25 and 50-yr setup calculated by SWAN model (see Section 4). 100-yr wave setup is from FIS study
3. Extrapolated from dataset

Table ES-3. Combined (swell and wind) effective wave heights and periods

Return Period (yrs)	Hs (ft)	Tp (s)
1	1.6	8.2 – 11.0
10	3.9	8.0 – 10.7
25	5.1	7.5 – 9.8
50	5.9	7.5 – 9.9

Table ES-4. Extreme depth-averaged current velocities

Return Period (yrs)	V (m/s)	V (ft/s)
1	0.3	1.0
10	0.6	2.0
25	0.9	3.0
50	1.1	3.6

All things considered, the site location is relatively protected with its only wave exposure being to the southeast direction. Wave modeling results demonstrate that ocean waves from the southeast are refracted such that most of the wave energy is directed towards headlands south of the marina site, which greatly reduces the wave exposure from the southeast. Additionally, site observations (by others) indicate that the site is a very quiescent location during typical operational conditions. The estimated 1-year return period condition exceeds the design guidelines for operational conditions, although this is expected to occur infrequently. The number of days that the wave heights would exceed the operational criteria is unknown and is beyond the scope of this study. If the Owner needs to ensure the operational criteria are not exceeded (other than during hurricane events), then additional infrastructure would be required (e.g., a floating wave attenuator). Alternatively, the Owner may accept the risk that the operational criteria may be exceeded one or more days per year. Overall, the site is expected to provide safe berthing for recreational boats during operational conditions except for a small fraction of the time.

Extreme hurricane conditions, such as the 50-yr return period storm, will result in wave heights exceeding the marina tranquility standards. The Owner, in consultation with other professionals, should determine the amount of acceptable risk for the marina facility, and then determine how to best mitigate the risk though physical risk reduction measures (e.g., designing to survive extreme events, incorporating factors of safety, etc.) and/or risk management through insurance.

1 Introduction

Water Environment Consultants, LLC (WEC) was contracted by Technomarine USA to complete a study to define the extreme wind, wave and water level conditions at the Summer's End project site on St. John, USVI (Figures 1-1 and 1-2). WEC was contracted to:

- Review and analyze available information from the Federal Emergency Management Agency to determine appropriate extreme wind and water levels at the project site;
- Review available Wave Information Study (WIS) hindcast data and determine extreme offshore wave conditions associated with 25 and 50-yr return period events;
- Conduct two-dimensional wave modeling to calculate the transformation of the offshore waves into Coral Bay and determine the extreme met-ocean climate; and
- Prepare a short report summarizing the study methods and results, to be provided in PDF format.

The original analysis was completed for Technomarine in 2016. The analysis was subsequently revised to include an evaluation of operational limiting conditions and a review of the met-ocean results in the context of the suitability of the site for marina development.

This report summarizes the results in the following sections:

- Section 2, Historic Storm Events – describes the effects of several major hurricanes that have affected the site since 1989;
- Section 3, Water Levels – summarizes astronomical tides and extreme water levels that may occur from hurricane events;
- Section 4, Extreme Wind Speeds – provides estimates of extreme wind speeds that may occur during hurricane events;
- Section 4, Extreme Waves – describes extreme offshores waves and estimates the extreme waves at the project site caused by offshore swell and locally generated wind waves;
- Section 5, Currents - describes the currents that may occur at the project site; and
- Section 6, Discussion & Recommendations – describes the interpretation of met-ocean conditions in relation to the suitability of the site for marina development, potential risks and additional marina infrastructure to mitigate those risks.

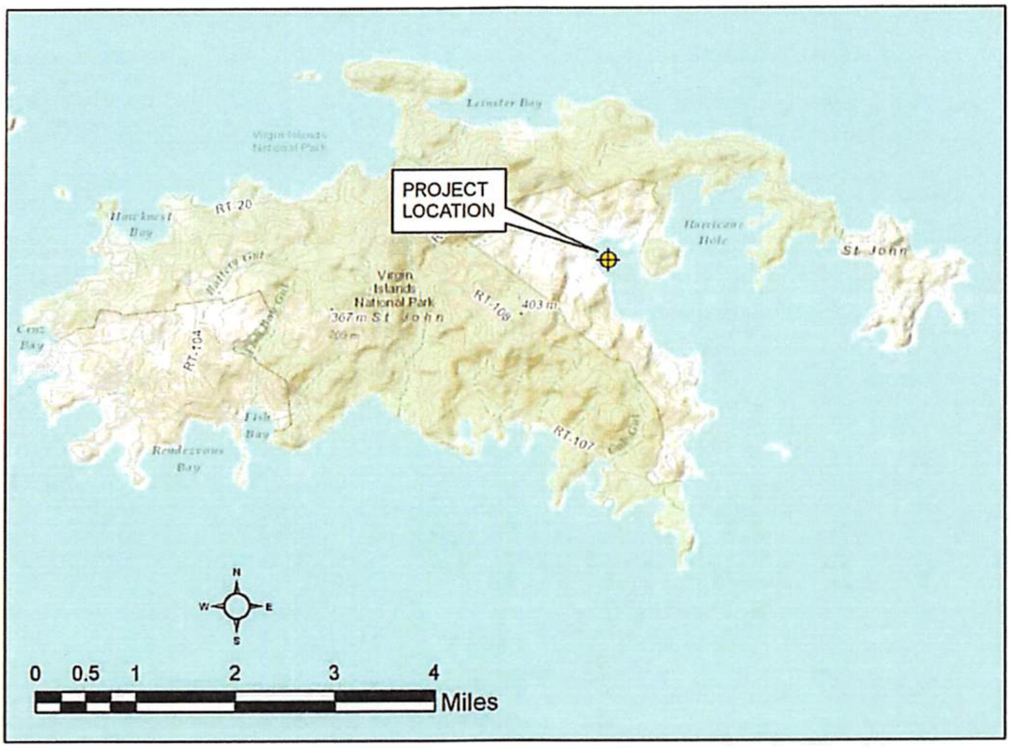

Figure 1-1. Project location map

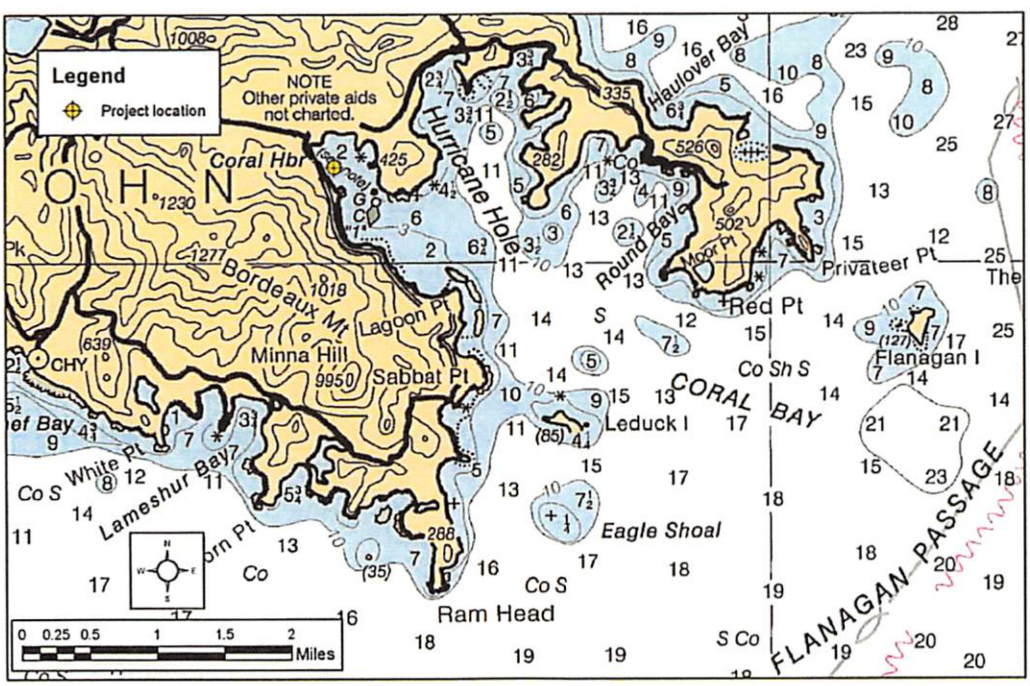

Figure 1-2. Navigation chart

2 Historic Storm Events

Four major hurricanes are described by the Federal Emergency Management Agency (FEMA) in their 2007 Flood Insurance Study for the USVI, including Hurricanes Hugo, Marilyn, Bertha and Lenny. In addition, two recent events – Hurricanes Irma and Maria – recently struck earlier this year with devastating consequences.

2.1 Hurricane Hugo, 1989

Hurricane Hugo was a Category 4 hurricane when it struck the Virgin Islands (National Oceanic and Atmospheric Administration [NOAA] 2007). Hugo caused severe coastal flooding the in Virgin Islands due to storm surges of as much as 11 ft (FEMA 2007). The storm passed directly over St. Croix, where the greatest damage occurred within the USVI. WEC did not find any official surge, wave or damage estimates for Coral Harbor in NOAA or FEMA documents. David Lyman posted to his web site his personal account of enduring Hugo while onboard is boat in Coral Bay (Lyman 2016):

> "There had been 55 boats anchored in Coral Bay when Hurricane Hugo arrived that September night in 1989. When the storm was over that next morning, there were just 5 boats still riding to their anchors...Many boats were blown ashore as other boats' mooring lines chaffed through or snapped, or small anchors dragged, entangling two, three or four boats in a mess as they were driven up on the beach by the wind, lifted by waves and surge far above the water line. When the hurricane left, boats were stacked three and four deep, 10 feet from the water's edge."

Lyman estimated wave heights in the bay exceeding 10 feet, although measured wave heights or maximum water levels are not available. His photographs of boats beached along the shoreline of the harbor (Figure 2-1) are evidence that damaging wind, surge and waves can occur in the harbor during hurricane events.

Figure 2-1. Boats on Coral Harbor shoreline following Hugo (Source: Lyman 2016)

2.2 Hurricane Marilyn, 1995

Hurricane Marilyn was nearly a category 3 hurricane when it struck the USVI (FEMA 2007). Storm surge in the USVI reached 6 to 7 feet, with an isolated surge of 11.7 feet reported in St Croix. The eye passed over St. Thomas, where approximately 80 percent of homes and businesses were destroyed. On St. John, approximately 30 percent of homes were destroyed (FEMA 2007). The approximate high water mark observed by the US Geological Survey in Coral Bay was 5.3 feet MSL (Torres-Sierra 1998).

2.3 Hurricane Bertha, 1996

The USVI were declared federal disaster areas following this storm. Bertha damaged almost 2,500 homes on St. Thomas and St. John.

2.4 Hurricane Lenny, 1999

Hurricane Lenny was unusual because of its extended west-to-east storm track. The hurricane caused an estimated 15 to 20 foot storm surge in Frederiksted on St. Croix. The National Ocean Service gage in Lime Tree Bay recorded a surge of 2.9 feet. A NOAA gage on St. Thomas recorded a surge of 1.8 feet.

2.5 Hurricane Irma, 2017

Hurricane Irma was the first Category 5 hurricane on record to hit the Leeward Islands and caused widespread and catastrophic damage across the Caribbean. On 6-Sept-2017, its path crossed just north of the US Virgin Islands with its eyewall landing just off of the British Virgin Islands. At landfall in BVI, Hurricane Irma reached its peak intensity with 185 mph (295 km/h) winds (https://en.wikipedia.org/wiki/Hurricane_Irma). Most of the buildings, infrastructure and boats in Coral Bay were still destroyed by Irma. At Lameshur Bay, St. John, a National Ocean Service tide gauge recorded a peak water level was 1.96 feet (https://tidesandcurrents.noaa.gov). Catastrophic damages reinforce the site is susceptible to damaging wind, surge and waves during hurricane events.

2.6 Hurricane Maria, 2017

Hurricane Maria was the 10th-most intense Atlantic hurricane on record. The hurricane reached Category 5 status before making landfall in Puerto Rico. Catastrophic damage and numerous fatalities occurred across the northeastern Caribbean, including the US Virgin Islands. Its northwesterly track towards Puerto Rico placed St. John just north and east of its path, exposing Coral Bay to hurricane force winds from the southeast and resulting storm waves propagating virtually unobstructed to the site from the offshore. A Sandy Point National Wildlife Refuge in St. Croix, sustained winds reached 99 to 104 mph (159 to 167 km/h) and gusted to 137 mph (220 km/h). (https://en.wikipedia.org/wiki/Hurricane_Maria) Weather stations on St. Croix recorded 5 and 10 inches of rain from the hurricane, and estimates for St. John and St. Thomas were somewhat less (National Weather Service 2017). A National Ocean Service tide gauge at Yabucoa Harbor, Puerto Rico, reported a water level of 4.3 feet above Mean Higher High Water (MHHW) at landfall (http://www.nhc.noaa.gov). At Lameshur Bay, St. John, a National Ocean Service tide gauge recorded a peak water level was 2.25 feet (https://tidesandcurrents.noaa.gov). Again, this most recent hurricane is yet another reminder that despite its generally quasi-sheltered location, the site is not exempt from damaging hurricane conditions.

3 Water Levels

3.1 Astronomical Tides

The NOAA published tidal datums for Lameshur Bay, St John (Station 9751381) are listed in Table 3-1. This is the closest tidal station to the project site. The mean tide range is small: Mean High Water (MHW) is approximately 0.7 feet above Mean Low Water (MLW). Also, note that the basis for the Virgin Islands Vertical Datum of 2009 (VIVD09) is Local Mean Sea Level for the National Tidal Datum Epoch 1983-2001.

Table 3-1. Tidal water level datums

Datum	Description	Elevation (ft MLLW)
HAT	Highest Astronomical Tide	1.24
MHHW	Mean Higher-High Water	0.82
MHW	Mean High Water	0.78
MTL	Mean Tide Level	0.42
MSL	Mean Sea Level	0.39
DTL	Mean Diurnal Tide Level	0.41
MLW	Mean Low Water	0.06
MLLW	Mean Lower-Low Water	0
VIVD09	Virgin Islands Vertical Datum of 2009	0.37
LAT	Lowest Astronomical Tide	-0.37

3.2 Extreme Water Levels

Extreme water levels from hurricane events have been evaluated by FEMA and are listed in the FIS for St. John (FEMA 2007). As noted in the FIS, there is a higher probability of landfalling storms on the south coast of St. John than the north coast because the prevailing storm direction is from the east and southeast. However, the bathymetry surrounding the island is steep, which limits the amount of surge caused by storms.

FEMA modeled hurricane storm surge at St. John using the ADCIRC hydrodynamic model and the PBL wind model. FEMA then analyzed coastal wave and water level elevations at specific cross-shore transects along the St. John shoreline. The closest transect location is J34, located in Coral Harbor, as shown in Figure 3-1. The reported extreme stillwater elevations for this transect are listed in Table 3-2 (note that WEC added an interpolated water level value for the 25-year return period, which is not included in the FIS). The stillwater elevation is the water level produced by the storm pressure and wind stress effects. In addition, wave setup adds an additional component of storm surge. The 1-percent-annual-chance (i.e., 100-year return period) stillwater plus wave setup listed in the FIS is 9.2 feet above local mean sea level (MSL), which includes a wave setup of 3.2 feet. WEC used the SWAN wave model (see Section 5 of this report) to calculate the wave setup component for the 10, 25 and 50-year return period events. Based on the SWAN model results, the FIS estimate of 3.2 of wave setup is conservative (i.e., high). The Base Flood Elevations for the project area are shown by FEMA Flood Insurance Rate Map in Figure 3-2.

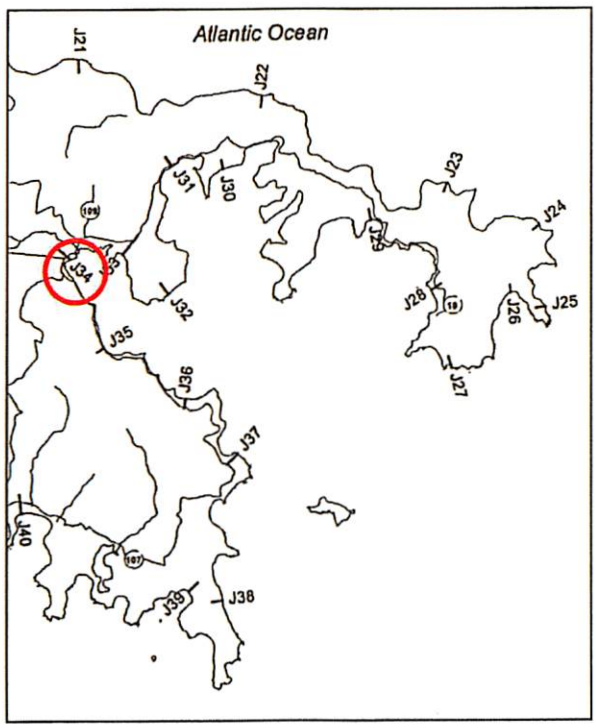

Figure 3-1. FIS transect location map (closest transect circled in red)

Table 3-2. Extreme stillwater and wave setup elevations

Event return period (yrs)	Stillwater elevation[1] (ft MSL)	Wave setup[2] (ft)	Stillwater + setup (ft MSL)
1	1[3]	0.8[3]	1.8
10	3.5	1.1	4.6
25	4.5	1.2	5.7
50	5.2	1.3	6.5
100	6	3.2	9.2

Notes:

1. 25-yr stillwater interpolated based on logarithmic trend of other data points.
2. 10, 25 and 50-yr setup calculated by SWAN model (see Section 4). 100-yr wave setup is from FIS study
3. Extrapolated from dataset

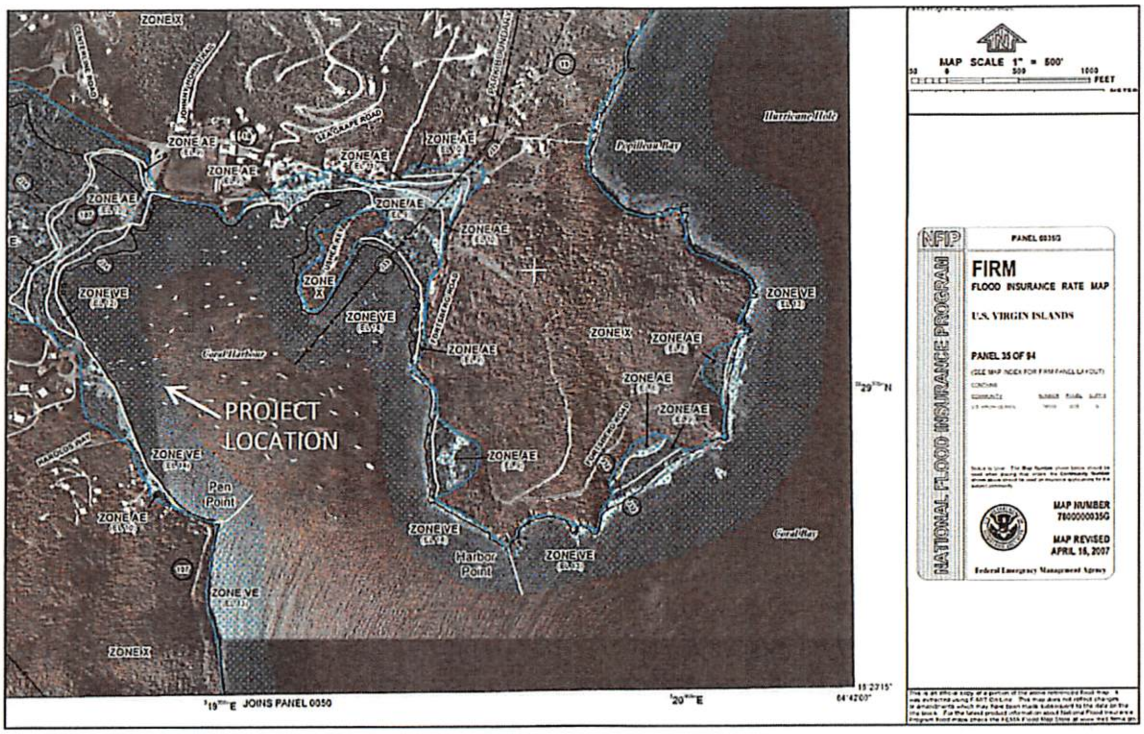

Figure 3-2. Flood Insurance Rate Map

3.3 Historical Sea Level Rise

Long-term water level monitoring by NOAA at Charlotte Amalie, VI indicates that the long-term historical trend in sea level rise (SLR) is approximately 0.65 ft per 100 years. SLR is expected to continue at a rate at least as high as the historical rate, and it may accelerate (although the degree of SLR acceleration is highly uncertain).

4 Winds

4.1 Typical Winds

Figure 4-1 summarizes the typical wind statistics for Cruz Bay, St. John. Winds typically blow from the east, east-southeast and southeast directions, and average 9 mph. Although the average winds are relatively weak, the prevailing winds are blowing in the worst-case direction relative to the site (i.e., the southeast quadrant), as the marina is exposed to the offshore waters from the southeast quadrant. Relatively speaking, locally generated wind waves from this direction will be greatest.

Statistics based on observations taken between 4/2012 - 4/2013 daily from 7am to 7pm local time. You can order the raw wind and weather data in Excel format from our historical weather data request page.

Month of year	Jan 01	Feb 02	Mar 03	Apr 04	May 05	Jun 06	Jul 07	Aug 08	Sep 09	Oct 10	Nov 11	Dec 12	SUM 1-12
Dominant Wind dir			n/a										
Wind probability >= 4 Beaufort (%)	23	40	n/a	29	28	19	19	20	2	14	3	11	10
Average Wind speed (mph)	5	8	NaN	9	9	9	9	9	2	3	5	5	6
Average air temp. (°F)	80	80	NaN	82	82	86	86	86	87	86	86	82	84
Select month (Help)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year

Wind dir. distribution Cruz Bay all year

Wind direction Distribution (%)

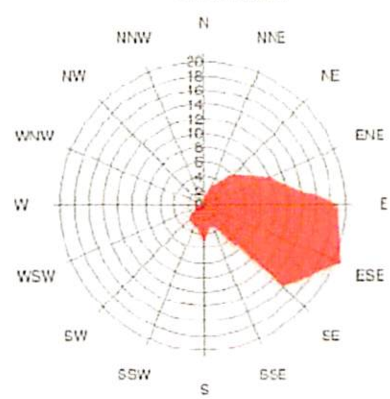

Figure 4-1. Annual Wind Rose Statistics for Cruz Bay, St. John (www.windfinder.com)

4.2 Extreme Winds

The most up-to-date design wind speeds are given by the national load standard, American Society of Civil Engineers (ASCE) 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010). The ASCE 7-10 extreme wind speeds for the Virgin Islands, given as 3-sec peak gust values, are listed in Table 4-1. The Technomarine design criteria require 1-minute duration wind speeds. Therefore, the wind speed durations were adjusted to 1-minute wind speeds based on the hourly average equivalent wind speed following the methods from the US Army Corps of Engineers (USACE) Coastal Engineering Manual (USACE 2006).

Table 4-1. Extreme wind speeds

Return Period (yr)	3-sec wind speed (mph)	1-min wind speed (mph)	Hourly wind speed (mph)
1	45[1]	33	27
10	74	55	44
25	112	83	67
50	130	96	78
100	143	106	85

1. Extrapolated from dataset

Understanding that Technomarine typically uses wind speed design criteria that correspond to the Saffir Simpson Hurricane Wind Scale, WEC correlated the extreme wind speeds in terms of hurricane wind scale. For the maximum wind speed with full occupancy, Technomarine typically assumes a Category 1 hurricane (sustained winds of 74-95 mph). For the maximum wind speed without boats, Technomarine typically assumes a Category 3 hurricane (sustained winds of 111-129 mph). For this site the Category 1 wind speed range corresponds roughly to a return period between 20 and 50 years. The Category 3 wind speed range corresponds to return periods greater than 100 years.

5 Waves

Coral Harbor is less than a half a mile wide, and the local fetch affecting the proposed marina is very short in most directions. The only significant fetch is to the south southeast, and extreme waves from this direction will be from locally generated wind waves and offshore swell during hurricane events. This section includes an assessment of the operational and extreme offshore waves and numerical modeling to estimate wind waves and swell in Coral Bay.

5.1 Typical Waves

Observations of the typical wave climate within Coral Harbor were documented by Applied Technology and Management (ATM) et al. (2014). From their observations intermittently over a period of 9-months from 2012 – 2014, wave heights never exceeded 0.15m (0.5ft). Visual observations are typically not wholly-accurate wave height measurements; in particular, it is likely that the observation was likely only noting the short period (2 – 3s) wind waves and any low frequency component was not included. Nevertheless, this provides a visual assessment of typical wave conditions observed at the site. ATM et al. (2014) also reported that "In the marina footprint however waves have been noted impacting the shore to the south which are as much as 1 ft in height."

Table 5-1. Wave Height Observations at Coral Harbor (extracted from ATM et al. 2014)

Date	Observed Wave Height (ft)	Observed Wave Height (m)
5/12/2012	0.33 - 0.50	0.10 - 0.15
5/22/2012	0.17 - 0.25	0.05 - 0.08
6/17/2012	0.17 - 0.25	0.05 - 0.08
6/18/2012	0.33 - 0.50	0.10 - 0.15
6/23/2012	0.33 - 0.50	0.10 - 0.15
6/31/2012	0.33 - 0.50	0.10 - 0.15
7/31/2012	-	-
8/2/2012	0.33 - 0.50	0.10 - 0.15
8/12/2012	0.33 - 0.50	0.10 - 0.15
9/14/2012	0.33 - 0.50	0.10 - 0.15
9/22/2012	0.33 - 0.50	0.10 - 0.15
10/7/2012	0.33 - 0.50	0.10 - 0.15
10/8/2012	0.50 - 0.50	0.15 - 0.15
11/13/2012	0.08 - 0.08	0.03 - 0.03
12/8/2012	0.17 - 0.25	0.05 - 0.08
1/16/2014	0.25 - 0.33	0.08 - 0.10
1/20/2014	-	-
1/24/2014	0.25 - 0.33	0.08 - 0.10
2/3/2014	0.33 - 0.50	0.10 - 0.15
2/25/2014	0.17 - 0.25	0.05 - 0.08

5.2 Extreme Waves

5.2.1 Offshore Waves

The USACE Engineering Research and Development Center (ERDC) provides high-quality coastal wave hindcast model estimates through the Wave Information Study (WIS) program. WIS data were obtained from the USACE for the WIS station closest to the project site (Station 61022), which is approximately 53 miles southeast of the project site (Figure 5-1).

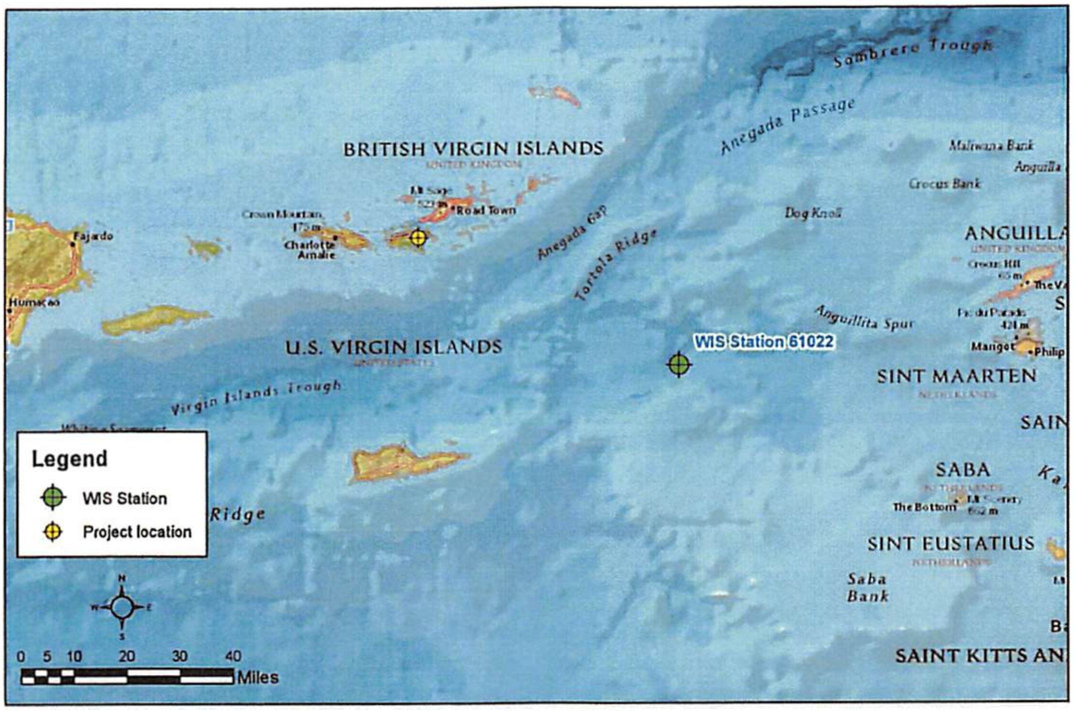

Figure 5-1. WIS station location

ERDC completed an extreme value analysis of the largest 33 events in the hindcast record, as shown in Figure 5-2. The resulting estimates of extreme significant wave heights for various return periods are summarized in Table 5-2. The peak periods for the largest six events (i.e., those equal to or greater than the 10-year return period event) range from 11.6 to 14.5 seconds. For these extreme events, longer wave periods are not directly correlated with higher wave heights, and therefore the design should assume that the wave periods for extreme events may range from 11 to 15 seconds.

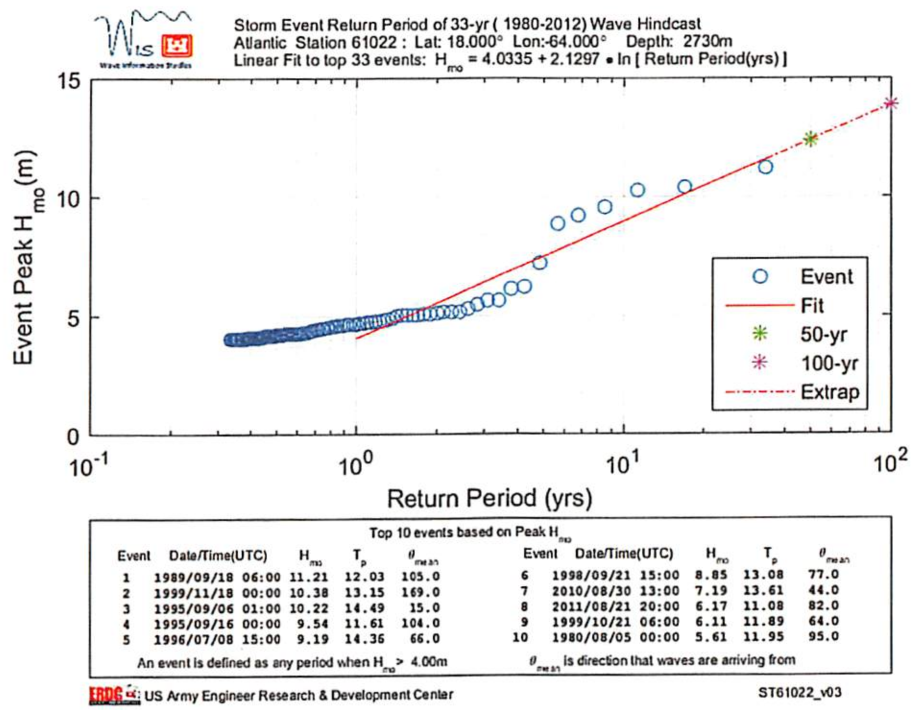

Figure 5-2. Extreme wave height analysis plot (Source: USACE 2016)

Table 5-2. Extreme offshore wave heights

Return Period (yr)	Hmo (m)	Hmo (ft)
1	4.1[1]	13.4
10	8.9	29.3
25	10.9	35.7
50	12.4	40.6
100	13.8	45.4

1. Extrapolated from dataset.

5.2.2 Marina Site Waves

Coral Harbor is well protected from ocean waves, but some wave energy will enter the harbor from the south southeast. WEC used two-dimensional numerical wave models to estimate the waves that will enter the harbor. WEC used CGWAVE to model the refraction, diffraction, shoaling and reflection of swell as it travels from the ocean, through Coral Bay and into Coral Harbor. WEC also used SWAN to model the wind wave growth from the local fetches. During extreme hurricane events, the site will be subjected to the combination of both the swell and wind waves.

5.2.2.1 Swell

CGWAVE (Panchang & Xu 1995) is a two-dimensional finite element model based on the elliptic mild-slope wave equation. CGWAVE includes processes include refraction and shoaling as waves propagate towards the project site from deeper waters offshore, as well as reflection and diffraction as waves encounter the shoreline.

WEC set up the CGWAVE model of Coral Bay using the local bathymetry surveyed at the project site combined with bathymetry digitized from the NOAA navigation chart for the USVI (chart #25641). The model includes a wave reflection coefficient of 0.25 assigned to the shoreline areas. Typical wave reflection coefficients range from 0.1 for flat beaches to 0.45 for rubble mound structures and 0.9 for vertical walls. An intermediate value was used here given the steep shorelines throughout most of the study area. WEC modeled a range of wave periods, 10 to 15 seconds, representative of the possible range of swell from extreme hurricane events. Modeled wave directions included east southeast, southeast and south southeast.

Example output from the CGWAVE model is shown in Figures 5-3 and 5-4. As shown by these figures, the bathymetry causes the swell traveling up Coral Bay to refract (i.e., bend) toward Harbor Point and toward the headland between Sanders Bay and Johnson Bay. Most of the wave energy is refracted towards these shorelines before reaching Coral Harbor. The wave refraction patterns are dependent on the wave length (a function of wave period), as shown by the slight differences between Figure 5-3 (a 13-second wave period) and Figure 5-4 (a 15-second wave period). The maximum modeled wave heights reaching the south side of the proposed marina are approximately 11 percent of the offshore wave height. Table 5-3 summarizes the modeled swell heights approaching the marina for the 10, 25 and 50-yr return period events, plus a calculated 1-yr return period height based on the modeled transformation coefficients for the other cases.

Table 5-3. CGWAVE modeled swell heights

Return Period (yrs)	Offshore Swell (ft)	Marina Site Swell (ft)
1	13.4	1.6[1]
10	29.3	3.2
25	35.7	3.9
50	40.6	4.5

1. Calculated based on average transformation coefficient

5.2.2.2 Wind Waves

The SWAN (Simulating WAves Nearshore) model was used to estimate the extreme wind waves that can occur from wind wave growth over the fetch to the southeast of the project site during hurricane events. SWAN is a two-dimensional steady-state spectral wave transformation model that simulates the growth and transformation waves in the nearshore region. The wave model simulates the following wave processes: wave propagation, shoaling, and refraction; wind wave growth; and wave dissipation

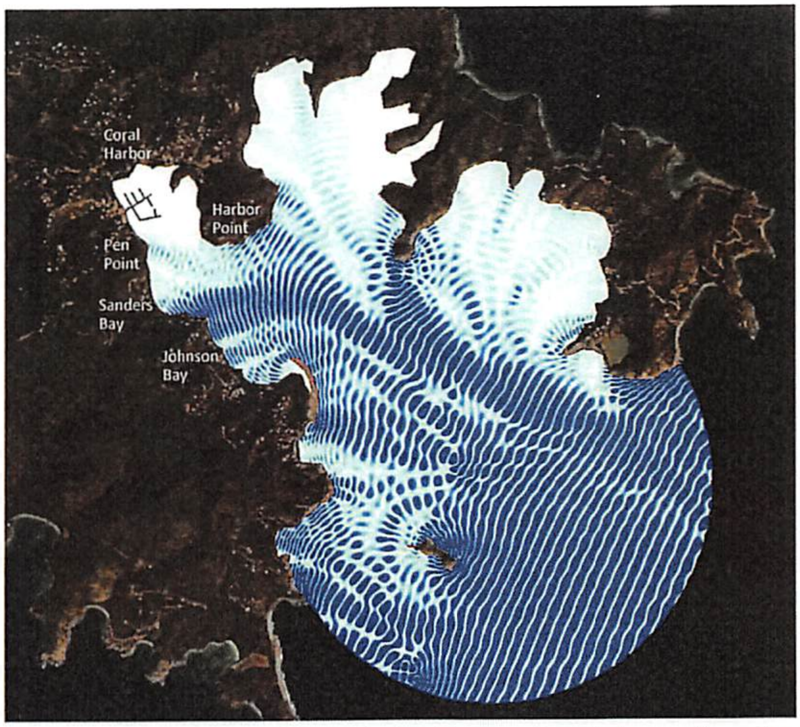

Figure 5-3. CGWAVE model results for 13-second period wave from ESE

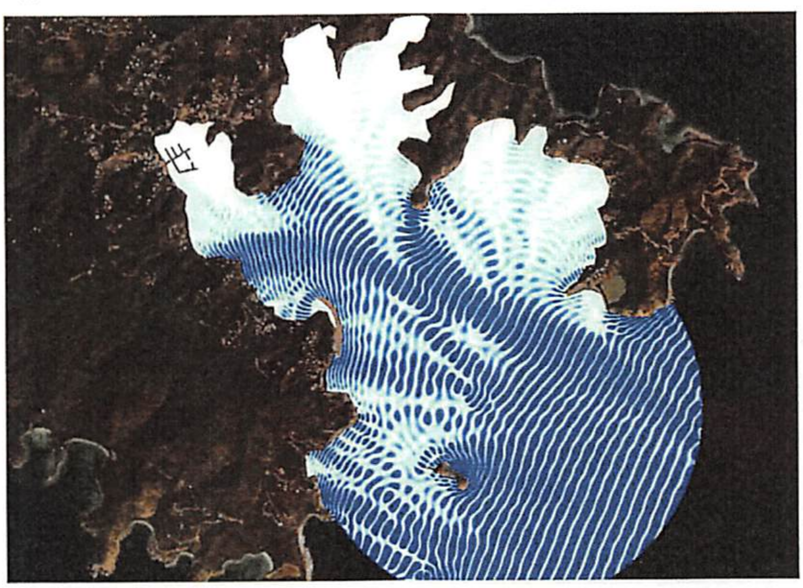

Figure 5-4. CGWAVE model results for 15-second period wave from ESE

from white capping, bottom friction, and depth-induced breaking. The SWAN wave model was set up using bathymetry digitized from the NOAA navigation chart for the USVI (chart #25641). The model was then used to simulate waves during winds from the east southeast, southeast and south southeast directions.

Example SWAN modeled significant wave heights are shown in Figure 5-5 for the 50-year return period wind from the south-southeast direction. The results for the various wave cases are shown in Table 5-4. In addition, the model was used to simulate the combined swell and wind wave conditions for the purposes of estimating wave setup in the harbor. The results for estimated wave setup are included in the water level estimates in Section 3.

5.2.2.3 Extreme Wave Conditions

The project site is an area where both offshore and local waves can exist and are propagating in the same direction (e.g., Hurricane Maria). As described in *Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters - Technical Memorandum* (FEMA 2008), for this scenario, the combined (swell and sea) wave height and combined wave period can be estimated as:

$$H \approx \sqrt{H_1^2 + H_2^2}$$

and

$$T \approx \frac{T_1 H_1^2 + T_2 H_2^2}{H_1^2 + H_2^2}$$

The estimated effective combined wave conditions are summarized in Table 5-5. The wave periods in this table include a range of values to account for the fact that the swell from the extreme storms may range from 11 to 15 seconds.

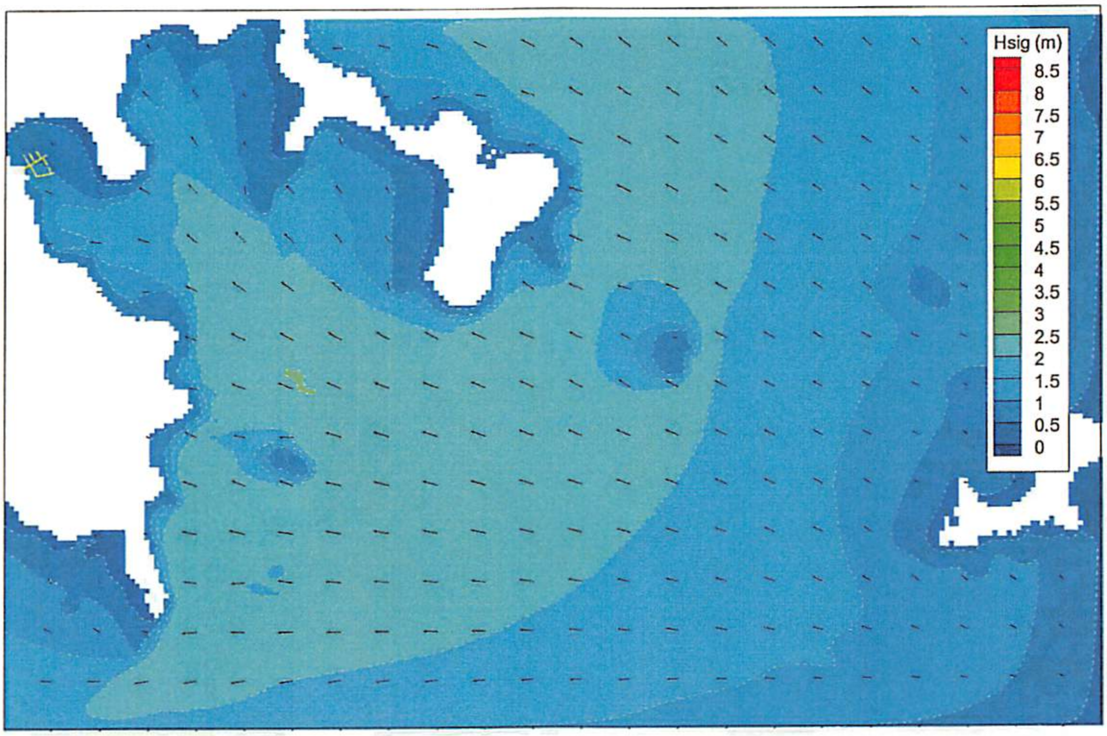

Figure 5-5. SWAN model results for 50-year return period wind from ESE

Table 5-4. SWAN modeled wind wave heights and periods at the project area

Return Period (yrs)	1-hour Wind Speed (mph)	Direction	Hs (ft)	Tp (s)
1	27	ESE	0.7[1]	2.0[1]
	27	SE	0.7[1]	2.0[1]
	27	SSE	0.4[1]	1.9[1]
10	44	ESE	2.3	2.3
	44	SE	2.3	2.3
	44	SSE	2.0	2.2
25	67	ESE	3.2	2.6
	67	SE	3.3	2.6
	67	SSE	3.1	2.5
50	78	ESE	3.7	2.7
	78	SE	3.8	2.7
	78	SSE	3.8	2.7

1. Interpolated from dataset

Table 5-5. Combined swell and wind wave heights and periods

Return Period (yrs)	Hs (ft)	Tp (s)
1	1.6	8.2 – 11.0
10	3.9	8.0 – 10.7
25	5.1	7.5 – 9.8
50	5.9	7.5 – 9.9

6 Currents

6.1 Typical Currents

Currents during typical astronomical tides and winds were measured at the project site using a current meter and were found to be on the order of 0.1 m/s, or 0.3 ft/s (Applied Technology and Management et al. 2014). Bioimpacts Inc (2017) measured currents at the site over a 2-year period intermittently spanning from December 2015 until June 2017 and concluded the currents at site were primarily influenced by tidal fluctuations and winds. The highest recorded current measurement was 0.6 ft/s with an average current less than 0.3 ft/s. Table 6-1 summarizes their measurements.

Table 6-1. Measured Currents at Site (Taken from Bioimpacts 2017)

Month/Year	CURRENTS			Tidal State	Winds
	18° 20.649'N 64° 42.847'W	18° 20.598'N 64° 42.824'W	18° 20.555'N 64° 42.804'W		
June-17	0.3ft/sec SW	0.3ft/sec W	0.3ft/sec SW	falling	E
	0.4ft/sec SW	0.2ft/sec SW	0.3ft/sec SW	falling	E
	0.3ft/sec SW	0.2ft/sec SW	0.3ft/sec SW	falling	E
	0.3ft/sec SW	0.2ft/sec SW	0.3ft/sec SW	falling	E
May-17	0.4ft/sec NNW	0.5ft/sec NNW	0.5ft/sec NNW	rising	SE
	0.4ft/sec NNW	0.6ft/sec NNW	0.5ft/sec NNW	rising	SE
	0.4ft/sec NNW	0.2ft/sec NW	0.5ft/sec NNW	rising	E
	0.6ft/sec NW	0.5ft/sec NW	0.4ft/sec NW	rising	E
April-17	0.2ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	NE
	0.2ft/sec SW	0.2ft/sec SW	0.3ft/sec SW	falling	NE
	0.1ft/sec W	0.1ft/sec W	0.3ft/sec NW	rising	E
	0.2ft/sec W	0.1ft/sec NW	0.3ft/sec NW	rising	E
	0.2ft/sec SW	0.2ft/sec W	0.3ft/sec SW	falling	SE
March-17	0.3ft/sec NW	0.4ft/sec NW	0.3ft/sec NW	rising	NE
	0.2ft/sec NW	0.2ft/sec NNW	0.4ft/sec NW	rising	NE
September-16	0.5ft/sec NNW	0.4ft/sec NNW	0.4ft/sec NNW	rising	E
	0.3ft/sec W	0.3ft/sec W	0.3ft/sec SW	falling	E
	0.4ft/sec NNW	0.3ft/sec NW	0.3ft/sec NNW	rising	SE
August-16	0.1ft/sec W	0.1ft/sec SW	0.1ft/sec NW	slack	SE
	0.1ft/sec SW	0.0ft/sec	0.2ft/sec NW	slack	SE
	0.5ft/sec NNW	0.5ft/sec NNW	0.5ft/sec NNW	rising	SE
	0.4ft/sec NW	0.5ft/sec NNW	0.5ft/sec NW	rising	SE
	0.2ft/sec W	0.2ft/sec SW	0.2ft/sec SSW	falling	E
July-16	0.5ft/sec NW	0.5ft/sec NW	0.4ft/sec NW	rising	SE
	0.3ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	SE
	0.2ft/sec SW	0.2ft/sec SW	0.2ft/sec SW	falling	SE
	0.1ft/sec SW	0.1ft/sec SSW	0.2ft/sec SSW	falling	E
June-16	0.3ft/sec SW	0.2ft/sec SW	0.4ft/sec SW	falling	SE
	0.4ft/sec NNW	0.5ft/sec NNW	0.7ft/sec NNW	rising	SE
	0.3ft/sec NW	0.4ft/sec NW	0.5ft/sec NNW	rising	E
May-16	0.4ft/sec NW	0.2ft/sec NW	0.4ft/sec NW	rising	E
	0.6ft/sec NW	0.5ft/sec NNW	0.4ft/sec NNW	rising	SE
	0.3ft/sec SSW	0.3ft/sec SSW	0.3ft/sec SSW	falling	SE
	0.2ft/sec SW	0.2ft/sec SSW	0.2ft/sec SSW	falling	SE
February-16	0.3ft/sec SSW	0.2ft/sec SSW	0.3ft/sec SSW	falling	ESE
	0.3ft/sec SW	0.2ft/sec SW	0.4ft/sec SW	falling	NE
January-16	0.4ft/sec WNW	0.5ft/sec NW	0.5ft/sec NNW	rising	NE
	0.4ft/sec NW	0.4ft/sec NW	0.4ft/sec NNW	rising	NE
	0.5ft/sec NW	0.5ft/sec NNW	0.4ft/sec NW	rising	NE
December-16	0.2ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	NE
	0.2ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	NE
	0.3ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	NE
	0.3ft/sec SW	0.3ft/sec SW	0.3ft/sec SW	falling	E
	0.4ft/sec NW	0.5ft/sec NNW	0.5ft/sec NW	rising	NE
	0.3ft/sec NNW	0.4ft/sec NW	0.5ft/sec NW	rising	NE
	0.4ft/sec NW	0.4ft/sec NNW	0.3ft/sec W	rising	NNE
	0.4ft/sec SSW	0.5ft/sec SW	0.4ft/sec SW	rising	NNE

6.2 Extreme Currents

Although typical wind and tide currents are small, extreme winds can result in much larger current speeds. The currents generated by hurricane wind speeds are complex and include currents it the direction of the surface wind, and as the wind generates a setup in the water levels, it also results in bottom currents in the opposite direction as the wind direction (Sheppard 2003). A three-dimensional hydrodynamic model would provide more accurate estimates of these currents under extreme wind conditions, but that level of analysis is beyond the scope of this study.

To provide a conservative estimate for this analysis, the bottom current in the opposite direction is neglected, and it is assumed that a fully-developed equilibrium logarithmic current profile forms in the direction of the wind (something that may or may not happen during hurricane conditions). An equation for depth-averaged alongshore current velocity is given by Dean and Dalrymple (1991):

$$V = \sqrt{\frac{8k \sin\theta}{f}} W \tan h \left(\frac{kf \sin\theta}{8} \frac{Wt}{h}\right)$$

where

W is the wind speed,
θ is the angle to the shoreline,
t is time;
h is the water depth,
f is the Darcy-Weisbach friction factor, and
k is the friction factor on the order of 10^{-6}.

The calculated depth-averaged current velocities reach equilibrium within 1 to 2 hours. These velocities are listed in Table 6-2. The resultant velocities are approximately 3 percent of the wind speed.

Table 6-2. Extreme depth-averaged current velocities

Return Period (yrs)	V (m/s)	V (ft/s)
1	0.3	1.0
10	0.6	2.0
25	0.9	3.0
50	1.1	3.6

7 Discussion & Recommendations

Using the site-specific met-ocean statistics presented above, the suitability of the site for marina development (specifically, the wave climate conditions for marina berthing and the need for additional wave protection) is evaluated to ensure the proposed marina configuration provides safe berthing for recreational boats. For reference, Figure 7-1 illustrates the proposed marina site plan overlain on the bathymetry.

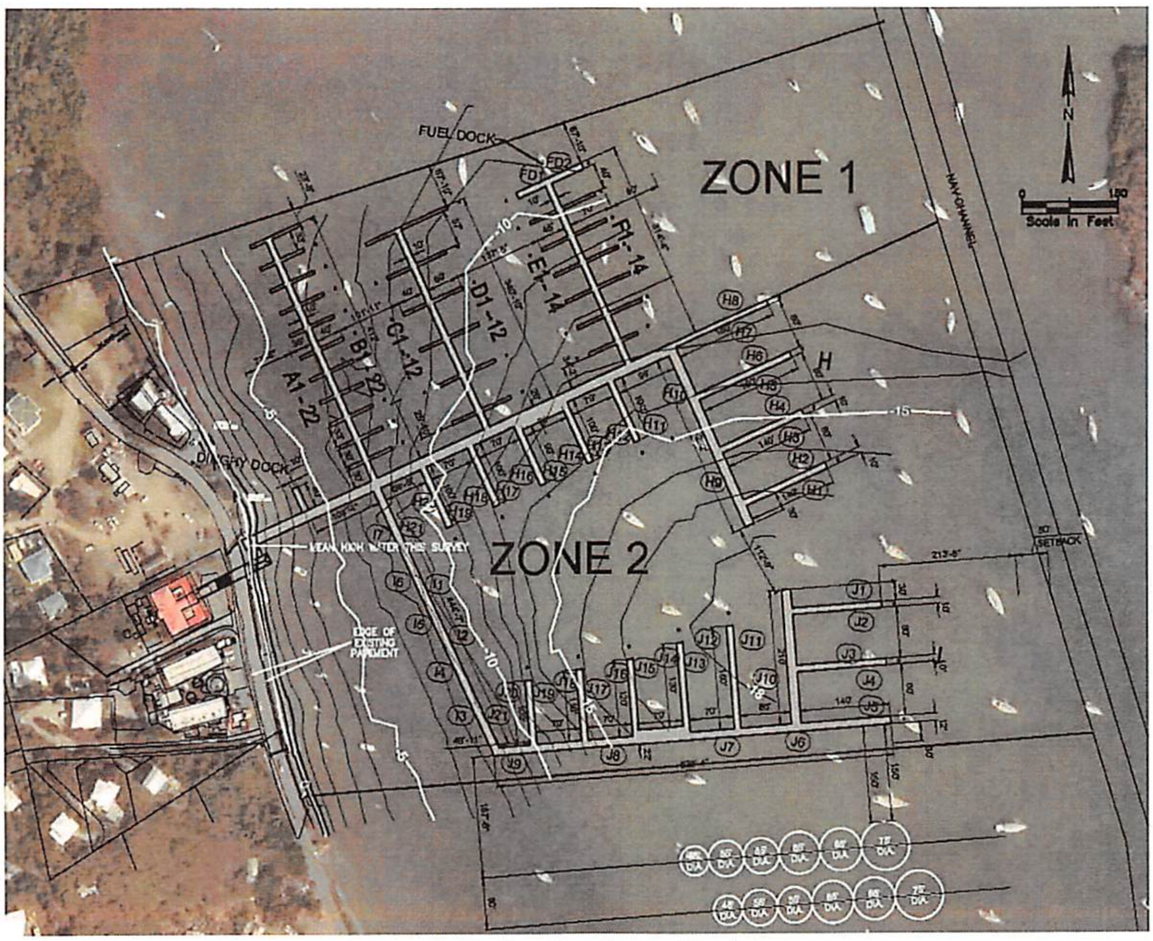

Figure 7-1. Marina site plan and bathymetry

7.1 Criteria

Table 7-1 and Table 7-2 below outline the small craft harbor wave tranquility criteria for good conditions and moderate conditions, respectively (ASCE 2012). Notably, these criteria are recommended guidelines and not absolute requirements; nevertheless, it is typical industry practice to design based on these criteria. As noted by ASCE (2012), "these criteria are far more stringent than those commonly accepted for craft left anchored freely in a protected embayment because the interaction of the vessel and the dock must also be considered." In the most simplistic sense, a 50-year return period design wave height exceeding the 2.0 – 2.5 ft criterion may cause damage to the facility, and a 1-year return period design

Table 7-1. Marina basin wave tranquility criteria for good conditions

Wave Direction Relative to Vessel	Significant Wave Height (ft) Not exceeded more than once per: Week	Year	50 Years
Head	0.5	1.0	2.0
Beam	0.3	0.5	0.8

Notes: 1. Multiply wave heights by 0.75 for "excellent" and 1.25 for "moderate" conditions.
2. For wave periods > 2 seconds.

Table 7-2. Marina basin wave tranquility criteria for moderate conditions

Wave Direction Relative to Vessel	Significant Wave Height (ft) Not exceeded more than once per: Week	Year	50 Years
Head	0.6	1.2	2.5
Beam	0.3	0.6	0.9

Notes: 1. For wave periods > 2 seconds.

wave height exceeding 1.0 – 1.2 ft will result in unacceptable operational conditions. The values presented in the tables above assume some level of vessel occupancy during storm events and are sensitive to vessel/dock orientation to incident wave direction. Floating docks are more vulnerable to damage by wave action than fixed docks. Commercially available floating dock design limits are typically for a 2.5-foot wave height, and this agrees with the 50-year head orientation under moderate conditions. The proposed marina will utilize fixed docks and can be designed to withstand greater wave heights than floating docks.

7.2 Site Suitability

The estimated extreme winds, water levels and currents are within the standard design range for most internationally recognized marina dock systems. However, the estimated wave conditions at the project site exceed the tranquility limits provided above.

In regard to operational conditions, the combined 1-year return period significant wave height was estimated to be 1.6 ft, whereas the limit was 1.0 – 1.2 ft for head-oriented berths and 0.5 – 0.6 ft for beam-oriented berths. The 1-year return period is often considered the operational (serviceability) limit to ensure safe mooring of vessels. While the 1-year return period wave heights have been estimated, the number of days that the wave heights would exceed the operational criteria is unknown and is beyond the scope of this study. Exceedance of the operational criteria (i.e., the 1-year wave criteria) over several days is much different than exceedance of the operational criteria for only a few hours. The operational impact on moored vessels would be substantially different as well. To ensure the operational criteria are not exceeded by the 1-yr return period event, additional infrastructure would be required (e.g., a floating wave attenuator). Alternatively, the Owner may accept the risk that the operational criteria may be exceeded one or more days per year. Estimation of the number of days per year that the operational criteria are exceeded would require additional analysis.

In regard to damage during hurricane events, the combined 50-year return period significant wave height was estimated to be 5.9 ft, whereas the limit was 2.0 – 2.5 ft for head-oriented berths and 0.8 – 0.9 ft for beam-oriented berths. The 50-year return period criteria are typically considered the design limits to avoid damage to the floating docks and related marina infrastructure. The proposed facility will utilize fixed docks. While the marina would not be suitable for vessel occupancy during extreme hurricane events, the survivability of the fixed docks will be dependent on the design level selected for the engineering design of the docks.

Based on the marina basin tranquility criteria, additional infrastructure improvements would be required to achieve conditions within the berthing guidelines; however, in review of the findings above, one must also consider the following points:

- The site-specific wind, wave and water level estimates above are based on limited modeling and analysis. Where possible, results have erred to the conservative (worst-case assumption); however, it is not possible to conclusively state whether all of the findings are entirely conservative based on the aforementioned analysis. A higher level of confidence in the findings would require more thorough analysis of the site supported by in-situ measurement data, all of which has been excluded from the present study.
- Site observations over a 9-month period and other anecdotal evidence from Bioimpacts (e.g., wave heights never greater than 1 ft during site visits) suggest that the estimated 1-year operational wave condition is conservative, and operational conditions at the marina would rarely be exceeded during typical annual conditions. However, these are visual observations (by others) and not wholly-accurate wave height measurements, and they are limited to observations on a small fraction of days during the year.
- A floating attenuator would partially improve the berth quiescent conditions; specifically, attenuating locally-generated wind wave conditions during operational conditions. Floating attenuators are very useful at attenuating a high percentage of the incident wave energy when wave periods range between 1 – 3 seconds. Such an attenuator would eliminate nominally 60% - 90% of the 1-year wind wave height estimate and further reduce its contribution to the combined 1-year sea-swell conditions estimated for the project area. A floating attenuator would do little to minimize the incident swell conditions as the wave period is beyond the performance range of these units.
- The operational criteria for berth tranquility of small craft harbors were developed based on berthing of the most common (length) range of recreational boats – boats ranging in length between nominally 15 ft – 45 ft. Obviously, smaller boats will be more susceptible to excessive movements from incident waves whereas large megayachts (80-ft and greater in length) will experience little movement to the same incident wave condition. The smallest berth in the proposed marina is 36 ft while the average size berth is approximately 60 – 70ft, and the largest berth is 140 ft in length. Given that the vessel size distribution for the proposed marina is skewed to much larger size vessels, one could argue a relaxation in the design criteria; specifically, in the context of the operational (weekly and 1-year return period) criteria.

- The proposed marina plan has a dock arrangement such that the largest vessels (megayachts) are positioned towards the seaward-end of the marina plan, and the smallest vessels are positioned towards the landward-end of the marina plan in the lee of the megayachts. During periods of high occupancy, these megayachts will attenuate incident waves, which will further reduce the wave conditions towards the leeward-end of the marina where smaller boats are moored.
- Without question, the site is not exempt from hurricane force conditions, as recent hurricanes (Irma and Maria), which left Coral Harbor completely destroyed, have unfortunately provided a recent reminder of this point. In order to mitigate the direct impact of the 50-year wave condition, additional coastal infrastructure (e.g., a rubble mound breakwater) would be required. Note that such infrastructure improvements would only mitigate the incident storm waves; hurricane force winds and water levels could still manage to destroy the marina facility without damaging waves.
- While there is a 2% annual risk of a 50-year return period storm occurring in a given year, the probability of a 50-year event occurring at some point over an assumed 25-year design life of the marina is 40%. Table 7-3 outlines the probabilities that an event will be exceeded over various periods of time. Exceedance of the extreme (design) event will cause damage and/or failure of the marina works. The Owner, in consultation with other professionals, should determine the amount of acceptable risk for the marina facility, and then determine how to best mitigate the risk though physical risk reduction measures (e.g., designing to survive extreme events, incorporating factors of safety, etc.) and/or risk management through insurance. Consideration of the total project cost over the life of the facility is a helpful method for evaluating design alternatives, particularly because physical risk reduction measures will increase initial project capital costs but they will also reduce long-term costs and could result in lower annualized costs. Such analysis is beyond the present study.

Table 7-3. Probability of natural hazard event occurrence for various periods of time (FEMA 2011)

Length of Period (Years)	Frequency – Recurrence Interval					
	10-Year	25-Year	50-Year	100-Year	500-Year	700-Year
1	10%	4%	2%	1%	0.2%	0.1%
10	65%	34%	18%	10%	2%	1%
20	88%	56%	33%	18%	4%	3%
25	93%	64%	40%	22%	5%	4%
30	96%	71%	45%	**26%**	6%	4%
50	99+%	87%	64%	39%	10%	7%
70	99.94+%	94%	76%	**51%**	13%	10%
100	99.99+%	98%	87%	63%	18%	13%

The percentages shown represent the probabilities of one or more occurrences of an event of a given magnitude or larger within the specified period. The formula for determining these probabilities is $P_n = 1-(1-P_a)^n$, where P_a = the annual probability and n = the length of the period.

The bold blue text in the table reflects the numbers used in the example in this section.

7.3 Summary

All things considered, the site location is relatively protected with its only wave exposure being to the southeast direction. Wave modeling results demonstrate that ocean waves from the southeast are refracted such that most of the wave energy is directed towards headlands south of the marina site, which greatly reduces the wave exposure from the southeast. Additionally, site observations (by others) indicate that the site is a very quiescent location during typical operational conditions. The estimated 1-year return period condition exceeds the design guidelines for operational conditions, although this is expected to occur infrequently. The number of days that the wave heights would exceed the operational criteria is unknown and is beyond the scope of this study. If the Owner needs to ensure the operational criteria are not exceeded (other than during hurricane events), then additional infrastructure would be required (e.g., a floating wave attenuator). Alternatively, the Owner may accept the risk that the operational criteria may be exceeded one or more days per year. Overall, the site is expected to provide safe berthing for recreational boats during operational conditions except for a small fraction of the time.

Extreme hurricane conditions, such as the 50-yr return period storm, will result in wave heights exceeding the marina tranquility standards. The Owner, in consultation with other professionals, should determine the amount of acceptable risk for the marina facility, and then determine how to best mitigate the risk though physical risk reduction measures (e.g., designing to survive extreme events, incorporating factors of safety, etc.) and/or risk management through insurance.

References

Applied Technology & Management, Inc., Bioimpact, Inc., and Cairone and Kaupp, Inc. 2014. Earth Change Plan & Environmental Assessment Report for the Development of the Upland Areas of the St. John Marina, Coral Bay, St. John, USVI. April 2014.

ASCE. 2010. Minimum Design Loads of Buildings and Other Structures. ASCE Standard ASCE 7-10.

ASCE. 2012. Planning and Design Guidelines for Small Craft Harbors. ASCE Manuals and Reports on Engineering Practice No 50. 3rd Edition.

Bioimpacts Inc. 2017. Appendix C-1 Response to NMFS PRD comments.

Dean, R.G. and R.A. Dalrymple. 1991. Water Wave Mechanics for Engineers and Scientists. World Scientific. p 162.

FEMA. 2007. Flood Insurance Study, U.S. Virgin Islands. Flood Insurance Study number 780000V000A. Revised April 16, 2007.

FEMA. 2008. Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters - Technical Memorandum.

Panchang, V. G. and B. Xu, 1995. "CGWAVE: A Coastal Wave Transformation Model for Arbitrary Domains," Technical Report, Dept. of Civil and Environmental Engineering, University of Maine, Oct. 1995.

Lyman. D. 2016. Experiencing Hurricane Hugo. Available: http://www.dhlyman.com/DHLyman.com/Hurricane_Hugo.html (April 2016)

National Oceanic and Atmospheric Administration. 2007. Errata Sheet to the Service Assessment Report Hurricane Hugo September 10-22, 1989.

National Weather Service. 2017. *"Major Hurricane Maria"*. SFO San Juan, November 22, 2017.

Sheppard. M. 2003. Wind Generated Currents in Shallow Water. Research project completed for Florida Department of Transportation. Available: http://www.dot.state.fl.us/rddesign/Drainage/Research/CE/Wind-Generated-Currents.pdf (April 2016)

Torres-Sierra, H. Storm-Tide Elevations Caused by Hurricane Marilyn on the U.S. Virgin Islands, September 15-16, 1995. U.S. Geological Survey Open-File Report 96-44.

USACE. 2008. Coastal Engineering Manual. Engineering Manual EM-1110-2-1100.