ST. JOHNS COUNTY, FLORIDA
South Ponte Vedra Beach, Vilano Beach, and Summer Haven Reaches

COASTAL STORM RISK MANAGEMENT PROJECT
DRAFT INTEGRATED FEASIBILITY STUDY AND ENVIRONMENTAL ASSESSMENT

APPENDIX A
Engineering

February 2016

US Army Corps of Engineers
Jacksonville District

U.S. ARMY CORPS OF ENGINEERS
JACKSONVILLE DISTRICT

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Background

St. Johns County is located on the northeast coast of Florida approximately midway between the Florida-Georgia state line and Cape Canaveral. The County is bounded to the north by Duval County and to the south by Flagler County. St. Johns County has approximately 42 miles of coastal shoreline, with about 24 miles extending from the Duval County line to St. Augustine Inlet, about 14 miles from St. Augustine Inlet to Matanzas Inlet, and about 3 miles from Matanzas Inlet to the Flagler County line. South Ponte Vedra Beach and Vilano Beach (denoted SPV and Vilano throughout this appendix) are located on the north side of St. Augustine Inlet and are the focus of the present study (Figure A - 1). Anastasia State Park and St. Augustine Beach are located south of the St. Augustine Inlet. Summer Haven, initially part of the study but screened out due to reasons described in the main report, is located south of Matanzas Inlet. A Federal Shore Protection Project is authorized for a contiguous portion of St. Augustine Beach and currently uses St. Augustine Inlet as a borrow source. The purpose of this study is to assess the feasibility of providing Federal Coastal Storm Risk Management (CSRM) measures to additional portions of the St. Johns County shoreline.

Problem Identification

Historically, beaches of St. Johns County have generally experienced substantial erosion due to the combined effects of winds, waves, and tides. The severity of erosion in some areas is indicated by the presence of protective structures such as seawalls and revetments, and the absence of any beach seaward of those protective structures. The objectives of the engineering analysis include the quantification of existing beach erosion and the design of corrective measures. Quantification efforts involve analysis of historical shoreline positions, estimation of alongshore sediment transport rates, and prediction of cross-shore losses of beach material due to storms. The results of those efforts serve as the basis for the design and analysis of beach nourishment measures, which could be employed to reduce storm damage in the project area.

Natural Forces

Winds

Local winds are the primary means of generating the small-amplitude, short period waves that are an important mechanism of sand transport along the Florida shoreline. Predominant winds from the east-southeast quadrant are generally mild in nature and occur in the spring and summer months. Elevated wind speeds from the north-northeast quadrant in fall and winter months occur during passage of northeasters which can cause extensive beach erosion and shorefront damage. Occasionally the area is impacted by the passage of tropical storms that can generate devastating winds, waves, and storm surge, which can cause direct damage to coastal structures and infrastructure or heavy erosion that can result in the undermining of coastal structures.

Wind data offshore of the project area is available from the U.S. Army Corps of Engineers (USACE) Wave Information Study (WIS) Program. WIS hindcast data are generated using the numerical hindcast model WISWAVE (Hubertz, 1992). The WIS hindcast database includes significant wave height, peak and mean wave period, peak and mean wave direction, wind speed, and wind direction. WISWAVE inputs include time varying wind fields overlaying a bathymetric grid.

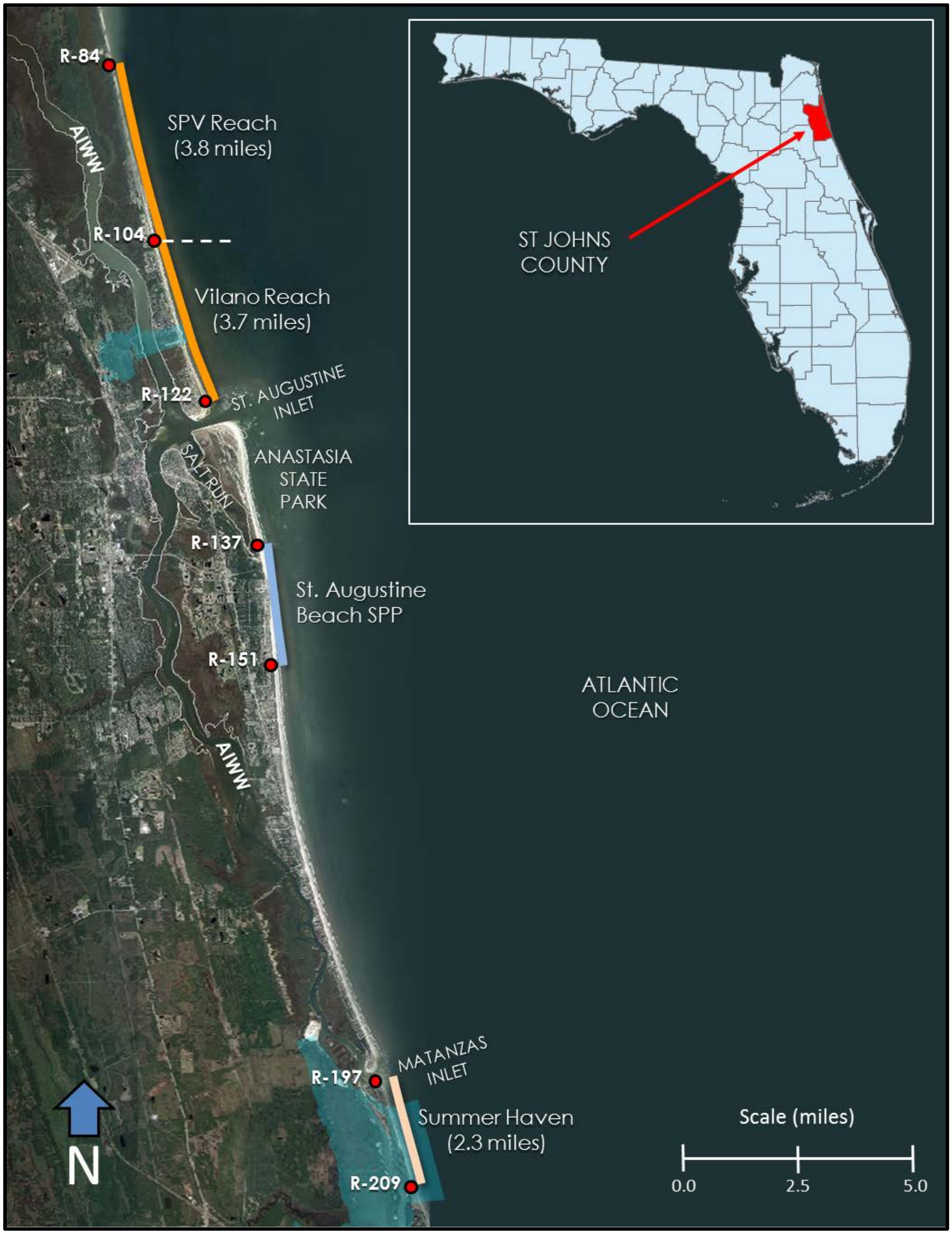

Figure A - 1. Project area and details.

There are 523 WIS stations along the Atlantic Coast. WIS Station 63416 is representative of offshore deep water wind and wave conditions for the project area since it is the closest station to the study area. As such, data presented in this section and the Waves section to follow use data output from Station 63416. Figure A - 2 provides a summary of wind data from WIS Station 63416, located at latitude 30.0° and longitude -81.08° (about 13.5 miles east of the project area; Figure A - 3), and shows the frequency of occurrence of wind speeds broken down into sixteen 22.5 degree angle-bands. The distribution of winds are consistent for all degree bands with percent occurrence varying from 4.5% to 8.3%. However, there is a slight increase of occurrence in wind from the east-northeast and south-southwest directions as seen in the wind rose in Figure A - 4. Note that generalizations of the wind fields in this section pertain to those directions that are important with respect to sediment transport. Therefore, offshore-directed winds may show greater frequency at the offshore WIS Station 63416 but are ignored when generalizing conditions.

Wind conditions in Coastal Florida are seasonal. A further breakdown of the wind data provides a summary of the seasonal conditions (Table A - 1). Between December and March, frontal weather patterns driven by cold Arctic air masses can extend as far as South Florida. These fronts typically generate northeast winds before the frontal passage and northwest winds behind the front. The northeaster behavior is responsible for the increased intensity of wind speed seen in the northeast sector winds during the winter months. Northeasters may result in wave conditions that can cause extensive beach erosion and shorefront damage.

ATLANTIC HINDCAST WAM4.5.1C : ST63416_v03
ALL MONTHS FOR YEARS PROCESSED : 1980 - 2012
STATION LOCATION : (-81.08 W / 30.00 N)
DEPTH : 22.0 m

PERCENT OCCURRENCE (X1000) OF WIND SPEED AND DIRECTION
CENTRAL LOCAL ANGLE BANDS OF (+/- 11.25 DEG)

NO. CASES : 289294

WND DIR DEG	WIND SPEED (M/S) <2.5	2.5-4.9	5.0-7.4	7.5-9.9	10.0-12.4	12.5-14.9	15.0-17.4	17.5-19.9	20.0-24.9	25.0-GREATER	TOTAL
0.0	242	1423	1864	1389	592	179	30	8	1	0	5728
22.5	254	1674	1960	1358	656	139	29	9	0	0	6079
45.0	293	2034	2449	1529	529	109	8	1	0	0	6952
67.5	328	2341	2760	1410	347	79	15	3	2	0	7285
90.0	406	2868	2543	871	163	48	8	2	2	0	6911
112.5	457	2934	2043	518	106	30	2	1	2	0	6093
135.0	518	3055	1900	430	98	29	3	2	1	0	6036
157.5	529	3273	1995	542	151	26	4	2	0	0	6522
180.0	587	3678	2821	926	253	54	8	0	0	0	8327
202.5	477	3404	2757	908	234	48	5	0	1	0	7834
225.0	396	3169	2683	795	218	41	4	0	2	0	7308
247.5	331	2330	1910	695	265	73	6	0	1	0	5611
270.0	276	1570	1313	776	426	121	17	2	2	0	4503
292.5	235	1160	1202	1022	711	274	57	2	0	0	4663
315.0	212	1138	1444	1276	781	259	37	3	1	0	5151
337.5	234	1181	1456	1248	648	133	23	1	3	0	4927
TOTAL	5775	37232	33100	15693	6178	1642	256	36	18	0	

MEAN WS (M/S) = 5.8 MAX WS (M/S) = 24.6 MEAN WIND DIR (DEG) = 351.0 FINITE

Figure A - 2. Wind percent occurrence by magnitude and direction.

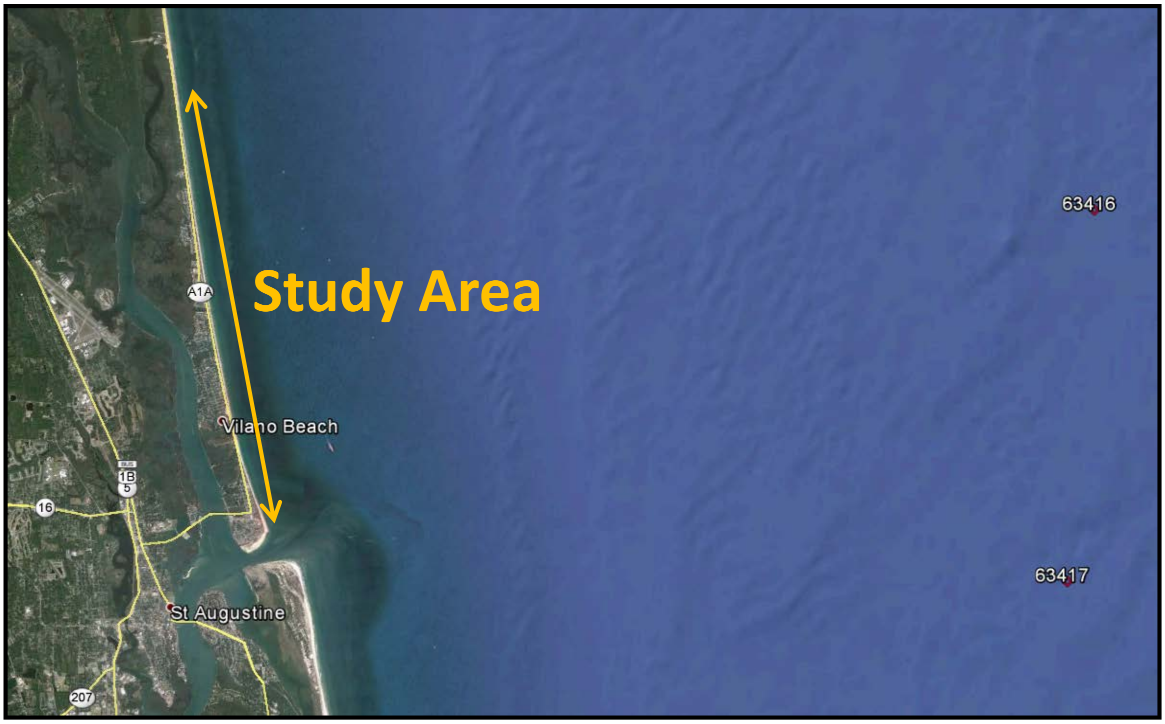

Figure A - 3. Location of WIS Station #63416 relative to study area.

The summer months (June through September) are characterized by southeast trade winds and tropical weather systems traveling toward the west and toward the northwest in the lower latitudes. Additionally, daily breezes onshore and offshore result from differential heating of land and water masses. These diurnal winds typically blow perpendicular to the shoreline and have less magnitude than Trade winds and northeasters. Daily breezes account for the general shift to east/southeast winds during the summer months when northeasters no longer dominate.

During the summer and fall months, tropical waves may develop into tropical storms and hurricanes, which can generate devastating winds, waves, and storm surge when they impact the project area. These storms contribute greatly to the overall alongshore and cross-shore sediment transport at the site. These intense seasonal events will be discussed in greater detail under the Storm Effects section.

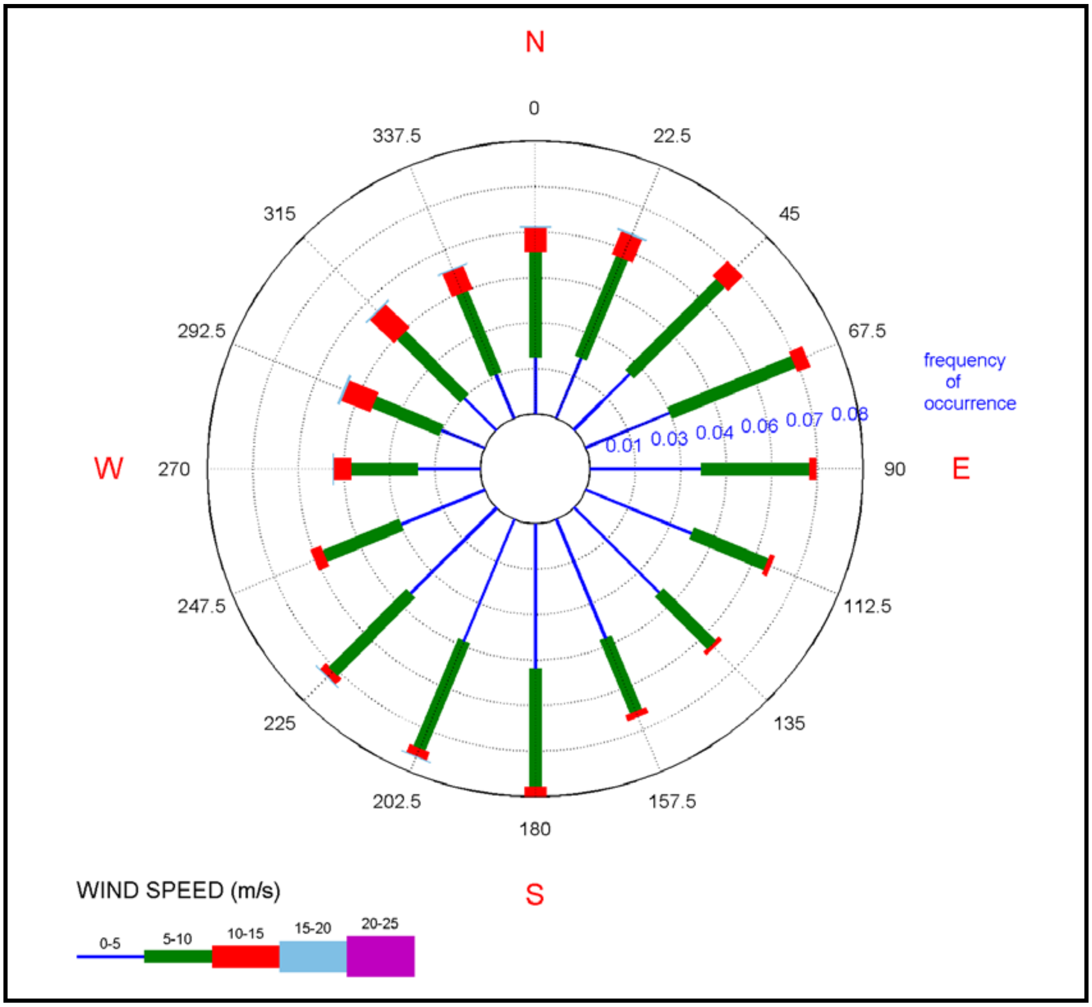

Figure A - 4. Wind rose for WIS Station 63416.

Table A - 1. Seasonal wind conditions.

Month	WIS Station #63416 (1980 – 2012)	
	Average Wind Speed (mph)	Predominant Direction (from)
January	15.4	NW
February	15.0	NW
March	14.3	S
April	13.0	S
May	11.4	S
June	10.7	SW
July	10.5	SW
August	10.3	SSW
September	12.1	ENE
October	14.1	NE
November	14.8	N
December	15.0	NW

Waves

The wave energy dissipation that occurs as waves enter the nearshore zone and break is the principal driver for sediment transport. Wave height, period, and direction, in combination with tides and storm surge, are the most important factors influencing the behavior of the beach and dune system. The SPV-Vilano study area is exposed to both short period wind-waves and longer period open-ocean swells originating predominantly from the northeast during spring, fall and winter months and from northeast to southeast during summer months.

Periodic erosion of the SPV-Vilano beach system and associated damage to upland development is attributable to large storm waves produced primarily by northeasters during the late fall and winter months and by tropical disturbances, including hurricanes, during the summer months. Because the study area is fully exposed to the open ocean in all seaward directions, the coastline is vulnerable to wave attack from distant storms (causing long period swells) as well as local storms (causing short period steep waves). Tropical storm passage is relatively frequent for the study area and even without landfall of a tropical storm, a system passing within several hundred miles may cause extensive erosion damage to the area.

Wave data for this report were obtained from the long-term USACE WIS hindcast database for the Atlantic coast of the U.S. This 33-year record extends from 1980 through 2012, and consists of a time-series of wave events at 3-hour intervals for stations located along the east and west coasts of the US, as well as the Gulf of Mexico and Great Lakes. The WIS station closest to the project area is #63416, located 13.5 miles offshore. The location of WIS station #63416 relative to the study area is shown previously in Figure A - 3.

Table A - 2 summarizes the percentage of occurrence and average wave height of the WIS waves by direction. It can be seen that the dominant wave direction is from the east with contributions from the east-northeast and east-southeast. This can be seen clearly in the wave rose presented in Figure A - 5. The total wave climate reflects both the open-ocean swell and more locally generated wind-waves.

Similar to wind conditions, wave conditions in Coastal Florida experience seasonal variability. The seasonal breakdown of wave heights is provided in Table A - 3 and shows that fall and winter months have an increase in wave height due to northeaster activity. The intensity and direction of these fall/winter wave conditions are reflected in the dominant southward transport of sediment and seasonal erosional patterns in the project area. In contrast, summer months experience milder conditions, with smaller wave heights with exception to the infrequent passage of a tropical cyclone. Overall, waves originating from the east to northeast quadrant dominate.

Table A - 2. Average wave heights (1980 to 2012).

Wave Direction (° True North)	Number of Occurrences	Percent Occurrence	Average Wave Height (ft)
0.0	4,884	1.7%	4.3
22.5	7,705	2.7%	4.3
45.0	20,266	7.0%	4.9
67.5	63,098	21.8%	4.9

Wave Direction (° True North)	Number of Occurrences	Percent Occurrence	Average Wave Height (ft)
90.0	110,585	38.2%	3.9
112.5	65,239	22.6%	3.3
135.0	8,359	2.9%	3.6
157.5	1,941	0.7%	3.6
180.0	849	0.3%	3.3
202.5	433	0.1%	3.3
225.0	348	0.1%	3.3
247.5	342	0.1%	3.3
270.0	471	0.2%	3.3
292.5	637	0.2%	3.6
315.0	1,451	0.5%	3.9
337.5	2,686	0.9%	4.3
Totals	**289,294**	**100.0%**	**3.9**

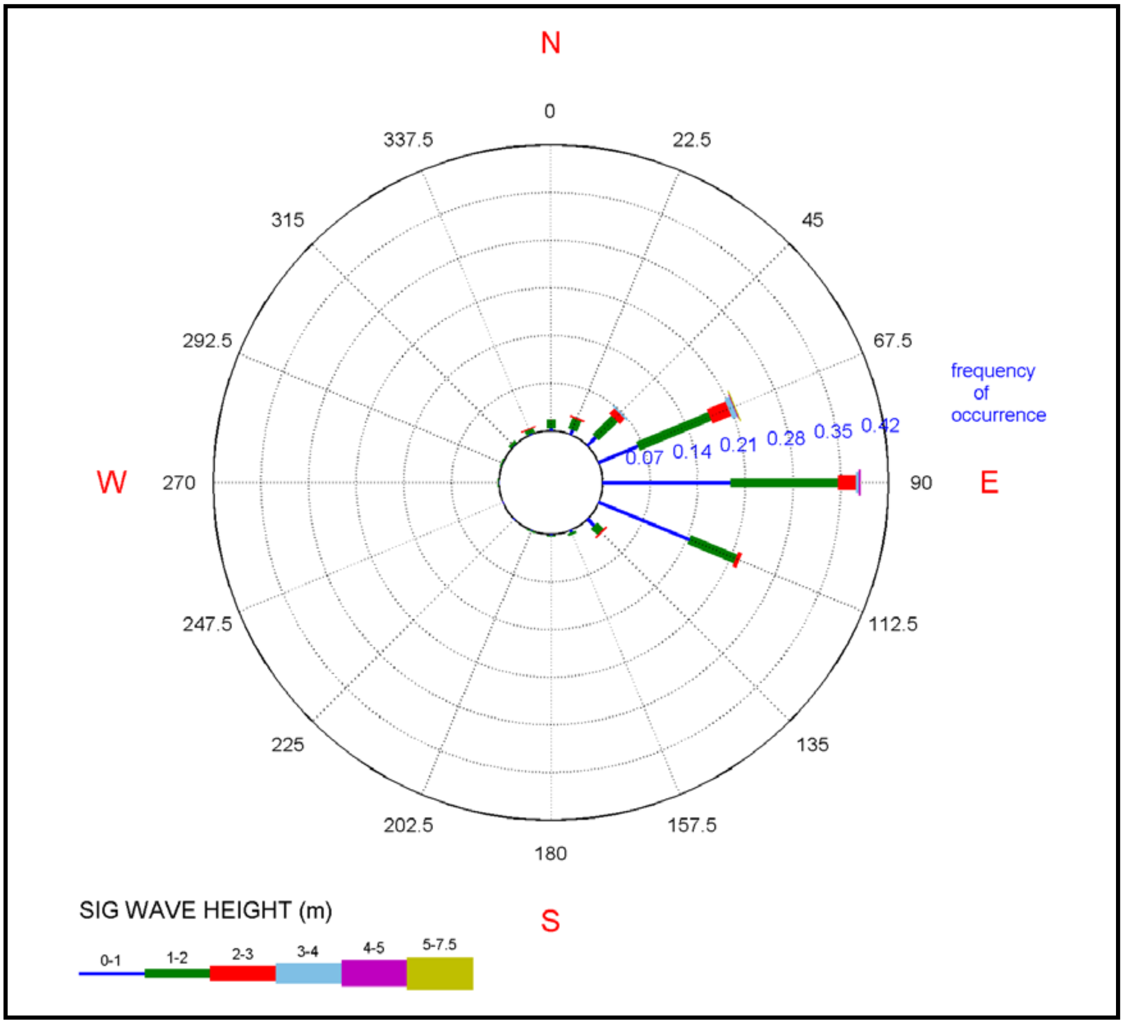

Figure A - 5. Wave rose for WIS Station 63416.

Table A - 3. Seasonal wave conditions.

Month	WIS Station #63416 (1980 – 2012)	
	Average Wave Height (ft)	Predominant Direction (from)
January	4.3	E
February	4.3	E
March	4.3	E
April	3.6	E
May	3.6	E
June	3.0	ESE
July	2.6	ESE
August	3.0	E
September	4.3	E
October	4.6	E
November	4.9	E
December	4.6	E

Tides and Currents

Astronomical tides are created by the gravitational effects of the moon and sun on the ocean and are well understood and predictable in magnitude and timing. The National Oceanic and Atmospheric Administration (NOAA) regularly publishes tide tables for selected locations along the coastlines of the United States and selected locations around the world. These tables provide times of high and low tides, as well as predicted tidal amplitudes.

Tides in St. Johns County area are semidiurnal, meaning two high tides and two low tides occur per tidal day. Tidal datums for St. Augustine Beach (NOAA station 8720587) and Vilano Beach ICWW (NOAA station 8720554) are summarized in Table A - 4 and Table A - 5, respectively. The St. Augustine Beach water level station was located on the St. Augustine Beach pier and represents open ocean water levels while the Vilano Beach water level station was located in the Intercoastal Waterway on the SR A1A bridge. The datums presented in Table A - 4 and Table A - 5 are based on tidal analysis periods of 6/1/1992 to 5/31/2000 and 10/1/2003 to 7/31/2004, respectively. The difference between Mean High Water (MHW) and Mean Low Water (MLW), known as the mean tide range, equals 4.61 ft at St. Augustine Beach and 4.24 feet at Vilano Beach.

Table A - 4. Tidal datums for St. Augustine Beach, FL.

Tidal Datum	Elevation Relative to NAVD88 (feet)
Mean Higher High Water (MHHW)	2.01
Mean High Water (MHW)	1.64
North American Vertical Datum (NAVD88)	0.00
Mean Tide Level (MSL)	-0.70
Mean Low Water (MLW)	-2.97
Mean Lower Low Water (MLLW)	-3.13

Table A - 5. Tidal datums for Vilano Beach, FL.

Tidal Datum	Elevation Relative to NAVD88 (feet)
Mean Higher High Water (MHHW)	1.86
Mean High Water (MHW)	1.53
North American Vertical Datum (NAVD88)	0.00
Mean Tide Level (MSL)	-0.56
Mean Low Water (MLW)	-2.71
Mean Lower Low Water (MLLW)	-2.89

Near-shore currents affect the supply and distribution of sediment on the sandy beaches of St. Johns County and are composed of alongshore and cross-shore components. Alongshore currents, induced by oblique wave energy, generally determine the long-term direction and magnitude of littoral transport. Cross-shore currents may have a more short term impact, but can result in both temporary and permanent erosion. The magnitude of these currents is determined by the wave characteristics, angle of waves from offshore, configuration of the beach, and the nearshore profile. For St. Johns County beaches, the net sediment transport is from north to south. This is due to the dominant wave activity from the northeast during the fall and winter months, particularly northeaster storms.

Since the southern portion of the study area is directly adjacent to the St. Augustine Inlet, currents are affected by the ebb and flood tidal flow through the inlet. The terminal groin structure on the north side of St. Augustine Inlet also provides varying degree of influence on nearshore currents depending on its exposure level. During the flood tide, currents along the beach within the inlet's area of influence will set with a larger inlet directed component (southerly in the study area) than areas outside the influence of the inlet. During the ebb tide, inlet-directed currents may still be present due to wave refraction around the ebb shoal as well as return flow eddies that develop from the ebb jet.

Storm Effects

The beaches of St. Johns County are influenced by tropical systems during the summer and fall and by northeasters during the late fall, winter, and spring. Although hurricanes typically generate larger waves and storm surge, northeasters typically have a greater cumulative impact on the shoreline due to longer storm duration and greater frequency of event occurrence.

Periodic and unpredictable hurricanes and coastal storms, with their energetic breaking waves and elevated water levels, can change the width and elevation of beaches and accelerate erosion (Figure A - 6). Storms erode and transport sediment from the subaerial beach into the active zone of storm waves. Once caught in the waves, this sediment is carried along the shore and re-deposited farther down the beach, or is carried offshore and stored temporarily in submerged sand bars. After storms pass, long period residual swell usually returns sediment from the sand bars to the beach, which is restored gradually to its natural equilibrium profile. While the beach profile typically recovers from storm energy as described, extreme storm events may cause sediment to leave the beach system entirely, sweeping it into inlets, into the back bay (over wash), or moving it far offshore into deep water where waves cannot return it to the beach. This causes the shoreline to recede, or move farther landward.

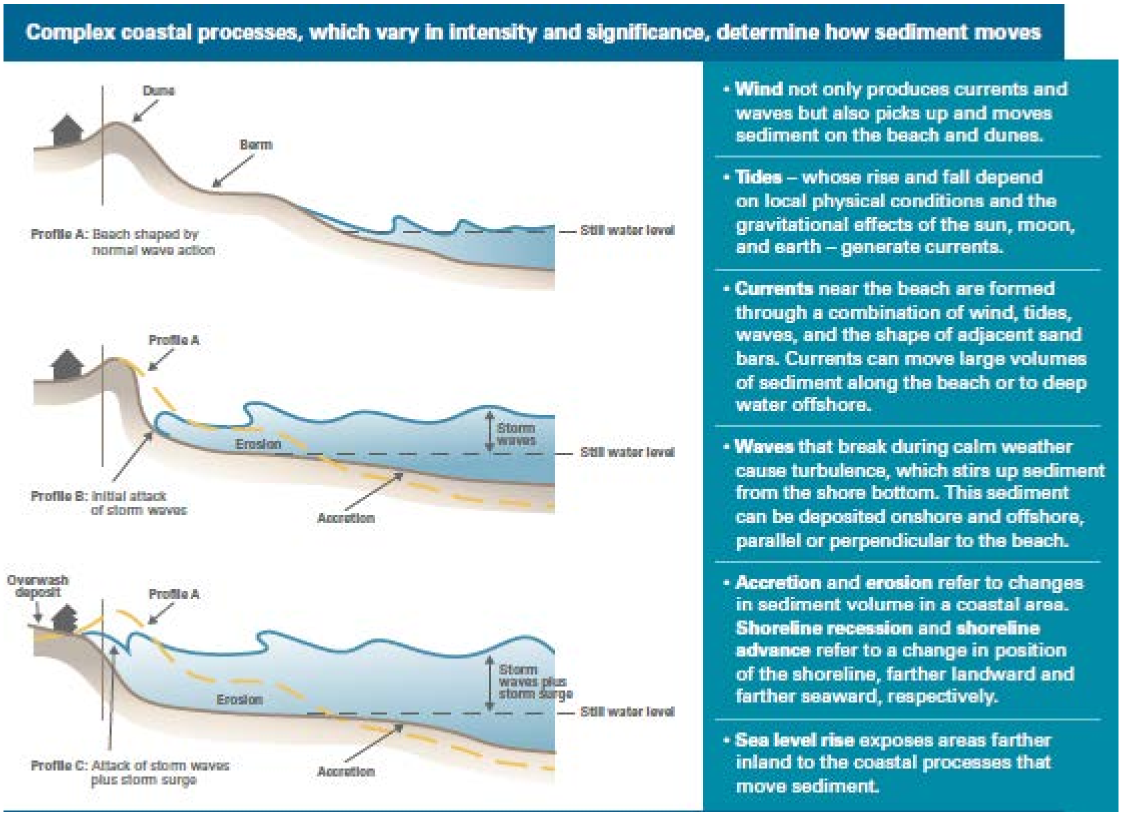

Figure A - 6. Typical coastal processes (from Shore Protection Assessment primer published by USACE-Engineer Research Development Center).

St. Johns County is located in an area of moderate hurricane activity. Figure A - 7 shows historic tracks of hurricanes and tropical storms from 1842 to 2014, as presented by the National Ocean Service (NOS) and available from the National Oceanic and Atmospheric Administration (NOAA, 2015). The shaded circle in the center of this figure indicates a 50-nautical mile radius (encompassing the entire St. Johns County shoreline) from St. Augustine Inlet. Based on National Hurricane Center (NHC) records, 18 hurricanes and 39 tropical storms have passed within this 50-mile radius over the 172-year period of record; or, one event of tropical storm force or greater passes within 50 miles of the study area every 3.0 years (NOAA, 2015). The 50-mile radius was chosen for display purposes in Figure A - 7 because any tropical disturbance passing within this distance, even a weak tropical storm, would likely cause erosive damage to the shoreline. Stronger storms are capable of producing significant damage to the coastline from far greater distances. Northern Florida has been affected by many hurricanes, but Hurricane Dora in 1964 may have had the greatest impact on St. Johns County. The storm surge in St. Augustine reached 12.0 feet (MSL-1929) during Dora (FDNR, 1987). The NHC data show that hurricanes and tropical storms pass within 50 nautical miles of the study area approximately every 3.0 years (NOAA, 2015). Shoreline erosion occurred during the series of hurricanes in 1999 that included Dennis, Floyd, and Irene. The northeasters of the 1960’s and of November 1984 are notable storms that caused intense erosion (Foster et al., 2000). Historic records indicate that an extra-tropical storm will impact the shoreline approximately 1.75 times per year (USACE, 1998).

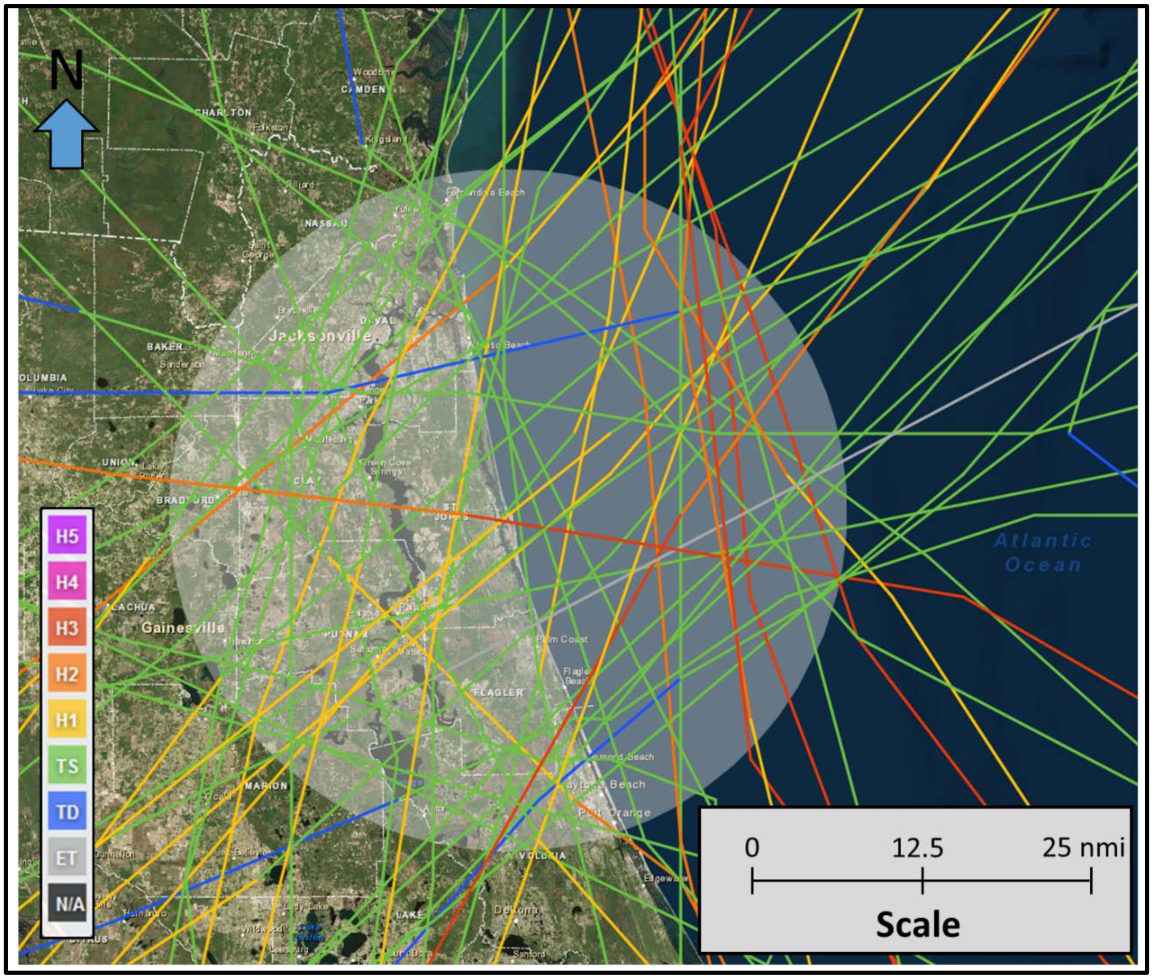

Figure A - 7. Historic tropical storm tracks passing within a 50-mile radius.

Storm Surge

Storm surge is defined as the rise of the ocean surface above its astronomical tide level due to physical forces. Surges occur primarily as a result of atmospheric pressure gradients and surface stresses created by wind blowing over a water surface. Strong onshore winds pile up water near the shoreline, resulting in elevated water levels along the coastal region and inland waterways. In addition, the lower atmospheric pressure which accompanies storms also contributes to a rise in water surface elevation. Extremely high wind velocities coupled with low barometric pressures (such as those experienced in tropical storms, hurricanes, and very strong northeasters) can produce very high, damaging water levels. In addition to wind speed, direction and duration, storm surge is also influenced by water depth, length of fetch (distance over water), and frictional characteristics of the nearshore sea bottom. Water level (with storm surge) time series are critical for input into shoreline response and coastal storm risk modeling applications.

The return period storm surge events can provide insight into the vulnerabilities of a given location through comparison with the existing topography. Table A - 6 provides peak storm surge heights by return period for St. Augustine Inlet, Florida. Storm surge levels versus frequency of occurrence presented in Table A - 6 were obtained from data compiled by the University of Florida for the Florida Department of Transportation (Sheppard and Miller, 2003).

Table A - 6. Peak storm tide elevations.

Storm Return Period	Peak Storm Surge Height		
(years)	ft-NGVD29	ft-NAVD88	ft-MSL
10	3.6	2.5	1.8
20	5.4	4.3	3.6
50	9.6	8.5	7.8
100	12.3	11.2	10.5
200	14.5	13.4	12.7
500	16.9	15.8	15.1

Sea Level Change

Relative Sea Level Rise

Relative sea level (RSL) refers to local elevation of the sea with respect to land, including the effects of lowering or rising land through geologic processes such as subsidence and glacial rebound. It is anticipated that the global mean sea level will rise within the next 100 years. To incorporate the direct and indirect physical effects of projected future sea level change on design, construction, operation, and maintenance of coastal projects, the USACE has provided guidance in the form of Engineering Regulation, ER 1100-2-8162 (USACE, 2013) and Engineering Technical Letter 1100-2-1 (USACE, 2014).

ER 1100-2-8162 provides both a methodology and a procedure for determining a range of sea level change estimates based on global sea level change rates, the local historic sea level change rate, the construction (base) year of the project, and the period of Federal participation for the project. Three estimates are required by the guidance, a Baseline (or “Low”) estimate, which is based on historic sea level rise and represents the minimum expected sea level change, an Intermediate estimate, and a High estimate representing the maximum expected sea level change. More details are provided in the referenced ER and ETL.

The SPV-Vilano project area is located approximately 31 miles from NOS gage #8720218 at Mayport, Florida. The historical local sea level rise rate taken from this gage was determined to be 2.40 mm/year (0.0079 ft/year; USACE, 2015). Given a project base year of 2020 a table of sea level change rates was produced for each of the three required scenarios through the end of Federal participation (Table A - 7). Figure A - 8 provides a graphic representation of the three levels of projected future sea level change for the 50-year planning horizon of the project (2020 to 2070) as well as an additional 50 years (to 2120).

The local rate of vertical land movement is found by subtracting regional MSL trend from local MSL trend. The regional mean sea level trend is assumed equal to the eustatic mean sea level trend of 1.7 mm/year (USACE, 2015). Therefore at SPV-Vilano, there is 0.70 mm/year of subsidence.

Table A - 7. Relative sea level rise for St. Johns County.

Year		Baseline (Historic)			Intermediate (NRC Curve I)			High (NRC Curve III)		
		mm	m	ft	mm	m	ft	mm	m	ft
Base Year	2020	0.0	0.00	0.00	0	0.00	0.00	0	0.00	0.00
	2025	12.0	0.01	0.04	20.3	0.02	0.07	46.5	0.05	0.15
	2030	24.0	0.02	0.08	41.9	0.04	0.14	98.6	0.10	0.32
	2035	36.0	0.04	0.12	64.9	0.06	0.21	156.3	0.16	0.51
	2040	48.0	0.05	0.16	89.2	0.09	0.29	219.8	0.22	0.72
25 Year	2045	60.0	0.06	0.20	114.9	0.11	0.38	288.8	0.29	0.95
	2050	72.0	0.07	0.24	141.9	0.14	0.47	363.5	0.36	1.19
	2055	84.0	0.08	0.28	170.3	0.17	0.56	443.9	0.44	1.46
	2060	96.0	0.10	0.31	200.1	0.20	0.66	529.9	0.53	1.74
	2065	108.0	0.11	0.35	231.2	0.23	0.76	621.6	0.62	2.04
50 Year	2070	120.0	0.12	0.39	263.6	0.26	0.86	718.9	0.72	2.36
	2075	132.0	0.13	0.43	297.4	0.30	0.98	821.9	0.82	2.70
	2080	144.0	0.14	0.47	332.6	0.33	1.09	930.5	0.93	3.05
	2085	156.0	0.16	0.51	369.1	0.37	1.21	1044.7	1.04	3.43
	2090	168.0	0.17	0.55	407.0	0.41	1.34	1164.7	1.16	3.82
75 Year	2095	180.0	0.18	0.59	446.3	0.45	1.46	1290.2	1.29	4.23
	2100	192.0	0.19	0.63	486.8	0.49	1.60	1421.4	1.42	4.66
	2105	204.0	0.20	0.67	528.8	0.53	1.73	1558.3	1.56	5.11
	2110	216.0	0.22	0.71	572.1	0.57	1.88	1700.8	1.70	5.58
	2115	228.0	0.23	0.75	616.7	0.62	2.02	1849.0	1.85	6.07
100 Year	2120	240.0	0.24	0.79	662.8	0.66	2.17	2002.8	2.00	6.57

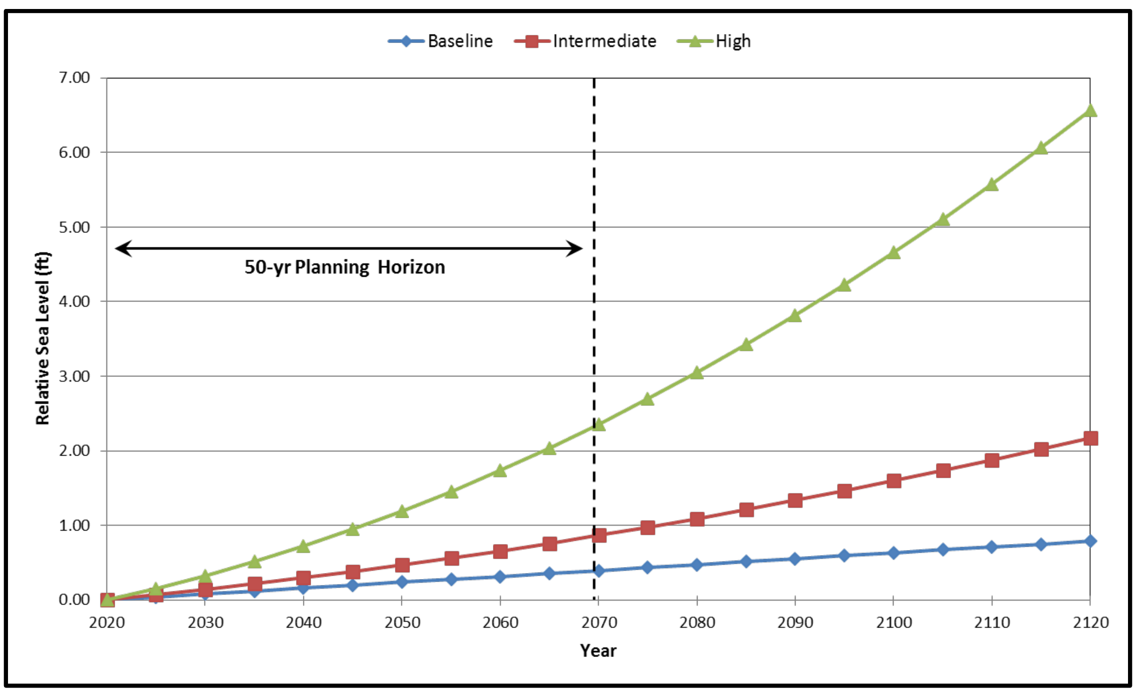

Figure A - 8. Relative sea level change, St. Johns County.

Beach Responses to Sea Level Change

ETL 1100-2-1 outlines methods and procedures to incorporate sea level change into the analysis, design and maintenance of projects. Per this guidance, this section evaluates how the sea level change scenarios outlined in the preceding section could affect future beach and shoreline behavior in the project area. The principal means by which sea level change would manifest itself on an open coast, sandy beach would be through changes to shoreline position and to beach volume. The below analyses are based on the assumption that sea level change would cause a change in the horizontal and vertical position of the beach profile. This phenomenon was first outlined by Bruun (1962). The theory states that an increase in water level causes the beach profile to shift upward and landward in response, in order to maintain an equilibrium shape. This shift causes both a shoreline change and a volumetric change as described herein.

Shoreline Change

Bruun (1962) proposed a formula for estimating the rate of shoreline recession based on the local rate of sea level change. This methodology also includes consideration of the local topography and bathymetry. Bruun's approach assumes that with a change in sea level, the beach profile will attempt to reestablish the same bottom depths relative to the surface of the sea that existed prior to sea level change. That is, the natural profile will be translated upward and shoreward to maintain equilibrium. If the alongshore littoral transport in and out of a given shoreline is equal, then the quantity of material required to re-establish the nearshore slope must be derived from erosion of the shore. The shoreline recession rate, *X*, resulting from sea level change can be estimated using Bruun's Rule, defined as:

$$X = \frac{-SW_*}{(h_* + B)} \qquad (1)$$

Where *S* is the rate of sea level change; *B* is the berm height (+8 ft NAVD88); h_* is the depth of closure (the depth beyond which there is no significant change over time in the shoreline profile; estimated to be approximately -20 ft NAVD88), and W_* is the width of the active profile (approximately 1400 ft). Figure A - 9 provides the resulting shoreline recession rate versus year for each of the three sea level rise scenarios.

The Bruun procedure is applicable to long straight sandy beaches with an uninterrupted supply of sand (which is applicable to the subject study area). Little is known about the rate at which profiles respond to changes in water level; therefore, this procedure should only be used for estimating long-term changes. The procedure is not a substitute for the analysis for historical shoreline and profile changes when determining historic (baseline) conditions. However, if little or no historical data is available, then historical analysis may be supplemented by this method to provide an estimate of the long-term erosion rates attributable to sea level rise. The offshore contours in the project area are not entirely straight and parallel; however, Bruun's Rule does provide an estimate of the potential shoreline changes within the project area attributable to a projected change in sea level.

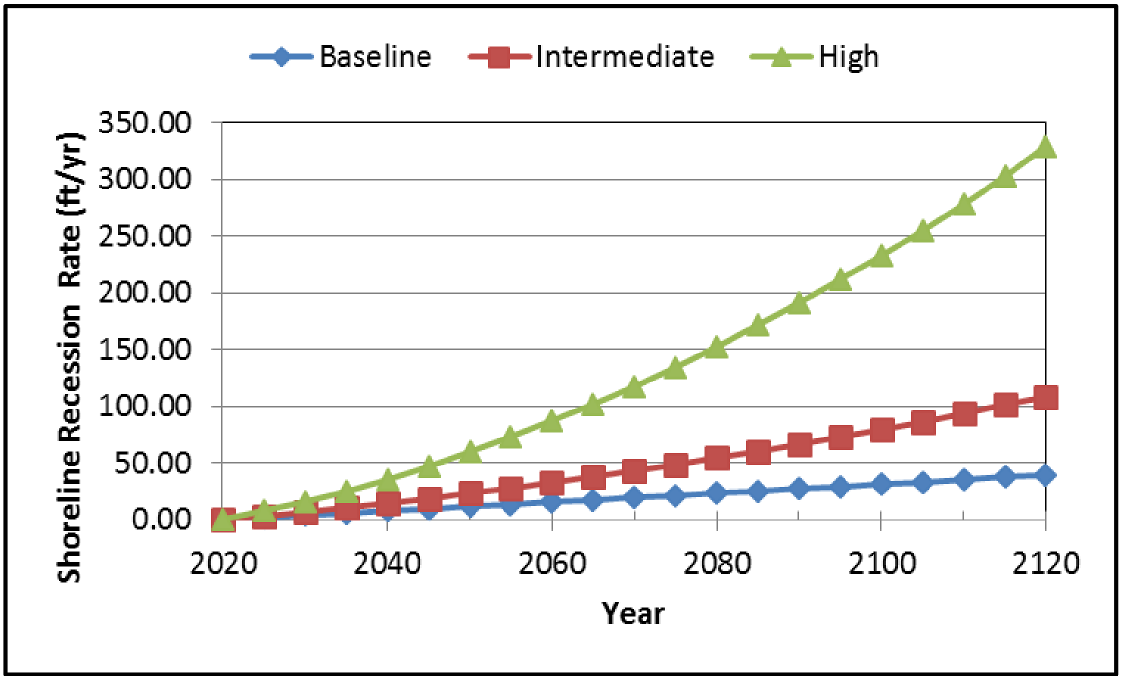

Figure A - 9. Estimated shoreline recession rate due to sea level rise.

Volumetric Change

Engineering Manual (EM) 1110-2-3301 (USACE, 1995) provides guidance on how to calculate beach volume based on berm height, depth of closure, and translation of the shoreline (in this case, shoreline recession). Assuming that as an unarmored beach erodes, it maintains approximately the same profile above the seaward limit of significant transport, the volume can be determined as:

$$V = (B + h_*)X \qquad (2)$$

Where variables are consistent with Equation 1. Figure A - 10 provides the resulting volume lost versus year for each of the three sea level rise scenarios.

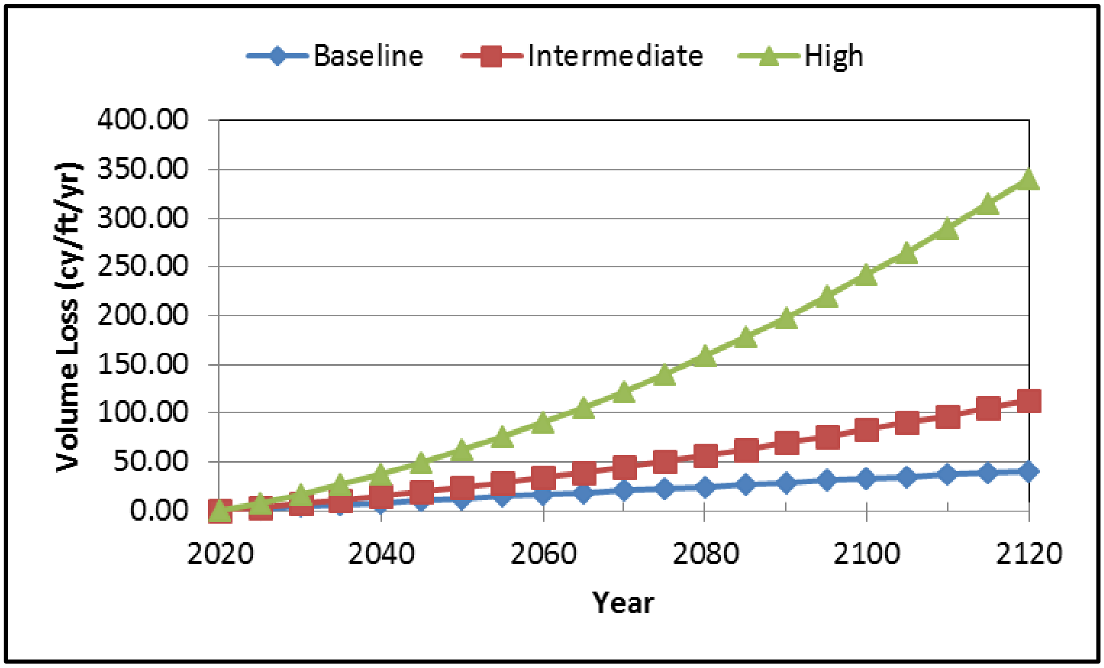

Figure A - 10. Estimated volume loss due to sea level rise.

Historical Shoreline Change

Changes in MHW position provide a historical view of the behavior of the shoreline. Beach profiles are traditionally gathered by the FDEP, local sponsors, and USACE. Available beach surveys for St. Johns County go back as far as 1883. However, the reliability of such historical profiles may be questionable. Additionally, some surveys were conducted over a span of years (i.e. 1970 to 1973) without any indication of which portion of the shoreline was surveyed during which year. Knowing the year of the survey data is vital to the accuracy of the annual recession rate. Therefore, based on a review of all available surveys, it was determined that profiles surveyed prior to 1972 would be excluded from the MHW analysis.

MHW shoreline positions were measured at each DNR survey monument location, for each survey, along the proper azimuth (75° for the study area; Figure A - 11). The distance given from each DEP monument to the MHW elevation for each year surveyed can be seen in Table A - 8. At each DEP R-monument in the study area the change in the MHW position and the rate of change was determined for the time period between each of the surveys as well as the overall change from 1972 to 2015. The rate of MHW position change for each monument for selected periods are displayed in Figure A - 12 and Table A - 9. The average MHW position change and change rate for each of the three study segments are presented in Table A - 10.

Shoreline changes fluctuate over time along the study area. The shoreline of St. Johns County has fluctuated throughout history seeing areas undergo both advancement and recession of the MHW position. This analysis showed that over the long term from 1972 to 2015 the study area, on average, has been receding. In the time between 1972 and 2015, the MHW in South Ponte Vedra receded an average of 1.3 ft/yr. In the Vilano Beach 1 segment the MHW receded 1.7 ft/yr on average while the Vilano Beach 2 segment directly north of the St. Augustine Inlet the MHW advanced seaward an average of 0.3 ft/yr.

Figure A - 11. Reference monument locations around study area.

Table A - 8. Study area distances between R-monuments and MHW.

Mon. ID	Sep 1972	Jun 1984	Sep 1986	Apr 1999	Aug 2003	Oct 2007	Dec 2009	Dec 2010	Feb 2012	Dec 2012	Jun 2013	July 2014	Aug 2015
84	214	164	198	163	168	140			156	159	145	156	150
85	186	176	212	167	158	139			151	132	169	132	121
86	162	150	159	157	152	120			171	153	179	163	225
87	177	127	150	146	140	103			111	127	97	130	153
88	143	178	179	140	143	89			109	104	120	101	100
89	187	156	182	159	147	103			117	117	104	133	140
90	178	171	158	142	134	95			102	102	104	100	99
91	205	193	190	181	153	130			133	142	133	119	125
92	199	224	228	184	170	145			164	145	141	138	127
93	160	164	143	145	139	114			125	118	108	100	93
94	199	226	223	161	166	133			147	139	143	143	126
95	221	192	175	176	173	150			160	165	161	158	130
96	182	148	200	138	139	115			127	128	125	112	92
97	179	140	142	125	132	99			108	96	100	81	93
98	203	162	169	159	161	121			136	148	132	115	171
99	192	196	217	162	158	136			151	139	142	142	150
100	190	179	188	156	151	131			147	142	139	138	130
101	232	194	195	190	186	144			155	136	140	150	151
102	197	205	196	167	162	147			152	135	131	144	157
103	209	223	198	190	181	160			158	166	141	151	162
104	207	213	198	192	176	158			162	147	154	144	158
105	225	207	199	206	188	160			176	177	179	169	151
106	194	171	181	169	154	130			152	139	135	129	120
107	199	170	161	151	146	128			153	139	134	139	117
108	205	175	174	167	141	134			151	127	137	134	132
109	237	184	187	188	170	154	164	160	173	163	162	172	150
110	242	208	219	199	179	150	160	161	172	165	151	167	166
111	215	175	183	178	160	133	165	147	146	127	128	131	132
112	211	171	173	157	152	135	129	132	126	122	139	128	148
113	208	170	183	178	152	136	134	126	124	118	92	123	136
114	191	158	147	160	128	134	114	121	120	119	119	109	121
115	191	175	171	160	126	134	107	127	131	135	132	145	142
116	228	191	181	164	127	141	105	133	145	148	137	140	138
117	224	207	201	174	135	159	106	155	162	168	164	143	155

Mon. ID	Sep 1972	Jun 1984	Sep 1986	Apr 1999	Aug 2003	Oct 2007	Dec 2009	Dec 2010	Feb 2012	Dec 2012	Jun 2013	July 2014	Aug 2015
118	243	240	213	191	157	192	172	194	184	193	184	181	182
119	213	255	241	188	179	211	188	189	173	186	159	208	203
120	183	287	300	204	211	229	184	169	177	190	163	244	246
121	229	247	235	225	274	271	220	214	274	266	302	319	311
122	262	279	243	403	380	341	449	453	455	312	379	372	342

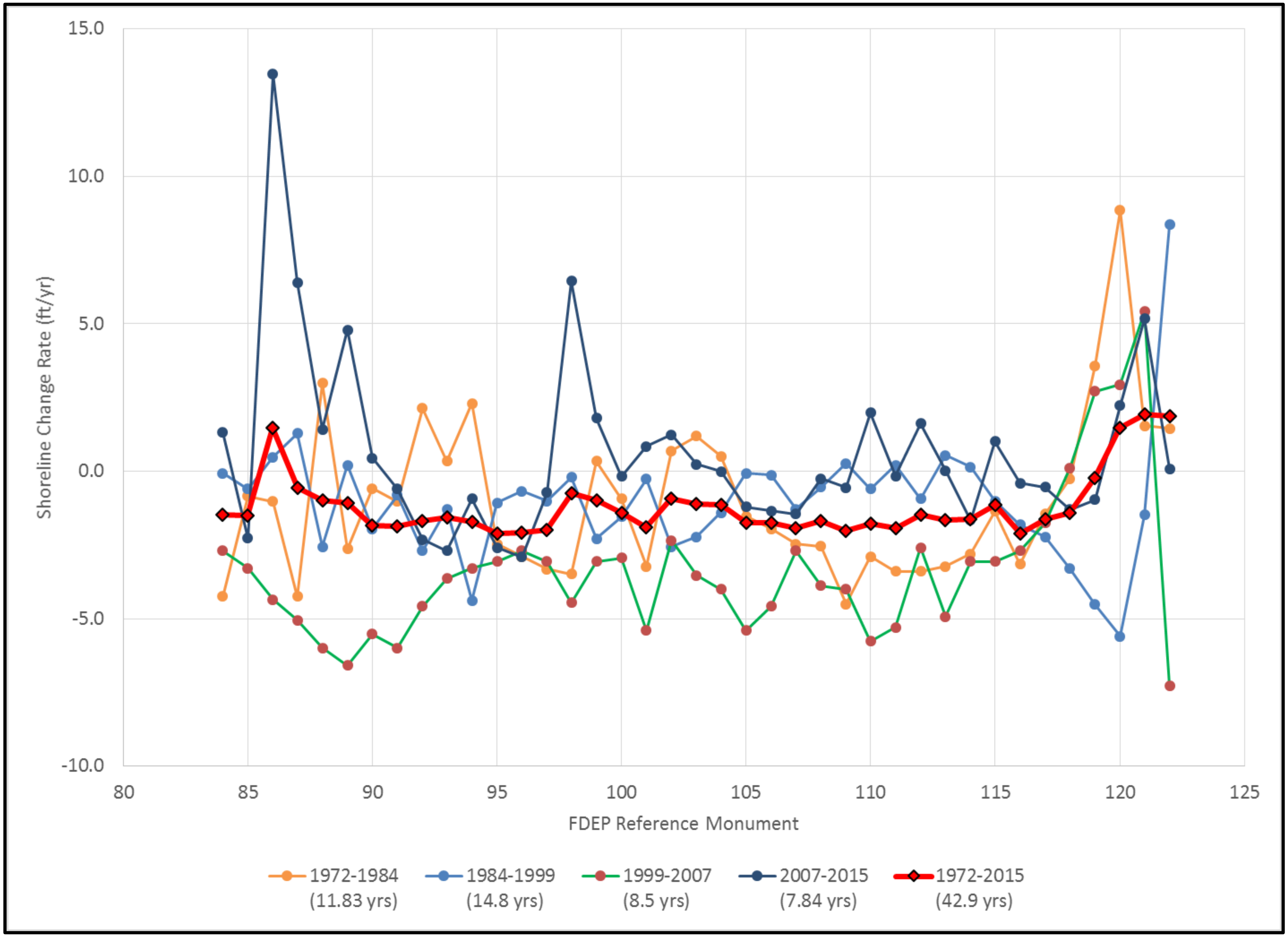

Figure A - 12. Study area shoreline change rates.

Table A - 9. Shoreline change rates (ft/yr) for selected periods.

Mon. ID	1972-1984 (11.83 yrs)	1984-1999 (14.8 yrs)	1999-2007 (8.5 yrs)	2007-2015 (7.84 yrs)	1972-2015 (42.9 yrs)
84	-4.3	-0.1	-2.7	1.3	-1.5
85	-0.9	-0.6	-3.3	-2.3	-1.5
86	-1.0	0.5	-4.3	13.5	1.5
87	-4.3	1.3	-5.1	6.4	-0.6
88	3.0	-2.6	-6.0	1.4	-1.0
89	-2.6	0.2	-6.6	4.8	-1.1
90	-0.6	-2.0	-5.5	0.5	-1.9
91	-1.0	-0.8	-6.0	-0.6	-1.9
92	2.1	-2.7	-4.6	-2.3	-1.7
93	0.3	-1.3	-3.6	-2.7	-1.6
94	2.3	-4.4	-3.3	-0.9	-1.7
95	-2.5	-1.1	-3.1	-2.6	-2.1
96	-2.9	-0.7	-2.7	-2.9	-2.1
97	-3.3	-1.0	-3.1	-0.7	-2.0
98	-3.5	-0.2	-4.5	6.4	-0.7
99	0.3	-2.3	-3.1	1.8	-1.0
100	-0.9	-1.5	-2.9	-0.2	-1.4
101	-3.2	-0.3	-5.4	0.8	-1.9
102	0.7	-2.6	-2.4	1.2	-0.9
103	1.2	-2.2	-3.5	0.2	-1.1
104	0.5	-1.4	-4.0	0.0	-1.1
105	-1.5	-0.1	-5.4	-1.2	-1.7
106	-2.0	-0.1	-4.6	-1.3	-1.7
107	-2.5	-1.3	-2.7	-1.5	-1.9
108	-2.6	-0.5	-3.9	-0.3	-1.7
109	-4.5	0.3	-4.0	-0.6	-2.0
110	-2.9	-0.6	-5.8	2.0	-1.8
111	-3.4	0.2	-5.3	-0.2	-1.9
112	-3.4	-0.9	-2.6	1.6	-1.5
113	-3.2	0.5	-4.9	0.0	-1.7
114	-2.8	0.1	-3.1	-1.7	-1.6
115	-1.4	-1.0	-3.1	1.0	-1.1
116	-3.1	-1.8	-2.7	-0.4	-2.1
117	-1.4	-2.2	-1.8	-0.5	-1.6
118	-0.3	-3.3	0.1	-1.3	-1.4
119	3.6	-4.5	2.7	-1.0	-0.2
120	8.8	-5.6	2.9	2.2	1.5
121	1.5	-1.5	5.4	5.2	1.9
122	1.4	8.4	-7.3	0.1	1.9

Table A - 10. Study segment shoreline change summary.

Project Segments		1972-1984 (11.83 yrs)	1984-1999 (14.8 yrs)	1999-2007 (8.5 yrs)	2007-2015 (7.84 yrs)	1972-2015 (42.9 yrs)
SPV (R84 - R104)	Avg Change (ft)	-11.5	-18.1	-34.7	8.6	-55.7
	Avg Change Rate (ft/yr)	-1.0	-1.2	-4.1	1.1	-1.3
VB 1 (R104 - R117)	Avg Change (ft)	-29	-9	-33	-2	-72
	Avg Change Rate (ft/yr)	-2.4	-0.6	-3.8	-0.2	-1.7
VB 2 (R117 - 122)	Avg Change (ft)	27	-22	3	6	14
	Avg Change Rate (ft/yr)	2.3	-1.5	0.4	0.8	0.3

Historical Volume Change

Volume change analysis can provide additional insight into beach changes beyond the shoreline change analysis. The variability of volume change rates is visible in the values presented in Table A - 11 and Figure A - 13. On average, volume change varied between 0.95 cy/ft/yr (1972 to 1986) and -6.18 cy/ft/yr (1999 to 2003) for an overall average change of -3.62 cy/ft/yr. Although the shoreline change suggests erosion of the profile during the 1972 to 1984 period, the volume change over a similar period (1972 to 1986) indicates that the shoreline recession is actually a redistribution of material along the profile since volume changes were actually positive. The final two periods analyzed in Table A - 11 (2003 to 2007 and 2011 to 2015) show the volume change rate along the study area to be consistent with the average of all the periods.

Table A - 11. Unit volume change rates for the study area.

Monument	Volume Change (cy/ft/yr)					
	1972 to 1986	1986 to 1999	1999 to 2003	2003 to 2007	2011 to 2015	Average
84	0.65	-8.56	4.42	-14.36		-4.46
85	2.28	-8.77	0.19	-8.81	-1.20	-3.26
86	1.77	-2.18	2.77	-11.71	-0.67	-2.00
87	1.26	-7.15	-4.00	-6.56	-6.69	-4.63
88	2.99	-10.19	3.76	-17.37	-0.96	-4.35
89	2.09	-9.47	0.34	-21.25	-6.32	-6.92
90	1.52	-6.87	-1.94	-12.67	4.25	-3.14
91	1.95	-4.32	-7.48	-6.04	-4.59	-4.10
92	2.54	-5.04	0.89	-9.86	-2.81	-2.86
93	1.68	-5.54	-9.17	1.24	-0.81	-2.52
94	1.22	-8.27	5.55	-6.83	-11.01	-3.87
95	-0.39	-2.43	-16.92	0.04	-15.50	-7.04
96	-0.07	-5.18	1.50	-7.77	-18.38	-5.98
97	-0.24	-10.22	-2.49	-3.20	-8.97	-5.02
98	-0.74	-4.61	-11.99	-2.31	-3.38	-4.61
99	1.39	-8.44	-17.55	5.00	1.95	-3.53
100	1.29	-5.41	-10.41	-4.04	-1.96	-4.11
101	1.68	-7.84	-7.43	-0.58	0.48	-2.74
102	2.00	-5.58	-2.06	-10.22	0.23	-3.13
103	3.94	-7.04	-14.85	4.56	-5.45	-3.77
104	4.14	-7.32	-18.01	-0.40	-12.54	-6.83
105	4.03	-7.19	-19.09	-0.64	0.37	-4.50

Monument	Volume Change (cy/ft/yr)					
	1972 to 1986	1986 to 1999	1999 to 2003	2003 to 2007	2011 to 2015	Average
106	2.45	-5.29	-15.54	-0.70	-3.07	-4.43
107	-0.56	-4.58	-12.73	2.15	-3.57	-3.86
108	0.14	-5.79	0.65	-11.69	-2.27	-3.79
109	-1.06	-4.20	-18.63	13.17	-1.68	-2.48
110	2.29	-4.21	-11.53	-6.67	-14.83	-6.99
111	-1.29	-8.33	-15.93	14.86	-1.50	-2.44
112	-3.07	-7.78	-8.31	-2.08	2.57	-3.73
113	-1.00	-7.02	-16.35	3.36	-1.57	-4.52
114	-0.91	-4.36	-6.11	-8.19	-0.30	-3.97
115	-0.57	-5.89	-0.56	-15.14	-2.96	-5.03
116	-1.08	-8.90	-11.09	1.90	-3.07	-4.45
117	-0.30	-9.12	-11.05	5.05	-4.50	-3.98
118	-0.88	-7.17	-6.58	8.54	-4.44	-2.11
119	1.13	-5.60	-3.07	11.02	-3.13	0.07
120	3.91	-7.88	1.99	0.12		-0.46
121	0.89	2.73	-9.47	5.58		-0.07
122	0.06	7.54	27.17	-16.92		4.46
Average	**0.95**	**-5.99**	**-6.18**	**-3.32**	**-3.95**	**-3.62**

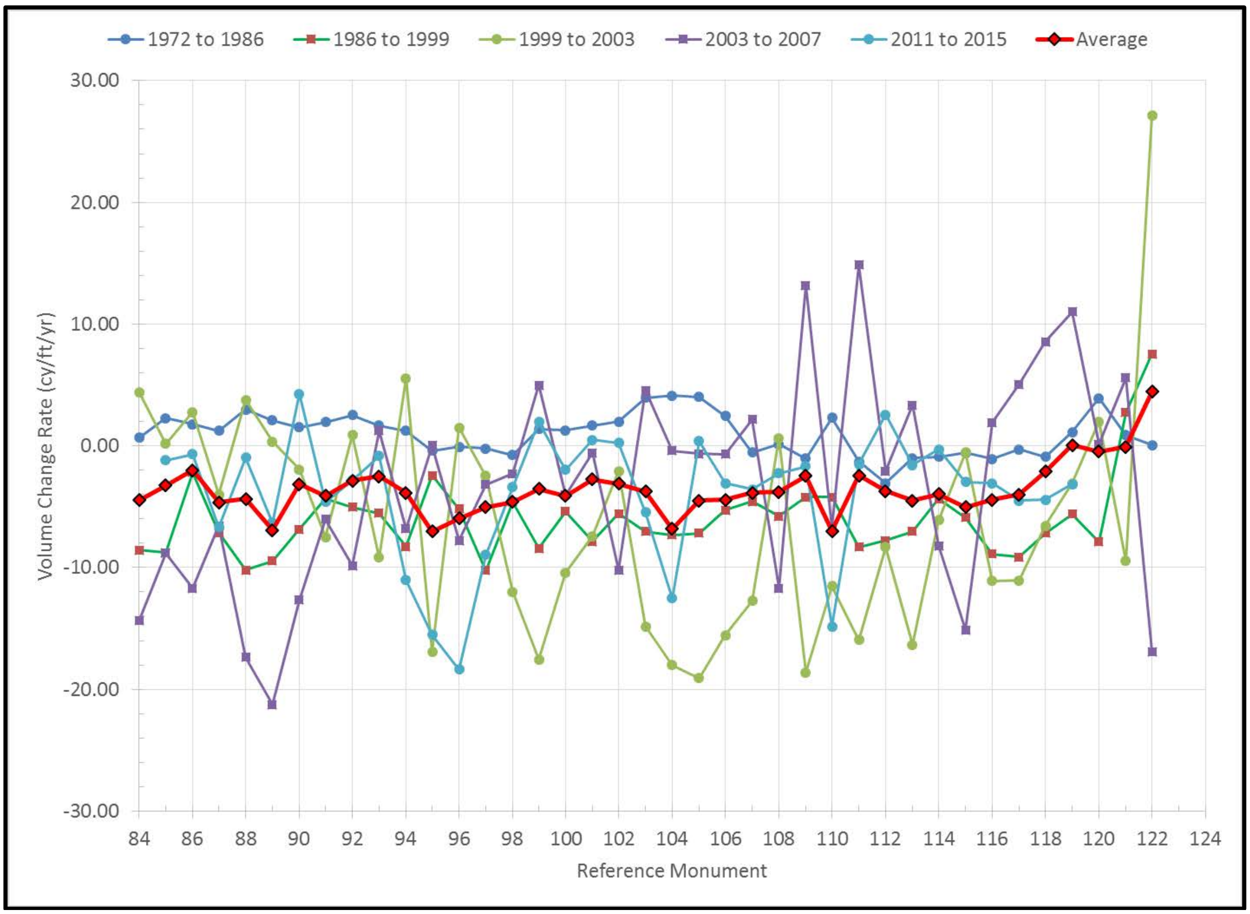

Figure A - 13. Unit volume change rates for the study area.

Effects of Adjacent Features

St. Augustine Inlet

St. Augustine Inlet features a Federal navigation channel and lies in central St. Johns County, approximately 34 mi south of the St. Johns River Inlet and 15 mi north of the Matanzas Inlet (

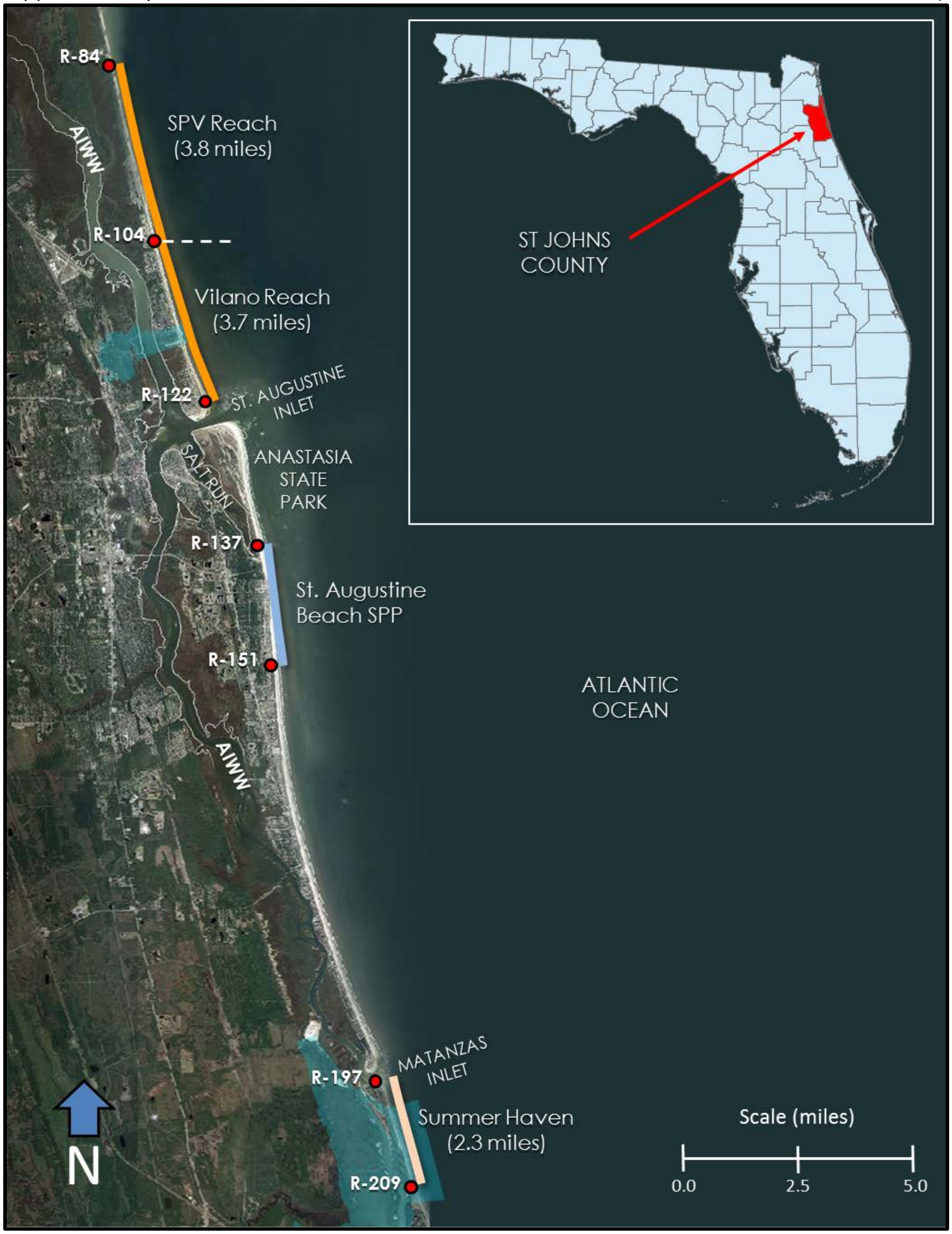

Figure A - 1). The inlet supports navigation and provides access to the Atlantic Ocean from the St. Augustine Harbor and the Intracoastal Waterway (ICWW). The extent of impacts to adjacent shorelines attributable to the Federal navigation channel was investigated using historical shoreline information dating as far back as 1859, prior to construction of Federal improvements in 1940. It was determined that prior to 1940, the shorelines adjacent to the existing inlet were characterized by cyclic periods of instability. Rates of shoreline change varied from as much as +250 feet/year to -250 feet/year. Much of this can be attributed to the unstable and migratory nature of the inlet prior to stabilization. Despite the mobility of the inlet at the time, available survey information suggests that the overall, average shoreline change rate prior to inlet stabilization was -4 feet/year.

Rates of erosion and accretion after inlet stabilization were estimated based on shoreline information obtained since construction in 1940. Inlet improvement involved cutting a new channel north of the natural inlet and constructing north and south structures at the inlet entrance: a sand-trap groin on the north side and a jetty on the south side. Immediately after construction shorelines adjacent to the inlet experienced fluctuating patterns of erosion and accretion as the old inlet migrated and filled, existing shoals dispersed and reformed, and the entire inlet/shoreline system sought equilibrium. Accounting for the years immediately after construction, as well as the addition of armor in various locations along the shoreline, and the addition of beach material obtained through maintenance dredging events, the average shoreline rate of change south of the inlet was determined to be -8 feet/year. With the exception of the post-construction stabilization period, shoreline erosion/accretion rates to the north of the inlet appear to have remained comparable to those of the pre-construction shoreline.

Assuming that any differences in shoreline change rates before and after inlet stabilization are indicative of the inlet's impacts on sediment transport processes, then approximately 50% of post-construction beach erosion south of the inlet has occurred due to impacts of the Federal navigation project. A previous inlet impact analysis, performed during preparation of the 1979 feasibility study (USACE, 1979) estimated that the inlet-induced component of total erosion occurring over approximately 5 miles of the project shoreline (south of the inlet) was about 50%. Therefore the analyses are comparable.

The St. Augustine Inlet contributes to beach erosion for areas north and south of the inlet by acting as a sediment sink (USACE, 2012). The sand trap groin structure on the north side of the inlet acts to stabilize southern portions of the study area by trapping sediment following the net southerly sediment transport direction. However, as designed, transport around the tip of the north sand-trap groin is prevalent and visible in the development of "Porpoise Point," the sand spit at the southern end of Vilano Beach adjacent to the navigation channel on the north side of the inlet. Flood tidal currents are responsible for diverting sediment from the north and south shorelines into the inlet. Once inside the inlet, sediments tend to settle out into flood shoals. Tidal ebb currents also divert sediments from the littoral system in the offshore direction where some of the sediment is deposited on the ebb shoal. Sediments not trapped in the ebb shoal complex are bypassed to the adjacent shorelines.

Based on USACE (2012), the inlet impounds 278,000 cy/yr of sediment originating from the adjacent north and south beaches. The sediment budget presented in USACE (2012) suggests that 248,800 cy/yr enters the inlet complex from the north beaches and 29,300 cy/yr originates from beaches to the south. Coupled with the net southerly transport of 150,000 cy/yr, the net deficit for the north and south beaches are 248,800 cy/yr and 179,300 cy/yr, respectively.

The Federally constructed hurricane and storm damage reduction project on St. Augustine Beach (construction initiated in 2001) addresses the down drift impacts of the inlet by facilitating inlet bypassing through dredging of the ebb shoal and nourishing down drift (south of the inlet) beaches. Since it was determined that 50% of the erosion south of the inlet was attributable to the Federal navigation project, the cost apportionment that would otherwise be paid by the local sponsor is further reduced and paid by the Federal government. For further details see USACE (1998).

Two governmental entities routinely dredge in the vicinity of St. Augustine Inlet. The Florida Inland Navigation District (FIND) acts as the local sponsor for ICWW maintenance, which places beach quality material along Anastasia Island State Park north of the existing St. Johns County SPP. USACE maintains the Federal navigation channel that traverses St. Augustine Inlet and often dredges the channel concurrently with renourishment of the existing St. Johns County Federal SPP since the inlet ebb shoal is the authorized borrow area. In 1996, 324,000 cubic yards (cy) of maintenance material from the navigation project was placed on St. Augustine beach between R-138 and R-148. Initial construction of the St. Johns County Federal Shore Protection Project in 2003 removed 4,500,000 cy from the ebb shoal with a portion of this volume placed in Anastasia State Park. This dredging event was followed by removal of 2,800,000 cy as part of the Federal emergency renourishment in 2005 as a result of the 2004 hurricane season. The latest renourishment of the St. Johns County SPP occurred in 2012 and placed 2,100,000 cy between R-139 and R-147.

Beach-*fx* Life-Cycle Shore Protection Project Evolution Model

Federal participation in Hurricane and Storm Damage Reduction (HSDR) projects is based on a favorable economic justification in which the benefits of the project outweigh the costs. Determining the Benefit to Cost Ratio (BCR) requires both engineering analysis (project performance and evolution) and planning (alternative analysis and economic justification). The interdependence of these functions has led to the development of the life-cycle simulation model Beach-*fx*. Beach-*fx* combines the evaluation of physical performance and economic benefits and costs of shore protection projects (Gravens et. al., 2007), particularly beach nourishment, to form the basis for determining the justification for Federal participation. This section describes the engineering aspects of the Beach-*fx* model.

Background & Theory

USACE guidance requires that flood damage reduction studies include risk and uncertainty (USACE, 2006). Beach-*fx* is an event-driven life-cycle model that satisfies this requirement by fully incorporating risk and uncertainty throughout the modeling process (input, methodologies, and output). Over the project planning horizon, typically 50 years, the model estimates shoreline response to a series of historically-based storm events. These plausible storms are randomly generated using a Monte Carlo simulation. The corresponding shoreline evolution includes not only erosion due to the storms, but also allows for storm recovery, post-storm emergency dune and/or shore construction, and planned nourishment events throughout the period of Federal participation for the project. Risk based damages to structures are estimated based on the shoreline response in combination with pre-determined Damage Functions for all structure types within the project area. Uncertainty is incorporated not only within the input data (storm occurrence and intensity, structural parameters, structure and contents valuations, and Damage Functions), but also in the applied methodologies (probabilistic seasonal storm generation and multiple iteration, life-cycle analysis). Results from the multiple iterations of the life-cycle are averaged and the economics of the project are determined.

The project site itself is represented by divisions of the shoreline referred to as “Reaches”. Because this term may also be used to describe segments of the shoreline to which project alternatives are applied, Beach-*fx* reaches will be referred to in this appendix as “Model reaches”. Model reaches are contiguous, morphologically homogenous areas that contain groupings of structures (residences, businesses, walkovers, roads, etc.), all of which are represented by Damage Elements (DEs). DEs are grouped within divisions referred to as Lots. Figure A - 14 shows a graphic depiction of the model setup. For further details about the specifics of Lot extents and DE grouping see the Economics Appendix.

Each model reach is associated with a representative beach profile that approximates the cross-shore profile and beach composition of the reach. Multiple model reaches may share the same representative beach profile while groupings of model reaches may represent a single design reach. For SPV/Vilano, the project area consists of a single design reach, originally divided into 37 model reaches. Table A - 12 provides model reach identifiers as well as corresponding FDEP R-monuments and Beach-*fx* representative profiles for each model reach. Model reaches are grouped assigned to representative profiles based on the similarity of profile characteristics (detailed in Pre-Storm Representative Profile Section).

Implementation of the Beach-*fx* model relies on a combination of meteorology, coastal engineering, and economic analyses and is comprised of four basic elements:

- Meteorologic driving forces

- Coastal morphology
- Economic evaluation
- Management measures

The subsequent discussion in this section addresses the basic aspects of implementing the Beach-*fx* model. For a more detailed description of theory, assumptions, data input/output, and model implementation, refer to Gravens et al. (2007), Males et al. (2007), and USACE (2009).

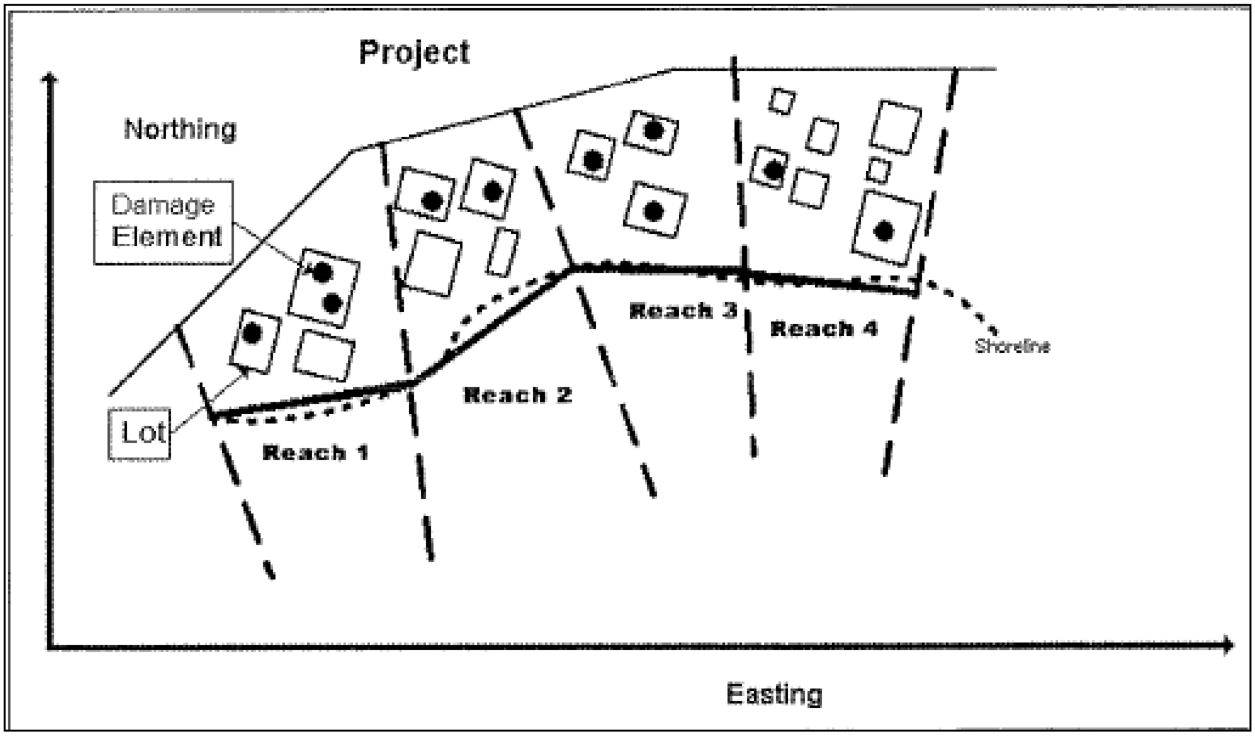

Figure A - 14. Beach-*fx* model setup representation.

Table A - 12. Model reaches, reference monuments, and Beach-*fx* representative profiles.

Model Reach	R-monument(s)	Beach-*fx* Profile #		Model Reach	R-monument(s)	Beach-*fx* Profile #
SPV84	84	1		SPV104	104	3
SPV85	85	1		SPV105	105	3
SPV86	86, 87	1		SPV106	106	3
SPV87	87	2		SPV107	107	3
SPV88	88	2		SPV108	108	3
SPV89	89	2		SPV109	109	3
SPV90	90	1		Vx110	110	3
SPV91	91	1		Vx111	111	3
SPV92	92	1		Vx112	112, 113	4
SPV93	93	1		Vx114	114	4
SPV94	94	2		Vx115	115	5
SPV95	95	2		Vx116	116	5
SPV96	96	2		Vx117	117	5
SPV97	97, 98	2		V2x118	118	6
SPV98	none	2		V2x119	119	6
SPV100	99, 100	3		V2x120	120	7

Model Reach	R-monument(s)	Beach-*fx* Profile #
SPV101	none	3
SPV102	101, 102	4
SPV103	103	4

Model Reach	R-monument(s)	Beach-*fx* Profile #
V2x121	121	7
V2x122	122	8

Meteorologic Driving Forces

The predominant driving force for coastal morphology and associated damages within the Beach-*fx* model is the historically based set of storms that is applied to the life-cycle simulation. Because the eastern coast of Florida is subject to seasonal storms- tropical storms (hurricanes) in the summer months and extra-tropical storms (northeasters) in the winter and fall months- the "plausible storms" dataset for St. Johns County is made up of both types. Derived from the historical record of the region, the plausible storm set is based on 46 tropical storms, occurring between 1887 and 1999 and 48 extra-tropical storms, occurring between 1980 and 1999.

Because tropical storm events tend to be of limited duration, passing over a given site within a single portion of the tide cycle, it is assumed that any of the historical storms could have occurred during any combination of tidal phase and tidal range. Therefore, each of the 46 tropical storms surge hydrographs was combined with possible variations in the astronomical tide. This was achieved by combining the peak of each storm surge hydrograph with the astronomical tide at high tide, mean tide falling, low tide, and mean tide rising for each of three tidal ranges corresponding to the lower quartile, mean, and upper quartile tidal ranges. This resulted in 12 distinct combinations for each historically based tropical storm and a total of 552 tropical storm conditions in the plausible storm dataset.

Due to their generally extended durations, extra-tropical storms in the historical record tend to occur over complete tide cycles. Therefore, it can be assumed that the storm hydrograph of each of the 48 historical extra-tropical storms is sufficient without combining with possible variations of the astronomical tide. The entire plausible storm suite therefore consists of a total of 600 tropical and extra-tropical storms.

In addition to the plausible storm dataset, the seasonality of the storms must also be specified. The desired storm seasons are based on the assumption that each plausible storm takes place within the season in which the original historical storm occurred. The probability of both tropical and extra-tropical storms is defined for each season through the Probability Parameter. The Probability Parameter is determined for each season and storm type by dividing the number of storms by the total number of years in the storm record (extra-tropical or tropical). Four storm seasons were specified for St. Johns County (Table A - 13).

Table A - 13. St. Johns County Beach-*fx* storm seasons.

Storm Season	Start Date	End Date	Probability Parameter Extra-Tropical Storm	Probability Parameter Tropical Storm
Extratrop. Winter/Spring	Dec 1	Apr 31	1.45	0.00
Tropical Early Summer	May 1	Jul 31	0.15	0.04
Tropical Peak	Aug 1	Sep 30	0.10	0.29
Extratrop./Tropical	Oct 1	Nov 30	0.70	0.07

The combination of the plausible storm dataset and the specified storm season allows the Beach-*fx* model to randomly select from storms of the type that fall within the season currently being processed. For each storm selected, a random time within the season is chosen and assigned as the storm date. The timing of the entire sequence of storms is governed by a pre-specified minimum storm arrival time. A minimum arrival time of 7 days was specified for St. Johns County. Based on this interval the model attempts to place subsequent storm events outside of a 14 day window surrounding the date of the previous storm (i.e. a minimum of 7 days prior to the storm event and a minimum of 7 days following the storm event). The model does allow the user to set different minimum arrival times for extra-tropical and tropical storms; however, the 7 day interval was considered suitable for both storm types. Also, due to the probabilistic nature of the model the minimum arrival time may be overridden as warranted during the course of the life-cycle analysis.

Coastal Morphology

The Beach-*fx* model estimates changes in coastal morphology through four primary mechanisms:

- Shoreline storm response
- Applied shoreline change
- Project-induced shoreline change
- Post-storm berm recovery

Combined, these mechanisms allow for the prediction of shoreline morphology for both with and without project conditions.

Shoreline Storm Response

Shoreline storm response is determined by applying the plausible storm suite that drives the Beach-*fx* model to simplified beach profiles that represent the shoreline features of the project site. As standard practice for obtaining storm response inputs for Beach-*fx*, application of the storm suite to the idealized profiles was accomplished employing the SBEACH coastal processes response model (Larson and Kraus, 1989). SBEACH is a numerical model which simulates storm-induced beach change based on storm conditions, initial profiles, and shoreline characteristics such as beach slope and grain size. Output consists of post-storm beach profiles, maximum wave height and wave period information, and total water elevation including wave setup. Pre- and post-storm profiles, wave data, and water levels can be extracted from SBEACH and imported into the Beach-*fx* Shore Response Database (SRD). The SRD is a relational database used by the Beach-*fx* model to pre-store results of SBEACH simulations of all plausible storms impacting a pre-defined range of anticipated beach profile configurations.

Pre-Storm Representative Profiles

In order to develop the idealized SBEACH profiles from which the SRD was derived, it was necessary to first develop representative profiles for the project shoreline. The number of representative profiles developed for any given project depends on the natural variability of shoreline itself. Historical profiles at each FDEP R-monument were compared over time, aligned, and then averaged into a composite profile representative of the shoreline shape at that given R-monument location. Composite profiles were then compared and separated into groupings according to the similarity between the following seven dimensions:

- Upland elevation

- Dune slope
- Dune height
- Dune width
- Berm height
- Berm width
- Foreshore slope

For St. Johns County, eight groupings of similarly dimensioned beach profiles were identified. Within each grouping, the composite profiles were then averaged into a single profile representative of a portion of the project shoreline. Using these representative profiles, idealized profiles describing the major dimensions of the profile were defined (Figure A - 15 through Figure A - 22). Below the zero foot-NAVD88 contour the idealized profile is the same shape as the average profile as it is assumed the average profile represents the equilibrium profile. Thus, the entire profile is not shown in Figure A - 15 through Figure A - 22. Table A - 12 provides dimensions for each of the idealized pre-storm Beach-*fx* profiles.

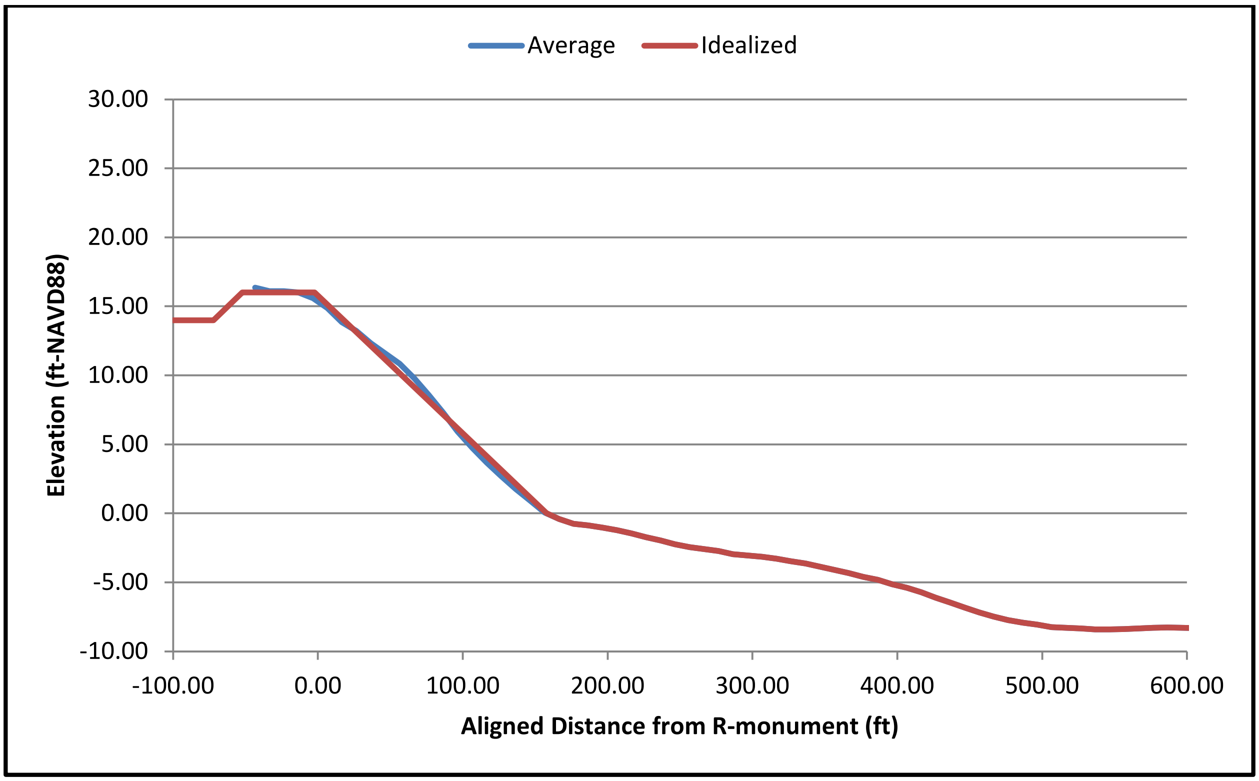

Figure A - 15. Averaged and idealized profiles: P1 grouping.

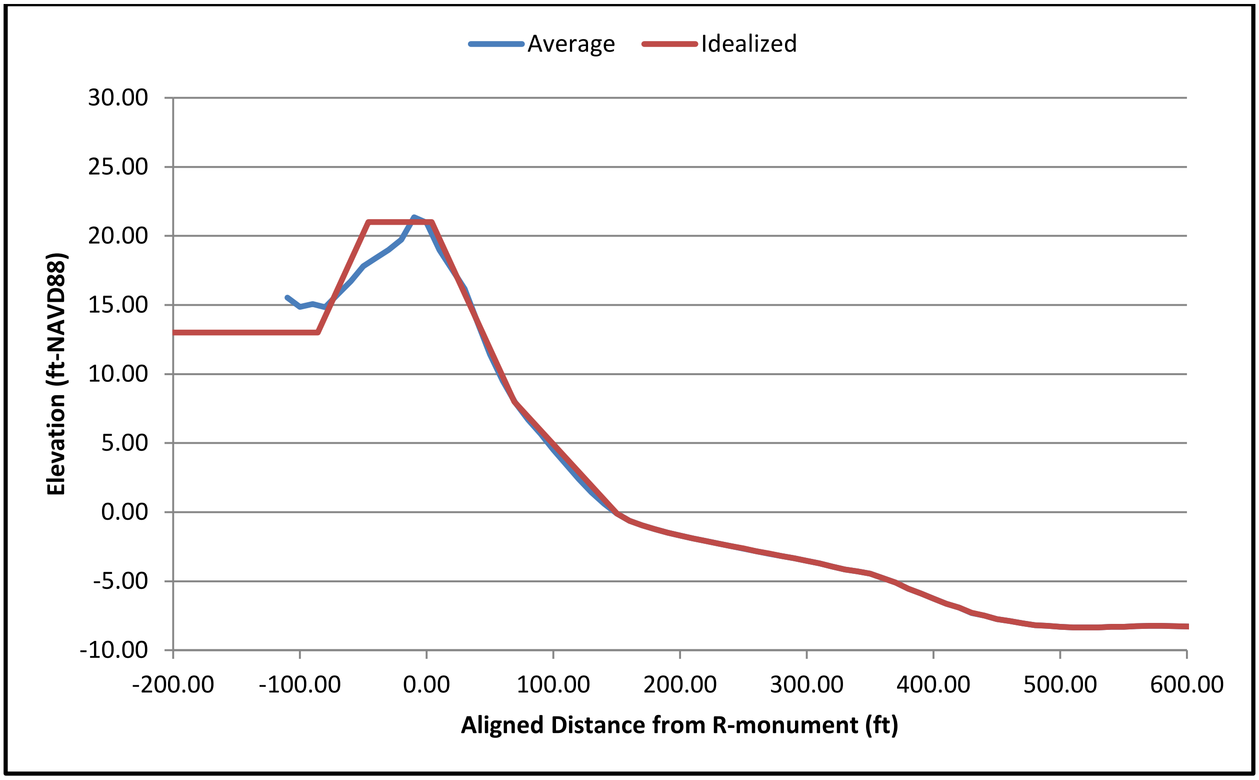

Figure A - 16. Averaged and idealized profiles: P2 grouping.

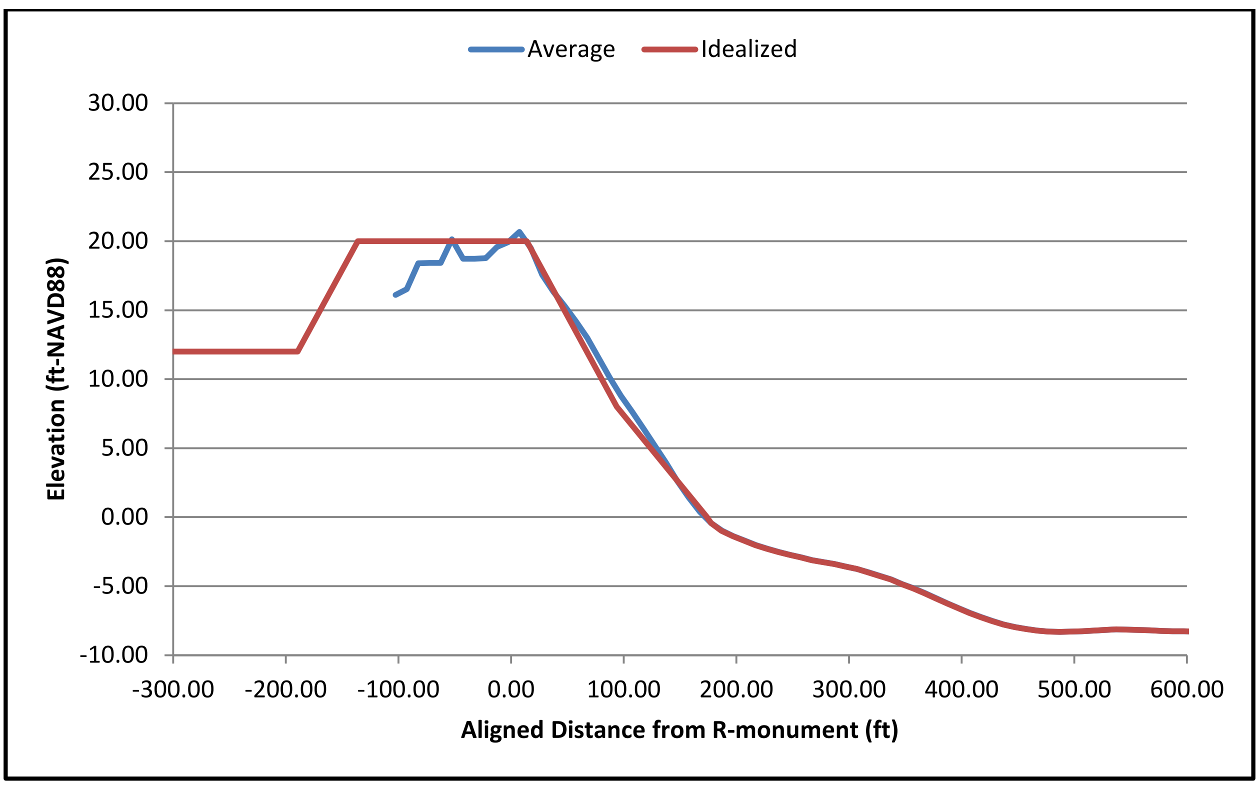

Figure A - 17. Averaged and idealized profiles: P3 grouping.

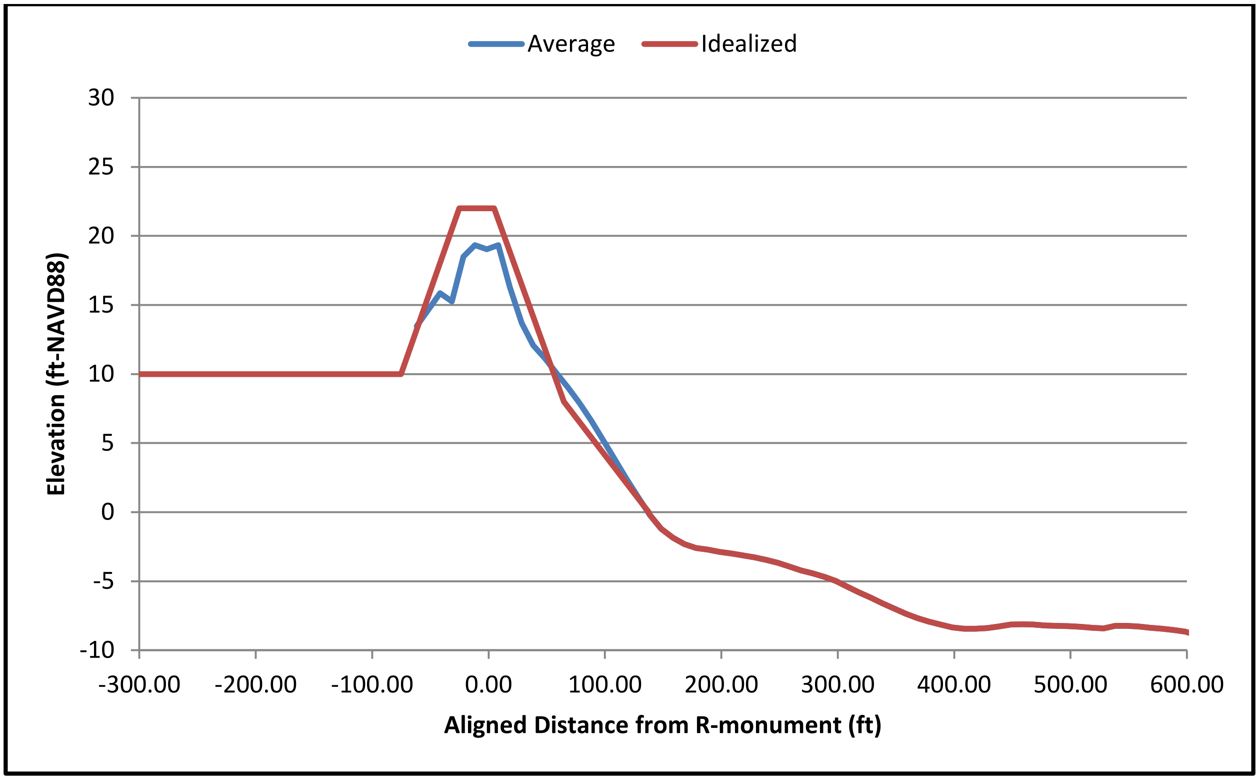

Figure A - 18. Averaged and idealized profiles: P4 grouping.

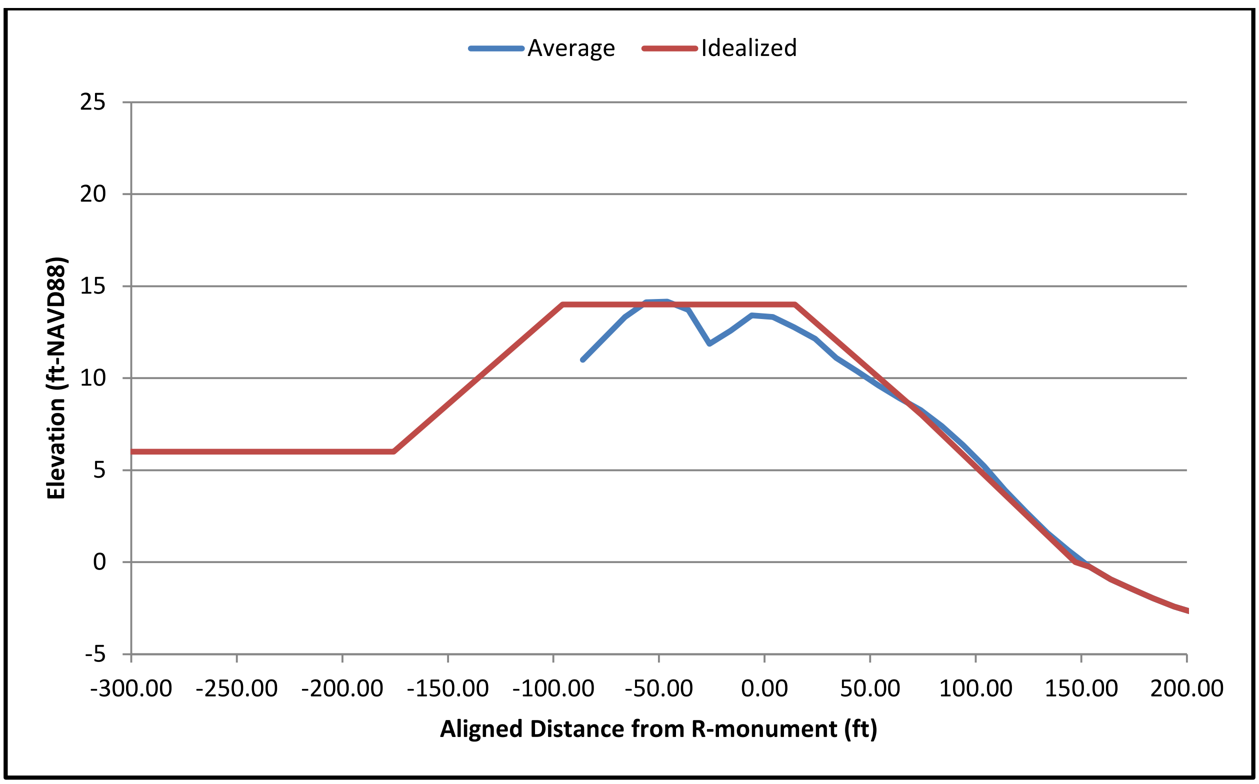

Figure A - 19. Averaged and idealized profiles: P5 grouping.

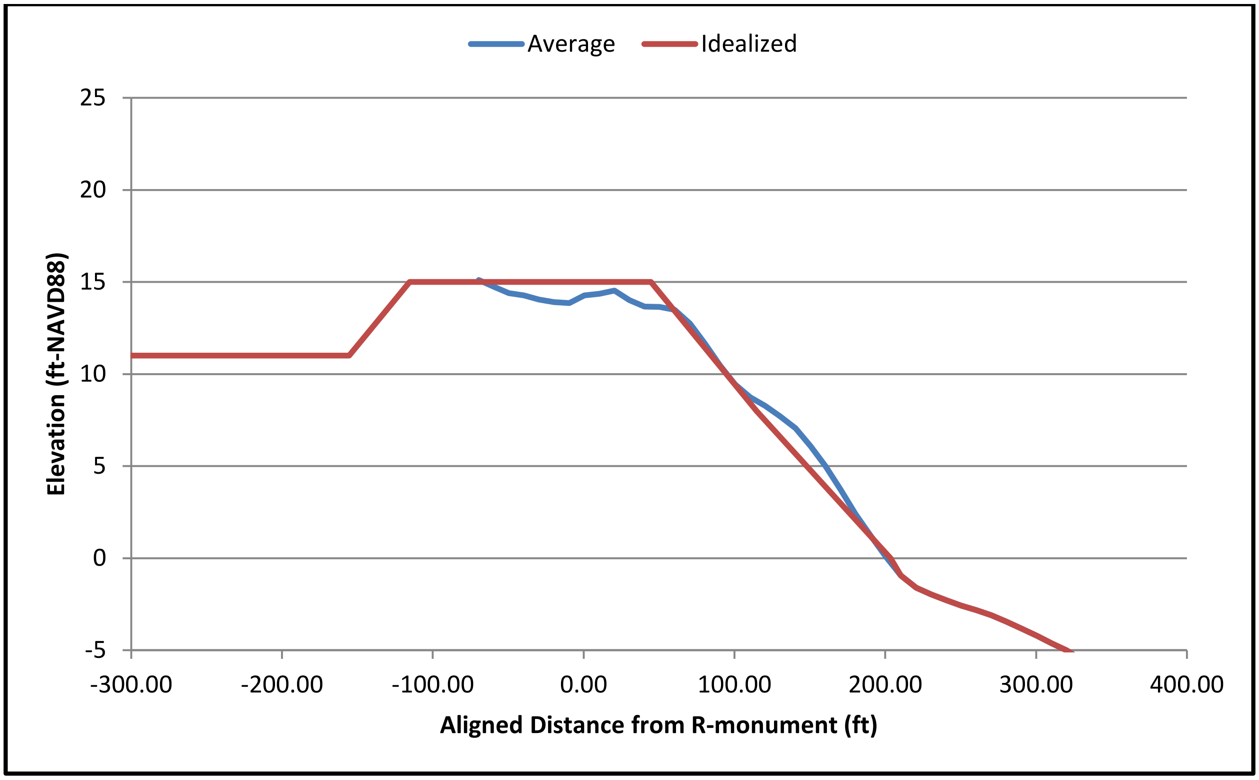

Figure A - 20. Averaged and idealized profiles: P6 grouping.

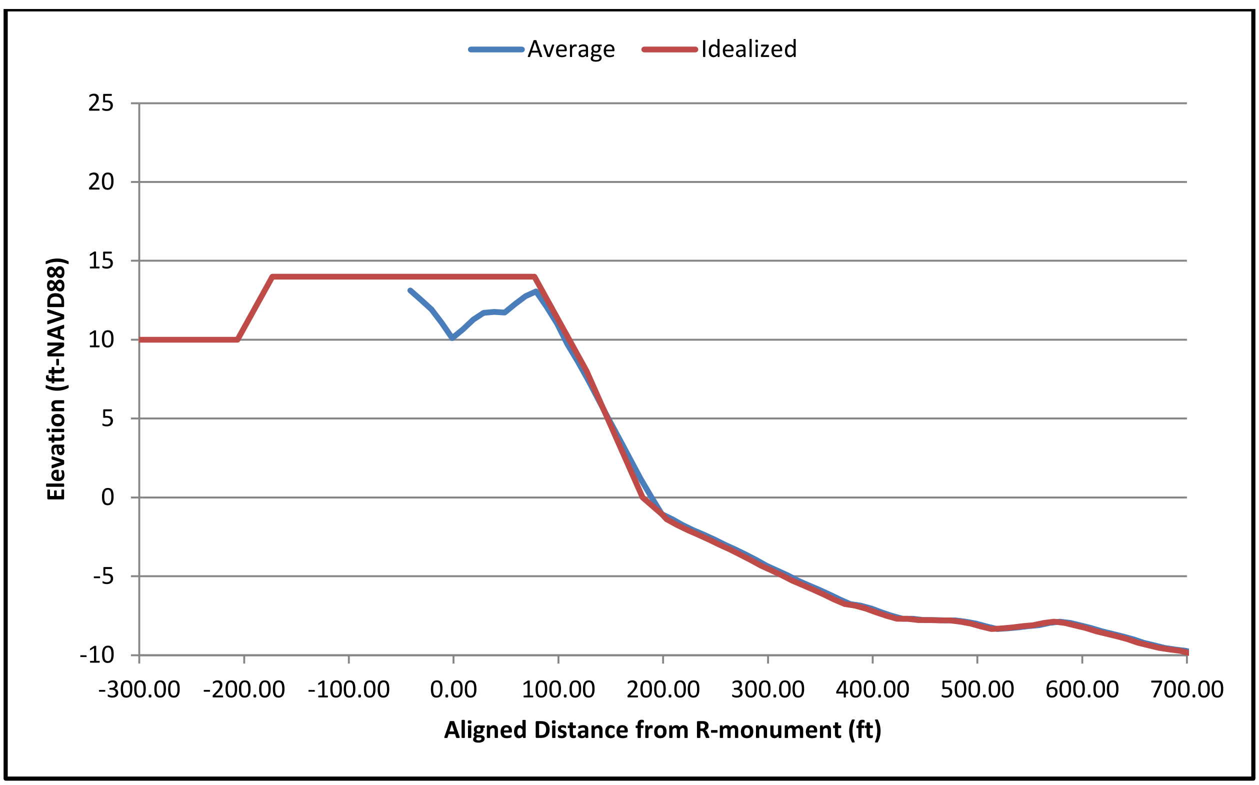

Figure A - 21. Averaged and idealized profiles: P7 grouping.

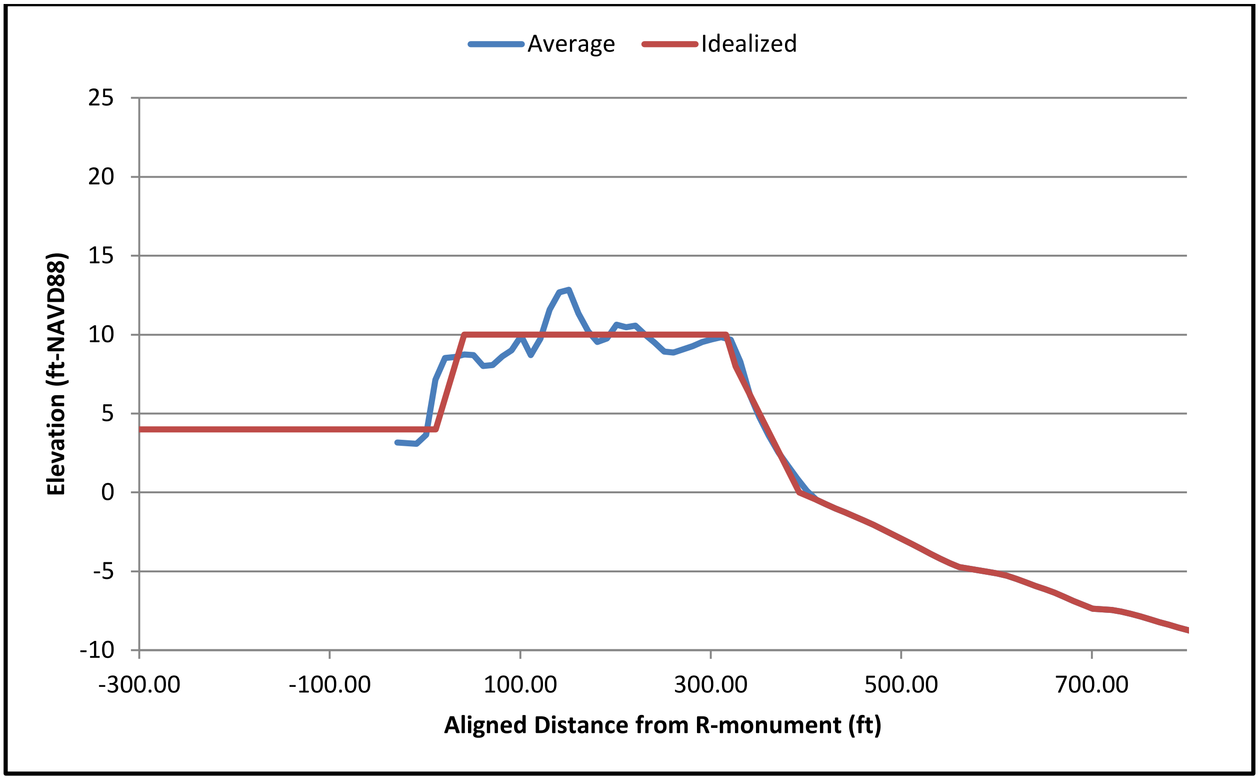

Figure A - 22. Averaged and idealized profiles: P8 grouping.

Table A - 14. Dimensions of idealized pre-storm representative profiles (existing).

Profile	R-monuments Represented	Upland Elevation (ft-NAVD88)	Dune Height (ft-NAVD88)	Dune Width (ft)	Dune Slope (V:H)	Berm Elevation (ft-NAVD88)	Berm Width (ft)	Foreshore Slope (V:H)
P1	R-84 to R-86; R-90 to R-93	14	16	50	1:10	8	0	1:10
P2	R-87 to R-89; R-94 to R-98	13	21	50	1:5	8	0	1:10
P3	R-100 to R-111	12	20	150	1:6.67	8	0	1:10
P4	R-112, R-114	10	20	30	1:5	8	0	1:9.09
P5	R-115 to R-117	6	14	110	1:10	8	0	1:9.09
P6	R-118 to R-119	11	15	160	1:10	8	0	1:11.11
P7	R-120 to R-121	10	14	250	1:8.33	8	0	1:6.67
P8	R-122	4	10	275	1:5	8	0	1:8.33

The idealized profiles described in Table A - 14 represent the existing shoreline condition. In order to provide Beach-*fx* SRD database entries representative of future shoreline conditions, with- and with-out the presence of a shore protection project, it was necessary to develop idealized profiles for a series of possible future conditions. Table A - 15 provides the array of future profile dimensions modeled for the study area. Note that elevations and slopes do not change between existing and future conditions.

Table A - 15. Range of dune and berm width values for representative profiles.

Profile	Dune Width				Berm Width			
	Minimum	Maximum	Increment	Added Dune Width	Minimum	Maximum	Increment	Added Berm Width
P1	10	70	10	20	0	100	20	100
P2	10	70	10	20	0	100	20	100
P3	15	180	15	30	0	100	20	100
P4	10	50	10	20	0	100	20	100
P5	10	130	20	20	0	100	20	100
P6	20	180	20	20	0	100	20	100
P7	25	275	25	25	0	100	20	100
P8	25	300	25	25	0	100	20	100

SBEACH Methodology

SBEACH simulates beach profile changes that result from varying storm waves and water levels. These beach profile changes include the formation and movement of major morphological features such as alongshore bars, troughs, and berms. SBEACH is a two-dimensional model that considers only cross-shore sediment transport; that is, the model assumes that simulated profile changes are produced only by cross-shore processes. Alongshore wave, current, and sediment transport processes are not included.

SBEACH is an empirically based numerical model, which was formulated using both field data and the results of large-scale physical model tests. Input data required by SBEACH describes the storm being simulated and the beach of interest. Basic requirements include time histories of wave height, wave period, water elevation, beach profile surveys, and median sediment grain size.

SBEACH simulations are based on six basic assumptions:

- Waves and water levels are the major causes of sand transport and profile change
- Cross-shore sand transport takes place primarily in the surf zone
- The amount of material eroded must equal the amount deposited (conservation of mass)
- A relatively uniform sediment grain size is distributed throughout the profile
- The shoreline is straight and alongshore effects are negligible
- Linear wave theory is applicable everywhere along the profile without shallow-water wave approximations

Once applied, SBEACH allows for variable cross shore grid spacing, wave transformation, randomization of input wave conditions, and water level setup due to wind. Output data consists of a final calculated profile at the end of the simulation, maximum wave heights, maximum total water elevations plus setup, maximum water depth, volume change, and a record of various coastal processes that may occur at any time-step during the simulation (accretion, erosion, over-wash, boundary-limited run-up, and/or inundation).

SBEACH Calibration

Calibration of the SBEACH model was performed during the 1984 period using wave height, wave period, and water level information from Hurricane Diana and a subtropical storm as inputs (Figure A - 23). Calibration of the model is required to ensure that the SBEACH model is tuned to provide realistic shore responses that are representative of the specific project location.

Pre- and post-storm shoreline profiles were obtained from FDEP from May 1984 and December 1984 surveys, respectively. Using the pre-storm profiles, SBEACH was then run with a range of values for an array of calibration parameters. Table A - 16 provides the relevant beach characteristic and sediment transport calibration parameters as well as their final calibrated values. Figure A - 24 provides a comparison of the measured data versus the calibrated model output. Calibration parameters were verified using wave height, wave period, and water level information from Hurricanes Floyd and Irene in 1999 and pre-and post-storm shoreline profiles from the March 1999 and October/November 1999 surveys which provided a better prediction of the post-storm survey than the calibration period (Figure A - 25).

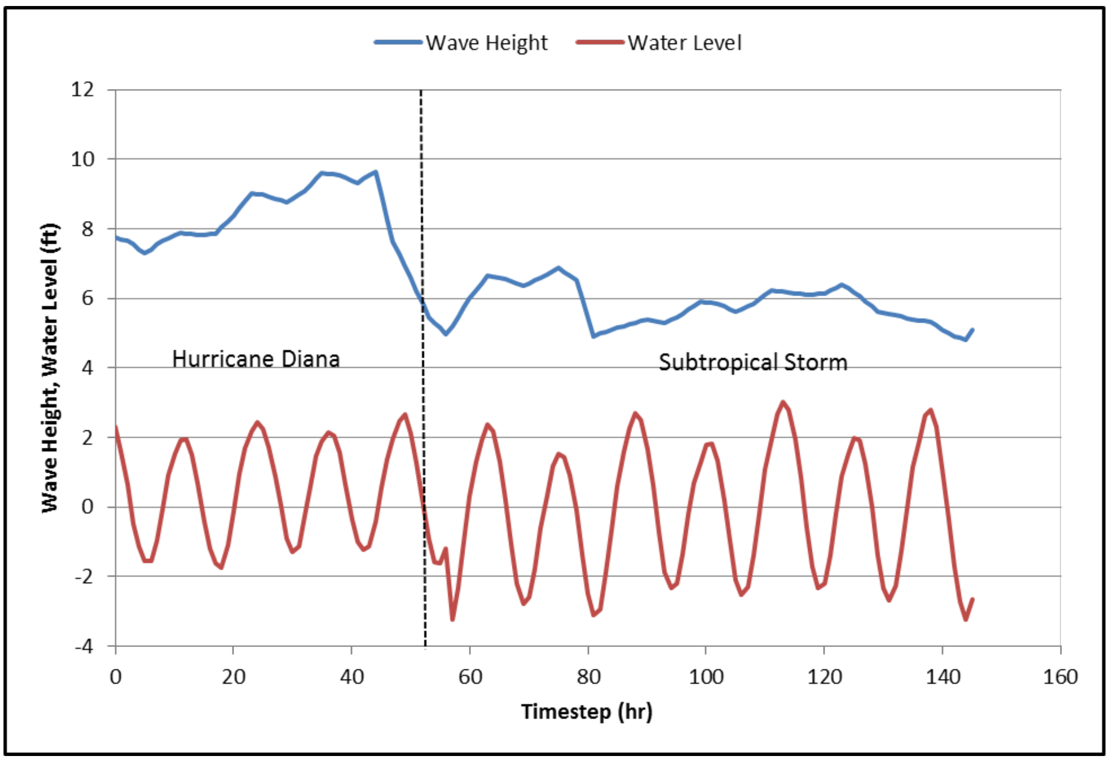

Figure A - 23. Hurricane Diana and 1984 subtropical storm wave and water level data for SBEACH calibration.

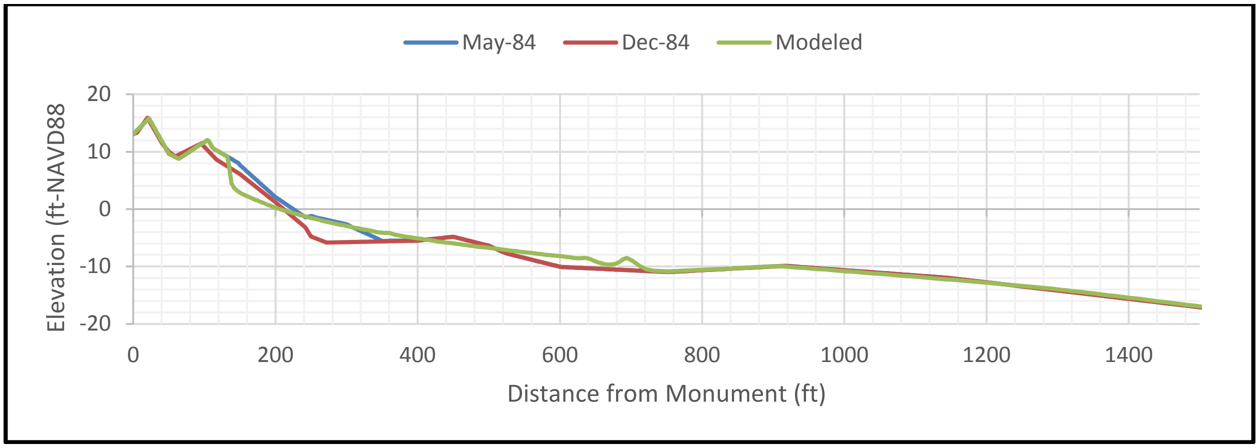

Figure A - 24. Measured and modeled profile change during SBEACH calibration.

Table A - 16. SBEACH calibrated beach characteristic and sediment transport parameters.

Beach Characteristic		Sediment Transport	
Parameter	**Calibrated Value**	**Parameter**	**Calibrated Value**
Landward Surf Zone Depth	1.0 ft	Transport Rate Coefficient	2.5e-06 (m^4/N)
Effective Grain Size	0.25 mm	Overwash Transport Parameter	0.005
		Coefficient for Slope-Dependent Term	0.005
Maximum Slope Prior to Avalanching	25	Transport Rate Decay Coefficient Multiplier	0.1
		Water Temperature	20°C

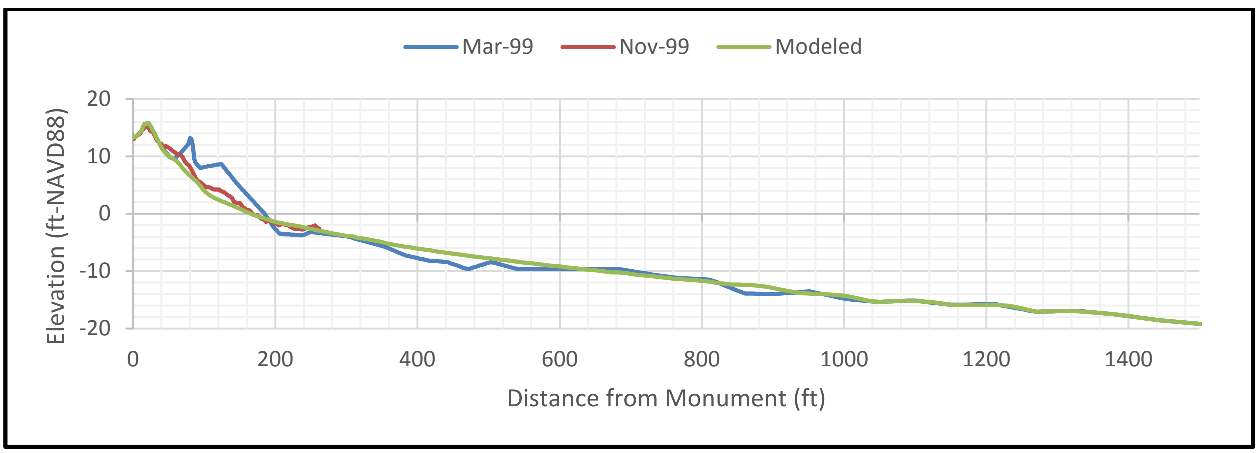

Figure A - 25. Measured and modeled profile change during SBEACH verification.

SBEACH Simulations

Calibrated SBEACH simulations were run for the idealized profiles, plus the range of widths provided in Table A - 15, in combination with each of the tropical and extra-tropical storms in the plausible storm database. From these profiles, changes in the key profile dimensions were extracted and stored in the SPV-Vilano Beach-*fx* SRD. The Beach-*fx* model extracts the profile responses stored in the SRD as storms impact the project area during the project simulation period which extends from present day through the anticipated initial construction date (2020) and ends with the 50-year planning horizon (2070).

Applied Erosion Rate

The Applied Erosion Rate (shoreline change rate in feet per year) is a Beach-*fx* morphology parameter specified at each of the model reaches. It is a calibrated parameter that, combined with the storm-induced change generated internally by the Beach-*fx* model, returns the historical shoreline change rate for that location.

The target shoreline change rate during Beach-*fx* calibration is an erosion or accretion rate derived from the MHW rate of change determined at each R-monument location (see **Historical Shoreline Change**). Although the MHW rate of change represents the historical behavior of the project shoreline, when it is calculated at single point locations, such as R-monuments, there is a high degree of variability between consecutive locations. This variability results in a similar variability in the Beach-*fx* results, specifically in project costs and predicted damages. Because this does not reflect actual shoreline behavior and leads to inconsistencies between adjacent economic reaches, the target shoreline change rate is determined by averaging adjacent MHW change rates to allow for smoother transitions along the length of the project shoreline. A three-monument smoothing window (approximately 3,000 ft) was applied four times to adequately reduce the noise in the MHW change rate signal. Figure A - 26 shows the smoothed target shoreline change rates along with the original MHW shoreline change rates (between 1972 and 2015) from which they were derived.

During Beach-*fx* calibration, Applied Erosion Rates were adjusted for each model reach and the Beach-*fx* model was run for 300 iterations of the 50-year project planning horizon. Calibration of the Beach-*fx* model is achieved when the rate of shoreline change, averaged over hundreds of life-cycle simulations, is equal to the target shoreline change rate.

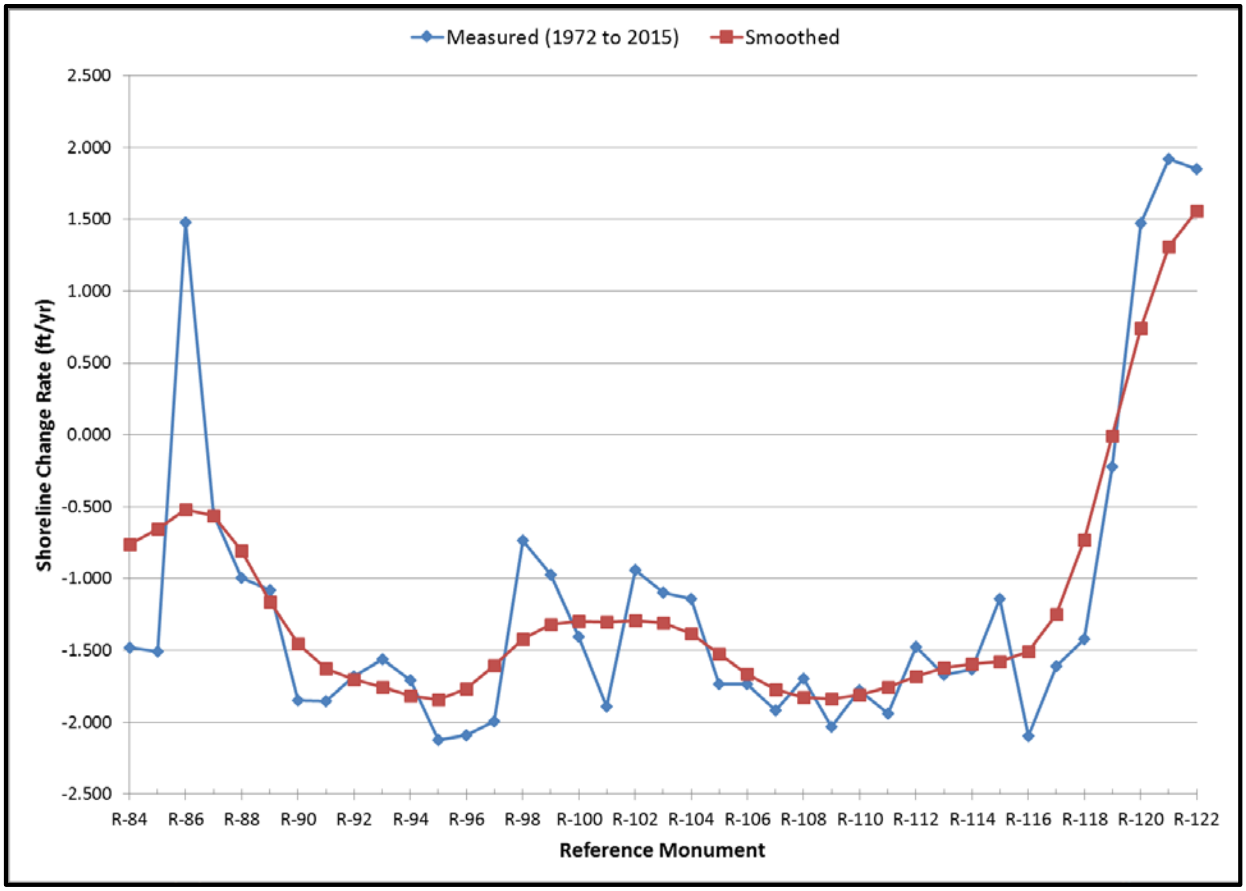

Figure A - 26. Target MHW shoreline change rate.

Project Induced Shoreline Change (GenCade)

The project induced shoreline change rate, also in feet per year, accounts for the alongshore dispersion of proposed beach nourishment alternatives. Beach-*fx* requires the use of shoreline change rates (beyond the Applied Erosion Rate) in order to represent the plan form diffusion of the beach fill alternatives after placement. The USACE one-dimensional shoreline change model GenCade was used to determine how the beaches of SPV and Vilano would respond to beach nourishment shore protection alternatives. The results from each beach fill alternative model run, including the no-fill without project condition, provided the plan form rate of change for each alternative. The difference in the rate of change for each alternative versus the without project condition was then used as input to the Beach-*fx* economic model to determine project benefits.

The GenCade model was developed by combining the USACE project-scale, engineering design-level shoreline change model GENESIS and the regional-scale, planning level model Cascade. The model can be set up and executed within the Surface-Water Modeling System (SMS) or executed as a stand-alone model through the MS-DOS interface and calculates the shoreline change and alongshore sediment transport due to waves. The original shoreline change model GENESIS was limited in its application to areas of sufficient distance from tidal inlets and to single littoral cells. By coupling the GENESIS and Cascade models, project areas represented in GenCade can span multiple littoral cells and include the features that separate the littoral cells such as inlets and structures (Frey et al., 2012).

Taylor Engineering, Inc. (2010) describes the development, calibration, and verification of the GenCade model in support of this study. The measured volume change over the 1986 to 1999 period was used to calibrate the model and verification of model parameters was determined using the volume changes measured over the period of 1999 to 2007. The Florida Coastal Forcing Project provided the wave data input for the GenCade model over the selected calibration and verification periods at the 10 m (33 ft) depth contour with dense alongshore spacing to account for any spatial differences in the wave field.

The unmodified shoreline used as input during model calibration served as the basis for determining the expected deviations in historic shoreline change rates that an advancement of the shoreline (i.e. beach nourishment) could cause. Beach nourishment alternatives explored in the present study were added in the GenCade model and simulated over the same period of time as the unmodified shoreline. The resulting shoreline changes were compared and the difference between the with-nourishment and without-nourishment model simulations represent the expected diffusion rate of a given beach nourishment alternative. Figure A - 27 shows the smoothed historic shoreline change rate (as presented in Figure A - 26), the diffusion rate predicted by GenCade (for a scenario of extending the shoreline 60 ft between reference monuments 103.5 and 116.5), and the superposition of the two. The result is an increase in shoreline recession rates within the simulated fill area (due to the project-induced perturbation of the shoreline) and a decrease in recession rates outside the fill area (as the fill material diffuses laterally outside the project limits). Figure A - 27 shows that immediately outside the project area the shoreline change rates become positive due to the influence of the simulated beach nourishment project.

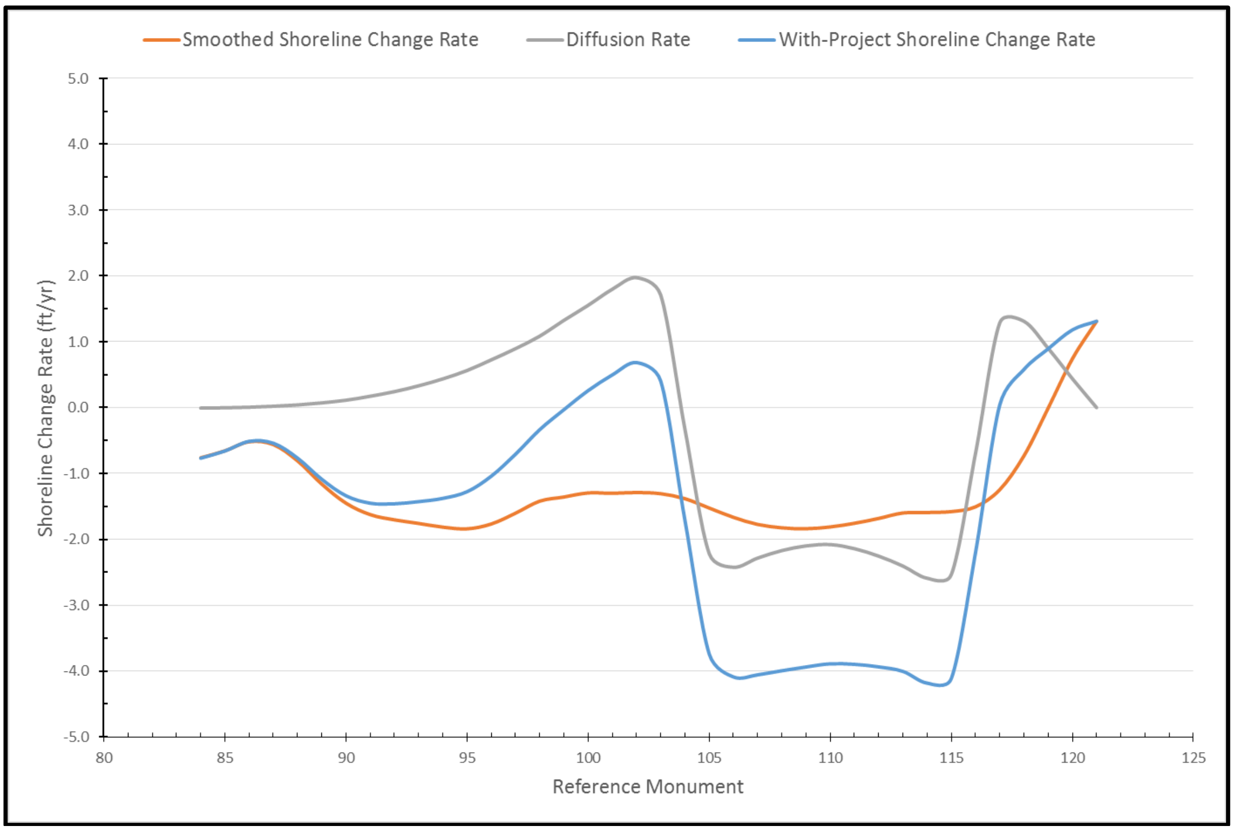

Figure A - 27. Shoreline change rates for with- and without-project simulations.

Post Storm Berm Recovery

Post storm recovery of eroded berm width after passage of a major storm is a recognized process. Although present coastal engineering practice has not yet developed a predictive method for estimating this process, it is an important element of post-storm beach morphology. Within Beach-*fx*, post-storm recovery of the berm is represented in an ad hoc procedure in which the user specifies the percentage of the estimated berm width loss during the storm that will recover over a given recovery interval. Based on review of available historical FDEP profiles that would qualify as pre- and post- storm, a recovery percentage of 90% over a recovery interval of 21 days was determined for the SPV-Vilano study area.

Economic Evaluation

The Beach-*fx* model analyzes the economics of shore protection projects based on the probabilistic nature of storm associated damages to structures in the project area. Damages are treated as a function of structure location and construction, the intensity and timing of the storms, and the degree of protection that is provided by the natural or constructed beach. Within the model, damages are attributed to three mechanisms:

- Erosion (through structural failure or undermining of the foundation)
- Flooding (through structure inundation levels)
- Waves (though the force of impact)

Although wind may also cause shoreline damage, shore protection projects are not designed to mitigate for impacts due to wind. Therefore, the Beach-*fx* model does not include this mechanism.

Damages are calculated for each model reach, lot, and damage element following each storm that occurs during the model run. Erosion, water level, and maximum wave height profiles are determined for each individual storm from the lookup values in the previously stored SRD. These values are then used to calculate the damage driving parameters (erosion depth, inundation level, and wave height) for each damage element.

The relationship between the value of the damage driving parameter and the percent damage incurred from it is defined in a user-specified “damage function”. Two Damage Functions are specified for each damage element, one to address the structure and the other to address its contents. Damages due to erosion, inundation, and wave attack are determined from the Damage Functions and then used to calculate a combined damage impact that reduces the value of the damage element. The total of all damages is the economic loss that can be mitigated by the shore protection project. A thorough discussion of the economic methodology and processes of Beach-*fx* can be found in the **Economics Appendix.**

Management Measures

Shoreline management measures that are provided for in the Beach-*fx* model are emergency nourishment, planned nourishment, and structure armoring. Emergency nourishments are generally limited beach fill projects conducted by local governments in response to storm damage. St. Johns County does not have a history of emergency nourishment. The absence of past emergency nourishment events prevents the assumption that future emergency nourishment events will occur, either with or without an authorized shore protection project in place. Therefore, this management measure was not included in the SPV-Vilano Beach-*fx* analysis.

Planned nourishments are handled by the Beach-*fx* model as periodic events based on design templates, triggers, and nourishment cycles. Nourishment templates are specified at the model reach level and include all relevant information such as order of fill, dimensions, placement rates, unit costs, and borrow-to-placement ratios. Planned nourishments occur when user defined nourishment triggers are exceeded and a mobilization threshold volume is met. At a pre-set interval (equal to one year in the present study), all model reaches which have been identified for planned nourishment are examined. In reaches where one of the nourishment threshold triggers is exceeded, the required volume to restore the design template is computed. If the summation of individual model reach level volumes exceeds the mobilization threshold volume established by the user, then nourishment is triggered and all model reaches identified for planned nourishment are restored to the design template.

Beach-*fx* allows for future armoring of structures if shoreline recession crosses an established threshold over the course of the life-cycle simulation. The threshold used for the SPV-Vilano study area is consistent with recent efforts to protect threatened structures within the study area and was set to approximately 30 ft from the seaward face of the structure for those properties deemed armorable. In locations where adjacent properties were already armored, the threshold trigger was set in line with the existing armor so a contiguous seawall was represented. A structure was determined to be armorable by following the regulations set forth by the State of Florida. If a structure was built prior to 1988 or the adjacent properties on either side are armored and the gap between them is less than 250 ft the structure is armorable.

Nourishment Design Templates

Beach-*fx* planned nourishment design templates are defined by three dimensions, the template dune height, template dune width, and template berm width. Berm elevations and dune and foreshore slopes remain constant based on the existing profiles. For the SPV and Vilano study area, each reach level nourishment template was developed based on a 0-, 10-, or 20-ft extension of the beach profile (from the dune to depth of closure) and five berm width extensions: 20-ft, 40-ft, 60-ft, 80-ft, and 100-ft. Model simulations were accordingly named with '0P20B' for a zero-foot extension of the entire profile with a 20-foot extension of the berm (Table A - 17). Template dune heights and widths in each case were set to the elevation and width of the existing idealized Beach-*fx* profile. Therefore, the existing 2015 dune and profile was rebuilt during model simulations if any portion of the dune template was damaged at the time a nourishment event was triggered.

Results from the future without-project (FWOP) model simulations provided the anticipated damages that occur over the 50-year life-cycle- which is the established time horizon for this study. The amount of damages varied throughout the study area due to differences in the existing profile conditions, historic erosion rates, and profile responses to storm events. Beach nourishment was not included for locations where FWOP damages were lower than the estimated cost of shore protection alternatives (see Economics Appendix Section 3.4.2). From the FWOP analysis, the area between R-92 and R-116 appeared most promising for testing future with project (FWP) beach nourishment alternatives. Given the lack of availability of public access in the SPV portion of the study area, and, therefore, the lack of Federal cost sharing for the area, an additional FWP area was identified and limited to only include R-103.5 to R-116.5 where adequate public access exists. Table A - 17 lists the initial FWP alternatives modeled in Beach-*fx* and the template dimensions of each alternative.

Table A - 17. Dune and berm features for initial array of nourishment alternatives.

Alternative Name	Project Length (mi)	Model Reaches	Dune Width Extension (ft)	Berm Width Extension (ft)
0P60B 104to116	2.6	104 to 116	0.00	60.00
0P40B 92to116	4.8	92 to 116	0.00	40.00
0P60B 92to116	4.8	92 to 116	0.00	60.00
10P60B 104to116	2.6	104 to 116	10.00	60.00
10P40B 104to116	2.6	104 to 116	10.00	40.00
0P40B 104to116	2.6	104 to 116	0.00	40.00
0P80B 104to116	2.6	104 to 116	0.00	80.00
10P80B 104to116	2.6	104 to 116	10.00	80.00
0P100B 104to116	2.6	104 to 116	0.00	100.00
20P20B 92to116	4.8	92 to 116	20.00	20.00
20P20B 104to116	2.6	104 to 116	20.00	20.00
10P20B 104to116	2.6	104 to 116	10.00	20.00
10P20B 92to116	4.8	92 to 116	10.00	20.00

Nourishment Triggers and Mobilization Threshold

Beach-*fx* planned nourishment design templates have three nourishment triggers: berm width, dune width, and dune height. Each trigger is a fractional amount of the corresponding template dimension that denotes the requirement for renourishment. During initial screening of project alternatives, the berm width, dune width, and dune height triggers were set at 0.5, 0.95, and 0.9, respectively, for alternatives which included a profile extension and a berm extension. Triggers were set at 0.0, 0.91, and 0.9, respectively, for alternatives which included only a profile extension. The mobilization threshold for all planned nourishment alternatives was varied depending on the projected volume requirement of each alternative. However, in order to maintain consistency among alternatives for comparison purposes, nourishment trigger volumes were set to approximately equal 80% of the calculated volume need of the alternative. Nourishment triggers and mobilization threshold are subjective parameters, based on local infrastructure and engineering judgment of when renourishment will become essential to the continued performance of the project. Application of these parameters for SPV and Vilano is discussed in greater detail in Project Volumes.

Beach-*fx* Project Design Alternatives

In order to determine the most effective and cost efficient protective beach design for the SPV and Vilano study area, alternatives were developed by combining the design reaches and nourishment templates discussed previously. Preliminary Beach-*fx* runs, limited to 30 iterations (as discussed in the Economics Appendix), allowed the initial array of alternatives to be screened down to those most likely to provide an effective and justified Federal project (see the **Main Text** for screening details). The remaining array of alternatives run with Beach-*fx* for 100 iterations of the 50-year period of Federal participation is presented in Table A - 18. These consist of two basic designs, a 40-ft or 60-ft extension of the berm and a 10-ft extension of the existing (2015) beach profile (from dune to depth of closure) with a 60-ft extension of the berm. No changes to the existing dune height were included. Typically the alternative with the greatest net benefits is selected as the tentatively selected plan (TSP), however, due to the lack of public access in South Ponte Vedra the TSP consists of a 60-ft berm between reference monuments 104 and 116 (Alternative ‘0P60B 104to116’).

Table A - 18. Array of alternatives with BCR and net benefits from 100-iteration model runs.

Alternative Name	Project Length (mi)	Model Reaches	Dune Width Extension (ft)	Berm Width Extension (ft)	BCR	Net Benefits ($)
0P60B 104to116	2.6	104 to 116	0.00	60.00	1.25	$ 8,573,000
10P60B 104to116	2.6	104 to 116	10.00	60.00	1.20	$ 7,473,000
0P40B 92to116	4.8	92 to 116	0.00	40.00	1.16	$ 8,844,000
0P60B 92to116	4.8	92 to 116	0.00	60.00	1.13	$ 7,957,000

Protective Beach Design

Based on Beach-*fx* model results and economic evaluation, project alternative '0P60B 104to116', a 60 ft extension of the berm and subaqueous beach profile, was identified as the TSP for nourishment of the beach between reference monument R-103.5 and R-116.5. A description of this shore protection plan is provided in the following sections.

Project Length

Project evaluation using Beach-*fx* started with the SPV and Vilano study area along 7.5 miles of shoreline, extending from FDEP monument R-84 to R-122 within St. Johns County. Note that this does not include the Summerhaven segment of the original study (R-197 to R-209), which was removed early in the study. The selected design provides a 60-ft wide berm extension for approximately 2.6 miles of the study area. The beach fill will be placed from R-103.5 to R-116.5 with tapers extending approximately 1,000 ft to the north of R-103.5 and approximately 1,000 ft to the south of R-116.5.

Project Design

The project design can be described by three factors, the dimensions of the dune, dimensions of the berm, and shoreline slopes.

Project Dune

Existing dune elevations in the project area are between 10 and 21 ft-NAVD88, generally increasing moving from south to north (Table A - 19). Evaluation of the design alternatives has shown that the existing elevations, when combined with berm and/or dune extension, provide sufficient protection. Therefore, no additional elevation is included in the selected design plan.

Table A - 19. Generalized dune characteristics of the study area.

Profile*	R-monuments	Dune Height (ft-NAVD88)	Dune Width (ft)
P1	R-84 to R-86; R-90 to R-93	16	50
P2	R-87 to R-89; R-94 to R-98	21	50
P3	R-100 to R-111	20	150
P4	R-112, R-114	20	30
P5	R-115 to R-117	14	110

Profile*	R-monuments	Dune Height (ft-NAVD88)	Dune Width (ft)
P6	R-118 to R-119	15	160
P7	R-120 to R-121	14	250
P8	R-122	10	275

*Shaded portions indicate areas that are not part of the TSP.

Within the alongshore limits of a given alternative the values in Table A - 19 provide the minimum dune dimensions required during a nourishment in the Beach-*fx* model. Within the area encompassed by the TSP (portions of Table A - 19 not shaded) the average dune width ranges between 110 ft and 150 ft with the exception of the R-112 to R-114 area which is only 30 ft wide. Alternatives that feature a profile extension include an additional 10-ft or 20-ft dune width beyond the minimum dune dimensions in Table A - 14, as discussed in the Nourishment Design Template section. Although the TSP alternative does not include an *extension* of the existing dune, any erosion of material from the existing idealized dune template (i.e. the 2015 generalized profile) is replaced during nourishment events in model simulations. Therefore, the existing 2015 idealized dune template will be restored accordingly during initial construction and renourishment of the project and is noted as an important feature of the HSDR project.

Project Berm

The design berm elevation for the project area is 8.0 ft-NAVD88, which is approximately at the natural berm elevation. Restricting the design berm elevation to the natural berm elevation minimizes scarping of the beach fill as it undergoes profile equilibration. Vertical scarps can hinder beach access by nesting sea turtles, and may also pose safety problems for recreational beach use. Other reasons for following the natural berm elevation are related to storm damage protection. A berm constructed at a lower elevation would increase the probability of overtopping by relatively frequent storms, thereby offering less protection to upland development and/or existing dunes. A higher berm elevation could result in problems related to backshore flooding due to excessive rainfall or wave overtopping. A higher berm may also be more susceptible to wind-induced erosion.

The TSP includes a design berm for the refined project area (R-103.5 to R-116.5) modeled in Beach-*fx* as a 60-ft extension of the +8 ft-NAVD88 contour sloping 1V:10H to the 0.0 ft-NAVD88 contour and a 60-ft extension of the existing subaqueous profile. Figure A - 28 shows the existing idealized profile and modeled nourishment template for Profile 3 (which represents R-100 to R-111). Construction of the TSP alternative will likely include a berm that is wider than what was modeled in order to account for the additional material below the 0.0 ft-NAVD88 contour and the difference in idealized/modeled slope versus practical construction slopes. Following construction and equilibration of the profile, the beach dimensions are expected to approximate the idealized profile.

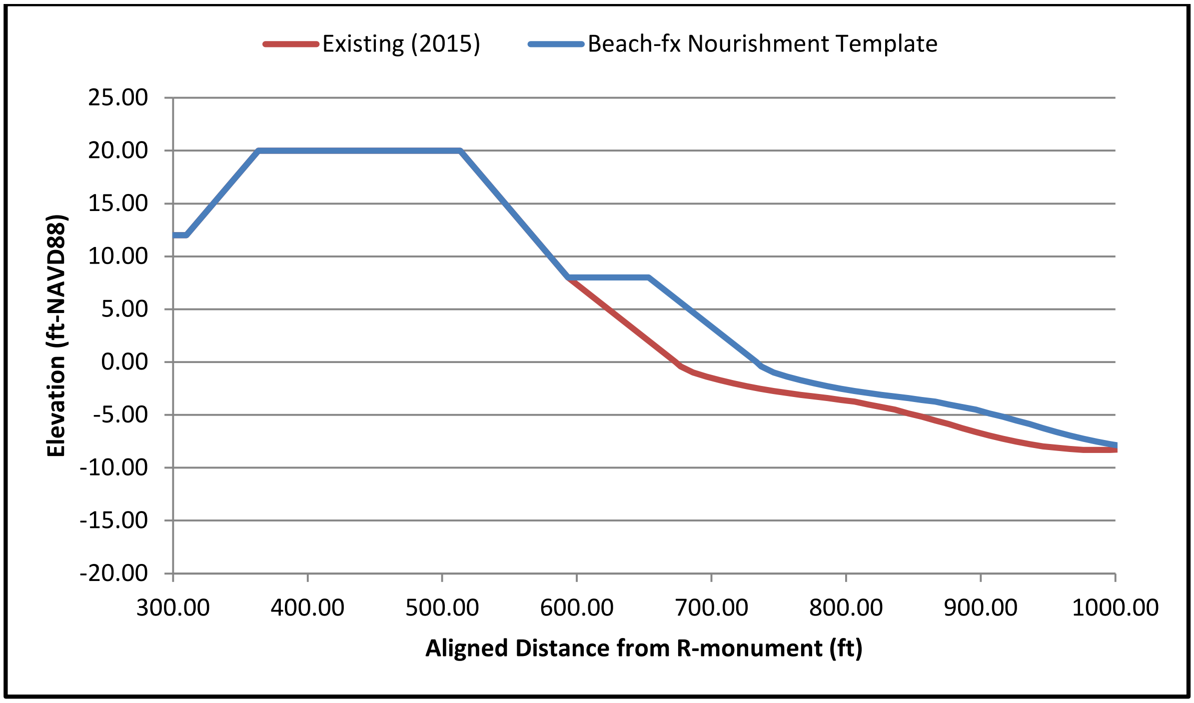

Figure A - 28. Existing profile and Beach-*fx* nourishment template for idealized Profile 3.

Project Beach Slopes

After adjustment and sorting of the placed material by wave action, the material is expected to adjust to an equilibrium beach slope, similar to the native beach. In SPV and Vilano, the native beach slopes in the area of the TSP vary between 1V (vertical) on 5H (horizontal) to 1V:10H at the dune, between 1V:9.09H and 1V:10H from the berm to 0.0 ft-NAVD88, and around 1V:50H from 0.0 to -12 ft-NAVD88. The estimate of the slope of the material after adjustment is based on averaging the beach profile slopes of the native beach from the mean low water datum to the approximate location of the 12 foot depth contour. Offshore slopes between the 0.0 ft and -30 ft-NAVD88 contour vary between 1V:75H and 1V:110H within the area of the TSP. Since, sand from the project borrow site was determined to be a near match to the gradation of the existing beach it is expected that the beach fill will equilibrate to a shape similar to the existing profile.

Table A - 20. Project dune and foreshore slopes.

Profile	R-monuments	Dune Slope	Foreshore Slope	Offshore Slope 0 to -12 ft-NAVD88	Offshore Slope 0 to -30 ft-NAVD88
P3	R-100 to R-111	1V:6.67H	1V:10.0H	1V:53H	1V:71H
P4	R-112, R-114	1V:5.00H	1V:9.09H	1V:50H	1V:93H
P5	R-115 to R-117	1V:10.0H	1V:9.09H	1V:48H	1V:110H

It is unnecessary and impractical to artificially grade beach slopes below the low water elevation since they will be shaped by wave action. For this reason, the front slope of the beach fill placed at the time of construction or future renourishment may differ from that of the natural profile. The angle of repose of the hydraulically placed material depends on the characteristics of the fill material and the wave climate in the project area. With steep initial slopes, the material will quickly adjust to the natural slopes.

Project Volumes

Traditionally, beach fill designs are presented as a set of three cross-sectional templates: the design template, which is based on an equilibrium profile translated seaward by the desired width of the berm or MHW extension; the advanced nourishment template, which represents the volume of material that is expected to erode between successive renourishment intervals; and, the construction template, which includes both the design and advanced fill quantities, but incorporates the wider berm and steeper slope that reflects the capabilities of the construction equipment. The design template is the minimum beach profile to be maintained, while the advance nourishment template contains the volume of material that anticipated to erode over the economically optimized renourishment interval while protecting the design template. This traditional approach, however, does not conform well to the probabilistic nature of the Beach-*fx* model or the methodology used for determining renourishment requirements.

Beach-*fx* begins with the desired design template (i.e. the 60 ft berm extension, Figure A - 28). Each life-cycle simulation then applies randomly generated storms, storm erosion, and natural background plus project-induced shoreline change rates. At one year intervals the model evaluates the resulting shoreline against two criteria (1) whether shoreline position at one or more reaches has exceeded one or more planned nourishment triggers and (2) whether the total volume presently required to fill the original design template exceeds the mobilization threshold. If both criteria are met then a nourishment event is initiated as long as the current simulation year is within the planned nourishment period (2020 to 2070 for this study). There are three planned nourishment triggers in Beach-*fx*: berm width, dune width, and dune height. Each trigger indicates what percentage of the design template berm width, dune width, or dune height must be present to *prevent* a renourishment (For example, a 90% (0.90) dune width trigger means that 90% of the total design template dune width must remain intact. If 10% or more of the template dune width is eroded, the first criteria for initiating a planned renourishment event has been met). Should any planned nourishment trigger be exceeded in one or more reaches, then Beach-*fx* computes the volume required to fill the original design template over all of the planned nourishment reaches and compares that volume to the mobilization volume threshold. If the mobilization volume threshold is exceeded a nourishment event over all planned nourishment reaches occurs and the model continues through the remainder of the life-cycle.

For the TSP alternative (‘0P60B 104to116’), the berm width, dune width, and dune height planned nourishment triggers were set at 0.5, 0.95, and 0.9, respectively. The mobilization threshold was initially set to 650,000 cy. Together, the triggers and the mobilization threshold allow for the optimization of the beach fill based on the physical dimensions of the project as well as assumptions regarding tolerable erosion limits and reasonable fill volumes. Sensitivity analysis of the nourishment triggers and mobilization threshold indicated that the mobilization threshold volume was the dominant parameter for optimizing project net benefits. Employing 100,000 cubic yard increments between 550,000 and 1,150,000 cy, a mobilization threshold of 750,000 was found to be (when combined with the above nourishment triggers) the most optimal threshold value for the ‘0P60B 104to116’ TSP alternative. Below the 750,000 cy threshold benefits were increased for an incrementally greater cost and above the optimized threshold costs were decreased for incrementally less benefits. For more details on mobilization threshold optimization see Economic Appendix Section 3.4.4.

Each complete Beach-*fx* model run consists of 100 iterations, each iteration representing the 50-year period of Federal participation for the project. Each iteration, therefore, has a unique volume requirement for initial construction and each renourishment. Based on the 0P60B alternative (100 iteration runs), a range of volumes was determined for each initial fill event and each subsequent renourishment event. Model runs were made for each of the three sea level rise cases: Base, Intermediate, and High. Table A - 21 provides volume statistics based on the 100 iterations of Beach-*fx* runs for the Base, Intermediate, and High sea level rise (SLR) scenarios. This table also provides the number of expected renourishment events.

Table A - 21. Beach-*fx* volume statistics for 100 iterations of TSP with SLR scenarios.

	Base SLR Scenario		
Volume Statistic	**Initial**	**Renourishment***	**Total**
Average (cy)	**1,310,000**	**866,000**	**4,267,000**
Max (cy)	1,844,000	993,000	5,790,000
Min (cy)	998,000	803,000	2,698,000
Std. Dev. (cy)	189,000	64,000	540,000

*Average interval 12 years

	Base SLR Scenario		
Volume Statistic	**Initial**	**Renourishment***	**Total**
Average (cy)	**1,424,000**	**851,000**	**5,205,000**
Max (cy)	1,950,000	978,000	6,438,000
Min (cy)	1,087,000	780,000	3,899,000
Std. Dev. (cy)	191,000	73,000	517,000

*Average interval 10 years

	Base SLR Scenario		
Volume Statistic	**Initial**	**Renourishment***	**Total**
Average (cy)	**1,614,000**	**788,000**	**6,829,000**
Max (cy)	2,114,000	867,000	8,160,000
Min (cy)	1,262,000	738,000	5,557,000
Std. Dev. (cy)	186,000	92,000	523,000

*Average interval 7 years

Project Construction

The TSP for the study area results in a 60 foot berm extension from the +8.0 ft-NAVD88 2015 contour out to the depth of closure between reference monuments 103.5 and 116.5. Table A - 22 lists the Northing and Easting coordinates of the +8.0 ft-NAVD88 2015 contour along the TSP project area. This contour includes perturbations due to the natural undulations of the shoreline as well as shoreline armoring (revetments) in select locations, but overall is rather smooth and straight. In order to ensure that the nourishment project provides the maximum benefit, it will be necessary during the project engineering and design (PED) phase to establish a smooth, relatively straight base construction line that will allow the project to perform as predicted during the Beach-*fx* shoreline analysis. The location of the +8.0 ft-NAVD88 2015 contour serves as the basis for creating the baseline which will be tailored to provide the approximate amount of material predicted for initial construction by Beach-*fx*.

As previously discussed, the front slope of the beach fill placed at the time of construction or future renourishment may differ from that of the natural profile. This reflects the capabilities of the construction equipment that will be used to build the shore protection project. Within the first year or two after placement of the beach fill, the construction profile will be reshaped by waves into an equilibrium profile, causing the berm to retreat to a position more characteristic of the project design template. Analysis of monitoring surveys collected during initial equilibration should anticipate higher erosion/recession rates than observed historically. However, the shoreline change rates over longer periods of time should reflect the pre-project erosion/recession rate plus the additional diffusion losses resulting from the beach fill.

Table A - 22. Reference monument control and location of the 2015 +8 ft-NAVD88 contour.

	Control		+8 ft NAVD88 Contour		
Monument	Easting*	Northing*	Range from Mon. (ft)	Easting*	Northing*
R-103	558,321.91	2,048,381.47	79.86	558,399.04	2,048,402.14
R-104	558,557.68	2,047,420.02	55.90	558,611.67	2,047,434.49
R-105	558,792.65	2,046,454.35	79.32	558,869.27	2,046,474.88
R-106	559,073.67	2,045,513.26	63.23	559,134.74	2,045,529.62
R-107	559,357.88	2,044,435.73	69.50	559,425.01	2,044,453.72
R-108	559,630.27	2,043,408.20	71.63	559,699.46	2,043,426.74
R-109	559,870.56	2,042,423.61	97.99	559,965.22	2,042,448.97
R-110	560,153.48	2,041,337.66	101.71	560,251.72	2,041,363.98
R-111	560,466.29	2,040,320.32	64.78	560,528.86	2,040,337.09
R-112	560,768.11	2,039,325.32	55.98	560,822.19	2,039,339.81
R-113	561,043.54	2,038,355.17	61.36	561,102.81	2,038,371.05
R-114	561,335.16	2,037,423.69	36.13	561,370.06	2,037,433.04
R-115	561,648.28	2,036,348.78	84.67	561,730.06	2,036,370.69
R-116	561,962.61	2,035,333.23	102.47	562,061.59	2,035,359.75
R-117	562,336.86	2,034,275.87	72.67	562,407.05	2,034,294.68

*Points reported in feet, NAD83/90.

Table A - 23. Mean high water range and coordinates from 2015 survey.

	Mean High Water Contour		
Monument	Range from Mon. (ft)	Easting*	Northing*
R-103	161.8	558,478.20	2,048,423.35
R-104	158.0	558,710.30	2,047,460.91
R-105	150.6	558,938.12	2,046,493.33
R-106	119.6	559,189.19	2,045,544.21
R-107	116.7	559,470.60	2,044,465.93
R-108	132.1	559,757.87	2,043,442.39
R-109	149.7	560,015.16	2,042,462.36
R-110	165.6	560,313.44	2,041,380.52
R-111	131.9	560,593.70	2,040,354.46
R-112	147.8	560,910.87	2,039,363.57
R-113	136.3	561,175.20	2,038,390.45
R-114	121.0	561,452.04	2,037,455.01
R-115	142.0	561,785.44	2,036,385.53
R-116	137.8	562,095.71	2,035,368.90
R-117	154.8	562,486.39	2,034,315.94

*Points reported in feet, NAD83/90.

Renourishment Events

Traditionally, renourishment events take place based on both an economically optimized renourishment interval and the physical performance of the project. Project performance, in the past, has been determined by assessing the condition of the design template. Should the design template be breached, the project is no longer providing the required level of protection and is considered for renourishment. Part of this consideration is how close in time the project may be to the designated renourishment interval.

While the basic principles of renourishment still apply, due to the probabilistic nature of Beach-*fx* and the way in which the model assesses renourishment requirements, a new means of assessing project performance must be employed. The former concepts of "design template" and "advance fill" are no longer applicable in the traditional sense. As shown in Figure A - 28 the existing dune and 60-ft berm extension template acts as the "advance fill", while the existing beach profile is the minimum acceptable profile (making it akin to what was formerly the "design template").

Planning of renourishment events will be based on performance of the project fill. A survey of the project area (such as a monitoring or post-storm survey) will be analyzed to determine if berm erosion is progressing as expected. Volume changes between the latest survey, the design template, and the construction template will be calculated. If the total volume required to restore the receded profiles exceeds the optimized mobilization threshold volume (750,000 cy), then a renourishment event is recommended. The decision to renourish may then be made based on traditional concerns, such as budget cycle and available funding. If projections of berm widths and volume changes suggest that a renourishment event is needed before the average interval found using Beach-*fx* (12 years) then the schedule should be adjusted accordingly.

Project Monitoring

Physical monitoring of the recommended project is necessary to assess project performance and to ensure that project functionality is maintained throughout the 50-year period of Federal participation in the project. The monitoring plan will be directed primarily toward accomplishing systematic measurements of the beach profile shape. Profile surveys should provide accurate assessments of dune and beach fill volumes and a basis for assessing post-construction dune and beach fill adjustments, as well as variation in the profile shape due to seasonal changes and storms. Monitoring will play a vital role in determining if project renourishment is necessary. Post construction monitoring activities include topographic and bathymetric surveys of the placement area (from the stable upland areas out to the depth of closure) and adjacent areas on an annual basis for 3 years following construction and then biannually until the next construction event. The cost for this post construction monitoring is included in the shared total project cost.

Other monitoring efforts include bathymetric surveying of the borrow site, which will be done as part of the pre-construction engineering and design (PED) phase prior to each nourishment. Since the existing St. Johns County SPP is required by permit to conduct surveys of the entire ebb shoal and inlet channel area, and the likelihood of such a permit requirement for construction of the present project, costs will likely be shared between the two projects.

Measured wind, wave, and water level information will be obtained from the best available existing data sources. This data will be applied in support of previously discussed monitoring efforts. It will also be used to periodically assess the state of sea level rise and to determine if reassessment of the project volumes and/or renourishment intervals based on an intermediate or high SLR case is required.

Summary

This appendix summarizes the engineering investigation and design of a shore protection project proposed for the SPV and Vilano portions of St. Johns County, Florida. The result of the study shows that shore protection measures are feasible for a portion of the study shoreline. Through optimization of developed alternatives, the selected alternative consists of beach nourishment/renourishment along approximately 2.6 miles of shoreline between FDEP monuments R-103.5 to R-116.5 at approximately 12-year intervals using material obtained from the St. Augustine Inlet complex.

The design beach fill template is characterized by a 60-ft extension of the 2015 beach profile between the berm (at +8 ft-NAVD88) and depth of closure (about -20 ft-NAVD88). Note that any erosion to the dry beach portion of the generalized 2015 profile (including the dune) will be restored during initial construction. The expected volume of nourishment material required for initial construction and renourishment events under the Base SLR case equal 1,310,000 cy and 866,000 cy, respectively. These values represent the average volumes from 100 unique Beach-*fx* model simulations that each spanned the 50 year planning horizon. Benefits and cost information for the selected alternative can be found in the Economics Appendix.

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