FLAGLER COUNTY, FLORIDA

HURRICANE AND STORM DAMAGE REDUCTION PROJECT
FINAL INTEGRATED FEASIBILITY STUDY AND
ENVIRONMENTAL ASSESSMENT

Appendix A

Engineering

US Army Corps
of Engineers ®
Jacksonville District

Flagler County Hurricane and Storm Damage Reduction Study

Final Integrated Feasibility Study and Environmental Assessment

Engineering Appendix

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Background

Flagler County is located on the northeast coast of Florida approximately midway between the Florida-Georgia state line and Cape Canaveral (Figure A- 1). The county is bounded to the north by St. Johns County and to the south by Volusia County. Flagler County has approximately 18 miles of sandy shoreline located on a coastal barrier island that varies in width from approximately 800 to 5,000 feet. The coast has no inlets or embayments and the beaches are typically fronted by steep dune faces or rock revetment. The Flagler County shoreline is subject to erosion caused by both and other natural shoreline processes. The purpose of this study is to assess the feasibility of providing Federal Hurricane and Storm Damage Reduction (HSDR) measures to portions of the Flagler County shoreline.

Four study reaches have been identified for Flagler County (Figure A- 1). One study reach (Marineland) is separate from the others, forming a north project segment. The Marineland reach extends from Florida Department of Environmental Protection (FDEP) DNR Monuments R-1 to R-4. The south project segment consists of the three remaining study reaches: Painters Hill (R-50 to R-60), Beverly Beach (R-60 to R-67), and Flagler Beach (R-67 to R-100). The purpose of this appendix is to document efforts related to the analysis of shoreline change and the design of remedial measures to alleviate beach erosion problems within the project area.

Problem Identification

In the past, beaches of Flagler County have generally experienced substantial erosion due to the combined effects of winds, waves, and tides. The severity of erosion in some areas is demonstrated 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 this appendix include quantification of existing beach erosion problems and the design of corrective measures. Quantification efforts involve analysis of historical shoreline positions, estimation of longshore 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 mechanisms of sand transport along the Florida shoreline. Flagler County lies at about 29° degrees latitude, slightly north of the tropical trade wind zone. Winds in this region vary seasonally with prevailing winds ranging from the northeast though the southeast. The greatest velocities originate from the north-northeast quadrant in winter months and from the east-southeast quadrant in the spring, summer, and early fall.

Figure A- 1. Flagler Project Location and Study Reaches

Wind data offshore of the project area is available from the USACE Wave Information Study (WIS) Program. WIS hindcast data are generated using the numerical hindcast model WISWAVE (Hubertz, 1992). WISWAVE is driven by wind fields overlaying a bathymetric grid. Model output includes significant wave height, peak and mean wave period, peak and mean wave direction, wind speed, and wind direction.

There are 523 WIS stations along the Atlantic Coast. WIS Station 63442 is representative of offshore deep water wind and wave conditions for the project area. Table A- 1 provides a summary of wind data from WIS Station 63422, located at latitude 29.58, longitude -81.0 (about 3 miles northeast of Flagler Beach - Figure A- 2). This table contains a summary of average wind speeds and frequency of occurrence broken down into eight 45 degree angle-bands. This table indicates that winds are fairly evenly distributed between the northeast and south directions. Due to its orientation, winds from the north-northeast to south-southeast have the most significant impact on the Flagler shoreline.

Table A- 1. Average Wind Conditions

Wind Direction (from)	WIS Station #63442 (1980 – 1999)	
	Percentage Occurrence (%)	Average Wind Speed (mph)
North	10.3	14.0
Northeast	15.3	13.4
East	14.6	11.1
Southeast	12.6	10.0
South	14.5	10.4
Southwest	13.4	10.9
West	9.5	13.3
Northwest	10.0	15.1

Wind conditions in Coastal Florida are seasonal. A further breakdown of the wind data provides a summary of the seasonal conditions (Table A- 2).

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. This post-frontal "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.

The summer months (June through September) are characterized by southeast trade winds and tropical weather systems traveling west to 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 longshore and cross-shore sediment transport at the site. These intense seasonal events will be discussed in greater detail under **Storm Effects** (page A-8).

Table A- 2. Seasonal Wind Conditions

Month	WIS Station #63442 (1980 – 1999)	
	Average Wind Speed (mph)	Predominant Direction (from)
January	14.6	N-NE
February	14.6	N-NE
March	14.0	E
April	12.0	E
May	10.2	E-SE
June	9.5	S-SE
July	9.5	S-SE
August	9.3	S-SE
September	10.7	E-NE
October	13.2	E-NE
November	14.0	E-NE
December	13.8	N-NE

Waves

The energy dissipation that occurs as waves enter the nearshore zone and break is the principal method of sediment transport. Wave height and period, in combination with tides and storm surge, are the most important factors influencing the behavior of the shoreline. The Flagler County study area is exposed to both short period wind-waves and longer period open-ocean swells originating predominantly from north-northeast to south-southeast directions.

Damage to the Flagler County shoreline and upland development is attributable to large storm waves produced primarily by tropical disturbances, including hurricanes, during the summer months and by “northeasters” during the late fall and winter 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 as well as local storms. Most hurricanes and tropical storms traversing northward through the Atlantic within several hundred miles of the east coast are capable of producing large swells. These swell can propagate long distances, causing erosion along the Flagler County shoreline.

Wave data for this report were obtained from the long-term USACE WIS hindcast database for the Atlantic coast of the U.S. This 20-year record extends from 1980 through 1999, 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 #63442, located 3 miles offshore of the study area in 66 feet of water. The location of WIS station #63422 relative to the study area is shown in Figure A- 2.

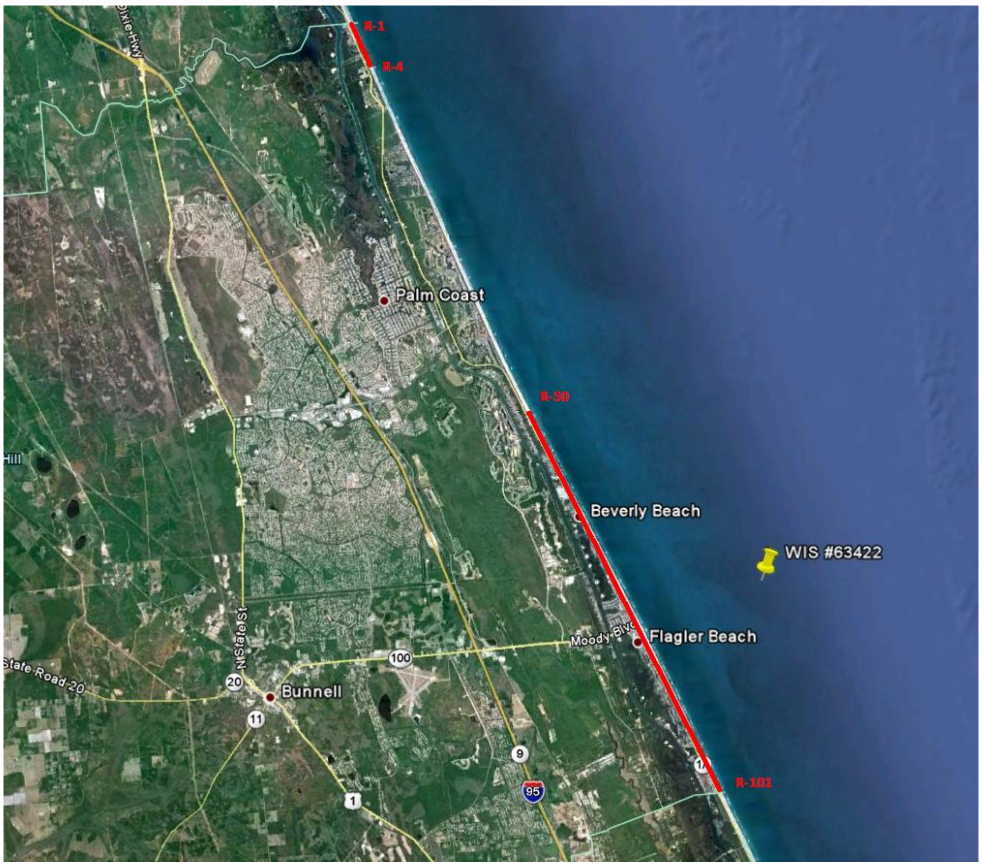

Figure A- 2. Location of WIS Station #63422 Relative to Project

Table A- 3 summarizes the percentage of occurrence and average wave height of the WIS waves by direction. It can be seen that the dominant wave directions range from northeast to southeast. This 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 provided in Table A- 4 shows that late 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 sediment transport and seasonal erosional patterns in the project area. In contrast, summer months experience milder conditions, with smaller wave heights. Overall, waves originating from the east to northeast quadrant dominate.

Table A- 3. Average Wave Heights (1980 to 1999)

Wind Direction (from)	WIS Station #63422 (1980-1999)	
	Percentage Occurrence (%)	Average Wave Height (ft)
North	9	4.5
Northeast	24	4.5
East	51	3.3
Southeast	12	2.7
South	2	3.1
Southwest	1	2.9
West	0	3.0
Northwest	2	3.6

Table A- 4. Seasonal Wave Conditions

Month	WIS Station #63422 (1980-1999)	
	Average Wave Height (ft)	Predominant Direction (from)
January	4.09	E-NE
February	4.07	E-NE
March	3.83	E-NE
April	3.33	E-NE
May	3.04	E-NE
June	2.61	E
July	2.24	E-SE
August	2.79	E
September	3.81	E-NE
October	4.58	E-NE
November	4.53	E-NE
December	4.15	E-NE

Wave periods have the same seasonality as wave heights. Table A- 5 provides a seasonal breakdown of percent occurrence by wave period. From this table, it can be seen that short period, locally-generated wind waves are common throughout the year. The yellow highlighted values show the dominant wave period for each month. None of these dominant periods are less than 5.0 seconds or greater than 6.0 seconds. It can also be seen that in the summer months the shortest period waves occur more frequently. During the fall and winter months more frequent higher-energy, longer-period storm swells occur. Note that the percentage of waves with period greater than 12.0 seconds increases from a low of 0.3% in June to a high of 13.4% in September (the height of hurricane season).

Table A- 5. Wave Period – Percent Occurrence

Wave Period (Sec)	Percent Occurrence by Wave Period Band											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
< 4.0	6.0	5.2	6.2	6.7	5.0	8.3	10.0	5.3	3.1	1.3	3.2	4.4
4.0 - 4.9	10.1	10.8	10.5	9.6	8.0	4.2	3.8	4.4	6.2	4.0	7.0	8.3
5.0 - 5.9	18.3	22.1	20.0	22.0	25.2	36.2	37.3	40.3	26.9	24.9	20.5	21.5
6.0 - 6.9	12.5	13.5	14.3	15.0	19.2	24.7	28.7	25.3	17.9	21.6	17.3	15.5
7.0 - 7.9	13.5	14.4	14.5	13.6	19.1	14.8	13.4	12.2	12.0	16.3	14.9	11.9
8.0 - 8.9	7.9	6.8	6.5	7.3	5.4	3.9	2.4	1.8	4.9	5.8	8.5	5.6
9.0 - 9.9	8.4	8.0	6.6	7.0	5.1	3.3	1.8	1.3	4.6	6.8	7.0	5.7
10.0 - 10.9	8.0	6.3	6.2	5.9	4.4	3.0	1.3	1.6	5.0	6.1	6.8	8.1
11.0 - 11.9	6.2	5.8	6.1	6.3	5.0	1.4	1.0	2.0	6.0	6.2	6.9	7.1
> 12.0	9.1	7.4	9.3	6.7	3.6	0.3	0.4	5.8	13.4	7.1	7.9	11.9

Tides and Currents

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

Tides in the Flagler County area are semidiurnal: two high tides and two low tides per tidal day (24 hours 50 minutes). Two measures of tidal range are commonly used: the mean tide range is defined as the difference between Mean High Water (MHW) and Mean Low Water (MLW), and represents an average range during the entire lunar cycle (27.3 days); and, the spring tide range is the average semidiurnal range which occurs semimonthly when the moon is new or full. The semidiurnal tides around Flagler Beach exhibit a mean tidal range of 3.64ft..

Presently, the nearest tide station to the project on the ocean side of the island is NOS Station 8720692 (State Road A1A Bridge), located at Matanzas Inlet approximately 17 miles north of Flagler Beach. The nearest tide station on the intracoastal side of the island is NOS Station 8720833 (Smith Creek, Flagler Beach), located directly west of Flagler Beach. Table A- 6 summarizes tidal data from both stations.

Table A- 6. Tidal Datums

Tidal Datum	Elevation Relative to MLLW	
	State Road A1A	Smith Creek
Mean High Water (MHW)	3.80	0.94
North Americal Vertical Datum (NAVD88)	2.28	0.78
Mean Tide Level (MSL)	1.95	0.52
Mean Low Water (MLW)	0.16	0.07
Mean Lower Low Water (MLLW)	0.00	0.00

The primary ocean current in the project area is the Florida Gulf Stream. With the exception of intermittent local reversals, it flows northward. The average annual current velocity is approximately 28 miles per day, varying from an average monthly low of 17 miles per day in November to an average monthly high of approximately 37 miles per day in July. The Gulf Streams lies approximately 60 miles offshore of the project area.

The near-shore currents in the project vicinity are not directly influenced by the Gulf Stream, but may be influenced indirectly via interaction with incident waves. Littoral currents affect the supply and distribution of sediment on the sandy beaches of Flagler County. Longshore 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 Flagler 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.

Influence of Matanzas Inlet (2.4 miles to the north) and Ponce de Leon Inlet (27 miles to the south) ebb and flood currents on local currents is negligible. In both cases the distance between the inlet and the project area places the project outside the influence of inlet tidal fluctuations.

Storm Effects

The shoreline of Flagler County is 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 often have a greater impact on the shoreline because of longer duration and greater frequency.

During intense storm activity, the shoreline is expected to naturally modify its beach profile**.** Storms erode and transport sediment from the 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. Periodic and unpredictable hurricanes and coastal storms, with their fierce breaking waves and elevated water levels, can change the width and elevation of beaches and accelerate erosion. After storms pass, gentle waves usually return sediment from the sand bars to the beach, which is restored gradually to its natural shape. 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 or far offshore into deep water where waves cannot return it to the beach. This may cause a permanent increase in the rate of shoreline recession.

Flagler County is located in an area of significant hurricane activity. Figure A- 3 shows historic tracks of hurricanes and tropical storms from 1858 to 2008, as recorded by the National Hurricane Center (NHC) and available from the National Oceanic and Atmospheric Administration (http://csc.noaa.gov/hurricanes/#). The shaded circle in the center of this figure indicates a 50-nautical mile radius (encompassing the entire Flagler county shoreline) from the center of the study area. Based on NHC records, 62 hurricanes and tropical storms have passed within this 50-mile radius over the 151-year period of record. Based on this chart, it can be seen that hurricanes and tropical storms pass within 50 nautical miles of the study area approximately every 2.4 years.

The 50-mile radius was chosen for display purposes in Figure A- 3 because any tropical disturbance passing within this distance, even a weak tropical storm, would be likely to produce some damage along the shoreline. Stronger storms are capable of producing significant damage to the coastline from far greater distances.

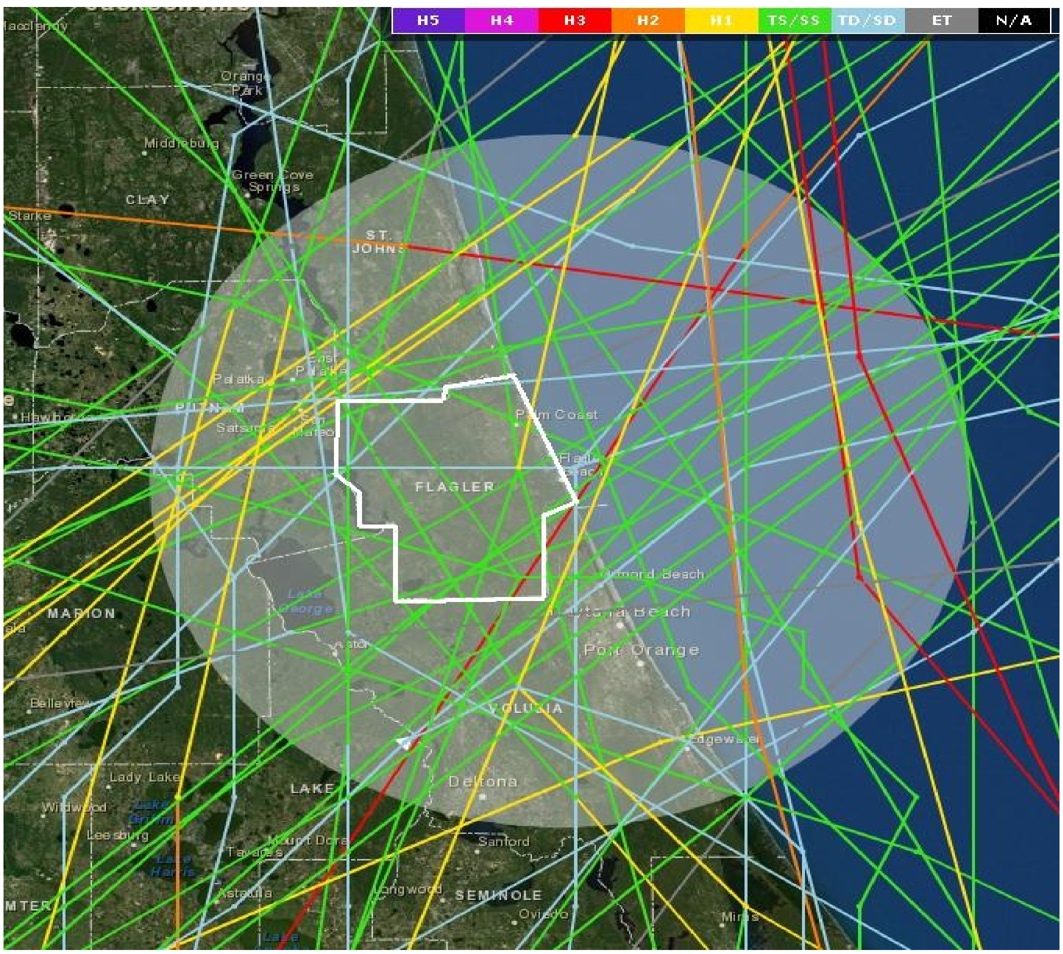

Figure A- 3. Historic storm tracks – Hurricanes and Tropical Storms (1858 – 2008, 50 mile radius)

In recent years, a number of named storms, passing within the 50 mile radius have significantly impacted the project area, including tropical storms Leslie (2000), Edouard (2002), Henri (2003), Charley (2004), Tammy (2005), and Fay (2008). Damages from these storms, as well as from more distant storms causing indirect impacts (Dennis, Floyd, and Irene in 1999; Gabrielle in 2001; Frances and Jeanne in 2004), included substantial erosion and damage from wind, wave, and water action.

Since the study area is exposed to the open ocean from northeast to southeast, the coastline is vulnerable to wave attack from distant storms as well. Most hurricanes and tropical storms traversing northward through the Atlantic passing within several hundred miles of the east coast are capable of producing large swells which are capable of causing erosion along the Flagler County shoreline.

Storm Surge

Storm surge is defined as the rise of the ocean surface above its astronomical tide level due to storm 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 super-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. An estimate of storm surge is required for the design of beach fill crest elevations. An increase in water depth may increase the potential for coastal flooding and allow larger storm waves to attack the shore.

The Flagler County SPP study area is a relatively low, flat barrier island and is susceptible to overtopping from extreme storm surges. Topographic surveys show that much of the island is less than 15 feet in elevation. Elevations of 15-20+ feet occur, but are almost exclusively along the oceanfront dune line. Flagler County Emergency Services (FlaglerEmergency.com) provides a hurricane storm-surge and evacuation map for public information (Figure A- 4). An examination of this map shows that virtually the entire study area would be inundated during even a Category 1 hurricane, should the storm make direct landfall in the Flagler County vicinity. In the event of a hurricane, only two evacuation routes from the barrier island exist: Palm Coast Parkway near the center of the county and the State Road 100 bridge about four miles north of the county line. The only continuous road extending along the length of the barrier island is Hwy A1A.

Storm surge levels versus frequency of occurrence were obtained from data compiled by the University of Florida for the Florida Department of Transportation (FDOT, 2003). Table A- 7 provides peak storm surge heights by return period for three locations in Flagler County: FDEP R-monuments R-0, R-55, and R-99. The storm surge elevations presented in this graph include the effects of astronomical high tide and wave setup.

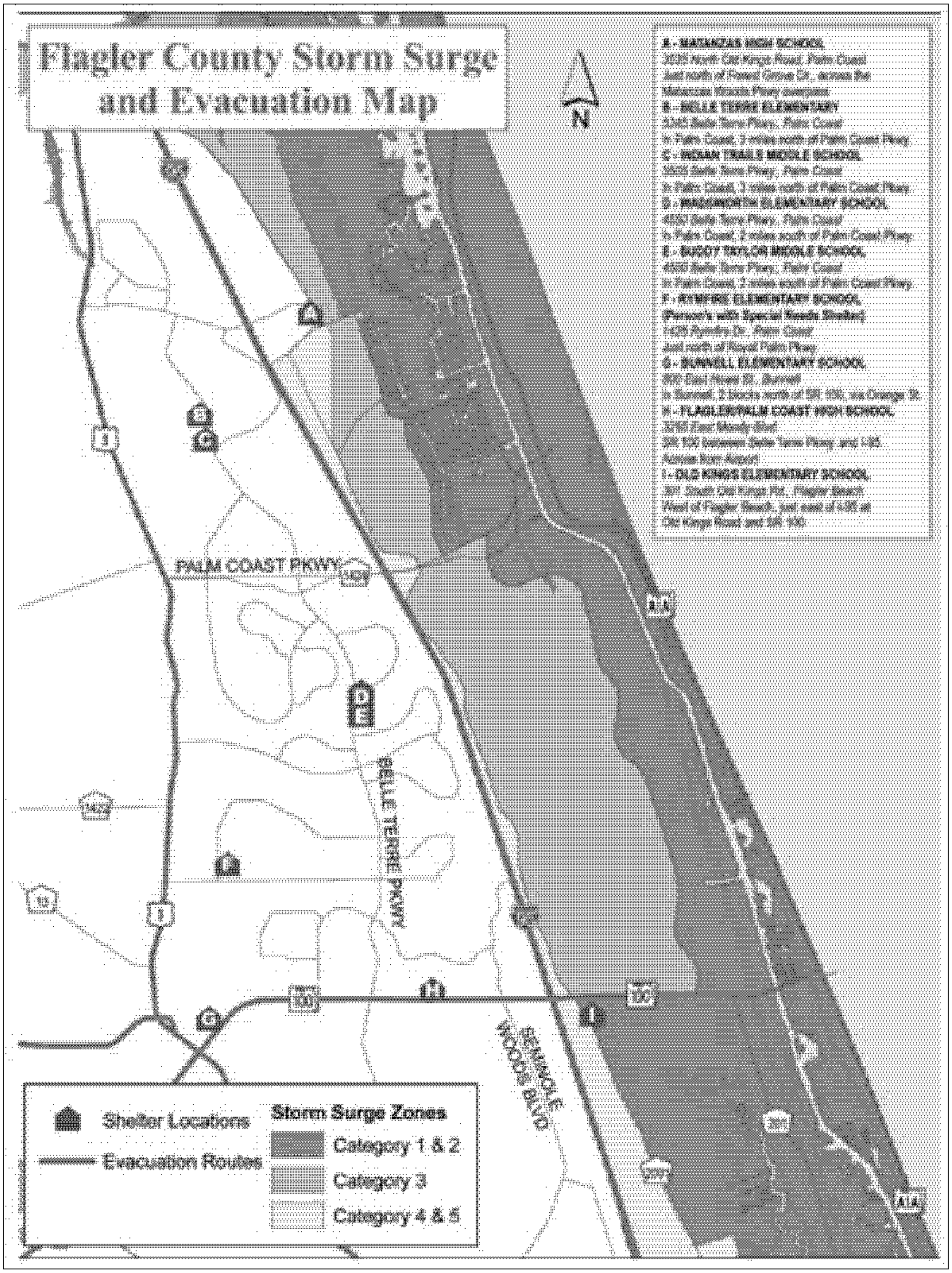

Figure A- 4. Storm Surge Zones, Flagler County, Florida.

Table A- 7. Storm Tide Elevations

Return Period (Years)	Total Storm Tide Level (Feet, NAVD88)		
	R-0	R-55	R-99
500	17.2	15.6	14.1
200	14.0	12.8	11.6
100	11.5	10.7	9.6
50	8.7	8.3	7.6
20	5.6	5.3	4.2
10	3.9	3.8	3.6

Sea Level Rise

Relative Sea Level Rise

Relative sea level (RSL) refers to local elevation of the sea with respect to land, including the lowering or rising of land through geologic processes such as subsidence and glacial rebound. It is anticipated that 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 U.S. Army Corps of Engineers (USACE) has provided guidance in the form of an Engineering Regulation, ER 1100-2-8162.

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 design life of 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 (NRC Curve I), and a High estimate (NRC Curve III) representing the maximum expected sea level change. All three scenarios are based on the following eustatic sea level rise (sea level change due to glacial melting and thermal expansion of sea water) equation:

$$E(t) = 0.0017t + bt^2$$

Where E(t) is the eustatic sea level rise (in meters); t represents years, starting in 1992 (the midpoint of the current National Tidal Datum Epoch of 1983-2001), and b is a constant equal to 2.71E-5 (NRC Curve I), 7.00E-5 (NRC Curve II), and 1.13E-4 (NRC Curve III). This equation assumes a global mean sea level change rate of +1.7mm/year.

In order to estimate the eustatic sea level change over the life of the project, the eustatic sea level rise equation is modified as follows:

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b({t_2}^2 - {t_1}^2)$$

Where t_1 is the time between the project's construction date and 1992 and t_2 is the time between the end of the project life and 1992. In order to estimate the required Baseline, Intermediate, and High

Relative Sea Level (RSL) changes over the life of the project, the eustatic sea level rise equation is further modified to include site specific sea level change as follows:

$$RSL(t_2) - RSL(t_1) = (e+M)\ (t_2 - t_1) + b(t_2^2 - t_1^2)$$

Where $RSL(t_1)$ and $RSL(t_2)$ are the total RSL at times t_1 and t_2, and the quantity $(e + M)$ is the local sea level rise in mm/year. Local sea level rise accounts for the eustatic change (0.0017mm/year) as well as uplift or subsidence and is generally available from the nearest tide gage with a tidal record of at least 40 years. The constant b is equal to 0.0 (Baseline), 2.71E-5 (Intermediate), and 1.13E-4 (High).

The Flagler project area is located approximately 60 miles from NOS gage #8720218 at Mayport, Florida. The historical sea level rise rate taken from this gage was determined to be 2.4 mm/year (0.0079 ft/year) (http://tidesandcurrents.noaa.gov/sltrends/index.shtml). In order to provide a more accurate estimate of local vertical land motion, the historical sea level rise rate is adjusted to account forregional trends. The local, adjustedsea level rise (*e*+M) at this location becomes 2.29 mm/yr (0.0075 ft/yr). Adjusted local sea level rise rates for NOS gages can be found at the USACE climate website: http://www.corpsclimate.us/ccaceslcurves.cfm. Given a project base year of 2016 and a project life of 50 years, a table of sea level change rates was produced for each of the three required scenarios. Table A- 8 shows the sea level change rates in five year increments, starting from the base year of 2014. Figure A- 5 provides a graphic representation of the three levels of projected future sea level change for the life of the project.

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. Therefore in Flagler County, there is 0.59 mm/year of subsidence.

Table A- 8. Relative Sea Level vs Year - Flagler County

	Baseline (Historic)			Intermediate (NRC Curve I)			High (NRC Curve III)		
	Year	mm	ft	Year	mm	ft	Year	mm	ft
Base Year	2016	9.16	0.03	2016	13.93	0.05	2016	29.05	0.10
	2021	20.61	0.07	2021	32.56	0.11	2021	70.44	0.23
	2026	32.06	0.11	2026	52.55	0.17	2026	117.49	0.39
	2031	43.51	0.14	2031	73.89	0.24	2031	170.18	0.56
	2036	54.96	0.18	2036	96.59	0.32	2036	228.53	0.75
25 Year	2041	66.41	0.22	2041	120.64	0.40	2041	292.52	0.96
	2046	77.86	0.26	2046	146.04	0.48	2046	362.17	1.19
	2051	89.31	0.29	2051	172.81	0.57	2051	437.46	1.44
	2056	100.76	0.33	2056	200.92	0.66	2056	518.41	1.70
	2061	112.21	0.37	2061	230.39	0.76	2061	605.00	1.98
50 Year	2066	123.66	0.41	2066	261.22	0.86	2066	697.25	2.29

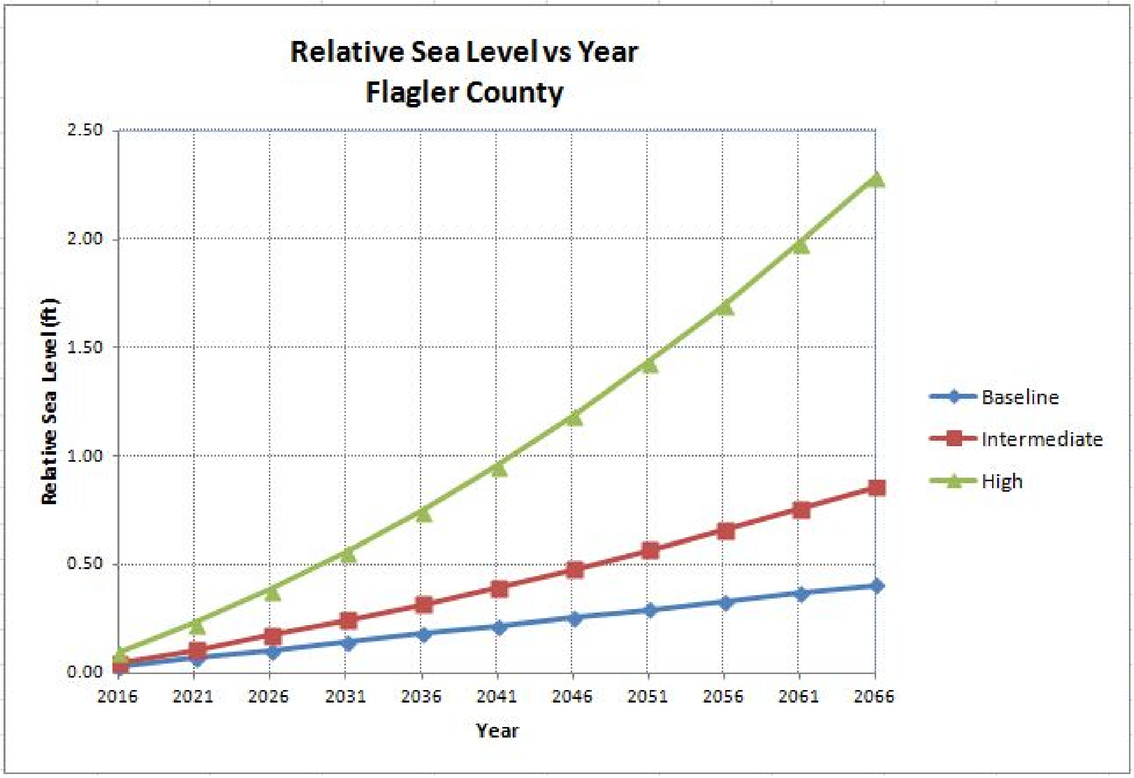

Figure A- 5. Relative Sea Level Rise, Flagler County

Beach Responses to Sea Level Change

This section evaluates how the above 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 Per 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

Per 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 longshore 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. Shoreline recession, X, resulting from sea level change can be estimated using Bruun's Rule, defined as:

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

Where S is the rate of sea level change; B is the berm height (approximately +10 feet NAVD88); h_* is depth of closure (the depth beyond which there is no significant change over time in the shoreline profile; estimated to be approximately -20 feet NAVD88); and W_* is the width of the active profile (approximately 1,800 feet). Figure A- 6 provides the resulting shoreline recession 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. 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.

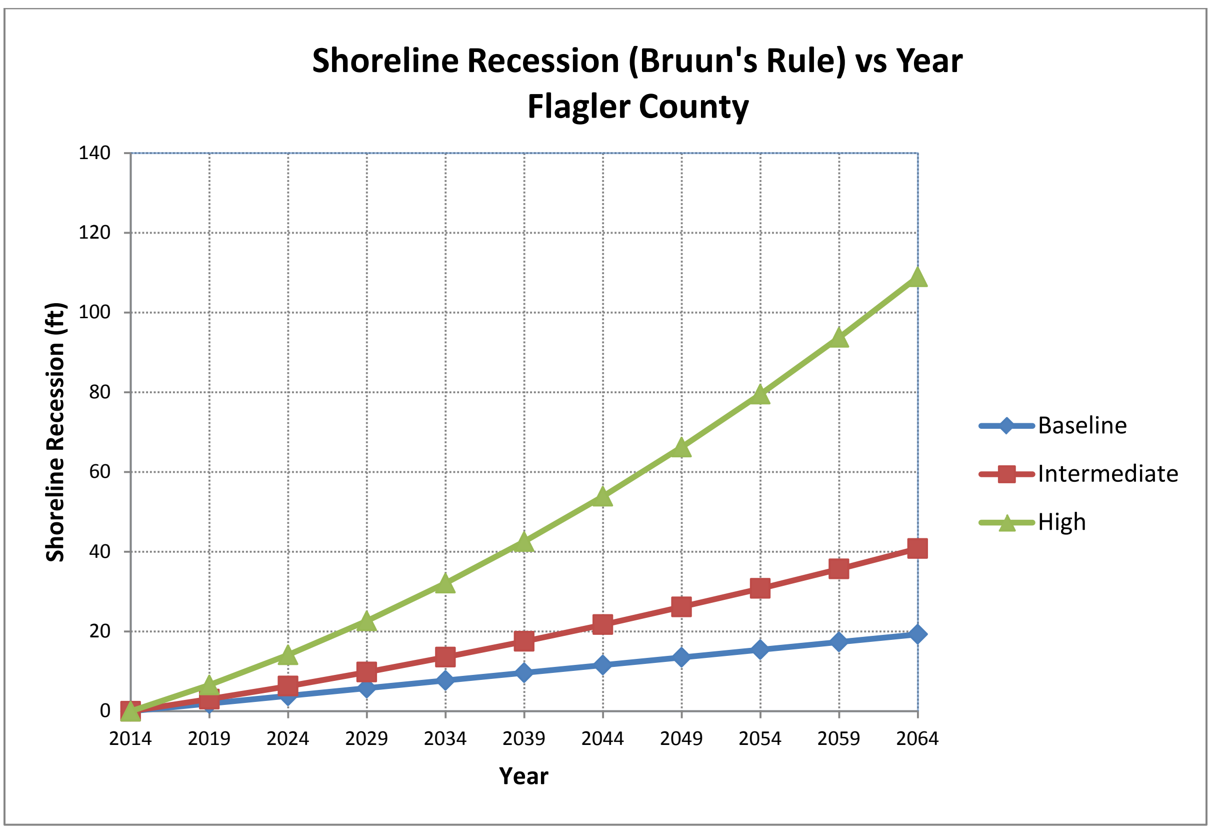

Figure A- 6. Shoreline Recession vs Year

Volumetric Change

Engineering Manual (EM) 1110-2-3301 (USACE, 1995) gives 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:

$$V = (B + h_*)X$$

Where B is the berm height, h_* is the depth of closure, and X is the horizontal translation of the profile. Figure A- 7 provides the resulting volume lost versus year for each of the three sea level rise scenarios.

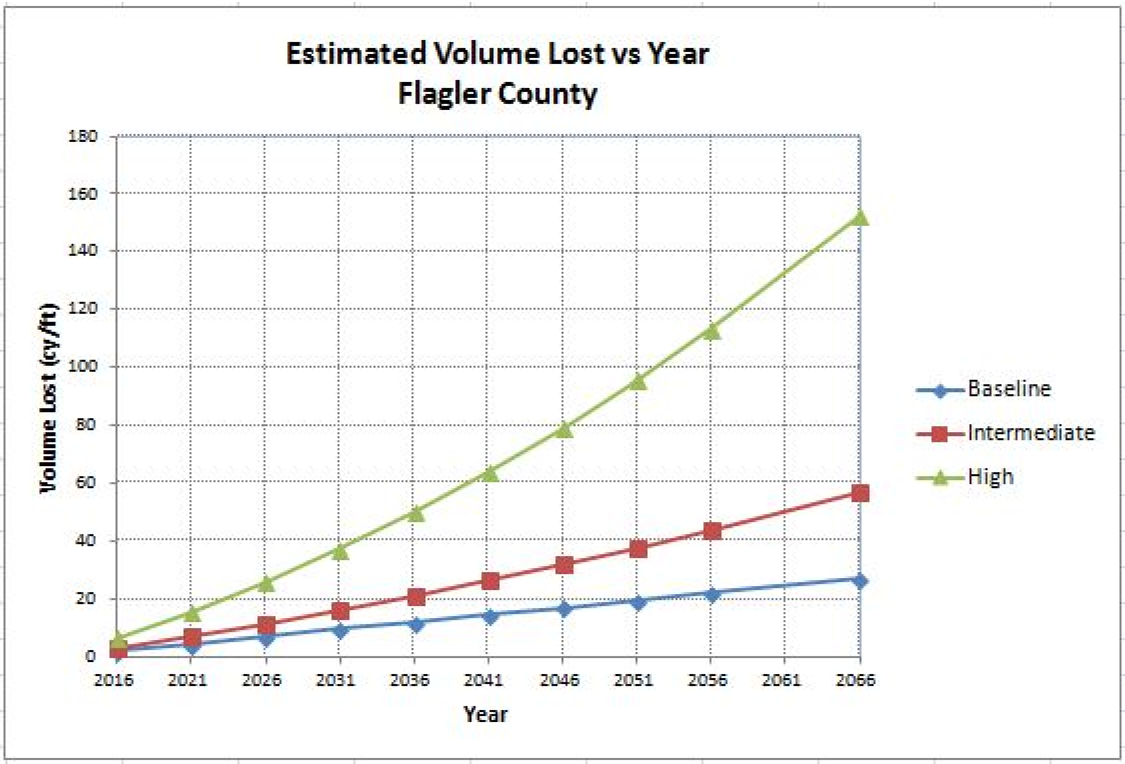

Figure A- 7. Estimated Volume Lost Versus Year

Historical Shoreline Change

Flagler County is unique compared to the counties to the north and south, in that the shoreline sediment contains a higher percentage of coarse shell hash which produces a larger median grain size and steeper beach profiles. The shoreline has mild concave curvature from north to south, transitioning to a headland at Flagler Beach. Shoreline irregularities along the generally curved shoreline are attributed to nearshore hard bottom exposed rock outcrops which influence shoreline erosion and accretion. A Florida Department of Environmental Protection shoreline change rate study conducted in

July of 1999 (FDEP, 1999) concluded that the beaches of Flagler County are subject to cyclic erosion and accretion but are relatively stable based on data from 1952 to 1993.

Changes in mean high water (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 Flagler County go back as far as 1872. However, the reliability of such historical profiles may be questionable. Therefore, based on a review of all available surveys, it was determined that profiles dating from 1972 to present would be used for the MHW analysis.

MHW shoreline positions were measured at each DNR survey monument location, for each survey, along the proper azimuth (70 degrees, measured from north, clockwise). Resulting differences in MHW position in both the north (R-1 to R-4) and south (R-50 to R-100) project segments are tabulated in Table A- 9. This table also provides the overall MHW rate of change for the period between 1972 and 2007.

In order to better interpret the shoreline change, the MHW position data was put into a graphical format (Figure A- 8). Note that lines only connect data between adjacent R-monument locations. Gaps indicate where R-monument measurements are missing. As seen in the figures, shoreline changes fluctuate over time along the study areas. Figure A- 9 provides a summary of all measured shoreline changes (1972 to 2007). This figure shows the cumulative changes bases on the data presented in

Table A- 9. While areas of accretion exist throughout the project over time, the overall trend of the shoreline is erosional.

The position of the MHW line varies along Flagler County project shoreline, with relatively small rates of change over the time period between 1972 and 2007 (Figure A- 10). Shoreline change rates for this period range from +1.06 to -2.40 feet per year with isolated areas of moderate erosion and accretion. Factors which contribute to this variation include the distribution of exposed rock in the surf zone and foreshore slope, as well as structures in the area. One structure of particular influence on longshore transport and beach erosion and accretion is the Flagler Pier at R-79. The pier tends to trap sand from longshore transport causing accretion north of the pier, as well as downdrift erosion about 2,000 feet south of the pier due to the interruption of longshore transported sand. From 1972 to 2007 the MHW rate of change was generally erosional along the study limits with annual erosion rates of -0.58 feet per year in the north project segment (R1 to R-4) and -0.59 feet per year in the south project segment (R-50 to R-100). Table A- 10 provides a further breakdown of annual shoreline rates of change rates by study reach.

Each of the study reaches, with the exception of Beverly Beach, have relatively consistent average shoreline rates of change, ranging from -0.58 ft/yr to -0.67 ft/yr. Due primarily to the stabilizing presence of a concrete and steel seawall over a significant portion of the reach, Beverly Beach experiences a lower shoreline rate of change, approximately -0.11 ft/yr.

Table A- 9. Mean High Water Shoreline Position Change

DNR Monument	MHW Shoreline Position Change in Feet 1972-1980	1980-1984	1984-1986	1986-1993	1993-2000	2000-2003	2003-2007	1972-2007	MHW Rate of Change (1972-2007) (feet/year)
R-1	-68			14	0	78	-85	-58	-1.66
R-2	-44					73	-87	-54	-1.54
R-3	13	56	-59	13	-53	105	-39	36	1.03
R-4	-53					22	-30	-5	-0.14
Average	-38	56	-59	14	-27	70	-60	-20	-0.58
R-50	-49					16	-13	-4	-0.11
R-51	-73			43	-1	-7	-1	-38	-1.09
R-52	-68	43	-6			1	-26	-44	-1.24
R-53	-63					12	-16	-49	-1.40
R-54	-17	18	-5	17	-14	0	4	3	0.09
R-55	-42					-8	22	3	0.09
R-56	-79					-1	-41	-84	-2.40
R-57	-34	36	12	-9	5	-13	-2	-5	-0.14
R-58	-43					-6	-1	-14	-0.40
R-59	-39					-7	13	20	0.57
R-60	-69					0	-15	-36	-1.03
R-61	-47	71	4	1	-20	3	2	14	0.40
R-62	-26					-9	29	37	1.06
R-63	-59	52	21	-28	17	-11	-20	-28	-0.80
R-64	-65					-5	24	1	0.03
R-65	-48					-1	-3	-11	-0.31
R-66	-31	34	-1	1	-8	-1	7	1	0.03
R-67	-28					-5	26	-8	-0.21
R-68	-24					22	-51	-16	-0.46
R-69	-8	10	21			-10	6	31	0.89
R-70	-7			-13	7	-10	-8	-18	-0.51
R-71	-21					-18	13	-2	-0.06
R-72	-58			-3	-7	-5	-38	-43	-1.23
R-73	-14	40	16			-14	-7	5	0.14
R-74	-16					-15	19	28	0.80
R-75	-21	55	-7	0	-11	11	-48	-21	-0.60
R-76	-24					48	-29	11	0.31
R-77	-30					15	-1	19	0.54
R-78	-13	34	16	-6	-35	10	-28	-22	-0.63
R 79	69					39	51	27	0.77
R-80	-55					44	-53	-41	-1.17
R-81	-71	52	-26	40	-41	33	-26	-39	-1.11
R-82	-51					40	-28	-20	-0.57
R-83	-23					7	-6	-2	-0.06
R-84	-29	30	3	-14	7	-6	35	26	0.74
R-85	-54					6	-28	-54	-1.54
R-86	-53					18	-51	-73	-2.09
R-87	-45	64	2	-11	-23	19	-71	-65	-1.86
R-88	-29					18	-96	-51	-1.46
R-89	-31					39	-97	-31	-0.89
R-90	-45	61	-39	21	25	40	-111	-48	-1.37
R-91	-69					15	-54	-78	-2.23
R-92	-63					13	-4	-35	-1.00
R-93	-65	54	-8	-9	-22	20	-39	-69	-1.97
R-94	-59					38	-24	-17	-0.49
R-95	-74					46	-48	-39	-1.11
R-96	-60	83	-42	34	-30	30	-30	-15	-0.43
R-97	-58					23	-59	-55	-1.57
R-98	-39					40	-29	-3	-0.09
R-99	-50	55	-28	54	-39	6	-22	-24	-0.69
R-100	-37					23	-24	-5	-0.14
Average	-45	47	-4	7	-11	11	-22	-21	-0.59

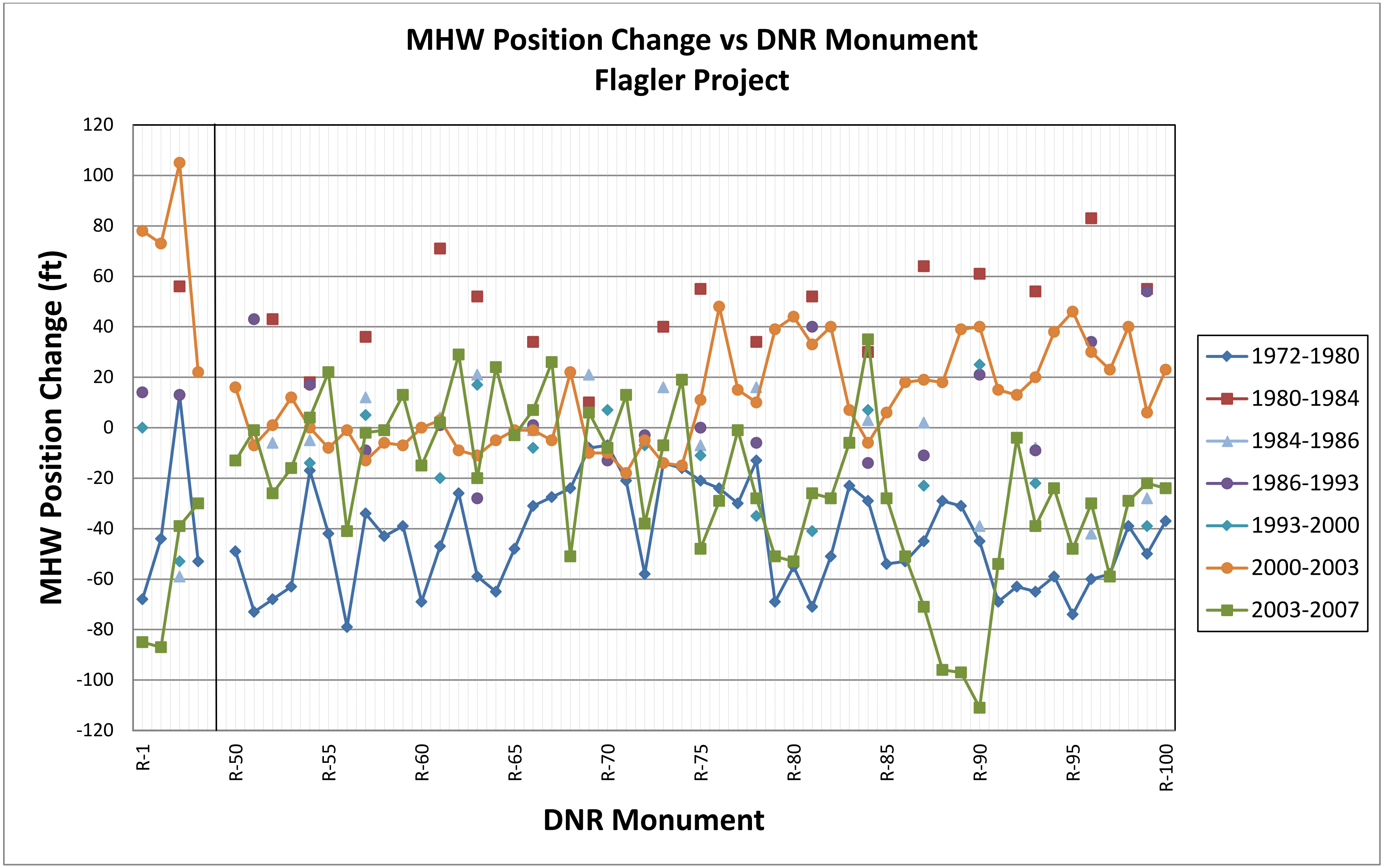

Figure A- 8. MHW Changes, Flagler County Study Area

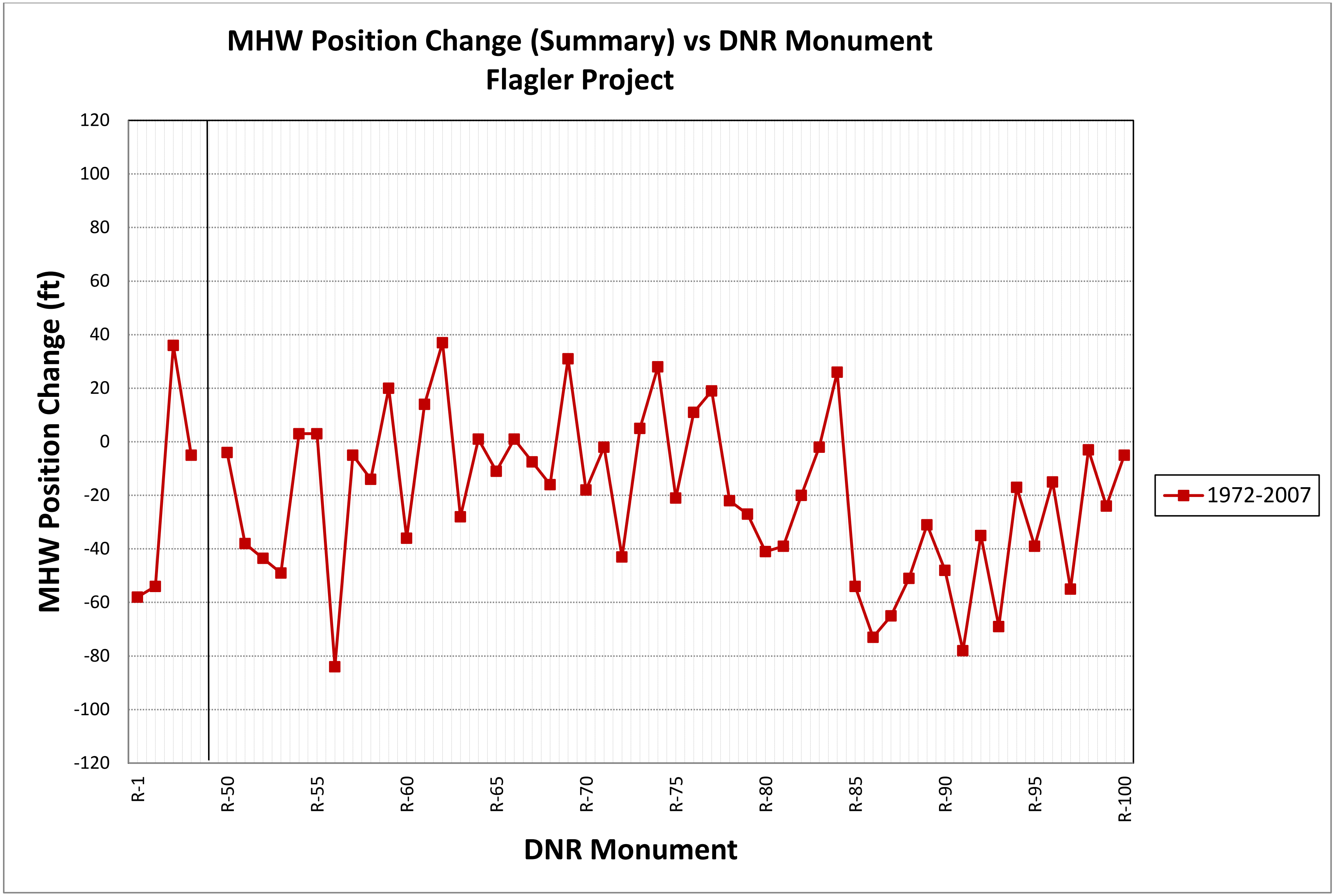

Figure A- 9. Summary of MHW Changes

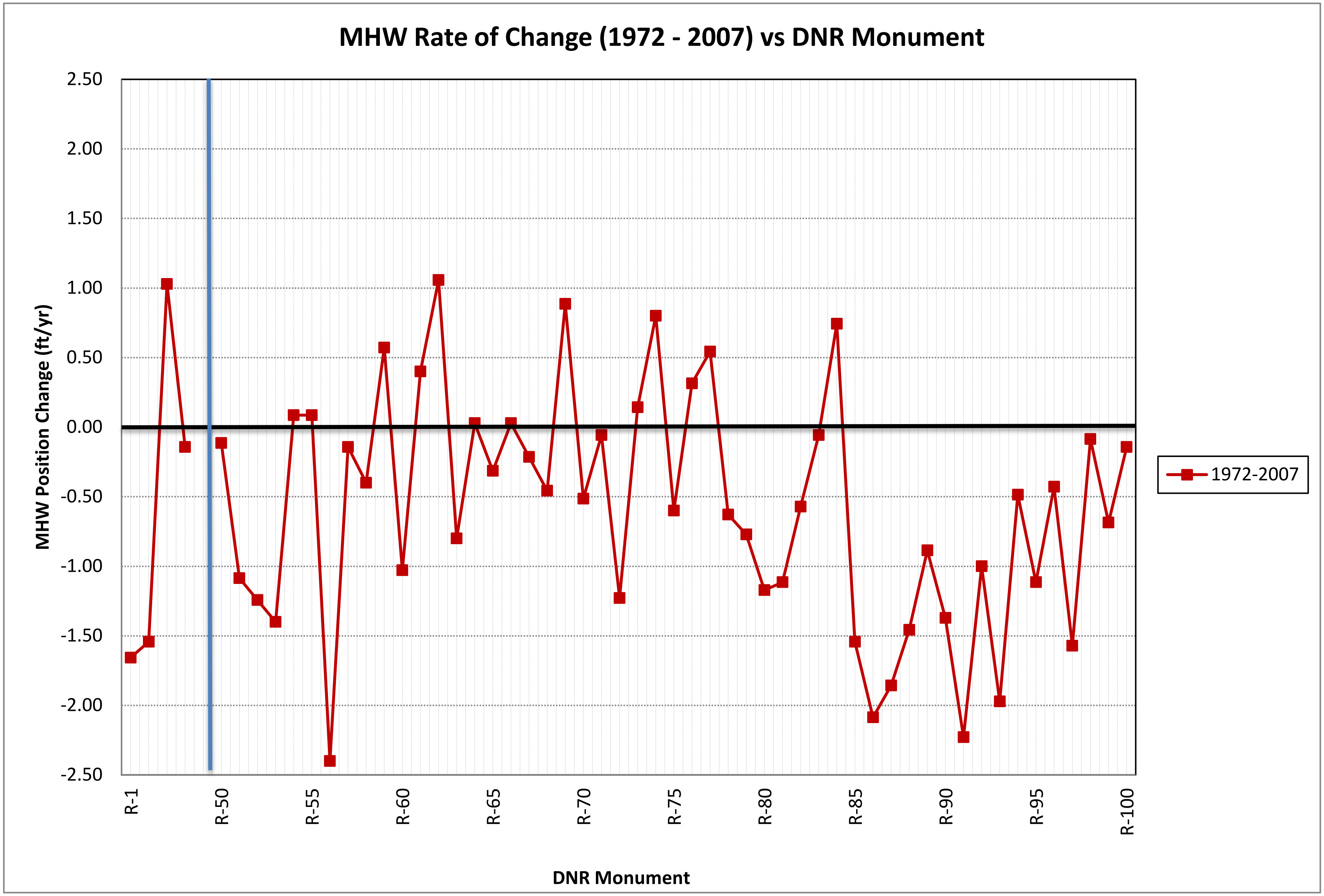

Figure A- 10. Shoreline Rate of Change (North and South Segments)

Table A- 10. Annual Shoreline Rate of Change by Study Reach

Project Segment	Study Reach	Location (DNR Monument)	MHW Rate of Change (1972 – 2007) (feet/year)
North	Marineland	R-1 to R-4	-0.58
Total (North)		R-1 to R-4	-0.58
South	Painters Hill	R-50 to R-60	-0.64
	Beverly Beach	R-60 to R-67	-0.11
	Flagler Beach	R-67 to R-101	-0.67
Total (South)		R-50 to R-101	-0.59
Total (Project)		R-1 to R-4, R-50 to R-101	-0.59

Existing Shoreline Armor

Historically, the threat that shoreline erosion has posed to both private and public infrastructure has resulted in coastal armoring throughout Flagler County. USACE shoreline surveys taken in February 2009 revealed that most of the existing shoreline armor could be found within the limits of the proposed project. Table A- 11 provides a summary of shoreline armor throughout Flagler County with armor outside of the study limits indicated. Further details of Flagler County coastal armoring can be found in the main report.

Table A- 11. Summary of Shoreline Armoring in Flagler County

Study Reach	R-Monument	Length (feet)	Description
Marineland	R-1 to R-2	1,350	Granite revetment at Marineland Aquatic Park
	R-1 to R-3	----	Five partially removed coquina groins
	R-2 to R-3	1,500	Steel seawall currently covered by dune and boardwalk
Varn Park*	R-49.3 to R-49.5	260	10’ tall stand-alone seawall with no structures behind the wall
Beverly Beach	R-60.5 to R-62.4	1,560	Concrete steel seawall fronting Camptown RV park
Flagler Beach	R-78.6 to R-79.4	565	Small section of aging concrete seawall
	R-80 to R-90	9,240	Coquina and granite revetment with regions of damage
	R-82	153	Concrete capped steel sheetpile seawall
	R-94.6 to R-94.8	152	Small segment of concrete seawall and small segment of wooden seawall
* Varn Park armor does not fall within the limits of the study area			

Effects of Adjacent Features

Shore Protection/Navigation Projects

There are no navigation projects in the vicinity of Flagler County that will affect the study area. Material dredged from the Intracoastal Waterway (IWW) near the Matanzas inlet has been placed on Summer Haven beach in the past. Although it is possible that sand from these activities migrates south to the Marineland reach, a review of the shoreline change data indicates that effects of this migration are negligible.

Inlet Effects

There are no inlets within Flagler County. The nearest inlets are Matanzas Inlet 2.4 miles to the north of Flagler County in St Johns County and Ponce de Leon Inlet 27 miles to the south of Flagler County in Volusia County. Matanzas Inlet is a relatively small inlet and is not maintained for navigation. The inlet has a history of migrating to the south, but is now stabilized with the south bridge abutment of the Highway A1A Bridge. Effects of Matanzas Inlet on the Flagler County shorelines to the south have not been quantified, but are expected to be negligible. Ponce de Leon inlet is distant enough and down drift of Flagler County and is therefore not expected to have an impact on the county's beaches.

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 lead 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

Beach-*fx* is an event-driven life-cycle model. USACE guidance (USACE, 2006) requires that flood damage reduction studies include risk and uncertainty. The Beach-*fx* model satisfies this requirement by fully incorporating risk and uncertainty throughout the modeling process (input, methodologies, and output). Over the project life-cycle, typically 50 years, the model estimates shoreline response to a series of historically based storm events. These plausible storms, the driving events, 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 life of the project. Risk based damages to structures are estimated based on the shoreline response in combination with pre-determined storm 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 over a range of possible values.

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- 11 shows a graphic depiction of the model setup. For further details about the specifics of Lot extents and DE grouping (see the Economics Appendix).

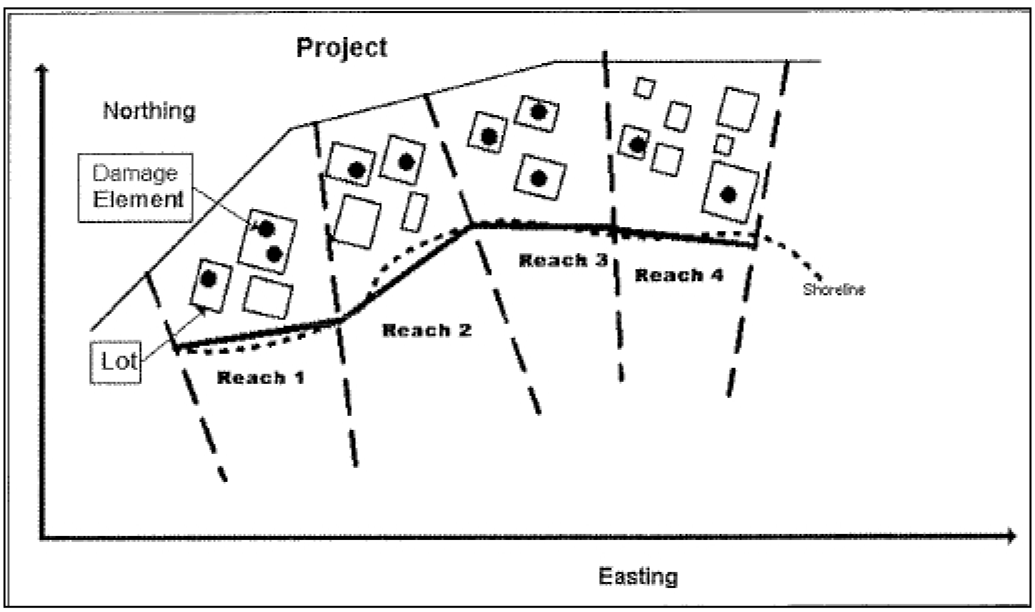

Figure A- 11. Beach-*fx* Model Setup Representation

Each model reach is associated with a representative beach profile that describes 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. Five design reaches were identified in the course of this study, Marineland, Reach A, Reach B, Reach C, and Reach D. Due to the presence of a robust revetment that precluded damages, the Marineland design reach was eliminated from further consideration early in the study (see main report for more detail). Combined, the four design reaches are composed of 50 model reaches, 315 lots, and 1,908 DEs. Table A- 12 provides the extent of the project and model reaches relative to FDEP R-monument locations.

Table A- 12. Extent of Design and Model Reaches

Design Reach	Model Reaches	R-monuments
Marineland[*]	R1-1 to R1-4	R-1 to R-4
Reach A (Painters Hill)	RA-1 to RA-10	R-50 to R-60
Reach B (Beverly Beach & North Flagler Beach)	RB-1 to RB-17	R-61 to R-79
Reach C (Central Flagler Beach)	RC-1 to RC-14	R-80 to R-94
Reach D (South Flagler Beach)	RD-1 to RD-5	R-95 to R-100
[*]The Marineland design reach was eliminated from consideration following the Feasibility Scoping Meeting		

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.

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 Flagler County is made up of both types. Derived from the historical record of the region, the Flagler 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. The 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 Flagler County (Table A- 13).

Table A- 13. Flagler 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 Flagler 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, based on SBEACH model results, extra-tropical storms did not have a significant impact on profile change in the study area. Therefore, the 7 day interval was considered suitable for both storm types. Also, due to the probabilistic nature of the mondel 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 set that drives the Beach-*fx* model to simplified beach profiles that represent the shoreline features of the project site. For the Flagler study, application of the storm set to the idealized profiles was accomplished with 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 give project depends on the natural variability of shoreline itself. First historical profiles at each FDEP R-monument were compared over time. Using the USACE Regional Morphology Analysis Package (RMAP) (Morang et al., 2009) each profile was aligned and then averaged into a composite profile representative of the shoreline shape at that given R-monument location. Next, each of the composite profiles were 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 Flagler County six 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 representing the major dimensions of the profile were defined (Figure A- 12 through Figure A- 17). Table A- 14 provides dimensions for each of the idealized pre-storm Beach-*fx* profiles.

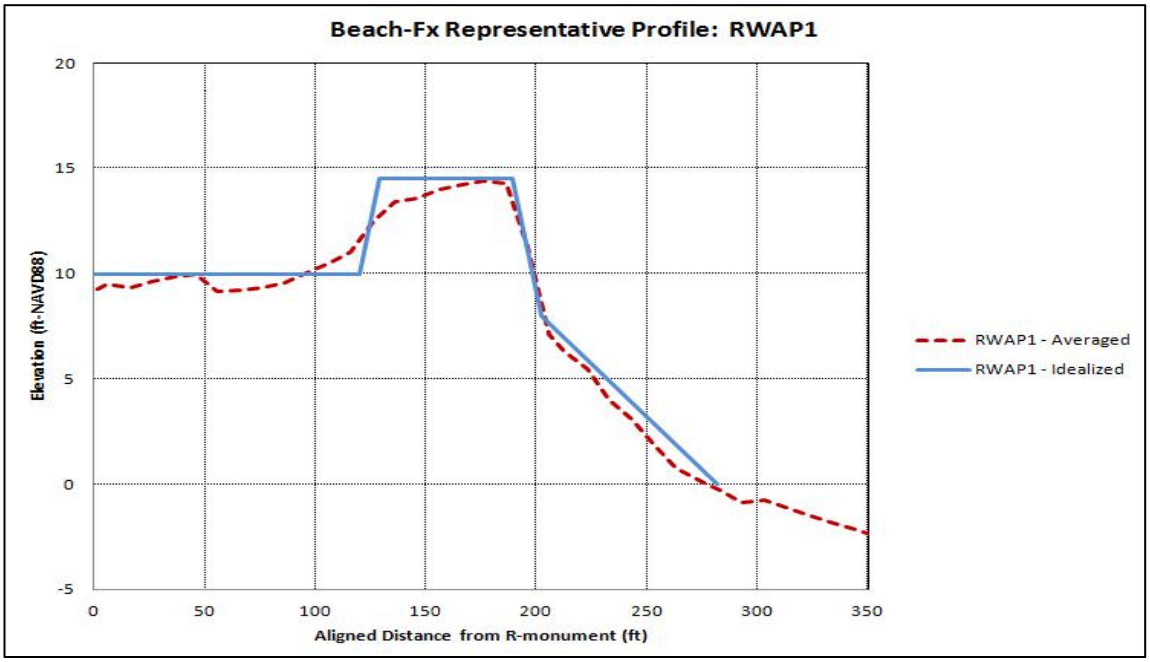

Figure A- 12. Averaged and Idealized Profiles: RWAP1 Grouping

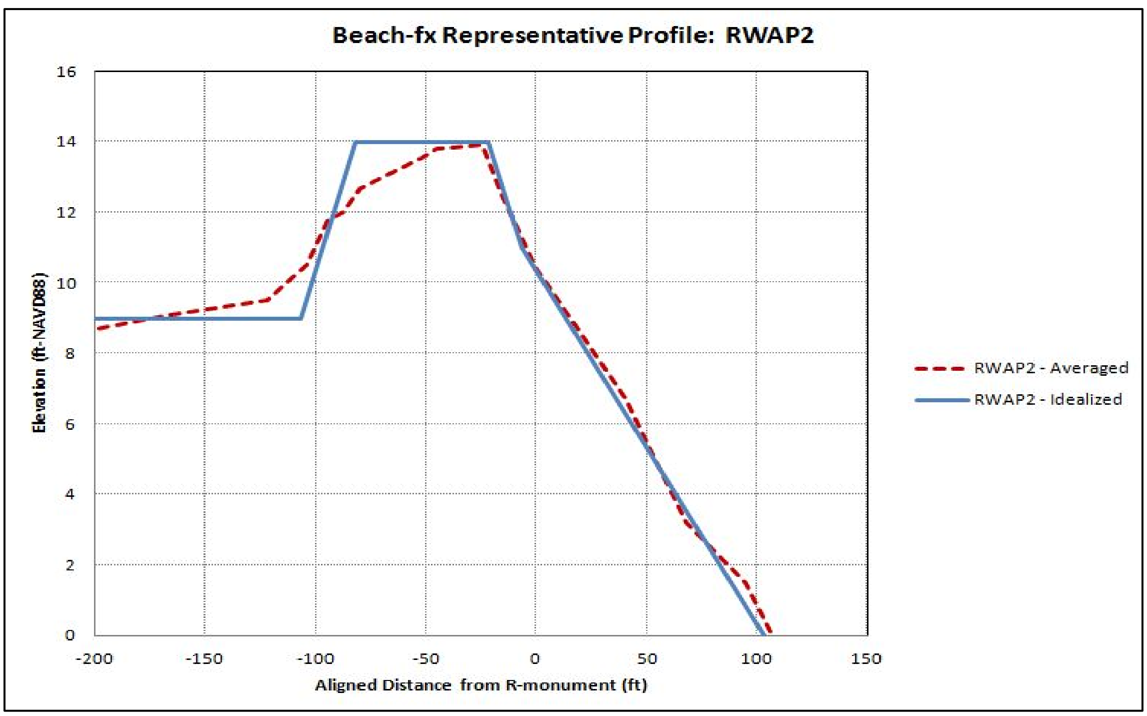

Figure A- 13. Averaged and Idealized Profiles: RWAP2 Grouping

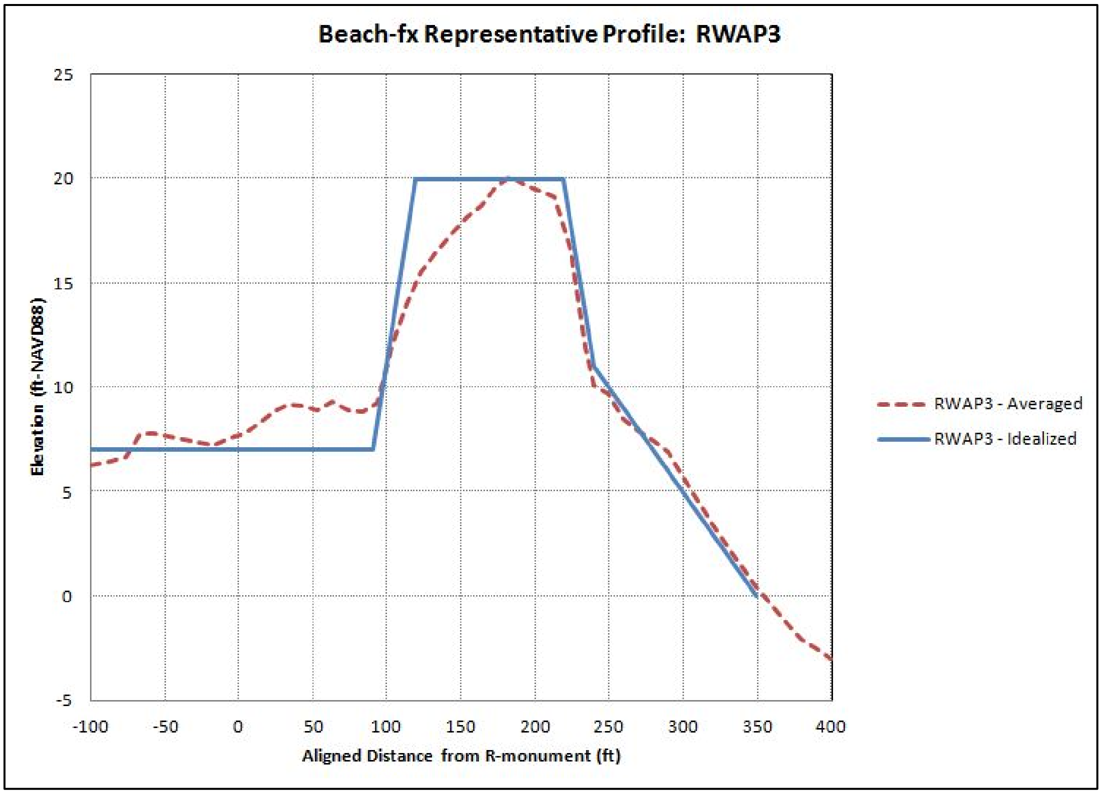

Figure A- 14. Averaged and Idealized Profiles: RWAP 3 Grouping

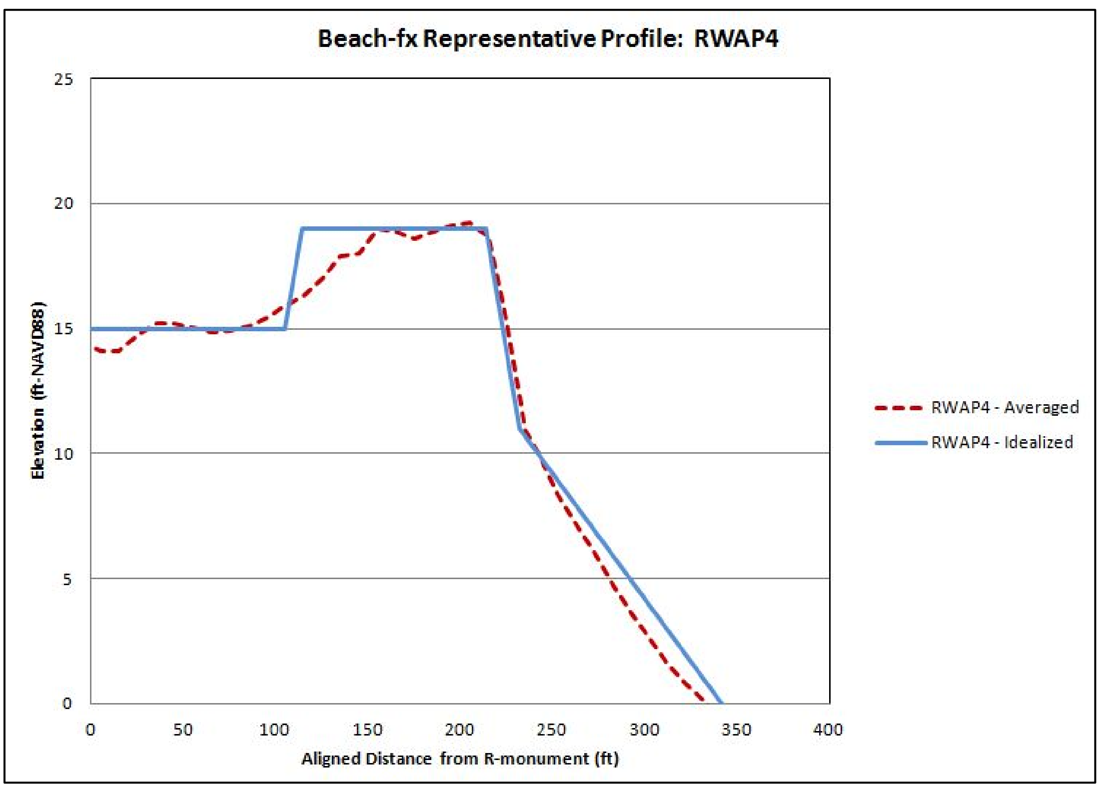

Figure A- 15. Averaged and Idealized Profiles: RWAP 4 Grouping

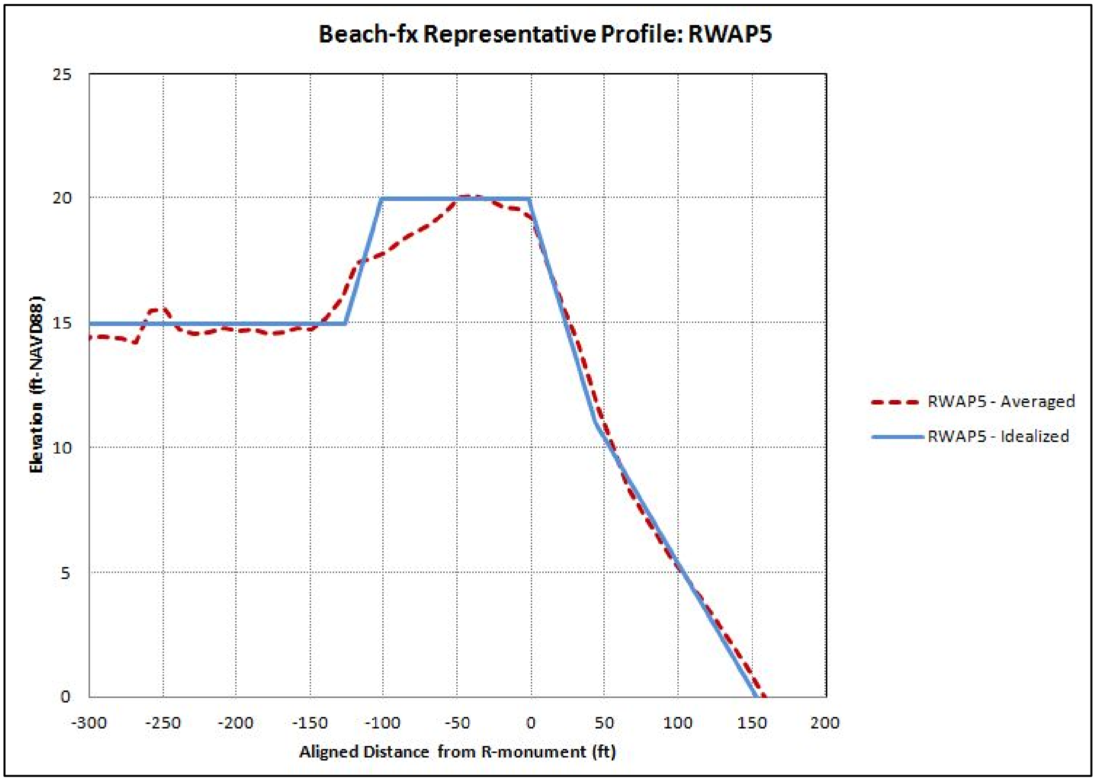

Figure A- 16. Averaged and Idealized Profiles: RWAP 5 Grouping

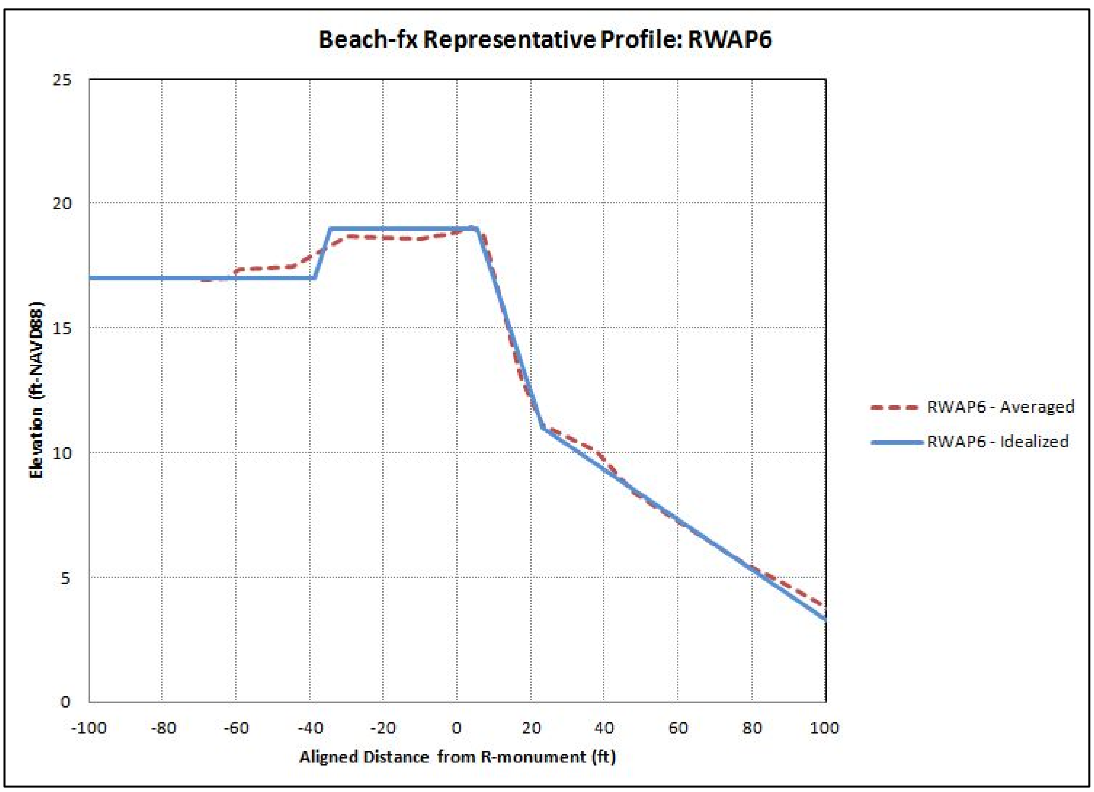

Figure A- 17. Averaged and Idealized Profiles: RWAP 6 Grouping

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, ft)	Berm Elevation (ft-NAVD88)	Berm Width (ft)	Foreshore Slope (V:H,ft)
RWAP1	R-1	10	14.5	60	1:2	8	0	1:10
RWAP2	R-3 to R-4	9	14	60	1:5	8	0	1:10
RWAP3	R-50 to R-54	7	19	100	1:2.2	11	0	1:10
RWAP4	R-55 to R-70, R-80 to R-88, R-95 to R-97	15	19	100	1:2.2	11	0	1:10
RWAP5	R-71 to R-79	15	20	100	1:5	11	0	1:10
RWAP6	R-89 to R-94, R-98 to R-100	17	19	40	1:2.2	11	0	1:10

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 Flagler County. Note that elevations and slopes do not change between existing and future conditions.

Table A- 15. Dimensions of Idealized Pre-Storm Representative Profiles (Future)

Profile (s)	Without Project Conditions		With Project Conditions					
	Dune Width (ft)	Berm Width (ft)	Dune Width (ft)	Berm Width (ft)				
RWAP1, RWAP2	45	0	60	0	20	40	60	80
	50	0	65	0	20	40	60	80
	55	0	70	0	20	40	60	80
	60	0	75	0	20	40	60	80
RWAP3, RWAP4, RWAP5	60	0	100	0	20	40	60	80
	65	0	105	0	20	40	60	80
	70	0	110	0	20	40	60	80
	75	0	115	0	20	40	60	80
	80	0						
	85	0						
	90	0						
	95	0						
	100	0						
RWAP6	20	0	40	0	20	40	60	80
	25	0	45	0	20	40	60	80
	30	0	50	0	20	40	60	80
	35	0	55	0	20	40	60	80
	40	0	40	0	20	40	60	80

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 longshore 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. Longshore 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)
- Relatively uniform sediment grain size throughout the profile,
- The shoreline is straight and longshore 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 refraction, randomization of input waves 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 using wave height, wave period, and water level information from Hurricane Dennis (August/September 1999) and Hurricane Floyd (September 1999) (Figure A- 18). 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.

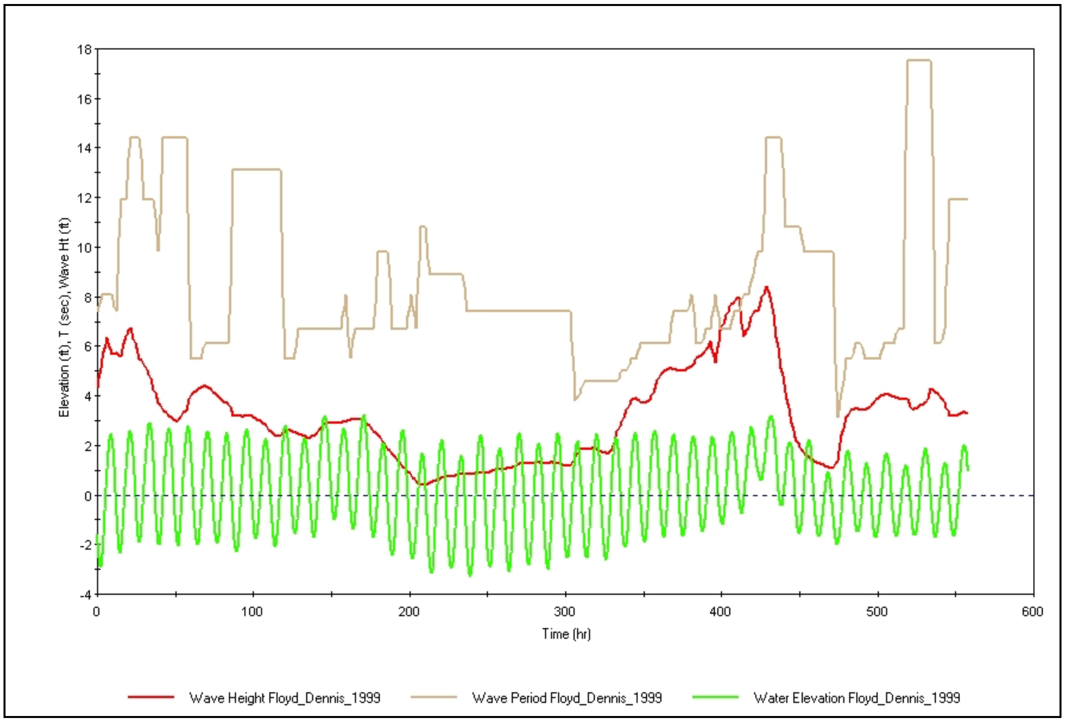

Figure A- 18. Hurricane Dennis and Floyd Wave and Water Level Data for SBEACH Calibration

Pre- and post-storm shoreline profiles were obtained from FDEP. 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.

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	0.5 ft	Transport Rate Coefficient	5e-007 (m4/N)
Effective Grain Size	0.44 mm	Overwash Transport Parameter	0.001
		Coefficient for Slope-Dependent Term	0.002
Maximum Slope Prior to Avalanching	45	Transport Rate Decay Coefficient Multiplier	0.1
		Water Temperature	20degC

SBEACH Simulations

Calibrated Flagler SBEACH simulations were run for each of the existing, future without project, and with project idealized profiles in combination with each of the tropical and extra-tropical storms in the plausible storm database. This resulted in 99,000 individual storm response profiles. From these profiles, changes in the key profile dimensions were extracted and stored in the Flagler Beach-*fx* SRD.

Applied Shoreline Change

The applied 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 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. Figure A- 19 shows the smoothed target shoreline change rates along with the original MHW shoreline change rates 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 hundreds of iterations over the 50-year project life cycle. Calibration is achieved when the rate of shoreline change, averaged over hundreds of life cycle simulations, is equal to the target shoreline change rate.

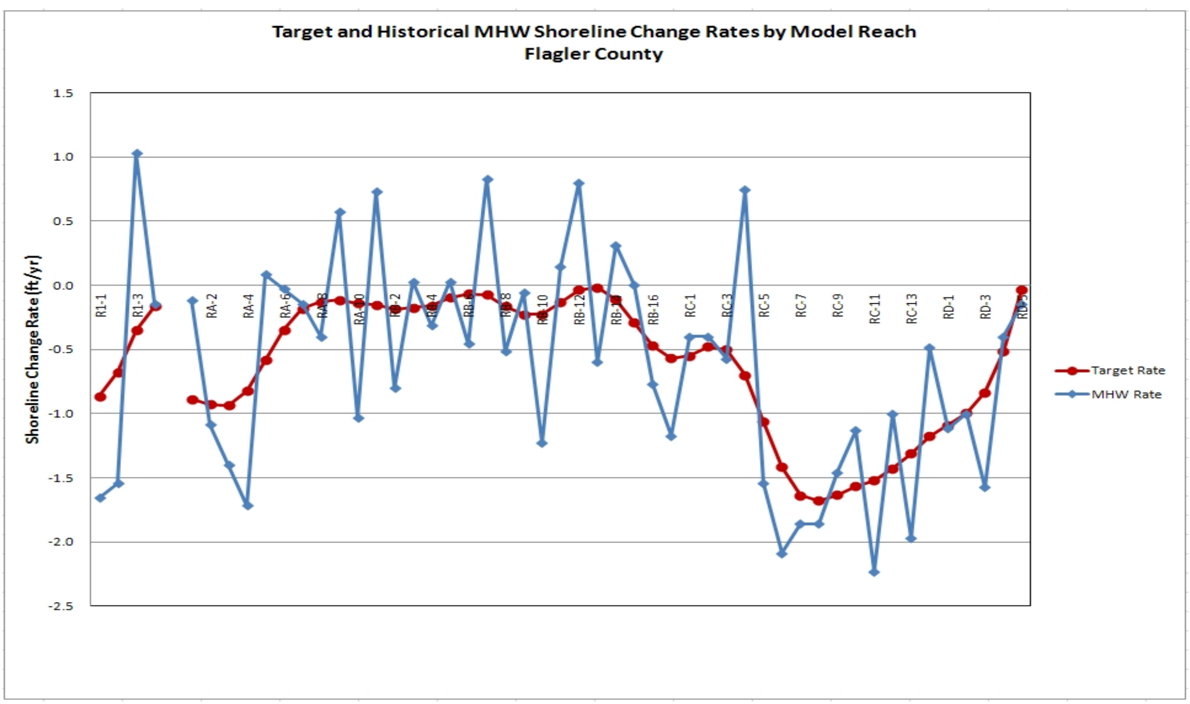

Figure A- 19. Target and Historical MHW Shoreline Change Rate

Project Induced Shoreline Change (GenCade)

The project induced shoreline change rate, also in feet per year, accounts for the alongshore dispersion of the placed beach nourishment material. Beach-*fx* requires the use of shoreline change rates in order to represent the planform diffusion of the beach fill alternatives after placement. The USACE one-dimensional shoreline change model GenCade was utilized to determine how the beaches at Flagler County, Florida would respond to shore protection alternatives, specifically beach nourishment. The results from each beach fill alternative model run, including the no-fill without project condition, provided the planform 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 longshore sand 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).

Beach Profile Survey Data

All available survey data for Flagler County and neighboring St. Johns and Volusia Counties were obtained from the FDEP. The shoreline data were analyzed to determine shoreline and volume changes between surveys as well as cumulatively. The cumulative change was averaged through time to arrive at average annual shoreline and volume change rates.

Shoreline Change (Countywide)

Table A- 17 provides the Mean High Water (MHW) shoreline changes calculated for Flagler County. This table is an expansion of Table A- 9 (Historical Shoreline Change), which included only the project areas.

Overall, between 1972 and 2007 the shoreline receded on average -16 feet (-0.4 feet per year), with the greatest cumulative changes equal to -78 feet (-2.2 ft/yr) at monument R-91 and +37 feet (+1.1 ft/yr) at monument R-62. This indicates that although the shoreline was generally receding, some areas exhibited shoreline advance, most notably between R-59 and R-84, as seen in Figure A- 20. During the 1984 to 1986/87 period, the average shoreline change equaled -8 feet with all changes being greater than -59 feet (R-3) and less than +21 feet (R-63 and R-69) (Figure A- 21). During the 1986/87 to 1993 period, shoreline change averaged 6 feet, with the greatest changes equal to -28 feet at R-63 and +54 feet at R-99.

Table A- 17. Flagler County Mean High Water Change

R-Monument	Mean High Water Change (feet)							
	1972 to 1980	1980 to 1984	1984 to 1986/87	1986/87 to 1993	1993 to 2000	2000 to 2003	2003 to 2007	Cumulative
1	-68			14	0	78	-85	-58
2	-44					73	-87	-54
3	13	56	-59	13	-2	54	-39	36
4	-53					22	-30	-5
5	-48					7	-6	-13
6	-43	27	7	1	-7	-2	1	-16
7	-34					-17	3	-15
8	-61					-5	-15	-18
9	-27	9	3	4	9	-19	2	-19
10	-52					5	-23	-24
11	-54					-8	12	16
12	-39	33	0	5	-6	-5	6	-6
13	-33					19	-14	-7
14	-43					10	-2	-1
15	-66	45	9	-8	-5	-5	-2	-32
16	-39					-13	2	3
17	-54	50	2			20	-33	-23
18	-24			-4	15	-6	-17	-11
19	-6					-23	8	4
20	-34					-3	-3	-29
21	-15	12	-11	16	-23	16	-10	-15
22	-62					9	9	-14
23	-84					10	-24	-42
24	-74			7	7	36	-47	-37
25	-59	61	-8			-18	-3	-12
26	-92					-2	-10	-30
27	-72	56	5	-13	8	8	-15	-23
28	-71					11	-3	-21
29	-70					9	-12	5
30	-96	62	9	3	-11	1	16	-16
31	-76					20	-9	-6
32	-69					8	-4	-13
33	-92	68	2	-10	9	0	0	-23
34						17	-10	
35	-59	70	-9			11	-7	7
36	-66				1	-2	-1	-10
37	-68					3	0	4
38	-39					-11	-7	-1
39	-38	29	1	3	17	-3	-2	7
40	-71					-3	-6	-8
41	-39					-12	-3	0
42	-45	51	-16	0	0	4	-17	-23
43	-30					23	-36	-19
44	-38					-2	-26	-35

R-Monument	Mean High Water Change (feet)							
	1972 to 1980	1980 to 1984	1984 to 1986/87	1986/87 to 1993	1993 to 2000	2000 to 2003	2003 to 2007	Cumulative
45	-42	70	-55	9	16	6	-6	-2
46	-24					-13	-18	13
47	-56					14	-26	-3
48	-45	73	-51	25	6	-9	-15	-16
49	-57					2	-25	-17
50	-49					16	-13	-4
51	-73	40	-39	43	-1	-7	-1	-38
52						1	-26	
53	-63					12	-16	-49
54	-17	18	-5	17	-14	0	4	3
55	-42					-8	22	3
56	-79					-1		
57	-34	36	12	-9	5	-13	-2	-5
58	-43					-6	-1	-14
59	-39					-7	13	20
60	-69					0	-15	-36
61	-47	71	4	1	-20	3	2	14
62	-26					-9	29	37
63	-59	52	21	-28	17	-11	-20	-28
64	-65					-5	24	1
65	-48					-1	-3	-11
66	-31	34	-1	1	-8	-1	7	1
67						-5	26	
68	-24					22	-51	-16
69	-8	10	21			-13	7	29
70	-7			-13	7	-10	-8	-18
71	-21					-18	13	-2
72	-58			-3	-7	-5	-38	-43
73	-14	40	16			-14	-7	5
74	-16					-15	19	28
75	-21	55	-7	0	-11	11	-48	-21
76	-24					48	-29	11
77	-30					15	-1	19
78	-13	34	16	-6	-35	10	-28	-22
79	-69					39	-51	-27
80	-55					44	-53	-41
81	-71	52	-26	40	-41	33	-26	-39
82	-51					40	-28	-20
83	-23					7	-6	-2
84	-29	30	3	-14	7	-6	35	26
85	-54					7	-26	-54
86	-53					18	-51	-73
87	-45	64	2	-11	-23	19	-71	-65
88	-29					19	-43	-51
89	-31					39	-37	-31

R-Monument	Mean High Water Change (feet)							
	1972 to 1980	1980 to 1984	1984 to 1986/87	1986/87 to 1993	1993 to 2000	2000 to 2003	2003 to 2007	Cumulative
90	-45	61	-39	21	-46	40	-40	-48
91	-69					15	-54	-78
92	-63					13	-4	-35
93	-65	54	-8	-9	-22	20	-39	-69
94	-59					38	-24	-17
95	-74					46	-48	-39
96	-60	83	-42	34	-30	30	-30	-15
97	-58					23	-59	-55
98	-39					40	-29	-3
99	-50	55	-28	54	-48	6	-13	-24
100	-37					23	-24	-5

Average	-47.5	47.3	-8.2	5.5	-6.9	8.6	-15.2	-15.9
Min	-96.0	9.0	-59.0	-28.0	-48.0	-23.0	-87.0	-78.0
Max	13.0	83.0	21.0	54.0	17.0	78.0	35.0	37.0
Std Dev	21.1	19.4	22.1	18.1	18.0	19.6	23.3	23.6
Average (ft/yr)	-5.9	11.8	-2.7	0.9	-1.0	2.9	-3.8	-0.5

Figure A- 21 suggests that sand waves are moving along the shoreline, likely in the direction of the dominant sediment transport (north to south), since the 1984 to 1986/87 period shows areas of erosion in the same location as the areas of accretion during the following 1986/87 to 1993 period. The five point moving averages support this observation with nodes and antinodes observed in Figure A-21.

Historical Volume Change

The beach profile survey data that was collected in 1972, 1987, 2003, and 2007 extended far enough offshore to adequately describe the profile volume of the given year, and all except for 2007 covered virtually the entire county; 2007 only covered the northern third of the county. The shoreward and seaward limits used to calculate volume changes were set to coincide where beach profile surveys crossed on the seaward face of the dune and again where the survey data converged offshore (depth of closure). Volumes were calculated for each profile as unit volumes (cubic yards per linear foot of beach) (Table A- 18) and actual volume changes (Table A- 19). Volumes between profile lines were computed by the average end area method where two adjacent profile unit volumes are averaged then multiplied by the distance between the profile lines to arrive at the volume in cubic yards.

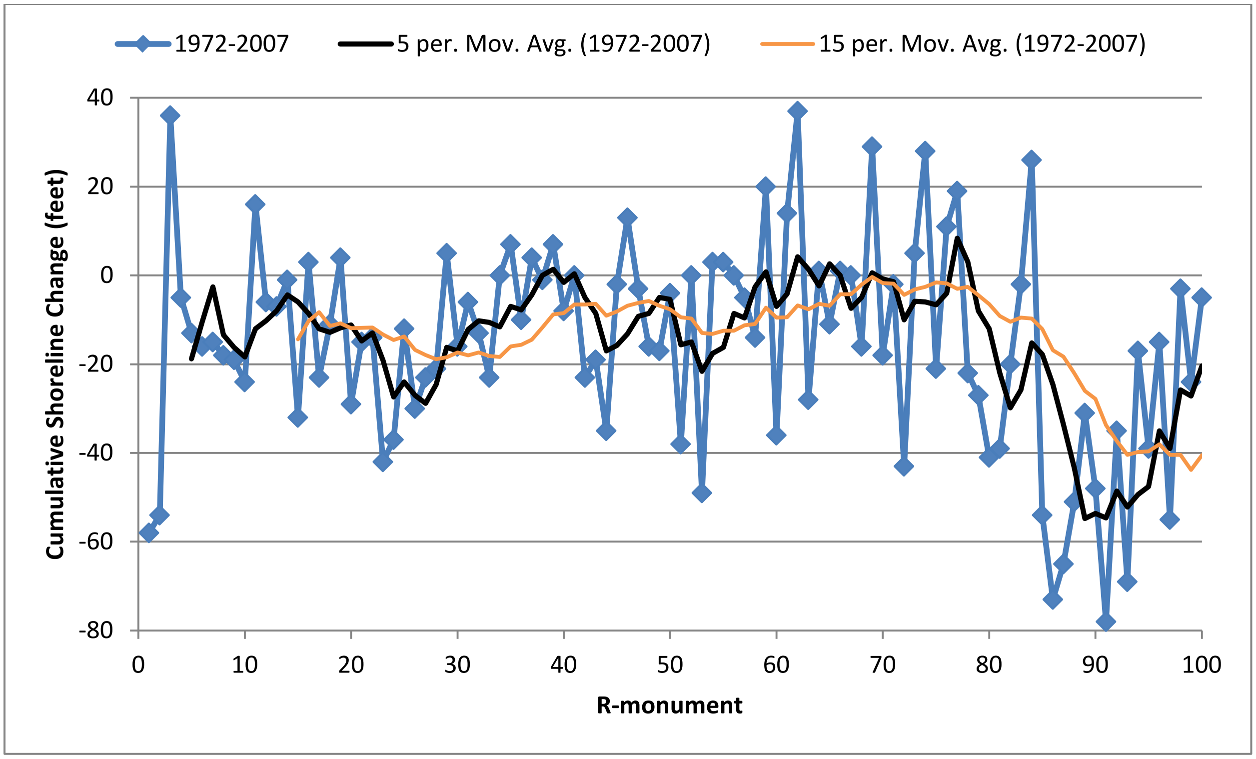

Figure A- 20. Flagler County Cumulative Shoreline Change (1972-2007)

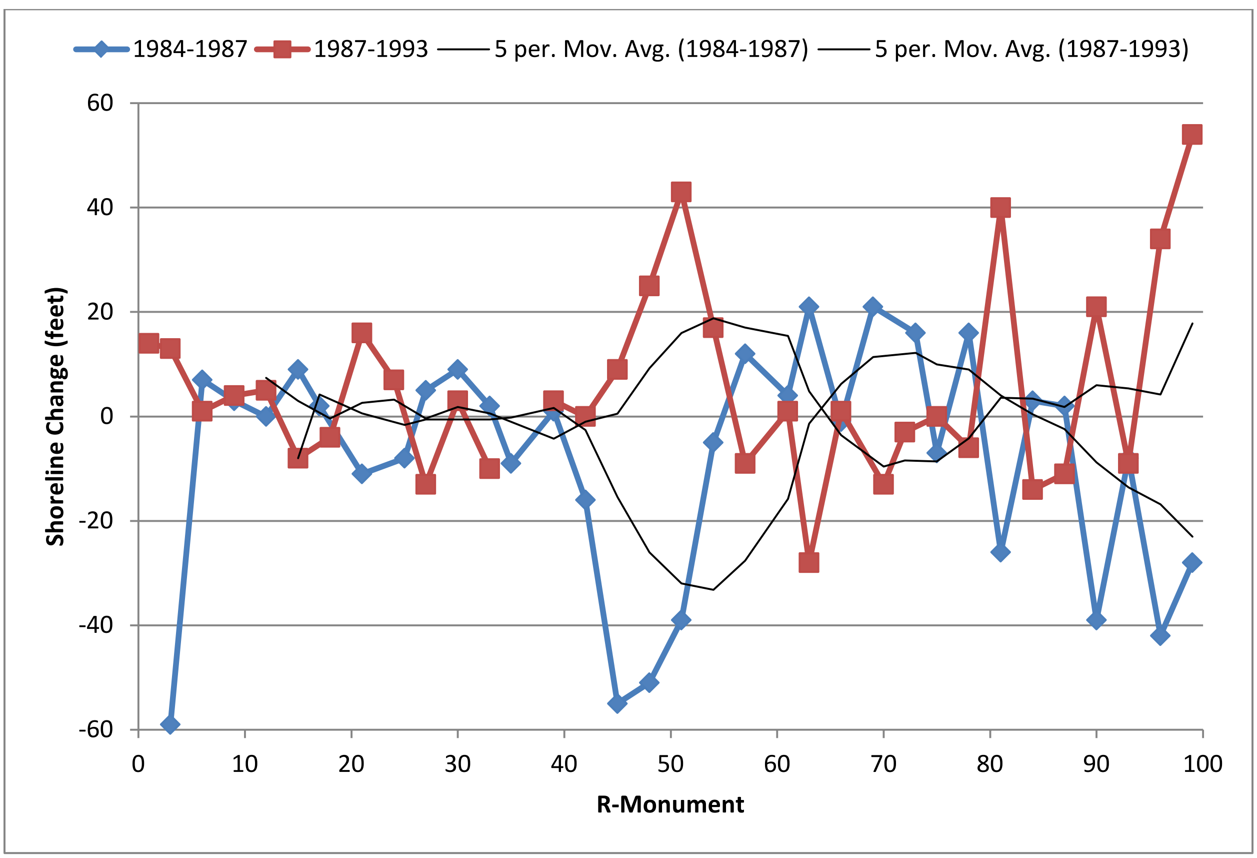

Figure A- 21. Flagler County Shoreline Change during 1984-1986/87 and 1986/87 to 1993.

The 1972, 1987, and 2003 surveys provided the most complete spatial and temporal coverage of Flagler County and were used to compute the volume change over each period. The average annual net volume change rate was also calculated. The great variability in the volume changes are noted as the beaches of Flagler County between the period between 1972 and 1987 *lost* 2.5 Mcy (-177,000 cy/year) and between 1987 and 2003 *gained* 1.6 Mcy (94,000 cy/year). The cumulative change between 1972 and 2003 was therefore equal to -931,000 cy (-30,000 cy/year) over the 31 year period.

Table A- 18. Flagler County Unit Volume Change

R-Monument	Distance Between Monuments (feet)	Unit Volume Change (cy/lf)		
		1972 to 1987	**1987 to 2003**	**2003 to 2007**
1		-74	81	-37
2	1001	-53	79	-36
3	1252	-33	119	-12
4	760	-26	88	-42
5	946	-26	56	-19
6	1005	-19	39	-27
7	965	-40	-1	16
8	920	-40	26	-10
9	1088	-61	0	11
10	991	-39	33	-20
11	930	-39	22	-7
12	976	-16	-1	-2
13	898	-28	5	-12
14	931	-28	33	-2
15	1078	-40	11	12
16	866	-22	30	-28
17	993	-22	26	-18
18	946	-3	16	-24
19	1006	-8	-24	6
20	950	-8	-3	-12
21	1096	-13	-19	-4
22	1137	-21	16	-18
23	817	-21	7	-13
24	986	-28	31	-34
25	1242	-18	-1	-18
26	874	-18	7	-48
27	836	-7	-2	-43
28	879	-23	4	-41
29	908	-23	-4	-33
30	1229	-38	44	-49
31	815	-26	53	-75
32	999	-26	57	-81
33	997	-13	25	-30
34	796	-11	2	-39
35	1249	-11	10	-44
36	937	-10	12	

R-Monument	Distance Between Monuments (feet)	Unit Volume Change (cy/lf)		
		1972 to 1987	1987 to 2003	2003 to 2007
37	966	-24	41	
38	947	-24	5	
39	976	-38	56	
40	1023	-20	14	
41	929	-20	18	
42	1009	-2	18	
43	918	-3	-14	
44	1050	-3	-23	
45	922	-4	-26	
46	1103	-6	10	
47	993	-6	-20	
48	965	-8	0	
49	946	-17	6	
50	1004	-17	20	
51	918	-26	12	
52	890	-16	27	
53	451	-16	23	
54	803	-6	7	
55	1027	-6	-18	
56	869	-6	-21	
57	725	-5	-48	
58	1104	-9	-9	
59	1102	1	-17	
60	784	-25	9	
61	926	-26	-6	
62	995	-39	6	
63	995	-38	-27	
64	882	-36	32	
65	1014	-34	-1	
66	936	-37	10	
67	917	-39	30	
68	1053	-46	36	
69	883	-37	12	
70	1064	-34	21	
71	969	-23	43	
72	940	-11	51	
73	952	-10	13	
74	948	0	26	
75	945	-24	23	
76	964	-22	58	
77	977	-47	29	
78	969	-69	32	
79	961	-73	64	
80	945	-97	30	
81	948	-72	0	
82	1006	-65	39	
83	903	-35	7	
84	1048	-31	-7	

R-Monument	Distance Between Monuments (feet)	Unit Volume Change (cy/lf)		
		1972 to 1987	1987 to 2003	2003 to 2007
85	732	-22	-27	
86	962	-26	-7	
87	946	-14	-40	
88	890	-22	23	
89	810	-19	40	
90	954	-37	30	
91	931	-33	33	
92	949	-44	31	
93	955	-51	10	
94	948	-46	35	
95	950	-58	23	
96	860	-52	46	
97	971	-57	16	
98	983	-56		
99	951			
100	951			
Average		-28	17	

Table A- 19. Flagler County Volume Changes

R-Monument	Distance Between Monuments (feet)	Volume Change (cy)		
		1972 to 1987	1987 to 2003	2003 to 2007
1				
2	1001	-63,688	79,900	-36,531
3	1252	-53,907	123,555	-30,099
4	760	-22,287	78,270	-20,550
5	946	-24,516	67,855	-28,550
6	1005	-22,599	47,492	-22,754
7	965	-28,476	18,290	-4,938
8	920	-36,779	11,699	2,785
9	1088	-54,877	13,990	116
10	991	-49,346	16,179	-4,781
11	930	-35,962	25,698	-12,831
12	976	-26,874	10,236	-4,700
13	898	-20,128	1,773	-6,604
14	931	-26,452	17,935	-6,637
15	1078	-37,114	23,680	5,241
16	866	-26,857	17,621	-7,263
17	993	-21,431	28,083	-23,050
18	946	-11,492	19,944	-20,051
19	1006	-5,283	-4,402	-8,971
20	950	-7,408	-12,877	-2,835
21	1096	-11,331	-11,771	-9,180
22	1137	-19,038	-1,818	-12,669
23	817	-16,820	9,329	-12,475
24	986	-24,099	18,964	-22,849
25	1242	-28,485	18,607	-31,910
26	874	-15,367	2,664	-28,804
27	836	-10,220	2,344	-37,959

R-Monument	Distance Between Monuments (feet)	Volume Change (cy)		
		1972 to 1987	1987 to 2003	2003 to 2007
28	879	-12,961	1,134	-36,735
29	908	-20,558	-19	-33,416
30	1229	-37,519	24,113	-49,942
31	815	-26,126	39,331	-50,551
32	999	-25,675	54,642	-77,834
33	997	-19,285	40,642	-55,002
34	796	-9,727	10,768	-27,522
35	1249	-14,301	7,817	-51,857
36	937	-10,002	10,434	
37	966	-16,328	25,499	
38	947	-22,611	21,817	
39	976	-30,135	29,779	
40	1023	-29,622	35,729	
41	929	-18,627	15,048	
42	1009	-11,241	18,180	
43	918	-2,444	1,654	
44	1050	-3,247	-19,598	
45	922	-3,249	-22,615	
46	1103	-5,614	-8,576	
47	993	-6,177	-4,888	
48	965	-7,099	-9,825	
49	946	-12,182	2,627	
50	1004	-17,323	12,893	
51	918	-19,874	14,732	
52	890	-18,783	17,409	
53	451	-7,304	11,348	
54	803	-9,040	12,054	
55	1027	-6,278	-6,087	
56	869	-5,124	-17,022	
57	725	-4,115	-24,846	
58	1104	-8,117	-31,369	
59	1102	-4,610	-14,545	
60	784	-9,498	-3,334	
61	926	-23,872	1,130	
62	995	-32,335	-97	
63	995	-38,262	-10,421	
64	882	-32,650	1,920	
65	1014	-35,415	15,671	
66	936	-33,555	4,260	
67	917	-35,307	18,049	
68	1053	-44,739	34,699	
69	883	-36,259	21,116	
70	1064	-37,734	17,269	
71	969	-27,927	31,193	
72	940	-16,059	44,334	
73	952	-9,938	30,224	
74	948	-4,707	18,510	
75	945	-11,268	23,432	

R-Monument	Distance Between Monuments (feet)	Volume Change (cy)		
		1972 to 1987	1987 to 2003	2003 to 2007
76	964	-22,043	39,170	
77	977	-33,331	42,607	
78	969	-55,789	29,629	
79	961	-68,016	45,964	
80	945	-80,540	44,192	
81	948	-80,325	14,164	
82	1006	-68,743	19,555	
83	903	-44,901	20,634	
84	1048	-34,488	6	
85	732	-19,579	-12,607	
86	962	-23,313	-16,627	
87	946	-19,179	-22,477	
88	890	-16,065	-7,863	
89	810	-16,289	25,471	
90	954	-26,570	33,630	
91	931	-32,427	29,227	
92	949	-36,303	30,081	
93	955	-45,324	19,682	
94	948	-46,019	21,484	
95	950	-49,304	27,418	
96	860	-47,146	29,673	
97	971	-52,740	30,416	
98	983	-55,250		
99	951			
100	951			
Total		-2,549,342	1,562,883	-771,709*
Number of Years		14.43	16.68	4.17*
Average Annual (cy/yr)		-176,701	93,717	-185,068*

*Note 2007 Survey only covered R-1 to R-35

Longshore Sediment Transport

The primary force that drives longshore sediment transport in Flagler County is the incident wave field. The net transport in Flagler County is generally north to south similar to other east coast Florida beaches as a result of the majority of annual wave energy originating from Northeaster conditions. However, during years of decreased Northeaster activity or in the calmer summer months dominated by smaller waves from southeast trade wind activity, the net annual transport can reverse to the north. The passage of tropical systems also promotes north-directed sediment transport.

The 1980 USACE Reconnaissance Report for Flagler County stated that the net annual southerly drift equaled 350,000 cy/yr (268,000 m^3/yr), based on Summaries of Synoptic Meteorological Observations (SSMO) data produced by the U.S. Navy (see Table 1). Further, the northward and southward transports equaled 263,000 cy/yr (201,000 m^3/yr) and 613,000 cy/yr (469,000 m^3/yr), respectively, resulting in a gross transport of 876,000 cy/yr (670,000 m^3/yr). Taylor (2002) reported that net transport in Flagler Beach ranged from 20,000-450,000 cy/yr (15,300-344,000 m^3/yr), using the CERC equation.

Table 1. Longshore Sediment Transport Estimates for Flagler County, FL.

Year	Report Title	Author	Sediment Transport Rates (cy/yr)
1980	Flagler County SPP Reconnaissance	USACE	+350,000 (Net) -263,000 (Northward) +613,000 (Southward) 876,000 (Gross)
2002	State Road A1A Shore Protection Evaluation Flagler Beach, Flagler County, FL	Taylor Engineering, Inc.	+20,000 to +450,000 (Net)
2007	Northeast Florida Atlantic Coast Regional Sediment Budget Nassau Through Volusia Counties	USACE	+177,000 at R-1 (Net); +154,000 at R-100 (Net)
2012	Regional Sediment Budget for St. Augustine Inlet and St. Johns County, FL, 1998/1999-2010	USACE	+100,000 (Net)

USACE (2007) studied the northeast Florida shoreline in order to develop a regional sediment budget using published volume change rates and transport rates as well as computing volume changes from surveys. The volume changes reported for Flagler County differed from those reported above and may be due to different calculation techniques; the 2007 study utilized digital terrain model surfaces to compare volume changes between surveys. The period from 1972 to 1987 was reported to change -208,000 cy/yr and by calculation the period from 1987 to 2003 was found to equal +240,000 cy/yr. So the total change over the 1972 – 2003 period equaled +32,000 cy/yr. The discrepancies were not further investigated, but more importantly and more relevant to the development of this study are the net transport rates that were calculated. The values equaled +177,000 cy/yr (with positive indicating southward transport) at the north end of the Flagler County cell, and +154,000 cy/yr at the south end using the calculated volume change within the cell equal to +23,000 cy/yr as previously indicated.

A recent report by USACE (2012-a) revised the regional sediment budget for St. Augustine Inlet and St. Johns County, FL. The analysis utilized a family of solutions scheme in order to converge on the final solution through the balancing of transport rates. At the southern end of the study area, at R-151- approximately 11.5 miles north of Flagler County, net transport was found to equal 100,000 cy/yr (76,000 m^3/yr) to the south. So with the more recent studies (2007 and 2012) in mind, net transport rates for Flagler should range from between 100,000 cy/yr (76,000 m^3/yr) to 177,000 cy/yr (135,000 m^3/yr). Further, if the volume change for Flagler calculated over the period from 1972 to 2003 were used in the 2007 USACE study, the upper range of net transport would thus be 207,000 cy/yr (158,000 m^3/yr). Given that these values of net transport are reduced from the original 1980 estimate by USACE, it is expected that the gross transport rate from that report (the only gross transport estimate available) over predicts the actual rate.

GenCade Model Setup

In order to fully capture the shoreline changes within Flagler County and to minimize any boundary effects to the study area, the GenCade model domain was extended to include about 2.5 miles of shoreline in St. Johns County to the north and a similar extension into Volusia County to the south. The northern boundary was therefore set to approximately coincide with FDEP monument R-198 in St. Johns County and the southern boundary extended to R-12 in Volusia County. The shoreline input data were limited however to the length of shoreline including R-201 in St. Johns County, R-1 through R-100 in Flagler County, and R-9 for Volusia County for the calibration and verification time periods. The model grid was located sufficiently landward so that the modeled shoreline would never intersect or recede landward of the grid. Additionally, the grid was oriented so that it was approximately parallel to the Flagler County shoreline, an azimuth of 158 degrees measured clockwise from due north (0°). Although Flagler County does have some shore protection structures in place, such as seawalls, revetments, and groins, the model setup ignored these structures since the seawalls and revetments are very rarely in contact with the ocean, and the groins are very porous with low crest heights, showing no signature of influence to the sediment transport that is typically observed with more prominent features.

The progression of the shoreline change modeling includes model calibration which tunes the model to better approximate the local conditions, the verification period which confirms that the model has been adequately adjusted during calibration, and finally the actual production runs which predict how engineering modifications to the system will react under typical conditions. Model guidance suggests that the GenCade model is most applicable to trending shorelines. Shoreline and volume changes were calculated and reviewed to select the best potential time periods where corresponding wave data were available. The wave data used for the analysis limited the number of surveys that could be used for direct comparison to the modeled results, specifically to the years between 1980 and 1999. The years selected for calibration and verification are bracketed by the surveys from 1984, 1987, and 1993.

The calibration start and end dates were set to a median date for each beach profile survey, or 8/1/1984 and 1/14/1987, respectively. The verification period immediately followed and covered the time period of 1/15/1987 to 07/31/1993. The calibration and verification periods were driven by wave data sets corresponding to the same dates of the beach profile surveys. For the production runs, engineering alternatives were introduced to the model space and shoreline changes were driven by using an average year of wave data repeated over ten years.

The computation time step was set at a three hour interval and modeled shoreline outputs were set to 168 hours (1 week). Grid cell sizes were set at 305 meters (~1,000 feet) which is approximately equal to the spacing of established monuments where profile line monitoring surveys originate. Beach profile surveys were analyzed through time to find the average berm height (11 feet; 3.35 meters) and depth of closure (20 feet; 6.1 meters) for the study area; these values are required model input and define the zone of sediment movement in the model. Geotechnical investigation of the beach sediments in Flagler County indicate that the median grain size (d50) was found to equal 0.44 mm. The calibration coefficients K1 and K2 were given the default values of 0.5 and 0.25, respectively. The lateral boundary condition was set to “pinned” meaning the shoreline doesn’t change position at either end. Further discussion of the lateral boundary condition is included in the Calibration section below. All other model parameter values were kept as default.

Wave Data

The GenCade model requires breaking wave heights to calculate potential sediment transport rates which are then used to calculate shoreline changes. Since breaking wave heights are rarely measured or readily available, GenCade includes an internal wave model that can transform waves from a given offshore depth to breaking depth using linear wave theory. If the offshore depth contours are believed to follow the shoreline as straight and parallel lines, using the internal wave model is recommended.

Although it is accepted that the contours off of Flagler County are generally rather straight and parallel, the offshore bathymetry as seen in Figure A- 22 suggests some features could influence the wave field. Since the degree of influence of the offshore features was unknown, the wave transformation model within the Coastal Modeling System (CMS-Wave) was used to shoal and refract the waves over the irregular bathymetry to a location seaward of the breaking depth. The transformed wave data output from the CMS-Wave model were then manipulated into the format required for input into GenCade.

The most complete wave data set available for Flagler County is a 20-year hindcast produced by USACE known as the Wave Information Studies. The wave hindcast is developed using computer models with observed wind fields as input. The wave model used for the Atlantic Coast, known as WISWAVE, is a discrete spectral wave model that solves the energy balance equation for the time and spatial variation of a 2-D wave spectrum from wind forcing (ERDC, 2012). The WIS database provides densely-spaced “virtual wave gages” between 15-20 meter (49-66 foot) water depths. The virtual wave gage closest to the study area is station 63422 located in 20 meters (66 feet) of water about 8.4 nautical miles from the shoreline at 29.58° latitude and -81.00° longitude.

Wave Transformation

To accomplish wave transformation from an offshore location to a nearshore location, CMS-Wave requires bathymetry data over the model domain and an offshore wave input. The National Geophysical Data Center (NGDC) of the National Oceanic and Atmospheric Administration (NOAA) maintains a database that integrates offshore bathymetry and land topography for the U.S. coastal zone known as the Coastal Relief Model (CRM). Through the internet interface, users can download data sets for one degree by one degree areas, or create custom grids to suit a project’s needs (NOAA, 2012).

For the Flagler County Feasibility Study, a custom CRM grid was created that extended well beyond the study area and offshore of the WIS station. The last complete FDEP beach profile survey of Flagler County from 2003 was merged and smoothed with the custom CRM bathymetry grid to provide the best available nearshore bathymetry dataset. The CMS-Wave model grid extended 4.2 miles north into St.

Johns County, 3.2 miles south into Volusia County, and 10.8 miles offshore with an azimuth of 202°, measured counter-clockwise from due north (0°). The grid resolution was varied in the cross shore direction from about 6.5 meters (21 feet) in the nearshore zone to 120 meters (400 feet) offshore, using five refinement points that were spaced increasingly farther apart moving offshore. The cell widths in the alongshore direction were held constant at about 150 meters (500 feet). There were a total of 77964 cells with 267 cells in the alongshore direction and 292 cells in the cross-shore direction.

The WIS dataset is available as hourly two-dimensional directional spectra or in three-hour parameter form that includes wave height, period, and direction; each parameter corresponds to the parameter value at the peak energy location in each spectrum. Since the format of the WIS spectra files are not consistent with the spectral files used in CMS-Wave, the WIS parameter files were used for input waves. Once imported into CMS-Wave, the parameters were used by the internal spectral generator to create spectral files that follow a generalized distribution of energy with the peak energy centered at the parameter values for each time step.

Three wave observation stations were included in the model setup so the waves transformed with CMS-Wave could be easily obtained as a model output. The three stations were located near or just offshore of the 5 meter (16 foot) contour and at the north, center, and south ends of the modeled area. This contour location was tested by running CMS-Wave with the month containing the largest wave event of the 20-year record to ensure that it was sufficiently deep so that the largest wave event was not breaking at this location.

By linear wave theory, the largest wave this location could observe before the breaking condition was met equals 3.75 meters (12.30 feet). During the month of the maximum observed wave height, the offshore height at WIS station 63422 equaled 8 meters (26 feet) at a peak period of 14.3 seconds. The nearshore wave height for this event after transformation from the offshore virtual wave gage location equaled 4.1 meters (13.5 feet) at the southern observation station, the maximum of the three stations. The model also reported that wave breaking did not occur indicating that the 5 meter depth was a reliable location to obtain transformed wave data over the entire 20 year period for input into GenCade. The output from the three observation stations was bracketed by the times corresponding to the calibration and verification periods, and manipulated into the GenCade input file format.

Calibration

The calibration procedure optimizes site specific model parameters for a given study area by comparing model predictions with measured shoreline change and calculated or published transport rates. It was previously noted that model guidance suggests that the model works best on shorelines with an established trend in its position change. One way to calibrate to shorelines exhibiting minimal trends in behavior is to compare the calculated longshore transport rates with the modeled values (USACE, 2012-b). For Flagler County, calibration of model parameters was the result of statistical analysis of the modeled versus measured shoreline. In addition, parameter values that resulted in unreasonable longshore transport rates were excluded from the analysis.

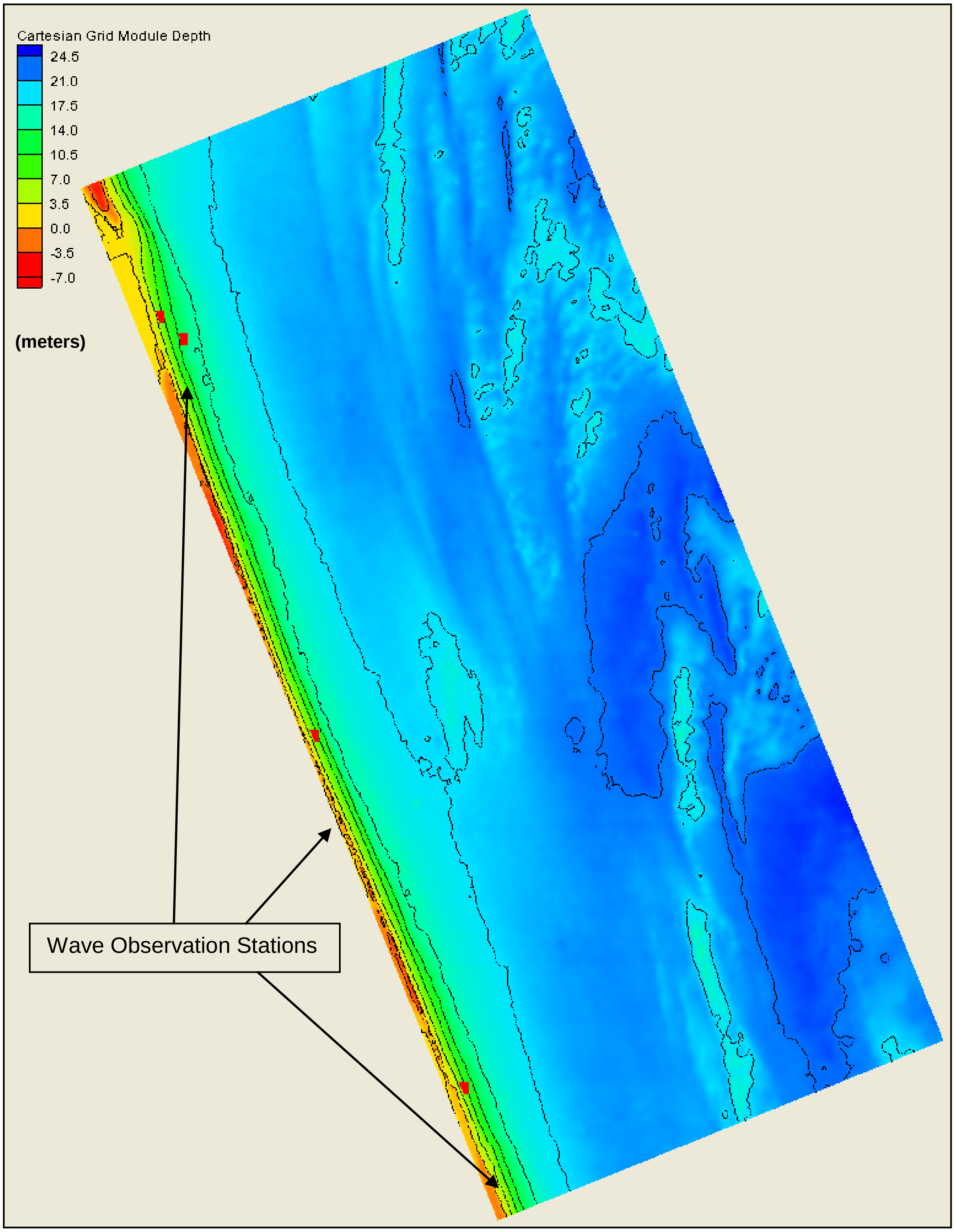

Figure A- 22. CMS-Wave Model Domain for Flagler County Feasibility Study

The calibration period was initially set up using all default parameter values with the 1984 measured shoreline as the initial shoreline and the three wave inputs that were output from the CMS-Wave model. Over the same model grid, the final surveyed shoreline for the calibration period from 1987 was added and saved. This allowed for comparison between the measured 1987 shoreline and the final shoreline calculated by the model, each referenced to grid-space. Additionally, the 1987 shoreline was ready for use following calibration in the model verification phase. The final shoreline for the verification period (1993) was similarly loaded into the model space and referenced to the model grid for later comparison with the 1987 initial shoreline condition for the verification phase.

The output from GenCade provided a calculated final 1987 shoreline plus sediment transport rates. Figure A- 23 shows the initial 1984 shoreline, modeled 1987 shoreline, and measured 1987 shoreline positions. The figure shows the model has heavily smoothed any undulation in the initial shoreline, including the embayment-like feature near the center of the study area. The measured versus modeled shoreline change reflects some of the discrepancies in shoreline positions, as seen in Figure A- 24. Some of the shoreline change magnitudes and directions match up well, but the majority of shoreline change does not and the worst areas between the modeled versus measured change are almost fully out of phase, resulting in a root mean square error (RMSE) of 11 meters (36 feet). Thus it was decided to incorporate a regional contour. Additionally, although the net annual transport rate calculated by the model seems to fall in the range provided by previous studies, the model is over predicting the gross rates (see Figure A- 25).

In order to preserve dominant regional shoreline characteristics in the absence of controlling structures, for example an embayment, GenCade provides the option to use a regional shoreline. If an open coast shoreline doesn't have specified sediment sources or sinks, or structures (such as the Flagler County shoreline), simulations over long time periods will result in a straight coastline. By providing a regional contour, the modeled shoreline will be guided in its evolution and eventually will assume a shape parallel to the regional contour if simulated over long enough time periods.

The regional contour was created using the mean high water location from the FDEP beach profile survey database by averaging the positions from the years of 1972, 1987, 1993, 2003, and 2007. Once the time-averaged shoreline was established, it was smoothed by applying a 3-point moving average. The regional contour was then formatted into the appropriate GenCade input file and loaded into the model. The results of the initial run using the regional contour with default parameter values can be seen in Figure A- 26 and Figure A- 27. Comparing the shoreline change figures (Figure A- 24 and Figure A- 27), one can see that the addition of the regional contour vastly improves the magnitudes and directions of shoreline change for the Flagler County study area, and the inflection points are more consistent with the measured data. The RMSE was reduced to 5.8 meters (18.9 feet) from 11.0 meters (36.0 feet) by adding the regional contour.

Following application of the regional contour, the first and most influential model parameter that was adjusted was the K1 transport coefficient. Changing the K1 value had direct noticeable effects to the longshore sediment transport rate. The magnitude of shoreline change also increased as K1 increased.

The K1 parameter was varied between 0.15 and 0.65, with the minimum achieved root mean square error result equal to 5.73 meters (18.8 feet) when K1 was set to 0.35. The average net annual transport predicted by GenCade with K1 equal to 0.35 ranged from -105,000 cy/yr (-80,000 m^3/yr) at the extreme north end of the county to around 146,000 cy/yr (112,000 m^3/yr) from the middle of the county southward; a negative transport rate indicates northbound sediment movement and a positive transport rate indicates southbound sediment movement. The gross transport for the same parameter model run was less varied across the model domain and equaled 719,000 cy/yr (550,000 m^3/yr) at the north end of the domain and 760,000 cy/yr (581,000 m^3/yr) at the south end, which is less than the USACE (1980) estimate of 876,000 cy/yr (670,000 m^3/yr).

The final K1 value of 0.35 resulted in an average annual shoreline change rate equal to -1.1 ft/yr. This is about two times as much as the historical average change rate of -0.5 ft/yr. Although this would indicate that the model is overpredicting shoreline recession, the longshore transport rates of the selected K1 value are at the bottom range of what was suggested in the literature. So reducing the K1 value in order to better align the modeled average shoreline change with the measured shoreline change over the calibration period would result in transport rates well below the lowest transport rate found in the literature. Further, when comparing the shoreline changes predicted by the model to the actual measured change, the calibration run with K1 set to 0.35 matched both the magnitude of shoreline change as well as the location of inflection points better than when K1 was set to 0.15.

The K2 longshore transport coefficient parameter is stated to only affect shoreline evolution in areas influenced by wave diffraction near structures (Frey et al. 2012). Nonetheless, K2 was varied during calibration following the selection of K1 equal to 0.35. Since the modeled Flagler County shoreline didn't include any structures, changes in the K2 value did not result in noticeable changes in modeled shoreline positions.

Other model parameters that had little effect on model solutions when varied were the median grain size (d_{50}) and the boundary type. The median grain size was varied between 0.10 mm and 0.65 mm with resulting RMSE values varying by less than 0.01 meters (0.03 feet). When the boundary type was set to a moving boundary that corresponded to the actual measured change at either end of the project during the calibration period, the result was not significantly improved. Due to the nature of the Flagler County shoreline, the values for the moving boundary applied during calibration would not be relevant during verification since the shoreline change direction and magnitude is rather ephemeral and not constant through time.

Therefore,
the lateral boundary condition was set to "pinned" meaning the shoreline doesn't change position at either end. The benefit of using a pinned boundary was to reduce any noise introduced by the oscillating nature of the shoreline which would be occur if using a moving boundary that reflects the actual measured change. For example during the calibration period the change at R-3 and R-99 equaled -59 and -28 feet, respectively. Those same monuments during the verification period equaled +13and +54 feet, respectively. The model was set up with the understanding that all parameters set during

calibration should be repeated during verification, and thus using a retreating shoreline during the verification period when measured data suggests an advancing shoreline didn't make sense. Tests were made during calibration using proxies for the long-term average shoreline change (0.5, 1.0, and 1.5 ft/yr), but the changes didn't improve the results. Further, the boundary was placed about 3.5 km north of the project area and 7.5 km south of the project area so problems with the boundary would be minimized in the area of interest.

The "Ismooth" model parameter, which represents the number of grid cells in the offshore contour smoothing window (a moving average applied to the shoreline to avoid instabilities caused by abrupt changes in shoreline orientation), was the only other parameter besides the K1 transport coefficient that exhibited some effect on the model solution. With the transport coefficient K1 set to 0.35, Ismooth was changed from the default value of 11 to a value of 1.0, 21, and 31. Changing the Ismooth parameter did not drastically change the fit between the modeled and measured shoreline, but the root mean square error was improved by 0.01 meters (0.03 feet) when using the value of 1.0. So the final Ismooth value used in verification and model runs on beach fill alternatives equaled 1.0 and resulted in an RMSE value of 5.72 meters (18.8 feet). The shoreline positions predicted from the final calibration run are seen in Figure A- 28, shoreline change for the same model run is found in Figure A- 29, and longshore transport is shown in Figure A- 30.

Verification

Using the parameters from the final calibration run, waves from the period between 15 January 1987 and 31 July 1993 were input along with the 1987 shoreline in order to model the verification period. As done with the calibration period, the final modeled shoreline position was compared with the final measured shoreline (1993) (Figure A- 31). For the majority of areas, the modeled shoreline followed the trend of the measured shoreline with some areas off in magnitude, typically predicting more erosion than measured, and other areas matching up very well in both trend and magnitude.

The shoreline changes predicted from the model are included with measured data in Figure A- 32, as well as the difference between the initial shoreline and the regional contour for reference. Between the 20km and 32km area of the grid space, the shoreline changes match extremely well. The areas where the model over-predicts erosion are more apparent in Figure A- 32 versus Figure A- 31. The influence of the regional contour on the final modeled shoreline is also apparent by the trend changes and minor perturbations in the two lines. The difference between the regional contour and the 1987 shoreline also largely follows the trends of the observed changes between 1987 and 1993, which suggests its validity. The RMSE for the validation run is included on Figure A- 32 and equaled 7.6 meters (24.9 feet). This value is greater than the calibration period, but deemed acceptable and validates that the model is properly calibrated. The longshore sediment transport rates during the verification period were reduced as compared to the final calibration run, but were determined acceptable (see Figure A- 33).

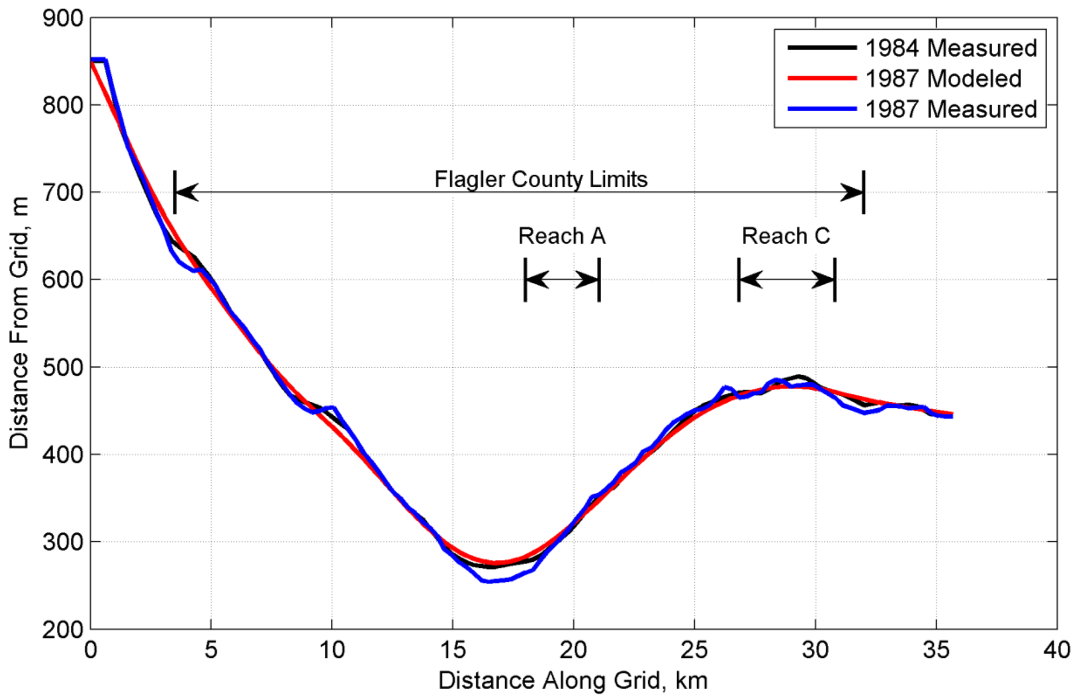

Figure A- 23. Modeled and Measured Shoreline Positions during Calibration Run #1

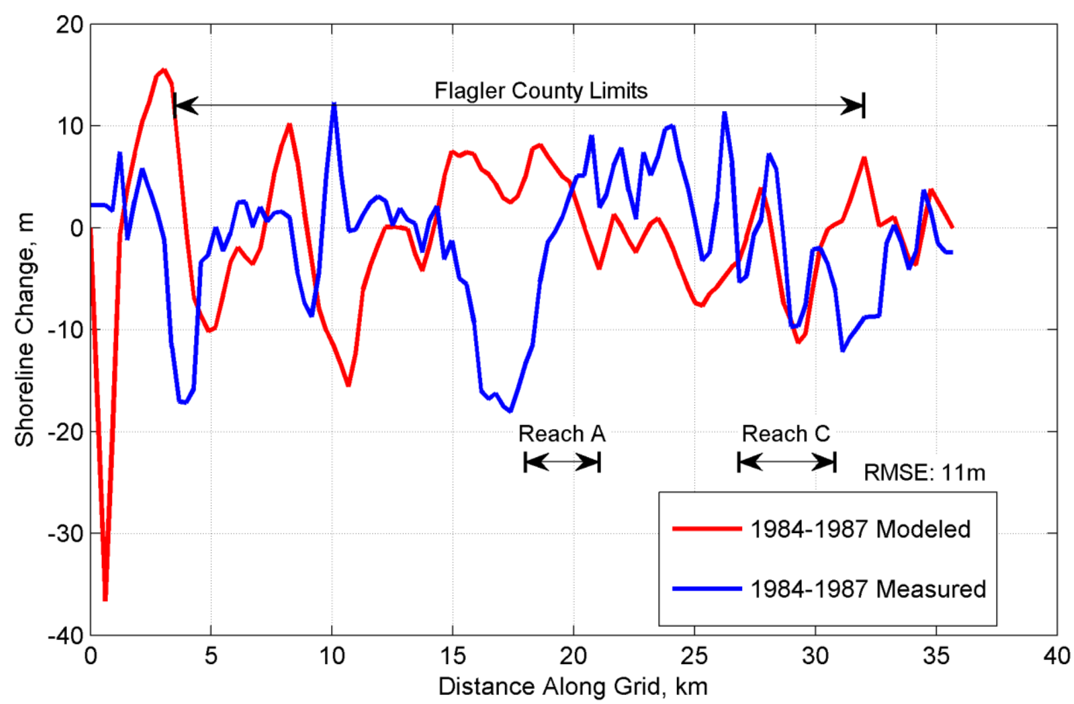

Figure A- 24. Modeled and Measured Shoreline Change during Calibration Run #1

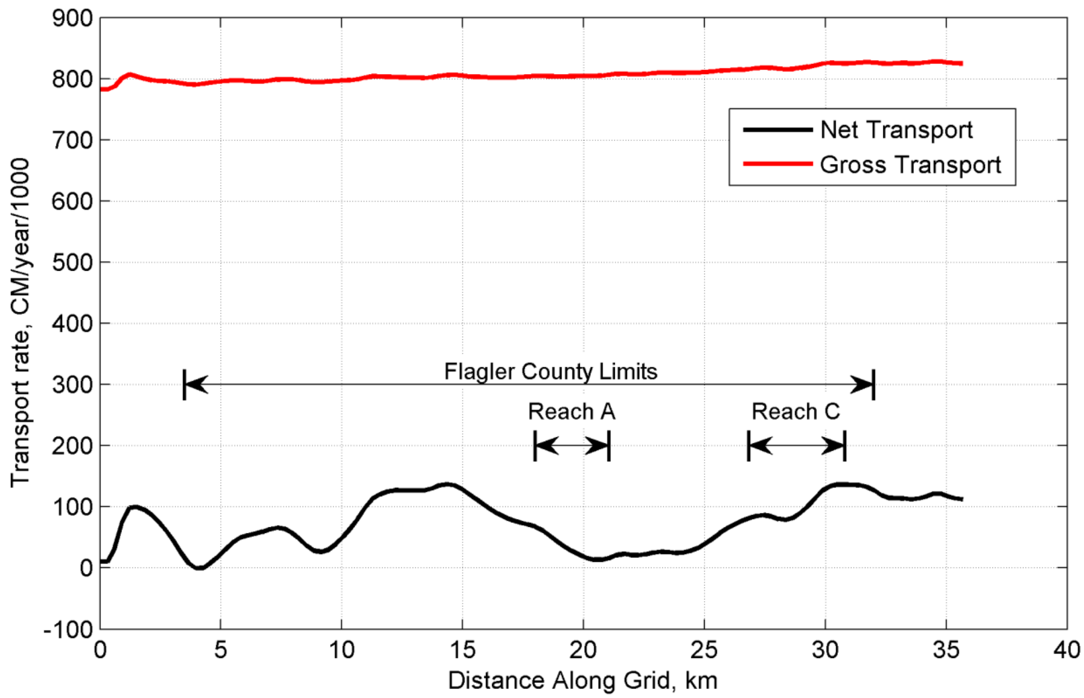

Figure A- 25. Longshore Sediment Transport for Calibration Run #1

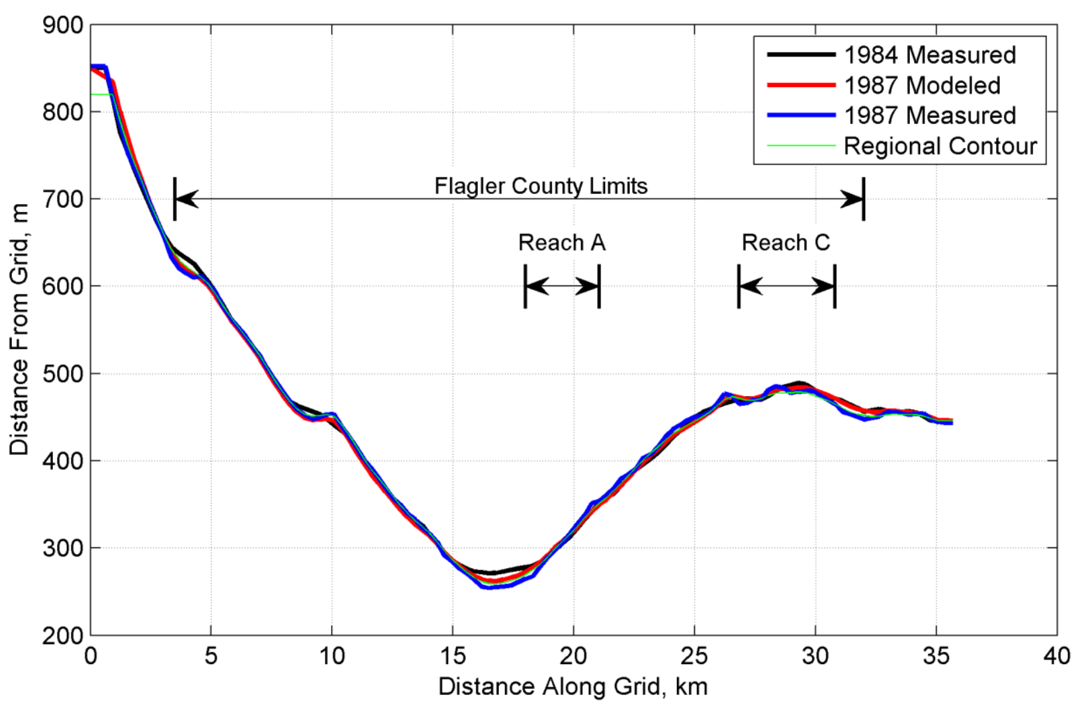

Figure A- 26. Modeled and Measured Shoreline Positions with Regional Contour

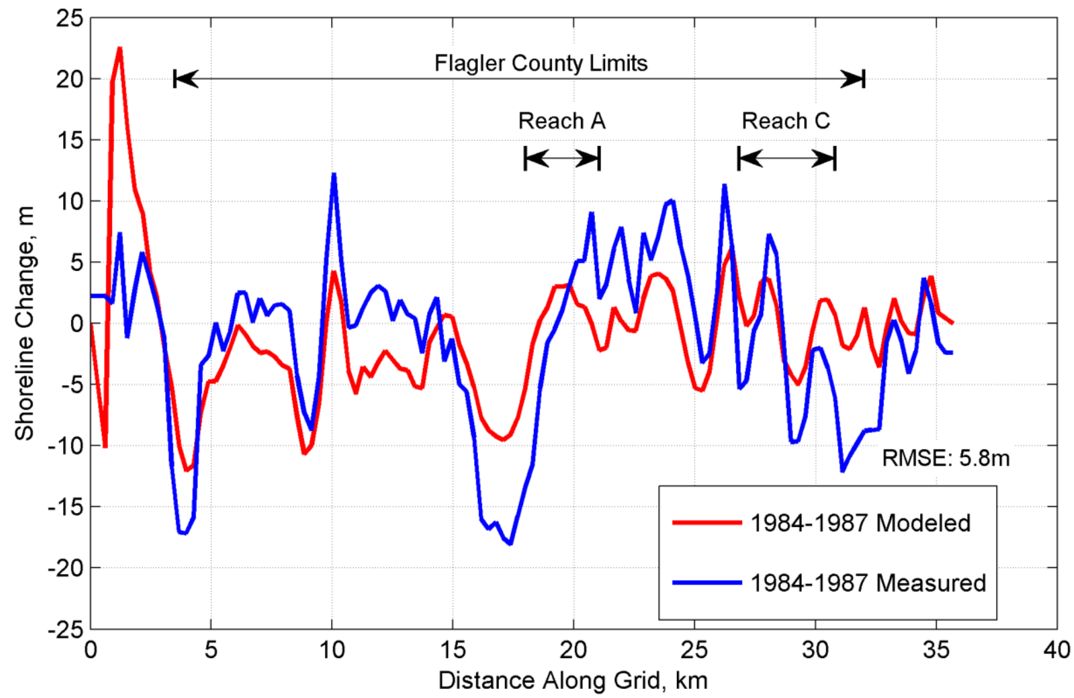

Figure A- 27. Modeled and Measured Shoreline Change with Regional Contour

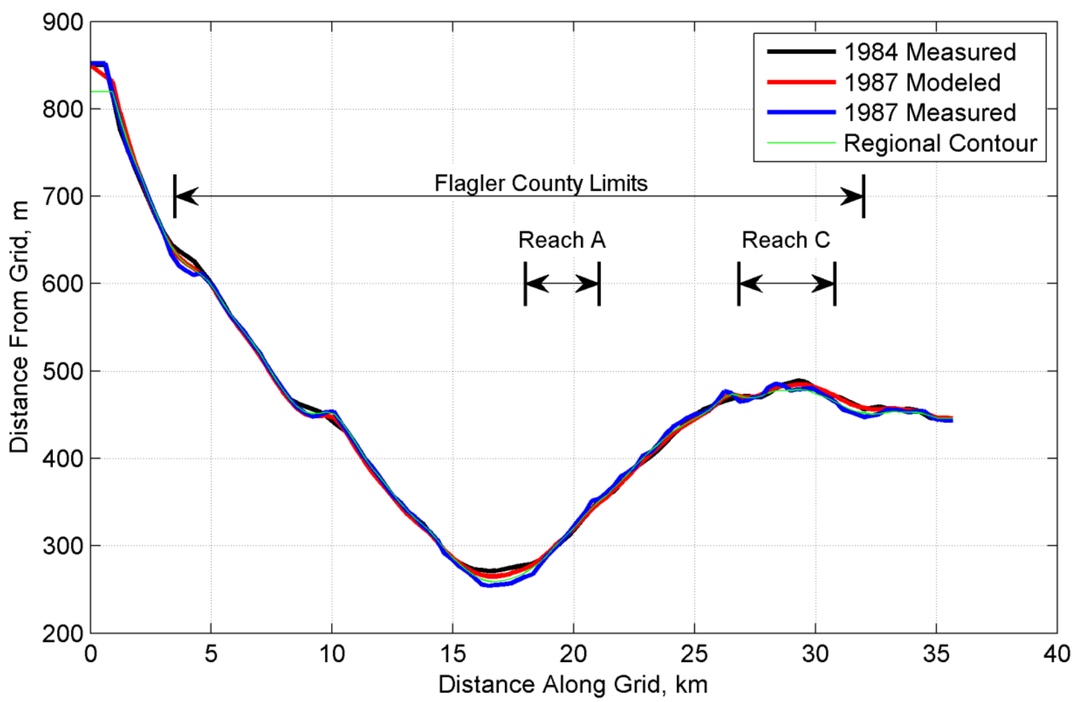

Figure A- 28. Final Calibration Modeled and Measured Shoreline Positions

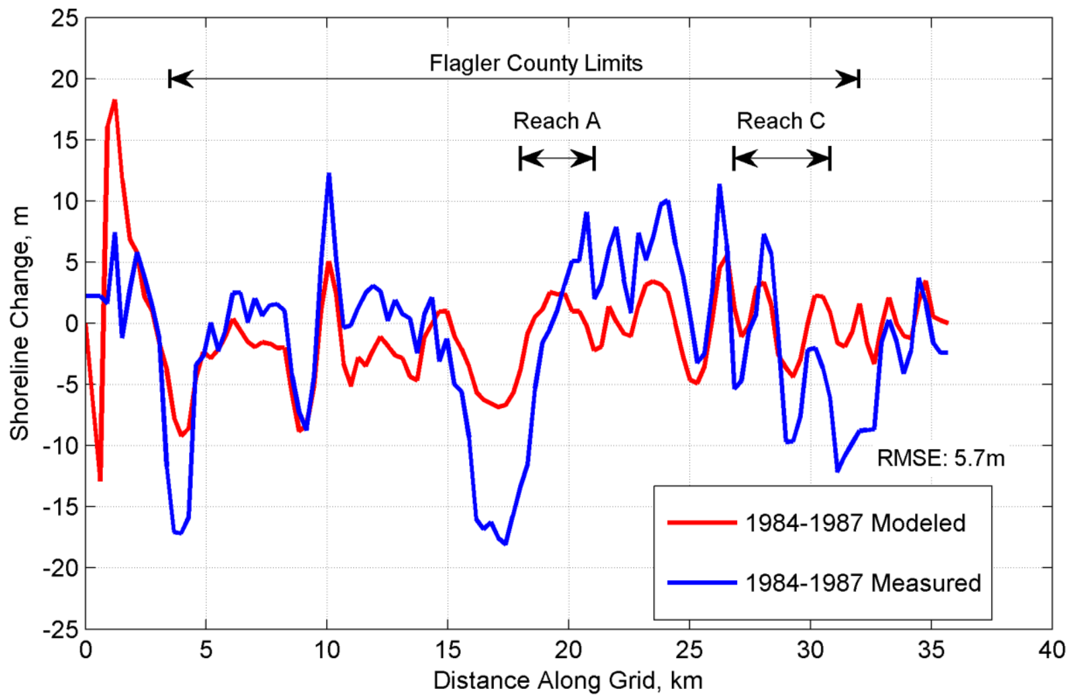

Figure A- 29. Final Calibration Modeled and Measured Shoreline Change

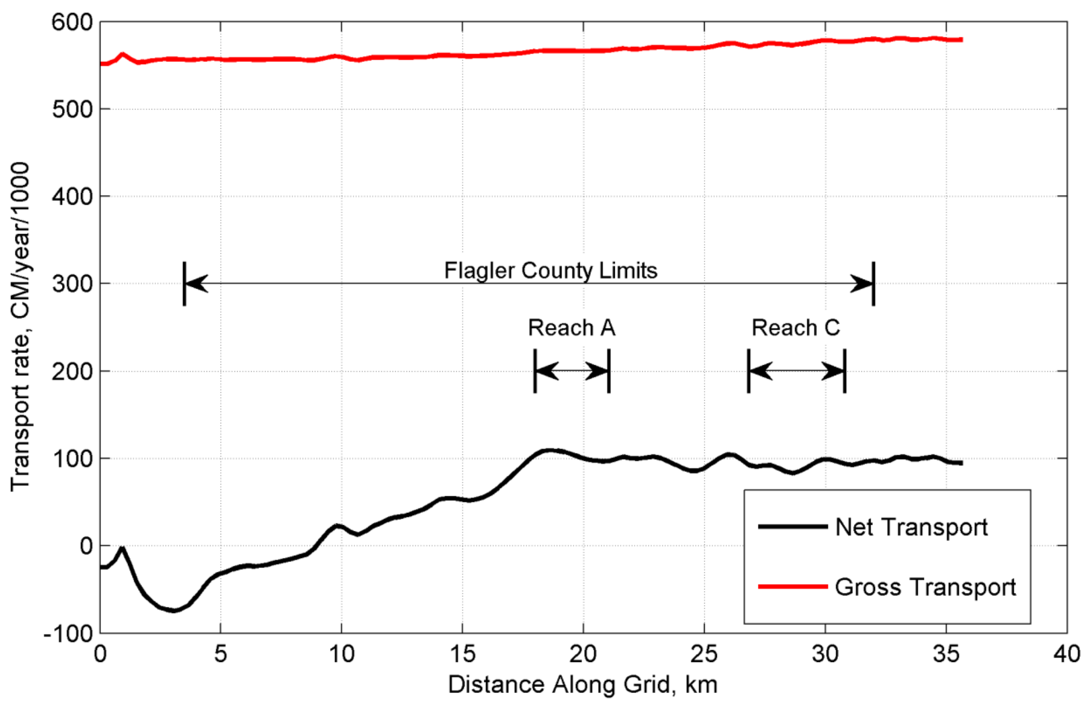

Figure A- 30. Final Calibration Longshore Sediment Transport

Model Results

The calibrated and verified GenCade model for Flagler County provides a means to test various proposed shoreline protection measures so that the most economically beneficial alternative can be determined. In order to compare proposed alternatives, an input shoreline condition with no modifications is first used to project the future background conditions. This model run is the future without-project condition. Then the various proposed alternatives are added to the initial input shoreline.

A beachfill is represented by extending the initial shoreline seaward by the amount specified for each alternative. For Flagler County, the input shoreline was extended seaward 10 feet to represent a dune only alternative, then 30 feet (10 foot dune + 20 foot berm), 50 feet (10 foot dune + 40 foot berm), 70 feet (10 foot dune + 60 foot berm) and 90 feet (10 foot dune + 80 foot berm) to represent a full beach nourishment template. The shoreline advances were applied to the areas that were determined to be favorable for a project to provide economic benefits (see Table A- 20, **Nourishment Design Templates**). The initial shoreline for all cases was based on the 1984 shoreline used during calibration. Figure A- 34 shows the initial model setup for both the with- and without-project conditions, focused on the areas where the nourishments are proposed and using the alternative with the maximum mean high water extension(70 feet).

The wave input for the production runs of project alternatives including the future without project condition was developed based on an average representative year. The entire 20 year wave dataset output by the CMS-Wave model was used as input into the GenCade model along with the 1984 shoreline. The average annual gross, net, northward, and southward transport outputs from the 20 years of the GenCade model period were amalgamated then averaged. The absolute relative difference of each year's different transport versus the 20-year mean was calculated and used to determine that 1991 best represented the average year. Ten year GenCade wave inputs were then developed by repeating the 1991 waves for each of the three stations used in the model simulations.

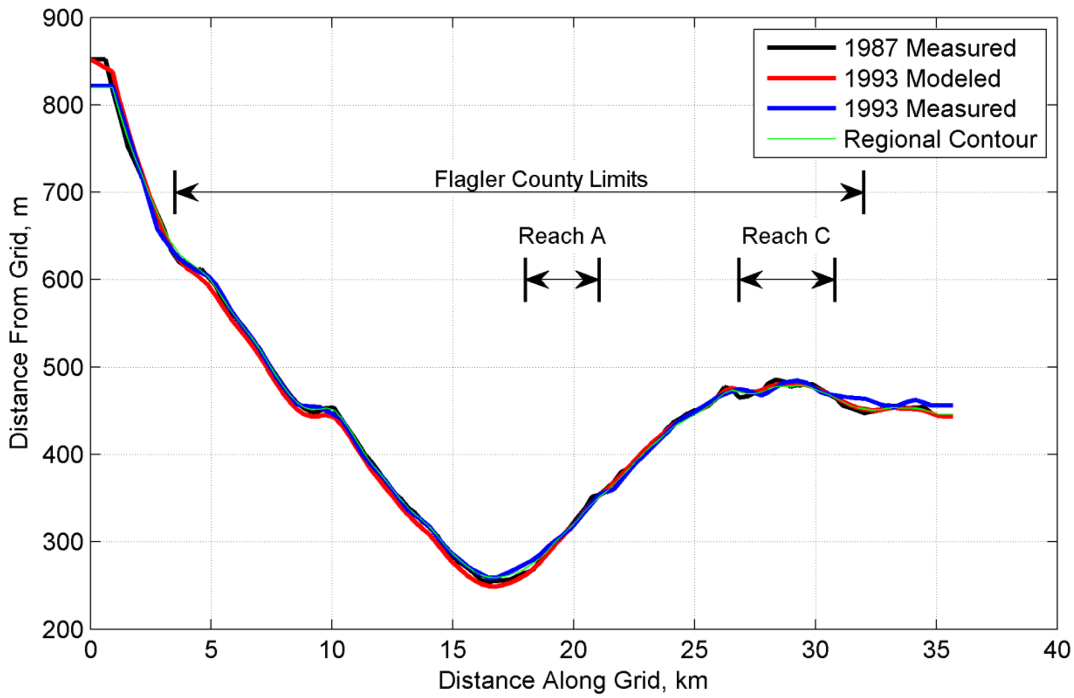

Figure A- 31. Verification Run Modeled and Measured Shoreline Positions

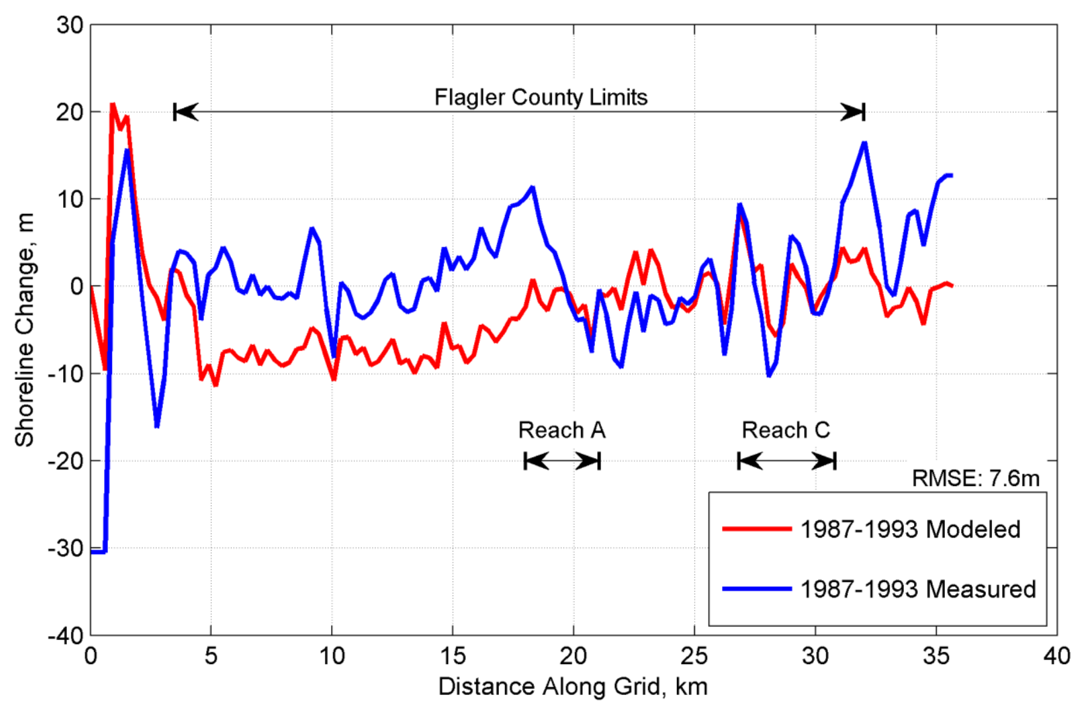

Figure A- 32. Verification Run Modeled and Measured Shoreline Change

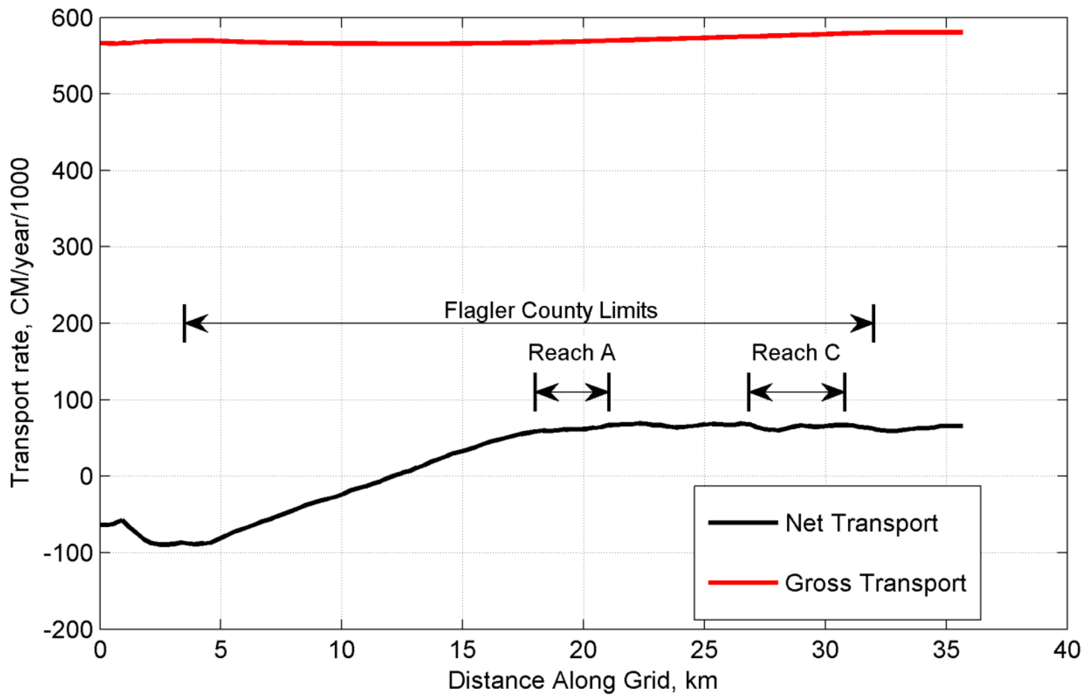

Figure A- 33. Verification Run Longshore Sediment Transport.

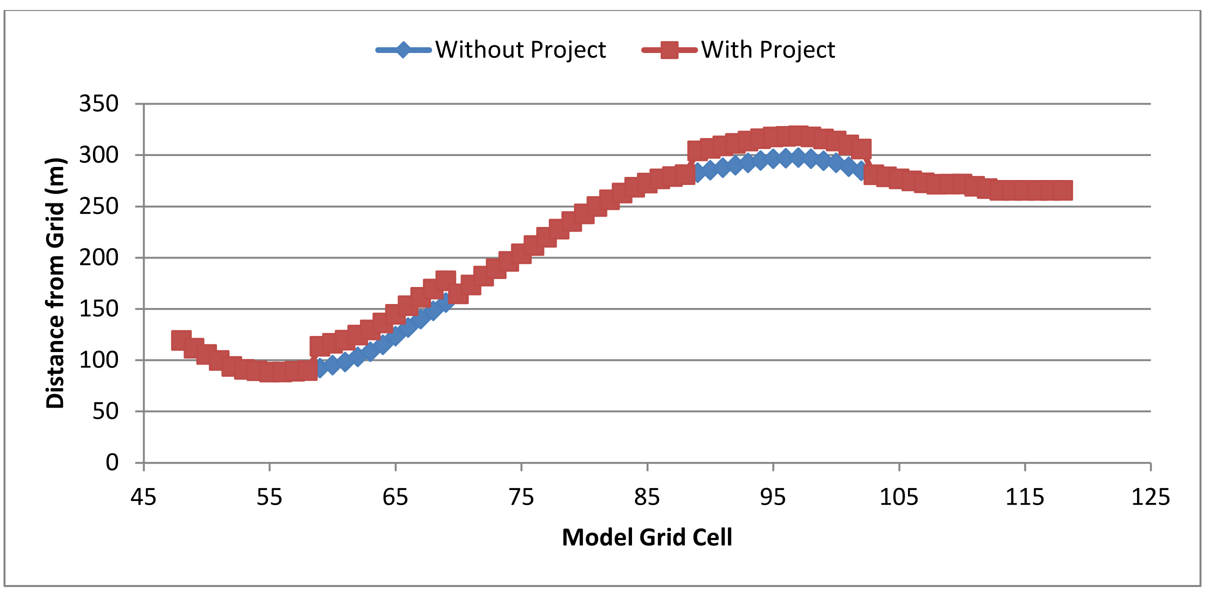

Figure A- 34. Initial model setup showing with and without project shorelines.

Future without Project Shoreline

The predicted shoreline changes without a nourishment project after a ten year simulation period is shown in Figure A- 35. Over the ten year period, the average change equaled -5.3 meters (-17.5 feet) or -0.5 m/yr (-1.75 ft/yr). The average values appear reasonable, but are greater than the historical average of -0.5 ft/yr. As previously discussed, the over-prediction of recession by the model was balanced with the amount of longshore sediment transport reported in the literature. The distribution as seen in Figure A- 35 is not uniform, giving rise to a standard deviation of 7.9 meters (25.9 feet). The area south of the embayment feature, from about 18 km south of the grid origin, shows mostly positive shoreline change over the 10 year simulation period, whereas the shoreline change for the area north of this point is largely negative.

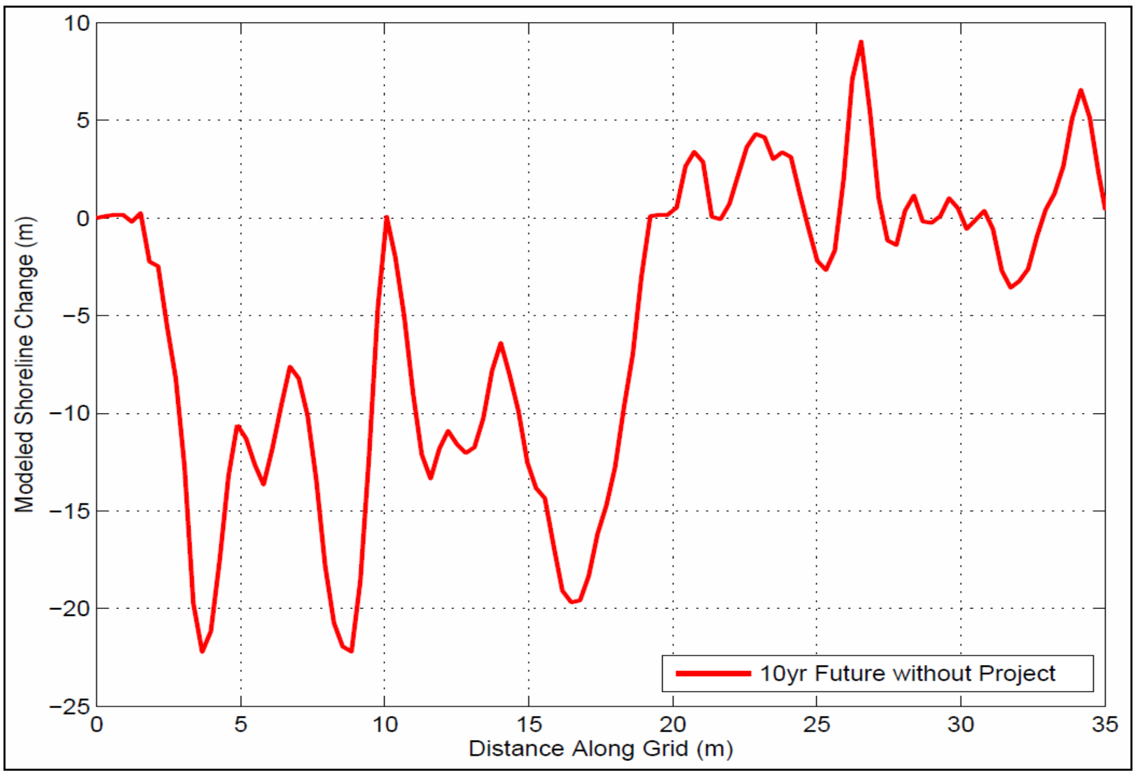

Figure A- 35. Future Without Project Predicted Shoreline Change

Future with Project Shoreline

The future with-project simulations featured a dune only option, represented by a 10 foot extension of the MHW shoreline, as well as dune+berm beachfill options of varying widths (30, 50, and 70 feet). The changes after 9 years of simulations were compared with the changes observed from the future without project run in order to determine the planform shoreline change rates that would be expected from a constructed project. Figure A- 36 and Figure A- 37 show two example model runs. Figure A- 36 shows shoreline change rates for fill alternatives in both Reaches A and C. Figure A- 37 provides the same results, but only the option to fill Reach C was examined. Similar results were obtained for all of the design alternatives that were carried through the full economic analysis of this study.

The greatest rates of shoreline change for the future with-project versus the future without project are found at the lateral boundaries of the beach fills. The change within the fill areas are shown as losses as the project equilibrates with the surrounding beaches and the nourishment material spreads laterally through diffusion. The beaches immediately adjacent to the fill areas receive the lost nourishment material resulting in positive shoreline change. Of the two examples shown, the greatest losses were predicted for Reach A and equaled more than 4 ft/yr. Reach C losses were about 3.75 ft/yr, so given the accuracy of the model, the change rates are essentially equal.

The project induced shoreline change rates calculated by Gencade and shown graphically in Figure A- 36 and Figure A- 37, do not take into account the improved performance of beach nourishments projects that comes with project maturation. That is, theory and beach nourishment experience has shown that dispersion losses at a beach nourishment project tend to decrease with the number of project nourishments. Based on the behavior of previous storm damage reduction projects along the east coast of Florida, it is assumed for the sake of this study that there will be a 20% reduction in shoreline change rates following each consecutive renourishment cycle.

Given that the final selected project alternative is a dune only feature, the over-prediction of average shoreline change with the selected calibration parameters has much less influence on the project economics than that of the greatest beach fill alternative. Further, the average change predicted by the model for the dune only alternative is very near the historical change rate of -0.5 ft/yr as seen in Figure A- 36 and Figure A- 37.

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 recovered 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 Flagler County.

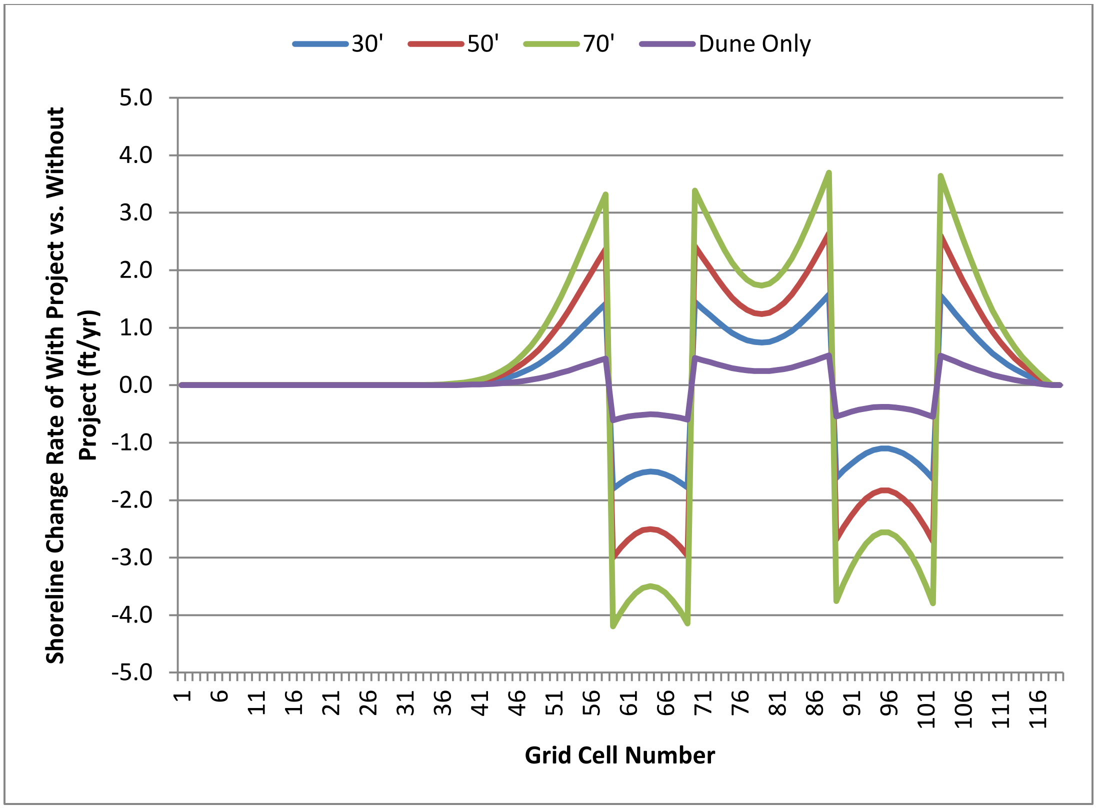

Figure A- 36. Modeled Shoreline Recession Rates for Reach A and C Nourishments

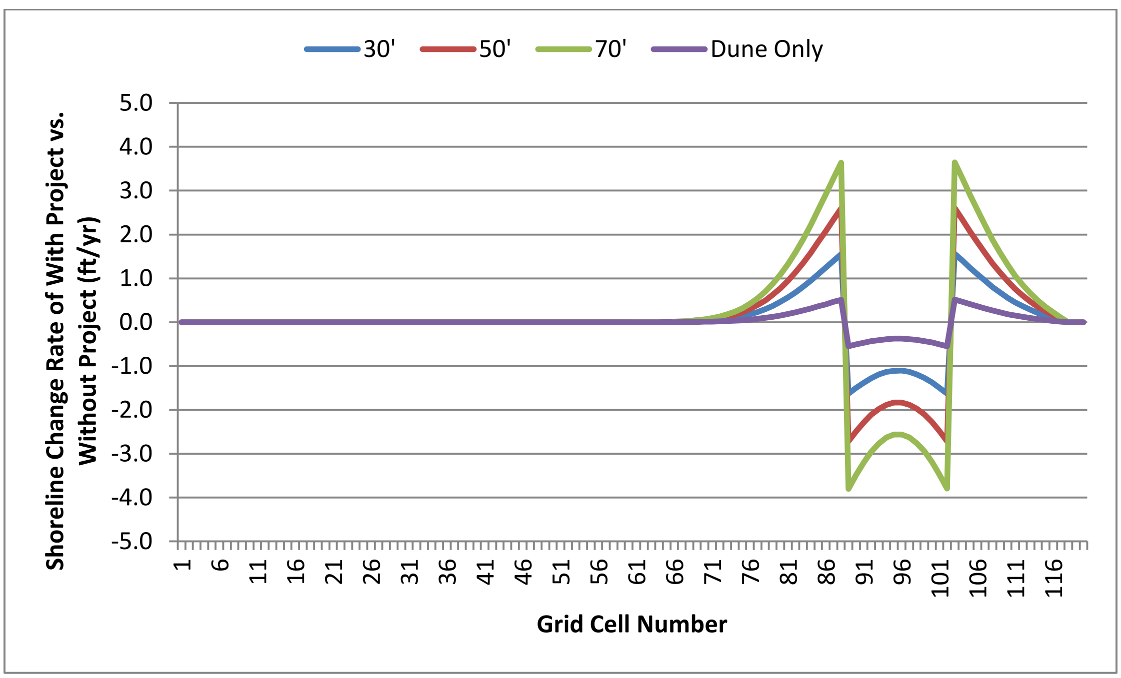

Figure A- 37. Modeled Shoreline Recession Rates for Reach C Nourishment

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 and planned nourishment.

Emergency Nourishment

Emergency nourishments are generally limited beach fill projects conducted by local governments in response to storm damage. Flagler 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 Flagler Beach-*fx* analysis.

Planned Nourishment

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, 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.

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 Flagler County, each model reach level template was developed based on a 10-foot extension of the dune and beach profile (to depth of closure) and four berm widths: 20-foot, 40-foot, 60-foot, and 80-foot. Template dune heights in each case were set to the elevation of the existing Beach-*fx* profile. Design templates were developed for each profile within the four viable design reaches, Reach A, Reach B, Reach C, and Reach D. Note that the Marineland design reach was previously eliminated from further study. Table A- 20 provides dimensions for each of the design templates.

Table A- 20. Flagler County Beach-*fx* Nourishment Design Templates

Design Reach	Beach-fx model Reaches	Beach-fx Profiles	Template Dune Height (ft-NAVD88)	Template Dune Width (ft)	Template Berm Width (ft)				
Reach A	RA-1 to RA-10	RWAP3	20	110	0	20	40	60	80
		RWAP4	19	110	0	20	40	60	80
Reach B	RB-1 to RB-17	RWAP4	19	110	0	20	40	60	80
		RWAP5	20	110	0	20	40	60	80
Reach C	RC-1 to RC-14	RWAP4	19	110	0	20	40	60	80
		RWAP6	19	50	0	20	40	60	80
Reach D	RD-1 to RD-5	RWAP4	19	110	0	20	40	60	80
		RWAP6	19	50	0	20	40	60	80

Nourishment Triggers and Mobilization Threshold

Beach-*fx* planned nourishment design templates have three nourishment triggers (1) berm width, (2) dune width, and (3) 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 dune and profile extension (with berm). Triggers were set at 0.0, 0.91, and 0.9, respectively, for alternatives which included only an extension of the dune and beach profile. The mobilization threshold for all planned nourishment alternatives was specified as 300,000 cubic yards. Nourishment triggers and mobilization threshold are subjective parameters, based on local infrastructure and engineering judgement of when renourishment will become essential to the continued performance of the project. Application of these parameters for Flagler County 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 Flagler County, alternatives were developed by combining the design reaches and nourishment templates discussed previously (Table A- 20). Preliminary Beach-*fx* runs, along with assessment of potential benefits, 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 final array of alternatives run with Beach-fx is presented in Table A- 21. These consist of two basic designs, a 10 foot hydraulic extension of the existing dune and beach profile (to depth of closure) and a 10 foot extension of the existing dune and beach profile combined with a 20 foot berm. No changes to the existing dune height were included.

Table A- 21. Final Array of Design Alternatives

Design Alternative	Description	Dune Height Extension (ft)	Dune Width Extension (ft)	Berm Width Extension (ft)
Reach A duneH	10’ extension of ReachA existing dune and beach profile	0	10	0
Reach A 30	10’ extension of ReachA existing dune and beach profile with a 20’ berm	0	10	20
Reach B duneH	10’ extension of ReachB existing dune and beach profile	0	10	0
Reach B 30	10’ extension of ReachB existing dune and beach profile with a 20’ berm	0	10	20
Reach C duneH	10’ extension of ReachC existing dune and beach profile	0	10	0
Reach C 30	10’ extension of ReachC existing dune and beach profile with a 20’ berm	0	10	20
Reach AC duneH	10’ extension of ReachA + ReachC existing dune and beach profile	0	10	0
Reach AC 30	10’ extension of ReachA + ReachC existing dune and beach profile with a 20’ berm	0	10	20

Protective Beach Design

Based on Beach-fx model results and economic evaluation, project alternative "ReachCduneH" (10' extension of ReachC existing dune and beach profile) was identified as the Recommended Plan for nourishment of Flagler County. A description of this shore protection plan is provided in the following sections.

Project Length

Nine miles of shoreline, extending from FDEP monument R-50 to R-100, within Flagler County was considered during project evaluation using Beach-*fx*. Note that this does not include the Marineland segment of the project (R-1 to R-4), which was removed early in the study. The selected design, ReachCduneH covers approximately 2.6 miles of the study area. The beach fill will be placed from R-80 to R-94 with tapers extending approximately 100 feet to the north of R-80 and approximately 100 feet to the south of R-94.

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 19 and 20 ft-NAVD88. Evaluation of the design alternatives has shown that the existing elevations, when combined with berm and/or dune extension, provide sufficient protection. No additional elevation is included in the selected design plan. For Reach C, the dune elevation is 19 ft-NAVD88.

Existing dune widths in the project area are variable. Between R-80 and R-88 the dune has an average width of approximately 100 feet. Between R-88 and R-94 the average width is approximately 40 feet. It should be noted that State Road A1A, which runs parallel to the project shoreline, is located within the dune. Based on the average dune widths, design widths for the 10 foot dune and profile extension alternative Reach C duneH are, therefore, 110 feet in the northern portion of Reach C and 50 feet in the southern portion.

Project Berm

The design berm elevation in Reach C is 11 ft-NAVD88, which approximates the natural berm elevation. Restricting the design berm elevation to the natural berm elevation minimizes scarping of the beachfill as it undergoes readjustment. Vertical scarps can hinder beach access by nesting sea turtles, and may also pose safety problems related to recreational beach use. Other reasons for mimicking 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.

Although the design berm for Reach C duneH is technically a 0 foot extension, construction of the dune and profile extension will result in an increase in the existing berm (see Project Construction). Figure A-38 shows a graphical representation of the Reach C duneH design alternative (110 foot dune template) as modeled by Beach-*fx*.

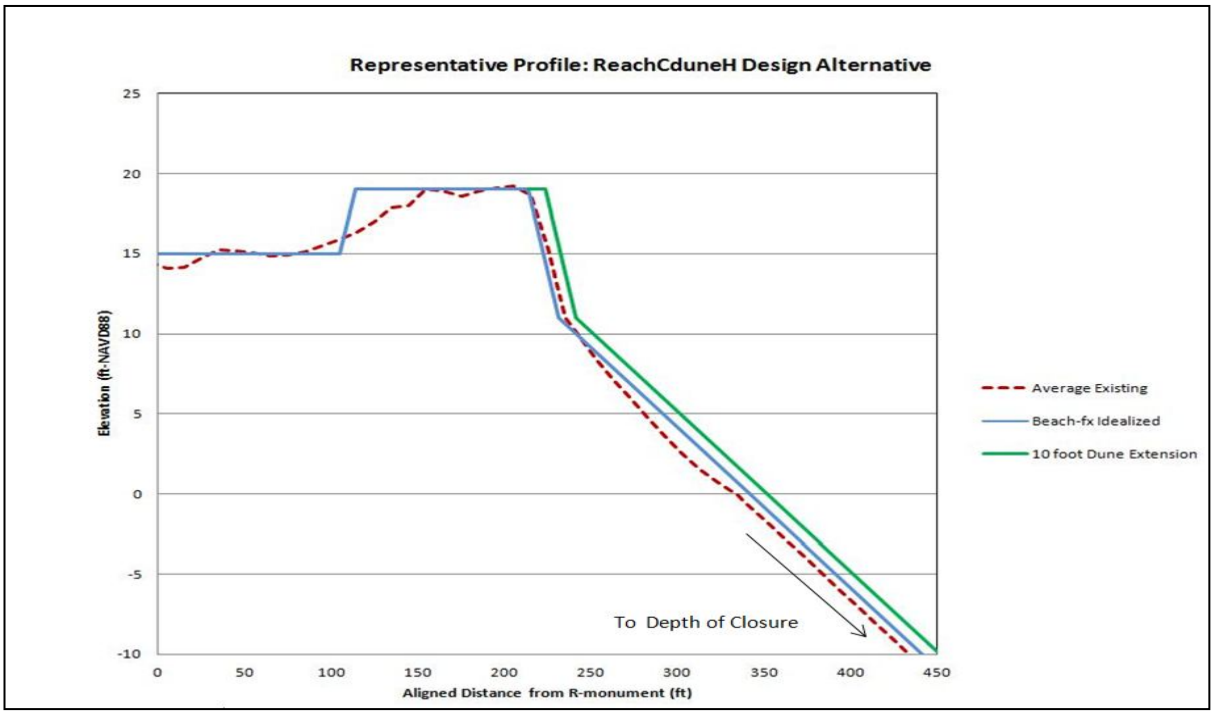

Figure A- 38. Graphical Representation of Reach C Dune Extension Alternative Beach-*fx* Profile

<u>Project Beach Slopes</u>

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 Flagler County, the native beach slopes in Reach C are estimated as a 1 (vertical) on 2.2 (horizontal) at the dune, 1 on 10 from the berm to MLW (-3.1 ft-NAVD88), and 1 on 40 to 1 on 70 below MLW. 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. Sand from the project borrow site was determined to be a near match to the gradation and shell content of the existing beach. This will allow the beach fill to equilibrate to a shape similar to the existing profile. Below the 12 foot depth contour, various bar type features appear in the profiles, making a representative slope difficult to determine.

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, beachfill 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 will dissipate through erosion 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 10 foot dune and profile extension, Figure A-38). Each life-cycle simulation then applies randomly generated storms, storm erosion, and natural background 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 renourishment event is initiated. 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 – existing dune plus fill extension - 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 the allowable erosion be exceeded in one or more reaches, then Beach-fx computes the volume required (over all of the *triggered* nourishment reaches) required to fill the original design template and compares that volume to the mobilization threshold. If the mobilization threshold is exceeded a renourishment over *all* planned nourishment reaches occurs and the model continues through the remainder of the life-cycle.

For Flagler County alternative Reach C duneH, the berm width, dune width, and dune height planned nourishment triggers were set at 0, 0.91, and 0.9, respectively. The mobilization threshold was set to 300,000 cubic yards. 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 threshold volume was the dominant parameter for optimizing project cost for an alternative in which the berm width has a zero value. Employing 25,000 cubic yard increments, a mobilization threshold of 300,000 was found to be (when combined with the above nourishment triggers), the most optimal threshold value. Decreasing the threshold, decreased the benefit to cost ratio. Increasing the threshold to 325,000 cubic yards produced a small increase in the benefit to cost ratio. However, it also allowed segments of dune to erode to beyond the existing project condition. This was not considered to be an acceptable assumption.

Each complete Beach-fx model run consists of 100 iterations, each iteration representing the life of the project. Based on the Reach C dune H 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- 22 provides the minimum, maximum, and average fill volumes for both initial and renourishment events over the life of the project. This table also provides the number of expected renourishment events.

Table A- 22. Project Volumes

Project Volumes (Averaged over 100 Beach-fx Life-cycle Iterations)				
Sea Level Rise Case	Volume Description	Initial Fill Volume (cy)	No. of Renourish. Events*	Average Volume per Interval (cy)
Base	Min - Max	300,000 – 370,000	4	300,000 – 350,000
	Average	330,000		320,000
Intermediate	Min - Max	300,000 – 370,000	5	300,000 – 350,000
	Average	330,000		320,000
High	Min - Max	350,000 – 410,000	8	310,000 – 370,000
	Average	370,000		330,000
*Due to its probabilistic nature, Beach-*fx* can result in a range of required renourishment events. However, for the Flagler County Recommended Plan (a relatively modest extension of the dune and profile), the minimum and maximum number of events was the same.				

Project Construction

The Recommended Plan for Flagler County results in a 10 foot seaward extension of the existing dune and beach profile out to depth of closure. Due to erosion, armor damage, and intermittent repairs and maintenance, the project shoreline does not presently have a smooth, consistent dune feature. In order to ensure that the nourishment project provides the maximum benefit, it is necessary to first establish a smooth, relatively straight base construction line that will allow the project to perform as predicted during the Beach-*fx* shoreline analysis.

In order to establish the project construction line, State Road A1A, which runs roughly parallel to the project shoreline, was identified as a reliable land based reference for developing a smooth, consistent project dune. The seaward crest of the dune was then identified as the shoreline profile reference point. Based on historical surveys, it was determined that the average distance between the eastern edge of A1A and seaward crest of dune (as measure at each FDEP R-monument) in Reach C is 20 feet. Therefore, the base construction line (defined as the “existing” seaward crest of the dune) is designated to be 20 feet east of, and parallel to, State Road A1A. The project shoreline would then add an additional 10 feet of width to the base construction line (“existing” dune). Figure A- 39 shows graphically the location of the measured (2011 survey), “existing”, and project dunes relative to the eastern edge of A1A. Note that this approach will ultimately provide a consistent level of protection to the road, which is the primary damageable infrastructure nearest the project.

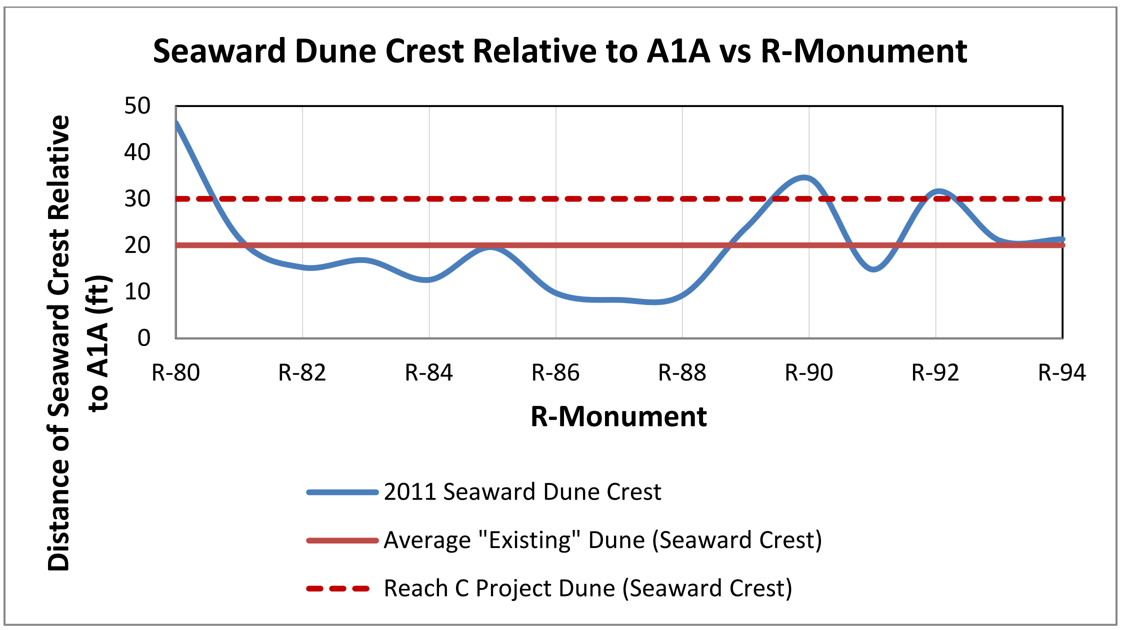

Figure A- 39. Measured and Design Dune Locations Relative to State Road A1A

Beach-*fx* estimates that initial construction of the Reach, C 10 foot dune and profile extension will require between 300,000 and 370,000 cubic yards of material. Using the 2011 survey (the most recently available reference), the designated construction line, and the project (10 foot dune and profile extension) design template, it was determined that the volume required for initial construction would be approximately 360,000 cubic yards. While this is above the Beach-*fx* average initial volume of 330,000 cubic yards, it is within 10% of the modeled values and is considered reasonable. Therefore, this volume is considered to be appropriate verification of the location of the base construction line and the validity of the project template. Because this volume is based on a conceptual layout and survey information that will be updated prior to construction, it will be used only for verification of the design dimensions and will not be used for cost estimating. Costs will continue to be based on average Beach-*fx* volumes.

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 beachfill, 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.

Based on the estimated initial fill volume, constructability considerations, and existing (2011) shoreline dimensions, a construction template applicable to Reach C was determined. The construction template (shown in Figure A- 40) consists of a 10 foot wide dune extension with a 1 on 3 slope, a 35.0 foot berm with a 1 on 100 slope, and foreshore fill extending to approximately -2 ft-NAVD88 with a slope of 1 on 5. This template, dimensioned for constructability, will then equilibrate into the project (10' dune and profile extension) template. The volume of material in the equilibrated profile (between the template and the "existing" condition) represents the material that is expected to erode between successive nourishment events.

Dune Walkovers

The Reach C project length (R-80 to R-94) contains forty-two dune walkovers. Walkovers are a mixture of twenty-one privately owned and twenty-one local government owned structures. On average, the structures are in good condition. Each crosses the dune within the project area and potentially will require replacement due to placement of the initial project fill. One time replacement costs of the twenty-one public structures is included in the total project cost. If needed, replacement of the remaining, privately owned structures will be a local responsibility to be covered in the perpetual storm damage reduction easement acquired by the Non-Federal Sponsor. Although the existing structures range from basic (Figure A- 40) to relatively elaborate (Figure A- 41), for feasibility level design and cost estimating purposes, a single dune walkover design (closely approximating the construction of existing public walkovers) is applied to all replacements (Figure A- 42). It should be noted that modification of this design may occur during the detailed design phase of the study.

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 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- 43 the entire 10 foot dune and beach profile 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”).

Figure A- 40. Dune Walkover – Basic

Figure A- 41. Dune Walkover – Elaborate

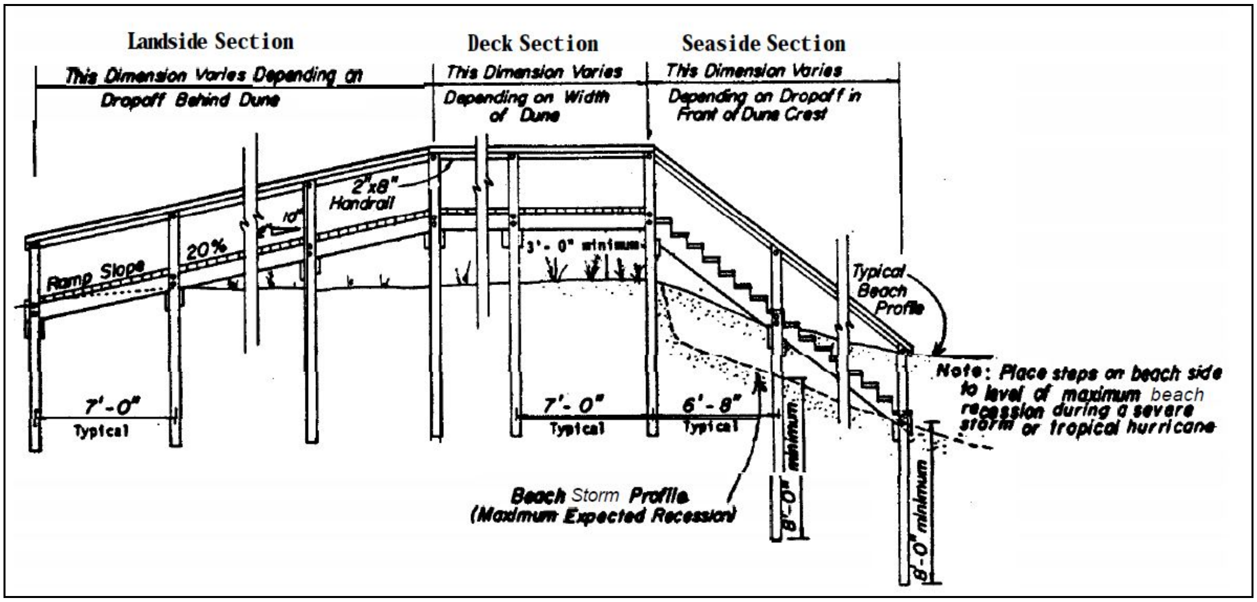

Figure A- 42. Dune Walkover – Feasibility Level Design Cross-section

Assessing the performance of the project fill now has two stages. First, a survey of the project area (such as a monitoring or post-storm survey) will be assessed to determine if the seaward crest of the dune at any of the R-monument locations within the project have receded past the Base Construction Line (Figure A- 43). If recession has occurred at one or more of the R-monuments, then a summation of the volume required to restore those profiles to the original construction template will be made. If the total volume required to restore the receded profiles exceeds the threshold volume, then a renourishment event is recommended. The decision to renourish may then be made based on traditional concerns, including such factors as budget cycle and available funding.

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 project life. 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 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 cost shared total project cost.

Other monitoring efforts include bathymetric mapping of the borrow site, which will be done as part of the pre-construction engineering and design (PED) phase prior to each nourishment.

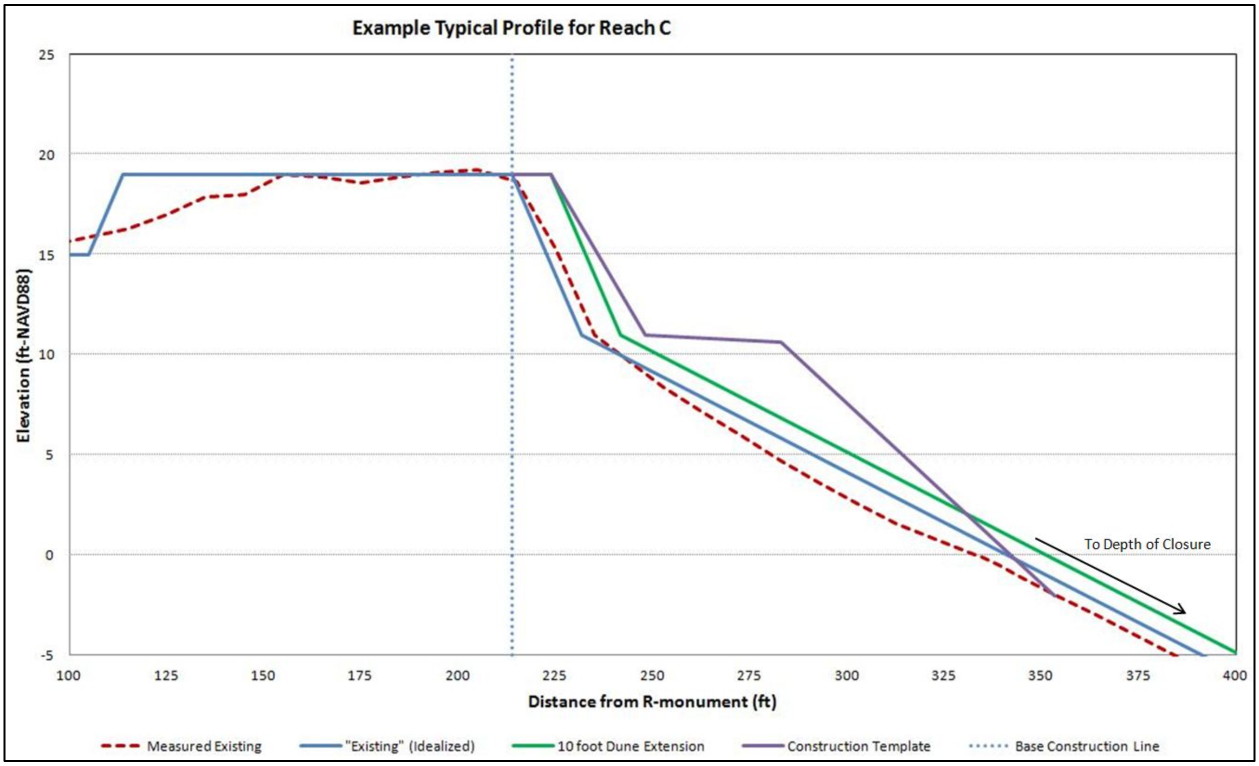

Figure A- 43. Typical Profile Sketch, Recommended Plan

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 of high SLR case is required.

Summary

This appendix summarizes the engineering design of a shore protection project proposed for construction in Flagler County, Florida. The project consists of beach nourishment/renouishment along approximately 2.6 miles of shoreline between FDEP monuments R-80 to R-94. The design beachfill template is characterized by a 10-foot extension of the 2008 profile from dune to depth of closure. The total expected volume of beachfill material required under the Base SLR case ranges from 1,500,000 to 1,770,000 cubic yards (average of 1,600,000 cubic yards). This includes 300,000 to 370,000 cubic yards (average of 330,000 cubic yards) for construction of the design beach profile. Total cost of initial project construction is estimated at $14,182,000. Those costs would include the plans and specifications surveys of the project area and borrow site for construction, and the cost of a volumetric survey after initial construction for payment. Future renourishment costs are estimated at $7,717,000 per nourishment, with periodic nourishment expected at approximately 11 year intervals. Assuming that the Base SLR case applies, an estimate of the total cost incurred over the 50-year project life is $44,962,000.

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