Appendix C1 – Hydrology & Hydraulics

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US Army Corps of Engineers
Philadelphia District

NEW JERSEY BENEFICIAL USE OF DREDGED MATERIAL FOR THE DELAWARE RIVER

FEASIBILITY REPORT AND INTEGRATED ENVIRONMENTAL ASSESSMENT

APPENDIX C - HYDROLOGY, HYDRAULICS, AND COASTAL

October 2017

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1.0 INTRODUCTION

1.1 OVERVIEW OF APPENDIX

This Hydrology, Hydraulics, and Coastal Appendix provides an overview of the analyses supporting the New Jersey Dredge Material Utilization (DMU) Coastal Storm Risk Management Feasibility Study. The majority of Appendix focuses on the coastal engineering analyses conducted in support of the beach restoration alternative evaluation and Beach-fx modeling effort.

1.2 STUDY AREA

The study area is located within the section of the Delaware River watershed, which lies within the State of New Jersey and the Delaware River itself. The north/south boundaries of the study area extend from Trenton, NJ to Cape May Point, NJ (Figure 1). The centerline of the Delaware River and Bay represents the western study area boundary and it extends approximately 135 miles from the Atlantic Ocean upstream to the head of tide at Trenton, New Jersey.

For the purposes of CSRM, the study area not only includes flood prone areas along the mainstem Delaware River and Delaware Bay, but also the tributaries of the Delaware which contribute to both tidal and fluvial flooding. Tributaries to the Delaware River and Bay within the study area include: Dennis Creek, Maurice River, Cohansey River, Stowe Creek, Alloway Creek, Salem River, Oldmans Creek, Raccoon Creek, Mantua Creek, Big Timber Creek, Cooper River, Pennsauken Creek, Rancocas Creek and Black Creek.

This feasibility study evaluated coastal storm-related damages in New Jersey occurring in two defined planning reaches within the Delaware River/Bay system. The “northern reach” is from the head of tide at Trenton, NJ down to the approximate river/bay boundary (around Alder Cove), while the “southern reach” extends south from the Alder Cove area (river/bay boundary) to the mouth of the Delaware Bay at Cape May Point, NJ.

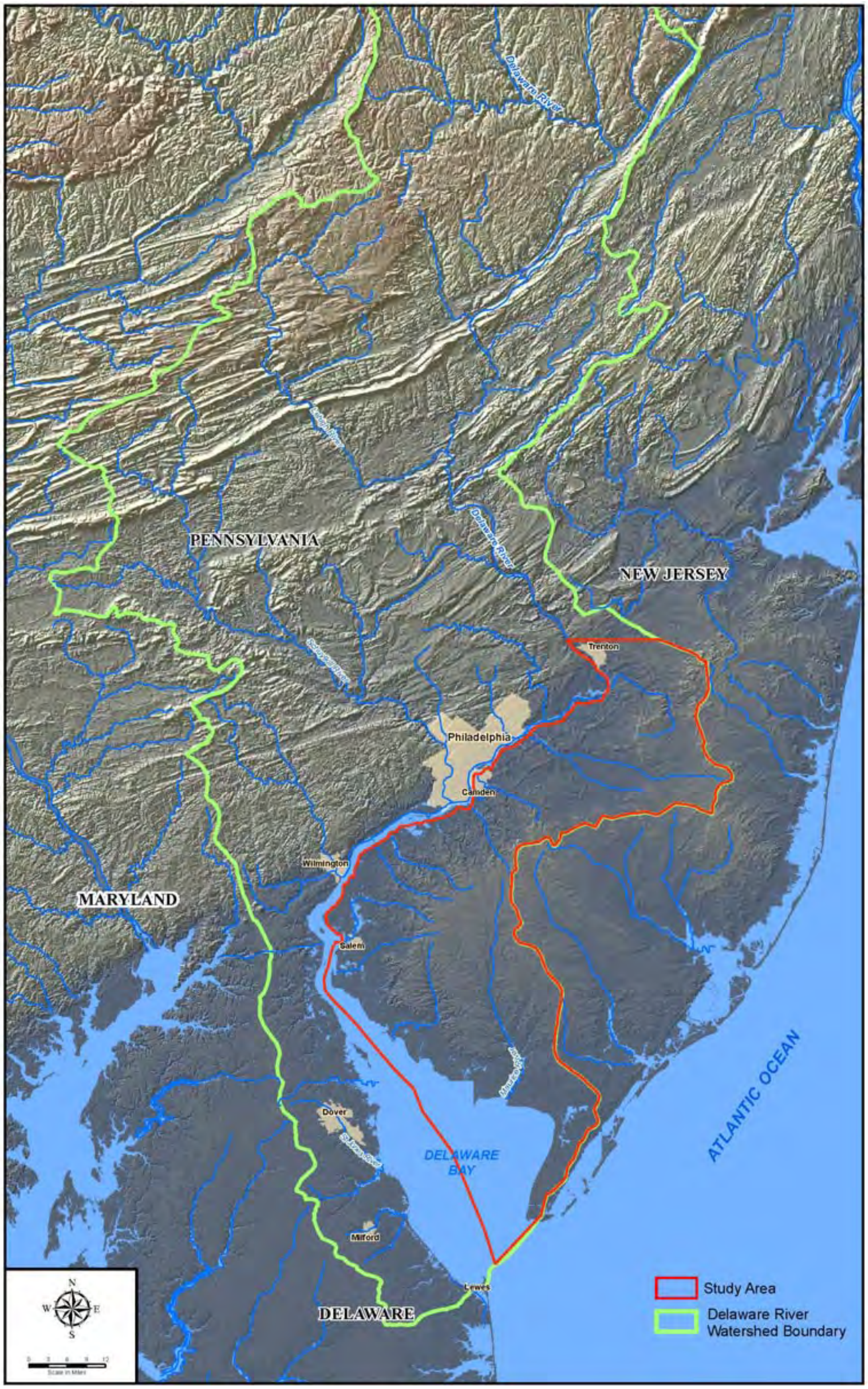


Figure 1: Study Area

2.0 SCREENING LEVEL ASSESSMENT

2.1 SCREENING LEVEL STAGE-PROBABILITY DATA

For the northern reach, stage-probability data for each of the DMU project sites were obtained directly from U.S. Army Engineer Research and Development Center (ERDC) in November 2015. These data were compiled by ERDC from the North Atlantic Coast Comprehensive Study (NACCS) results, originally finalized in January 2015, but subsequently updated to incorporate model refinements, and new data as they became available. Additional information regarding the NACCS modeling study is provided in Section 3.5.

NACCS modeling output supplied by ERDC was reported at each Save Point, which is a point in the modeled area at which results such as water surface elevation, wave height, etc., are saved, for a total of 18,977 discrete locations throughout the NACCS study area. These data were provided in both spreadsheet form, and as Google Earth KML format for use in GIS systems. As the NACCS numerical modeling utilized a coupled surge and wave model (ADCIRC + STWAVE), and for the results utilized for screening (Base+Tides conditions), reported water levels explicitly accounted for effects of storm surge, wave setup, and tides, but required incorporation of actual wave height effects (i.e. wave crest elevations). As such, two separate data sets were supplied by ERDC: one for static water level or stillwater elevation (SWEL), and one for wave height, again reported at each model node in the NACCS study area. Both data sets were supplied at various average recurrence intervals (ARI) from 1- to 10,000-yr, with the mean (average) value reported, including multiple upper confidence limits (84th, 90th, 95th percentile, etc.). For later incorporation into HEC-FDA, conversion from ARI to annual exceedance probability (AEP) was completed using the reciprocal (e.g. 2-yr ARI = 1/2, or an AEP of 0.5, or 50% annual chance exceedance (ACE)). NACCS model results were originally supplied in metric units, and were subsequently converted from meters, MSL to feet, NAVD88 through conversion values provided by ERDC.

Following data conversion, one half (0.5) the wave height was added linearly to the SWEL to account for wave effects, resulting in the wave crest elevation, or total water level (TWL), at each model save point, again across various ARI, and multiple confidence intervals. The one half (0.5) fraction is an approximation based on the simplifying assumption of linear wave theory. Wave height is the difference in elevation between the wave crest and wave trough. In linear wave theory, the total wave height (crest to trough) is vertically symmetrical about the still water level that is, the wave crest is ½ the of the wave height above the still water level. This was deemed sufficiently detailed for screening level decisions.

For each study location within the northern reach, multiple proximate save points (typically 3 to 5) were compiled. SWEL, wave height, and TWL data for compiled save points were plotted and reviewed to determine a representative save point at each study location. Additionally, as uncertainty varied spatially throughout the NACCS modeling domain, ERDC also provided estimates of epistemic uncertainty for each save point, to further qualify confidence in the model results, allowing screening of save points for use at each of the DMU study locations. In general, stage-probability data varied only slightly across each individual study location, and as such it was determined that data from a single representative save point was sufficient to describe anticipated water levels at each study location, to inform project screening. In total, two base stage-probability curves were determined for each study location: SWEL, and SWEL + ½ Wave

Height, each reported with the mean and multiple confidence limits. Figure 2 below depicts example location with NACCS Save Points used in screening assessment, with Table 1 showing all NACCS Save Points used during screening, by location. Figure 3 shows an example of output data from NACCS analysis for one location.

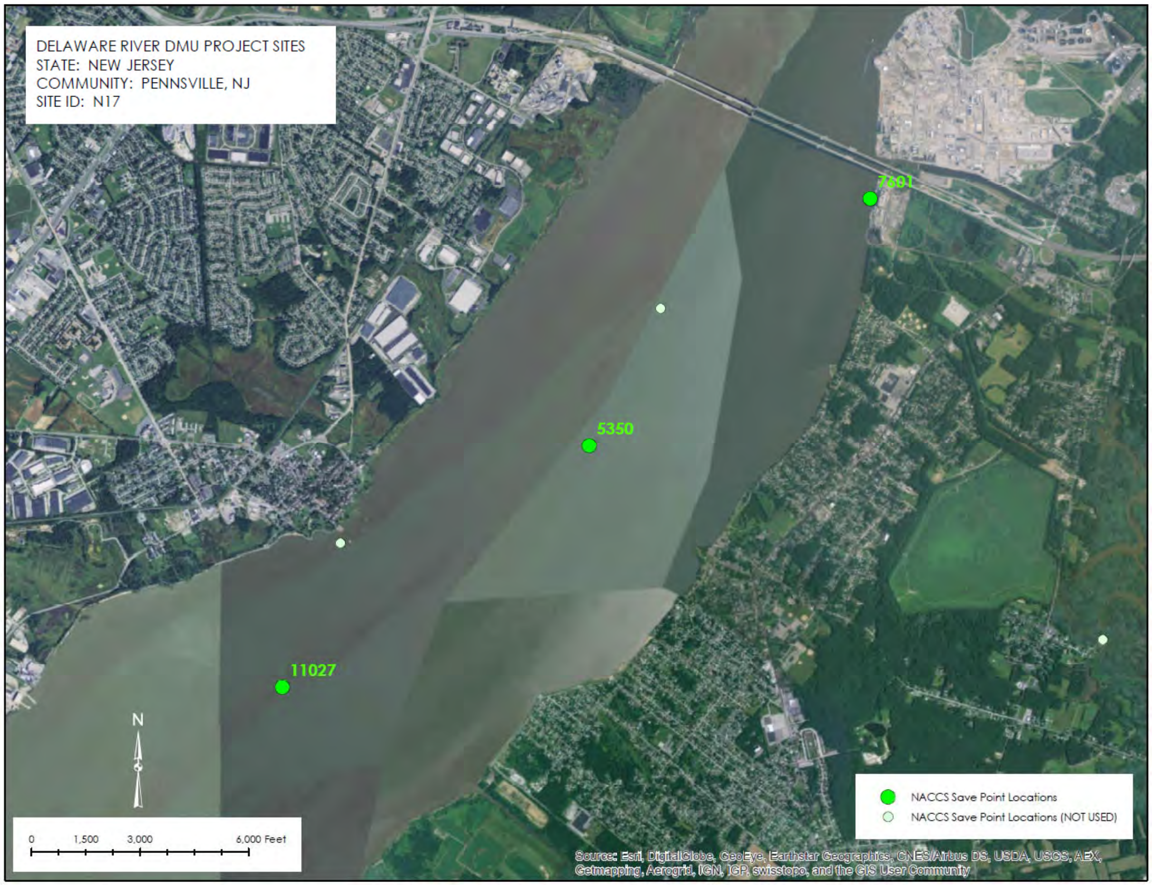


Figure 2: Example NACCS Save Point Map

Table 1: NACCS Save Points Used for Initial Screening

Site ID	N15	N17	N25	N26	N27	N28	N33
Location / Municipality	Penns Grove	Pennsville	Bivalve	Shellpile	Port Norris	Maurice River	Villas
NACCS Save Point ID	11109	13295	13403	13403	13403	11185	15258
	11030	5349	13402	13402	13402	11192	11168
	11100	11102	13404	13404	13404	11184	13425
	13322	11024	13396	13396	13396	13409	11169
	5351	7601	11191	11191	11191		15268
		11028	11185	11185	11185		11205
		5350					
		7158					
		11027					
		7600					
		7599					
		11112					

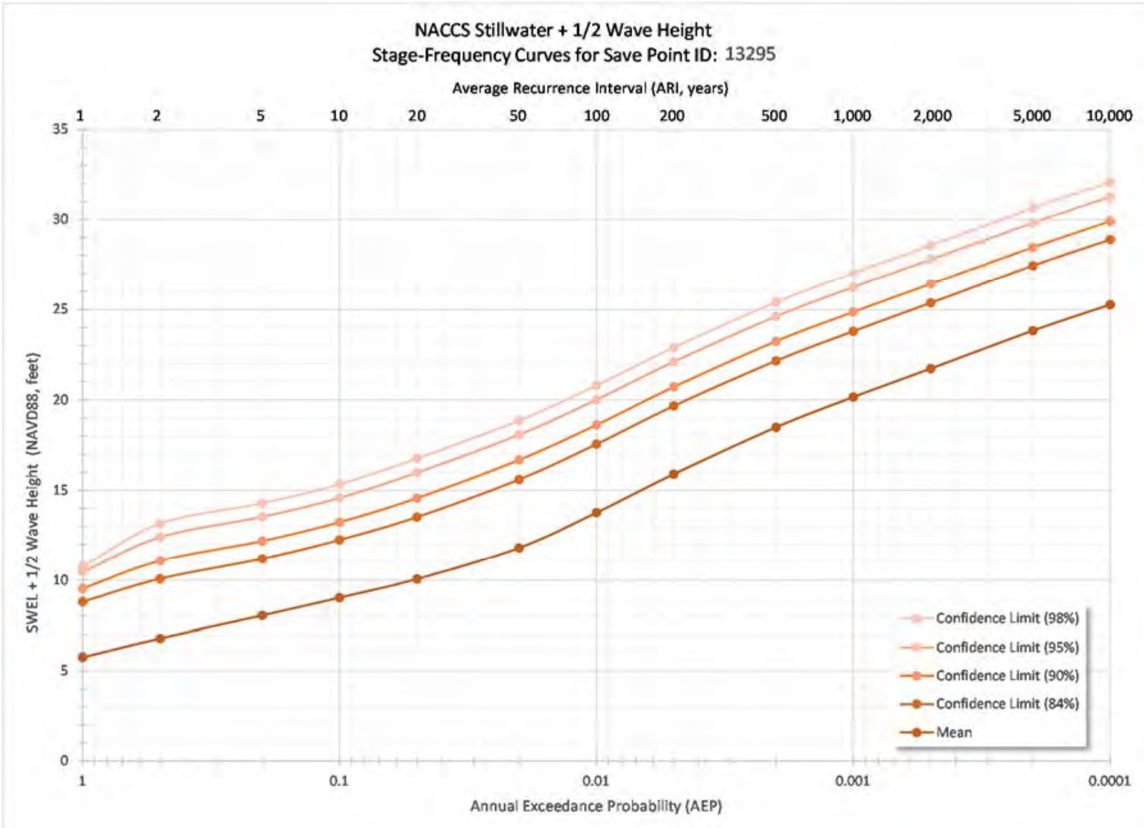


Figure 3: Example NACCS Save Point Output compiled for screening (note: confidence limits shown convey epistemic uncertainty only, not sampling uncertainty)

2.2 SCREENING LEVEL RELATIVE SEA LEVEL CHANGE (RSLC) ANALYSIS

In accordance with USACE ER 1100-2-8162, potential effects of RSLC on overall water levels were analyzed for each study location, over a 50-yr economic analysis period and a 100-yr planning horizon. Given the size and scope of potential projects, and associated anticipated timing, a base year for RSLC analysis of 2020 was used, with future years of 2070 and 2120. For each study location, the most appropriate NOAA gage (typically closest geographically) was determined, and RSLC adjustments were calculated for the future years using published RSLC rates, for the three recommended curves: USACE Low, USACE Intermediate, and USACE High. Table 2 summarizes the NOAA gage utilized for each study location. For screening purposes, these RSLC adjustments were added linearly to the base stage-probability curves discussed above, resulting in a total of eight stage probability curves compiled for each study location, again each with mean and multiple confidence limits for the economic analysis. These stage-probability curves are:

- Base year (2020) SWEL
- Future year (2070) SWEL + RSLC USACE Low
- Future year (2070) SWEL + RSLC USACE Intermediate
- Future year (2070) SWEL + RSLC USACE High

- Base year (2020) SWEL + 1/2 Wave Height
- Future year (2070) SWEL + 1/2 Wave Height + RSLC USACE Low
- Future year (2070) SWEL + 1/2 Wave Height + RSLC USACE Intermediate
- Future year (2070) SWEL + 1/2 Wave Height + RSLC USACE High

Given the anticipated size of any protection features, and negligible effects to stage of the tidal Delaware River and Bay, all stage-probability curves were utilized for both without and with-project conditions. As discussed above, at study locations where wave data was unreported, only SWEL curves were produced, for four total curves rather than eight.

Table 2: Nearest NOAA Gage used for Screening Level Sea Level Change Calculations

Site ID	Location / Municipality	Nearest NOAA Gage
N15	Penns Grove	8551910, Reedy Point, DE
N17	Pennsville	8551910, Reedy Point, DE
N25	Bivalve	8536110, Cape May, NJ
N26	Shellpile	8536110, Cape May, NJ
N27	Port Norris	8536110, Cape May, NJ
N28	Maurice River	8536110, Cape May, NJ
N33	Villas	8536110, Cape May, NJ

2.3 SCREENING LEVEL TOPOGRAPHIC REVIEW

Available topographic data and bathymetric data at each study location was compiled and reviewed in ArcGIS to further inform initial screening. Specifically, topographic-bathymetric combination (topobathy) LiDAR data from 2014 was available for the majority of the study area. This was supplemented with topographic LiDAR from 2009 where necessary for coverage of the entire floodplain for a few locations in the upper extent of the study area. All elevation data were reprojected, and converted as necessary, to horizontal datum of State Plane New Jersey, NAD83, feet, and a vertical datum of NAVD88, feet, for consistent use with the NACCS stage-probability curves.

At each study location, ArcGIS was utilized to cut profiles, laid out perpendicular to the shoreline. Multiple profiles were utilized at each location to estimate existing level of protection, continuity of protection features, as well as potential impacts of with-project features. Topography at each location was also reviewed to qualitatively assess potential incremental benefits to increasing level of protection. Further, profiles were utilized for feasibility level quantity estimates of with-project conditions at each study location. FEMA Flood Insurance Rate Maps were also utilized to inform initial screening. Figure 4 shows an example of topographic profile placement and Table 3 below summarizes estimated level of existing protection at each of the study locations.

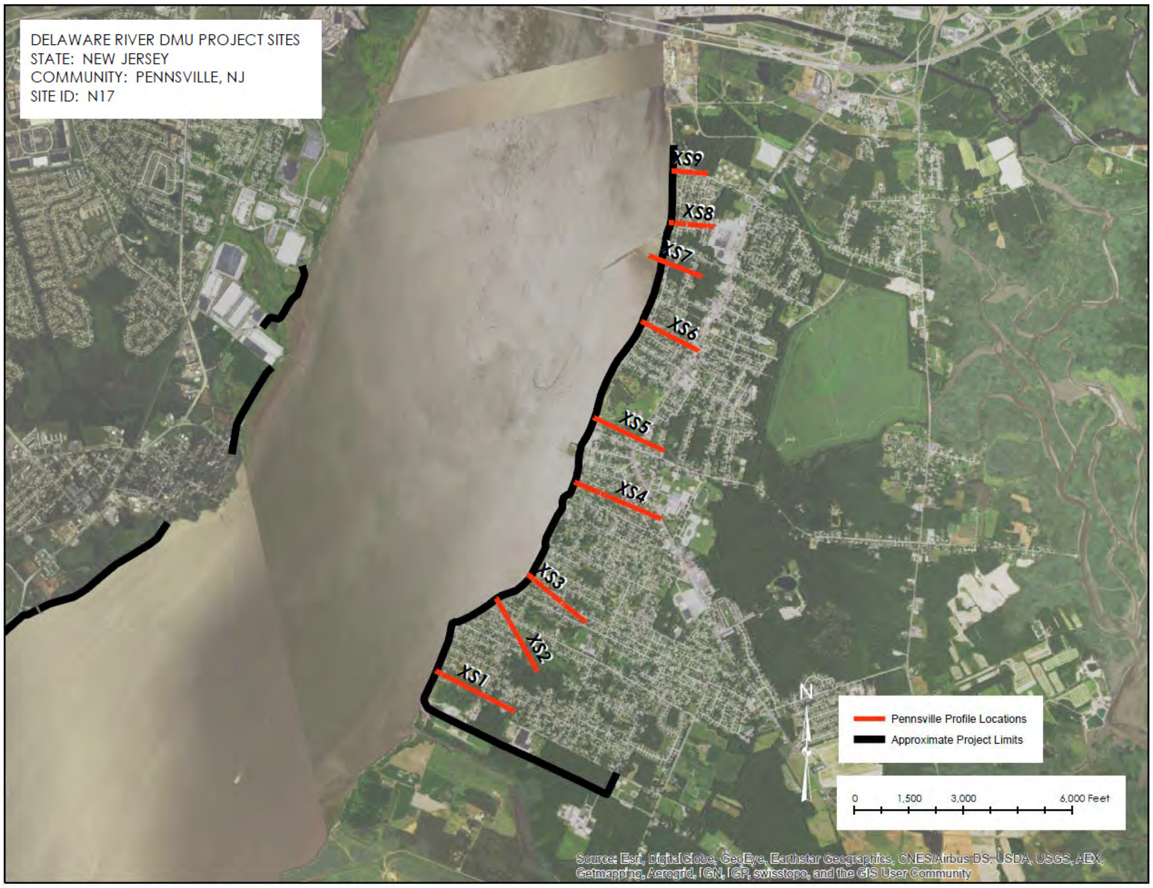


Figure 4: Example profile locations for screening

Table 3: Existing level of protection from topographic assessment

Site ID	Location / Municipality	Approx. Elevation of High Ground / Existing 'Protection' (ft, NAVD88)
N15	Penns Grove	7.0 to 8.0
N17	Pennsville	7.5 to 9.5
N25	Bivalve	6.0 to 6.5
N26	Shellpile	6.0 to 6.5
N27	Port Norris	6.0 to 6.5
N28	Maurice River	10.0 to 12.0
N33	Villas	9.0 to 12.0

2.4 SCREENING LEVEL SUMMARY

Per the screening methodologies applied in Section 2.1 through 2.3 above, the original intent was to use the two stage-probability curves generated by the NACCS numerical modeling as inputs to the HEC-FDA model to estimate the economic benefits of a beach restoration project at the CSRM problem areas. However, after further analysis, the PDT divided the study area into two planning reaches (northern reach and southern reach) based on the differing characteristics of the waterway in each reach.

In the northern reach, the width of the waterway is relatively smaller and the principal CSRM damages are due to inundation related to coastal storm surge (which includes wave radiation stresses), as occurs during tropical storms, hurricanes or nor'easters. However, in the southern reach, the width of the bay (fetch) increases and allows wind to generate greater wave energy at the shoreline, so that waves create an additional risk mechanism beyond inundation alone. Due to the additional damage mechanisms, the southern reach experiences CSRM damages from the combined effects of inundation, waves and storm erosion, analogous to the damage mechanisms experienced on the open ocean coast. Consideration of these additional damage mechanisms led to the inclusion of additional sites in the southern planning reach: Gandys Beach, Fortescue, Reeds Beach, Pierces Point, and Del Haven.

As qualitative screening, supported by a Value Engineering study, ruled out the CSRM problem areas in the northern planning reach (riverine portion of the study area), it became apparent that HEC-FDA was not the appropriate model to evaluate the sites in the southern reach. Therefore, Villas (N33) and five other sites (Gandys Beach, Fortescue, Reeds, Pierces, and Del Haven) were further analyzed with Beach-fx, as described in subsequent sections.

3.0 EXISTING CONDITIONS

This section provides a description of the hydraulic and coastal existing conditions at the six sites carried forward for to further evaluation as a beach restoration alternative. The six sites are (from north to south): Gandys Beach, Fortescue, Reeds Beach, Pierces Point, Del Haven, and Villas. Included in this section is a description of the tides, sea level change, winds, waves, NACCS model results, and historical shoreline changes. Figure 5 shows the location of the six sites as well as some of the tidal stations, wave buoy stations, and NACCS Save Points used throughout the study.

3.1 ASTRONOMICAL TIDES

Daily tidal fluctuations at the project site are semi-diurnal, with two highs and two lows per 24-hour day. Tidal ranges in Delaware Bay increase with distance above the mouth of the bay and reach a local maximum in the vicinity of Gandys Beach and Fortescue. Figure 6 shows the mean maximum tidal height in Delaware Bay. Tidal datum relationships at three NOAA stations in the study area are presented in Table 4. Fortescue Creek is used in this study to represent tidal conditions at Gandys Beach and Fortescue. Brandywine Shoal Light is used in this study to represent Reeds Beach and Pierces Point. Cape May, NJ is used to represent Del Haven and Villas.

Table 4: Tidal Datum Relationships

Datum[1]	Fortescue Creek	Brandywine Shoal Light	Cape May
MHHW	3.20	2.60	2.43
MHW	2.80	2.16	1.99
NAVD88	0.00	0.00[2]	0.00
MSL	-0.03	-0.29	-0.45
MLW	-3.05	-2.74	-2.86
MLLW	-3.22	-2.90	-3.02

Notes: [1]Tidal datums based on 1983-2001 Tidal Epoch, [2]NAVD88 based on NOAA's VDATUM Software

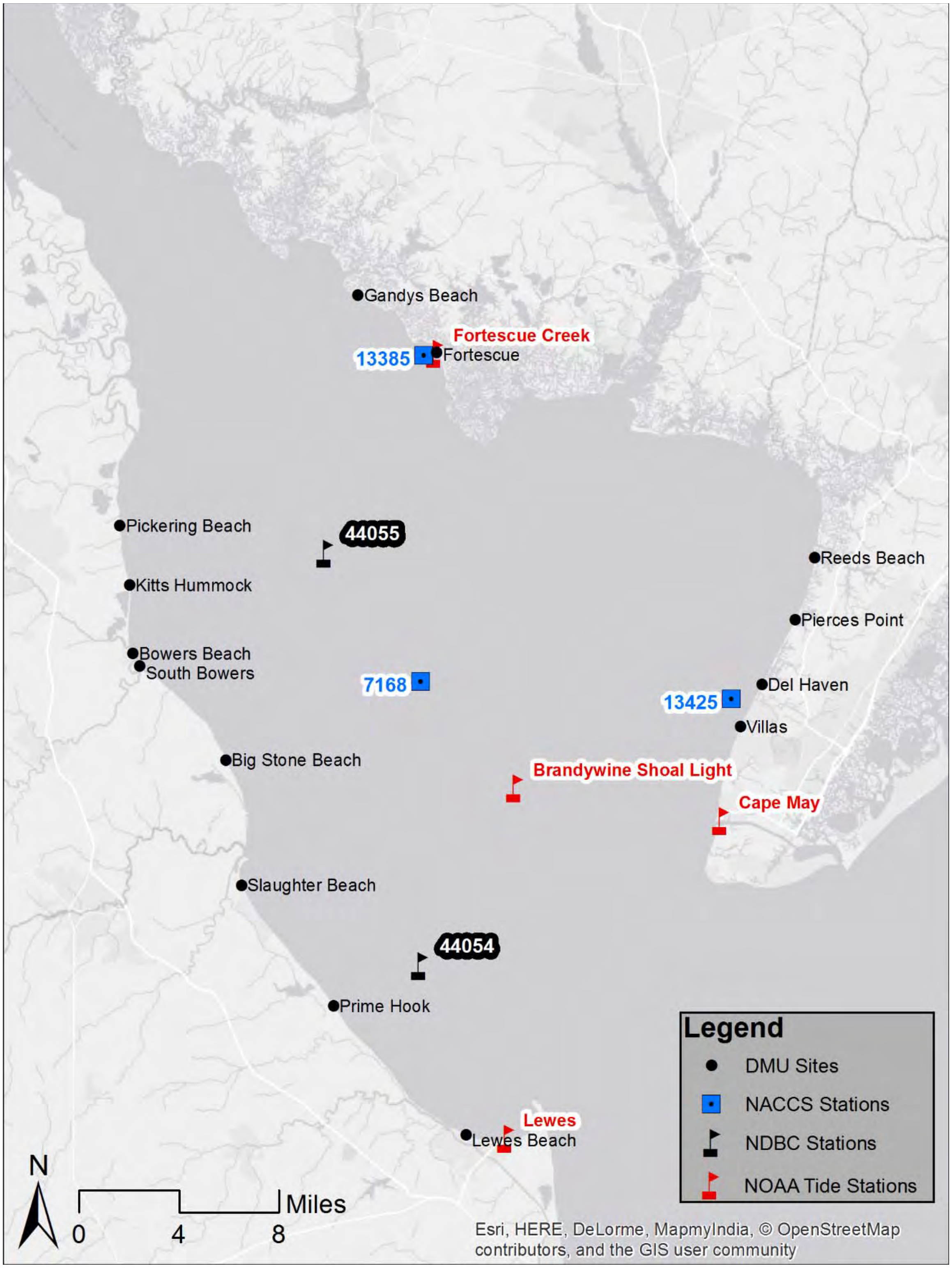


Figure 5: Existing Condition Data

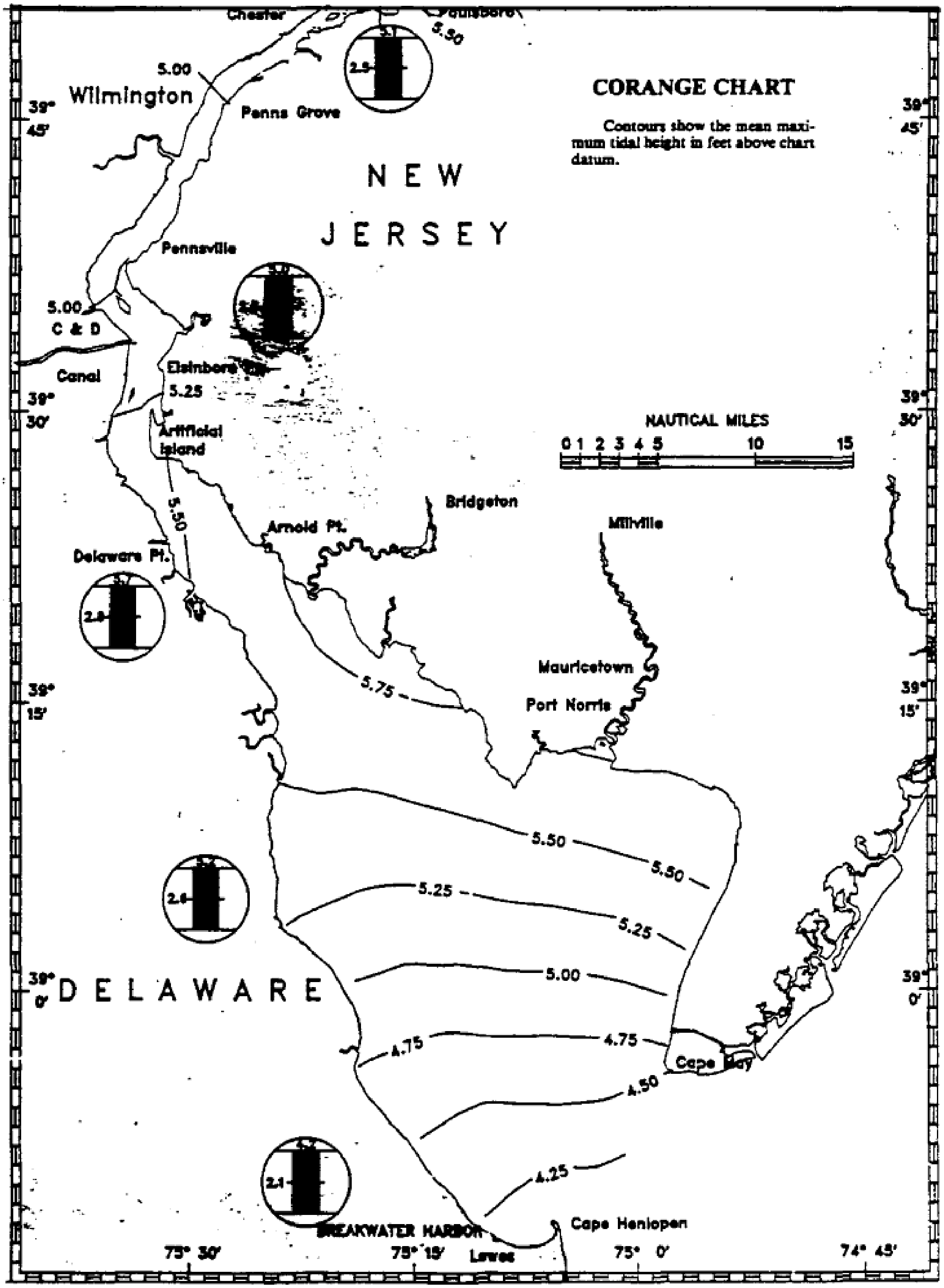


Figure 6: Maximum Tidal Range Contours (NOAA, 1988)

3.2 SEA LEVEL CHANGE

In accordance with ER 1100-2-8162, potential effects of relative sea level change (RSLC) on overall water levels were analyzed for each study location, over a 50-yr economic analysis period and a 100-yr planning horizon. A RSLC may be composed of both an absolute mean sea level change component and a vertical land movement change component. Historical RSLC and USACE SLC scenarios for this study are based on NOAA tidal records at Lewes, DE (Figure 7). Lewes, DE was selected over Cape May, NJ because the tidal record length at Lewes, DE is several decades longer than Cape May, NJ. Table 5 presents RSLC projections for the three USACE scenarios: Low/Historical, Intermediate, and High. A graphical display of the three RSLC scenarios over the 100-yr planning horizon is presented in Figure 8.

Table 5: USACE Sea Level Change Scenarios

Year	USACE - Low (ft, MSL[1])	USACE - Int (ft, MSL[1])	USACE - High (ft, MSL[1])
1992	0.0	0.0	0.0
2020	0.3	0.4	0.6
2045	0.6	0.8	1.6
2070	0.8	1.4	3.1
2095	1.1	2.0	5.0
2120	1.3	2.8	7.4

[1]Mean Sea Level based on National Tidal Datum Epoch (NTDE) of 1983-2001

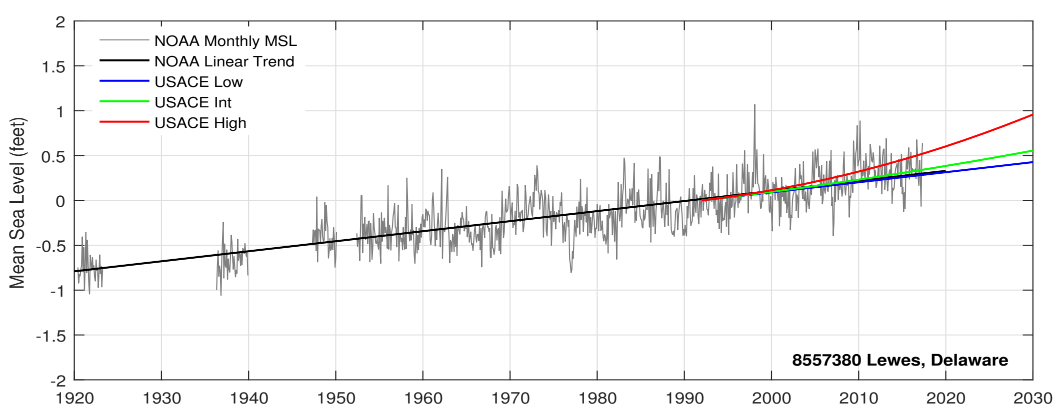


Figure 7: Historical Relative Sea Level Change at Lewes, DE

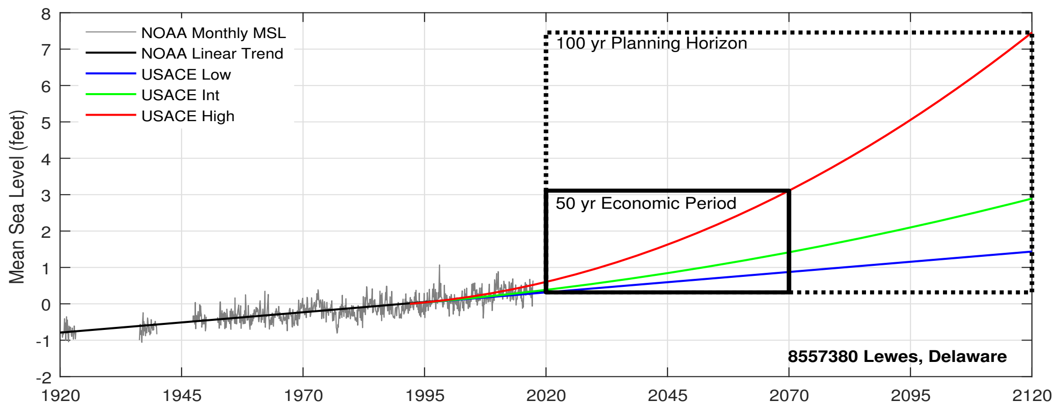


Figure 8: Relative Sea Level Change Projections at Lewes, DE

3.3 WINDS

The prevailing wind direction reported at the Brandywine Shoal Light in Lower Delaware Bay is from the northwest. The annual wind rose diagram in Figure 9 shows that the most frequent and strongest wind directions (greater than 26 knots) are from the northwest. However, relatively strong winds (greater than 18 knots) occur from all directions. Seasonal wind roses, as seen in Figure 10, show that the wind regime varies from season to season, with the stronger winter winds prevailing from the northwest and the majority of the summer winds prevailing from the south. However, some of the strongest winds (highest velocity) observed throughout the year are from the northeast (USACE 1998).

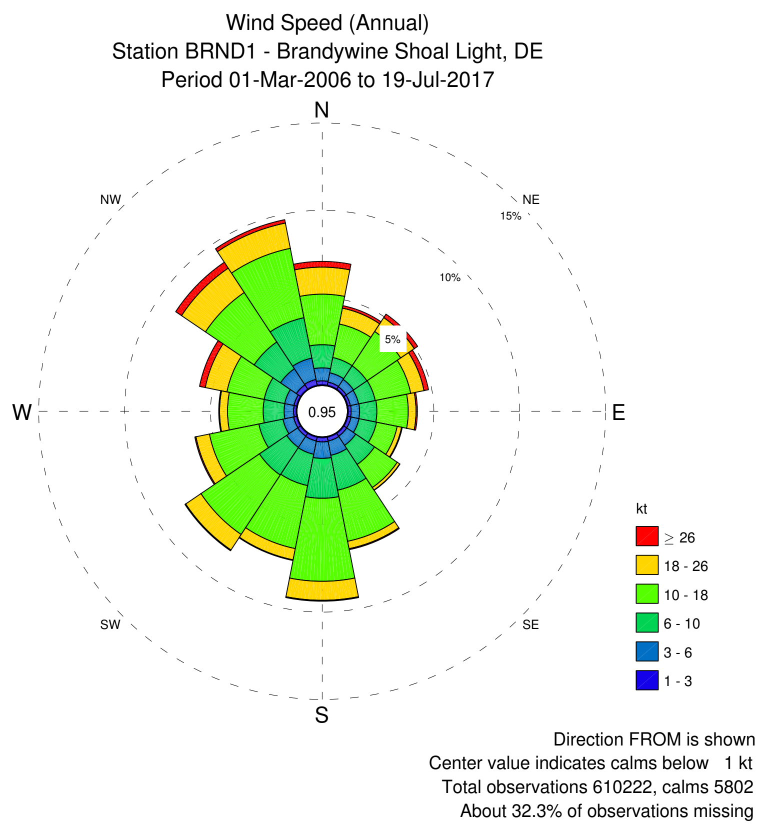
3.4 WAVES

Waves within Delaware Bay may be generated by local winds or propagate from the ocean through the mouth of the Bay. Further away from the mouth of the Bay the wave direction is associated with the wind direction and prevailing fetch. Two NOAA National Data Buoy Center (NDBC) stations are available inside Delaware Bay, 44054 and 44055. Table 6 shows the location and available record length at these two buoy stations. Station 44054 is located near the mouth of the Bay and is exposed to a combination of local winds and waves that propagate through the mouth of the Bay. Station 44055 is located farther up the Bay and is primarily exposed to locally generated waves. Wave roses for these two stations, Figure 11 and Figure 12, show that the primary difference between this two stations is that the 44054 is exposed to significant more direction from the east (i.e. propagating from ocean through the mouth of the Bay). Station 44054 is only located about 4 miles offshore of the Delaware Coastline and as a result wind generated waves from the SW quadrant don’t have open water fetch to grow into significant waves.

The six sites under consideration in this study are sheltered from ocean waves propagating through the mouth of the Bay. Therefore, the general wave conditions at the sites is best characterized by station 44055. However, the wave directions at each of the 6 sites will vary based on the prevailing open water fetch direction and lengths. Wave Height probability of exceedance at Station 44055 is shown in Figure 13 and the joint probability between the wave height and peak wave period is shown in Figure 14. The joint probability figure shows that the largest wave heights at Station 44055 are short waves with peak wave periods between 2 and 6 seconds.

Table 6: NOAA NDBC Wave Data

Buoy Station	Latitude (deg. N)	Longitude (deg. W)	Water Depth (ft)	Record Length
44054	38.883	75.183	26	2017-2-6 to 2008-1-29
44055	39.122	75.256	n/a	2017-6-6 to 2008-1-29



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	7.08	4.66	5.18	4.81	4.04	3.25	3.94	6.58	9.38	7.21	8.11	6.06	4.52	5.78	8.76	9.68	99.05
26+	0.33	0.14	0.24	0.27										0.34	0.41	0.20	2.36
18–26	1.55	0.93	0.95	0.79	0.45	0.26	0.17	0.43	1.09	0.66	1.07	0.80	0.46	1.29	1.68	1.48	14.05
10–18	2.89	1.95	2.36	2.11	1.83	1.23	1.59	2.98	4.75	3.68	4.09	2.93	2.07	2.21	3.41	4.05	44.15
6–10	1.33	0.90	0.94	0.99	1.01	1.04	1.32	2.04	2.29	1.83	1.91	1.43	1.22	1.22	1.85	2.36	23.64
3–6	0.74	0.54	0.51	0.49	0.49	0.47	0.63	0.83	0.94	0.78	0.77	0.63	0.55	0.55	1.11	1.25	11.28
1–3	0.24	0.20	0.18	0.17	0.19	0.21	0.22	0.24	0.26	0.23	0.22	0.20	0.17	0.18	0.31	0.35	3.57

Figure 9: Brandywine Shoal Light Annual Wind Rose

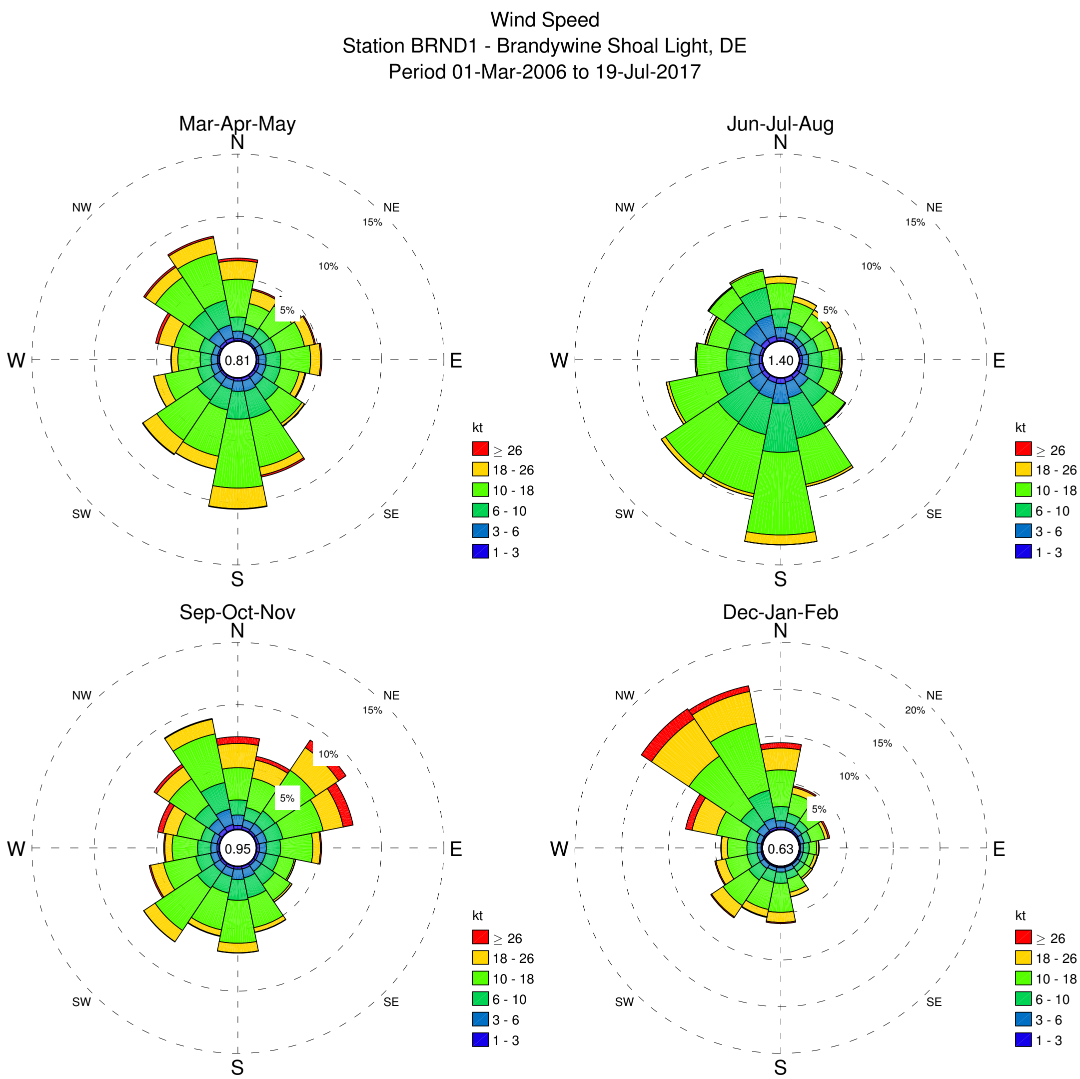
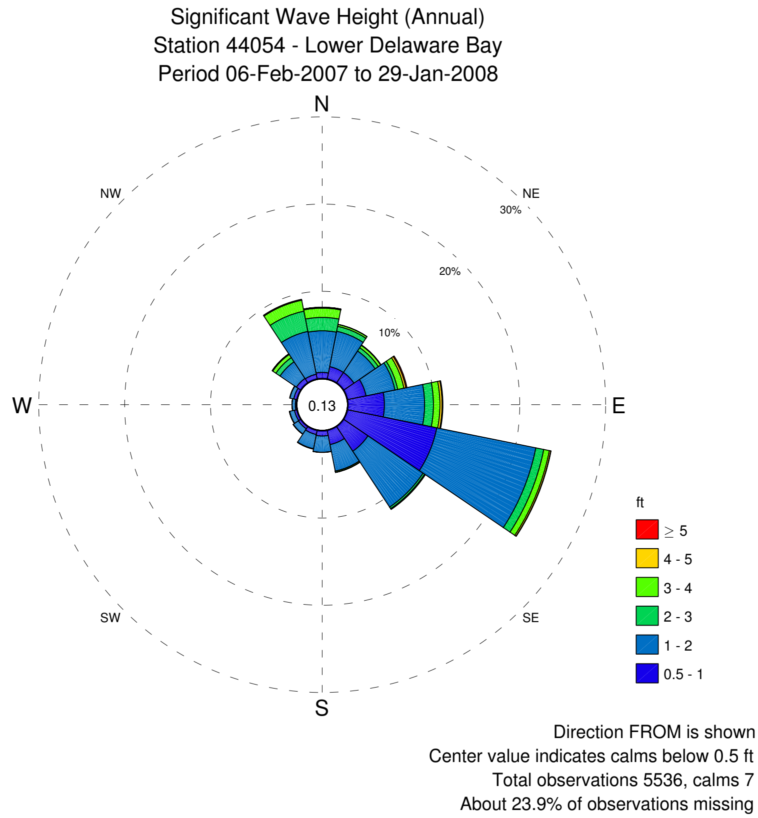


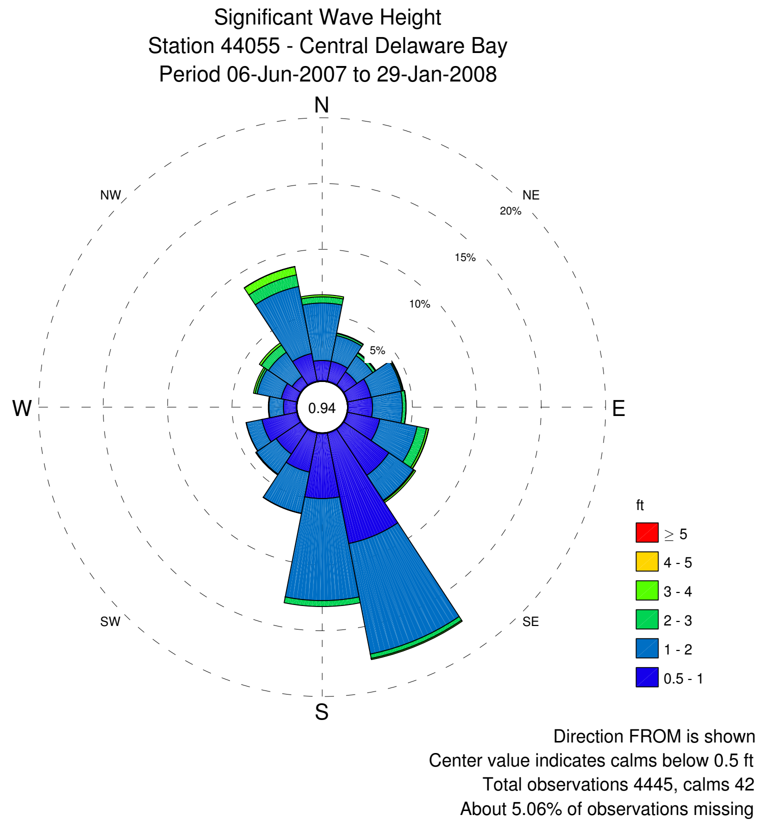
Figure 10: Brandywine Shoal Light Seasonal Wind Roses



Percentage of Occurrence

Significant Wave Height, ft	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	8.18	6.39	5.24	7.06	11.05	24.17	11.45	4.93	2.47	2.20	1.08	0.90	0.52	0.87	4.01	9.32	99.87
> 5				0.13													0.31
4 – 5				0.23	0.25	0.16										0.11	1.01
3 – 4	1.07	0.22	0.33	0.74	0.76	0.67	0.11	0.13							0.43	1.28	5.73
2 – 3	1.54	0.58	0.45	0.33	1.03	0.94	0.22								0.58	2.37	8.22
1 – 2	4.77	4.14	2.96	3.43	4.68	11.76	7.71	3.11	1.86	1.66	0.72	0.58	0.38	0.45	2.29	5.00	55.51
0.5 – 1	0.70	1.45	1.45	2.20	4.24	10.60	3.41	1.66	0.60	0.49	0.33	0.29	0.13	0.38	0.61	0.56	29.10

Figure 11: Buoy 44054 Annual Wave Rose



Percentage of Occurrence

Significant Wave Height, ft	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	6.55	3.76	3.01	4.43	4.52	6.43	6.73	17.57	13.16	6.25	4.21	4.03	2.18	3.44	3.87	8.91	99.06
≥ 5																	
4 – 5																	
3 – 4	0.16					0.16	0.13							0.13	0.18	0.63	1.75
2 – 3	0.47	0.16	0.22		0.27	0.74		0.34	0.45					0.16	0.67	0.90	4.63
1 – 2	4.36	2.00	1.48	2.41	2.32	2.95	2.29	8.57	7.76	3.19	1.84	1.24	1.12	1.91	2.20	5.20	50.87
0.5 – 1	1.55	1.55	1.24	1.87	1.91	2.56	4.23	8.57	4.95	3.04	2.29	2.77	1.01	1.24	0.81	2.16	41.75

Figure 12: Buoy 44055 Annual Wave Rose

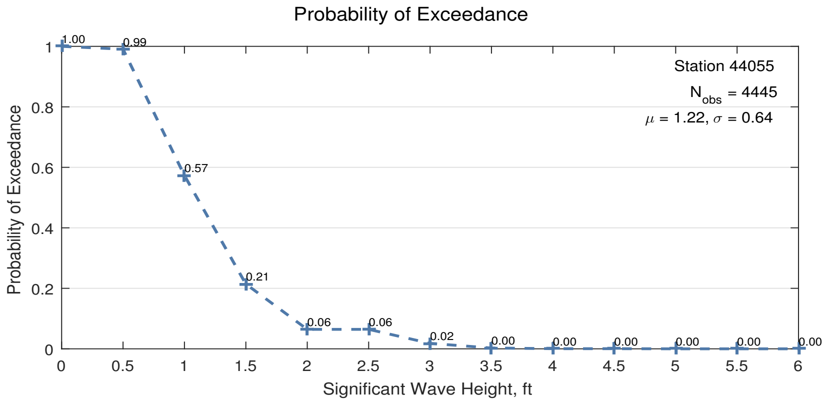


Figure 13: Buoy 44055 Probability of Exceedance

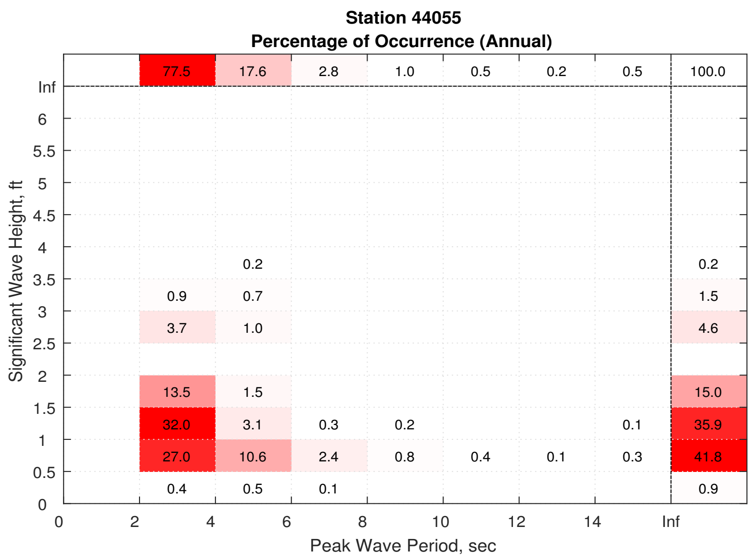


Figure 14: Buoy 44055 Joint Probability of Wave Height and Period

3.5 NORTH ATLANTIC COAST COMPREHENSIVE STUDY (NACCS)

The North Atlantic Coast Comprehensive Study (NACCS) was authorized under the Disaster Relief Appropriations Act, PL 113-2, in response to Superstorm Sandy. The Act provided the USACE up to $20 Million to conduct a study with the goal to (1) reduce flood risk to vulnerable coastal populations, and (2) promote resilient coastal communities to ensure a sustainable and robust coastal landscape system, considering future sea level change and climate change scenarios.

As part of the NACCS, the US Army Engineer Research and Development Center (ERDC) completed a coastal storm wave and water level modeling effort for the U.S. North Atlantic Coast. This modeling study provides nearshore wind, wave, and water level estimates and the associated marginal and joint probabilities critical for effective coastal storm risk management. This modeling effort involved the application of a suite of high-fidelity numerical models within the Coastal Storm Modeling System (CSTORM-MS) to 1050 synthetic tropical storms and 100 historical extra-tropical storms. Documentation of the numerical modeling effort is provided in Cialone et al. 2015 and documentation of the statistical evaluation is proved in Nadal-Caraballo et al. 2015. Products of the study are available for viewing and download on the Coastal Hazards System (CHS) website: https://chs.erdc.dren.mil/.

NACCS modeling results are applied throughout the NJ DMU study to define wave and water level Annual Exceedance Probabilities (AEP) and in the development of the Beach-fx storm suite. Model results at two save points, #13425 and #13385, are used to characterize the nearshore wave and water level conditions at the 6 sites. The location of these save points in relation to the 6 sites is shown in Figure 5. Water level and wave height AEP at these two save points are presented in Table 7 and Table 8 respectively. The water level AEP are based on the “Base + Linear superposition of 96 random tides” simulations and the mean confidence interval. The wave height AEP are based on the “Base Conditions + 1 random tide” simulations and the mean confidence interval.

The water levels reported in Table 7 represent the peak water level observed during a storm due to the combination of storm surge and astronomical tide. Theoretically wave setup could also contribute to the peak water level, however the save points are located in relatively deep water outside the surf zone where wave setup should be small. The water level does not include individual wave crests which may increase the instantaneous water surface by approximately 0.5 times the wave height (applying linear wave theory).

Table 7: NACCS Water Level Annual Exceedance Probability

Return Period (years)	Average Annual Exceedance Probability	#13425 (ft, NAVD88)	#13385 (ft, NAVD88)
1	100.0%	4.0	4.3
2	50.0%	4.6	4.9
5	20.0%	5.3	5.7
10	10.0%	5.7	6.3
20	5.0%	6.2	6.9
50	2.0%	7.0	8.1
100	1.0%	7.9	9.5
200	0.5%	8.9	11.0
500	0.2%	10.1	12.8

Table 8: NACCS Wave Height Annual Exceedance Probability

Return Period (years)	Average Annual Exceedance Probability	#13425 Hs (ft)	#13385 Hs (ft)
1	100.0%	4.7	4.0
2	50.0%	5.4	4.7
5	20.0%	6.1	5.5
10	10.0%	6.4	6.1
20	5.0%	6.7	6.7
50	2.0%	6.9	7.2
100	1.0%	7.2	7.5
200	0.5%	7.6	7.8
500	0.2%	8.3	8.4

3.6 SHORELINE CHANGE

The purpose of the historic shoreline change analysis is to document the past behavior of the study area's shorelines, in order to make a reasonable estimate of the long-term shoreline change rates. Previously documented shoreline change rates along the study area were reviewed and are summarized in Table 9. The alongshore extent corresponding to each location in Table 9 is shown in Figure 15. In addition to the prior studies, a new shoreline change analysis (Attachment C) was completed at Villas and Del Haven using long profile survey data from 1995 and LiDAR data from 2014. There is considerably less information available on shoreline change rates at Gandys Beach and Fortescue. Observed shoreline changes at Fortescue were adjusted based on past beach fill activities to determine what the shoreline change rate would likely have been in the absence of these activities.

It is evident from Table 9 that there is fairly good agreement between previously reported shoreline change rates and more recent analyses by the Stockton College Coastal Research Center

(2016) and USACE (2016). The greatest uncertainty appears to be at Reeds Beach, with reported values ranging between -3 ft/yr and 0 ft/yr. However, the more recent analyses show that the shoreline at Reeds Beach has been stable with shoreline change rates up to -1 ft/yr.

Recommended Future Without Project (FWOP) shoreline change rates for the NJ DMU project, Table 11, are a synthesis of all the available shoreline change data in study area with greater emphasis on newer data.

Table 9: Historical Shoreline Change Rates (ft/yr) from Prior Studies – Cape May County

<table>
<tr><th rowspan="2">Location</th><th>USACE 1960</th><th>USACE 1991</th><th>FEMA 1993</th><th>USACE 1998a</th><th>USACE 1998b</th><th>CRC 2016</th><th>USACE[2] 2016</th></tr>
<tr><td>1842 to 1957</td><td>1842 to 1957</td><td>1842 to 1986</td><td>1943 to 1995</td><td>1842 to 1994</td><td>1995 to 2016</td><td>1995 to 2014</td></tr>
<tr><td>Goshen Creek</td><td rowspan="2">-3.0[1]</td><td rowspan="2">-3</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Reeds North</td><td rowspan="3">-2</td><td></td><td>0</td><td>0</td><td></td></tr>
<tr><td>Reeds South</td><td rowspan="2">-0.2</td><td rowspan="2">0</td><td></td><td>-1</td><td></td><td></td></tr>
<tr><td>Pierces Point</td><td></td><td>-1</td><td></td><td></td></tr>
<tr><td>Del Haven</td><td rowspan="2">+1.0</td><td rowspan="2">+1</td><td rowspan="2">0</td><td>-0.6</td><td></td><td></td><td>-0.1</td></tr>
<tr><td>Villas North</td><td>-0.2</td><td></td><td>+1.5</td><td>+1.5</td></tr>
<tr><td>Villas South</td><td rowspan="2">-2.3</td><td rowspan="2">-2</td><td>-2</td><td>-1.4</td><td></td><td></td><td>-0.9</td></tr>
<tr><td>North Cape May</td><td>+3</td><td>+1</td><td></td><td>0</td><td>+0.1</td></tr>
</table>

[1]Shoreline change reported for Reeds Beach to Goshen Creek
[2]Analysis conducted by Philadelphia District in support of this study, Attachment A

Table 10: Historical Shoreline Change Rates (ft/yr) from Prior Studies – Downe Township

Location	USACE 1991	HMM 2016	USACE[2] 2017
	1943 to 1995	1930 to 2013	1943 to 1995
Gandys Beach		-2.5	
Fortescue	-1		-2.5[1]

[1]Shoreline change rate in absence of past beach fill activities
[2]Analysis conducted by Philadelphia District in support of this study, Attachment A

Table 11: Recommended FWOP Shoreline Change Rates (ft/yr)

Location	Characterization	Shoreline Change (ft/yr)
Gandys Beach	Moderate Erosion	-2.5
Fortescue	Moderate Erosion	-2.5
Reeds Beach	Stable to Low Erosion	-1
Pierces Point	Stable to Low Erosion	-1
Del Haven	Stable	0
Villas North	Stable to Accretion	+0.5
Villas South	Moderate Erosion	-1.5

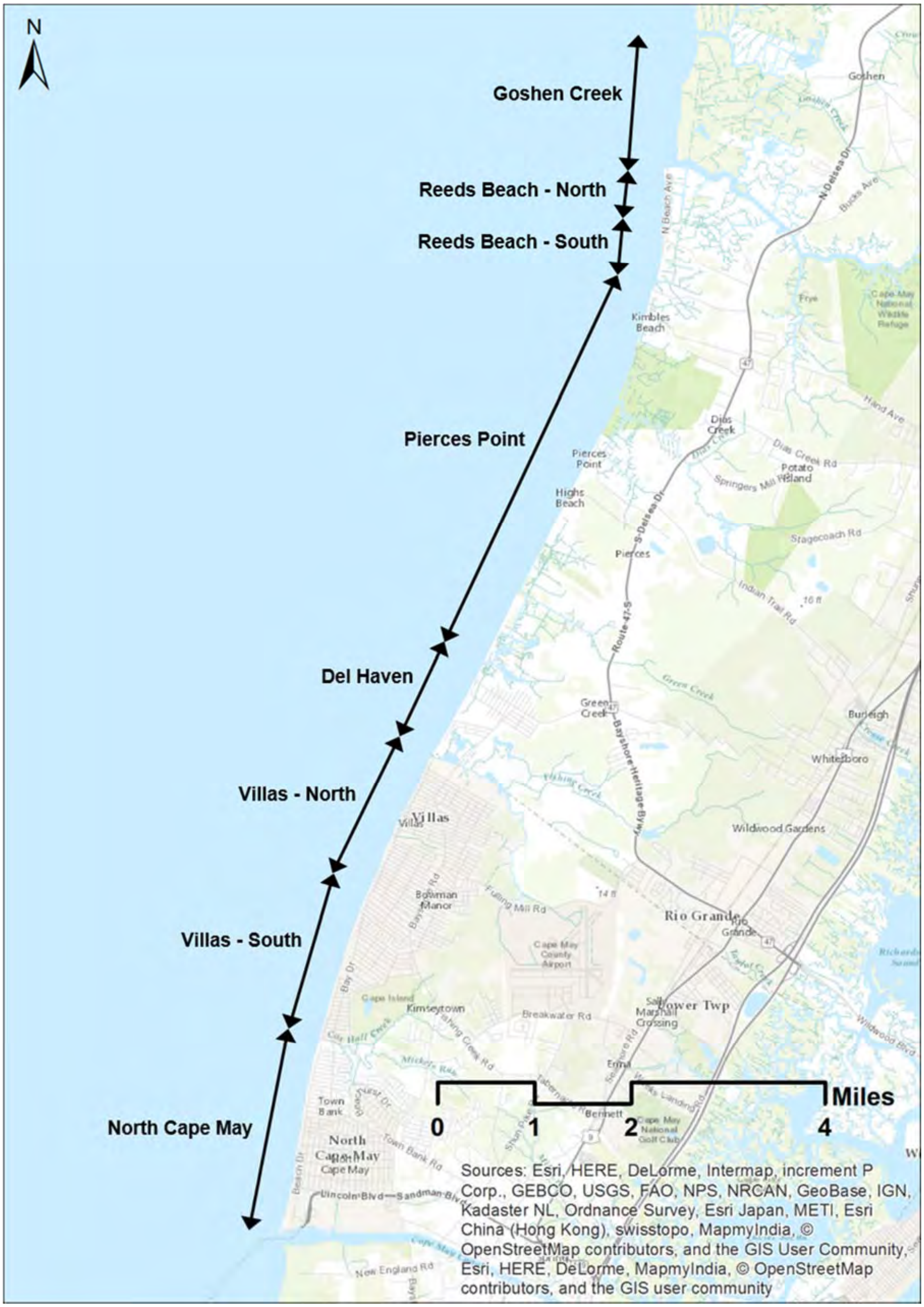


Figure 15: Shoreline Change Analysis Locations – Cape May County

4.0 BEACH-FX INPUT DATA

4.1 REPRESENTATIVE PROFILES

BEACH-FX SIMPLIFIED PROFILE

Due to the complexity of natural beach profiles, Beach-fx employs a simplified or idealized beach profile, representing key morphological features defined by points as shown in Figure 16 (Gravens et al. 2007). The simplified profile represents a single trapezoidal dune with a horizontal berm and a horizontal upland landward of the dune feature. The submerged portion of the profile is represented by a detailed series of distance-elevation points or as an equilibrium profile (Gravens et al. 2007). Some of the features of the simplified profile are taken as constant, not varying with storm response or management measures to reduce the number of profile permutations in the Storm Response Database (SRD) and improve computational efficiency. The beach profile variables that may be changed by storms are: dune width, dune height, berm width, and upland width. The constant values are: upland elevation, dune slope, berm elevation, foreshore slope, and the shape of the submerged profile. Thus, in response to a storm, the berm can erode or accrete (change in berm width), the dune can change height and/or width, and can translate landward resulting in an upland width change (Gravens et al. 2007).

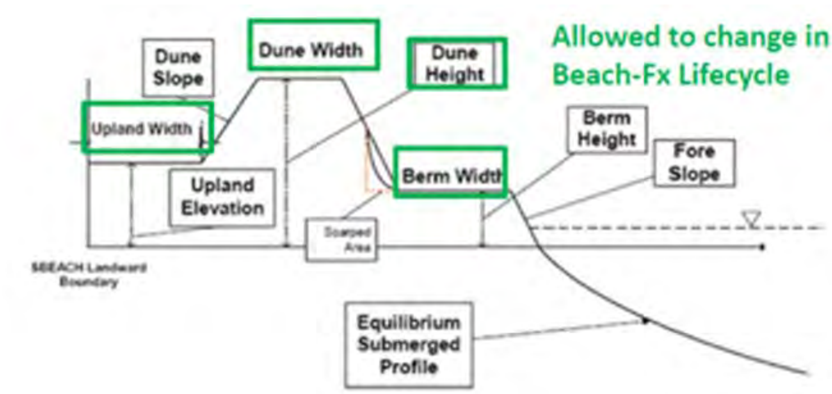


Figure 16: Beach-fx Simplified Profile

BEACH-FX MORPHOLOGY TYPES

Beach-fx supports three different morphology types as shown in Figure 17 and described below:

- Low Upland (LU): upland elevation < dune/berm elevation
- Low Berm (LB): berm elevation < upland/dune elevation
- High Upland (HU): upland elevation >= dune elevation

The most prevalent morphology types in the study area are LU and LB. However, the HU does occur in some portions of Villas where the upland elevations can exceed 14 feet NAVD88. The HU morphology type does not allow a dune that is lower than the upland elevation (invalid type shown in Figure 17. During the development of the representative profiles in Villas, it was important to understand the valid morphology types in Beach-fx.

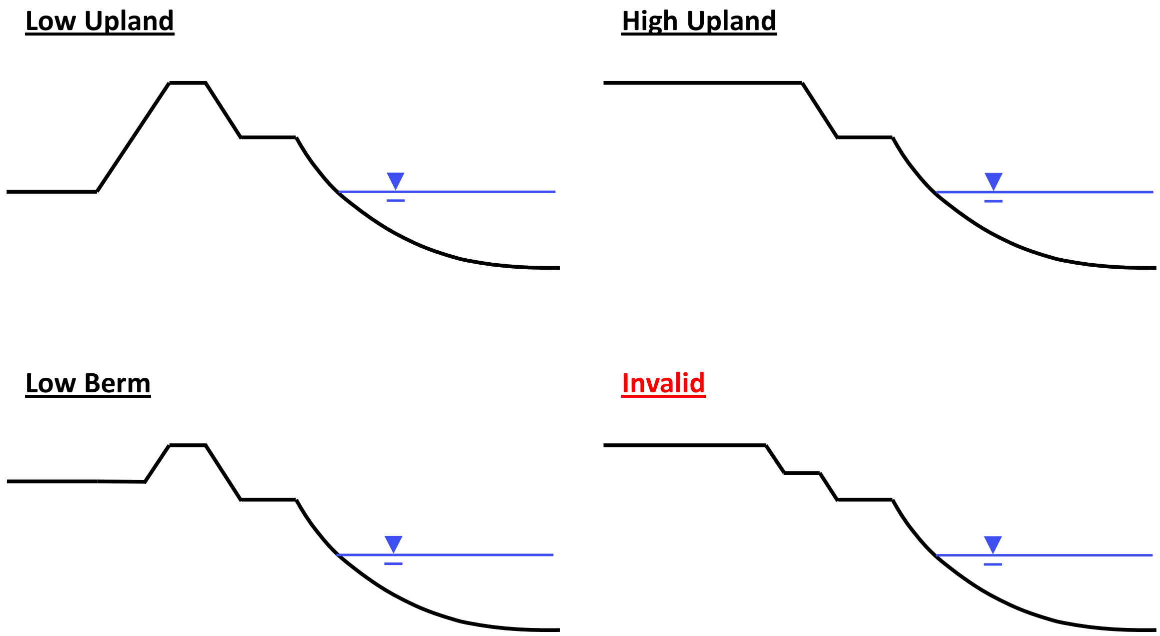


Figure 17: Beach-fx Morphology Types

Representation of the With-Project alternatives presents another challenge in Beach-fx. Many of the project sites are LB morphology type characterized with a relatively low and wide dune with houses located on the dune. With-Project alternatives at these sites include higher dunes, which would be constructed in front of the existing dune and houses (top left panel of Figure 18). These alternatives could actually have two dunes: (1) lower existing dune and (2) higher design dune. However, double dunes are not a valid morphology type in Beach-fx, so a modified representation of the alternatives is required. Figure 18 shows an example of two With-Project alternatives encountered in the project area and the approach to representing them within the allowable Beach-fx morphology types.

Another constraint within the Beach-fx framework is that the landward dune toe for all With-Project alternatives is the same as the Existing Conditions. The top right panel of Figure 18 shows an example of how the Existing and Design profile must share the same landward dune toe. With this constraint in mind, a conscious effort was made during the development of the representative profiles and existing conditions to place the landward dune toe seaward of houses where possible.

Developing representative profiles for Beach-fx is part science and part art, and the developer must balance the tradeoffs between more representative profiles and a better characterization of the existing conditions versus the resources required to model the additional representative profiles in SBEACH and Beach-fx. The developer must also balance the tradeoffs of accurately capturing the existing conditions versus accurately capturing potential With-Project alternatives.

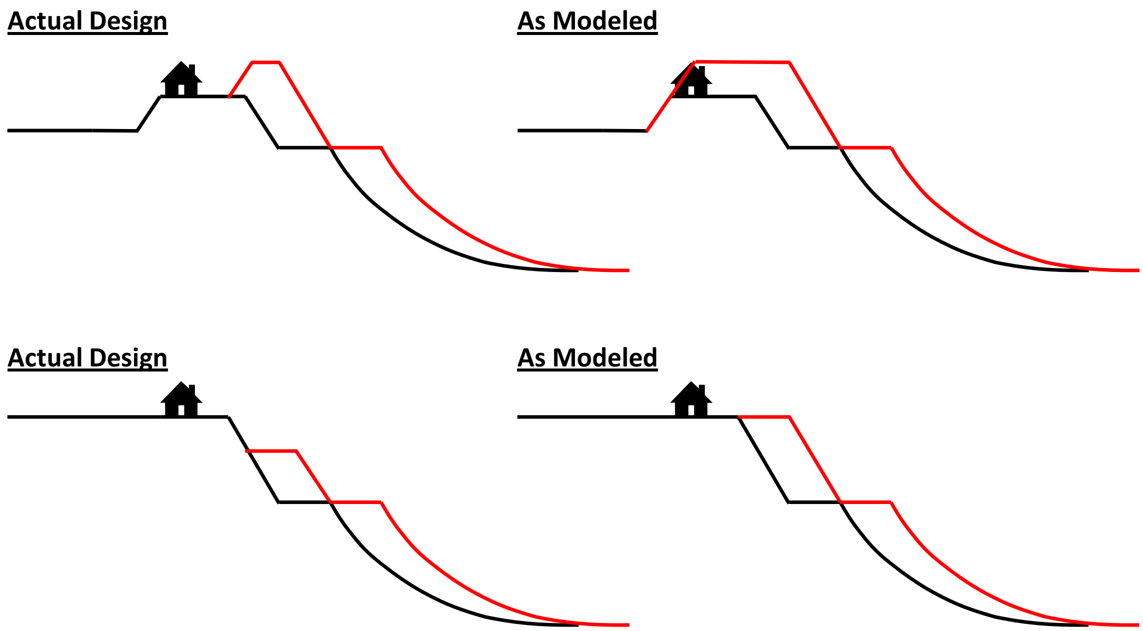


Figure 18: Beach-fx With-Project Constraints

DATA SOURCES

Three data sources were used to characterize the representative beach profiles:

- 2014 NOAA Post-Sandy Topobathymetric LiDAR
- 2015 NAP Beach Profile Survey of Gandys Beach and Fortescue
- 2017 NAP Beach Profile Survey of Reeds Beach, Pierces Point, Del Haven, and Villas

The 2014 LiDAR data was generally used to characterize the subaerial portion (dune & berm) of the profile, especially in Cape May County where survey data wasn't available until later in the study. To facilitate beach profile analyses, profiles were "cut" every 1,000 feet along the shoreline. The 2015 and 2017 NAP survey data were used to define the submerged profiles.

SUBMERGED PROFILES

The mean submerged profiles at each site were determined by first aligning all the profiles for a given site at +2 ft NAVD88, and then by calculating the mean of all the aligned submerged profiles. Figure 19 shows an example of the mean submerged profile at Del Haven. Conditions were similar enough along all of the sites except Villas to only have one submerged profile per site represent the entire site. Three submerged profiles were required at Villas to adequately capture the variability in the submerged profile conditions. The conditions at Gandys Beach and Fortescue were similar enough, both subaerial and submerged, that a single representative profile was adequate to represent both sites. A sensitivity analysis was performed at Gandys Beach and Fortescue to Superstorm Sandy to verify that a single submerged profile was adequate. Figure 20 shows the modeled dune and berm changes, which are morphological responses tracked in

Beach-fx, are very similar for both submerged profiles; hence, it was determined that a single submerged profile was adequate for Gandys Beach and Fortescue.

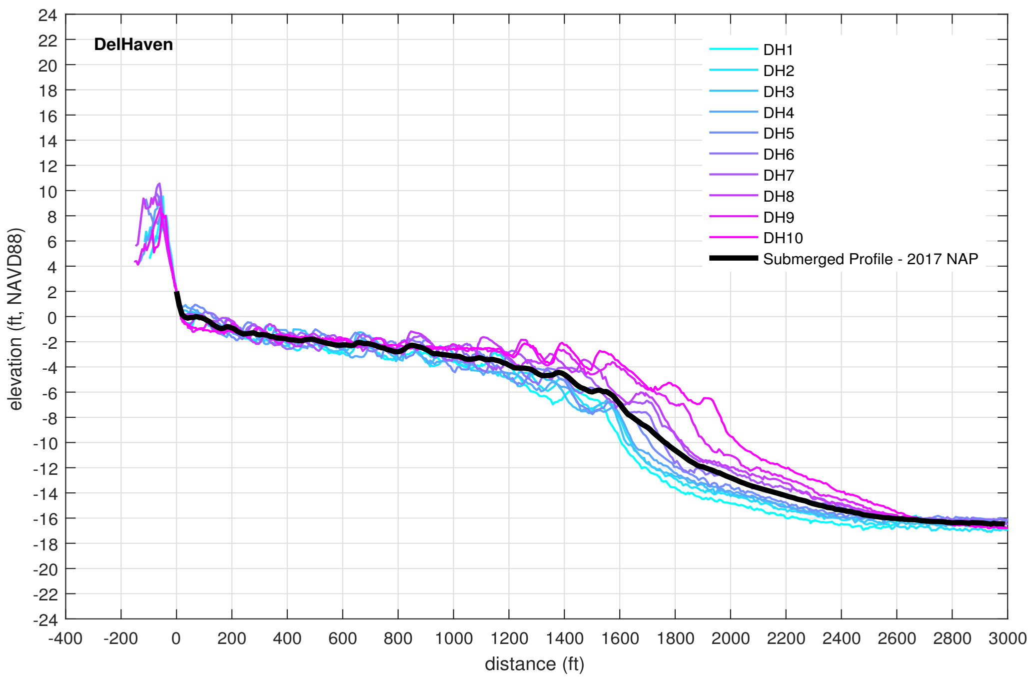


Figure 19: Submerged Profile at Del Haven

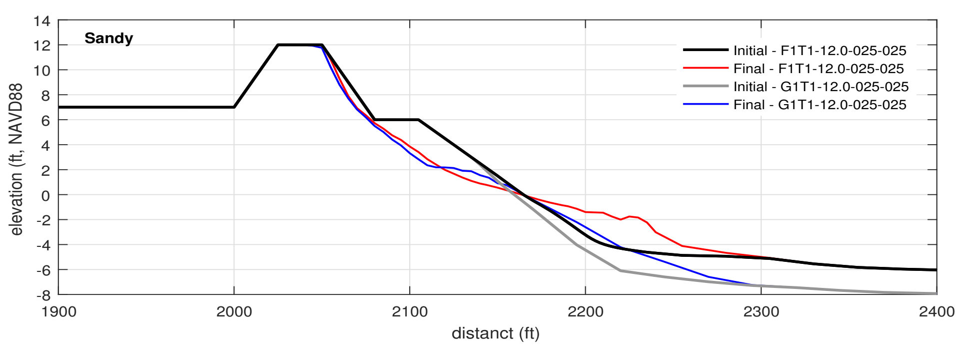


Figure 20: SBEACH Sensitivity to Submerged Profile at Gandys Beach and Fortescue

SUBAERIAL

The subaerial simplified profile parameters (Figure 16) were characterized at each site using a Matlab algorithm developed by ERDC-CHL that groups together similar profiles based on dune height and centers the profiles along the dune. The algorithm determines the representative upland elevation, dune elevation, berm elevation, berm width, dune slope, and foreshore slope.

An example of the algorithm for the “high” dune profiles at Del Haven is shown in Figure 21. The results of this analysis were primarily used to determine the characteristic foreshore slopes and dune slopes.

During the subaerial analysis, it became apparent that it was difficult to identify a berm elevation because none of the profiles exhibited a flat berm or gently sloping berm. A review of the 1999 Feasibility Reports for the study area (USACE 1999a, USACE 1999b) found that the proposed plan for Reeds Beach and Pierces Point had a berm elevation of +5.5 ft and the proposed plan for Villas and Del Haven had a berm elevation of +4.7 ft NAVD88. A review of the original LiDAR surface data revealed that in areas where the beach is the widest, there is a relatively flat berm around the +5 to +6 ft NAVD88. Based on these two data sources, a representative berm elevation of +5 ft NAVD88 was selected for Reeds Beach, Pierces Point, Del Haven, and Villas. The LiDAR data at Gandys Beach and Fortescue indicated that there was a relatively flat berm at +6 ft NAVD88 in the only area with a wide beach (adjacent to the jetty at Fortescue Creek).

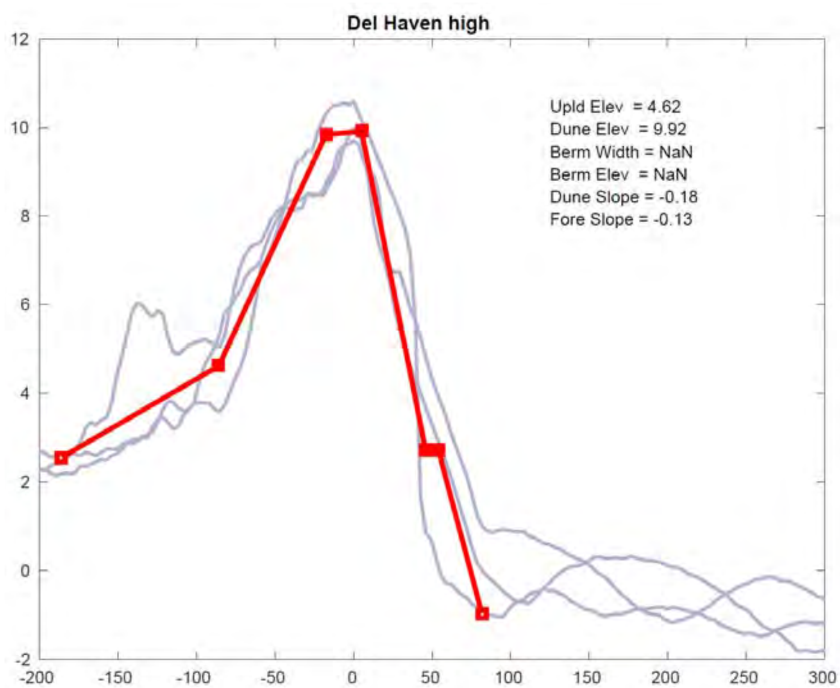


Figure 21: Example of Subaerial Profile Characterization at Del Haven

The remaining subaerial profile characteristics, upland elevation, dune height, dune width, and berm width were determined manually through trial and error by plotting the representative profile against existing profile data at each Beach-fx Reach. This trial and error process also took into consideration the cross-shore alignment of the representative profile, location of existing houses, and potential With-Project alternatives. Figure 22, Figure 23, and Figure 24 show an example of the final Beach-fx Alignment and representative profiles at Reaches 4 and 5 in Del Haven.

A complete overview of the representative subaerial profiles and envelope of existing profile data is shown in Figure 25 to Figure 34. The selected representative profiles strike a balance between accurately capturing the existing conditions and With-Project alternatives, as well as limiting the number of unique profiles and SBEACH model simulations.

EXISTING CONDITION REPRESENTATIVE PROFILES

The complete set of representative profiles is provided below in Table 12. Gandys Beach and Fortescue actually use the same representative profile and set of SBEACH simulations. All of the sites except Villas only required one representative profile. Due to distinct differences in the submerged profiles and high variability in dune and upland conditions at Villas, several representative profiles were required.

Table 12: Representative Profiles

Site	Profile Name	Submerged Profile	Upland Elv. (ft*)	Berm Elv. (ft*)	Dune Slope	Foreshore Slope	Dune Elv. (ft*)	Dune Width (ft)	Berm Width (ft)	Upland Width (ft)
Gandys	F1	Fortescue_Avg	6.5	6	0.20	0.1	6.5	0	0	1,000
Fortescue	F1	Fortescue_Avg	6.5	6	0.20	0.1	6.5	0	0	1,000
Reeds Beach	RB1	Reeds_02	5.5	5	0.10	0.1	5.5	0	0	800
Pierces Point	PP1	Pierces_Point	4.5	5	0.15	0.1	6	10	0	800
Del Haven	DH1	DelHaven	6	5	0.20	0.1	8	25	0	800
Villas North	VN1	Villas_North1	8	5	0.15	0.1	10	40	20	800
Villas North	VN2	Villas_North2	10	5	0.15	0.1	11	40	20	800
Villas North	VN3	Villas_North1	8	5	0.15	0.1	8	0	0	800
Villas South	VS1	Villas_South	10	5	0.20	0.1	12	25	0	800
Villas South	VS2	Villas_South	14	5	0.20	0.1	16	25	0	800

*All elevations are in feet NAVD88

Figure 22: Beach-fx Alignment – Del Haven

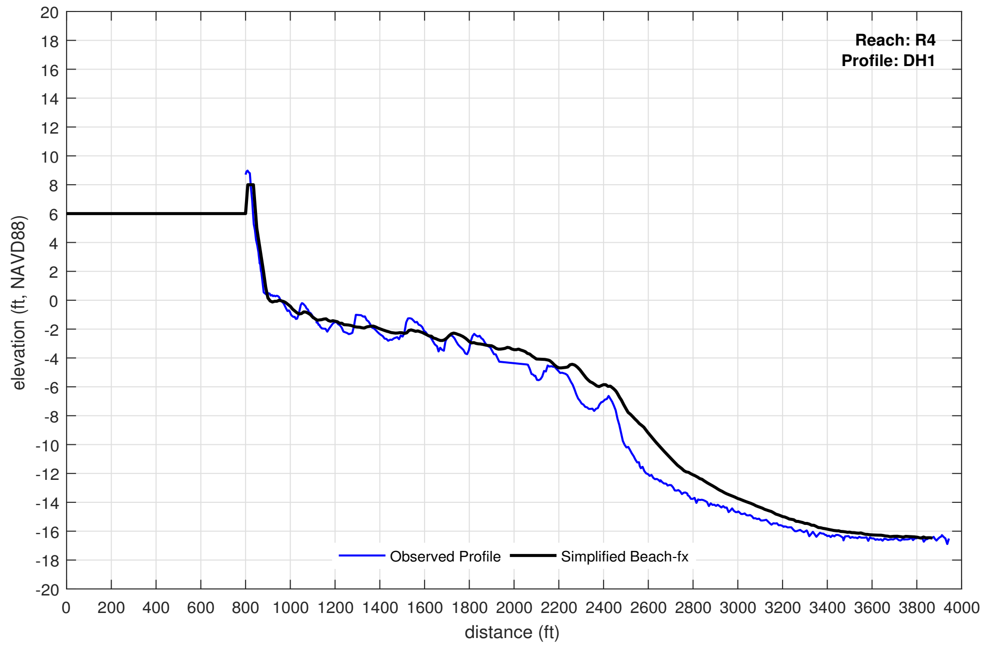


Figure 23: Trial and Error at Del Haven, Beach-fx Reach 4

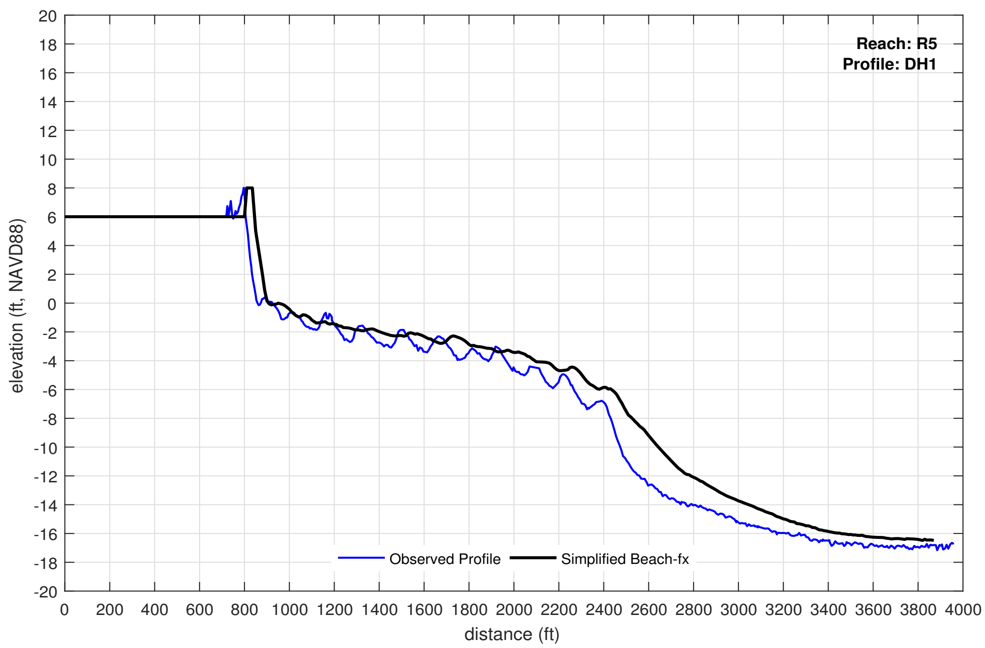


Figure 24: Trial and Error at Del Haven, Beach-fx Reach 5

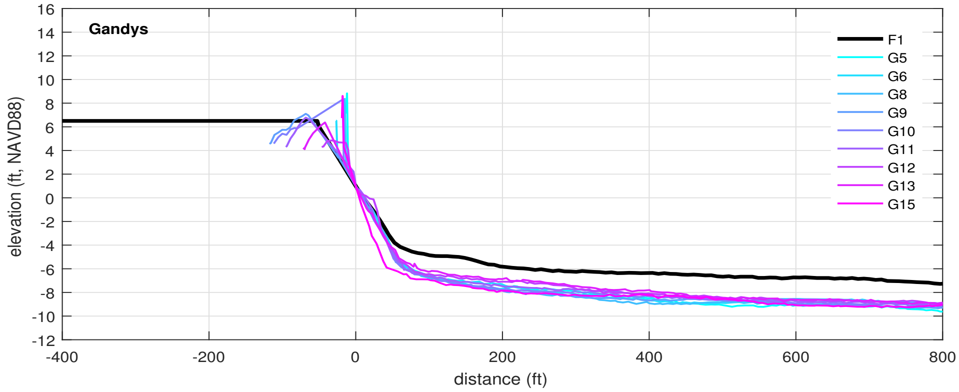


Figure 25: Representative Profile F1 – Gandys Beach

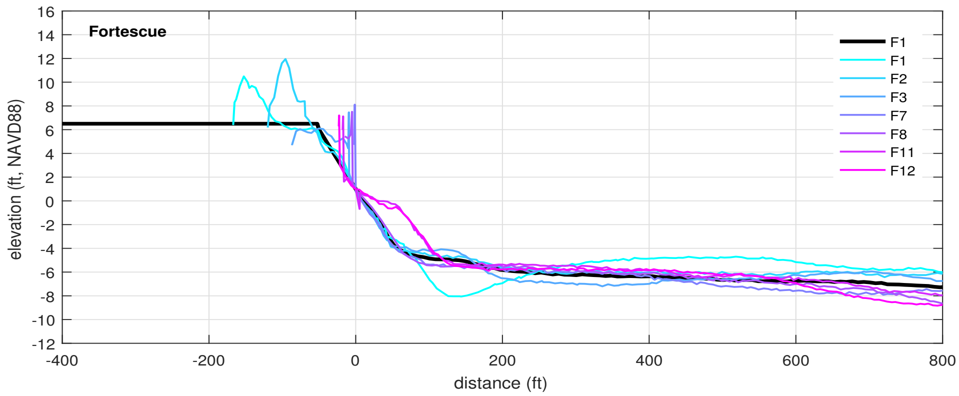


Figure 26: Representative Profile F1 – Fortescue

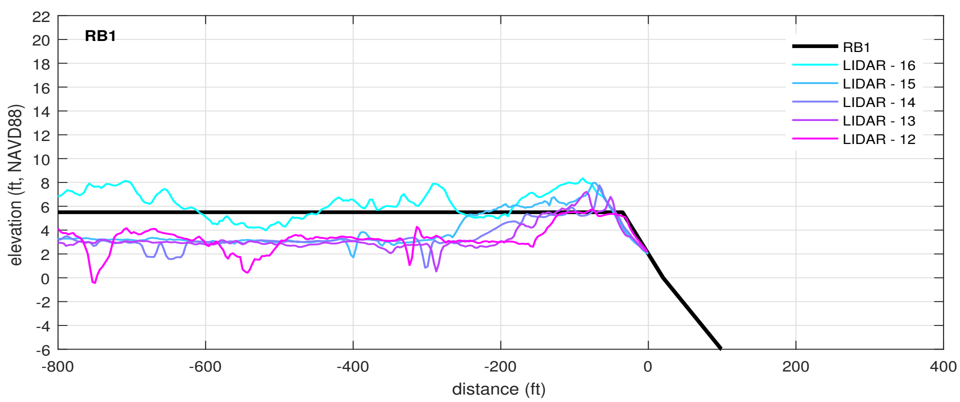


Figure 27: Representative Profile RB1 – Reeds Beach

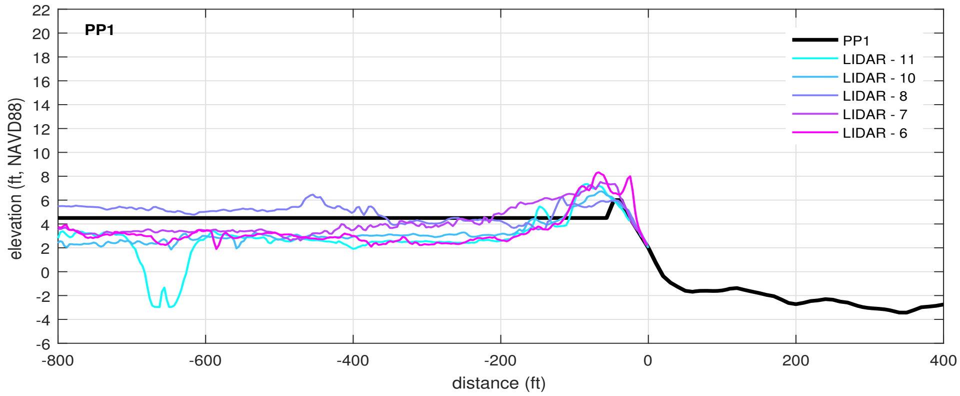


Figure 28: Representative Profile PP1 – Pierces Point

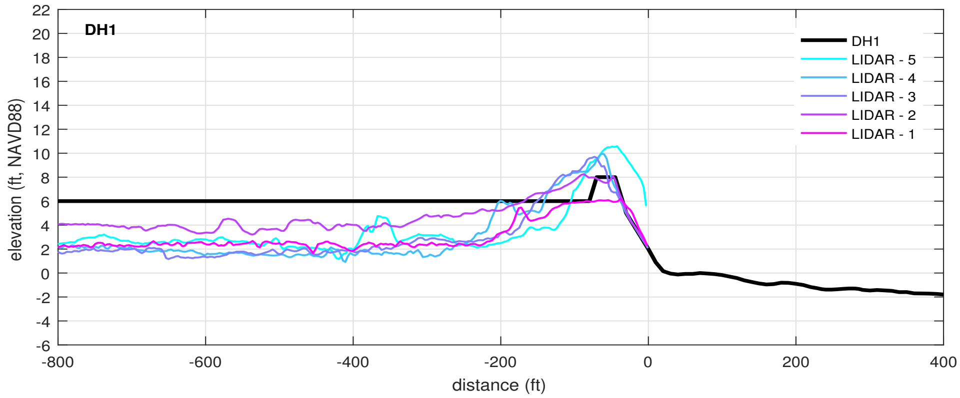


Figure 29: Representative Profile DH1 – Del Haven

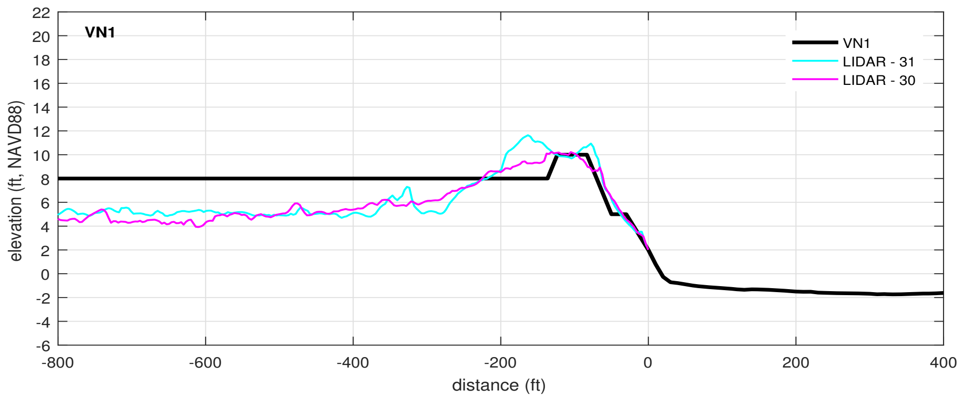


Figure 30: Representative Profile VN1 – Villas North

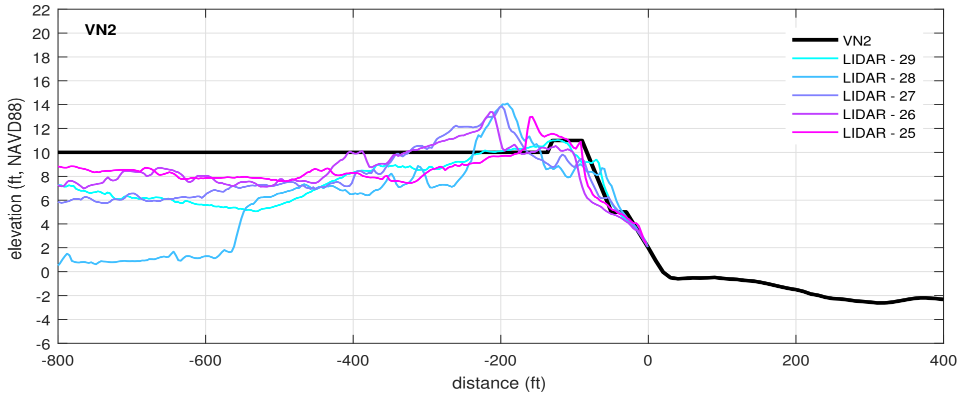


Figure 31: Representative Profile VN2 – Villas North

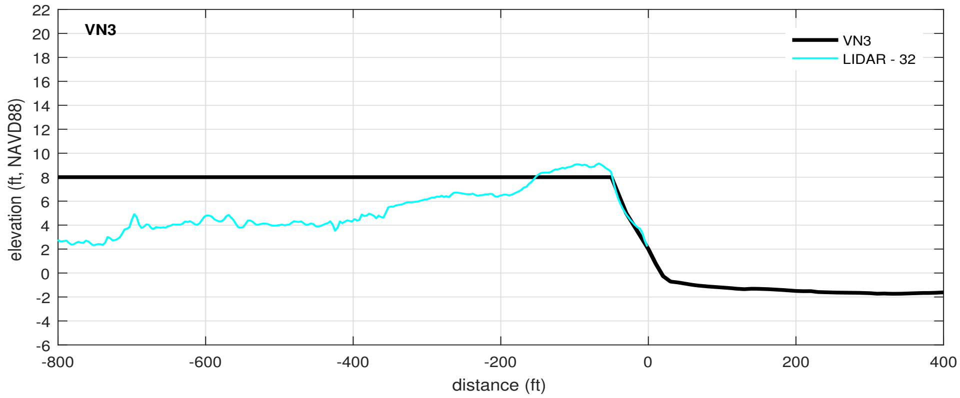


Figure 32: Representative Profile VN3 – Villas North

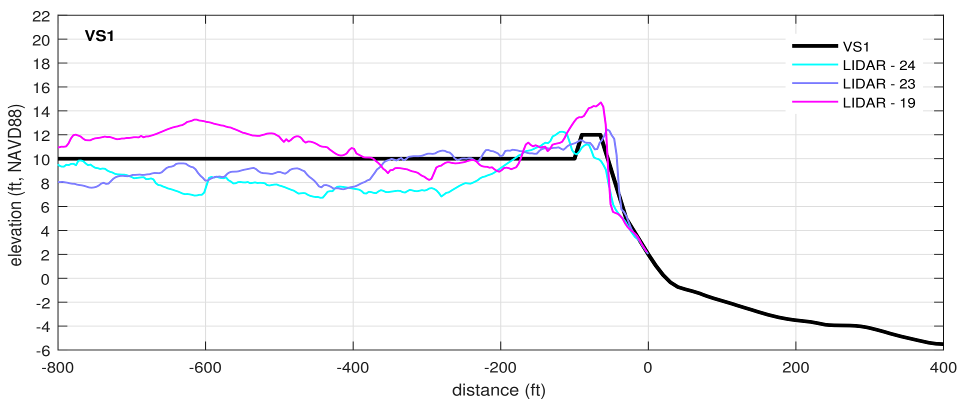


Figure 33: Representative Profile VS1 – Villas South

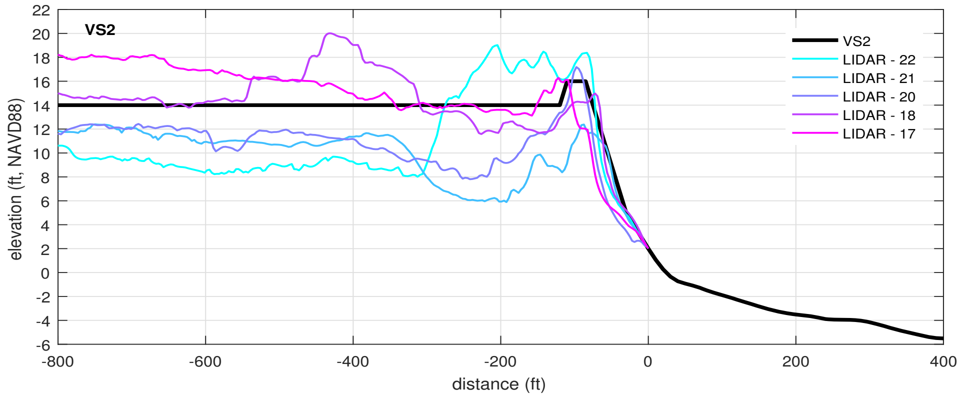


Figure 34: Representative Profile VS2 – Villas South

DEPTH OF CLOSURE

Dean and Dalrymple (2002) define the depth of closure as the "*offshore depth beyond which beach profiles taken over time at a given site coincide."* Seaward of this depth, although the waves can move sediment, the net sediment transport does not result in significant changes in mean water depth." The depth of closure is generally either determined from repeated cross-shore profile surveys or estimated using formulas based on wave statistics. Fortunately, repeated cross-shore profile surveys are available at two locations in project area from the New Jersey Beach Profile Network (NJBPN) collected by the Richard Stockton College of NJ Coastal Research Center (2013). Section 4.3 provides additional detail about the NJBPN.

Repeated profile surveys from 1995 to 2016 are available at Reeds Beach and Villas North. The survey data at Reeds Beach (Figure 35) clearly indicates a depth of closure of -6 ft NAVD88, whereas the survey data at Villas North (Figure 36) indicates a depth of -1 ft NAVD88. At both of these sites the depth of closure appears to coincide with the transition from the steep foreshore to the gentle sloping offshore portion of the profile. The wave conditions at the two sites are fairly similar and underscore the difficulty of trying to use wave statistics to estimate the depth of closure in Delaware Bay. Based on the observations at these two sites it is believed that the inflection point between the steep foreshore and more gentle offshore profile is a better indicator of the depth of closure. For simplicity, two depth of closure values were selected for the NJ DMU study, -3 ft and -6 ft NAVD88. Gandys Beach, Fortescue, Reeds Beach, and Villas South have deeper nearshore profiles and are best characterized by a depth of closure of -6 ft NAVD88. Pierces Point, Del Haven, and Villas North have shallower nearshore profiles and are characterized by a depth of closure of -3 ft NAVD88.

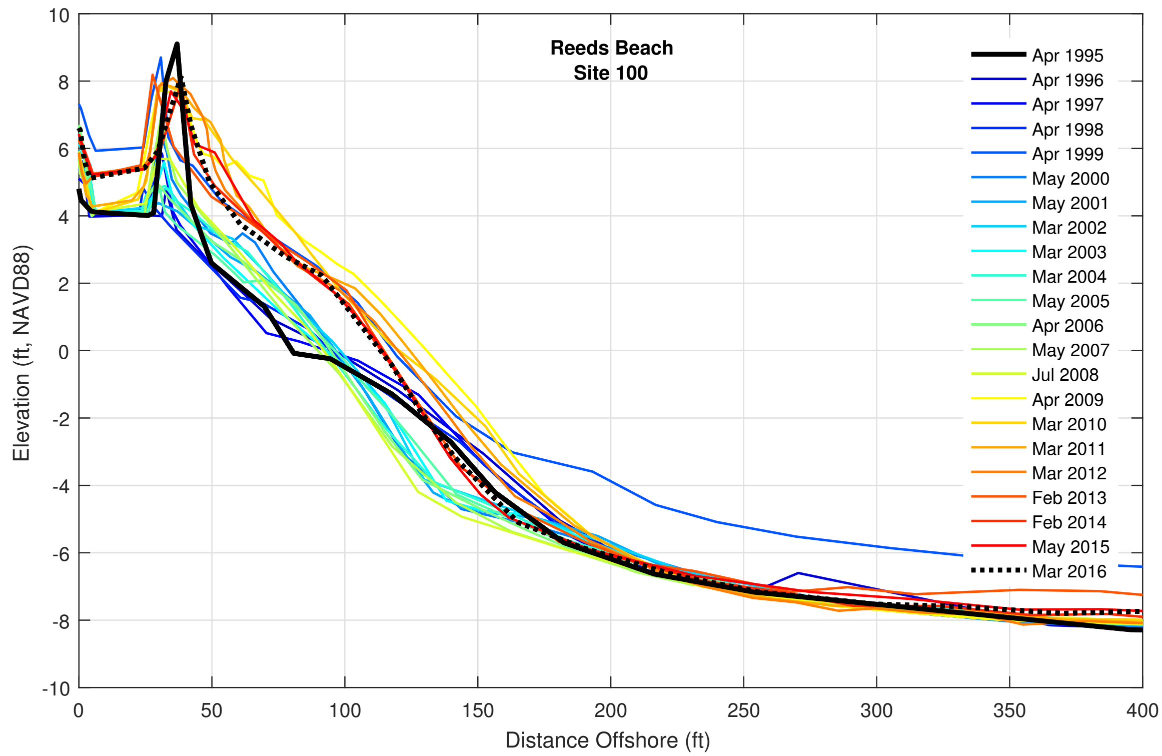


Figure 35: NJBPN – Reeds Beach (1995-2016)

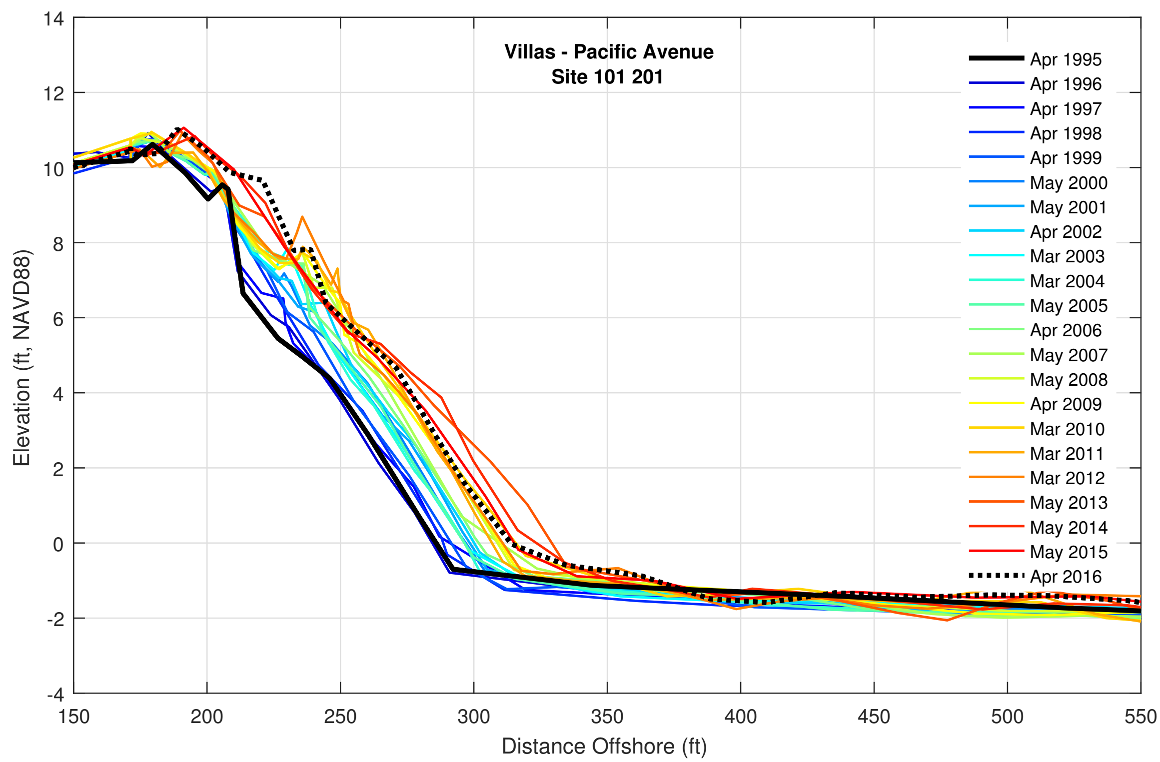


Figure 36: NJBPN – Villas North (1995-2016)

4.2 STORM SUITE

GANDYS BEACH AND FORTESCUE

This section summarizes the procedure used to develop a representative storm suite for Gandys Beach & Fortescue, NJ. Fifteen (15) tropical and 10 extratropical storms were identified as representative events characterizing the 1050 probabilistic tropical storms and 100 historical extratropical storms available for the study area in the Coastal Hazards System (CHS). Relative probabilities of the selected storms were computed by summing the individual relative probabilities of the storms each selected storm represents. The tidal analysis consisted of generating three idealized cosine tides (high, medium and low amplitude) and combining a high tide, mid-tide falling, low tide, and mid-tide rising at the peak surge of the water elevation time series. The 12 tidal combinations for each storm resulted in the generation of a total of 300 unique plausible storm events. NACCS Save Point 13385 was used in the analysis.

Identification of Representative Storms and Estimation of Relative Probabilities

Of the 1050 synthetic tropical storms available in the Coastal Hazards System (CHS), 389 storms have a storm track that pass within a 200km radius of the project site location (Figure 37). Because storms with a peak surge below 0.5m (~1.64 ft.) are assumed not to be damage producing, a peaks over threshold analysis was performed to eliminate these storms. Through this analysis, the tropical storms were further reduced from 389 to 321, and the extra-tropical storms were reduced from from 100 to 77.

The storms were then placed into bins based on the peak surge elevation. By performing a K-means clustering analysis on the tropical storm peaks, the lower and upper surge limits of each bin were defined. K-means clustering is a method used to groups points together that are more similar to each other than to the points in another cluster. In this particular method, the user selects the number of clusters, *K*, and the algorithm places *K* arbitrary “centroids” in the data set. The nearest neighbor to each “centroid” is determined, thus defining the initial clusters. A new, actual centroid of each cluster is calculated, and the nearest neighbor search is performed again. This process is repeated until the centroids no longer move. By performing this analysis in one-dimensional space on the peak surge each cluster is representative of a storm bin. The lower and upper limits of each bin are calculated as the average of the peak surge where one bin ends and the next begins. For example, the limit between bins 2 and 3 is defined as the average of the highest peak in bin 2, and the lowest peak in bin 3.

Because there are significantly fewer extra-tropical storms, these bins were set up manually. Table 13 and Table 14 summarize the tropical and extra-tropical bins, respectively.

Table 13: Tropical Storm Bin Ranges and Number of Storms in Each Bin

Bin Number	Peak Surge Limits (ft. MSL)	Storms in Bin
1	1.64-2.11	31
2	2.11-2.71	42
3	2.71-3.05	30
4	3.05-3.66	38
5	3.66-4.30	36
6	4.30-5.00	36
7	5.00-5.88	28
8	5.88-6.65	18
9	6.65-7.48	16
10	7.48-8.59	20
11	8.59-9.71	11
12	9.71-10.56	7
13	>10.56	8

Table 14: Extra-Tropical Storm Bin Ranges and Number of Storms in Each Bin

Bin Number	Peak Surge Limits (ft. MSL)	Storms in Bin
1	1.64-2.13	9
2	2.13-2.46	15
3	2.46-2.79	11
4	2.79-3.12	11
5	3.12-3.45	9
6	3.45-4.10	13
7	4.10-4.59	5
8	>4.593	4

After the storms were placed into their bins, the hydrographs were shifted along the time axis to align the peak surge. Bins 3 and 4 of the tropical storms and bins 2 and 6 of the extra-tropical storms were further divided into short and long duration storms within these storm bins. One storm was selected from each bin that represents all storms in that bin (Figure 38).

The relative probability of the selected storm is calculated as the sum of the relative probabilities of the storms that it represents. CHS provides a relative probability of occurrence for each of the synthetic tropical storms ensuring that large storm events do not occur at the same rate as smaller events. Conversely, because the extra-tropical storms are based on historical observations each storm possesses the same probability of occurrence. Table 15 and Table 16 show the selected representative storms and their relative probabilities. The tropical storm probabilities were normalized by the relative probability of the storms in bin 13.

Table 15: Tropical Selected Storms and Probabilities

Bin Number	Number of storms in Bin	Number of rep. storms	Selected Storms	Normalized Relative Probability
1	31	1	165	6.72
2	42	1	145	10.65
3	19	2	179	4.05
	11		236	1.36
4	23	2	190	10.02
	15		1008	1.80
5	36	1	530	7.32
6	36	1	139	6.40
7	28	1	297	3.62
8	18	1	526	2.49
9	16	1	143	1.45
10	20	1	623	2.55
11	11	1	123	1.25
12	7	1	93	0.30
13	8	1	36	1.00

Table 16: Extra-Tropical Storms and Probabilities

Bin Number	Number of storms in Bin	Number of rep. storms	Selected Storms	Relative Probability
1	9	1	1984-02-29	9
2	7	2	1978-12-25	7
	8		1978-04-26	8
3	11	1	1987-01-02	11
4	11	1	1982-10-25	11
5	9	1	1998-01-28	9
6	8	2	1980-10-25	8
	5		1983-12-12	5
7	5	1	1968-11-12	5
8	4	1	1974-12-02	4

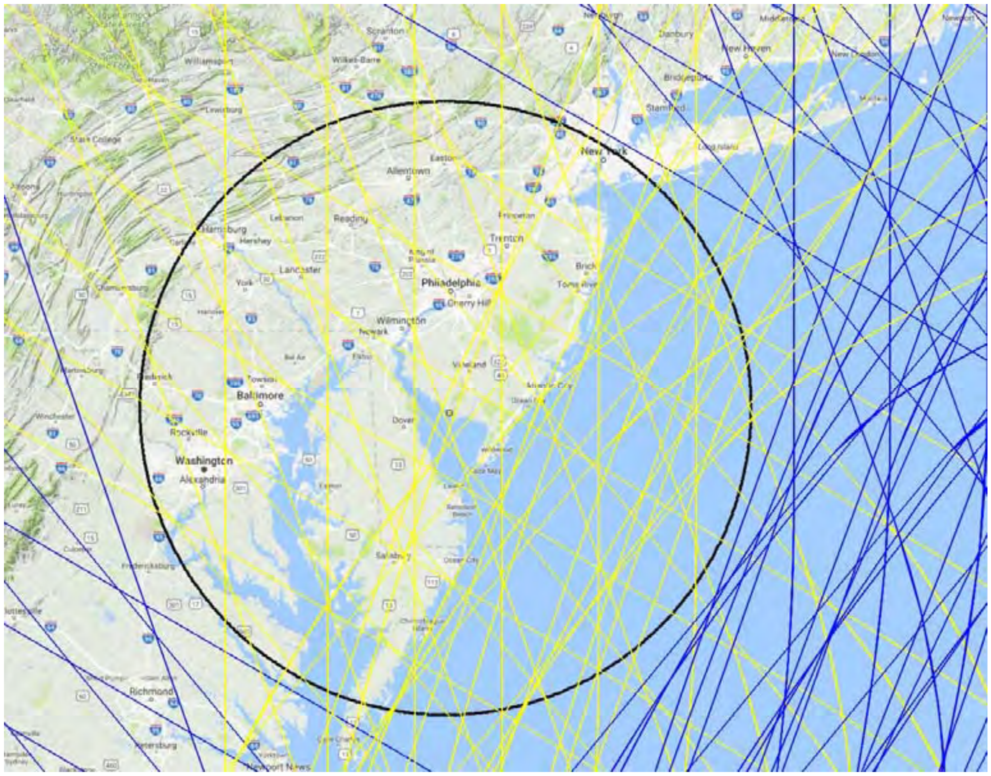


Figure 37: Storms within 200 km radius of site location

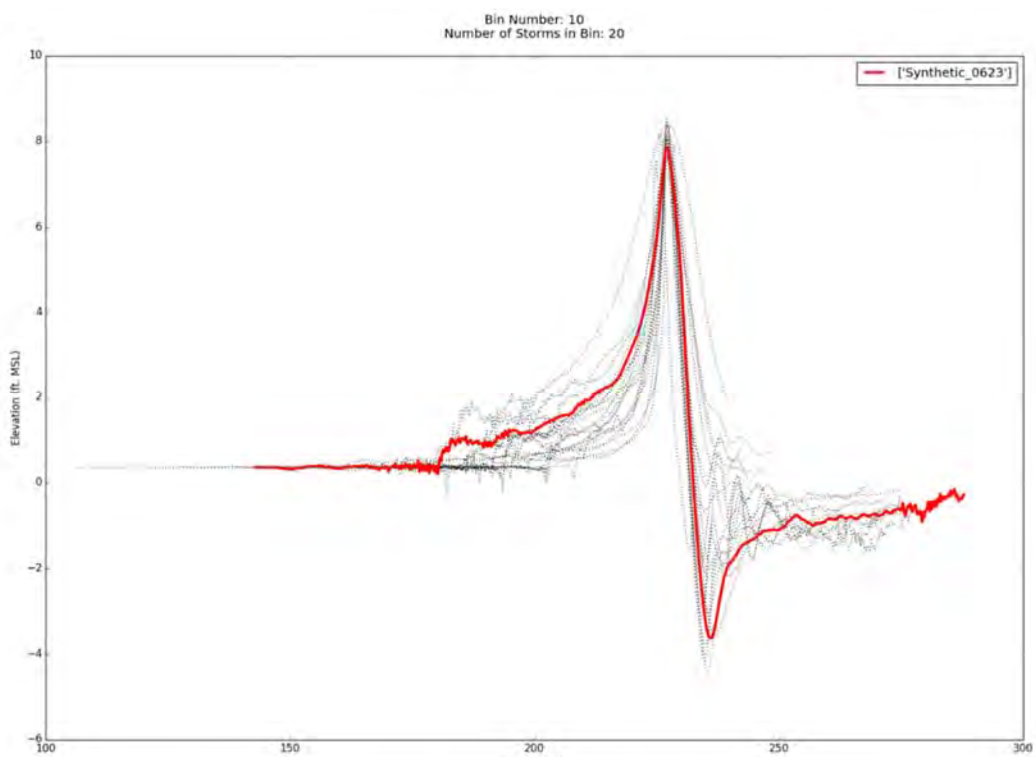


Figure 38: Selected Representative Storm for Bin 10

Wave Time Series

The wave, surge, and peak period time series of the representative storms were plotted together and were trimmed to start and end at the same time (Figure 39). There were three cases (tropical storms 143, 145, and 165) where the wave and peak period time series ended at a point that would result in the surge being trimmed immediately after the peak. In these cases, the waves were reflected across the peak value, and the peak period was increased step wise from the point of being cut off to a value of 8 seconds (Figure 40).

Tidal Analysis

The tidal analysis estimates the high, medium, and low tidal ranges for the site location, and are combined with the surge hydrograph to develop water elevation time series. A 20-year tide was created for this site location, and the probability density and cumulative distribution functions of tide elevation (CDF shown in Figure 41) were developed. From the tidal CDF, the statistically weighted idealized tide values associated with the lowest 1/16th, next 1/8th, next 1/16th, central 1/2, next 1/16th, next 1/8th, and highest 1/16th were computed. The difference between the high, medium and low idealized tide elevations are the idealized tidal ranges. Table 17 shows the CDF range values, the associated tidal elevations, and cosine approximations.

Table 17: Idealized tidal elevation associated with CDF values

Tide	CDF Range	CDF Average	Elevation (ft.)	Cos. Approx. (ft.)
HL	0-0.0625	0.03125	-3.30	-3.44
ML	0.0625-0.1875	0.125	-2.61	-2.65
LL	0.1875-0.25	0.21875	-2.10	-2.08
M	0.25-0.75	0.5	0.00	0.00
LH	0.75-0.8125	0.78125	2.07	2.08
MH	0.8125-0.9375	0.875	2.69	2.65
HH	0.9375-1	0.96875	3.58	3.44

Three semidiurnal cosine tides were created using the computed representative tidal amplitudes. The tides were then added to the surge elevation time series such that peak surge aligned with high-tide, mid-tide falling, low tide, and mid-tide rising. The combination of the three tides at the four tidal phases resulted in 12 plausible total water elevation time series for each representative storm. Figure 42 shows the storm surge hydrograph for storm 623 (black line) and the plausible total water level hydrographs corresponding to the three estimated tidal amplitudes when peak surge occurs at high tide.

Specification of Storm Seasons

The North Atlantic Coastal Comprehensive Study (NACCS) reports that the tropical storm season spans 6 months from June-November, with the distribution of storms as follows: June-0.04, July-0.04, August-0.26, September-0.48, October-0.12, and November-0.06. The probability of a storm occurring in a given month is defined as the storm distribution multiplied by the storm occurrence rate. The storm occurrence rate is provided by CHS as an attribute of the CHS Save Point.

The extra-tropical storm season spans October-March and a uniform distribution of storm occurrence across the six month season is assumed. The extratropical storm occurrence rate or average number of storms per year is calculated as the number of storms above the threshold divided by the number of years spanned (1938-2012). The probability of a storm occurring in a given month is the rate of storm occurrence divided by the number of months in the extratropical storm season (6). The tropical and extratropical storm seasons are summarized in Table 18.

Table 18: Summary of tropical and extra-tropical storm seasons

Month	Probability of Tropical	Probability of Extra-Tropical
January	0	0.171111111
February	0	0.171111111
March	0	0.171111111
April	0	0
May	0	0
June	0.007008	0
July	0.007008	0
August	0.045552	0
September	0.084096	0
October	0.021024	0.171111111
November	0.010512	0.171111111
December	0	0.171111111

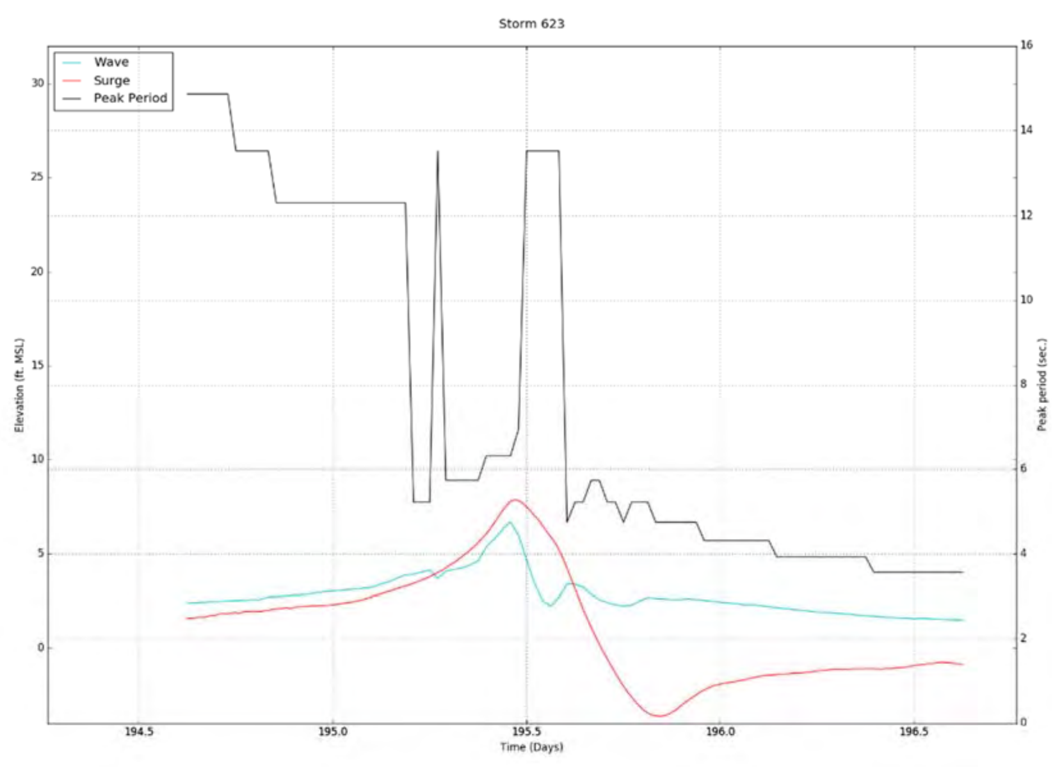


Figure 39: Plot of Surge, Wave and Peak Period Time Series

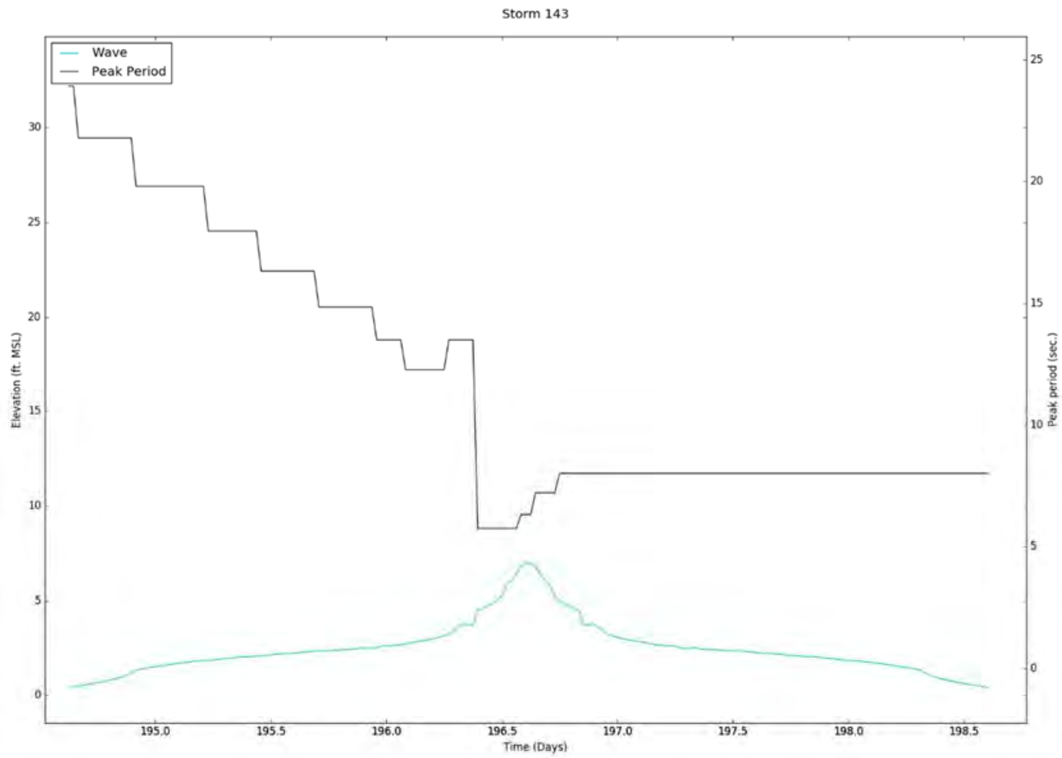


Figure 40: Plot of reflected wave time series and peak period at 8 seconds

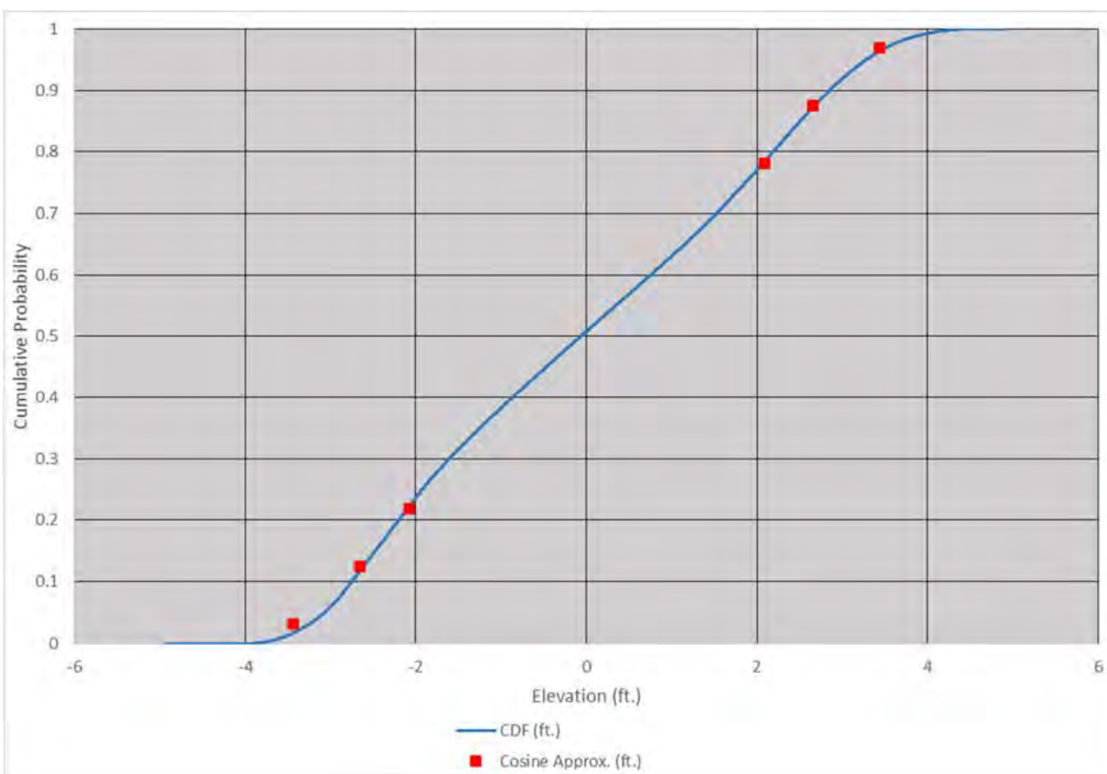


Figure 41: CDF and Cosine Approximation of Tides

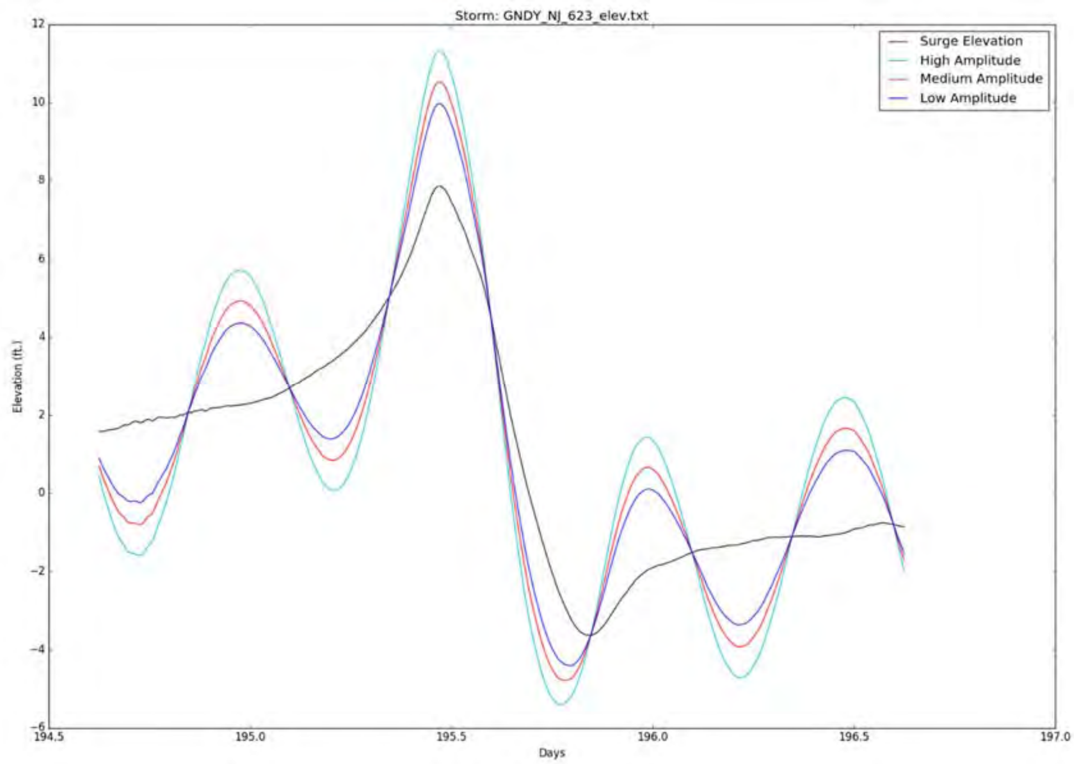


Figure 42: Three tidal amplitudes combined with surge at high tide

CAPE MAY COUNTY

A separate storm suite was developed for the Cape May County sites (Reeds Beach, Pierces Point, Del Haven, and Villas) using ADCIRC Save Point 13425. The approach applied for Cape May is similar to the approach described above for Gandys Beach and Fortescue. The primary difference is that the storm bins and selection of representative storms was completed manually. After clustering the storms based on peak storm surge, time series of storm surge values for storms within each cluster were aligned at the peak and examined to select representative storms for each cluster. The first 6 tropical storm surge bins were divided into short and long duration storms. Figure 43 shows the aligned storm surge hydrographs for the 100-Yr return period cluster with the red bold lines depicting the representative storms.

The final storm suite, shown in Table 19 and Table 20, includes 10 extratropical storms and 19 tropical storms. Relative storm probabilities were calculated using the same approach as Downe Township and the storm seasons (Table 18) are also the same for Cape May County.

The high medium and low tidal amplitudes (3.0, 2.23 and 1.74 ft) were obtained from 20-year-long equilibrium tide at National Oceanic and Atmospheric Administration (NOAA) station 855889, Brandywine Shoal Light, DE. CHS provided conversion factor from MSL to NAVD88 of -0.354 at ADCIRC Save Point 13425.

Table 19: Extra-Tropical Storms and Probabilities – Cape May

Storm Surge (ft)	Wave Height (ft)	No. of Storms	Representative Storms ID	Storm Name	Relative Probability
> 4	> 4	2	7	1950-11-25	1.0
	< 4	3	17	1962-03-07	1.5
3.5-4	> 4	2	27	1972-02-19	1.0
	< 4	7	76	1998-01-28	3.5
3-3.5	> 5	1	26	1972-02-04	0.5
	< 5	16	11	1953-11-07	8.0
2.5-3	> 5	1	34	1977-10-14	0.5
	< 5	19	53	1987-01-02	9.5
2-2.5	> 4	8	6	1947-03-02	4.0
	< 4	10	67	1994-12-24	5.0

Table 20: Tropical Storms and Probabilities – Cape May

Return Period (yr)	Storm Surge (ft)	No. of Storms	Representative Storms ID	Relative Probability
2	3.35	62	360	33.719
			243	33.719
5	4.20	81	362	31.559
			545	31.559
10	4.66	46	399	18.077
			530	18.077
20	5.09	79	300	24.496
			549	24.496
50	6.14	62	209	23.156
			222	23.156
100	7.28	48	170	10.194
			528	10.194
200	8.23	17	38	2.279
			193	2.279
500	9.22	15	45	7.535
1,000	9.84	9	196	2.869
2,000	10.43	6	168	1.000
5,000	11.12	1	36	0.058
10,000	11.65	3	43	0.264

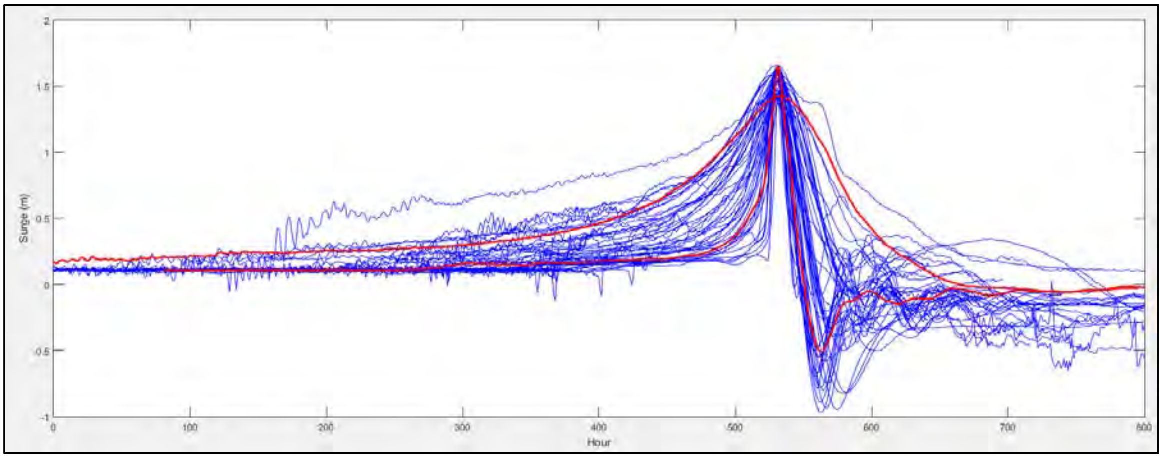


Figure 43: Hydrographs for the 100-Yr return period cluster (selected storms in red)

4.3 SBEACH MODELING

SBEACH OVERVIEW

Storm-Induced BEAch CHange Model, SBEACH, is a one-dimensional model, developed by the United States Army Corps of Engineers (Larson and Kraus 1989, Larson et al. 1989), which simulates cross-shore erosion of beaches, berms, and dunes under storm water levels and waves. SBEACH calculates beach profile change using an empirical morphologic approach with emphasis on beach and dune erosion. In model simulations, the beach profile progresses to an equilibrium state based on the initial profile, median grain size, and storm conditions (wave height, wave period, wave condition, wind speed and direction, and water level). The model also simulates overwash and dune lowering.

SBEACH is primarily used in this study to build the Beach-fx Storm Response Database (SRD). The SRD is a lookup table that stores the morphological profile responses (i.e. change in berm width and dune width/height) and damage driving parameters (i.e. wave height, water level, and vertical erosion). The SRD is based on approximately over a million SBEACH simulations for a range or possible beach profile configurations and storm conditions.

SBEACH MODEL SETTINGS

SBEACH model settings, Table 21, are the same as used in the Delaware Dredge Material Utilization (DE DMU) Study and are based on ERDC-CHL past applications and experiences. Model settings were validated based on Hurricane Sandy observations as described in the section below.

Table 21: SBEACH Model Settings

SBEACH Parameter	Value
Landward surf zone depth (ft)	1
Effective grain size (mm)	0.33
Maximum slope prior to avalanching (deg)	30
Transport rate coefficient (m^4/N)	$1.5e^{-6}$
Overwash transport parameter (K_B)	0.001
Coefficient for slope-dependent term (m^2/s)	0.002
Transport rate decay coefficient multiplier	0.5
Water temperature (°C)	20

SBEACH simulations were performed on using variable grid spacing that generally uses 2 ft grid cells from the landward boundary to the 0 ft contour, 5 ft grid cells from the 0 ft contour to about the -4 ft contour, 10 ft grid cells from the -4 ft contour to about the -6 ft contour, and then 20 ft grid cells to the seaward end of the profile. Simulations were conducted with a time step of 1-minute and wave height randomization activated with 10% variability.

The only parameter that is different from the DE DMU is the effective grain size (0.33 mm). Geotechnical analysis of beach samples collected in 1995 and subsequent compositing determined that the native mean grain size was 0.31 mm at Villas/Del Haven and 0.33 mm at Reeds Beach/Pierces Point.

An effective grain size of 0.33 mm is applied at Gandys Beach and Fortescue even though sediment samples collected by the Philadelphia District in September 2016 indicate that the existing grain size at these two sites is coarser (D50 of 0.5 mm). Modeling coarse grain sizes in SBEACH (greater than about 0.4 mm) is not recommended unless there is measured data to calibrate the model, which there is not at Gandys Beach and Fortescue. Sediment transport in SBEACH is based on an equilibrium energy dissipation determined from the input grain size, and simulations with coarse sediments could result in concrete-like profile responses, unrealistic for the current study.

HURRICANE SANDY MODEL VALIDATION

SBEACH model validation was completed using available pre- and post-Superstorm Sandy beach profile surveys at three locations in the project area. Superstorm Sandy survey data and observations are available from the New Jersey Beach Profile Network (NJBPN) collected by the Richard Stockton College of NJ Coastal Research Center (2013). Unfortunately, there are not any NJBPN or other profile data available at Gandys Beach or Fortescue suitable for model validation.

Wave and water level boundary conditions for the Superstorm Sandy model simulations were obtained from the NACCS modeling results at stations 13421 (Reeds Beach) and 13425 (Villas and North Cape May). Figure 44 and Figure 45 show the nearshore wave and water level conditions at NACCS station 13421 and 13425 during Superstorm Sandy. While Sandy's storm track and wind orientation may have spared Delaware Bay from the relatively high storm surges observed north of Atlantic City, Sandy generated very large waves in Delaware Bay that were directed at the bay shore of Cape May County.

On November 9th of 2012, the Richard Stockton College of NJ Coastal Research Center collected photographs and surveys to wading depths at profiles #100, #101/#201, and #102 in the project area (Figure 46). Figure 47, Figure 49, and Figure 51 present the pre- and post-sandy photographs and profile surveys at Reeds Beach, Villas, and North Cape May, respectively. NAP was unable to obtain digital records of the November 9th surveys, but digital records for long profile surveys from November 19th and 21st are available and are plotted against the SBEACH model results in Figure 48, Figure 50, and Figure 52.

The reason why the earlier beach profile surveys from November 8th and 9th are included here is to try and best capture the conditions at sites before any major recovery or cleanup efforts were undertaken. However, even by the time of the November 9th survey at Reeds Beach, Figure 47, sand on the roadway had been transferred back to the beach in the form of a series of dune-like piles. By the time of the November 19th survey the piles of sand are even larger and could lead to false conclusion that the dune survived Superstorm Sandy.

Overall, the SBEACH settings previously used in the DE DMU study produced acceptable results. Model results are in very good agreement with observations at Villas, with a slight over-prediction of erosion at Reeds Beach and under-prediction of erosion at North Cape May.

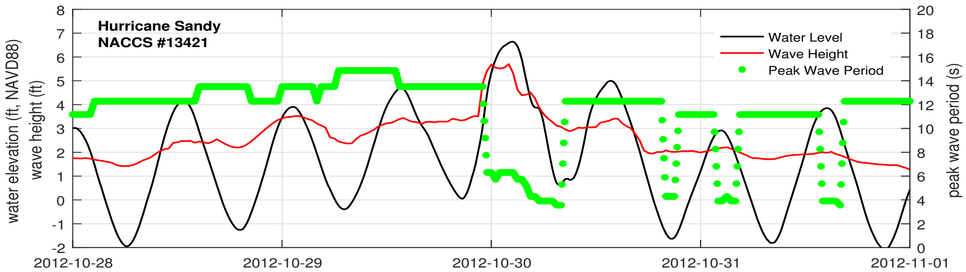


Figure 44: Superstorm Sandy Boundary Conditions at Reeds Beach

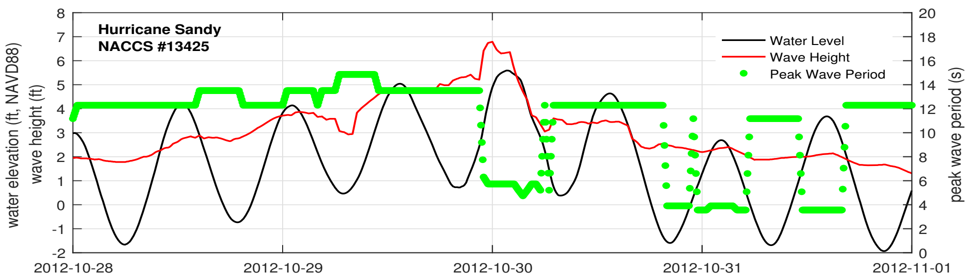


Figure 45: Superstorm Sandy Boundary Conditions at Villas

MATRIX OF SIMULATIONS

Over 1 million SBEACH simulations were performed to create the Storm Response Database (SRD) for Beach-fx. The SRD is a pre-generated set of beach profile responses to storms for the storm suite, and for a range of profile configurations that are expected to exist under different scenarios of storm events and management actions, such as beach nourishment (Gravens et al. 2007). The complete matrix of SBEACH simulations is shown in Attachment C.4. Beach-fx supports non-uniform increments in dune height, dune width, and berm width; however, it was more efficient in this case to setup the model simulations using uniform increments, 5 ft in dune width and 10 ft for berm width. Dune heights range from as low as the upland elevation to as high as + 18 ft NAVD88.

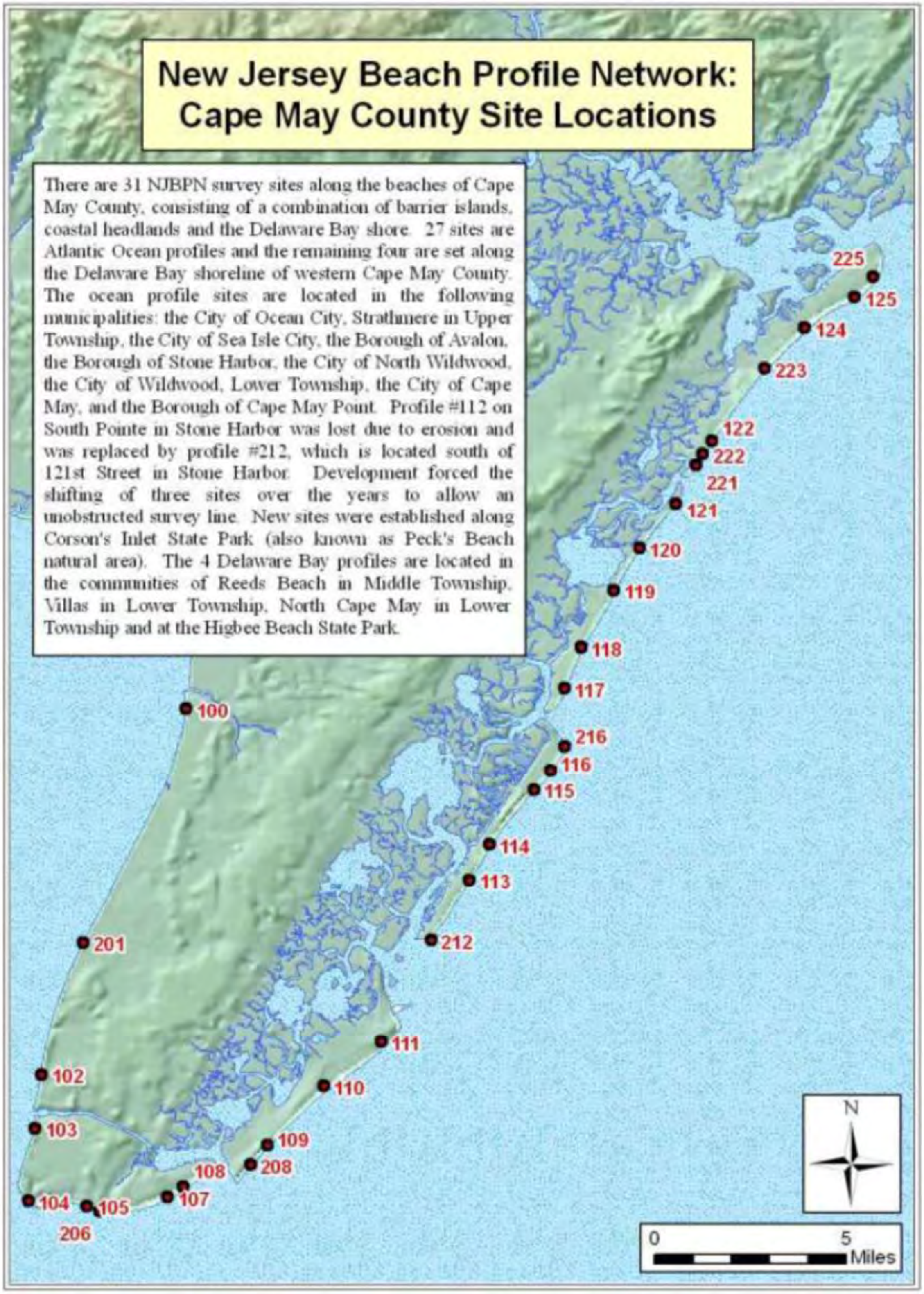


Figure 46: New Jersey Beach Profile Network: Cape May County Locations

The left photograph was taken March 5, 2012 looking south. The dune was vegetated and the beach was higher in elevation compared to the post-Sandy picture on the right taken November 9, 2012. The sand in the center of the right photo has been transferred from the roadway back to the beach as a series of piles.

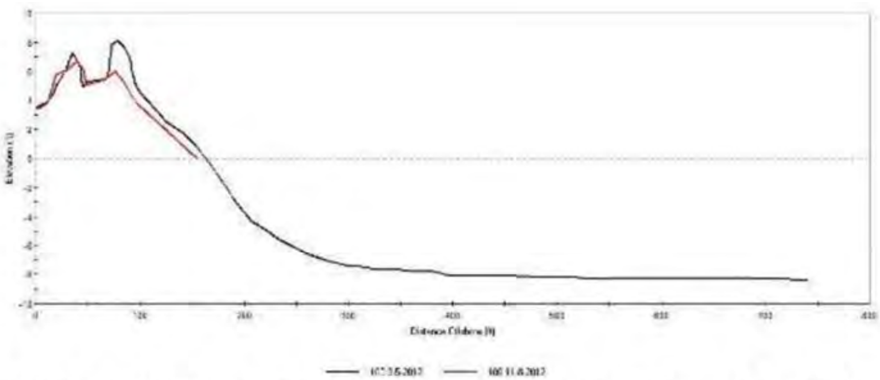


Figure 2. Reeds Beach, Cape May County is located on the western shoreline facing into Delaware Bay. This beach was nourished using dredge material from Bidwell Creek to the north in 2010. The "dune" between the road and the bay was removed and the sand pushed across the road into the salt marsh. The shoreline retreated 8 feet as well. The lost material will not return to the beach except for the material excavated from the roadway.

Figure 47: Reeds Beach (100) NJBPN Superstorm Sandy Observations

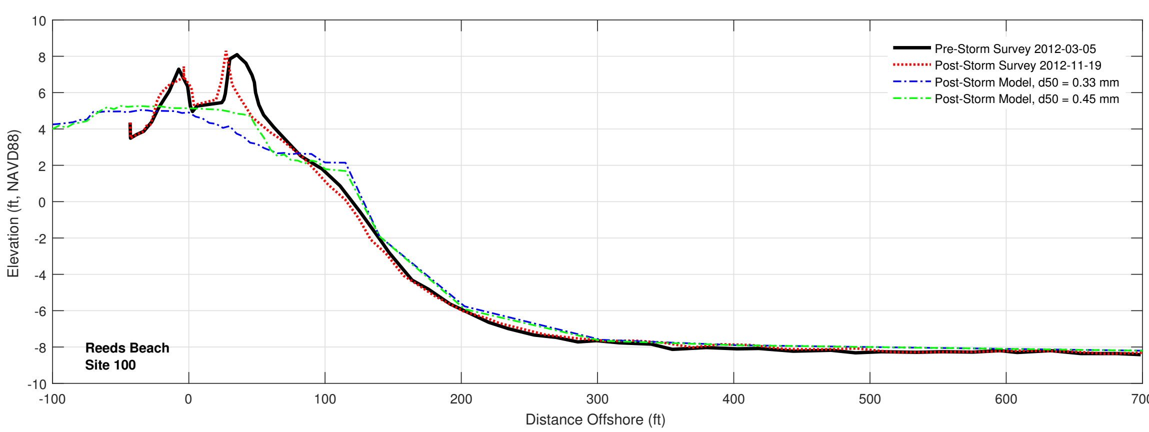
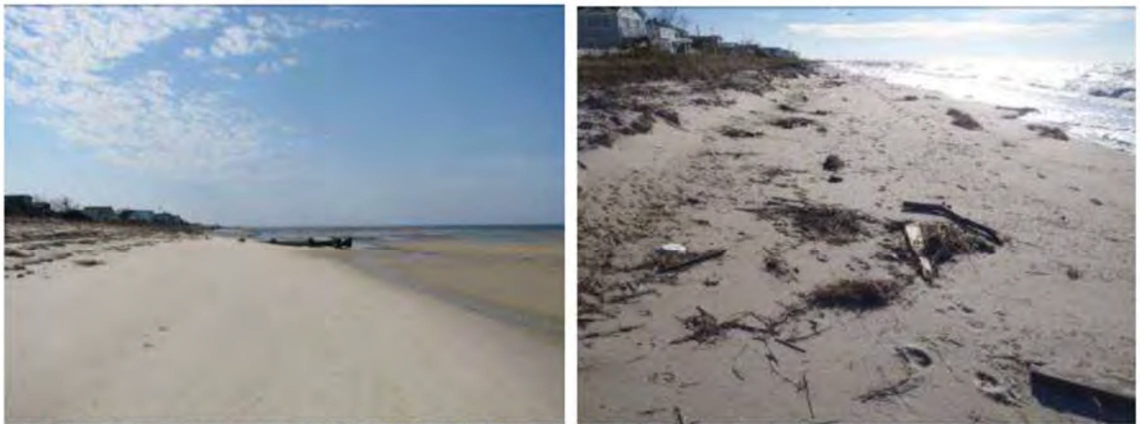


Figure 48: Reeds Beach Superstorm Sandy SBEACH Model Results



The left view was taken March 7, 2012 looking south along the beachfront. By November 9th the storm impact was found to have eroded the beach into the toe of the dune reducing the beach elevation and creating a minor scarp. The height of the uplands bluff prevented local wave or tidal flooding.

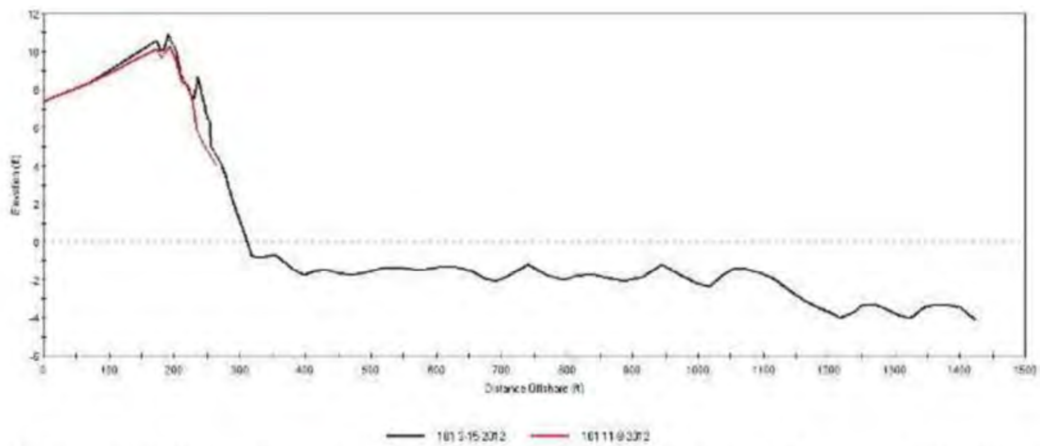


Figure 3. The wide shelf terrace likely saw sediment shifted around, but little erosion vertically appeared to have drastically changed the situation. The small foredune was removed and waves lowered the beach elevation, likely moving sand onto the nearshore terrace segment.

Figure 49: Villas (101) NJBPN Superstorm Sandy Observations

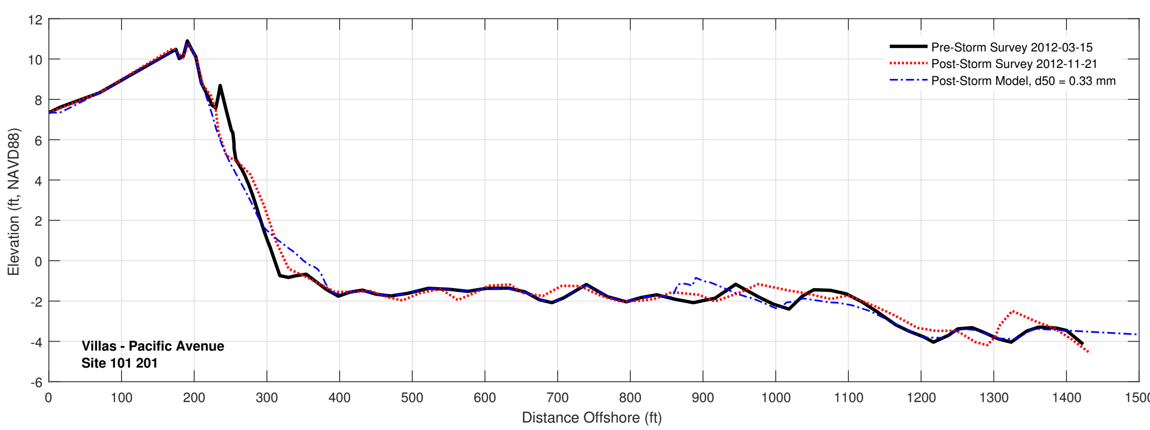


Figure 50: Villas Superstorm Sandy SBEACH Model Results

At Whitter Avenue the drainage line shows the extent of beach erosion between March 15, 2012 and after Sandy on November 9th. Sand was moved offshore onto the terrace, but the dune withstood the majority of the Delaware Bay wave assault on this shoreline.

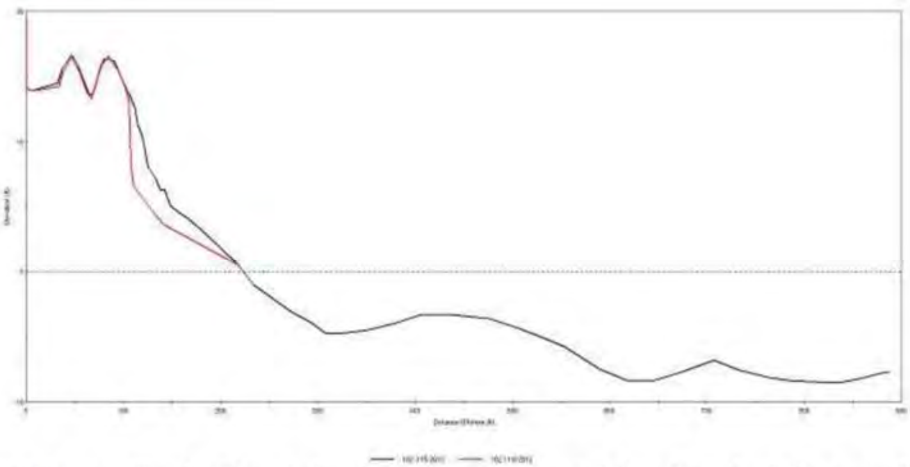


Figure 4. The cross section shows the erosion into the seaward toe of the dune with the post-storm beach slope meeting the pre-storm line at a much lower slope angle. Sand was transported offshore at the expense of the existing beach and dune.

Figure 51: North Cape May (102) NJBPN Superstorm Sandy Observations

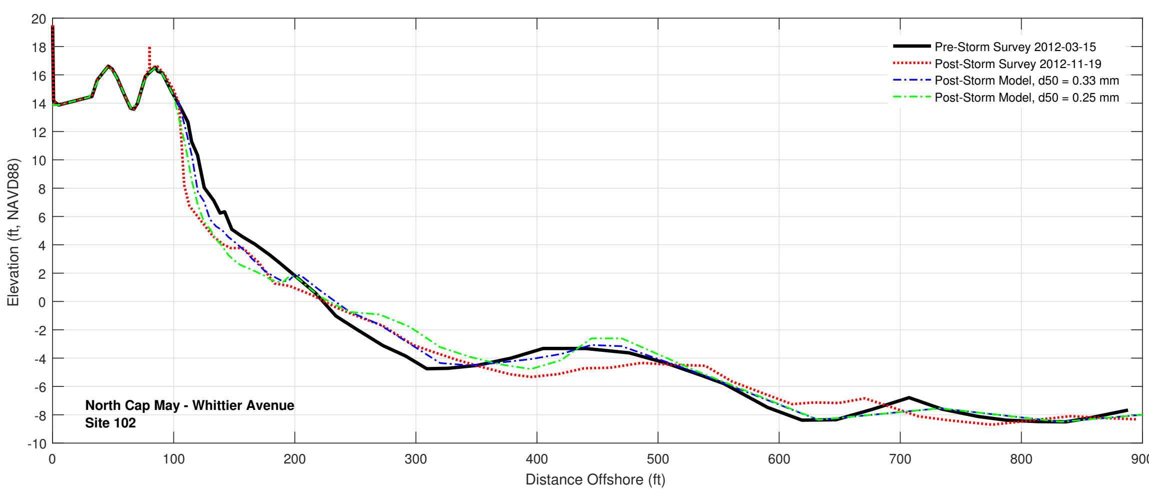


Figure 52: North Cape May (102) Superstorm Sandy SBEACH Model Results

SBEACH MODEL RESULTS

SBEACH modeling results, Figure 53, are presented for the existing conditions at 3 sites: Reeds Beach, Villas North, and Villas South. Hurricane Sandy was selected to show sample SBEACH results even though it is not part of the storm suite because it provides a good frame of reference for evaluating the SBEACH results. The results shown in Figure 53 are all displayed at the same scale to facilitate comparison between sites. The most striking observation is that at Reeds Beach the horizontal erosion is greatest and the entire profile is inundated during the peak of the storm. The existing condition dunes at Villas North and South are high and wide enough to prevent the profile from being inundated. The second observation is that there is considerably more dune erosion at Villas South than Villas North. This is likely due to two factors: (1) Villas North has a 20 ft wide berm that provides a buffer for the dune, and (2) Villas South has a deeper submerged profile allowing larger waves to attack the dune.

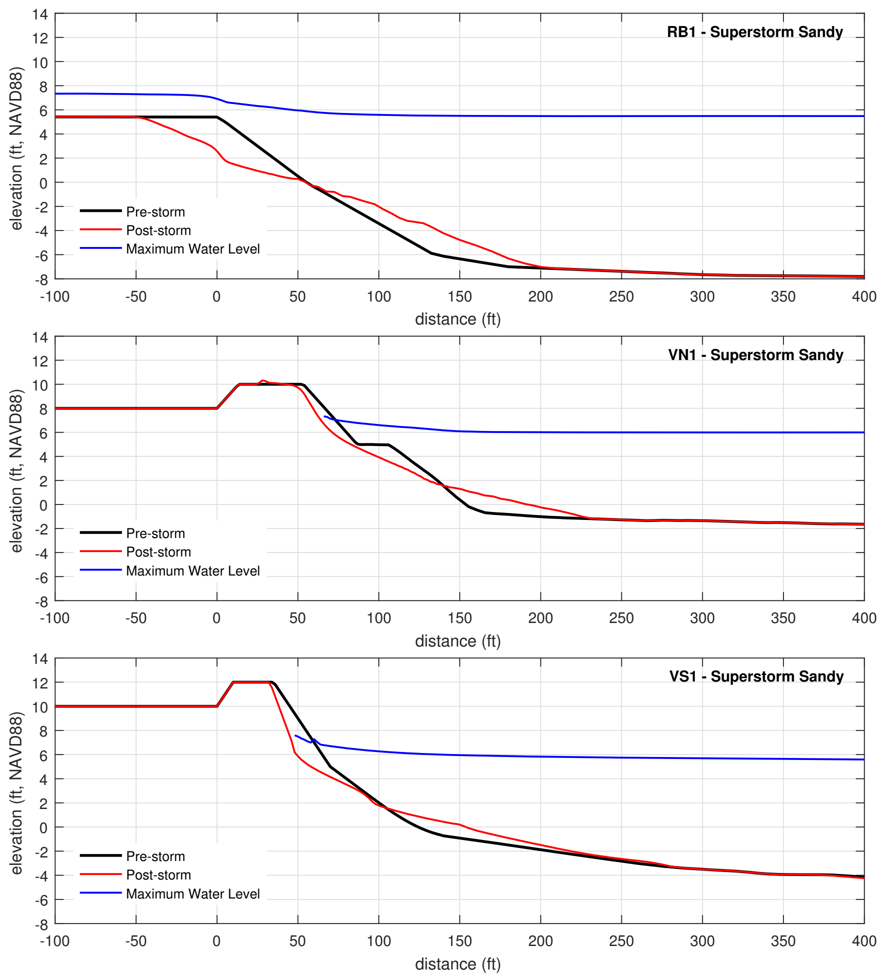


Figure 53: SBEACH Model Results for Superstorm Sandy at Reeds, Villas North, and Villas South

BACK BAY FLOODING

All of the communities evaluated in Beach-fx are also exposed to "back bay flooding," a term used to described flooding occurring from the landward side or marsh side of the communities. Figure 54 shows an aerial image of Reeds Beach, highlighting the flow of water from Delaware Bay into the marshes that border the landward side of the beach sites. Beach restoration alternatives at these sites may reduce erosion damages, wave damages, and even block the flow of water from the seaward side of beach, but they will do nothing to stop back bay flooding. Beach-fx is able to capture back bay flooding by applying single peak water level for each storm event and using the greater value of the two values: (1) seaward water level from SBEACH, and (2) back bay flooding elevation. Peak water elevations for each storm at the nearshore NACCS stations are used to define the back bay flooding elevations in the model.

Figure 54: Conceptualization of Back Bay Flooding at Reeds Beach

4.4 DIFFUSION LOSSES

OVERVIEW

Beach nourishment projects constructed on a long beach represent a perturbation or planform anomaly, which under wave action, will spread out along the shoreline (Dean, 2002). This process is illustrated in Figure 55, which shows waves interacting with the beach nourishment causing sediment transport away from the anomaly and smoothing or spreading out of the sediment (Dean & Grant 1989). The term "spreading out" losses actually refers to a redistribution of the sediment and not a total loss to the system but rather a loss from the region in which the sediment is placed (Dean & Grant 1989). This process is referred to as "beachfill diffusion" since the process is modeled analytically using the one-dimensional diffusion equation, first utilized by Pelnard-Considere (1956). Diffusion losses within the study area could be significant at many of sites and be several times greater than the background erosion rates, thus having an outsized effect on periodic nourishment quantities.

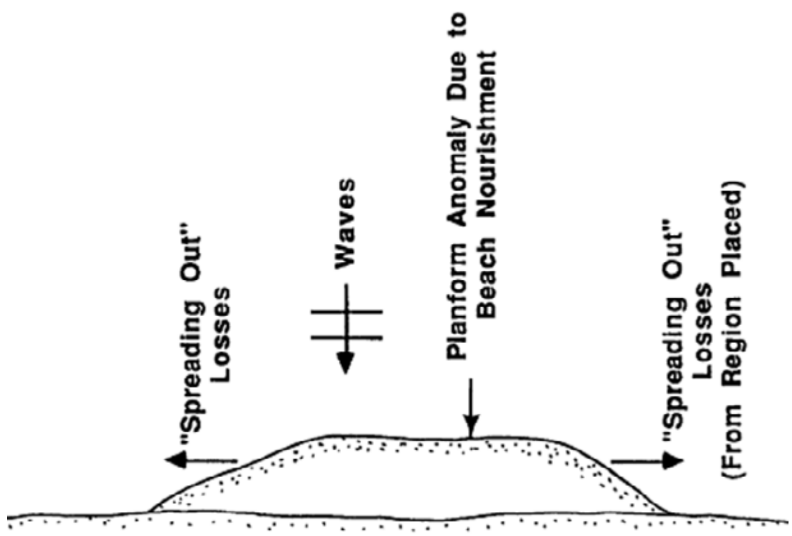


Figure 55: "Spreading Out" losses occurring from diffusion

Beachfill diffusion is modeled analytically in this study using solutions to the Pelnard-Considere equation for a rectangular planform anomaly on an infinitely long shoreline. Losses are primarily a function of the wave energy, alongshore length of beach nourishment, and cross-shore width of planform anomaly. The non-dimensional solution to the equation is shown in Figure 56, where t' is a non-dimensional representation of time based on the ratio of the alongshore length (l) of beach nourishment anomaly, time (t) after construction, and longshore diffusivity (G). The longshore diffusivity is a function of how energetic the wave environment. Figure 56 shows how the planform anomaly spreads out over time. The non-dimensional form of time indicates that rate at which diffusion occurs is a function of the diffusivity and alongshore length. Locations with more wave energy will have a larger longshore diffusivity and t' will increase. Similarly, as the alongshore length decreases, t' increases. An example solution to the Pelnard-Considere equation for a 4,000 foot-long beach nourishment project is shown in Figure 57. The bottom panel of Figure

57 shows the fraction of sand volume remaining and the impact of background erosion, which is linearly added to diffusion losses.

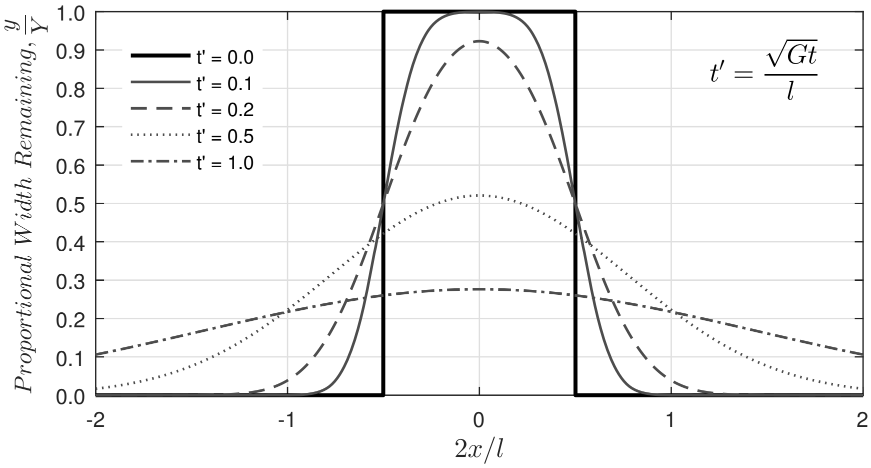


Figure 56: Non-dimensional Shoreline Evolution

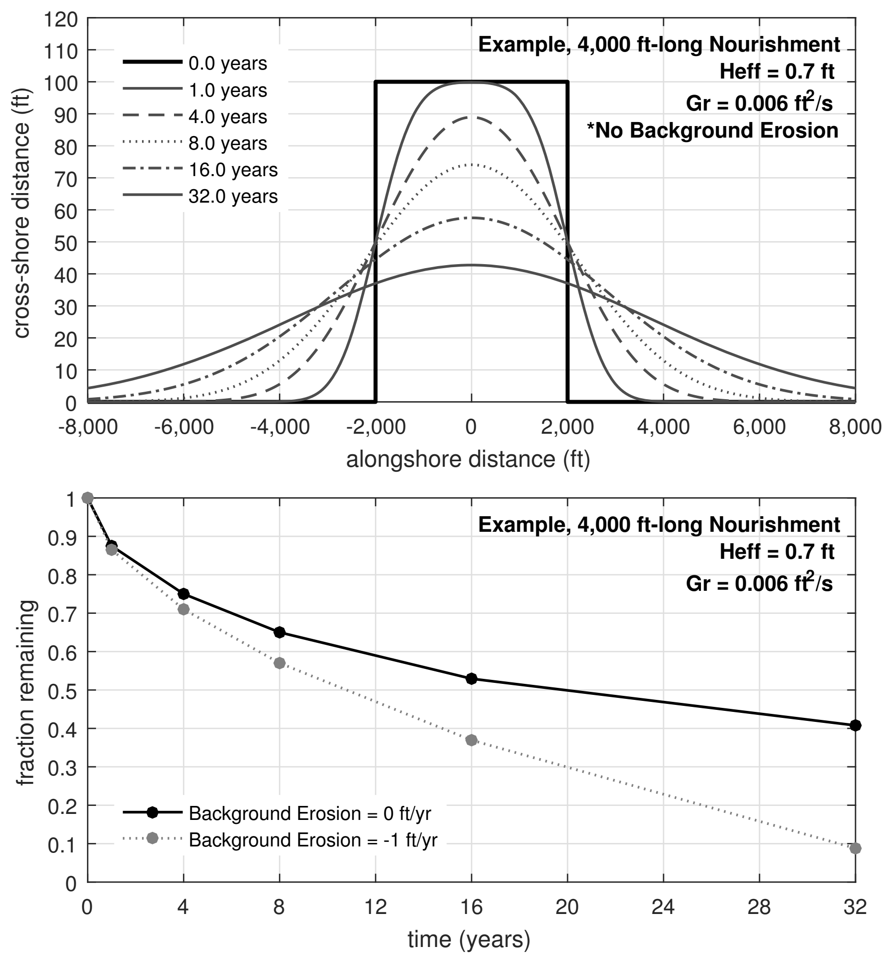


Figure 57: Example of Diffusion Losses at 4,000 foot-long Nourishment Project

EFFECTIVE WAVE HEIGHT

Wave conditions at NOAA NDBC Buoy 44055 located in the middle of Delaware Bay are used to calculate the effective wave height and longshore diffusivity, G. The formula for calculating the effective wave height, H_{eff}, is provided below from Dean and Grant (1989):

$$H_{eff} = \left[\frac{1}{N}\sum_{n=1}^{N}\left(K_s H_{s,n}\right)^{2.4}\right]^{\frac{1}{2.4}}$$

where Hs is the significant wave height, and Ks is the shoaling coefficient and equal to 0.735. An effective wave height of 0.7 feet and representative wave period of 3.4 seconds was calculated for Buoy 44055.

The formula for the longshore diffusivity, G, is also provided below from Dean (2005) including the effects of wave refraction around the planform:

$$G = \frac{K H_b^{\frac{5}{2}}\sqrt{g/\kappa}}{8(s-1)(1-p)(h_* + B)}\frac{C_b}{C_*}$$

Where K is the sediment transport factor (0.78), H_b is the breaking wave height taking here as the effective wave height, g is the acceleration of gravity, κ is the wave breaking index (0.78), s is the sediment specific gravity (2.65), p is the in-place sediment porosity (0.35), h_* is the depth of closure, B is the berm height, C_b is the wave celerity at breaking, and C_* is the wave celerity at the depth of closure. The longshore diffusivity is 0.0143 ft^2/s and 0.0061 ft^2/s at sites with a depth of closure of -6 ft and -3 ft NAVD88 respectively.

TERMINAL GROINS

There are two existing terminal groins or jetties in the study area: (1) northern end of Reeds Beach, and (2) northern end of Fortescue Creek. A third terminal groin is proposed at the northern end of Gandys Beach, the justification for this groin is provided in Section 5.3. These three terminal groins would significantly reduce diffusion losses. Dean & Grant (1989) describe a simple approach to incorporate terminal groins in the diffusion analysis. The recommended approach for a single terminal groin is to increase the effective length of the nourishment to twice the physical length of the project and apply background erosion rates that account for the influence of the terminal groin. By doubling the effective length of the nourishment, diffusion losses are cut in half. The same approach, doubling the effective length of the project, was also applied at the southern end of Del Haven where the nourishment project would tie-into the adjacent nourishment project at Villas.

APPLICATION TO PROJECT AREA

Diffusion calculations were first performed at the sites based on the planform anomaly length (i.e. alongshore length of nourishment) and a range of possible planform anomaly widths (ΔY). This analysis resulted in a site-specific lookup table relating planform anomaly widths to diffusion rates, where the diffusion rate represents the average loss over the entire nourished beach after 4 years. At the time the diffusion analysis was completed, the anticipated periodic nourishment cycle was 4 years.

After generating the lookup table of diffusion rates, the next step was to determine the planform anomaly width for different With-Project alternatives. The planform anomaly width is measured as the cross-shore difference between the existing condition shoreline position and With-Project shoreline position. To simplify the analysis, the representative profile from Beach-fx was used to represent the existing conditions. Figure 58 and Figure 59 show an example of the process used to determine the planform anomaly width for various With-Project alternatives at Gandys Beach, Fortescue and Villas South.

The results of the diffusion analysis for the tentatively optimized With-Project alternative is presented in Table 22.

Table 22: Diffusion Results

Site	Design	Length (ft)	ΔY (ft)	Diffusion (ft/yr)
Gandys	WP6.5B50	2,890[1]	50	-4.4
Fortescue	WP6.5B50	4,400[1]	50	-2.8
Reeds	WP5.5B50	5,300[1]	50	-1.2
Pierces	WP6B50	3,000	50	-5.9
Del Haven	WP8B50	5,600[1]	50	-1.7
Villas North	WP8B50	16,400[2]	45	-1.0
Villas South	WP12B50a	16,400[2]	50	-1.2

[1]Effective Length is twice as long due to terminal groins and adjacent projects
[2]Entire length of Villas was used in calculations with design specific planform anomalies (ΔY).

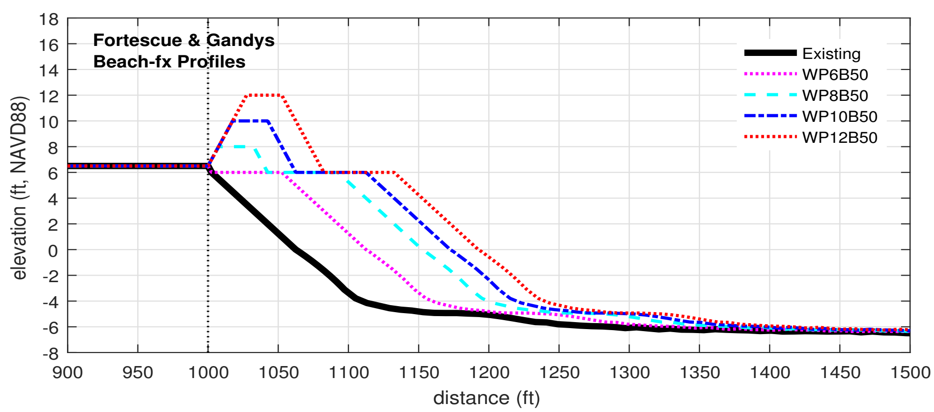


Figure 58: Beach Nourishment Alternatives at Gandys Beach and Fortescue

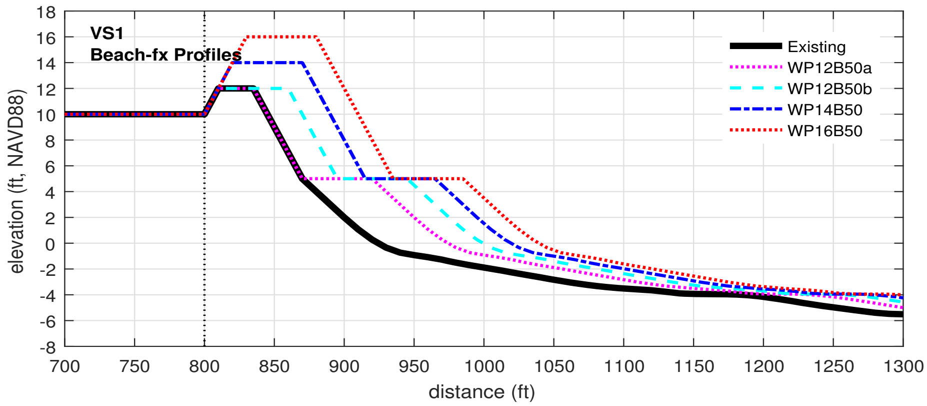


Figure 59: Beach Nourishment Alternatives at Villas South

4.5 SEA LEVEL CHANGE IMPLEMENTATION IN BEACH-FX

Mark Gravens (2011) provides a detailed description of how sea level change (SLC) is implemented in Beach-fx. A brief overview of this paper is provided here as well as a discussion of the site specific inputs for the NJ DMU.

SLC is implemented in Beach-fx based on four assumptions:

1. Natural berm elevation will rise in concert with rising sea surface (supports #2);
2. Pre-computed beach profile responses in Shore Response Database are equally valid at the end of the project life cycle as they are at the beginning of the project life cycle;
3. Water surface and wave elevations may be incrementally increased by an amount equal to the estimate amount of SLC;
4. Bruun Rule (1962) may be used to estimate additional shoreline recession associated with SLC.

Based on these assumptions, Beach-fx only requires two additional site specific inputs to evaluate the effect of sea level change, historic rate of SLC, and average profile slope over active beach. A third parameter, G_a, may also be included in the Bruun Rule calculation to account for the loss of fines from an eroding upland. Beach-fx has its own internal sea level change calculator, consistent with ER 1100-2-8162, and is able to calculate the mean sea level at any point in time for all three SLC scenarios (Low, Intermediate, and High). The historic rate of SLC in the study area is +0.0105 ft/yr (Lewes, DE). The average profile slope over the active beach profile, ϑ, was estimated to be 1V:30H based on profile surveys in the project area. G_a was set to the default value of 1.0 for this study.

Shoreline recession associated with SLC is modeled in Beach-fx after Bruun (1962).

$$R = \frac{S}{\theta} G_a$$

S = change in sea level

θ = average profile slope over active beach profile

R = horizontal recession of beach

G_a = factor relating volume of eroded material required to yield a unit volume of compatible beach sand, accounting for the loss of fines from eroding upland

Application of the Bruun Rule to the study area, Table 23, reveals that historic rate of sea level change is responsible for 0.3 feet of background shoreline erosion per year. The 0.3 feet of shoreline erosion associated with the historic rate of SLC is a component of historical background erosion rate. Therefore, the potential impact of SLC in the Intermediate and High SLC scenarios is the net increase in shoreline change relative to the historic rate (Δ Shoreline Change). Table 23 shows that the Intermediate and High SLC scenarios could increase shoreline erosion by 0.3 ft/yr and 1.3 ft/yr, respectively.

Table 23: Bruun Rule Results

SLC Scenario	SLC[1] (ft)	Shoreline Recession (ft)	Shoreline Change (ft/yr)	Δ Shoreline Change[2] (ft/yr)
Low/Historic	0.53	-16	-0.3	0.0
Int.	1.00	-30	-0.6	-0.3
High	2.48	-74	-1.5	-1.3

[1]Projected sea level change from 2020 to 2070.

[2]Increase in shoreline change relative to historical background erosion.

5.0 BEACH-FX OUTPUT DATA

5.1 MORPHOLOGICAL EVOLUTION

Morphological results from the Future Without Project (FWOP) Beach-fx model simulations are presented in this section. A detailed discussion of Beach-fx and the economic results are presented in the Main Report and Economics Appendix. The focus of this section is to verify that the morphological evolution simulation in Beach-fx is consistent with the inputs described in the previous section.

Beach-fx simulates profile morphology changes through five mechanisms:

1. Storm-induced morphology change based on SBEACH model results stored in SRD
2. Post-storm berm width recovery
3. SLC-induced shoreline change (Bruun Rule)
4. Applied shoreline change rate
5. Project-induced shoreline change (e.g. diffusion losses)

Together, the first four factors make up the long-term background erosion rate. For this study, the applied shoreline change rate was used as a calibration parameter to ensure that Beach-fx is reproducing the long-term background erosion rates (Table 11). It is noted that model calibration is performed under the Low SLC scenario. A berm width recovery factor of 95%, which is fairly standard value, was applied in this study. The 5th factor, project-induced shoreline change, was applied in the With-Project simulations and set equal to the diffusion losses (Table 22).

The FWOP Beach-fx model results for all 300 lifecycle simulations are presented here for the Intermediate SLC scenario. Each lifecycle simulation is performed over a 55-year period from 2017 to 2072. Figure 60 to Figure 69 show the existing condition profile in 2017 and FWOP profile at the end of the simulation in 2072. The light red lines represent the FWOP profile at the end of each iteration. The thick red line represents the average profile in 2072. Not surprisingly, the greatest erosion is observed at the sites with the highest background erosion rates: Gandys Beach, Fortescue, and Villas South. Reeds Beach and Pierces Point also experience significant erosion. Since the background erosion rates at Del Haven and Villas North are stable, it is not surprising the Beach-fx simulations show fairly stable conditions. Figure 60 to Figure 69 do not capture the post-storm conditions, so it is possible that the profiles eroded back even further during a storm event before recovering (i.e. 95% berm width recovery factor).

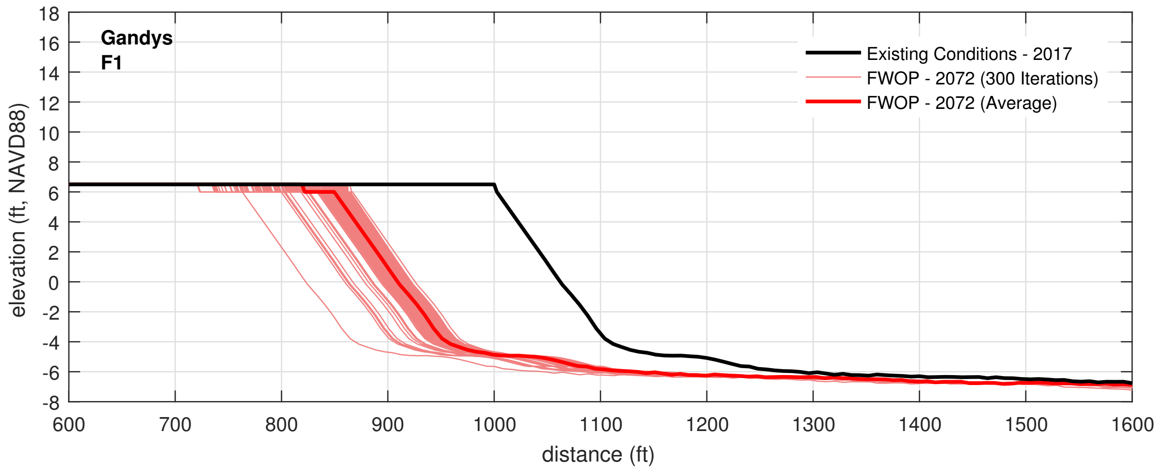


Figure 60: FWOP Morphology at Gandys Beach (F1)

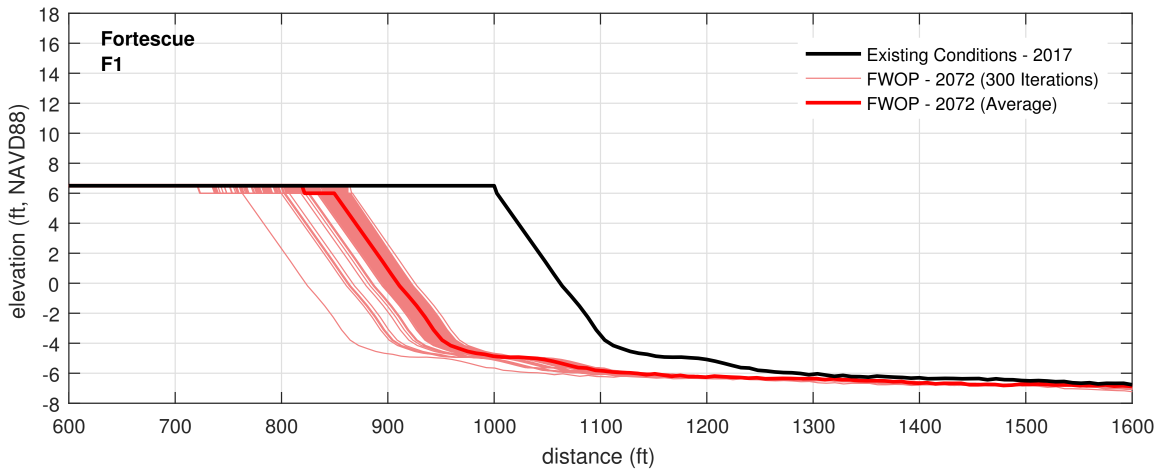


Figure 61: FWOP Morphology at Fortescue (F1)

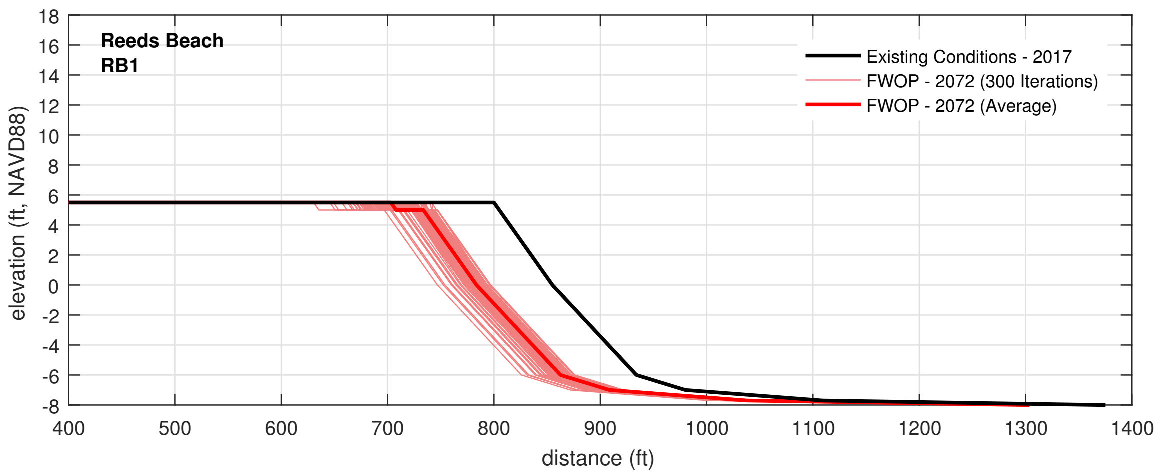


Figure 62: FWOP Morphology at Reeds Beach (RB1)

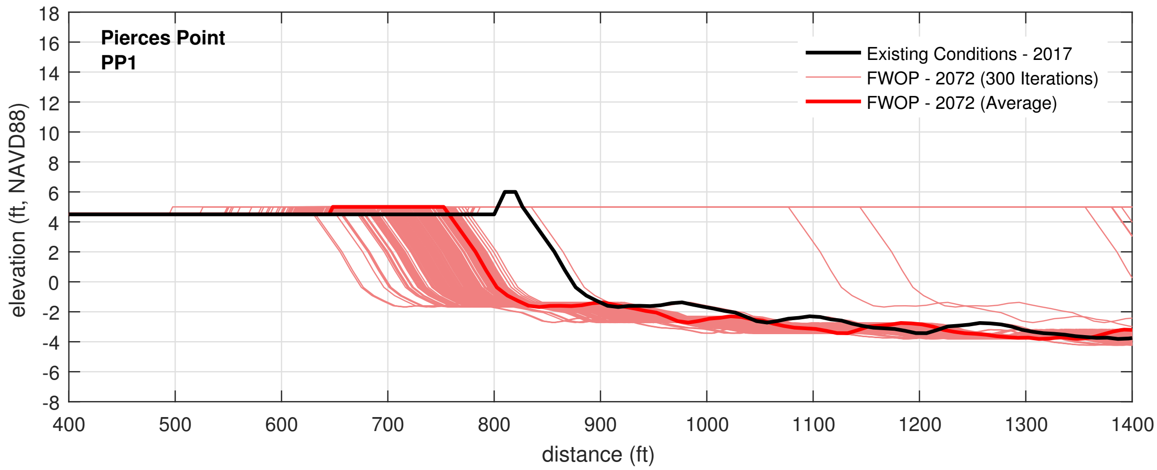


Figure 63: FWOP Morphology at Pierces Point (PP1)

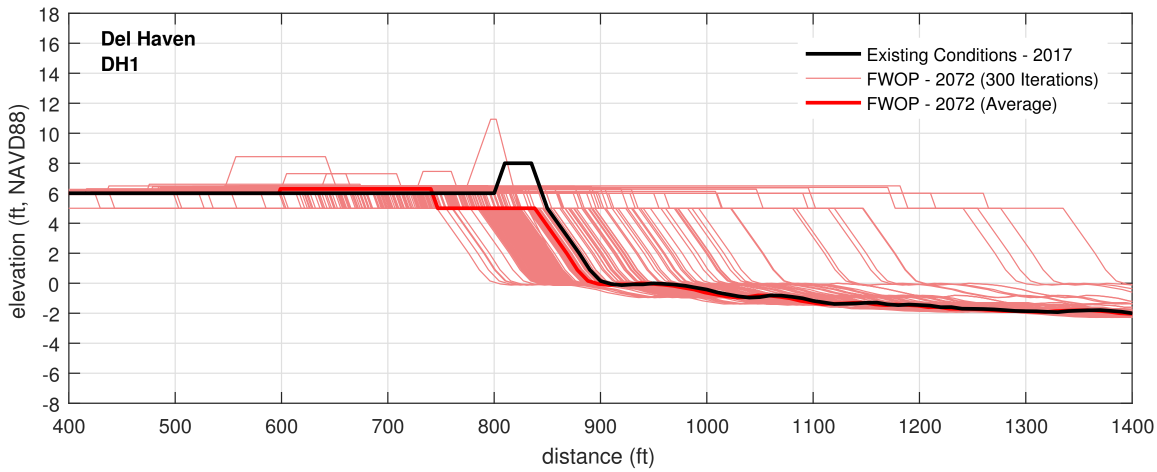


Figure 64: FWOP Morphology at Del Haven (DH1)

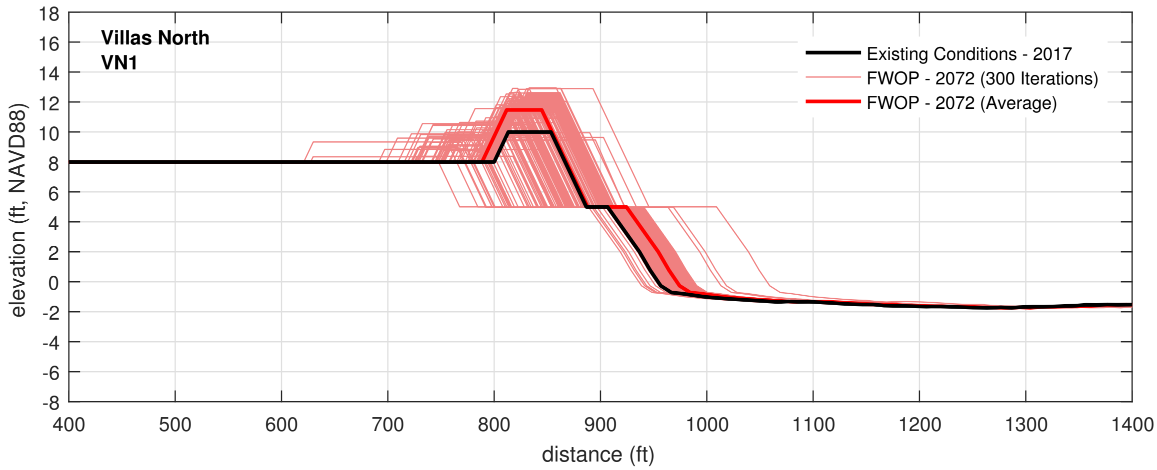


Figure 65: FWOP Morphology at Villas North (VN1)

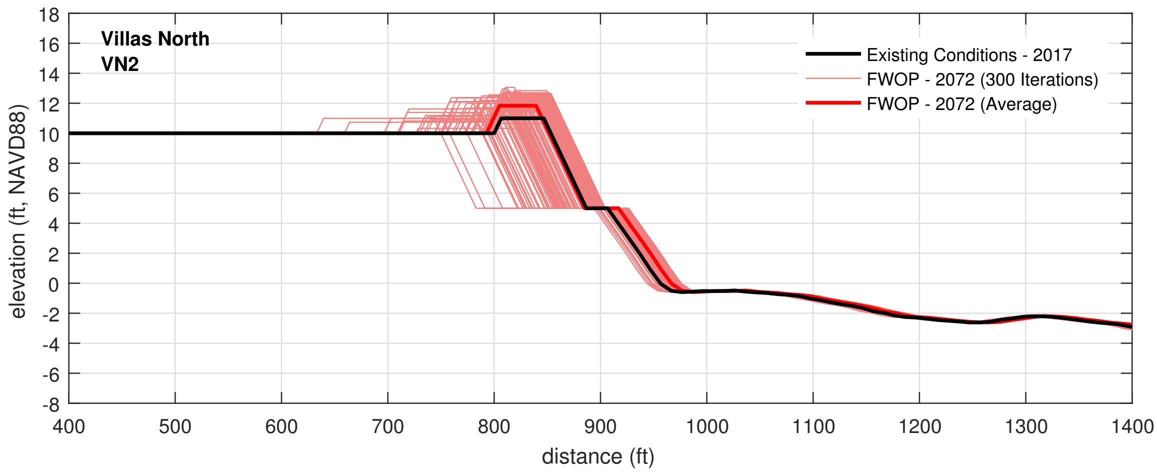


Figure 66: FWOP Morphology at Villas North (VN2)

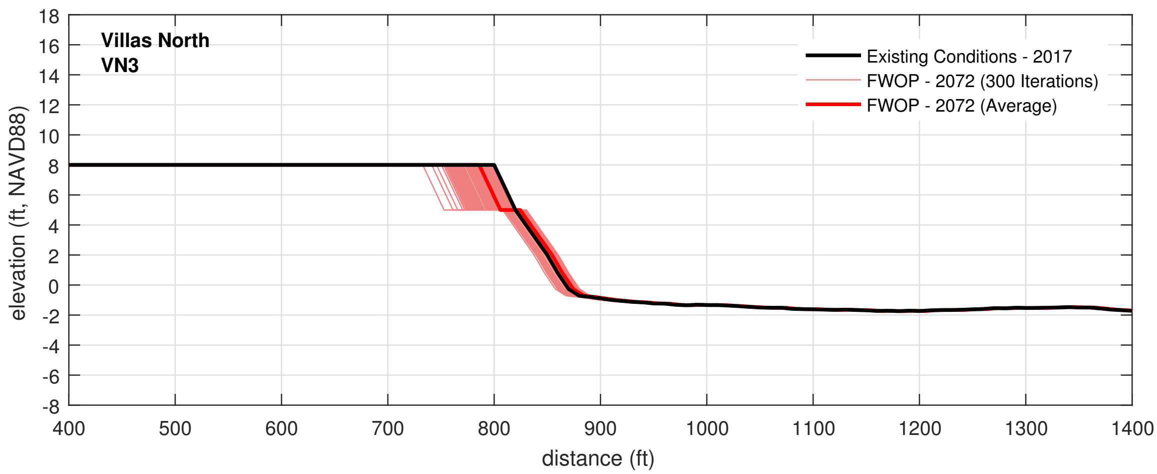


Figure 67: FWOP Morphology at Villas North (VN3)

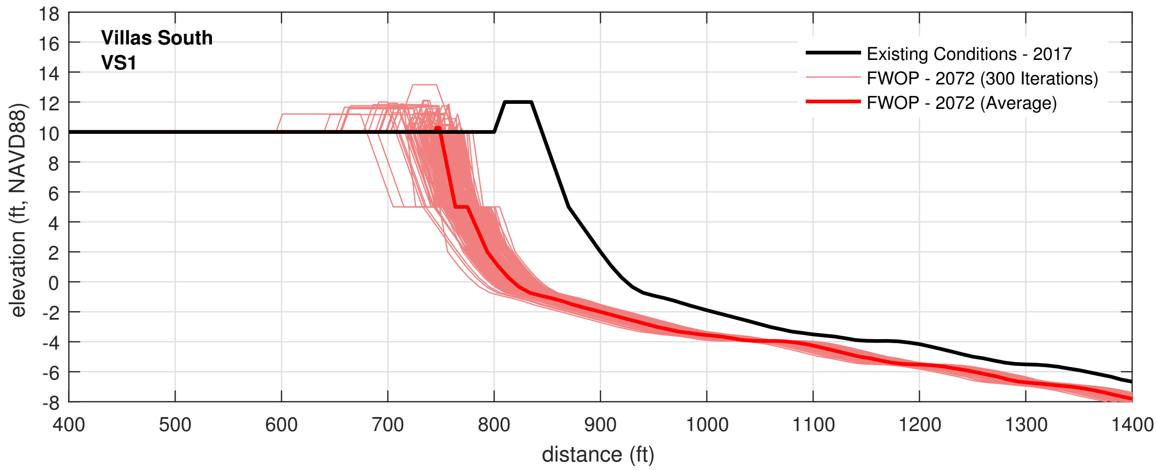


Figure 68: FWOP Morphology at Villas South (VS1)

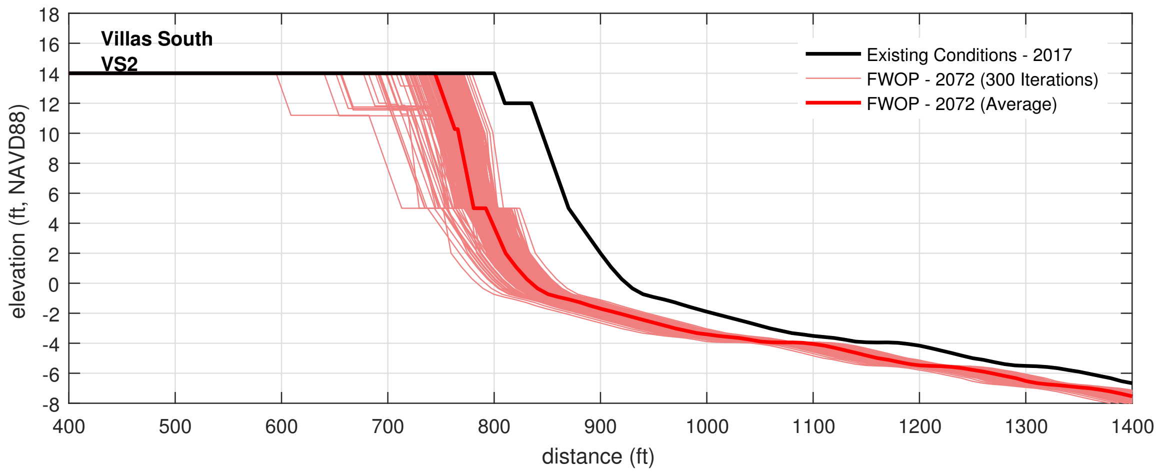


Figure 69: FWOP Morphology at Villas South (VS2)

5.2 NOURISHMENT FILL QUANTITIES

INITIAL CONSTRUCTION

A comparison of the Civil Engineering estimate of initial construction quantities and the calculated number in Beach-fx is presented in Table 24. Considering that Beach-fx is based on simplified representation of the profile, and that in some instances a single profile is used to represent an entire project site, the agreement between the Civil Engineering estimate and Beach-fx is considered good. The largest differences occur at Pierces where the Civil Engineering estimate is based on a shorter project length of approximately 2,000 feet and Del Haven where the Civil Engineering estimate was completed for a +12 ft NAVD88 dune, not the +8 ft NAVD88 dune modeled in Beach-fx.

Table 24: Initial Construction Quantities – Beach-fx Result

<table>
<tr><th>Site</th><th>Reach Length (ft)</th><th>Engineering Estimate (cy)</th><th>Beach-fx Result (cy)</th></tr>
<tr><td>Gandys</td><td>3,100</td><td>145,000</td><td>114,000</td></tr>
<tr><td>Fortescue</td><td>5,590</td><td>193,000</td><td>205,000</td></tr>
<tr><td>Reeds</td><td>4,840</td><td>264,000</td><td>155,000</td></tr>
<tr><td>Pierces</td><td>5,900</td><td>65,000[2]</td><td>124,000</td></tr>
<tr><td>Del Haven</td><td>5,290</td><td>287,000[1]</td><td>105,000[1]</td></tr>
<tr><td>Villas North</td><td>8,140</td><td rowspan="2">470,000</td><td>173,000</td></tr>
<tr><td>Villas South</td><td>8,515</td><td>314,000</td></tr>
</table>

[1] Engineering estimate for Del Haven is based on design dune at +12 ft NAVD88, Beach-fx design template has dune at +8 ft NAVD88.

[2] Engineering estimate for Pierces Point is based on a 2,000-ft long project

PERIODIC NOURISHMENT

Table 25 presents the periodic nourishment fill quantities for an 8-year dredging cycle at each site based on the background erosion rate, diffusion losses, and SLC-induced erosion. An 8-year dredging cycle was selected to reduce project costs. At shorter dredging cycles, the required periodic nourishment fill quantities became relatively small for a dredging operation and it may not be sensible to mobilize a dredge.

A comparison of the engineering estimate and Beach-fx result is shown in Table 26. Since Beach-fx is calibrated to the historical background erosion rate, it is not surprising that two estimates are in very good agreement.

Table 25: Periodic Nourishment Quantities – Engineering Estimate

Site	Length (ft)	Background Erosion (ft/yr)	Diffusion Losses (ft/yr)	Int. SLC Erosion (ft/yr)	Periodic Nourishment (cy/operation)
Gandys	3,100	-2.5	-4.4	-0.3	79,000
Fortescue	5,590	-2.5	-2.8	-0.3	111,000
Reeds	4,840	-1	-1.2	-0.3	39,000
Pierces	5,900	-1	-5.9	-0.3	101,000
Del Haven	5,290	0	-1.7	-0.3	25,000
Villas North	8,140	+0.5	-1.0	-0.3	15,000
Villas South	8,515	-1.5	-1.2	-0.3	83,000

Table 26: Periodic Nourishment Quantities – Beach-fx Result

Site	Length (ft)	Engineering Estimate (cy/operation)	Beach-fx Result (cy/operation)
Gandys	3,100	79,000	80,000
Fortescue	5,590	111,000	113,000
Reeds	4,840	39,000	40,000
Pierces	5,900	101,000	101,000
Del Haven	5,290	25,000	19,000
Villas North	8,140	15,000	20,000
Villas South	8,515	83,000	115,000

5.3 TERMINAL GROIN JUSTIFICATION

Previous studies at Gandys Beach and Fortescue have identified the need for coastal structures as a complementary component of any beach restoration project due to the sediment deficit in the system, high background erosion rates, and diffusion losses (Hatch Mott McDonald, 2016). A terminal groin at the northern end of Gandys Beach, Figure 70, is absolutely critical to anchoring the beach restoration project and limiting end losses. The existing terminal groin at the northern end of Fortescue, Figure 71, is in poor condition and will be too short to effectively limit sediment transport into Fortescue Creek.

A simple analysis was performed to show the cost of the terminal groins at Gandys Beach and Fortescue is far less than additional nourishment costs without the terminal groins. As discussed in Section 4.4, the diffusion losses at Gandys Beach and Fortescue would be twice as high without the terminal groins, resulting in higher beach fill placement quantities and possibly a shorter nourishment cycle (not included in this analysis). The analysis shows that the cost of the terminal groin, $700,000, is paid back during the first nourishment cycle (8 years). The additional diffusion losses at Gandys Beach and Fortescue, 48,500 cy/cycle and 55,700 cycle, would cost $1,586,000 and $2,131,000 respectively during the first nourishment operation. Over a 50-year project life, the cumulative savings on periodic nourishment would greatly exceed the initial cost of the terminal groins.

Figure 70: Terminal Groin Location at Gandys Beach

Figure 71: Terminal Groin Location at Fortescue

6.0 CONCLUSIONS

This Hydrology, Hydraulics, and Coastal Appendix details the technical analyses supporting the New Jersey Dredge Material Utilization (DMU) Coastal Storm Risk Management Feasibility Study. The majority of coastal work focused on supporting the Beach-fx modeling effort. The SBEACH modeling work, shoreline change rates, and diffusion losses are critical components of Beach-fx that ultimately drive the economic damages, beach restoration quantities and costs, and plan selection. Several recommendations to the PDT were made based on the HHC technical analyses:

- Extend beach nourishment cycle from 4 years to 8 years to reduce project costs;
- Add terminal groins at Gandys Beach and Fortescue to reduce project costs over 50 years;
- Split Villas into two separate sites, Villas North and South, based on distinct differences in topo-bathymetric conditions and historical shoreline changes.

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ATTACHMENT C.1 SHORELINE CHANGE ANALYSIS

October 2017

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1.0 SHORELINE CHANGE ANALYSIS

1.1 EXISTING SHORELINE CHANGE RATES

The purpose of the historic shoreline change analysis is to document the past behavior of the study area's shorelines, in order to make a reasonable estimate of the long-term shoreline change rates. Previously documented shoreline change rates along the study area were reviewed and are summarized in Table 1. The alongshore extent corresponding to each location in Table 1 is shown in Figure 1. In addition to the prior studies, a new shoreline change analysis (USACE, 2016) was completed at Villas and Del Haven using long profile survey data from 1995 and LIDAR data from 2014. There is considerable less information available on shoreline change rates at Gandys Beach and Fortescue. Observed shoreline changes at Fortescue were adjusted based on past beach fill activities to determine what the shoreline change rate would likely have been in the absence of these activities.

It is evident from Table 1 that there is fairly good agreement between previously reported shoreline change rates and more recent analyses by the Stockton College Coastal Research Center (2016) and USACE (2016). The greatest uncertainty appears to be at Reeds Beach, with reported values ranging between -3 ft/yr and 0 ft/yr. However, the more recent analyses show that the shoreline at Reeds Beach has been stable with shoreline change rates up to -1 ft/yr.

Recommended shoreline change rates for the NJ DMU project, Table 3, are a synthesis of all the available shoreline change data in study area with greater emphasis on newer data.

Table 1: Historical Shoreline Change Rates (ft/yr) from Prior Studies – Cape May County

<table>
<tr><th rowspan="2">Location</th><th>USACE 1960</th><th>USACE 1991</th><th>FEMA 1993</th><th>USACE 1998a</th><th>USACE 1998b</th><th>CRC 2016</th><th>USACE 2016</th></tr>
<tr><th>1842 to 1957</th><th>1842 to 1957</th><th>1842 to 1986</th><th>1943 to 1995</th><th>1842 to 1994</th><th>1995 to 2016</th><th>1995 to 2014</th></tr>
<tr><td>Goshen Creek</td><td rowspan="2">-3.0[1]</td><td rowspan="2">-3</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Reeds North</td><td rowspan="3">-2</td><td></td><td>0</td><td>0</td><td></td></tr>
<tr><td>Reeds South</td><td rowspan="2">-0.2</td><td rowspan="2">0</td><td></td><td>-1</td><td></td><td></td></tr>
<tr><td>Pierces Point</td><td></td><td>-1</td><td></td><td></td></tr>
<tr><td>Del Haven</td><td rowspan="2">+1.0</td><td rowspan="2">+1</td><td rowspan="2">0</td><td>-0.6</td><td></td><td></td><td>-0.1</td></tr>
<tr><td>Villas North</td><td>-0.2</td><td></td><td>+1.5</td><td>+1.5</td></tr>
<tr><td>Villas South</td><td rowspan="2">-2.3</td><td rowspan="2">-2</td><td>-2</td><td>-1.4</td><td></td><td></td><td>-0.9</td></tr>
<tr><td>North Cape May</td><td>+3</td><td>+1</td><td></td><td>0</td><td>+0.1</td></tr>
</table>

[1]Shoreline change reported for Reeds Beach to Goshen Creek

Table 2: Historical Shoreline Change Rates (ft/yr) from Prior Studies – Downe Township

Location	USACE 1991	HMM 2016	USACE 2017
	1943 to 1995	1930 to 2013	1943 to 1995
Gandys Beach		-2.5	
Fortescue	-1		-2.5[1]

[1]Shoreline change rate in absence of past beach fill activities

Table 3: Recommended Shoreline Change Rates (ft/yr)

Location	Characterization	Shoreline Change (ft/yr)
Gandys Beach	Moderate Erosion	-2.5
Fortescue	Moderate Erosion	-2.5
Reeds Beach	Stable to Low Erosion	-1
Pierces Point	Stable to Low Erosion	-1
Del Haven	Stable	0
Villas North	Stable to Accretion	+0.5
Villas South	Moderate Erosion	-1.5

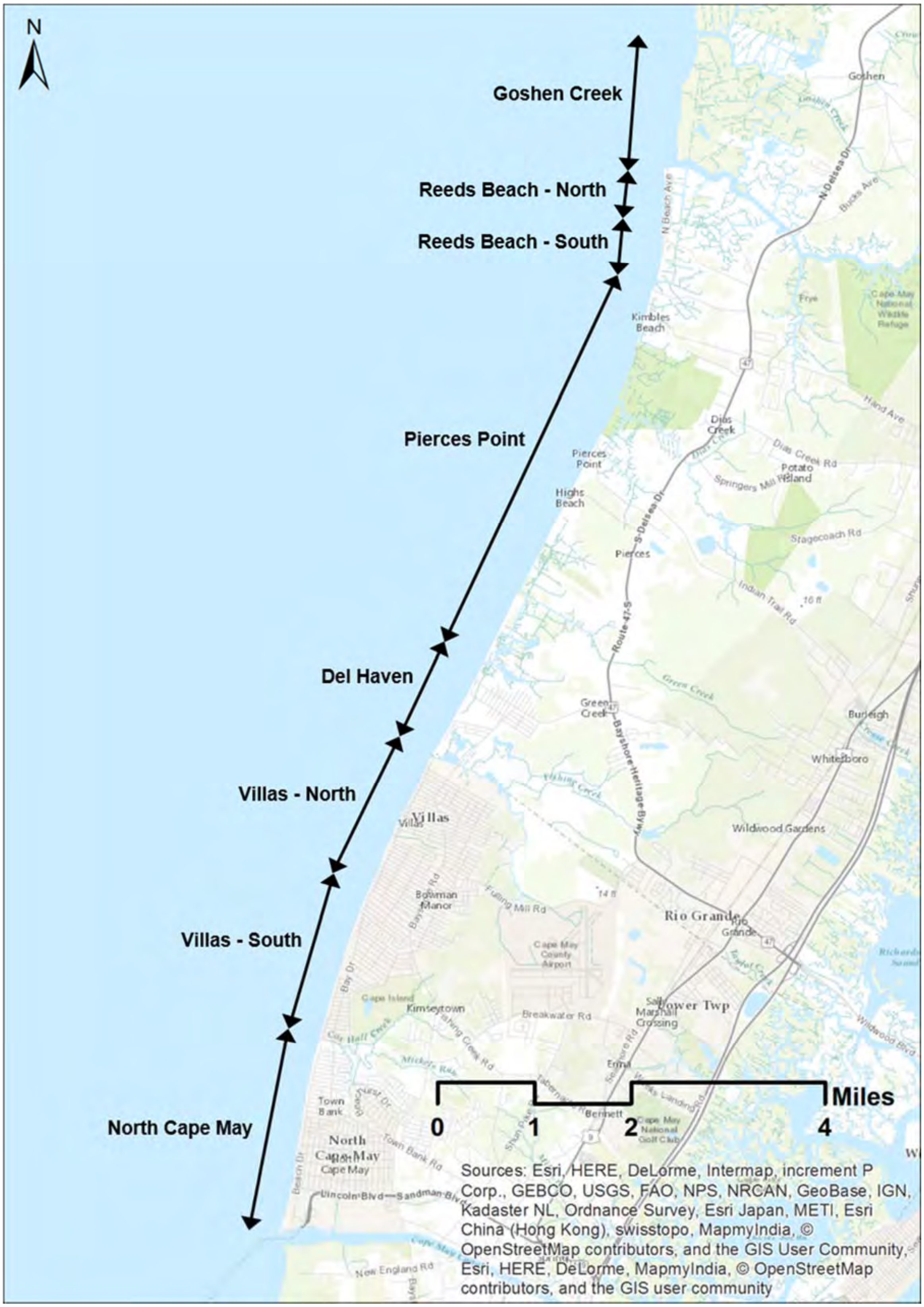


Figure 1: Shoreline Change Analysis Locations

1.2 PRIOR STUDIES

USACE, 1960

The U.S. Army Engineer District, Philadelphia (1960) conducted a shoreline change analysis by comparing high water shorelines along 24 profile lines surveyed in 1956 and 1957 with previous surveys. Table A-4 lists the net changes in highwater shorelines determined in that study.

TABLE A-4

SHORE LINE CHANGES

Average Net Changes of High Water Shore, for Sections Listed
(expressed in feet, bayward +, landward —)

Section and Profile Line No.	1842-85	1885-1948	1948-57	1842-1957
Cape May Canal to Villas Profile lines 2 to 7	-160	-110	(1)	-270
Villas to south of Green Creek Profile lines 8 & 9	+ 50	-110	+170	+110
South of Green Cr. to Reeds Beach Profile lines 10 to 12	+ 40	- 90	+30	- 20
Reeds Beach to Goshen Cr. Profile lines 13 to 15	-110	-240	(1)	-350
Goshen Cr. to West Cr. (2) Profile line 16	-800	-600	-100	-1500
West Cr. to Riggins Ditch Profile lines 17 to 19	-190	- 50	- 30	- 270
Riggins Ditch to East Pt. Profile lines 20 to 24	+ 40	-170	+120	- 10

(1) Intermittent sections of advance and recession, with little net change indicated for the entire reach.

(2) Streams within this reach have had considerable effect upon the configuration of the shore line.

USACE, 1991

In August 1991, the Corps conducted a review of the Delaware Bay and its tributaries to determine the magnitude, location, and effect of the shoreline erosion problems under the scope of the Delaware Bay Coastline - New Jersey and Delaware Reconnaissance Study. In order to quantify shoreline change rates, a variety of sources were utilized including aerial photography and pertinent reports of previous analyses. A primary source of information for the Delaware shoreline of the bay was a series of shoreline change maps produced by the University of Maryland Laboratory for Coastal Research under contract from DNREC. These maps were produced from NOS "T" sheets and aerial photography using a Metric Mapping technique described in Galgano and Leatherman (1988). Two doctoral theses were used to quantify shoreline behavior: Maurmeyer (1978) and Phillips (1985), the latter a review of the New Jersey bay shore from Arnolds Point to Moores Beach. In general, the shoreline along the Delaware and New Jersey shores of the Delaware Bay has had a long-term history of erosion as the sea level rises and continues to flood the Delaware River valley. This sea level rise along with wind generated direct wave attack have been two major contributing factors in erosion along both shorelines of the bay.

The shoreline eroded at 3 ft/yr from 1842 to 1957 around Reeds Beach but was stable from Reeds beach down to Green Creek for the same time period. In addition, shoreline changes from just south of Green Creek to Villas resulted in 1 ft/yr accretion from 1842 to 1957, and 2 ft/yr erosion from Villas to Cape May Canal for the same time period (USACE, 1960). Recent ground-level reconnaissance at Reeds Beach revealed the high water mark to be threatening many structures along the beach. The existing shoreline at Villas consists of a slightly wider beach in general than at the other communities and a more prominent dune system.

Listed in Table 10 are the estimated shoreline change ranges along the major communities along Delaware Bay. If beach fill projects have been implemented for a community, an attempt was made to use pre-beach fill shoreline changes or use estimates after filtering out beach fills to eliminate the beach fill effect.

Table 10
Shoreline Change Rates
Delaware Bay - New Jersey Shore
(Historical time period shown in parenthesis)

SITE	CHANGE RATE (FT/YEAR)
Sea Breeze	(1940-1978) -2 to -5
Fortescue	(1940-1978) -1
Maurice River Cove	(1940-1978) -3 to -12
East Point to Thompsons Beach	(1842-1957) Stable
Moores Beach	(1842-1957) -2 to -6
Reeds Beach	(1842-1957) -3
Green Greek to Villas	(1842-1957) +1
Villas to Cape May Canal	(1842-1957) -2

FEMA, 1993

In 1993 FEMA completed a pilot study for the historical shoreline changes for Cape May County (FEMA, 1993). A total of 808 transects along the bayshore were analyzed for shoreline changes. The shoreline positions were obtained from NOAA T-Sheets dated back to 1842 and 1971, 1977 and 1986 ortho-photographs. The Cape May County bayshore has been divided into 10 subsections as described below. Subsuction 1A to Subsection 1F are discussed below.

Subsection 1A, Figure 2,extends from West Creek to north of Bidwell Creek. This section is an unstabilized shoreline that has historically experienced erosion. The coast is largely composed of marsh, with some limited narrow stretches of sandy beach. The largest amount of historic erosion (just over 20 feet per year) occurred near Dennis Creek which corresponds to the area where the shoreline changes orientation and where waves propagates from the southwest.

Subsection 1B, Figure 3, extends from Bidwell Creek to Pierces Point. This section includes Reeds Beach, Cooks Beach and Kimbles Beach. The shoreline in this subsection is relatively straight, oriented along a generally north-northeast by south-southwest axis. The shoreline is fronted by narrow sandy beaches, backed by extensive marshlands. Coastal development is limited to a series of small coastal villages and towns. Erosion is fairly uniform and low, averaging 2 feet per year.

Subsection 1C, Figure 4, is similar to subsection 1B but with lower erosion rates. Average erosion rates are about 1 foot per year.

Subsection 1D, Figure 5, extends approximately from the town of Sunray Beach to north of Town Bank. This section includes Miami Beach, Villa, Highland Beach and Cape May Beach. Development here is much more extensive than up-bay, with wider beaches backed by progressively less marshlands. Erosion rates are small with some accretional areas. At the southernmost end erosion rates again approach 2 feet per year.

Subsection 1E, Figure 6, is a highly developed area which includes Town Bank and North Cape May. It is a highly developed area characterized by a series of groins, the first of which was placed in the 1930's. As a result, the typical erosion/accretion patterns closely reflect the presence of these groins, with updrift accretion and downdrift erosion at each of the groins. The erosion rate for pre-groin period (1883-1936) average nearly 2 feet per year as compared to the post-groin period (1936-1986) of alternating erosion/ accretion pattern.

Subsection 1F, Figure 7, covers approximately 2,700 feet of shoreline updrift of the Cape May Canal Inlet. The Inlet was opened and stabilized in the 1930's. The period of post-stabilization from 1943 to 1986 is characterized by an accretionary trend. At the north side of the inlet, the annual growth rate is nearly 7 feet since 1943. The pre-groin period (1883-1936) shows an erosion rate of nearly 2 foot per year.

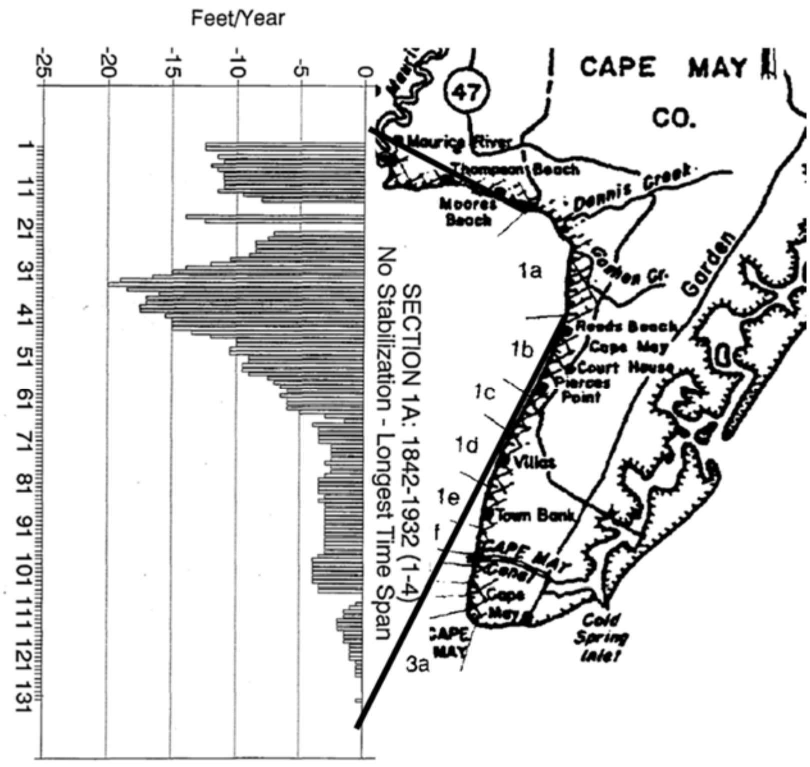


Figure 2: FEMA Historical Shoreline Changes Section 1A 1842-1932

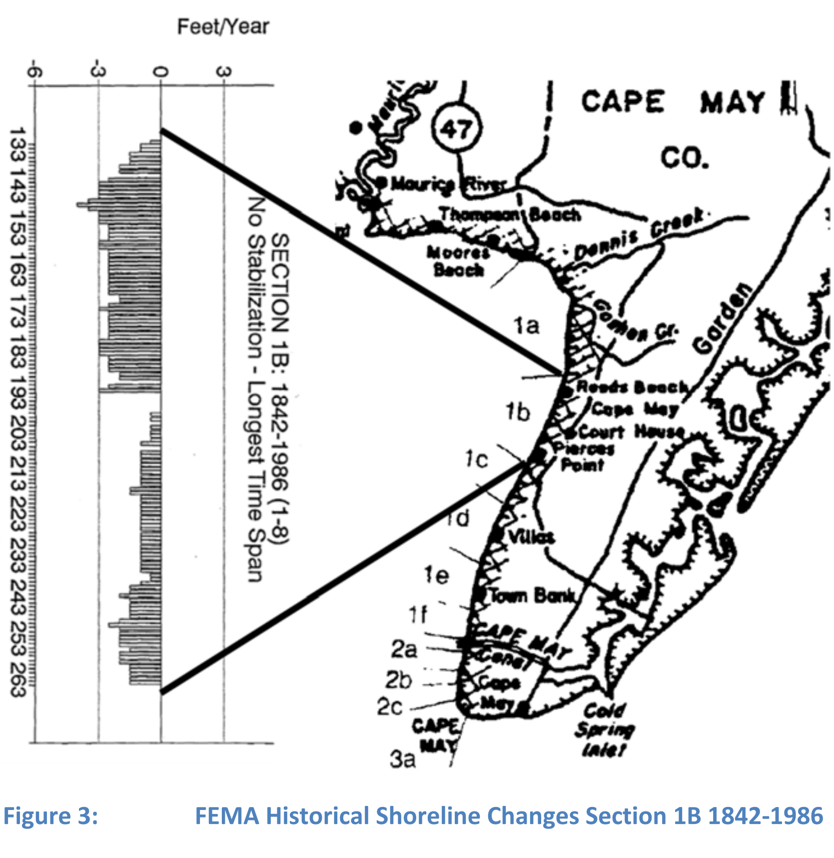


Figure 3: FEMA Historical Shoreline Changes Section 1B 1842-1986

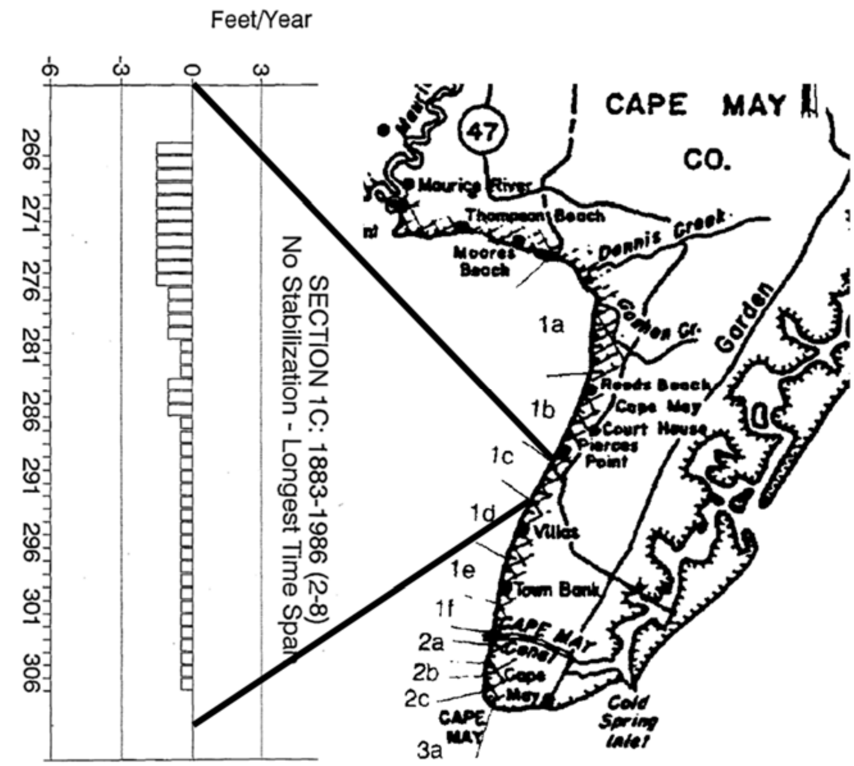


Figure 4: FEMA Historical Shoreline Changes Section 1C 1883-1986

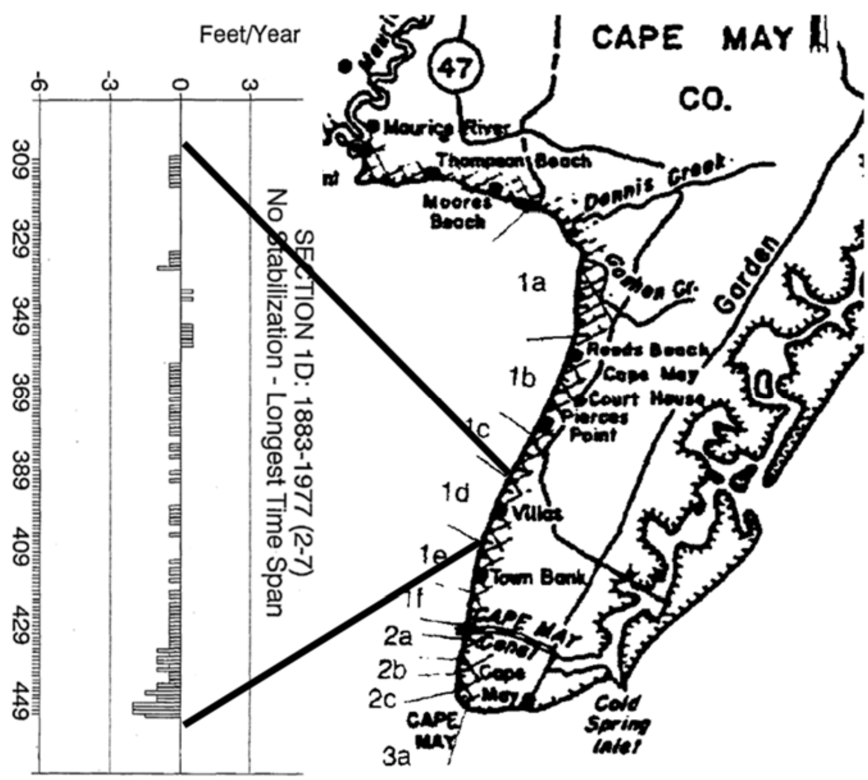


Figure 5: FEMA Historical Shoreline Changes Section 1D 1883-1977

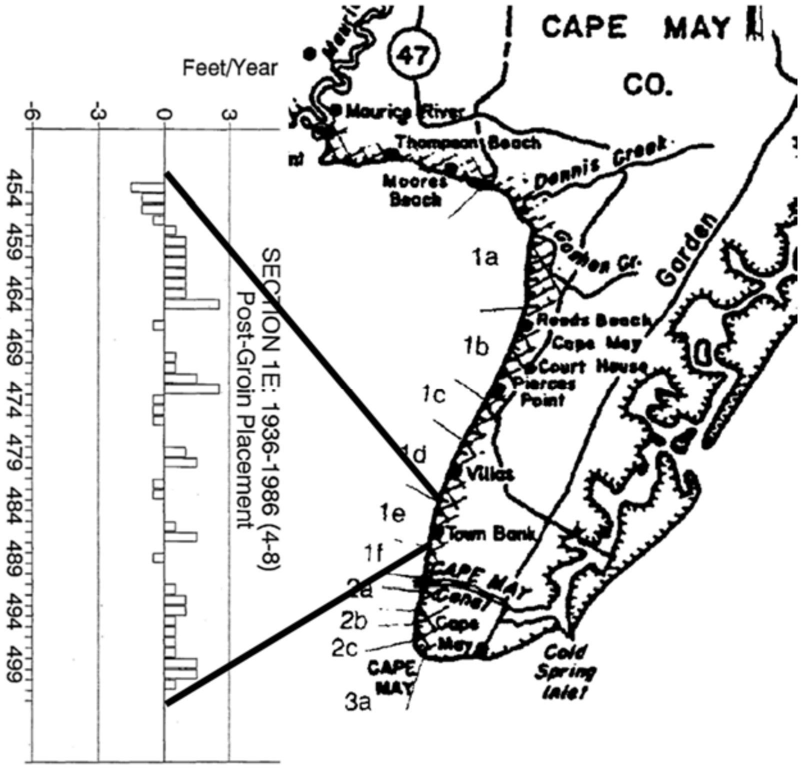


Figure 6: FEMA Historical Shoreline Changes Section 1E 1936-1986

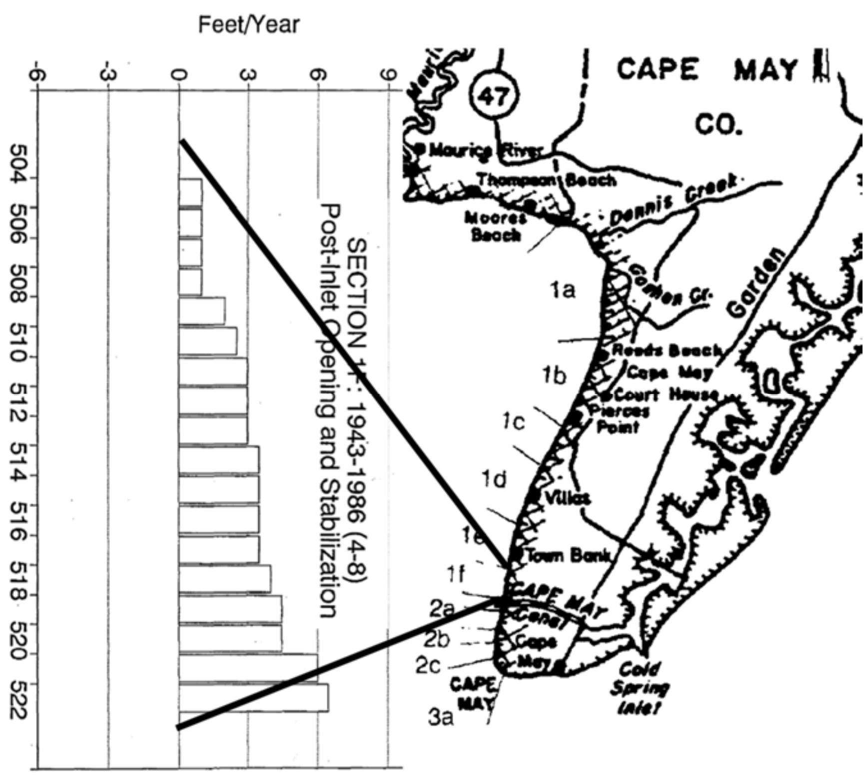


Figure 7: FEMA Historical Shoreline Changes Section 1F 1943-1986

USACE, 1998A

A historic shoreline analysis was conducted to document past shoreline behavior between cape May Canal and Norbury's Landing. Both New Jersey Department of Environmental Protection, Division of Coastal Resources and U.S. Army Corps of Engineers, Philadelphia have studied this area and have found varying periods of both erosion and accretion. No nourishment or Cape May Canal channel maintenance material has been placed in the study area.

Shoreline movement has been affected by local shore protection structures. A variety of structures of different designs and construction materials have been built by the State, municipalities and private interests. Cape May Canal located immediately south of the study area was completed in 1942 and two stone jetties were constructed at the Delaware Bay entrance in 1943. After the 1930's, seven groins were constructed in North Cape May and Town Bank which presently continue to trap sediment.

In order to update previous studies, shoreline change was measured along fourteen transect lines coinciding with the 1995 beach survey using historical digital shoreline maps. This technique is similar to that used in FEMA (1993). Digitized shoreline position maps from the years 1879/85, 1932/36, 1943, 1971, and 1977 were examined. These maps were prepared by the University of Maryland Laboratory for coastal Research using National Ocean Survey (NOS) "T" sheets and aerial photography. Additionally, a 1995 shoreline from digital photogrammetry was provided by Michael Baker, Jr. Inc. The 1971 shoreline was subsequently dropped from this analysis because of gaps in the data. Shoreline positions were determined by digitizing the wet/dry sand interface which is usually discernible on aerial photographs and are therefore dependent on wave conditions and water elevation during the high tide that preceded the survey used to develop the map. Shorelines determined in this way can be taken as an approximation of the MHW position. Shoreline change rates are shown in Table 11.

The study area was .delineated into six cells, as shown in Figure 45 and described in Table 12, based on physical, hydraulic, and economic factors. Table 13 shows the arithmetic average shoreline change rates for each cell in the study area. Results from this analysis indicate that although there have been varying periods of both erosion and accretion, the shoreline is eroding approximately 1 foot per year in the northern portion (cells 3-6) and accreting at approximately 1 foot per year in the southern portion (cells 1,-2) of the study area.

Table 11
Cape May Villas And Vicinity Historical Shorelines
Shoreline Change Rates in ft/yr

(Dashed line indicates no data available)

LINE	1879/85-1932/36	1932/36-1943	1943-1977	1977-1995	1943-1995
CMV-1	-0.6	-11.2	8.4	-0.3	4.8
CMV-2	-1.5	-4.3	2.4	2.0	2.3
CMV-3	-1.4	-1.0	1.5	-0.6	0.8
CMV-4	-1.3	-1.8	2.1	0.1	1.4
CMV-5	----	----	0.0	0.4	0.1
CMV-6	-1.5	10.2	-1.4	-1.4	-1.4
CMV-7	-0.8	5.4	-1.2	-2.1	-1.5
CMV-8	-0.6	6.7	-1.4	4.4	0.6
CMV-9	-0.2	5.4	-2.1	3.4	-0.2
CMV-10	-0.3	3.2	-0.3	1.3	0.3
CMV-11	1.1	-2.7	-0.1	-1.7	-0.7
CMV-12	-0.1	-5.0	-0.4	-0.3	-1.4
CMV-13	-0.8	-1.3	1.4	-0.5	0.8
CMV-14	-1.0	3.7	-0.8	-1.6	-1.1

Table 12
Cell Delineation

CELL	DESCRIPTION
1	Cape May Canal to Beach Plum Drive
2	Beach Plum Drive to Delview Road: Aluminum Bulkhead
3	Delview Road to Rosewood Avenue
4	Rosewood Avenue to Florida Avenue
5	Florida Avenue to end of Millman Lane: Timber Bulkhead
6	end of Millman Lane to Norbury's Landing

Table 13
Cape May Villas & Vicinity Average Shoreline Change Rates
Shoreline Change Rates in ft/yr

Line	1879/85-1932/36	1932/36-1943	1943-1977	1977-1995	1943-1995
Cell 1	-1.2	-4.6	3.6	0.3	1.5
Cell 2	----	----	0.0	0.4	0.1
Cell 3	-1.5	10.2	-1.4	-1.4	-1.4
Cell 4	-0.5	5.2	-1.3	1.8	-0.2
Cell 5	1.1	-2.7	-0.1	-1.7	-0.7
Cell 6	-0.6	-0.9	0.1	-0.8	-0.6

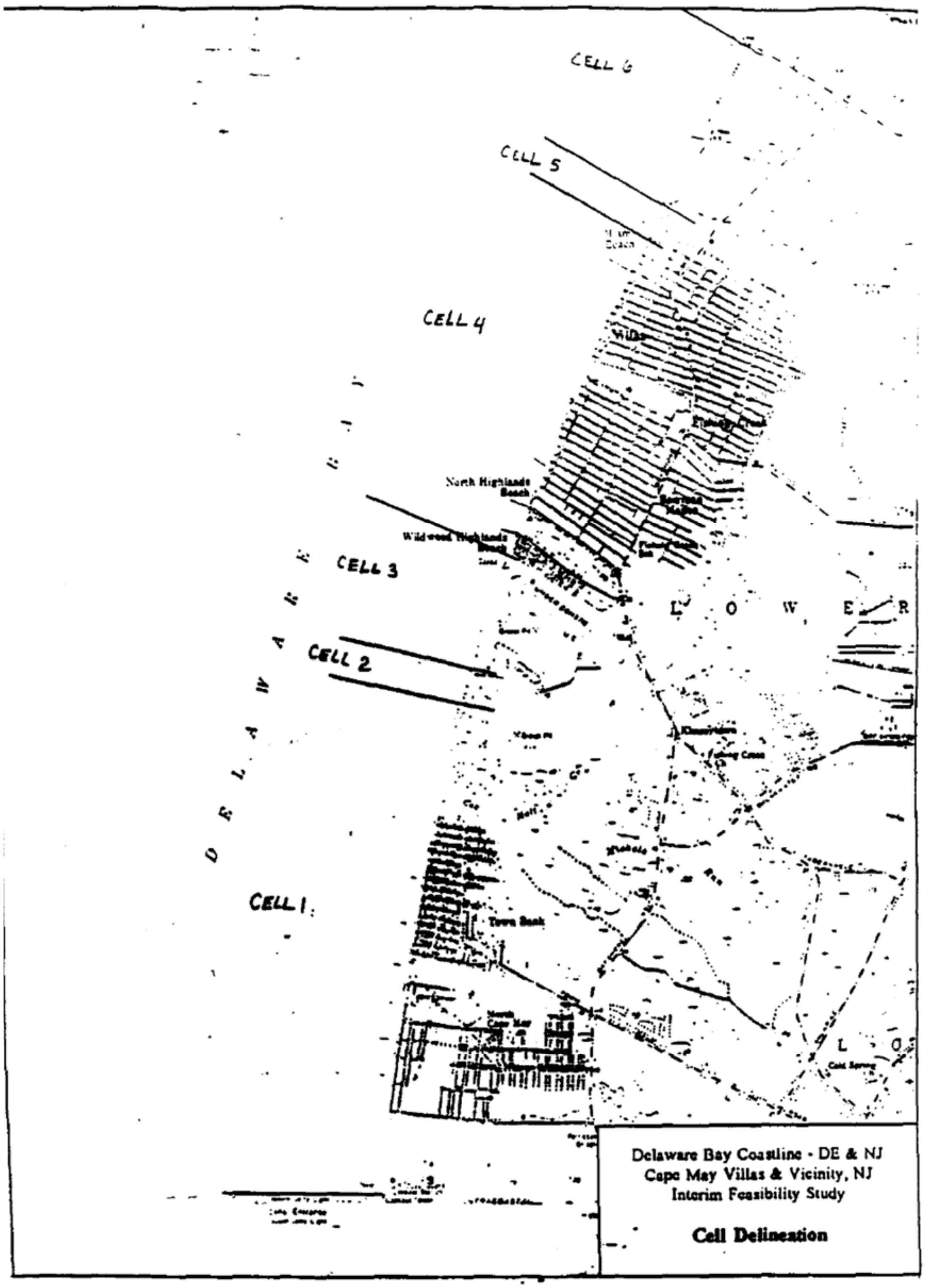

CELL 6
CELL 5
CELL 4
Villas
North Highlands Beach
Wildwood Highlands Beach
CELL 3
CELL 2
D E L A W A R E
L O W E R
Town Bank
CELL 1
Delaware Bay Coastline - DE & NJ
Cape May Villas & Vicinity, NJ
Interim Feasibility Study
Cell Delineation

FIGURE 45

USACE, 1998B

The shoreline history of much of the Pierces Point - Reeds Beach study area has been characterized by a small background erosion trend for the last century. Topographic maps, nautical charts, and aerial photographs together demonstrate that the study area has generally experienced loss of beach leading to conditions which exist at present. Historic shoreline mapping provided by NJDEP shows shoreline location for the study area obtained from historic topographic maps and aerial photography. The dates of mapped shorelines include: 1836-42; 1879-85; 1932-36; 1943; 1971; 1977; and 1994.

A review of these shoreline position data reveal that there have been periods of small accretion interspersed with periods of small erosion. The overall condition at Pierces Point has involved less than 100 feet of shoreline retreat from the date of the earliest mapped shoreline (1836-42) to the most recent (1994), for an average long-term erosion rate of less than one foot per year. A similar condition exists for the southern half of the Reeds Beach shoreline, with the northern half experiencing stability to small accretion since the period around 1935 when a jetty was constructed on the south side of the Bidwell Creek entrance.

HATCH MOTT MACDONALD, 2016

A shoreline change analysis was conducted at Gandys Beach using historical aerials from 1930 and 2013. Shoreline change rates were calculated based on the distance from the observed shoreline from a fixed baseline. Figure 8 shows the shoreline change rates near Gandys Beach. The average shoreline change rate along Gandys Beach was found to be -2.5 ft/yr (erosive). The analysis also showed accelerated shoreline change rates north of Gandys Beach, and is believed to be most likely due to a rapid depletion in sediment source.

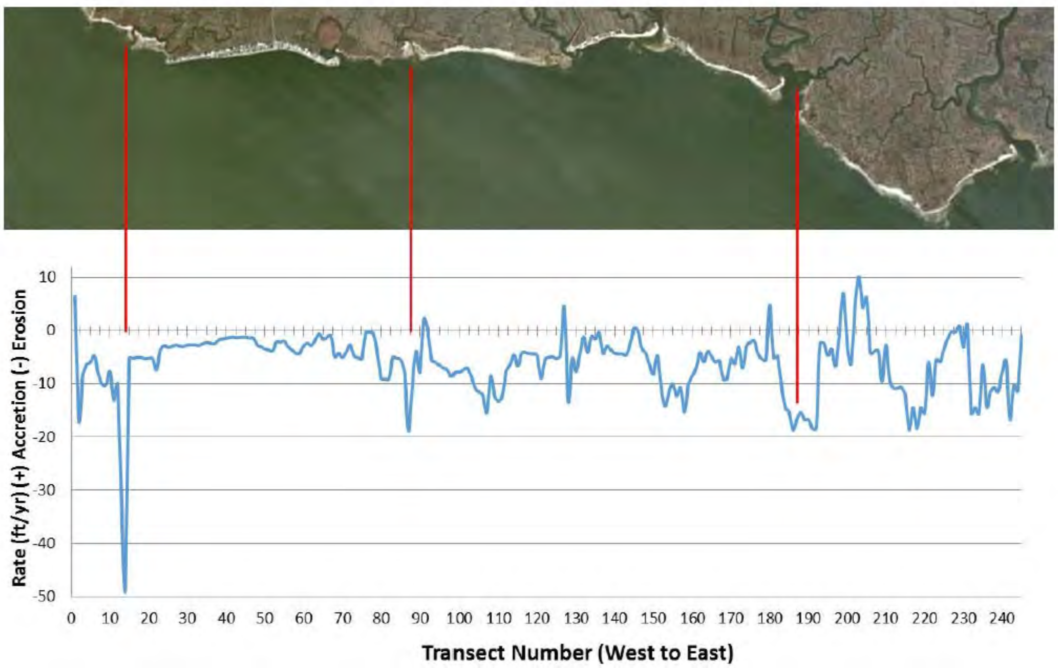


Figure 8: HMM Shoreline change rates at Gandys Beach

NJBPN, 2016

In 1986, The Richard Stockton College Coastal Research Center (CRC) established the New Jersey Beach Profile Network (NJBPN) for the purpose of monitoring shoreline conditions along New Jersey's coast. NJBPN consists of over 100 beach profile sites along the entire shoreline, including the Raritan and Delaware Bays. Figure 9 shows the NJBPN locations in Cape May County. The profile sites are spaced approximately one mile apart, with at least one site located in each oceanfront municipality. The dune, beach, and nearshore are surveyed at each profile site twice a year (fall and spring), and analyzed for seasonal and multiyear changes in shoreline position and sand volume. Reports on all beach profiles are published annually.

There are 3 beach profiles sites in the study area:

- Site 100 Reeds Beach
- Site 101/201 Pacific Avenue, Villas
- Site 102 Whittier Avenue, North Cape May

All the available survey data from 1986 to 2016 was provided to the USACE by Coastal Research Center of Stockton College. Figure 10 to Figure 12 show the evolution of the beach profile at the three sites over the last 20 years.

The beach profile data at Reeds Beach shows that the beach was fairly stable until 2008 when dredged sediment from Bidwell Creek was placed at Reeds Beech. From 2008 onward the beach has experienced a relatively small erosional trend.

The beach profile data at Villas shows that the beach is accretional with an approximately 30 feet of shoreline advance since 1985 (1.5 ft/yr).

The beach profile data at North Cape May shows that beach was accretional until Hurricane Sandy in 2012, retreated in response to Sandy, and since stabilized.

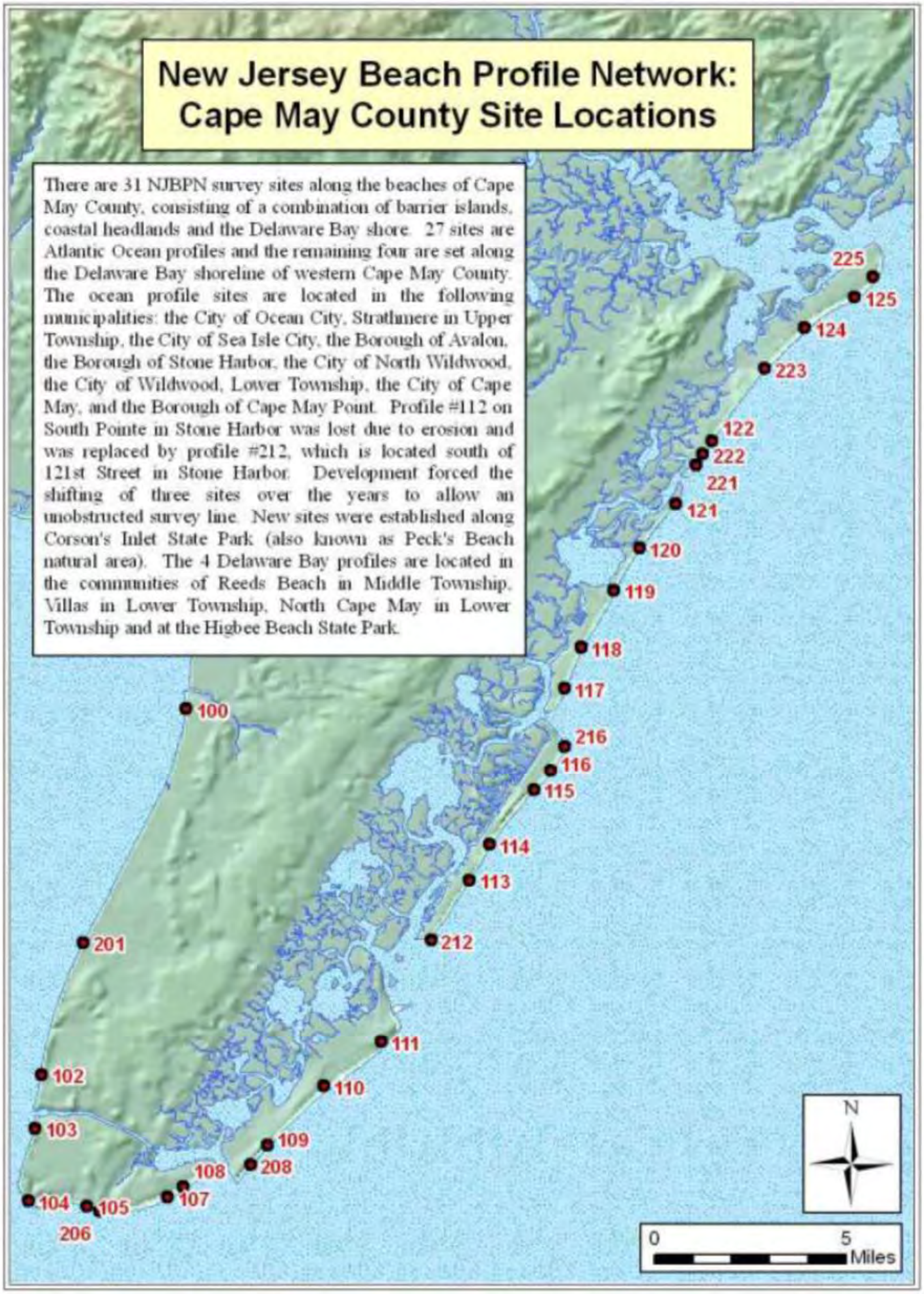


Figure 9: NJBPN Profile Locations – Cape May County

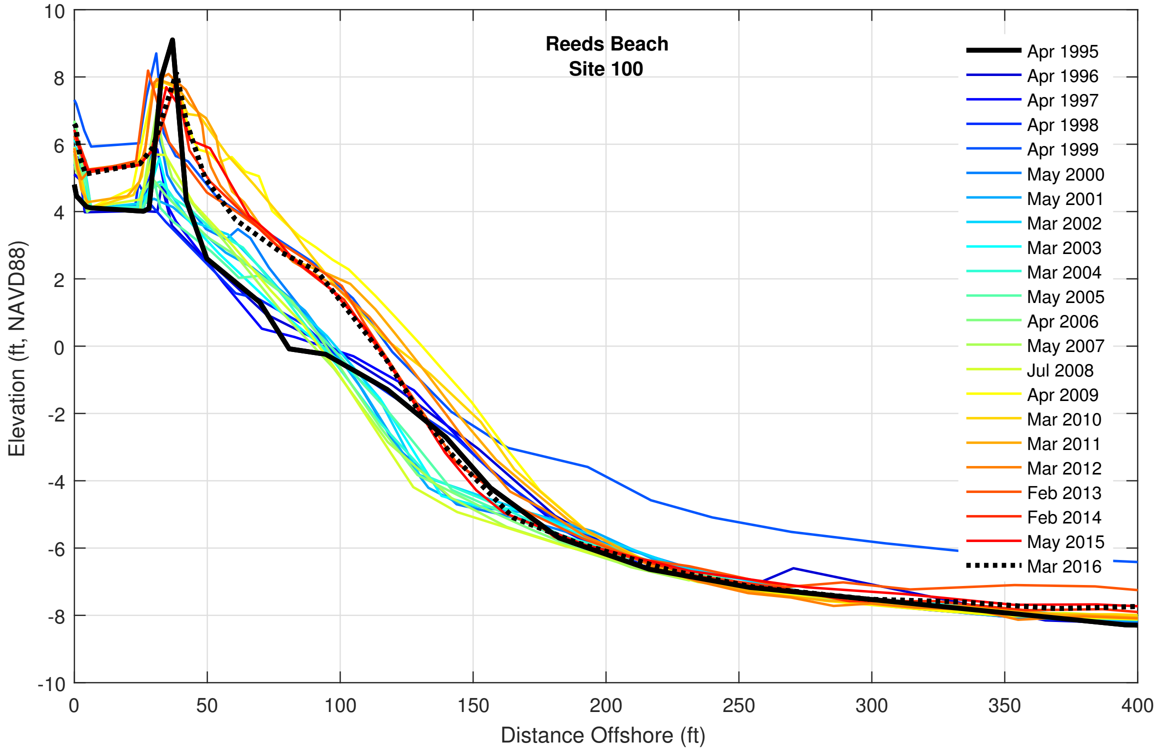


Figure 10: NJBPN Site 100, Reeds Beach

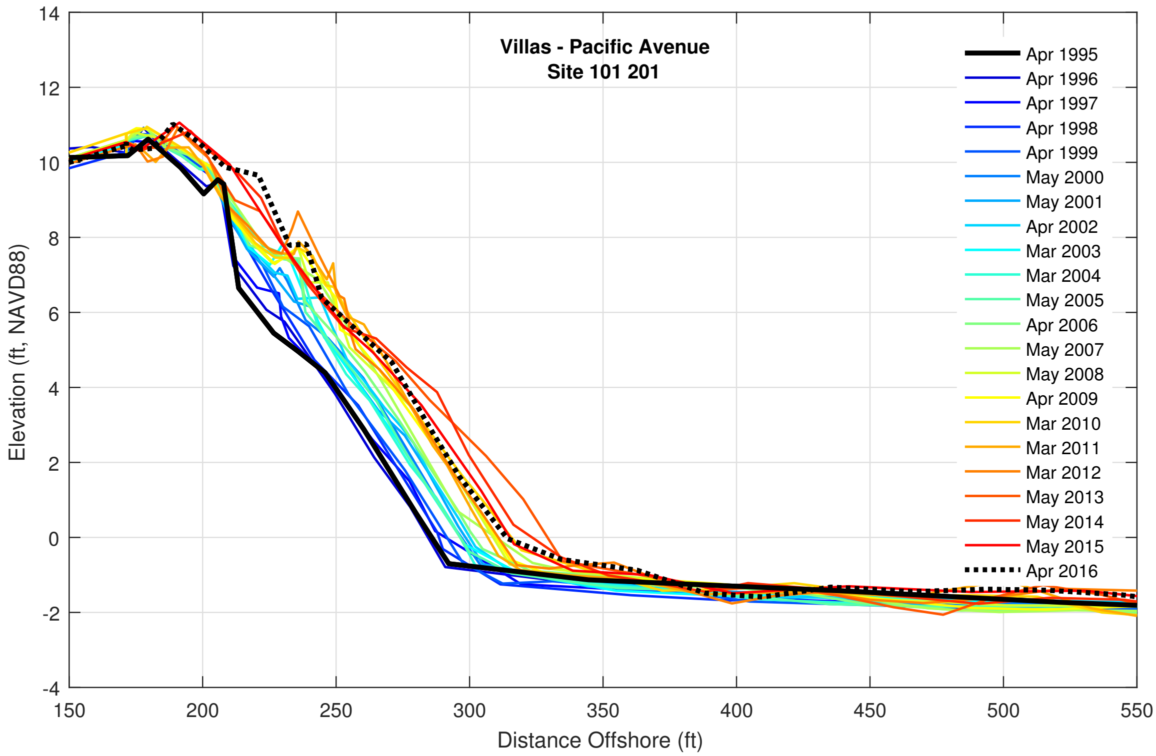


Figure 11: NJBPN Site 101/201, Villas

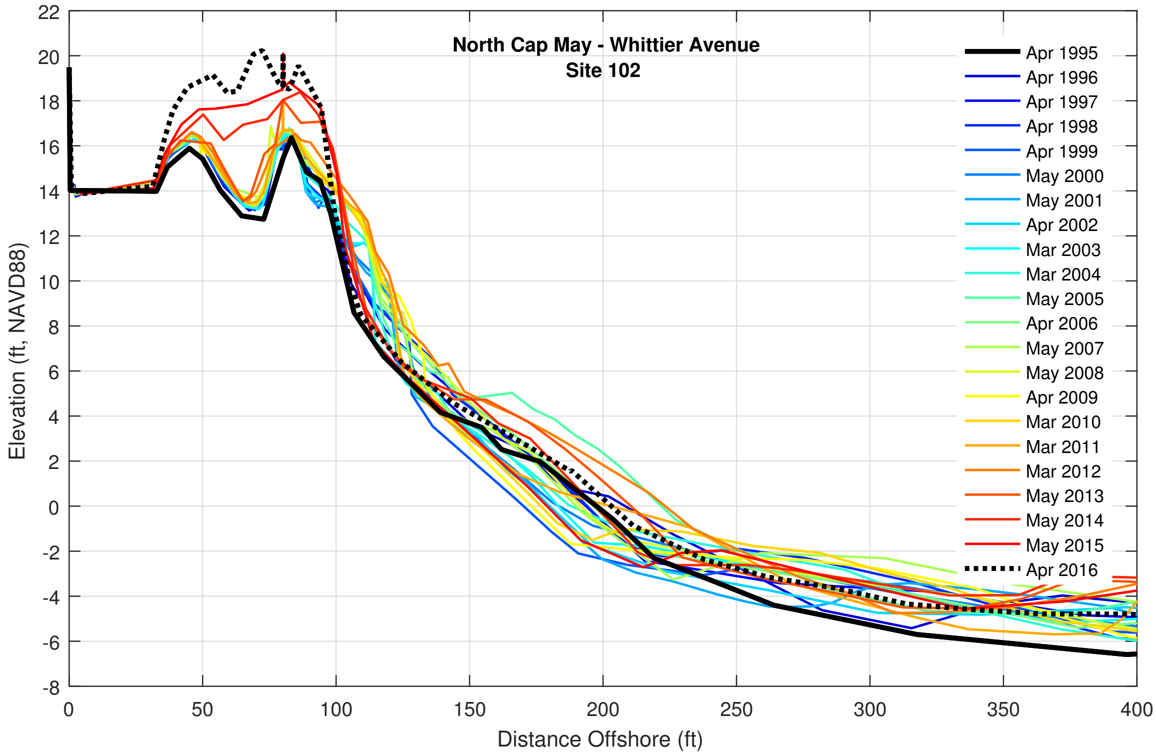


Figure 12: NJBPN Site 102, North Cape May

USACE, 2016

Fourteen profile lines were surveyed between Cape May Canal and Norbury's Landing in June 1995 in order to establish existing beach conditions for the study area (Figure 13). Locations of the survey lines are shown in Figure 12. LIDAR data from 2014 was extracted along fourteen profile lines to evaluate profiles changes from 1995-2014. Table 4 presents the observed shoreline changes at the fourteen survey lines and the average value across geographic reaches. Figure 14 to Figure 23 present a comparison of observed profile data from CMV-5 to CMV-14.

Table 4: Observed Shoreline Changes 1995-2014

Survey Line	Shoreline Change Rate (ft/yr)	Average (ft/yr)
CMV-1	2.5	+2.5
CMV-2	0.5	
CMV-3	-0.4	+0.1
CMV-4	0.0	
CMV-5	-0.3	
CMV-6	-1.6	-0.9
CMV-7	-0.9	
CMV-8	1.3	
CMV-9	2.3	+1.7
CMV-10	1.7	
CMV-11	1.5	
CMV-12	0.4	
CMV-13	-0.3	+0.1
CMV-14	0.2	

Shoreline Change Measured at +2 ft NAVD88 Contour

Figure 13: Villas 1995 Survey Line Locations

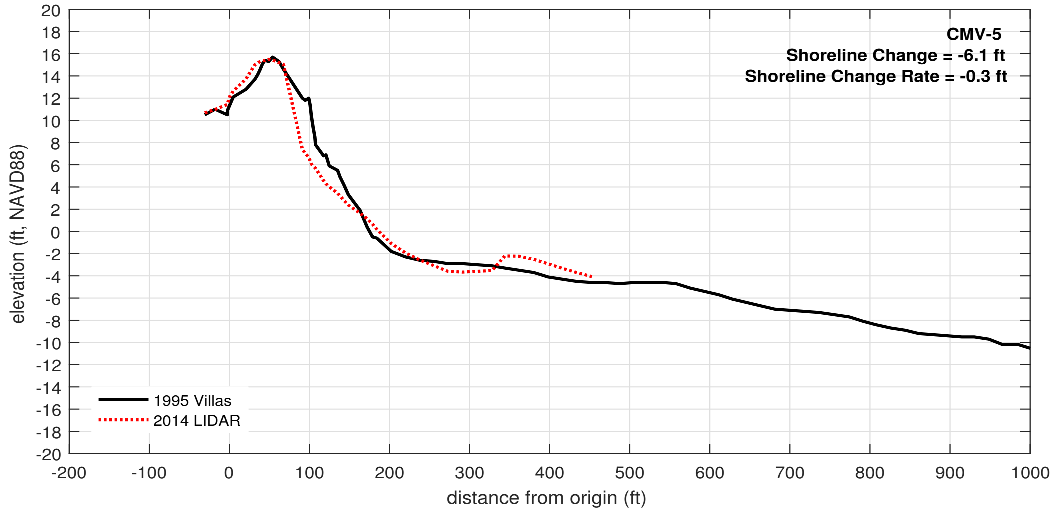


Figure 14: 1995 Villas CMV-5

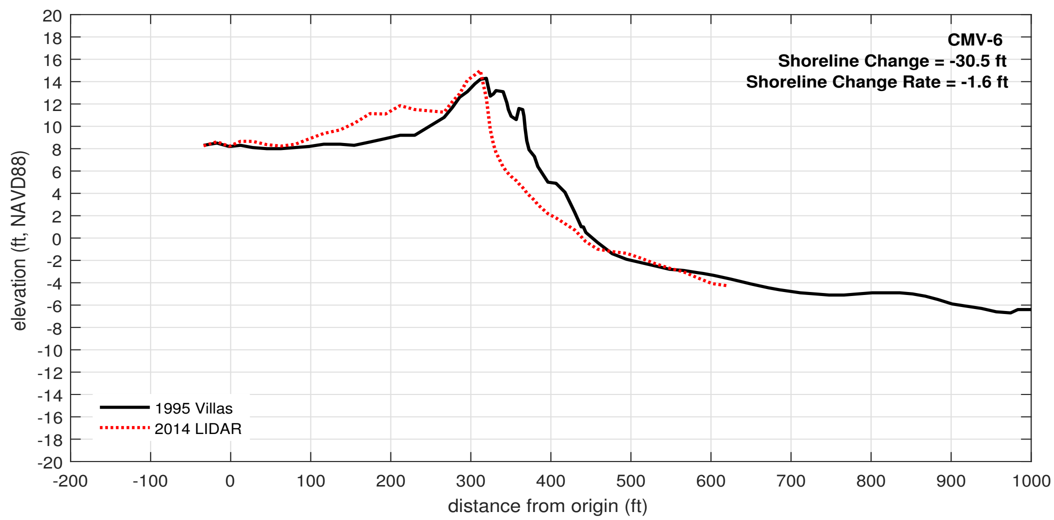


Figure 15: 1995 Villas CMV-6

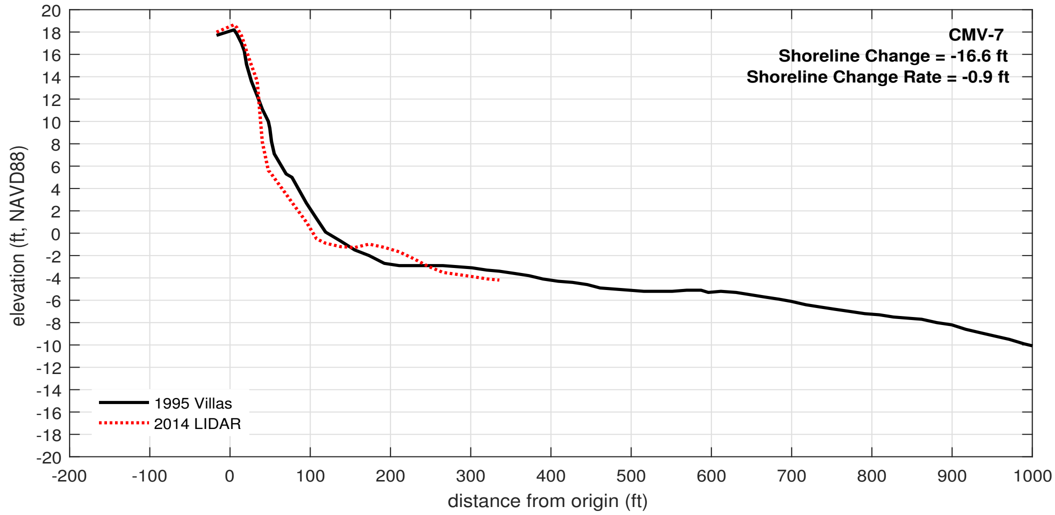


Figure 16: 1995 Villas CMV-7

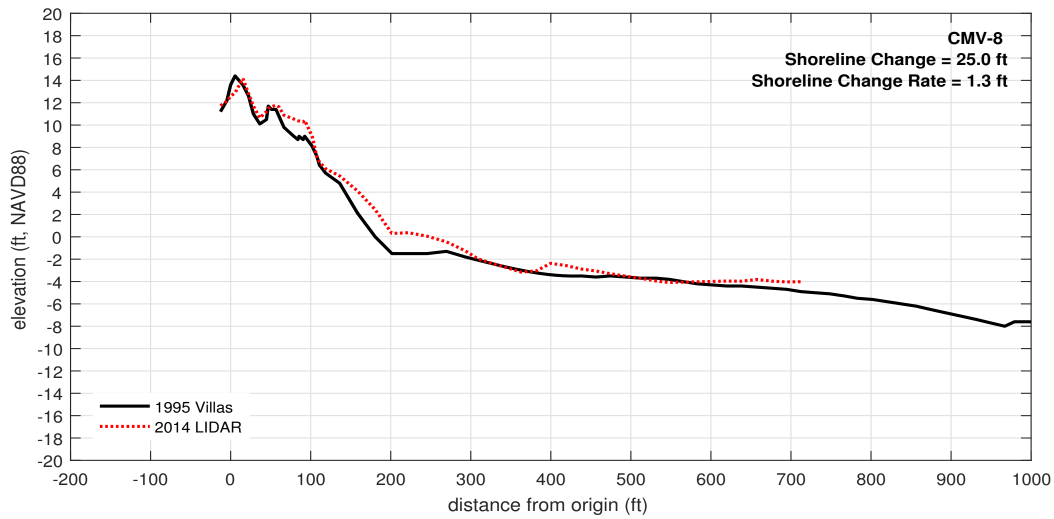


Figure 17: 1995 Villas CMV-8

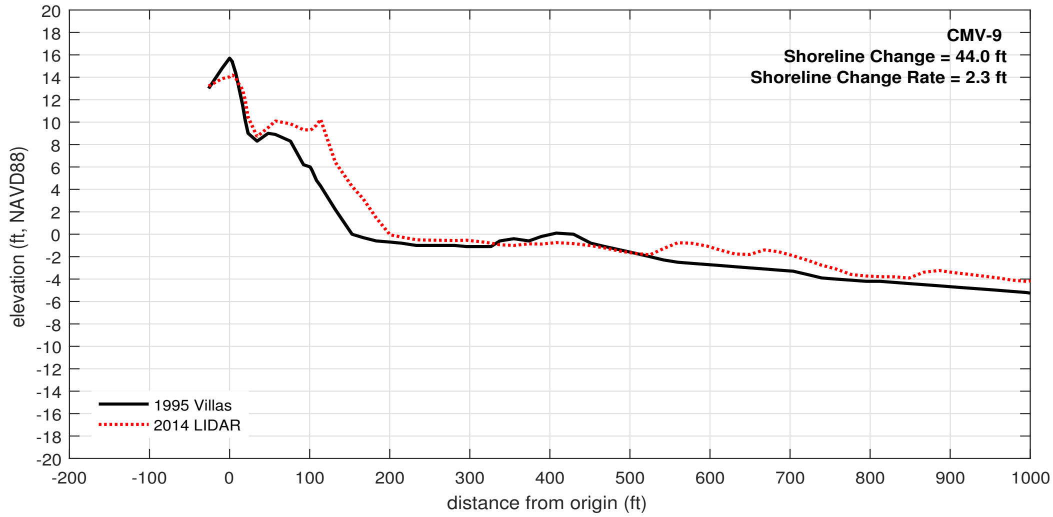


Figure 18: 1995 Villas CMV-9

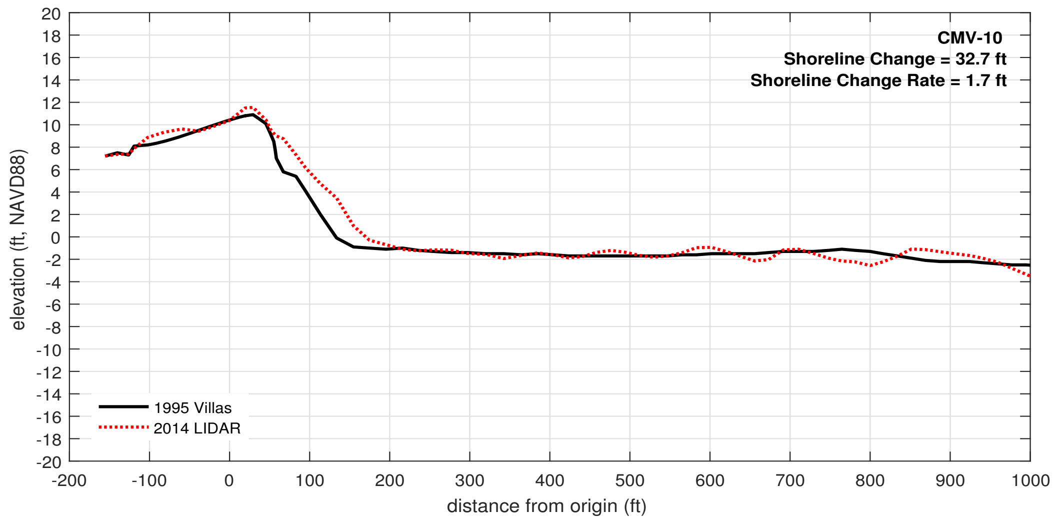


Figure 19: 1995 Villas CMV-10

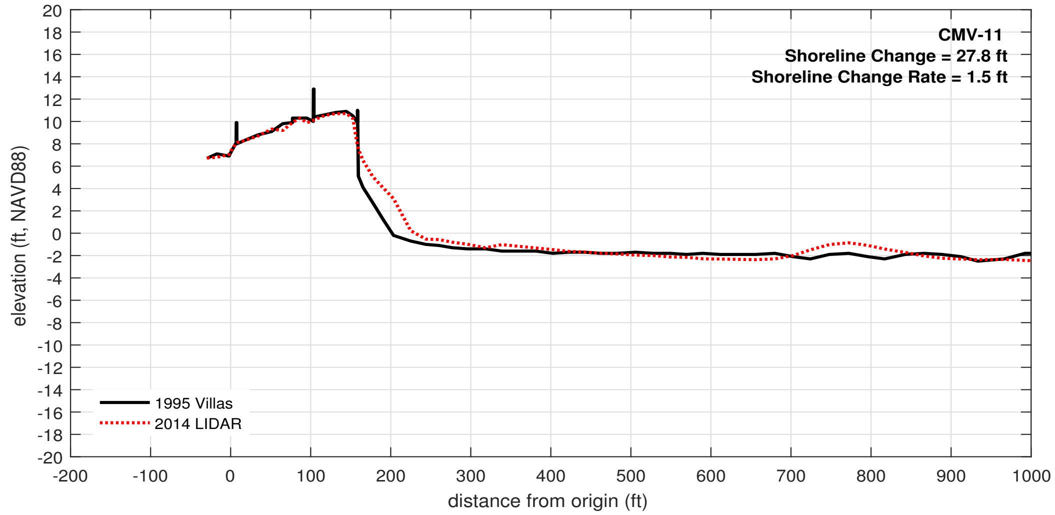
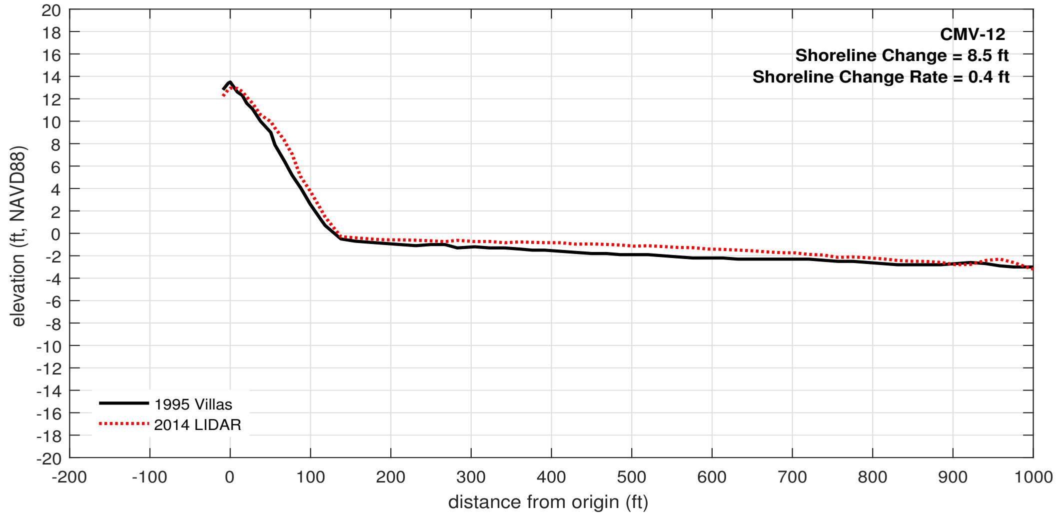


Figure 20: 1995 Villas CMV-11



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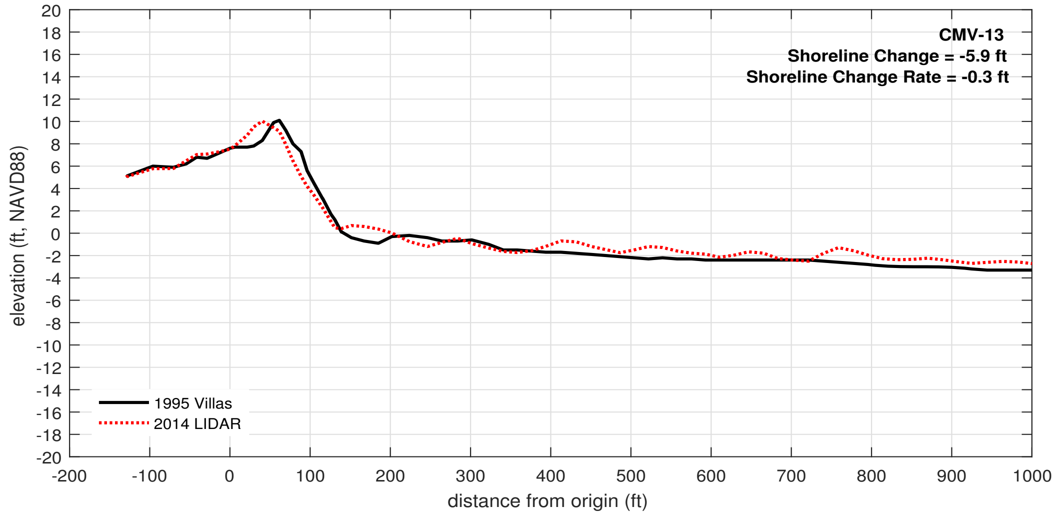

Figure 22: 1995 Villas CMV-13

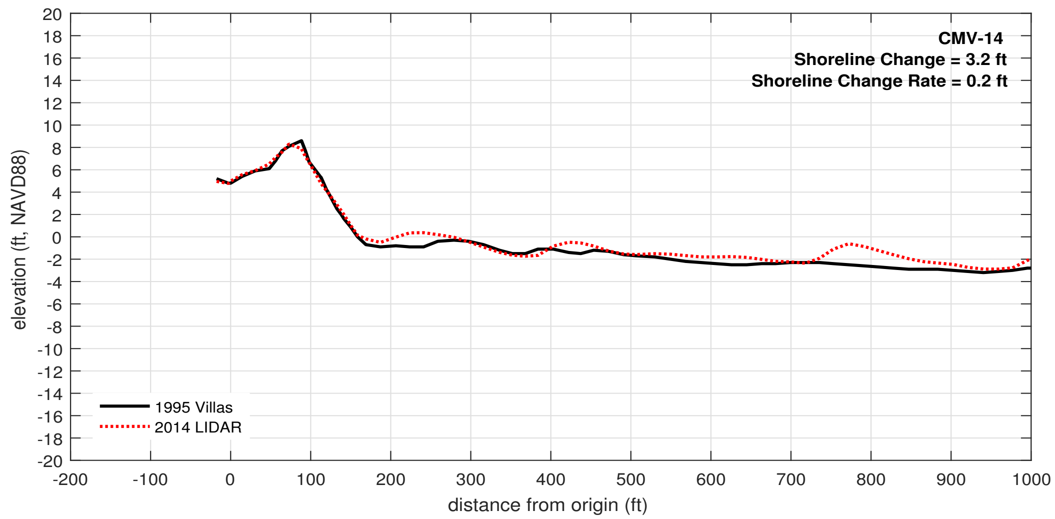

Figure 23: 1995 Villas CMV-14

USACE, 2017

The USACE 1991 Delaware Bay Coastline - New Jersey and Delaware Reconnaissance Study indicated that the historical shoreline change rate at Fortescue, NJ was 1 ft/yr, based on NOS "T" sheets and aerial photography from 1940 and 1978. However, over this same time period Fortescue Creek has been dredged 2 to 3 times per decade with some of the dredged material being beach fill compatible sand and placed along Fortescue (per NJDEP). Therefore, the observed shoreline changes may not reflect the background shoreline change rates that would occur at Fortescue in the absence of beach fill placement. It is estimated that approximately 15,000 cy of sand was placed at Fortescue during each dredging operation. If 2.5 operations occurred per decade, then the resulting annual quantity of sand placed at Fortescue was 3,750 cy/yr. Based on an assumed active profile height of 12 feet, and alongshore length of 5,000 feet, the impact of the of the beach fill on shoreline change rates is estimated to be +1.7 ft/yr. Therefore, the estimated shoreline change rate in the absence of beach fill is approximately -2.7 ft/yr, or -2.5 ft/yr rounded to the nearest half foot.

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U.S. Army Corps of Engineers, Philadelphia District, 1998b. Reeds Beach and Pierces Point, NJ. Interim Feasibility Study. Final Appendix A, Engineering Technical Appendix.

ATTACHMENT C.2 TOPOBATHYMETRIC DATA

October 2017

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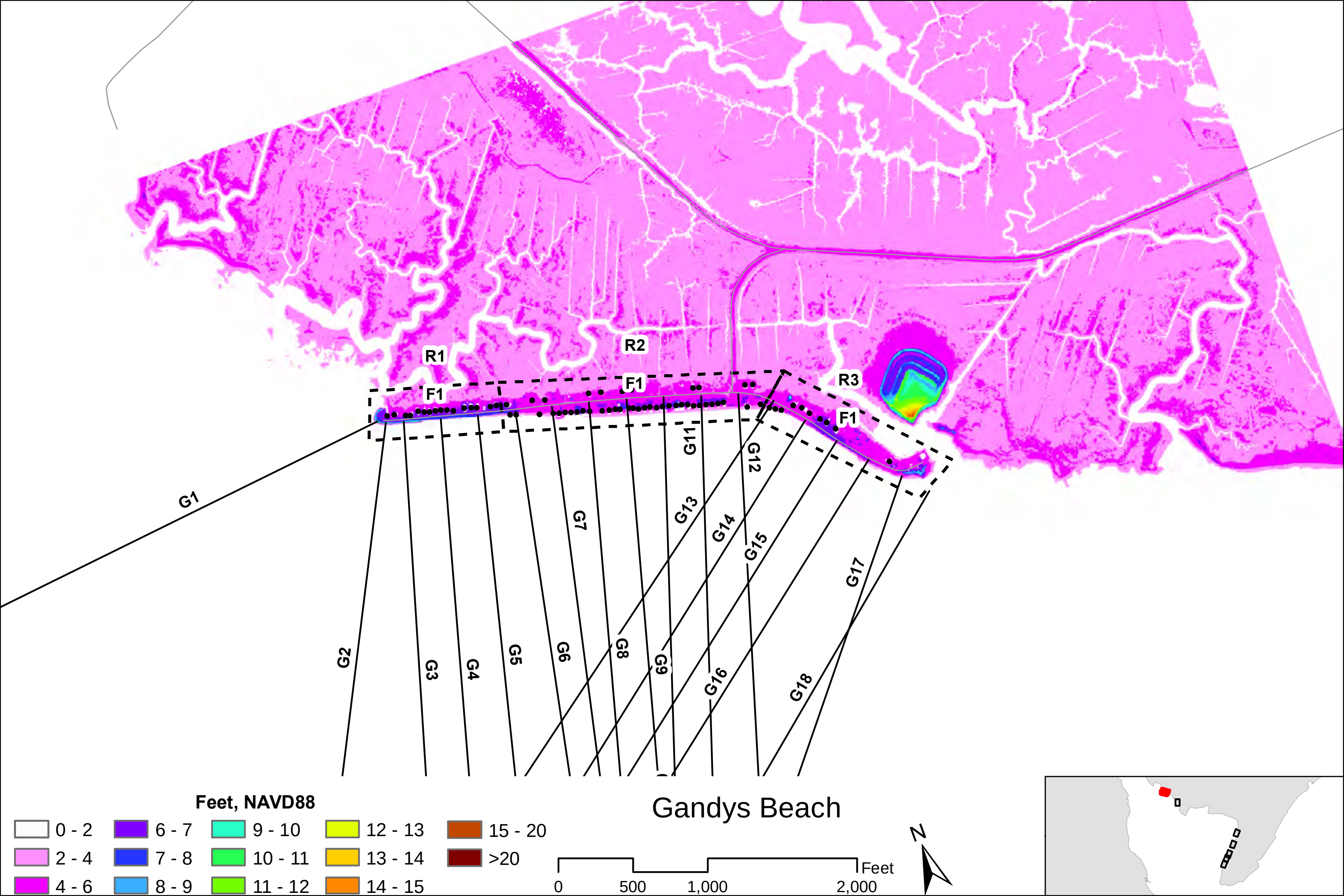

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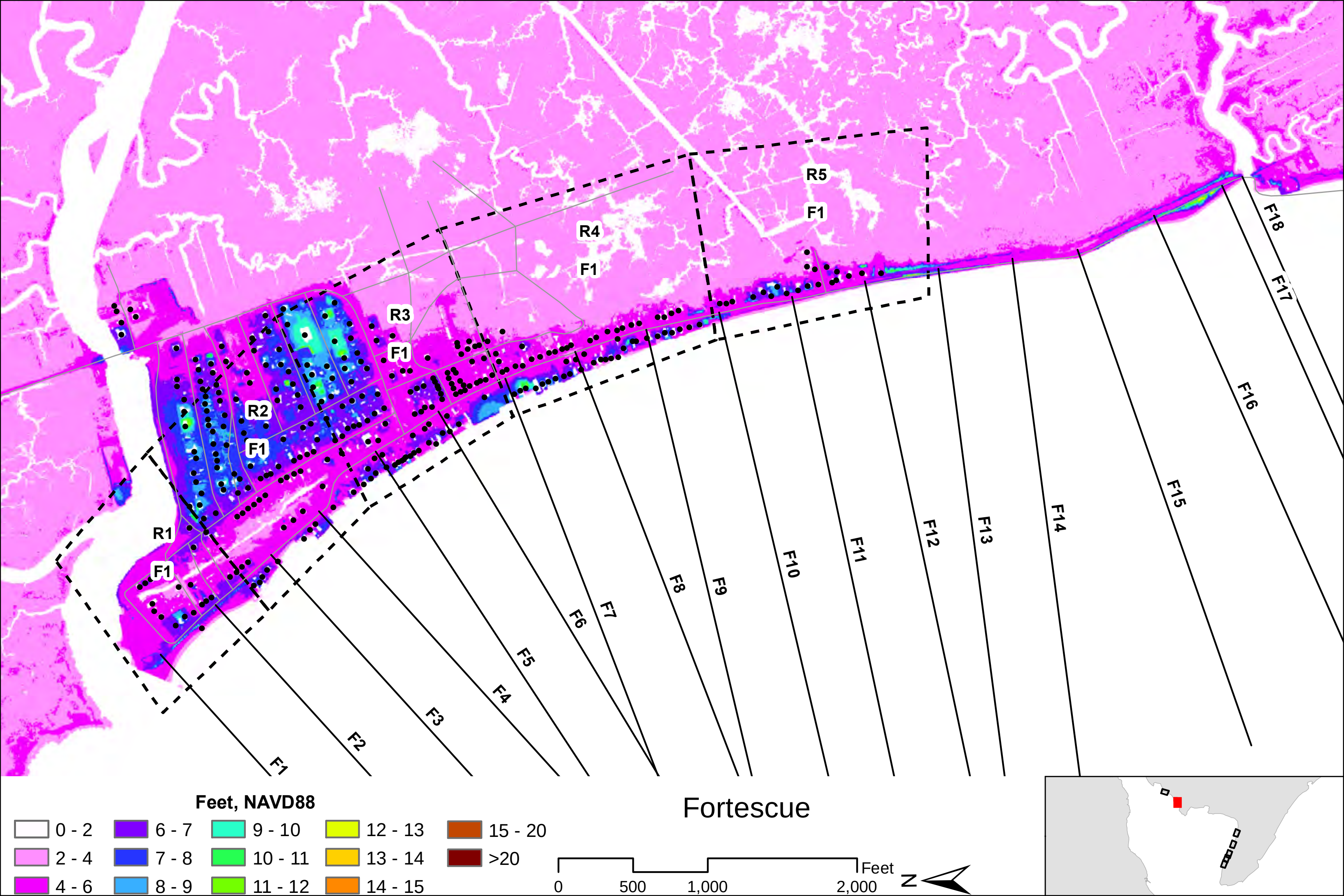

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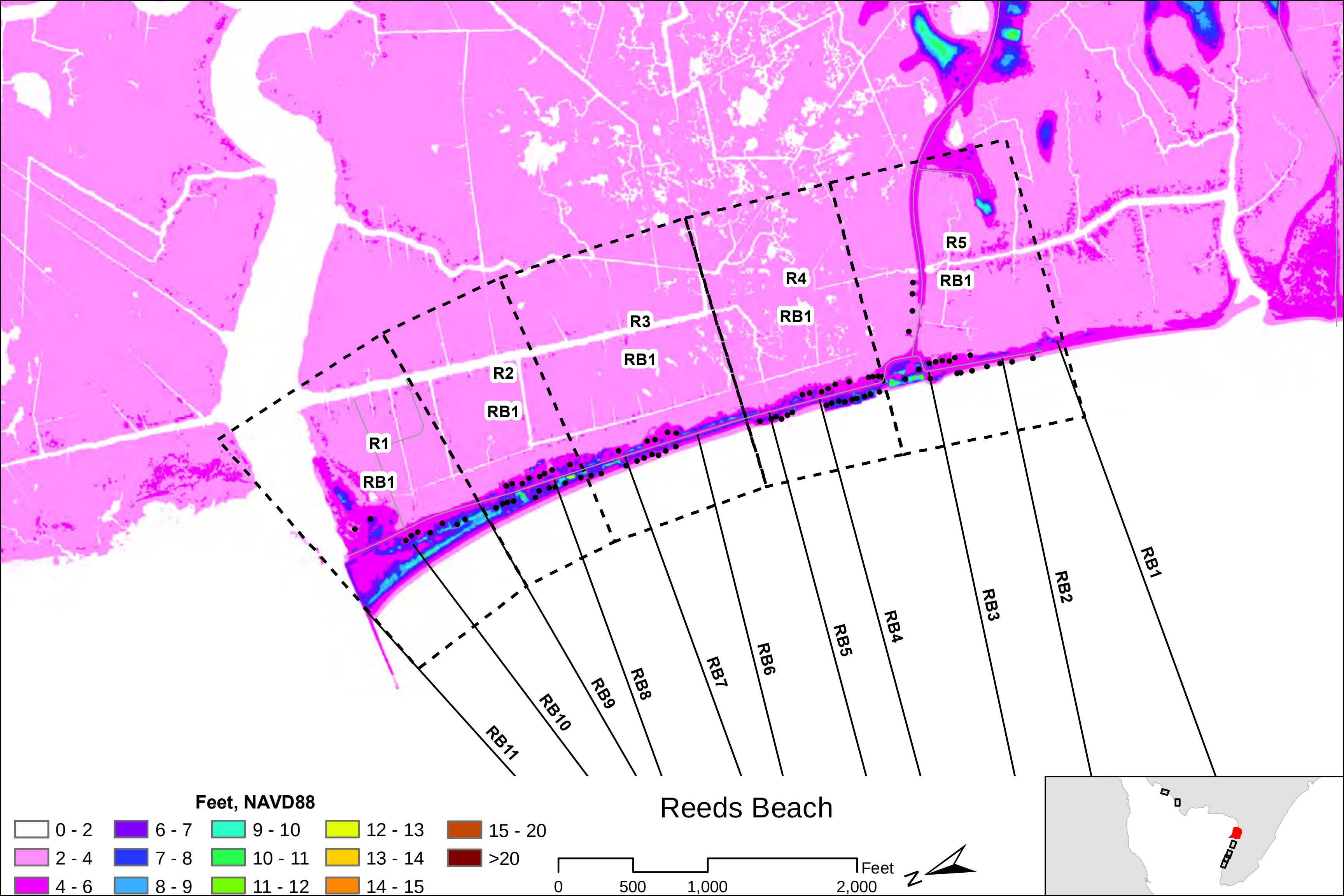
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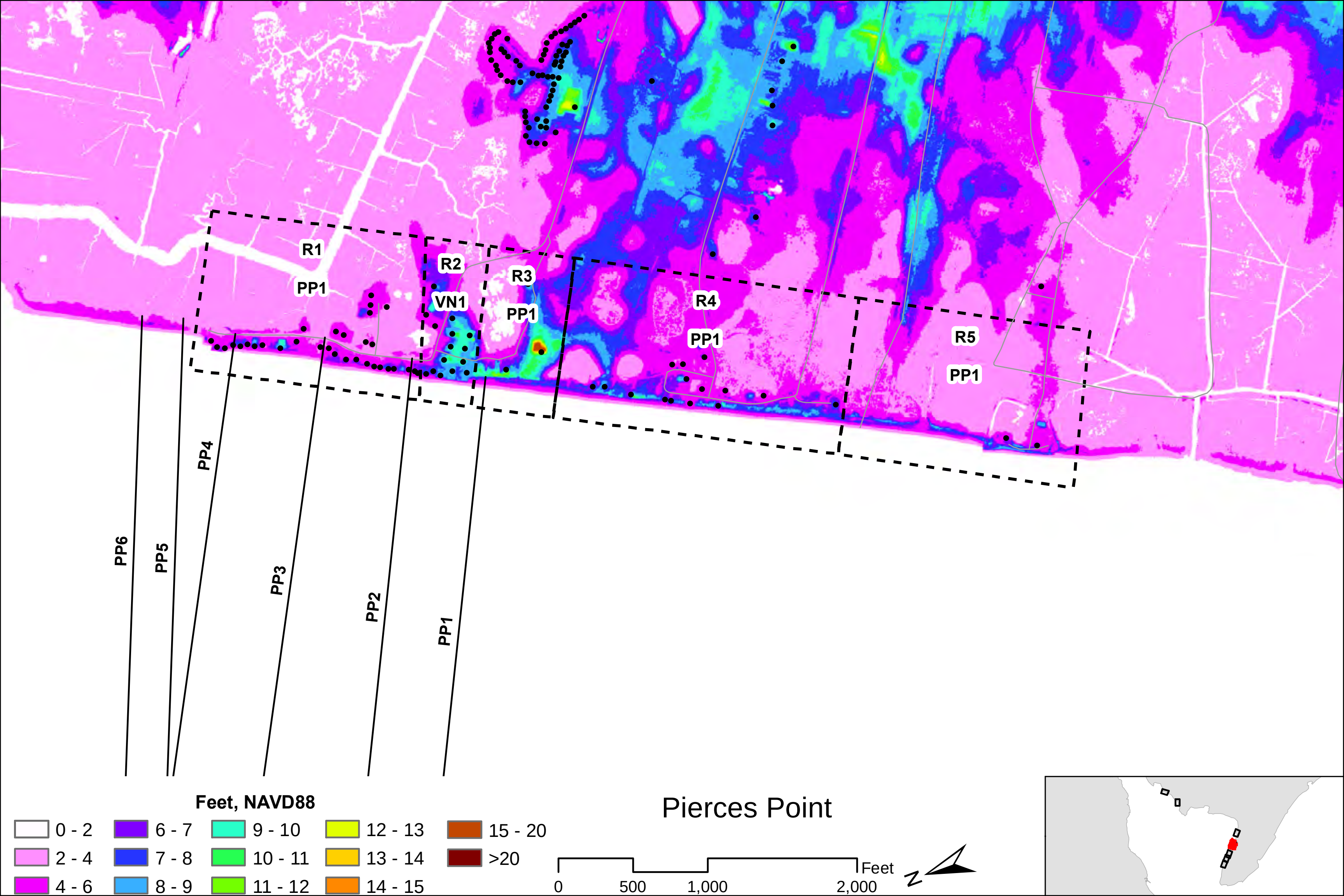
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Feet, NAVD88
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2 - 4
4 - 6
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7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
Pierces Point
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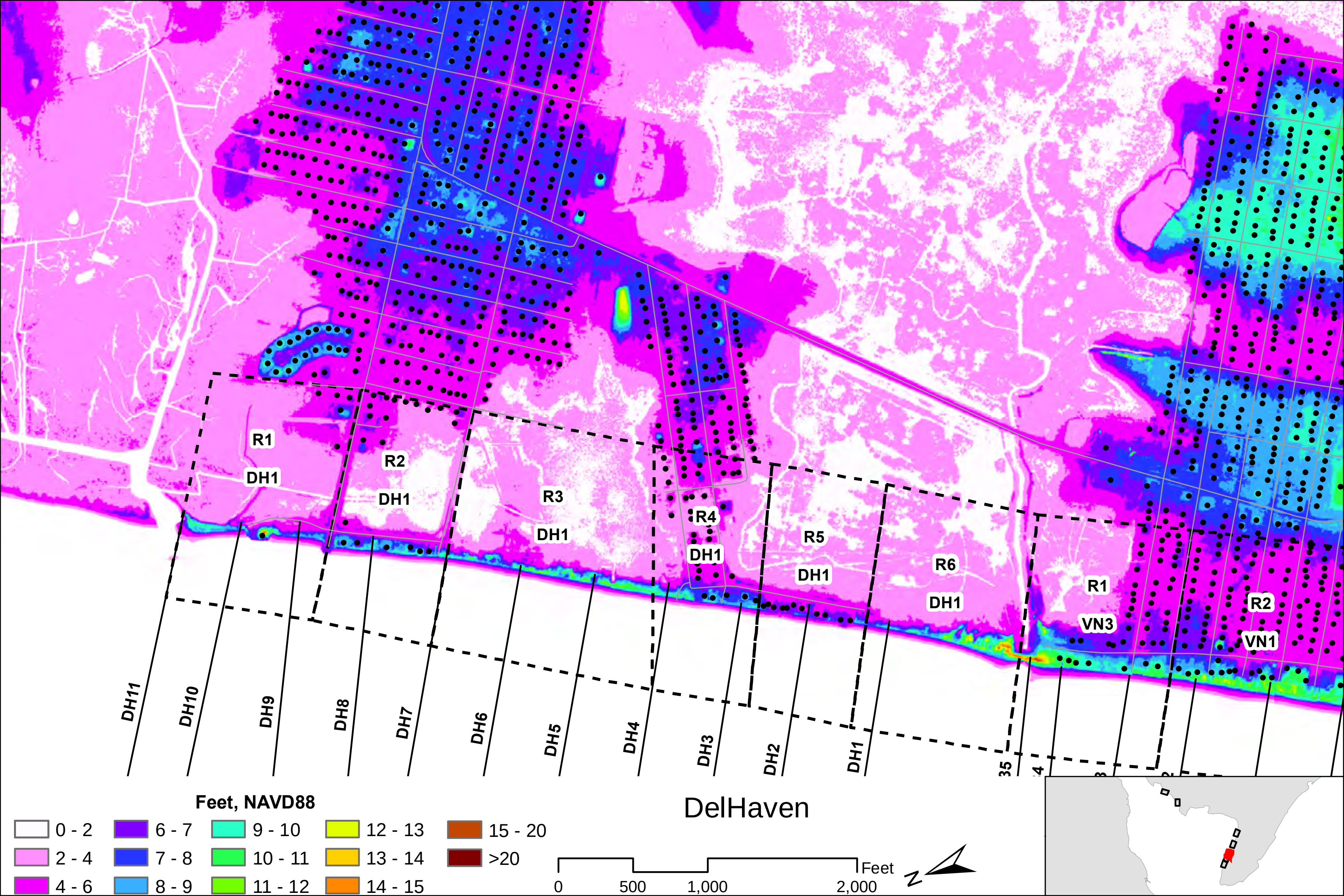
2,000
Feet
N



R1
DH1
R2
DH1
R3
DH1
R4
DH1
R5
DH1
R6
DH1
R1
VN3
R2
VN1
DH11
DH10
DH9
DH8
DH7
DH6
DH5
DH4
DH3
DH2
DH1
Feet, NAVD88
0 - 2
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4 - 6
6 - 7
7 - 8
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14 - 15
15 - 20
>20
DelHaven
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Feet
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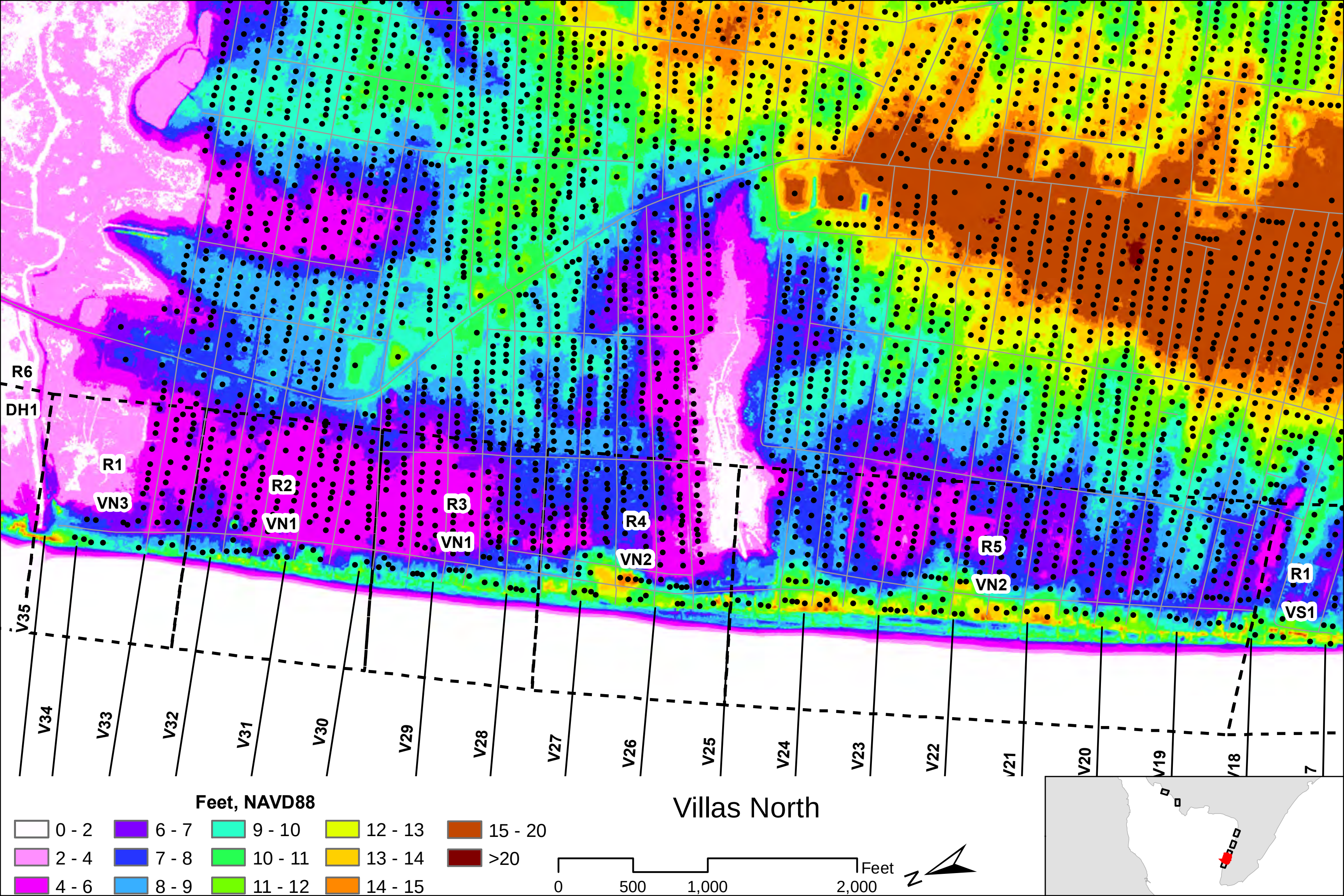

Villas North
Feet, NAVD88
0 - 2
2 - 4
4 - 6
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8 - 9
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12 - 13
13 - 14
14 - 15
15 - 20
>20
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1,000
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Feet
N
R6
DH1
R1
VN3
R2
VN1
R3
VN1
R4
VN2
R5
VN2
R1
VS1
V35
V34
V33
V32
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V29
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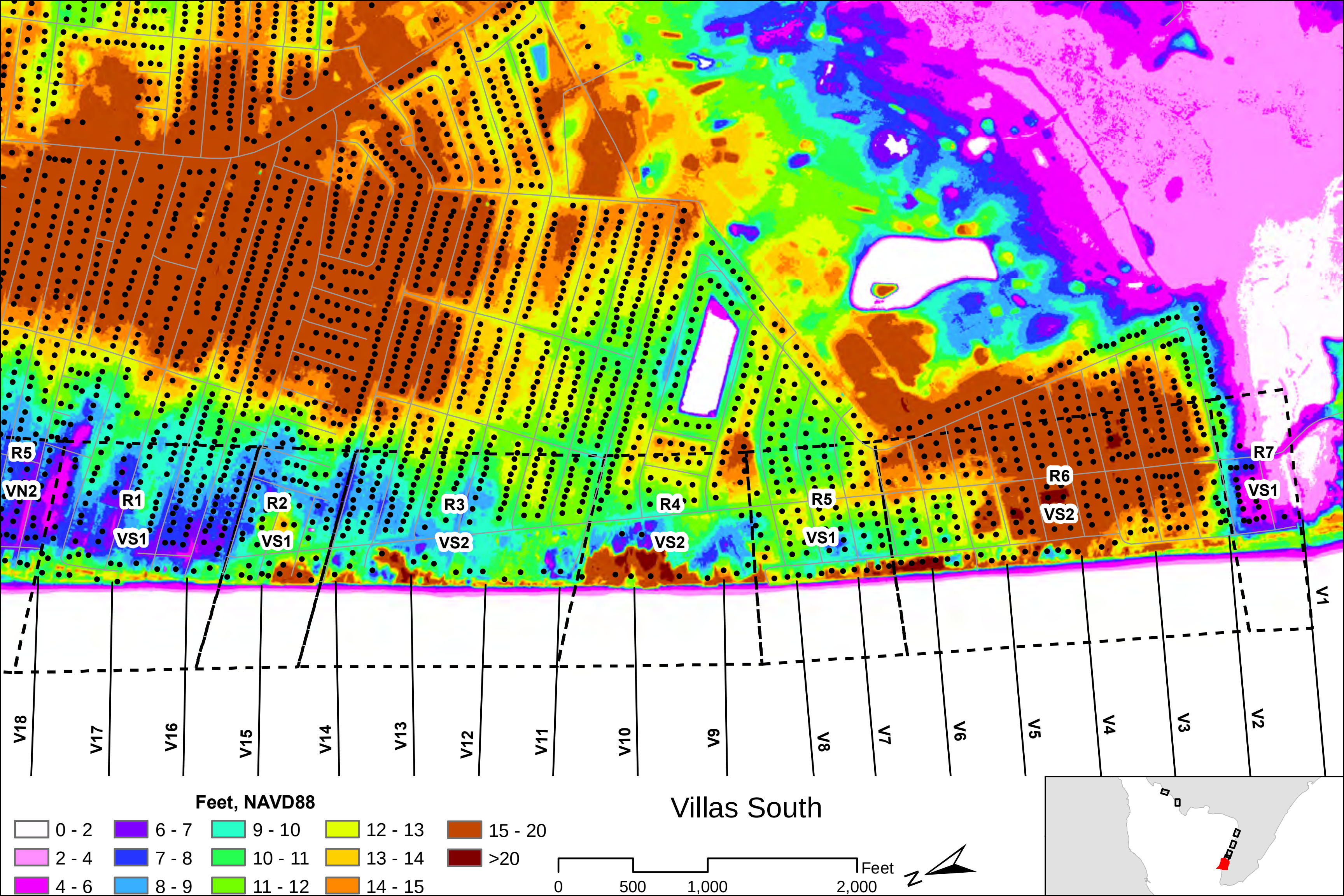
V26
V25
V24
V23
V22
V21
V20
V19
V18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R1
VS1
R2
VS1
R3
VS2
R4
VS2
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VS1
R6
VS2
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VS1
V18
V17
V16
V15
V14
V13
V12
V11
V10
V9
V8
V7
V6
V5
V4
V3
V2
V1
Feet, NAVD88
0 - 2
2 - 4
4 - 6
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7 - 8
8 - 9
9 - 10
10 - 11
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14 - 15
15 - 20
>20
Villas South
0
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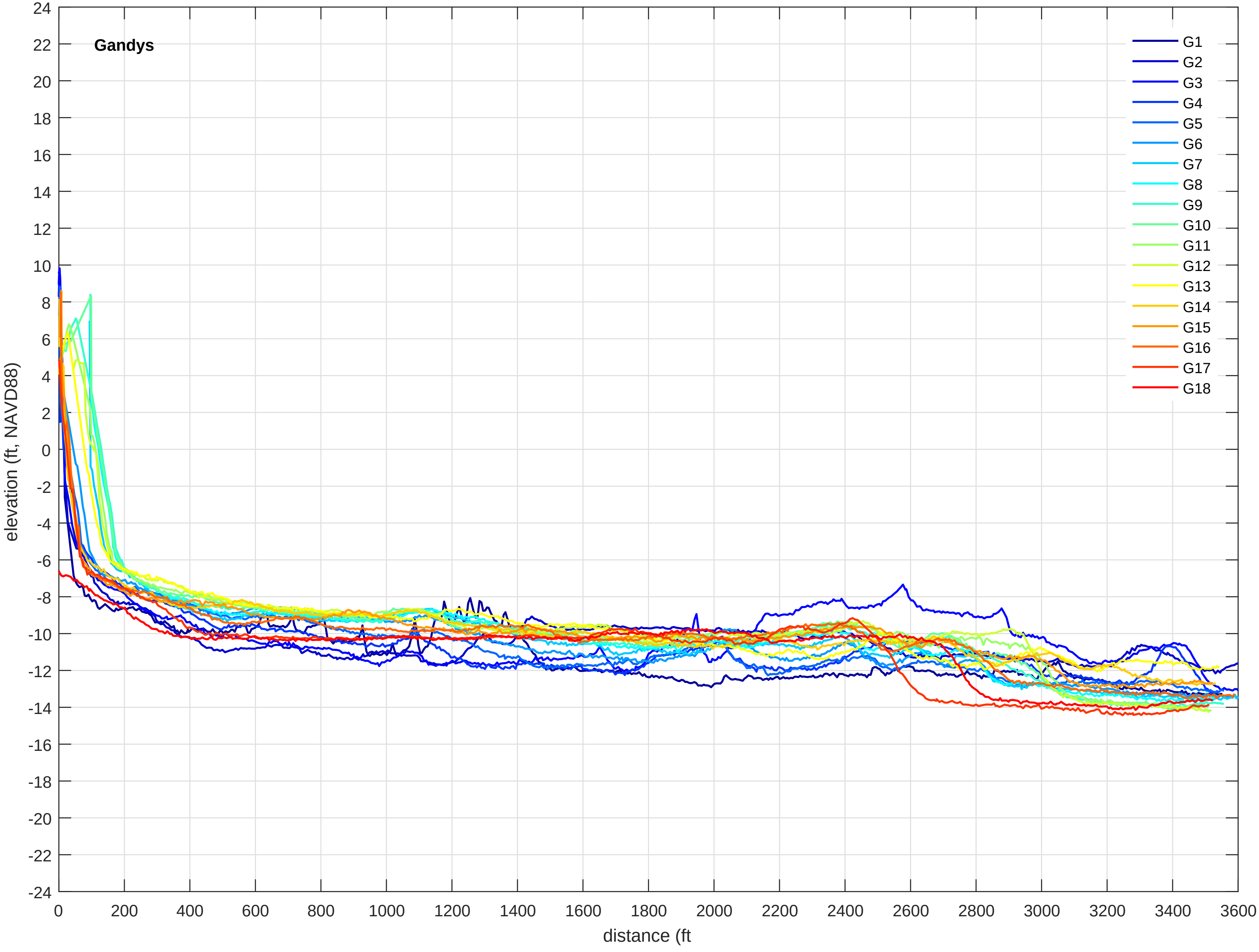
2,000
Feet
N


Gandys
G1
G2
G3
G4
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G6
G7
G8
G9
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G11
G12
G13
G14
G15
G16
G17
G18
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
10
8
6
4
2
0
-2
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-6
-8
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-12
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-16
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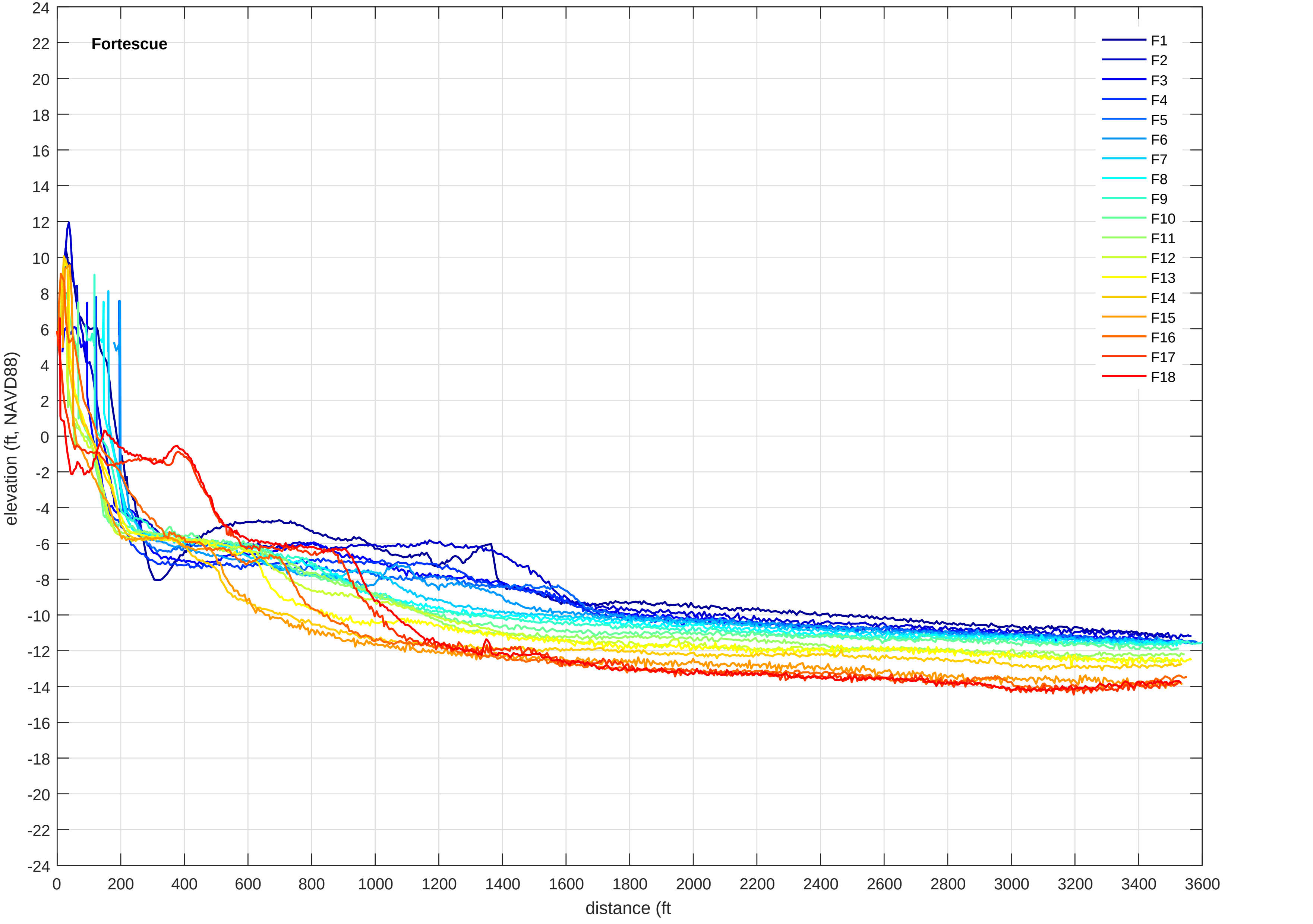

Fortescue
F1
F2
F3
F4
F5
F6
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F8
F9
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F11
F12
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F14
F15
F16
F17
F18
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
10
8
6
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2
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-2
-4
-6
-8
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-12
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-16
-18
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-22
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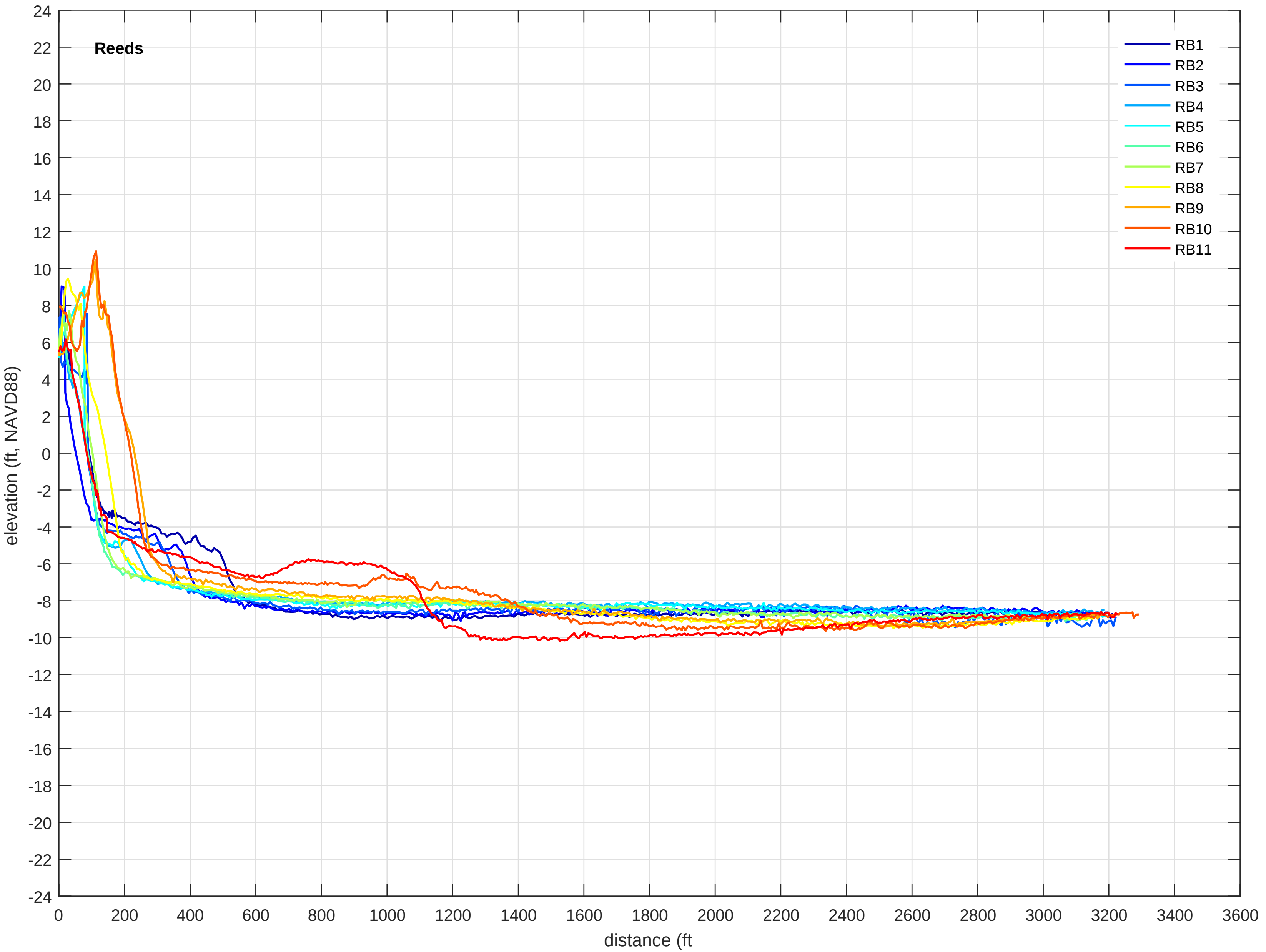

Reeds
RB1
RB2
RB3
RB4
RB5
RB6
RB7
RB8
RB9
RB10
RB11
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
10
8
6
4
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-2
-4
-6
-8
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-16
-18
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-22
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3000
3200
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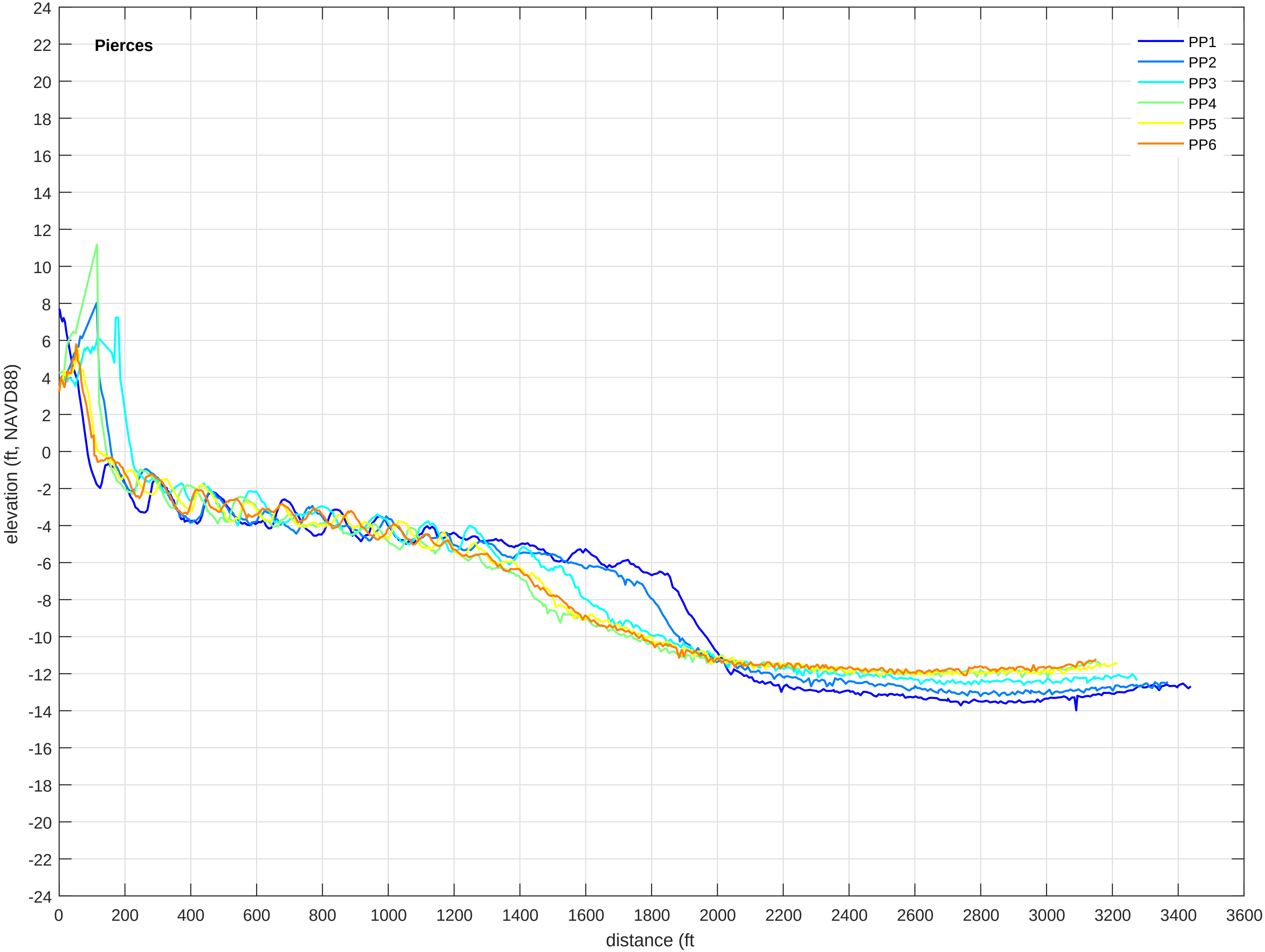
3600


Pierces
PP1
PP2
PP3
PP4
PP5
PP6
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
10
8
6
4
2
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-2
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-6
-8
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-12
-14
-16
-18
-20
-22
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2600
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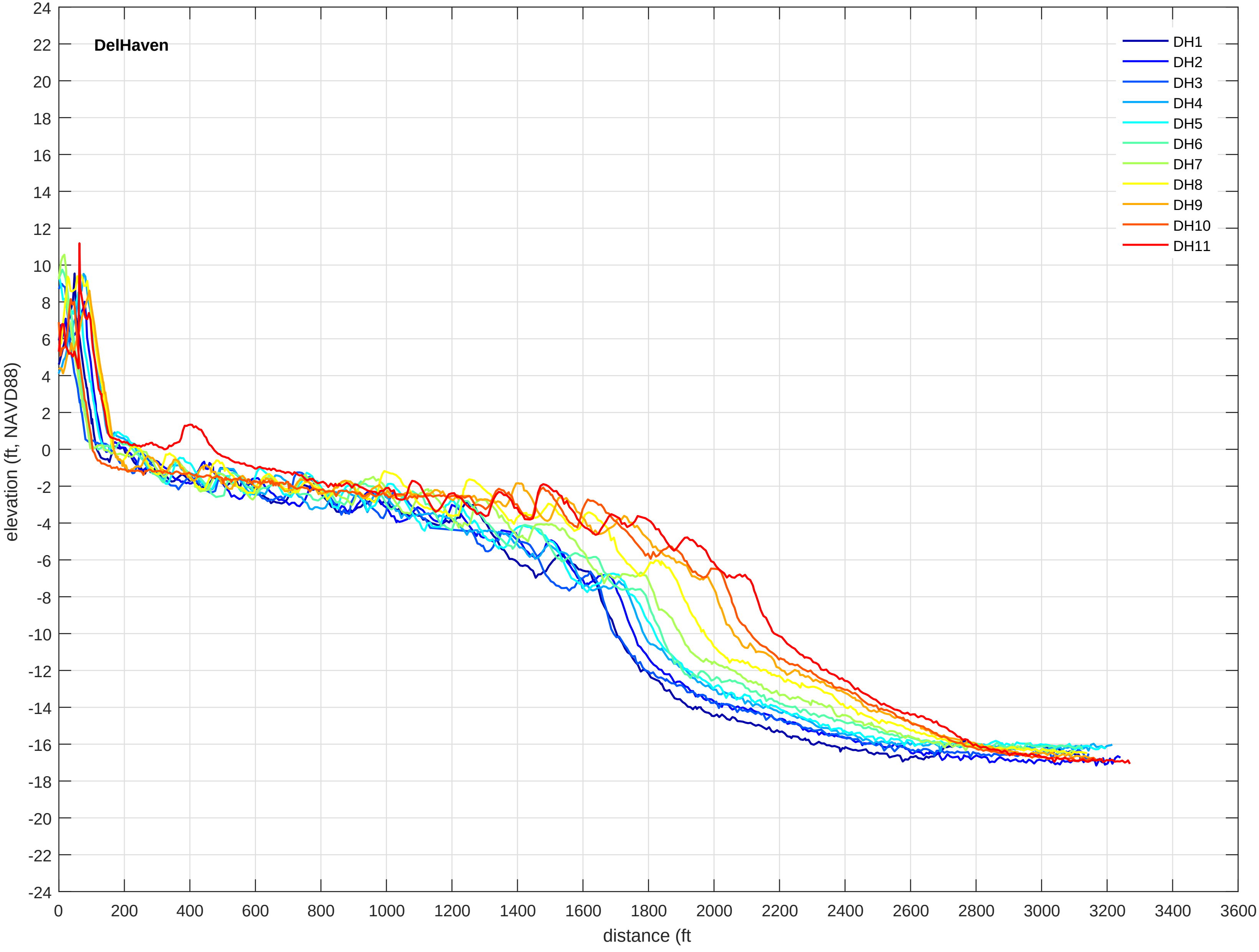
3600



DelHaven
DH1
DH2
DH3
DH4
DH5
DH6
DH7
DH8
DH9
DH10
DH11
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
10
8
6
4
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0
-2
-4
-6
-8
-10
-12
-14
-16
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-22
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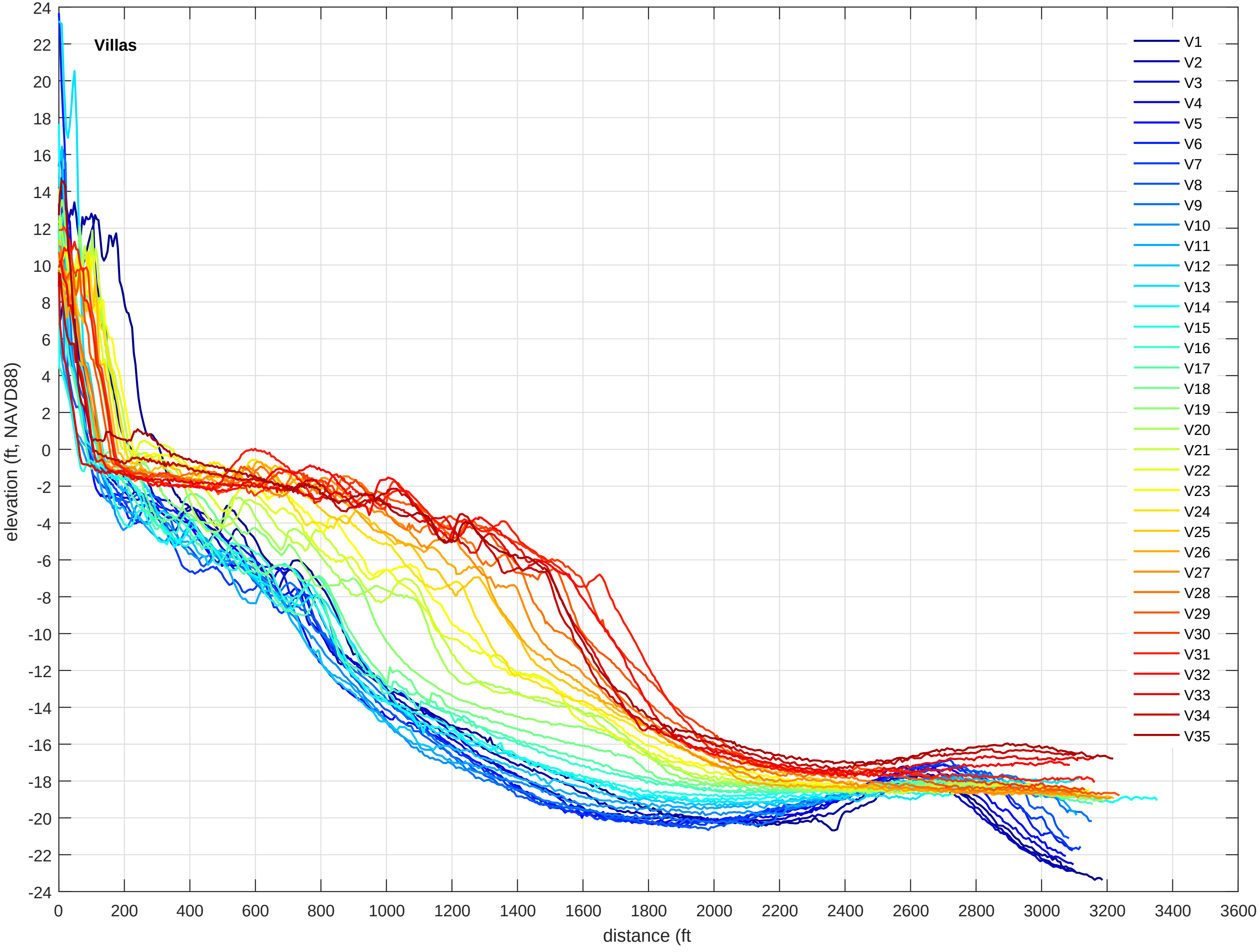

Villas
elevation (ft, NAVD88)
distance (ft
24
22
20
18
16
14
12
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8
6
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-22
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2600
2800
3000
3200
3400
3600
V1
V2
V3
V4
V5
V6
V7
V8
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V10
V11
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V13
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V16
V17
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V21
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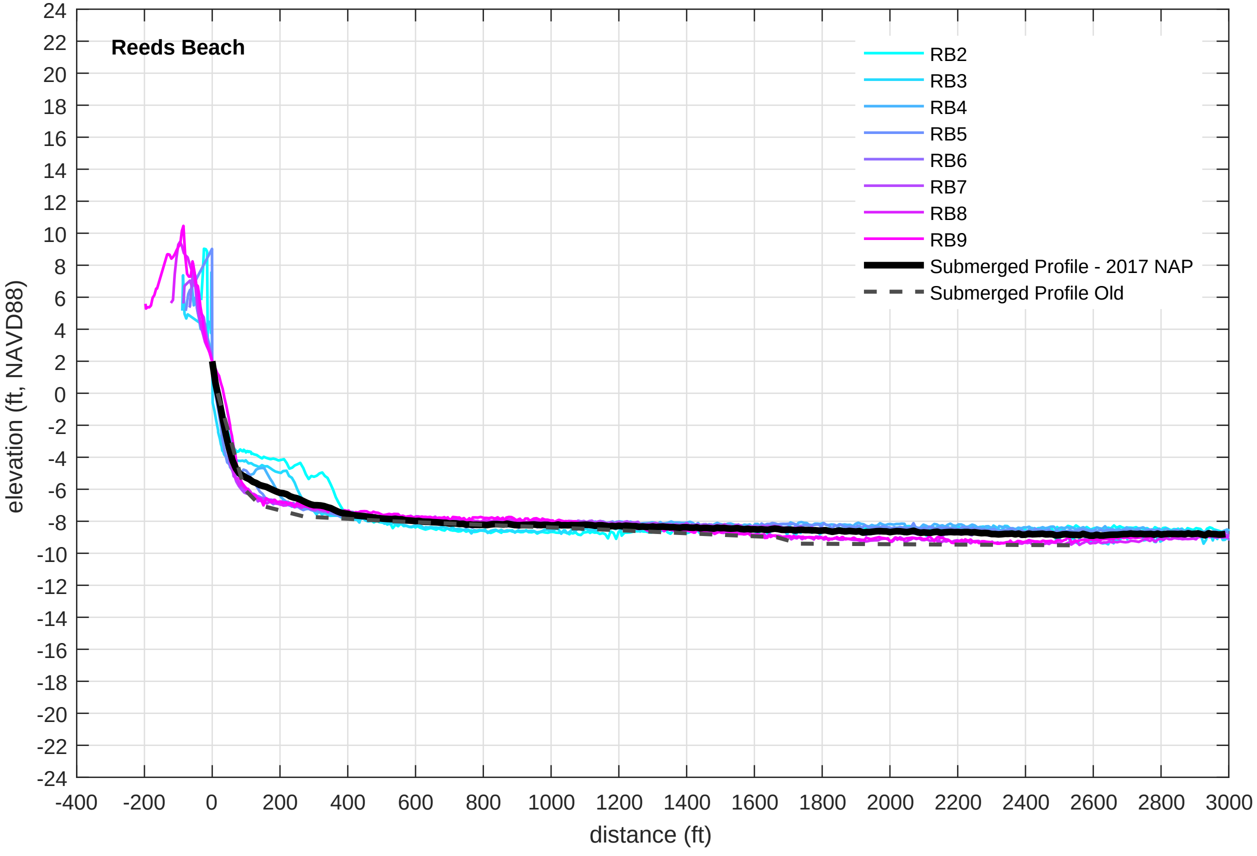
V27
V28
V29
V30
V31
V32
V33
V34
V35


Reeds Beach
RB2
RB3
RB4
RB5
RB6
RB7
RB8
RB9
Submerged Profile - 2017 NAP
Submerged Profile Old
elevation (ft, NAVD88)
distance (ft)
24
22
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18
16
14
12
10
8
6
4
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-2
-4
-6
-8
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-12
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-16
-18
-20
-22
-24
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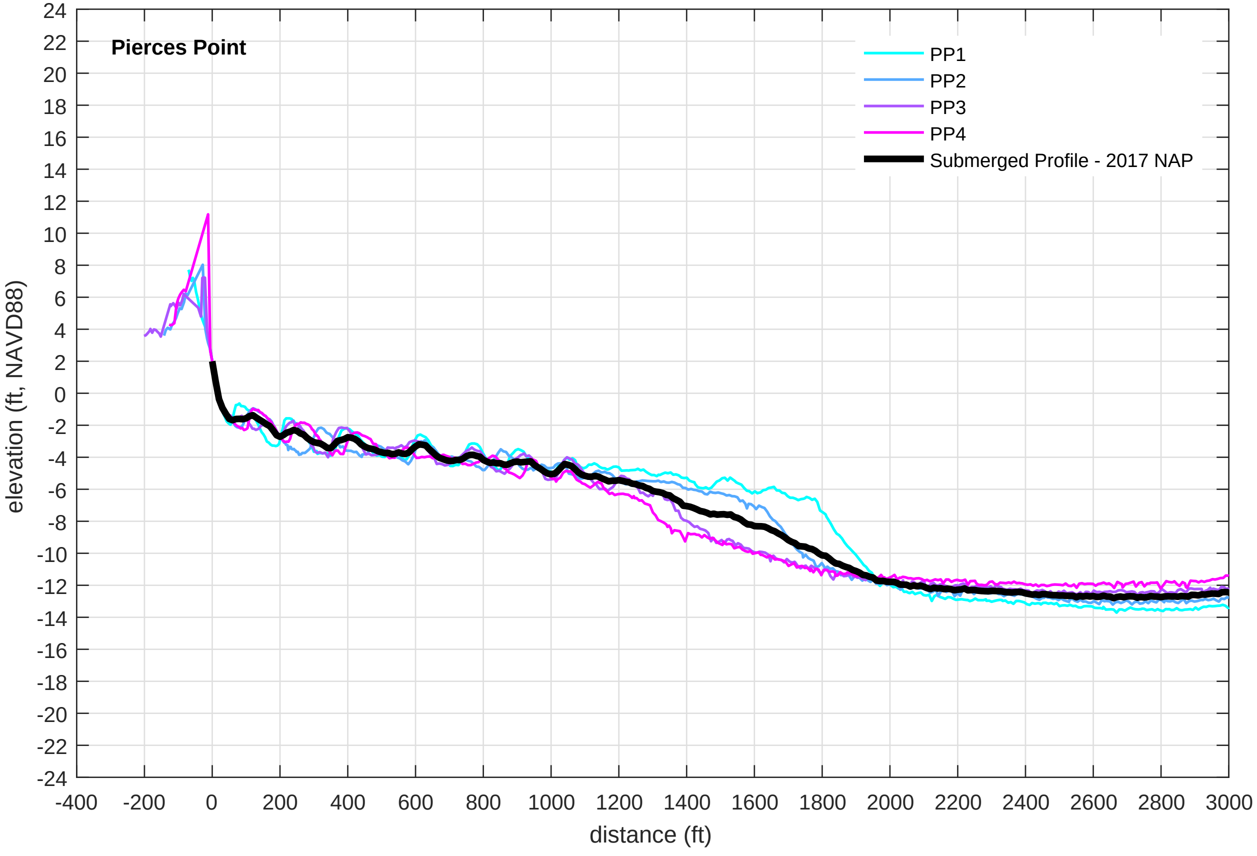
Pierces Point
PP1
PP2
PP3
PP4
Submerged Profile - 2017 NAP
elevation (ft, NAVD88)
distance (ft)
24
22
20
18
16
14
12
10
8
6
4
2
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-2
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-6
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-10
-12
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-16
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-22
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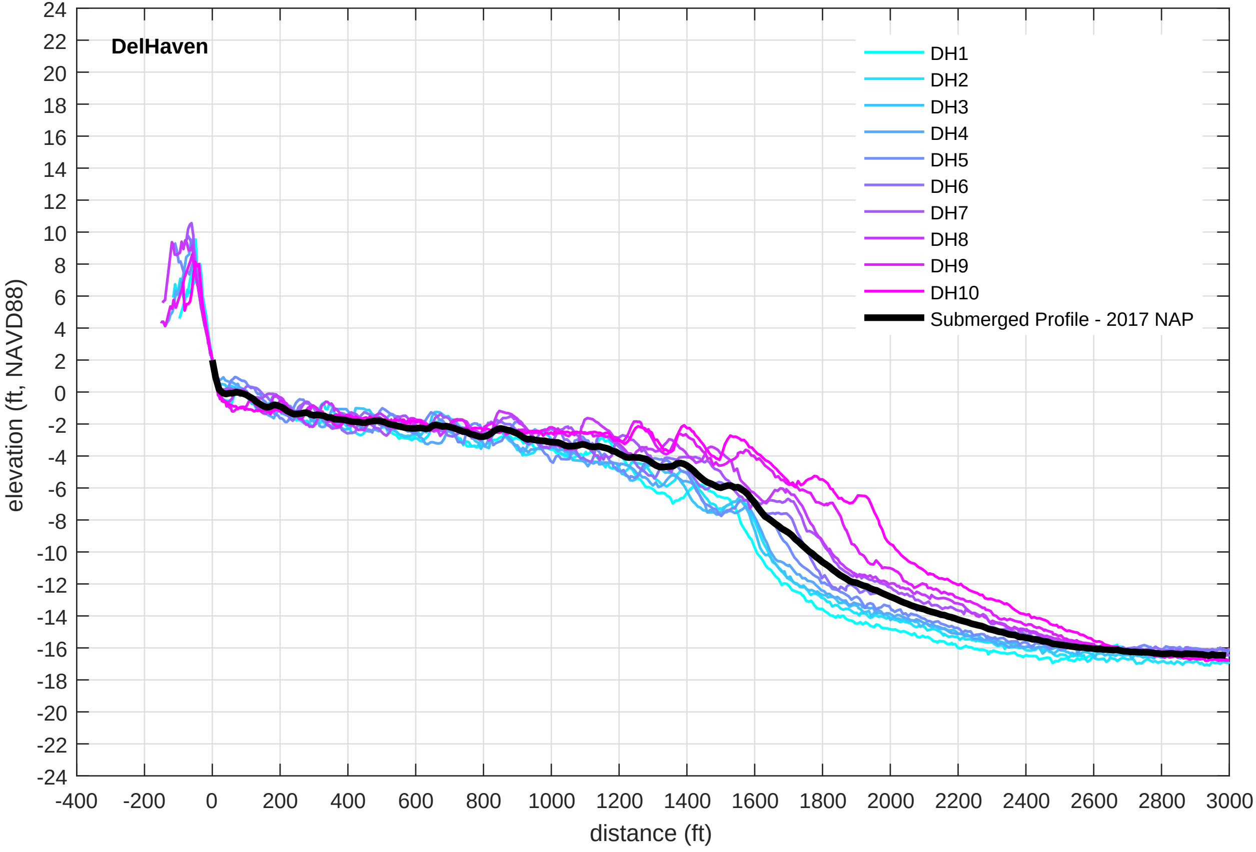
2800
3000


DelHaven
DH1
DH2
DH3
DH4
DH5
DH6
DH7
DH8
DH9
DH10
Submerged Profile - 2017 NAP
elevation (ft, NAVD88)
distance (ft)
24
22
20
18
16
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-2
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-6
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-10
-12
-14
-16
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-20
-22
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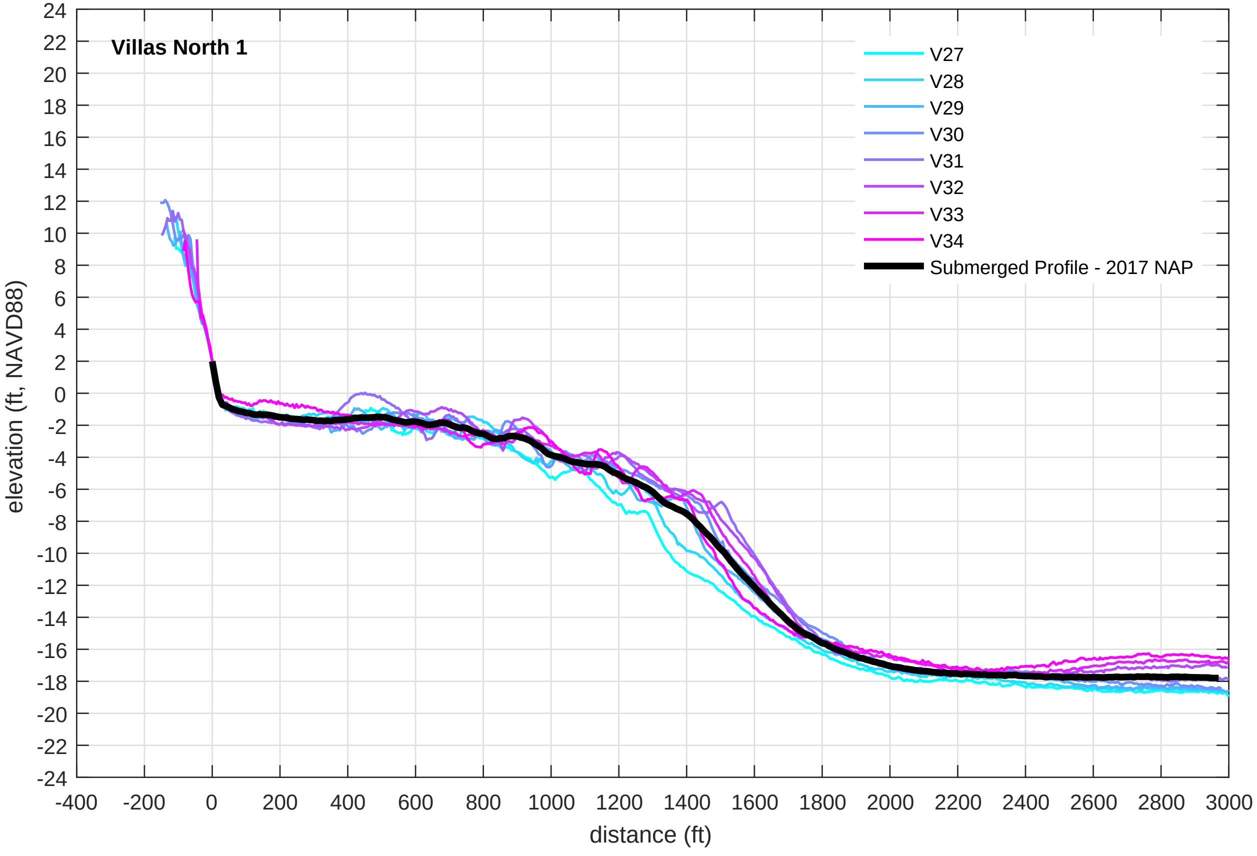
Villas North 1
V27
V28
V29
V30
V31
V32
V33
V34
Submerged Profile - 2017 NAP
elevation (ft, NAVD88)
distance (ft)
24
22
20
18
16
14
12
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8
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4
2
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-2
-4
-6
-8
-10
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-16
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-22
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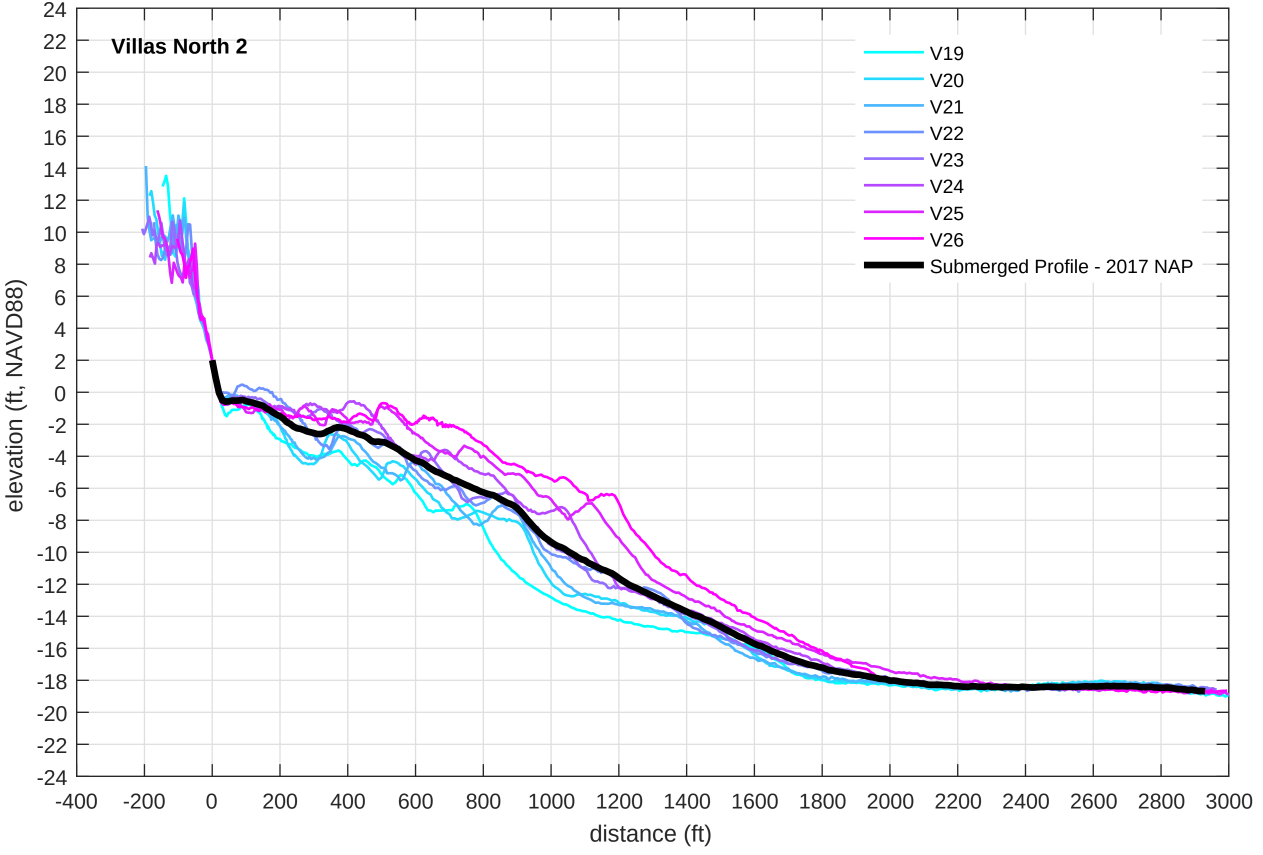

Villas North 2
V19
V20
V21
V22
V23
V24
V25
V26
Submerged Profile - 2017 NAP
elevation (ft, NAVD88)
distance (ft)
24
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
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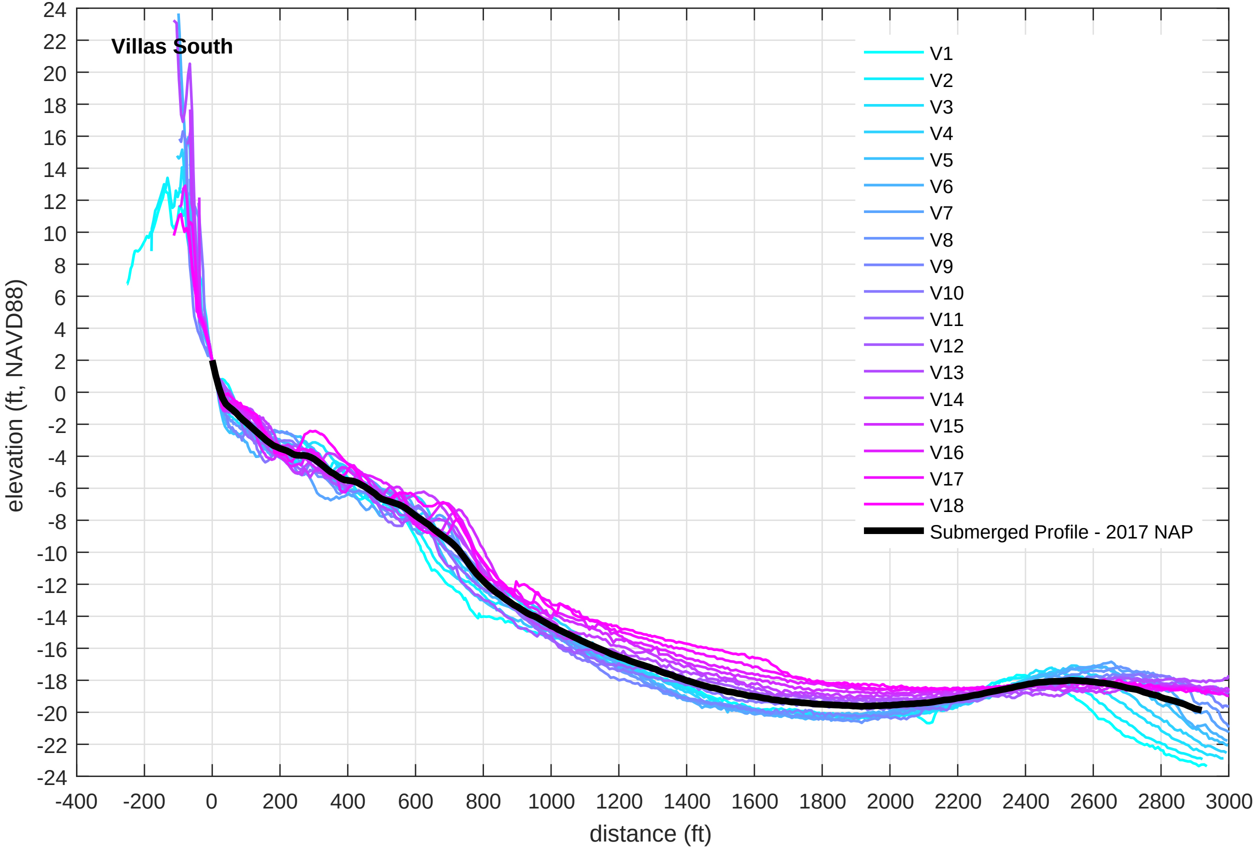
3000


Villas South
V1
V2
V3
V4
V5
V6
V7
V8
V9
V10
V11
V12
V13
V14
V15
V16
V17
V18
Submerged Profile - 2017 NAP
elevation (ft, NAVD88)
distance (ft)
24
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-400
-200
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2000
2200
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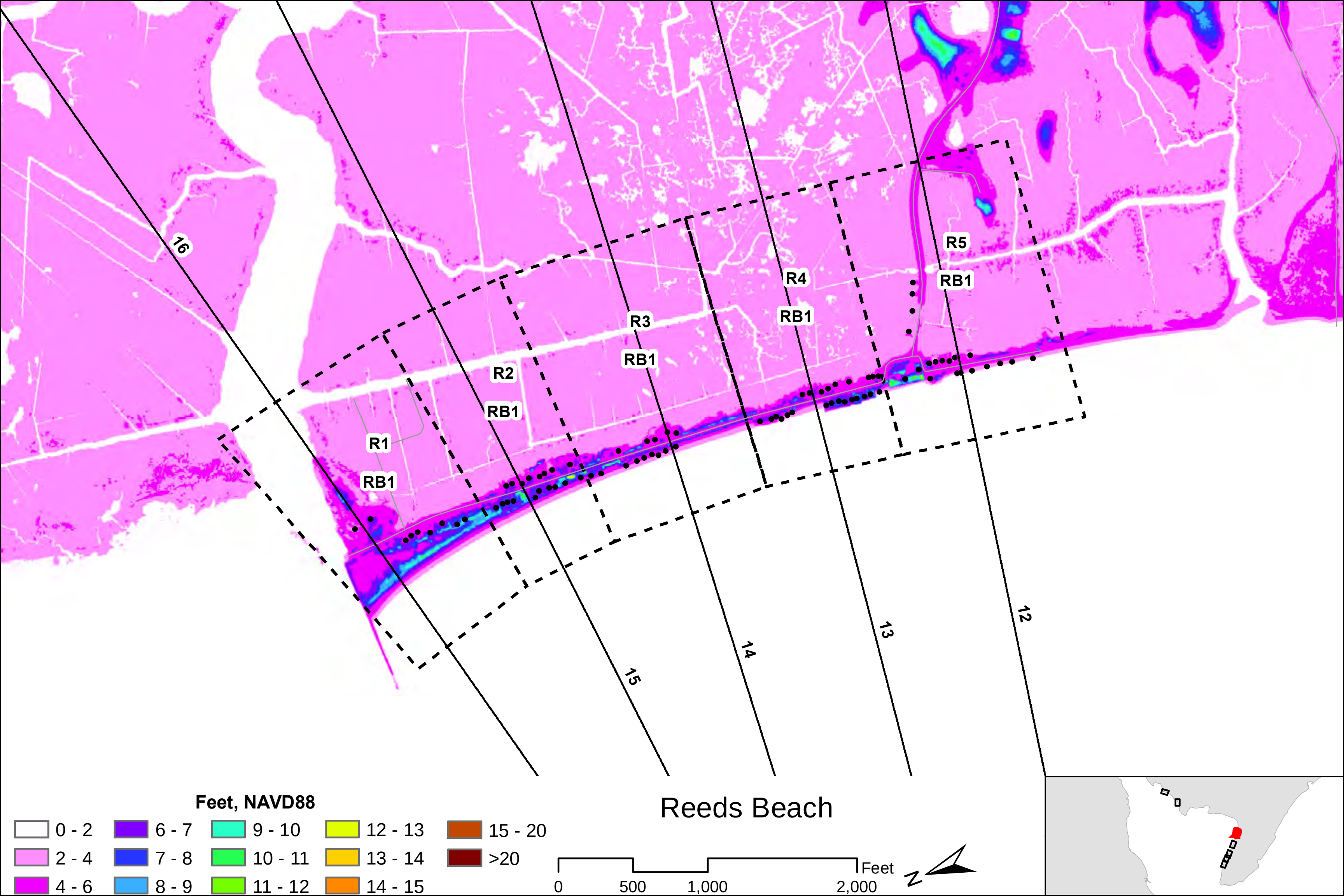
2600
2800
3000


16
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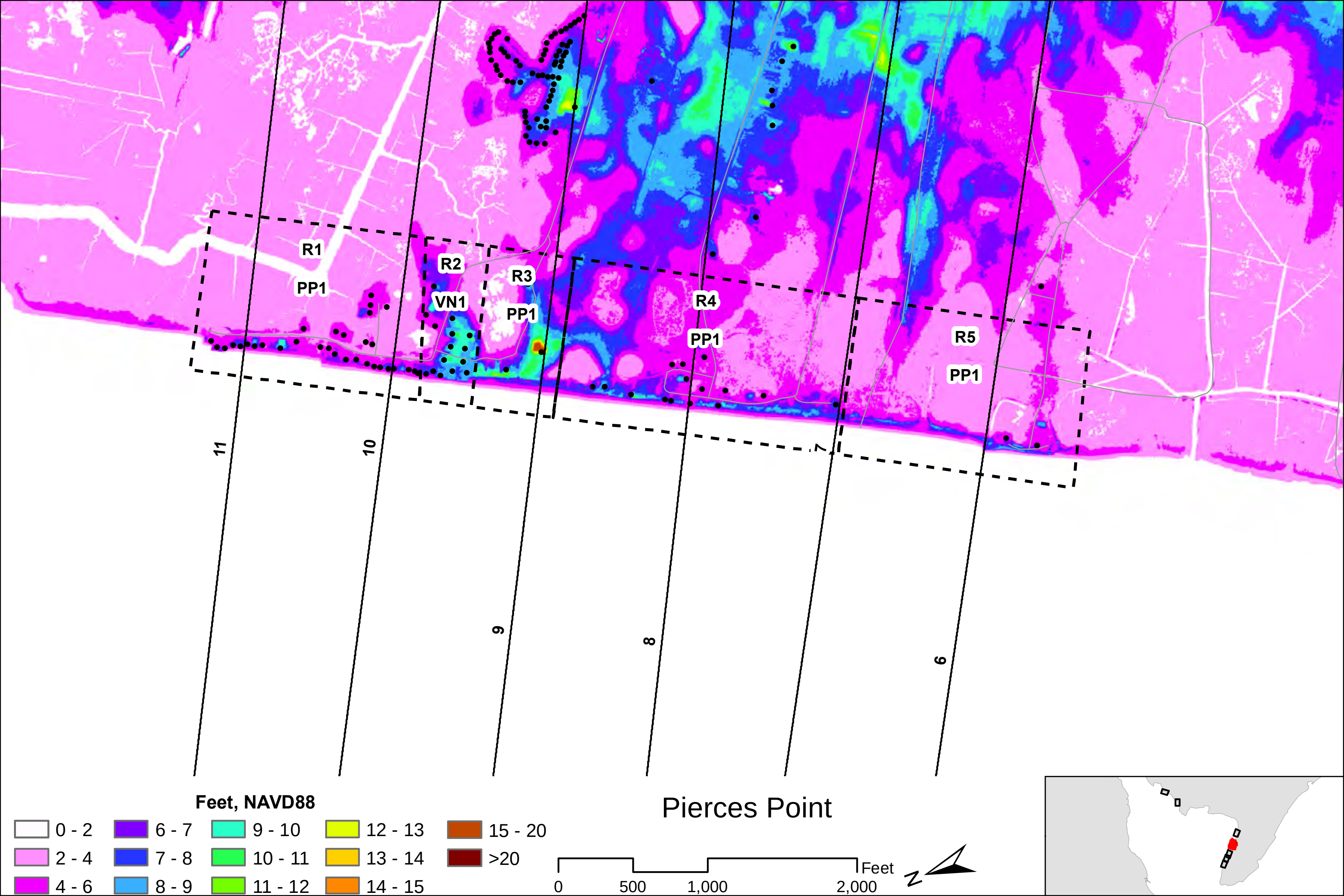
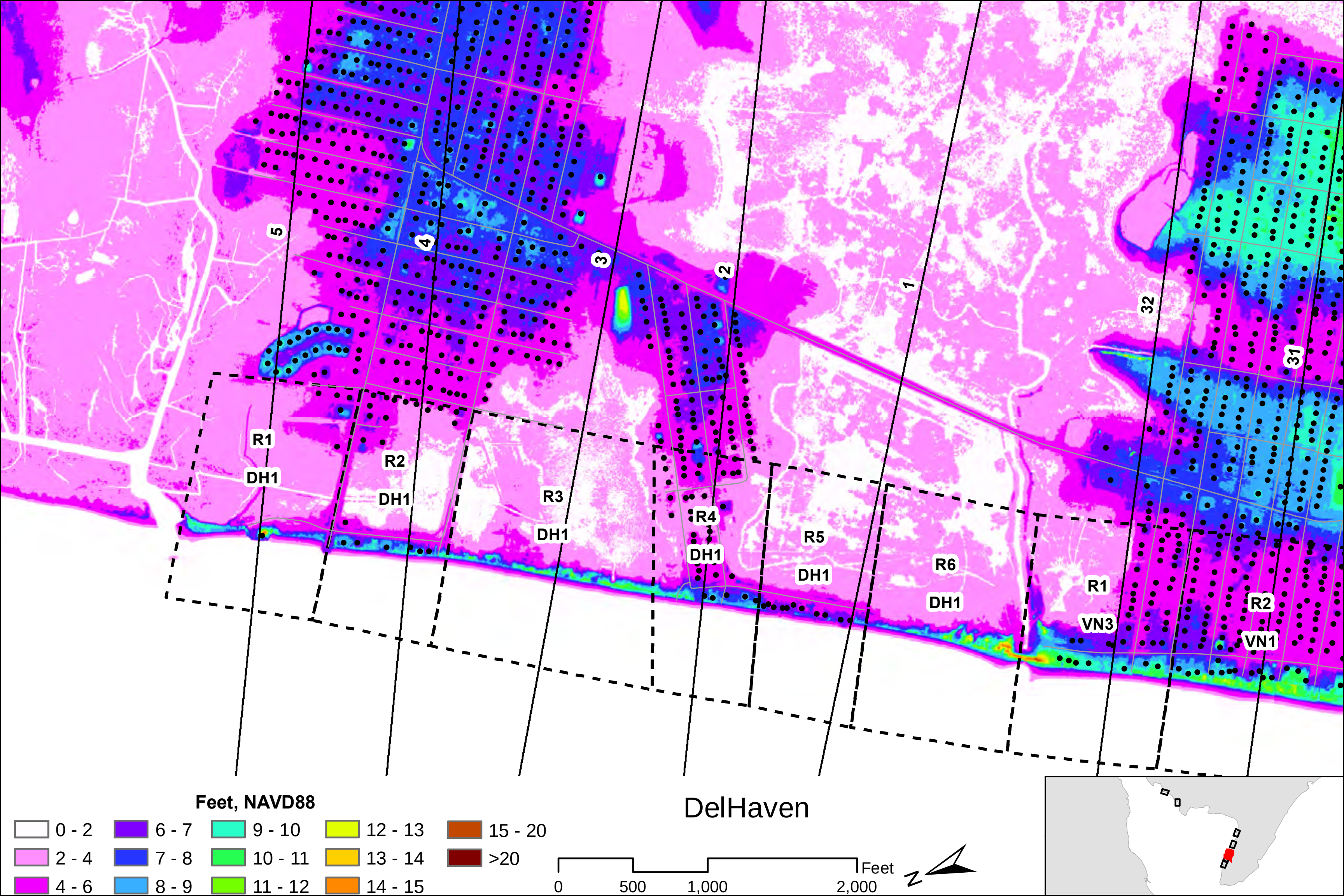
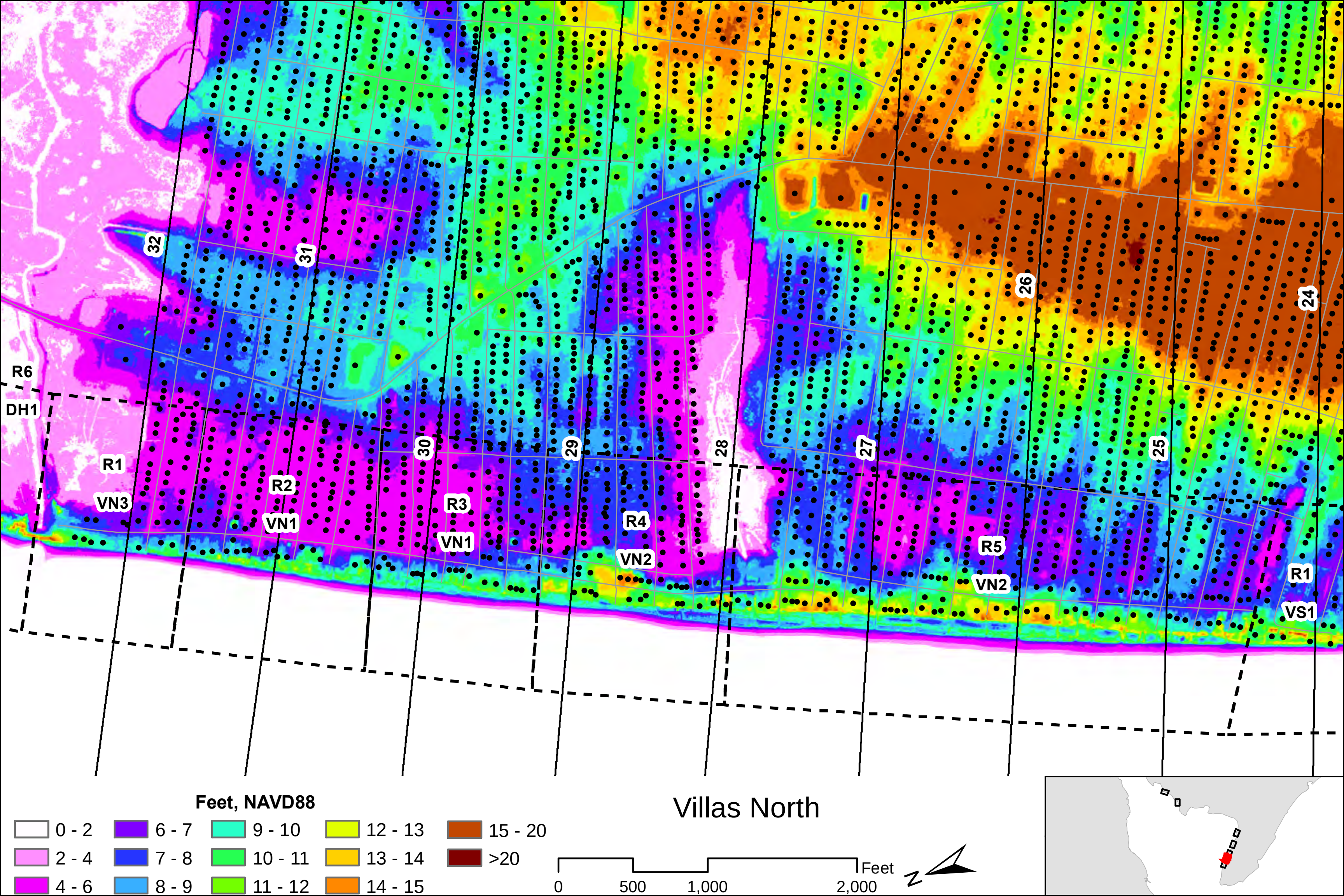
14
13
12
R1
RB1
R2
RB1
R3
RB1
R4
RB1
R5
RB1
Feet, NAVD88
0 - 2
2 - 4
4 - 6
6 - 7
7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
Reeds Beach
0
500
1,000
2,000
Feet
N


R1
PP1
R2
VN1
R3
PP1
R4
PP1
R5
PP1
11
10
9
8
7
6
Feet, NAVD88
0 - 2
2 - 4
4 - 6
6 - 7
7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
Pierces Point
0
500
1,000
2,000
Feet
N


5
4
3
2
1
32
31
R1
DH1
R2
DH1
R3
DH1
R4
DH1
R5
DH1
R6
DH1
R1
VN3
R2
VN1
Feet, NAVD88
0 - 2
2 - 4
4 - 6
6 - 7
7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
DelHaven
0
500
1,000
2,000
Feet
N



32
31
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28
27
26
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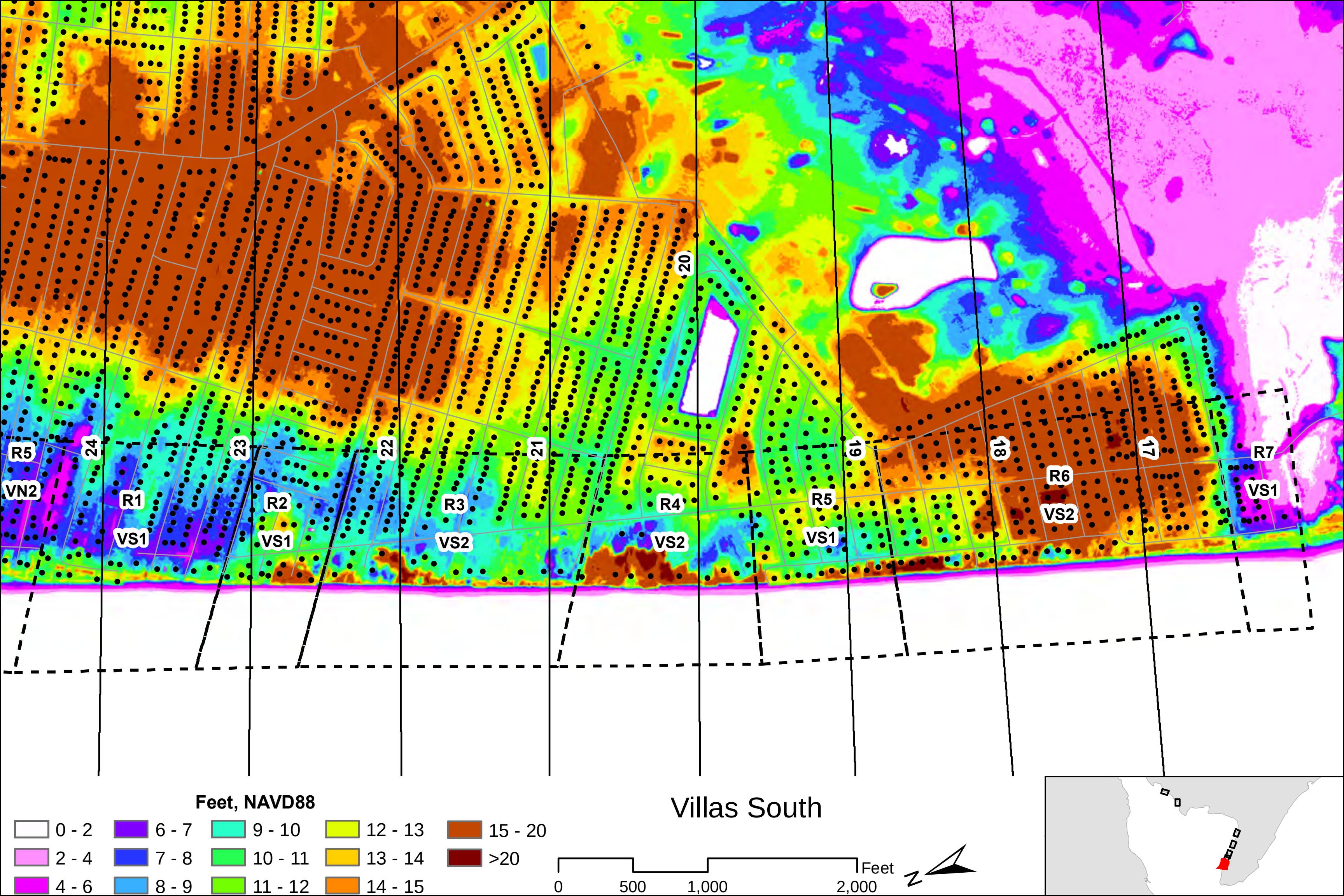
24
R6
DH1
R1
VN3
R2
VN1
R3
VN1
R4
VN2
R5
VN2
R1
VS1
Feet, NAVD88
0 - 2
2 - 4
4 - 6
6 - 7
7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
Villas North
0
500
1,000
2,000
Feet
N



24
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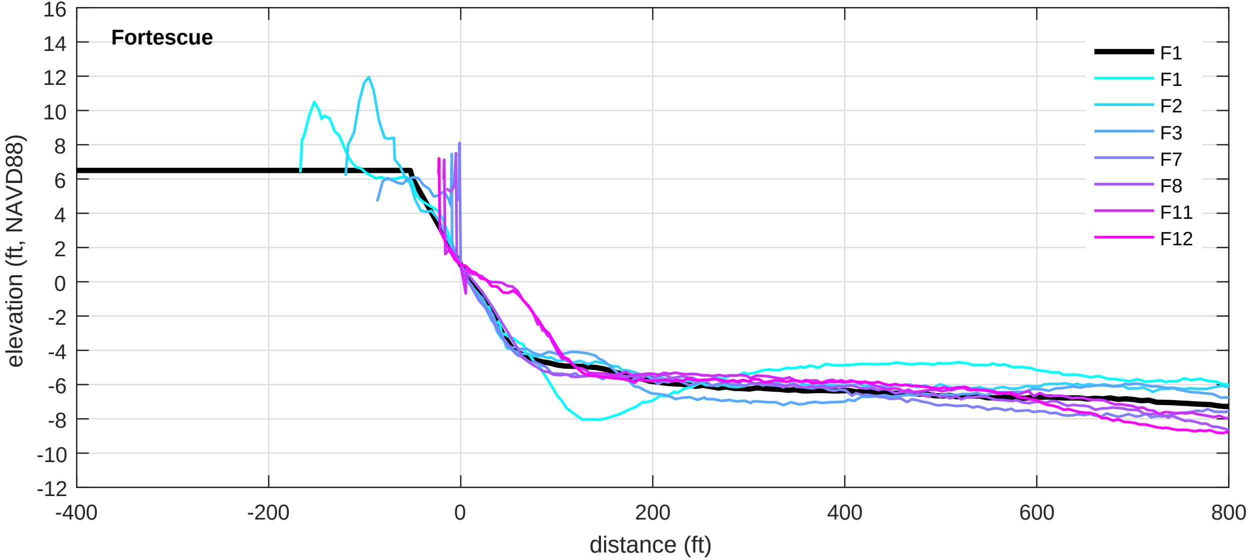
21
20
19
18
17
R5
VN2
R1
VS1
R2
VS1
R3
VS2
R4
VS2
R5
VS1
R6
VS2
R7
VS1
Feet, NAVD88
0 - 2
2 - 4
4 - 6
6 - 7
7 - 8
8 - 9
9 - 10
10 - 11
11 - 12
12 - 13
13 - 14
14 - 15
15 - 20
>20
Villas South
0
500
1,000
2,000
Feet
N


Fortescue
F1
F1
F2
F3
F7
F8
F11
F12
elevation (ft, NAVD88)
distance (ft)
16
14
12
10
8
6
4
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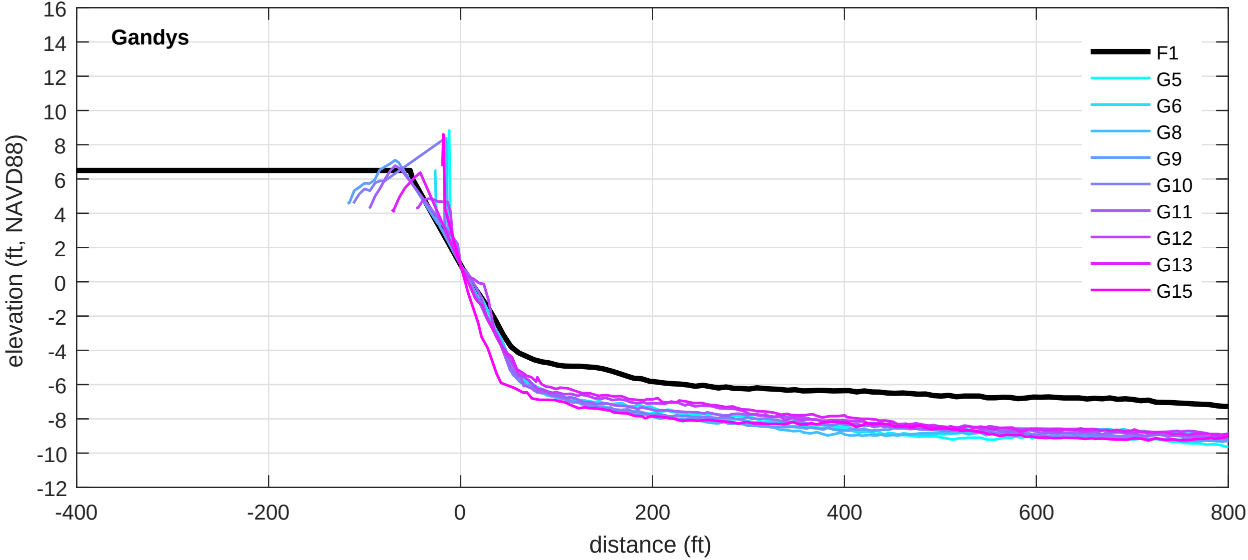
-4
-6
-8
-10
-12
-400
-200
0
200
400
600
800



Gandys
F1
G5
G6
G8
G9
G10
G11
G12
G13
G15
elevation (ft, NAVD88)
16
14
12
10
8
6
4
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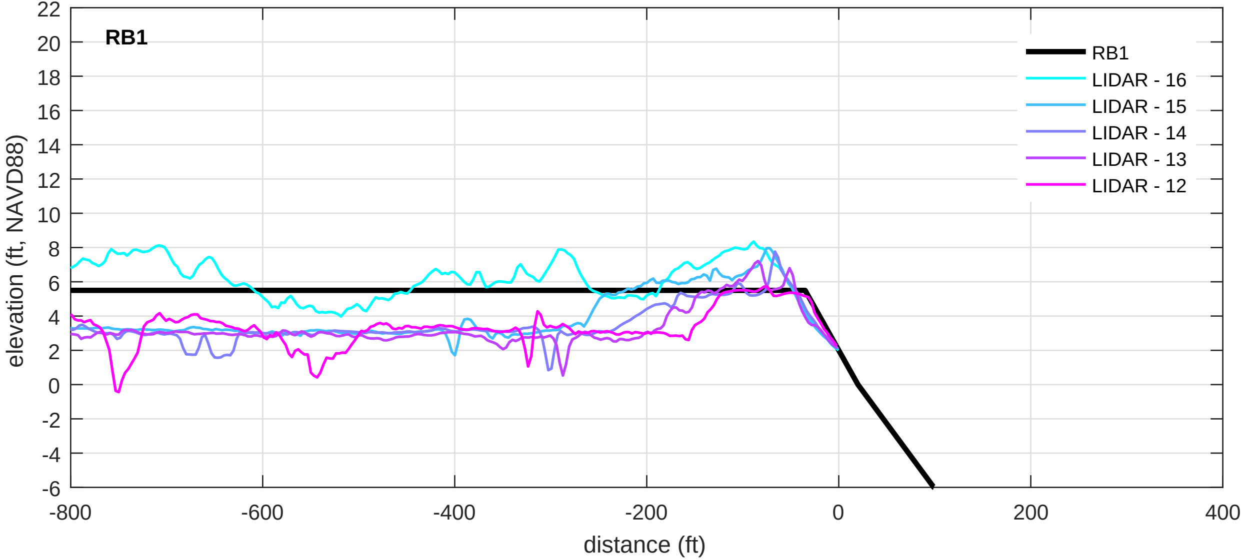
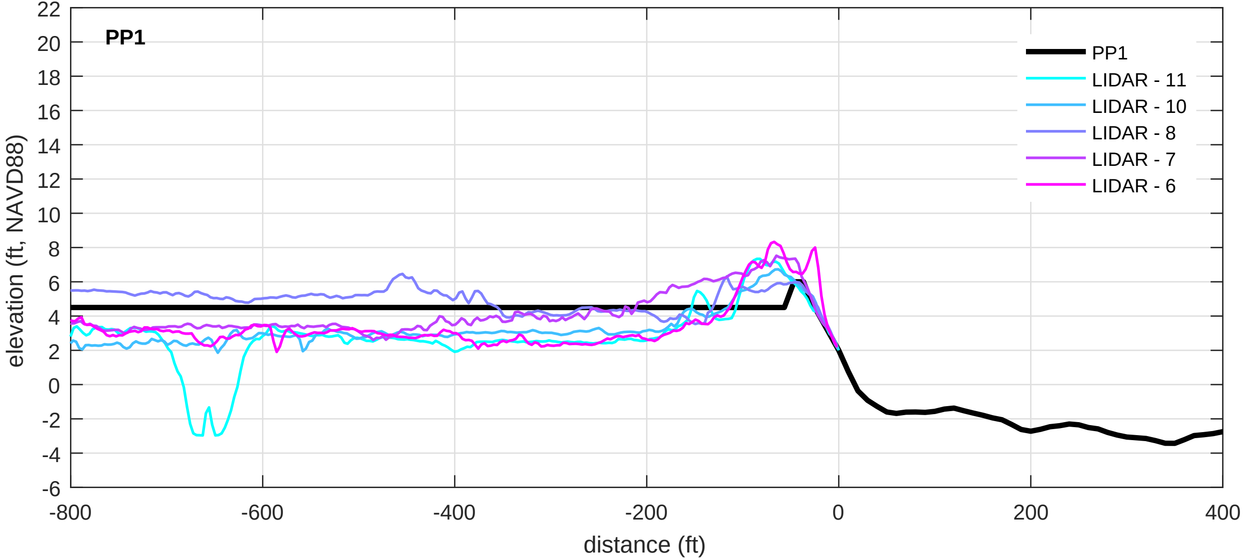
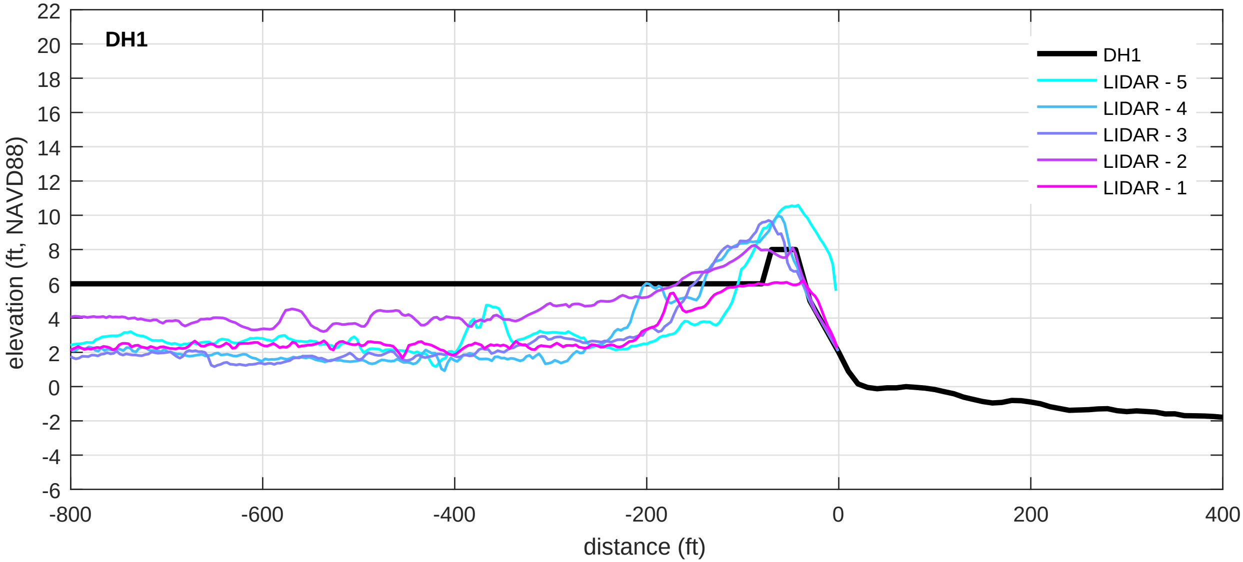
-4
-6
-8
-10
-12
-400
-200
0
200
400
600
800
distance (ft)


RB1
RB1
LIDAR - 16
LIDAR - 15
LIDAR - 14
LIDAR - 13
LIDAR - 12
elevation (ft, NAVD88)
distance (ft)
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-800
-600
-400
-200
0
200
400



PP1
PP1
LIDAR - 11
LIDAR - 10
LIDAR - 8
LIDAR - 7
LIDAR - 6
elevation (ft, NAVD88)
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-800
-600
-400
-200
0
200
400
distance (ft)



DH1
DH1
LIDAR - 5
LIDAR - 4
LIDAR - 3
LIDAR - 2
LIDAR - 1
elevation (ft, NAVD88)
distance (ft)
22
20
18
16
14
12
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-2
-4
-6
-800
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-400
-200
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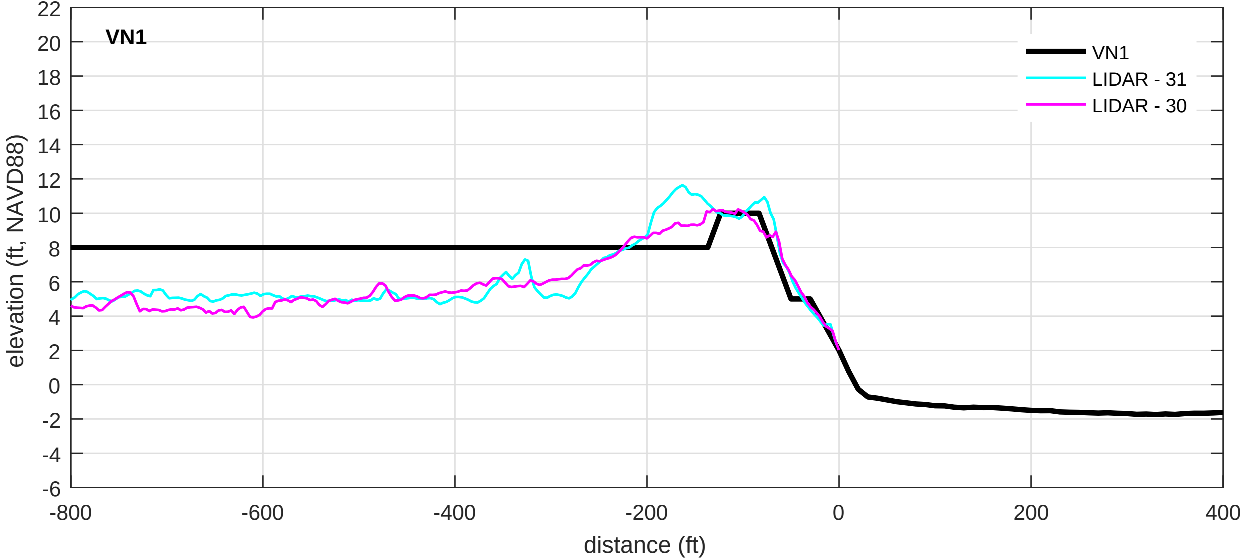
200
400



VN1
VN1
LIDAR - 31
LIDAR - 30
elevation (ft, NAVD88)
distance (ft)
22
20
18
16
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10
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6
4
2
0
-2
-4
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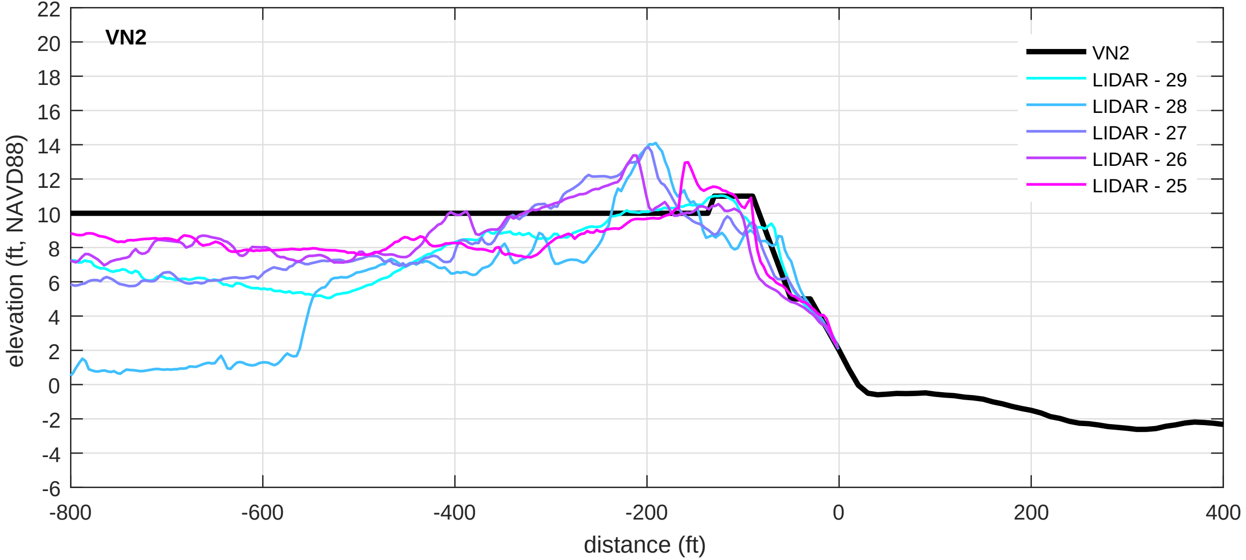
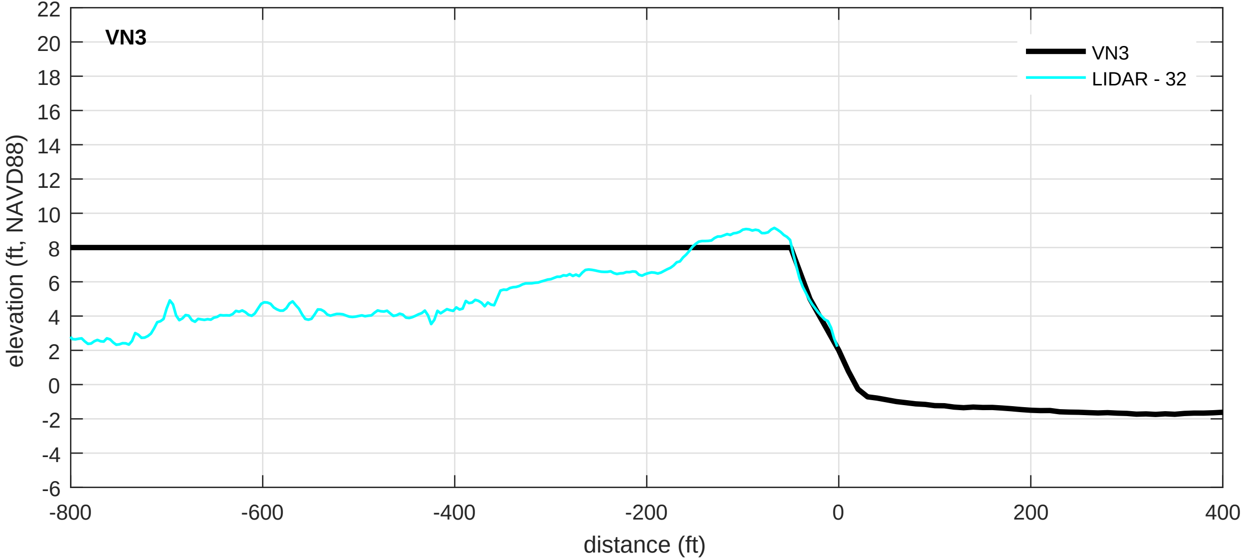
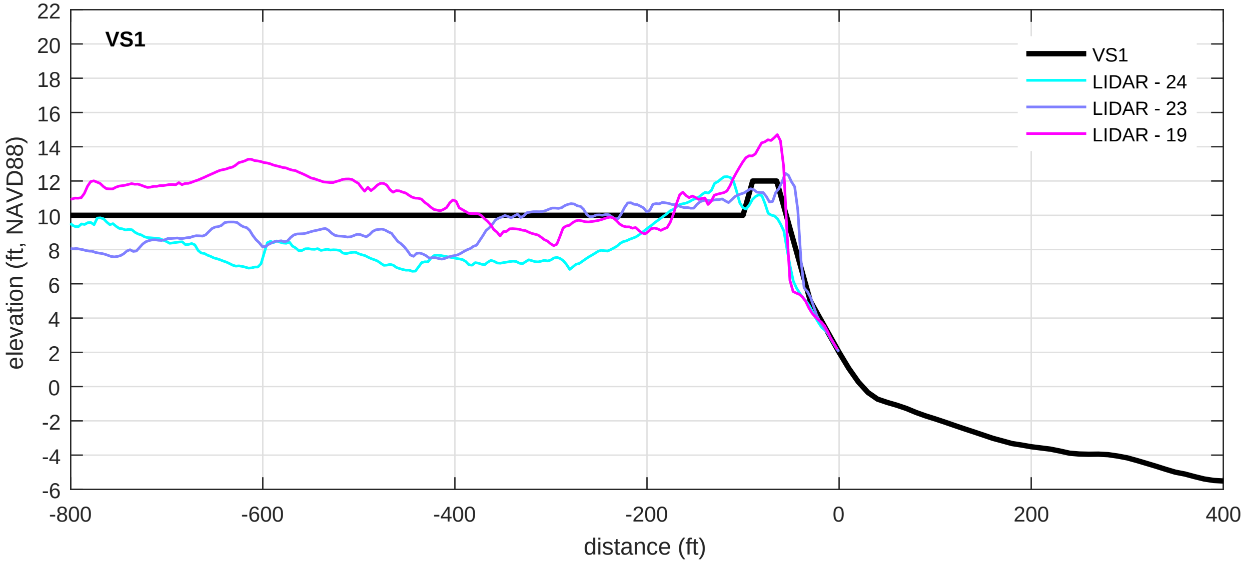
-400
-200
0
200
400

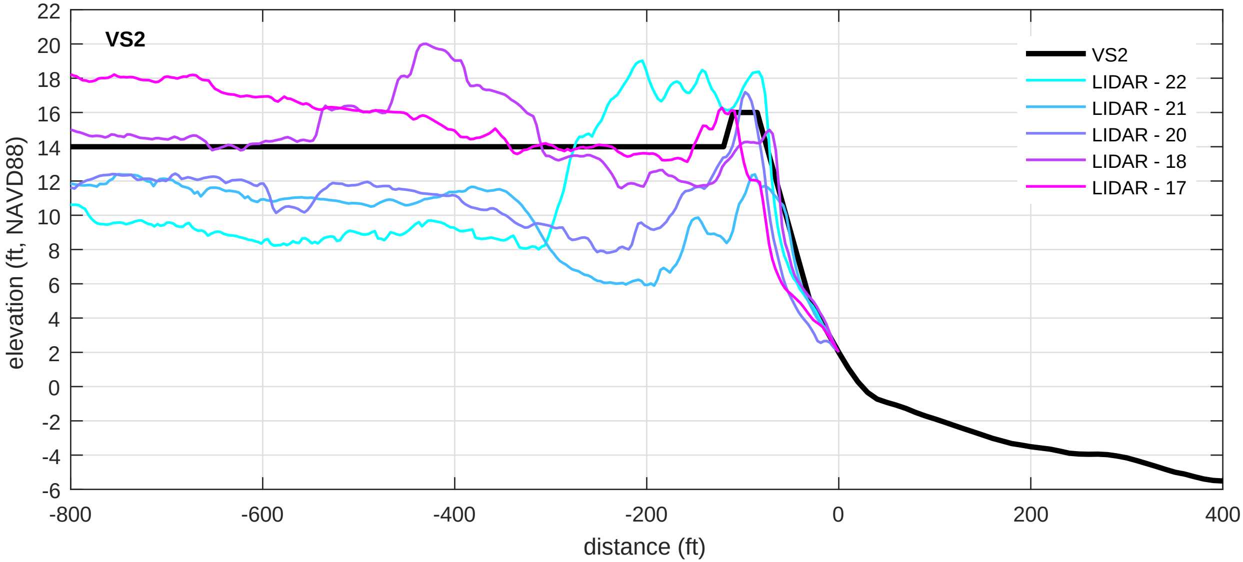


VN2
VN2
LIDAR - 29
LIDAR - 28
LIDAR - 27
LIDAR - 26
LIDAR - 25
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
elevation (ft, NAVD88)
-800
-600
-400
-200
0
200
400
distance (ft)


VN3
VN3
LIDAR - 32
elevation (ft, NAVD88)
distance (ft)
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-800
-600
-400
-200
0
200
400



VS1
VS1
LIDAR - 24
LIDAR - 23
LIDAR - 19
elevation (ft, NAVD88)
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-800
-600
-400
-200
0
200
400
distance (ft)


VS2
VS2
LIDAR - 22
LIDAR - 21
LIDAR - 20
LIDAR - 18
LIDAR - 17
elevation (ft, NAVD88)
distance (ft)
22
20
18
16
14
12
10
8
6
4
2
0
-2
-4
-6
-800
-600
-400
-200
0
200
400

ATTACHMENT C.3 BEACH-FX WITH PROJECT DUNE ALIGNMENT

October 2017

BEACH-FX:
WITH PROJECT DUNE ALIGNMENT

New Jersey DMU

21 July 2017

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OVERVIEW

Purpose:

- Provide With Project Beach-fx inputs
  - Dune dimensions that reflect civil design constraints
  - Diffusion losses.

Sites:

- Gandys
- Fortescue
- Reeds
- Pierces
- Del Haven
- Villas North
- Villas South

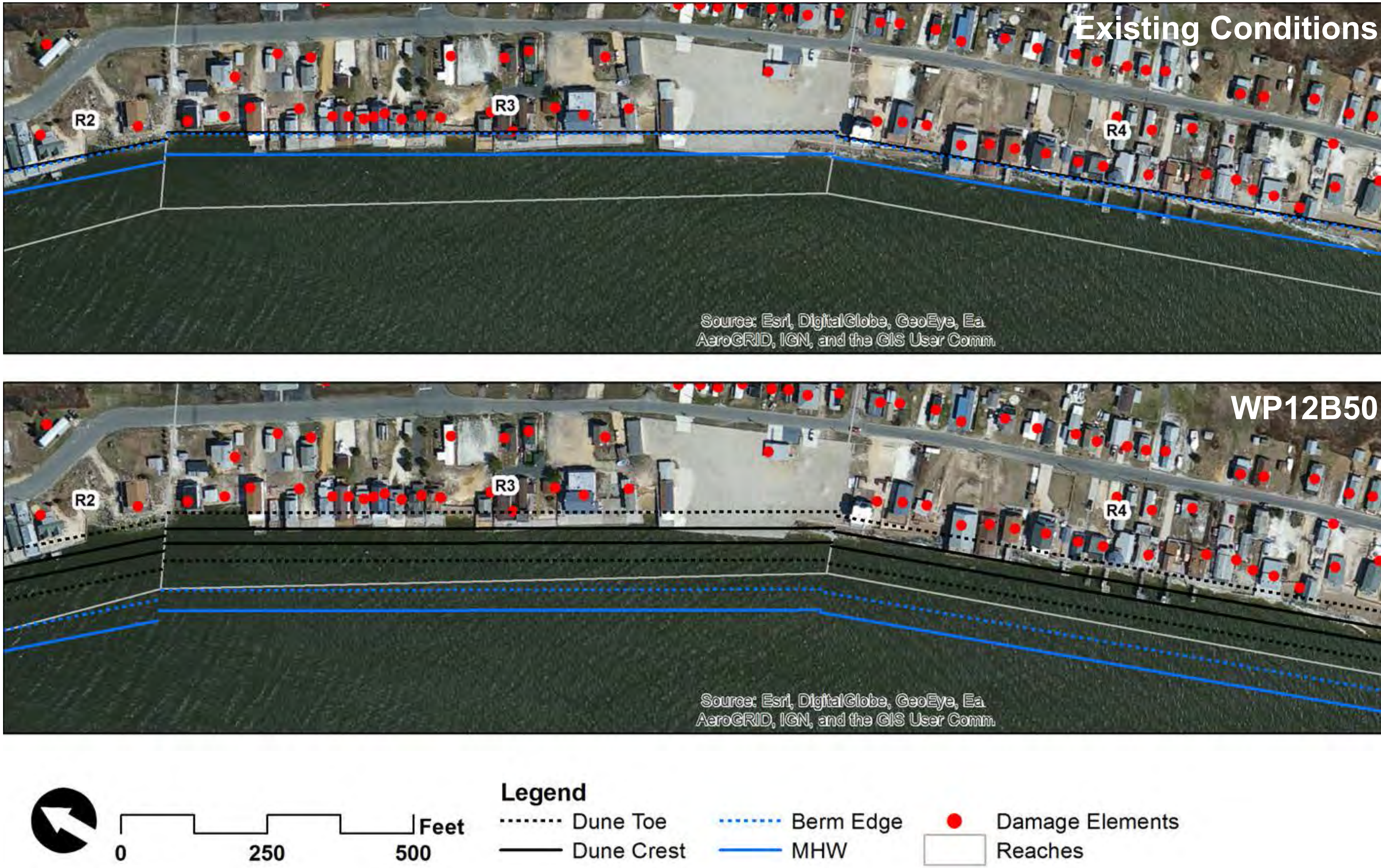
GANDYS ALIGNMENT

Feet
0 250 500

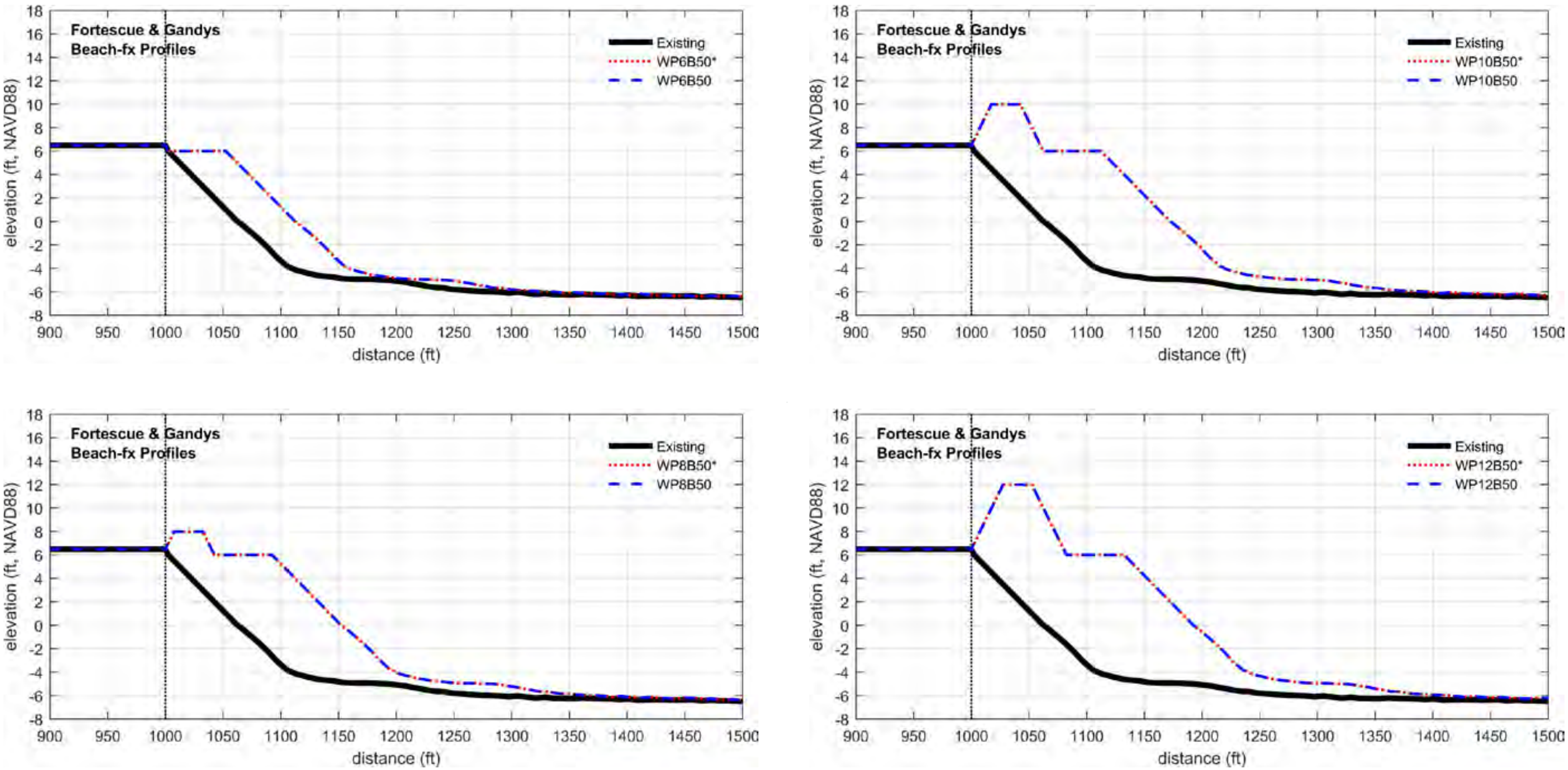
Legend

- Dune Toe
- Dune Crest
- Berm Edge
- MHW
- Damage Elements
- Reaches

FORTESCUE BEACH ALIGNMENT



GANDYS AND FORTESCUE – DESIGN VS BEACH-FX



*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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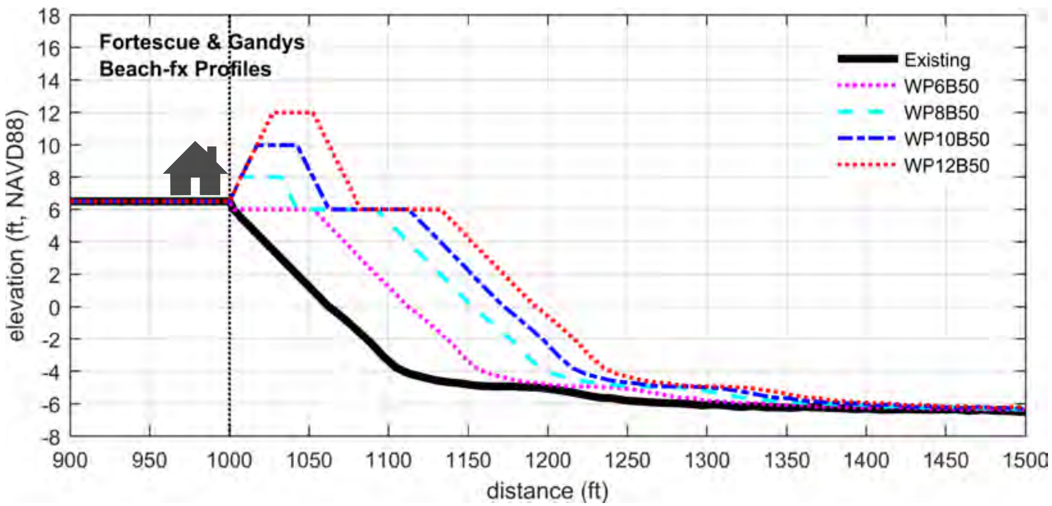
GANDYS – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+6.5	0	0	50	-2.2
WP6B50	+6.5	0	50	100	-4.4
WP8B50	+8	25	50	140	-6.2
WP10B50	+10	25	50	160	-7.0
WP12B50	+12	25	50	180	-7.9



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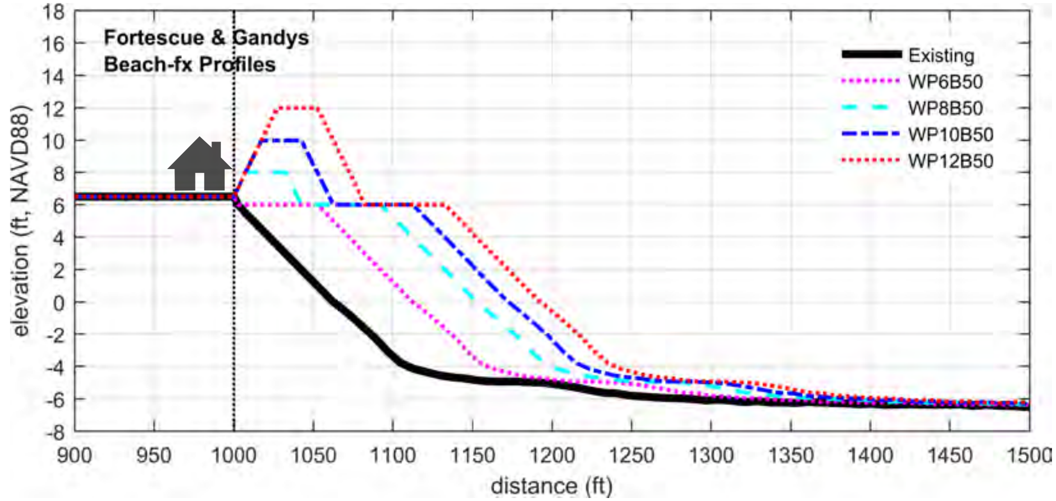
FORTESCUE – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+6.5	0	0	50	-1.4
WP6B50	+6.5	0	50	100	-2.8
WP8B50	+8	25	50	140	-3.9
WP10B50	+10	25	50	160	-4.5
WP12B50	+12	25	50	180	-5.0



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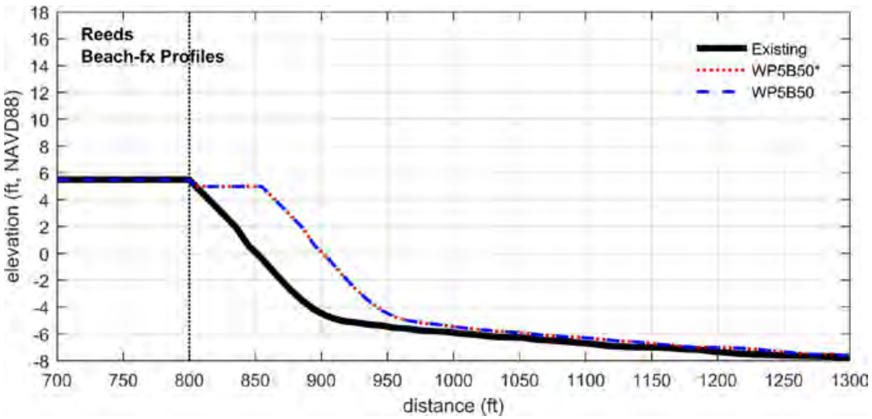
REEDS BEACH ALIGNMENT

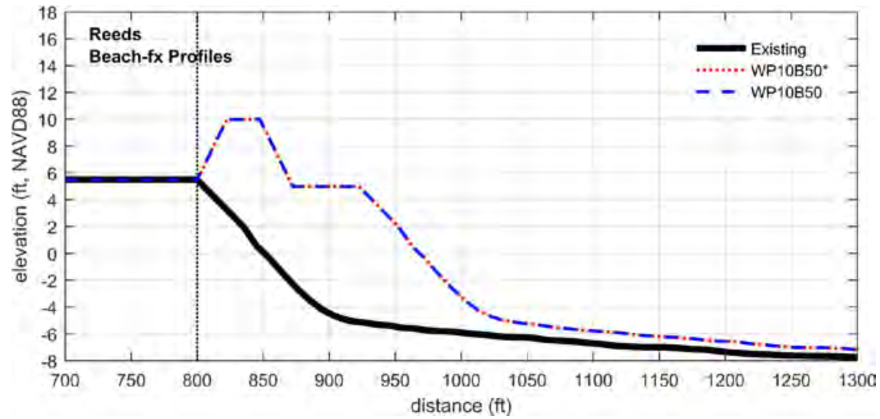
Feet: 0, 250, 500

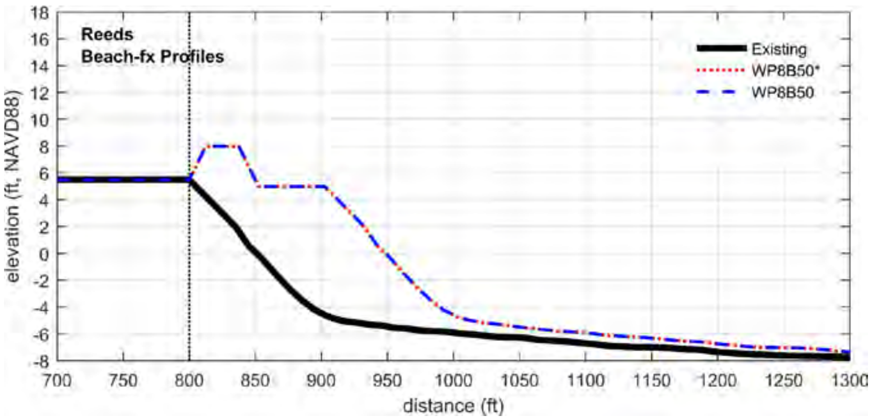
Legend

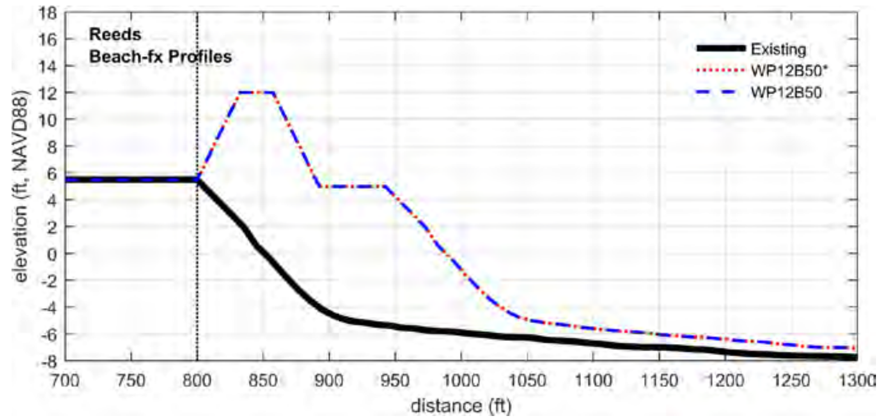
- Dune Toe (dotted black line)
- Dune Crest (solid black line)
- Berm Edge (dotted blue line)
- MHW (solid blue line)
- Damage Elements (red dot)
- Reaches (box)

REEDS – DESIGN VS BEACH-FX









*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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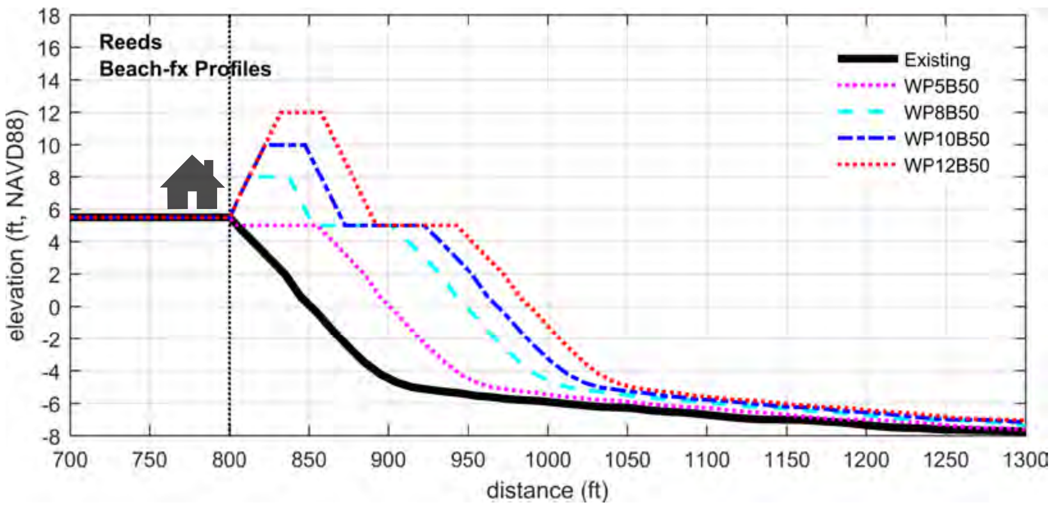
REEDS – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+5.5	0	0	0	0
WP5B50	+5.5	0	50	50	-1.2
WP8B50	+8	25	50	97.5	-2.3
WP10B50	+10	25	50	117.5	-2.8
WP12B50	+12	25	50	137.5	-3.3



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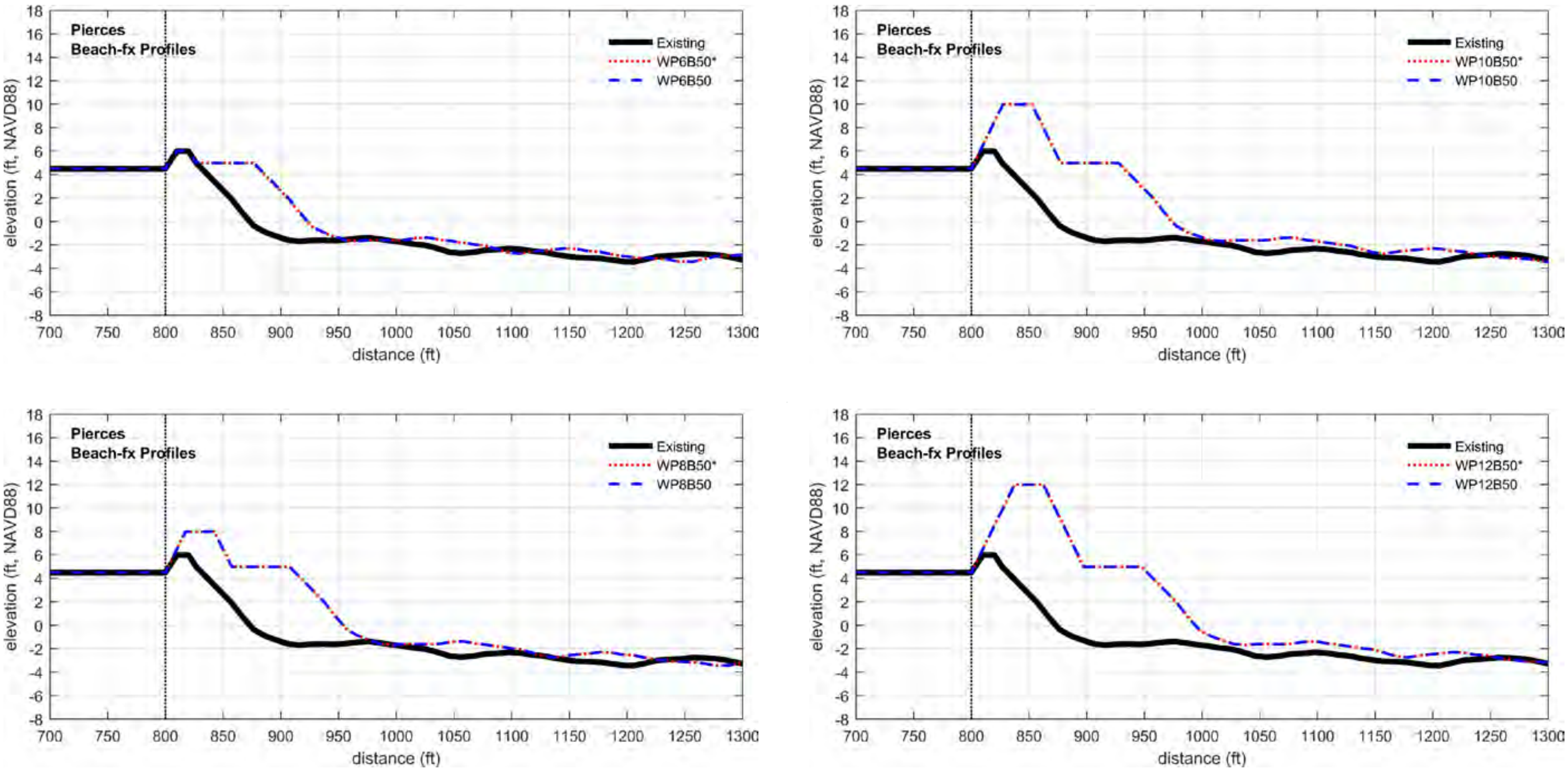
PIERCES BEACH ALIGNMENT

PIERCES – DESIGN VS BEACH-FX



*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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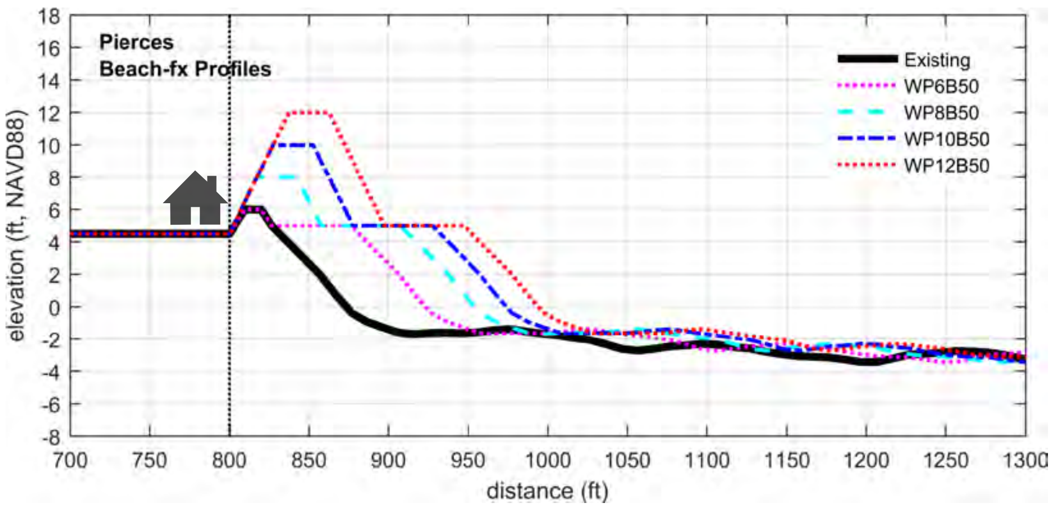
PIERCES – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+6	10	0	0	0
WP6B50	+6	10	50	50	-5.9
WP8B50	+8	25	50	81	-9.5
WP10B50	+10	25	50	101	-11.9
WP12B50	+12	25	50	121	-14.2



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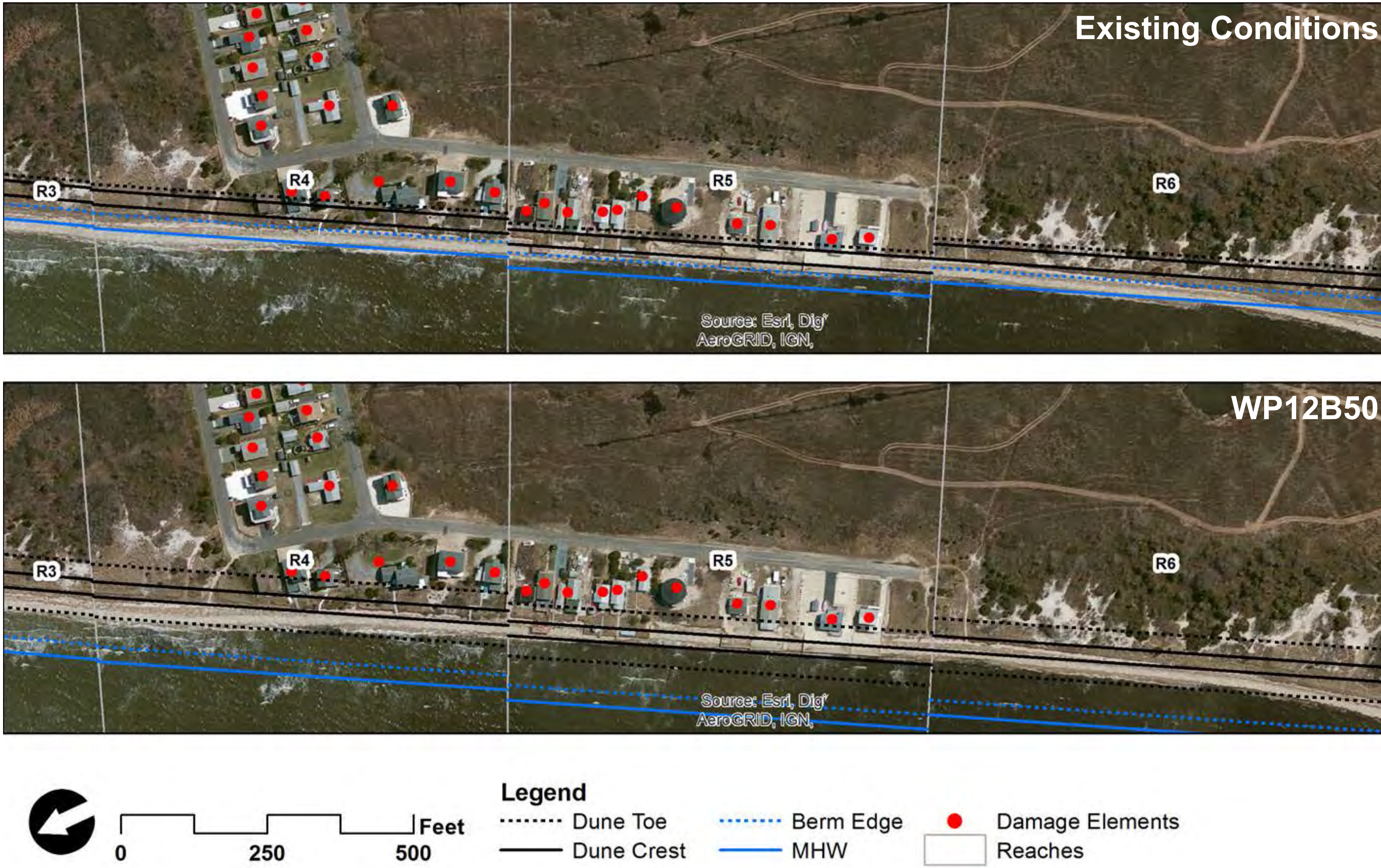
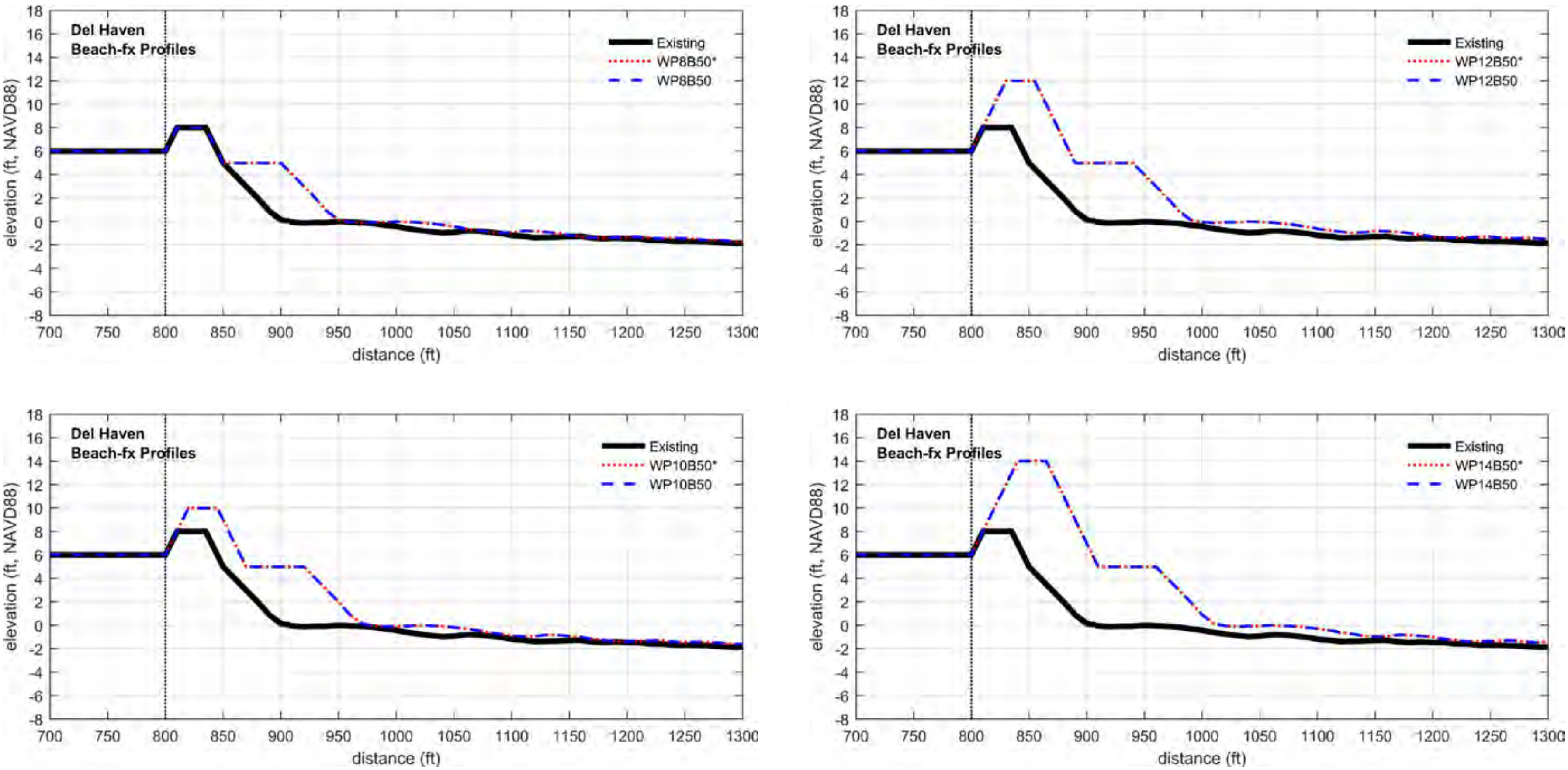
DEL HAVEN BEACH ALIGNMENT



DEL HAVEN – DESIGN VS BEACH-FX



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Blue profile represents Modeled profile

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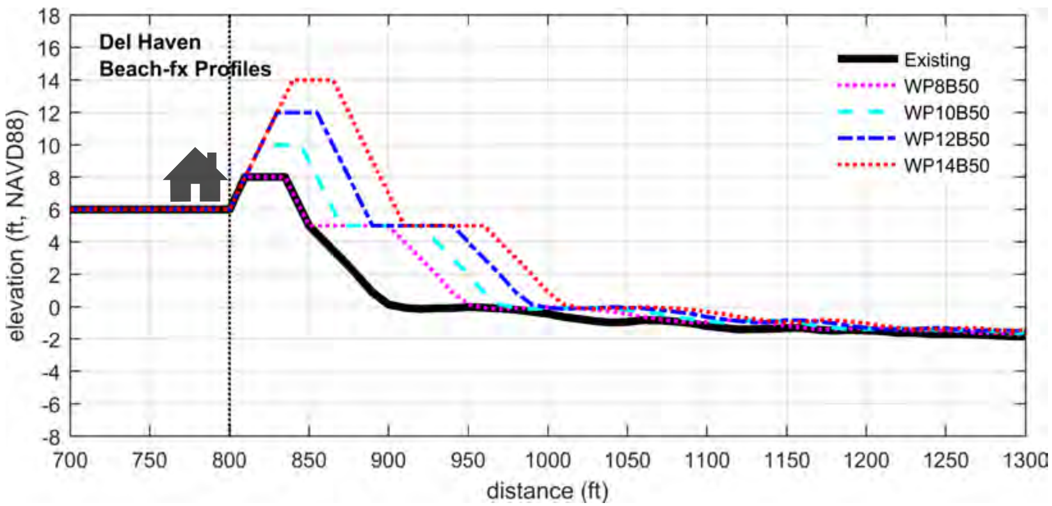
DEL HAVEN – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+8	25	0	0	0
WP8B50	+8	25	50	50	-1.7
WP10B50	+10	25	50	70	-2.4
WP12B50	+12	25	50	90	-3.0
WP14B50	+14	25	50	110	-3.7



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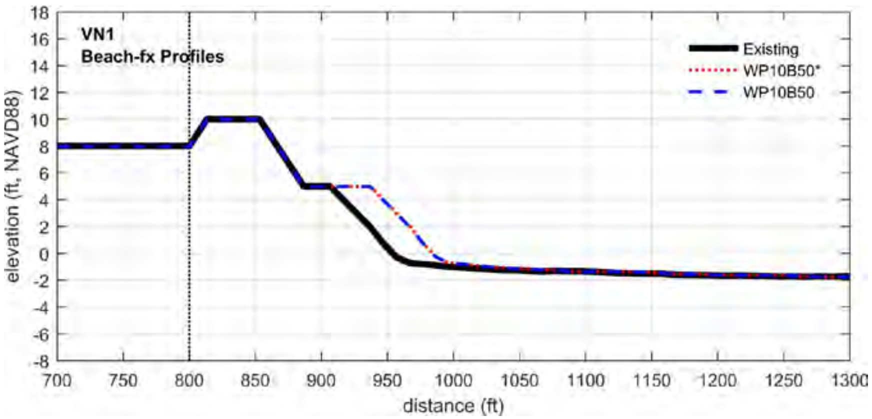
VILLAS NORTH BEACH ALIGNMENT

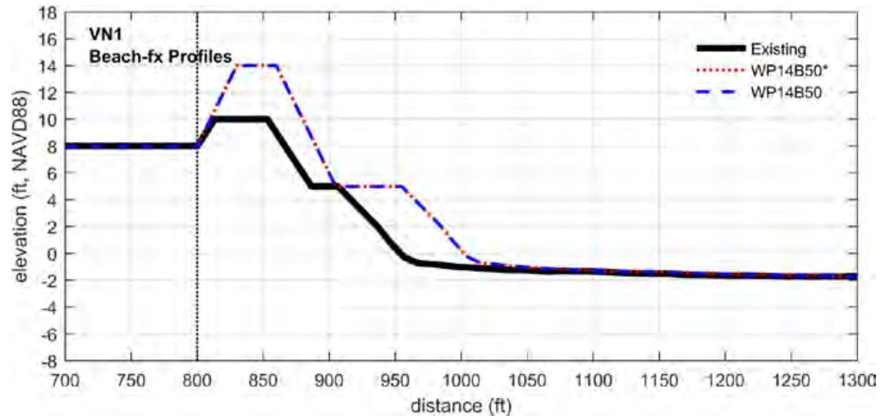
0 250 500 Feet

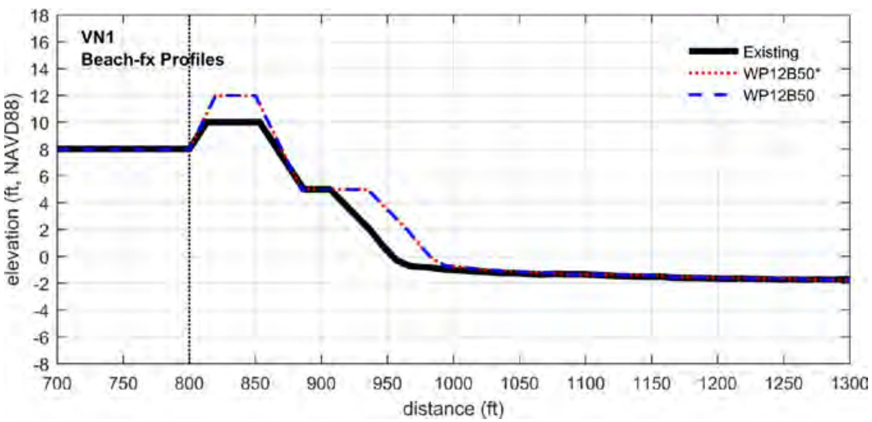
Legend

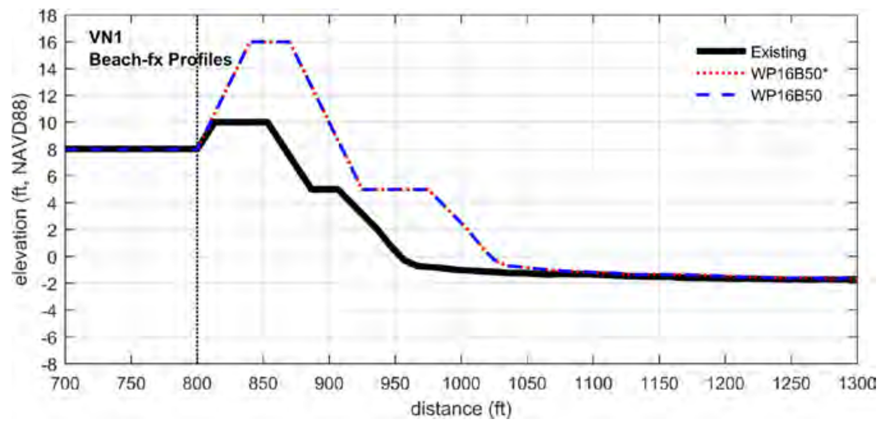
- Dune Toe
- Dune Crest
- Berm Edge
- MHW
- Damage Elements
- Reaches

VILLAS NORTH 1 – DESIGN VS BEACH-FX









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Blue profile represents Modeled profile

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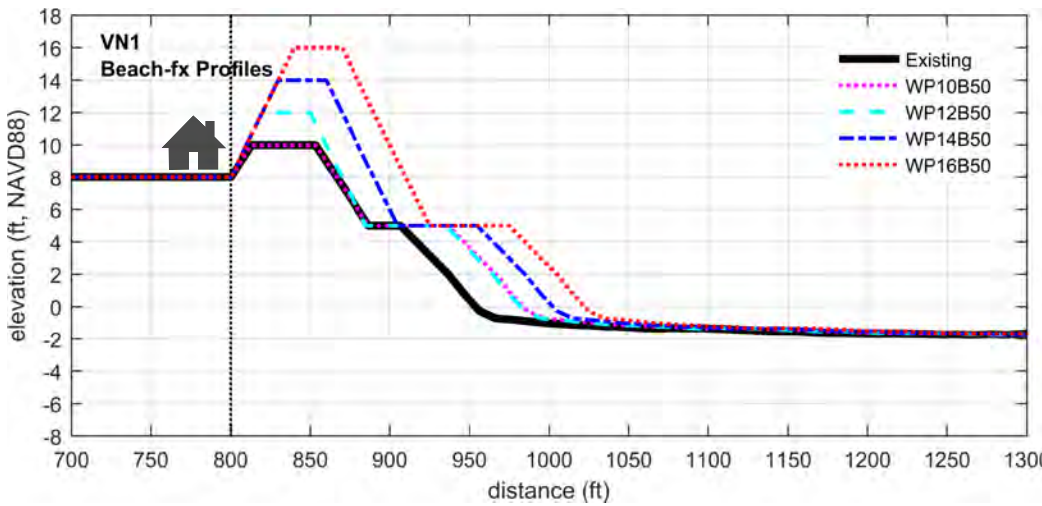
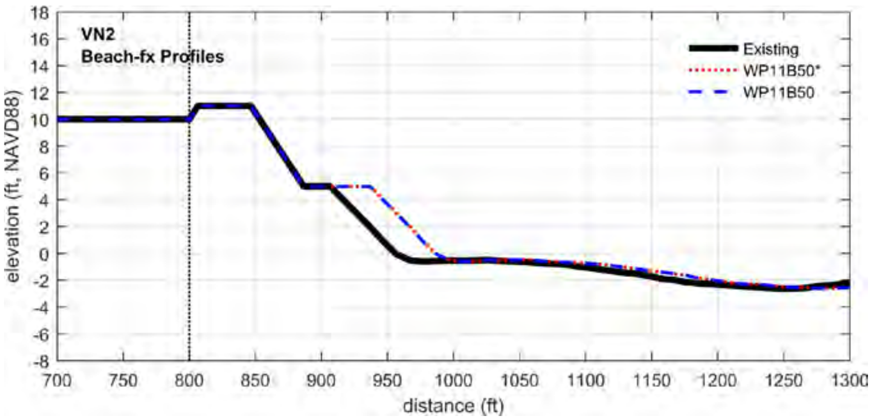
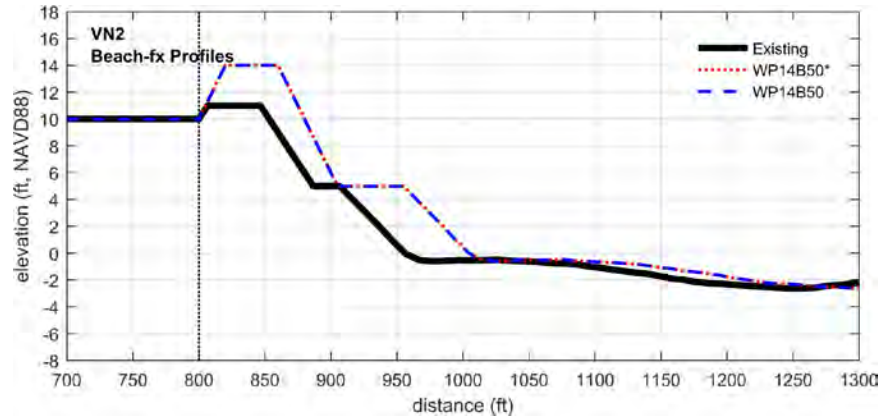
VILLAS NORTH 1 – OPTIMIZATION ALTERNATIVES

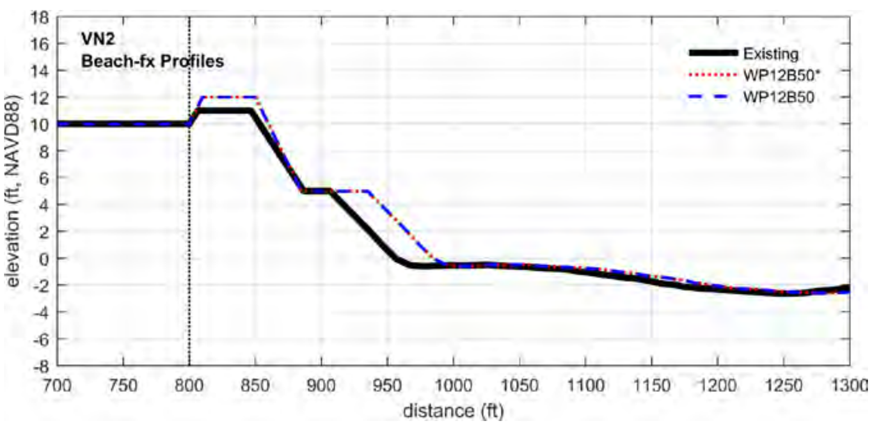
Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+10	40	20	0	0
WP10B50	+10	40	50	30	-0.7
WP12B50	+12	30	50	30	-0.7
WP14B50	+14	30	50	50	-1.2
WP16B50	+16	30	50	70	-1.6

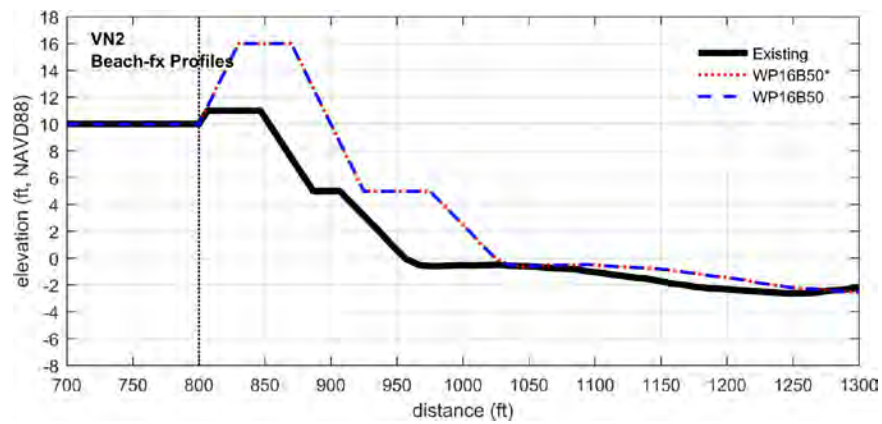


VILLAS NORTH 2 – DESIGN VS BEACH-FX









*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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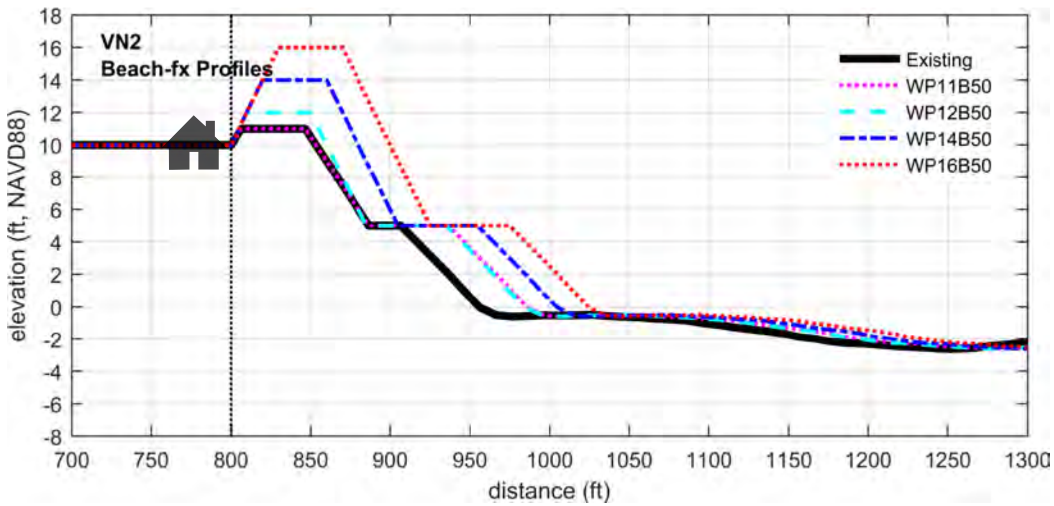
VILLAS NORTH 2 – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+11	40	0	0	0
WP11B50	+11	40	50	30	-0.7
WP12B50	+12	40	50	30	-0.7
WP14B50	+14	40	50	50	-1.2
WP16B50	+16	40	50	70	-1.6



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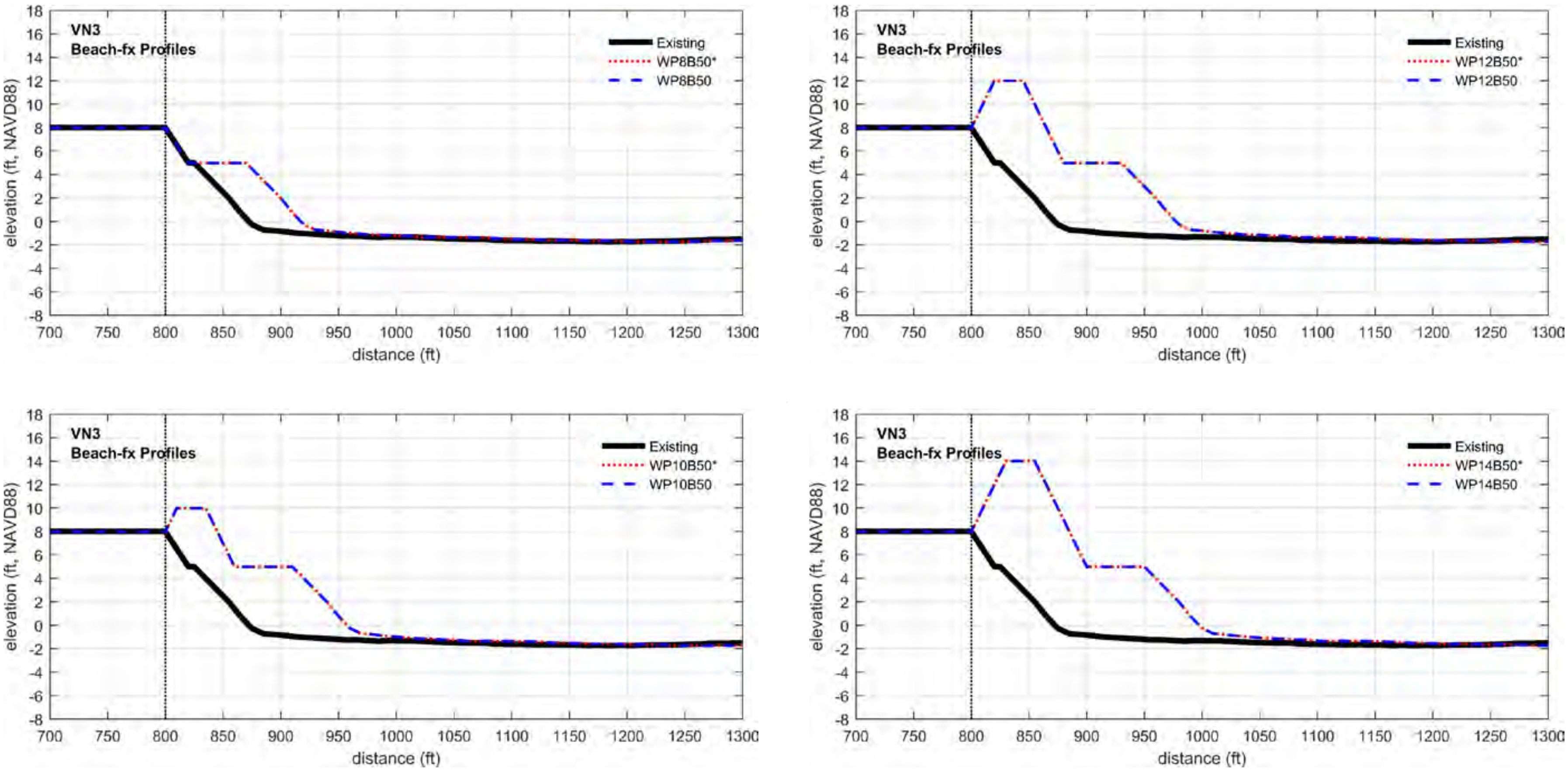
VILLAS NORTH 3 – DESIGN VS BEACH-FX



*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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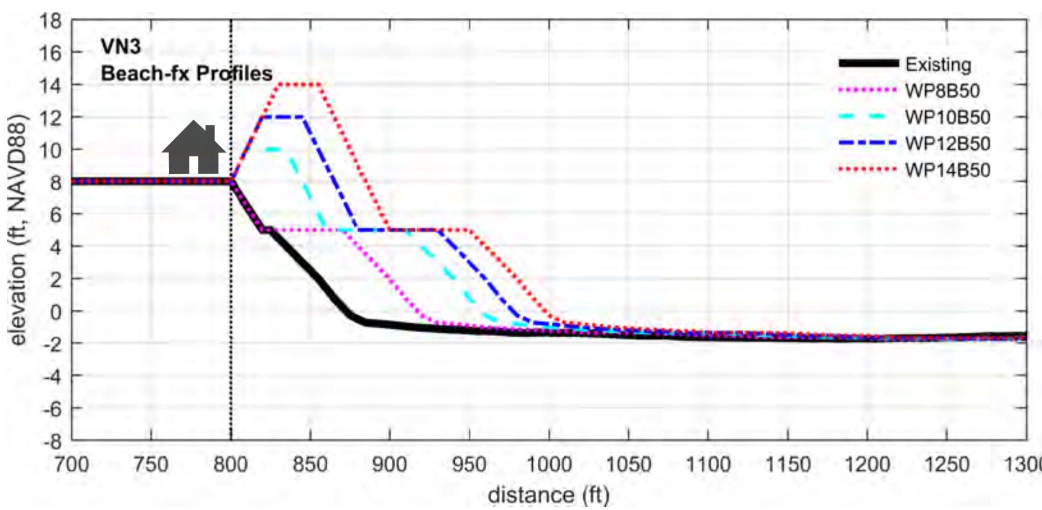
VILLAS NORTH 3 – OPTIMIZATION ALTERNATIVES

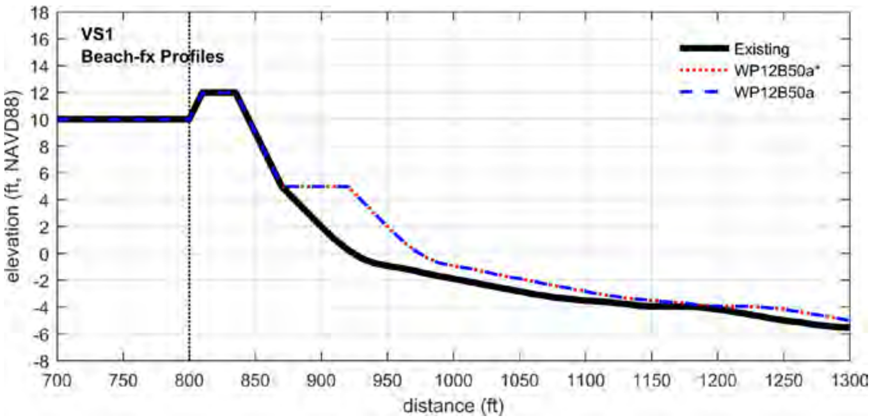
Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+8	0	5	0	0
WP8B50	+8	0	50	45	-1.0
WP10B50	+10	25	50	85	-2.0
WP12B50	+12	25	50	105	-2.4
WP14B50	+14	25	50	125	-2.9

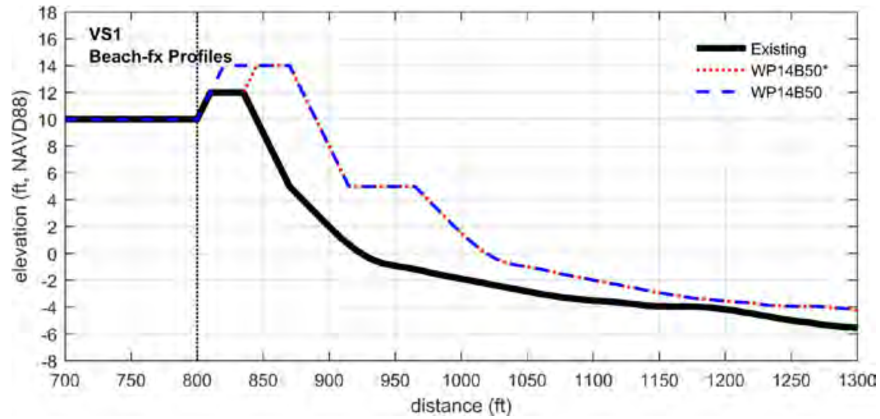


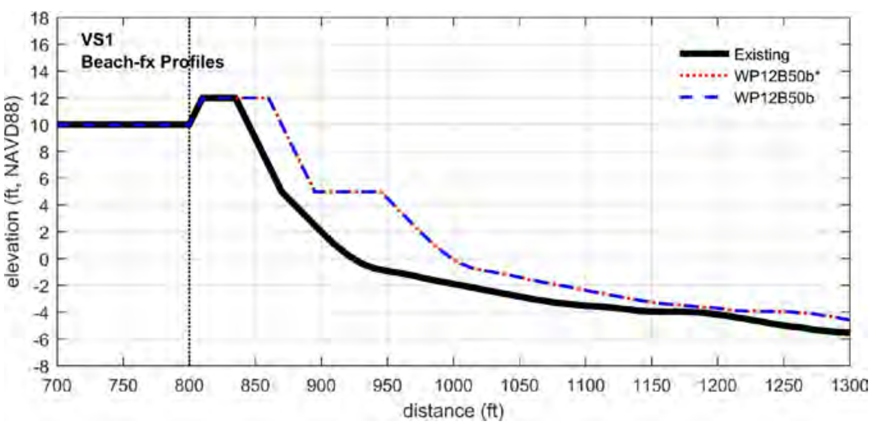
VILLAS SOUTH BEACH ALIGNMENT

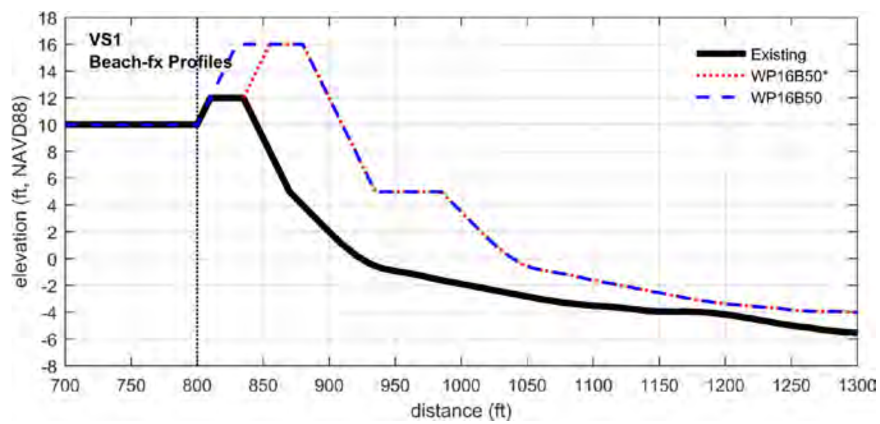
VILLAS SOUTH BEACH ALIGNMENT

VILLAS SOUTH 1 – DESIGN VS BEACH-FX









*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

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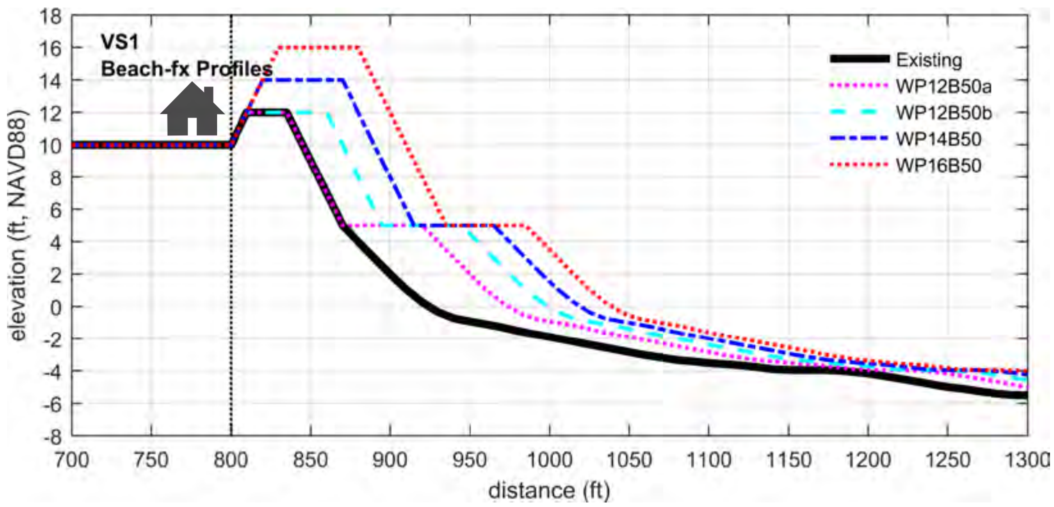
VILLAS SOUTH 1 – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+12	25	0	0	0
WP12B50a	+12	25	50	50	-1.2
WP12B50b	+12	50	50	75	-1.7
WP14B50	+14	50	50	95	-2.2
WP16B50	+16	50	50	115	-2.7



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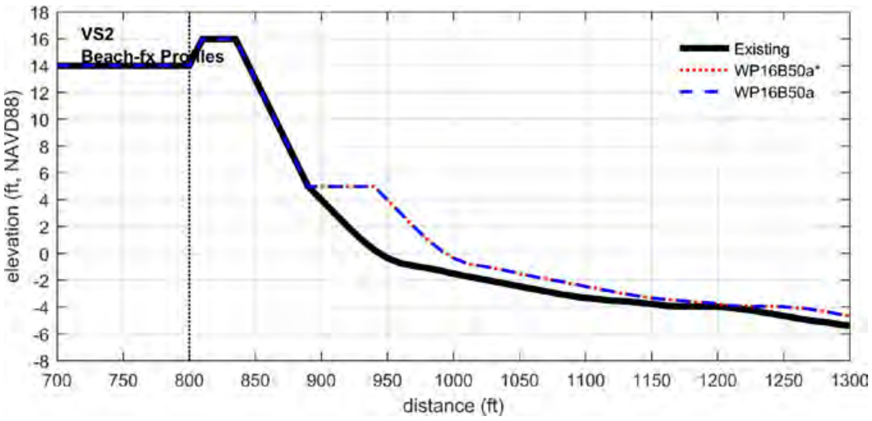
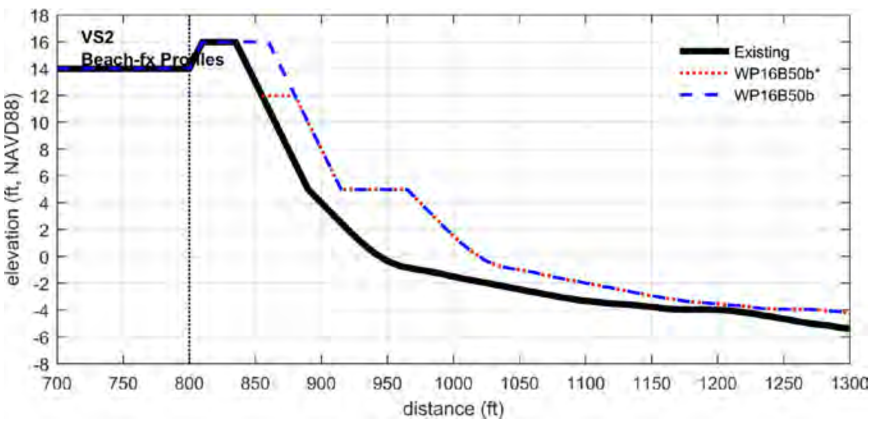
VILLAS SOUTH 2 – DESIGN VS BEACH-FX

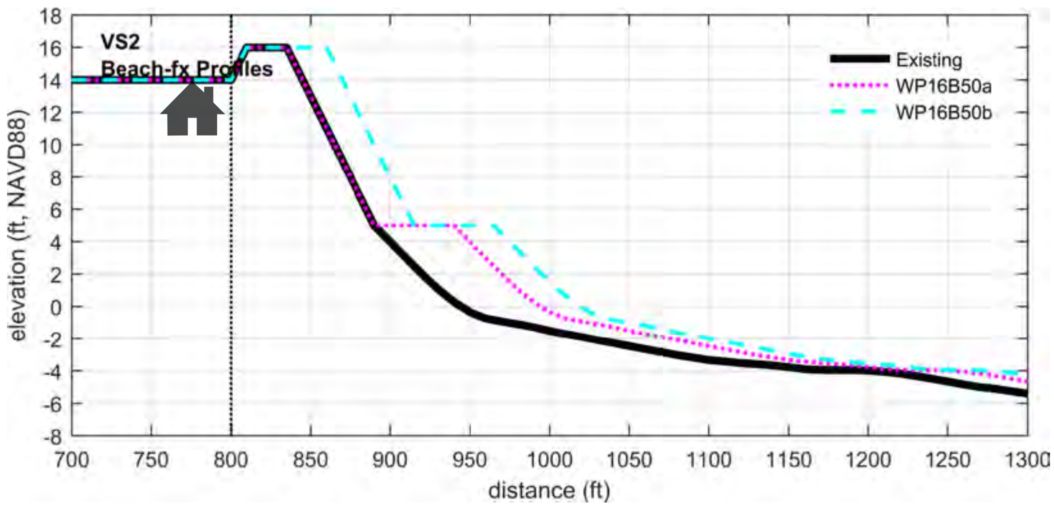




*Red profile closely represents Civil Design profile.
Blue profile represents Modeled profile

VILLAS SOUTH 2 – OPTIMIZATION ALTERNATIVES

Alternative	Dune Height (ft, NAVD88)	Dune Width (ft)	Berm Width (ft)	ΔY (ft)	Diffusion (ft/yr)
Existing	+16	25	0	0	0
WP16B50a	+16	25	50	50	-1.2
WP16B50b	+16	50	50	75	-1.7



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ATTACHMENT C.4 SBEACH MATRIX OF SIMULATIONS

October 2017

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Table C.4.1: SBEACH Matrix of Simulations

Site	Profile Name	Fixed SBEACH Parameters: Upland Elv. (ft, NAVD88)	Berm Elv. (ft, NAVD88)	Dune Slope	Foreshore Slope	Variable SBEACH Parameters: Dune Elevations (ft, NAVD88)	Dune Widths (ft)	Berm Widths (ft)	Simulations: Profile Permutations	Storms	Simulations	Subtotal
F1 - Existing	F1	6.5	6	0.2	0.1	6.5	0	0 10 20 30 40 50 60 70 80 90 100	9	300	2,700	
F1 - Existing	F1	6.5	6	0.2	0.1	7, 8, 9, 10	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	216	300	64,800	
F1 - Design	F1	6.5	6	0.2	0.1	11, 12, 14, 16, 18	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	270	300	81,000	148,500
RB1 - Existing	RB1	5.5	5	0.1	0.1	5.5	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
RB1 - Existing	RB1	5.5	5	0.1	0.1	6, 7, 8, 9	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	396	348	137,808	
RB1 - Design	RB1	5.5	5	0.2	0.1	10, 11, 12	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	198	348	68,904	
RB1 - Design	RB1	5.5	5	0.2	0.1	14, 16, 18	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	198	348	68,904	279,444
PP1 - Existing	PP1	4.5	5	0.15	0.1	4.5	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
PP1 - Existing	PP1	4.5	5	0.15	0.1	5, 6, 7, 8, 9, 10	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	396	348	137,808	
PP1 - Design	PP1	4.5	5	0.2	0.1	11, 12	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	132	348	45,936	
PP1 - Design	PP1	4.5	5	0.2	0.1	14, 16, 18	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	198	348	68,904	256,476
DH1 - Existing	DH1	6	5	0.2	0.1	6	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
DH1 - Existing	DH1	6	5	0.2	0.1	7, 8, 9, 10, 11	0, 5, 10, 15, 20, 25, 30	0 10 20 30 40 50 60 70 80 90 100	385	348	133,980	
DH1 - Deisgn	DH1	6	5	0.2	0.1	12, 14, 16, 18	0, 5, 10, 15, 20, 25	0 10 20 30 40 50 60 70 80 90 100	264	348	91,872	229,680
VN1 - Existing	VN1	8	5	0.15	0.1	8	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
VN1 - Existing	VN1	8	5	0.15	0.1	9, 10, 11	0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60	0 10 20 30 40 50 60 70 80 90 100	429	348	149,292	
VN1 - Design	VN1	8	5	0.2	0.1	12, 14, 16, 18	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	396	348	137,808	290,928
VN2 - Existing	VN2	10	5	0.15	0.1	10	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
VN2 - Existing	VN2	10	5	0.15	0.1	11, 12	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	198	348	68,904	
VN2 - Design	VN2	10	5	0.2	0.1	14, 16, 18	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	297	348	103,356	176,088
VS1 - Existing	VS1	10	5	0.2	0.1	10	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
VS1 - Existing	VS1	10	5	0.2	0.1	11	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	99	348	34,452	
VS1 - Existing	VS1	10	5	0.2	0.1	12, 14, 16, 18, 20	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	495	348	172,260	210,540
VS2 - Existing	VS2	14	5	0.2	0.1	14	0	0 10 20 30 40 50 60 70 80 90 100	11	348	3,828	
VS2 - Existing	VS2	14	5	0.2	0.1	16, 18, 20	0, 5, 10, 15, 20, 25, 30, 35, 40	0 10 20 30 40 50 60 70 80 90 100	297	348	103,356	107,184
												1,698,840