Rockefeller Refuge Gulf Shoreline Stabilization Demonstration Project (ME-18)

Post Construction Monitoring

Prepared for:

State of Louisiana

Coastal Protection and Restoration Authority

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EXECUTIVE SUMMARY

Three alternative shoreline protection designs consisting of a reef breakwater, a reef breakwater with lightweight aggregate core (LWAC), and a gravel/crushed stone beach fill were constructed as test sections in a demonstration project at the Rockefeller Wildlife Refuge in Cameron Parish, Louisiana. Construction of the demonstration project was completed in December 2009. To compare the effectiveness of each alternative at reducing erosion, each test section was monitored for approximately 14 months following construction. The analysis presented herein was performed to help determine which method of shoreline protection is best suited for construction along the full 9.2-mile project reach.

The monitoring program included collection of field data to document performance of the shoreline protection alternatives. Metocean data stations were installed to collect wave, wind, and tide data for approximately six months. Concurrent measurements of waves offshore and leeward of the breakwaters were included to assess wave transmission for each breakwater and better characterize the local wave climate. In addition, the project area was surveyed approximately every four months to document structure geometry, shoreline and nearshore profile changes, and structure settlement. A separate control area was surveyed to assess shoreline retreat at a location without shoreline protection. Aerial photography was obtained to visually document the conditions during the surveys.

Based on parameters such as wave attenuation, reduction of shoreline recession, cost, and constructability, the reef breakwater with LWAC is recommended for implementing the full 9.2 mile project. Although the other structural alternative, the reef breakwater, was marginally effective at reducing erosion and is less expensive than the reef breakwater with LWAC, further design optimization is unlikely to result in a variation that meets the original design criteria in terms of protection to the refuge shoreline. If the design criteria are relaxed, the reef breakwater should be given further consideration. The beach fill alternative was the least effective at protecting the refuge shoreline, although inference of its performance was limited by the relatively short length of the test installation. Because of its relatively high cost, constructability challenges, and poor performance, the beach fill is not recommended for further consideration.

The data assembled and collected as part of this monitoring program should be applied for final design of the full 9.2 mile project. In particular, required crest elevation and width of the breakwater should be refined based on the documented observations of foundation settlement and wave transmission. Preliminary projections for construction of the full 9.2 mile project with the reef breakwater with LWAC range from approximately $87,400,000 to $98,900,000 depending on the actual level of design optimization that can be realized. For a hypothetical construction budget of $50,000,000, it is estimated that a project length of approximately 4.7 miles could be constructed.

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ACRONYMS

ASCE	American Society of Civil Engineers
CIRIA	Construction Industry Research and Information Association
CIAP	Coastal Impact Assistance Program
CPRA	Coastal Protection and Restoration Authority
CWPPRA	Coastal Wetlands Planning Protection and Restoration Act
DNR	Louisiana Department of Natural Resources
H_{mo}	Spectral Significant Wave Height
H_S	Significant Wave Height
H_{10}	Average of highest 10 percent of all wave heights
HDR	HDR Engineering, Inc.
JCLS	John Chance Land Surveys, Inc.
LADOTD	Louisiana Department of Transportation and Development
LDWF	Louisiana Department of Wildlife and Fisheries
LWA	Lightweight Aggregate
LWAC	Lightweight Aggregate Core
MHHW	Mean Higher High Water
MLLW	Mean Lower Low Water
NAVD	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
OCPR	Office of Coastal Protection and Restoration
RSLR	Relative Sea Level Rise
SPM	Shore Protection Manual
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WIS	Wave Information Studies

1.0 PROJECT OVERVIEW

1.1 Introduction and Purpose

To combat erosion and mitigate the loss of wetlands along the Rockefeller Wildlife Refuge shoreline, the Louisiana Coastal Protection and Restoration Authority (CPRA\OCPR) teamed with the National Marine Fisheries Service and Louisiana Department of Wildlife and Fisheries to implement the Rockefeller Refuge Gulf Shoreline Stabilization Project. The project intent is to halt erosion along a 9.2 mile portion of the refuge located west of Joseph Harbor Bayou ("Joseph Harbor"). Due to the extremely soft soil and other challenging site conditions, a smaller demonstration project utilizing several alternative shoreline protection designs was constructed in Fall/Winter 2009. Monitoring and evaluation of the demonstration project is presented herein.

The overall project location is the western 9.2 miles of Gulf shoreline along the refuge from Joseph Harbor to Beach Prong (Figure 1); the demonstration portion of the project is located just west of Joseph Harbor. The shoreline along the project reach is predominantly composed of soft marine clays with a veneer of crushed shell perched along the crest of an escarpment that defines the shoreface. The beach is backed by extensive marsh. The area is directly exposed to waves and currents from the open Gulf of Mexico, and high tides and storm events contribute to erosion along the refuge shoreline that averages over 30 ft/yr (Byrnes *et al*. 1995). Once constructed, the full 9.2 mile project is expected to reduce the loss of marsh habitat by 920 acres over the 20 year design life (LaCoast.gov 2003). Shiner Moseley (2006) provides additional project background and design information. A detailed summary of project construction is presented in HDR (2010).

For the relatively unique alternatives considered for protection of the refuge shoreline, limited or lacking available design guidance posed a challenge in predicting wave dissipation and transmission and soil/structure interaction. In addition, coastal processes along mud coasts are not well understood, and the innovative designs proposed likely had some redundancy or limitations based on the assumptions made. For the current demonstration project, two reef breakwater alternatives and a gravel beach fill alternative were constructed. To address some of the unknowns and better understand the performance characteristics of the constructed alternatives, a monitoring plan was developed and implemented. During 14 months following construction, the three constructed alternatives were monitored through collection of field data. Subsequent performance analysis as well as evaluation of cost, constructability, and other parameters has been applied to develop recommendations for final design and implementation of the full 9.2-mile project. The overall objectives of this monitoring program were to:

- Compare performance of each alternative;
- Determine which alternative is most suitable for a full 9.2-mile project;
- Present data to aid in design optimization for the most suitable alternative.

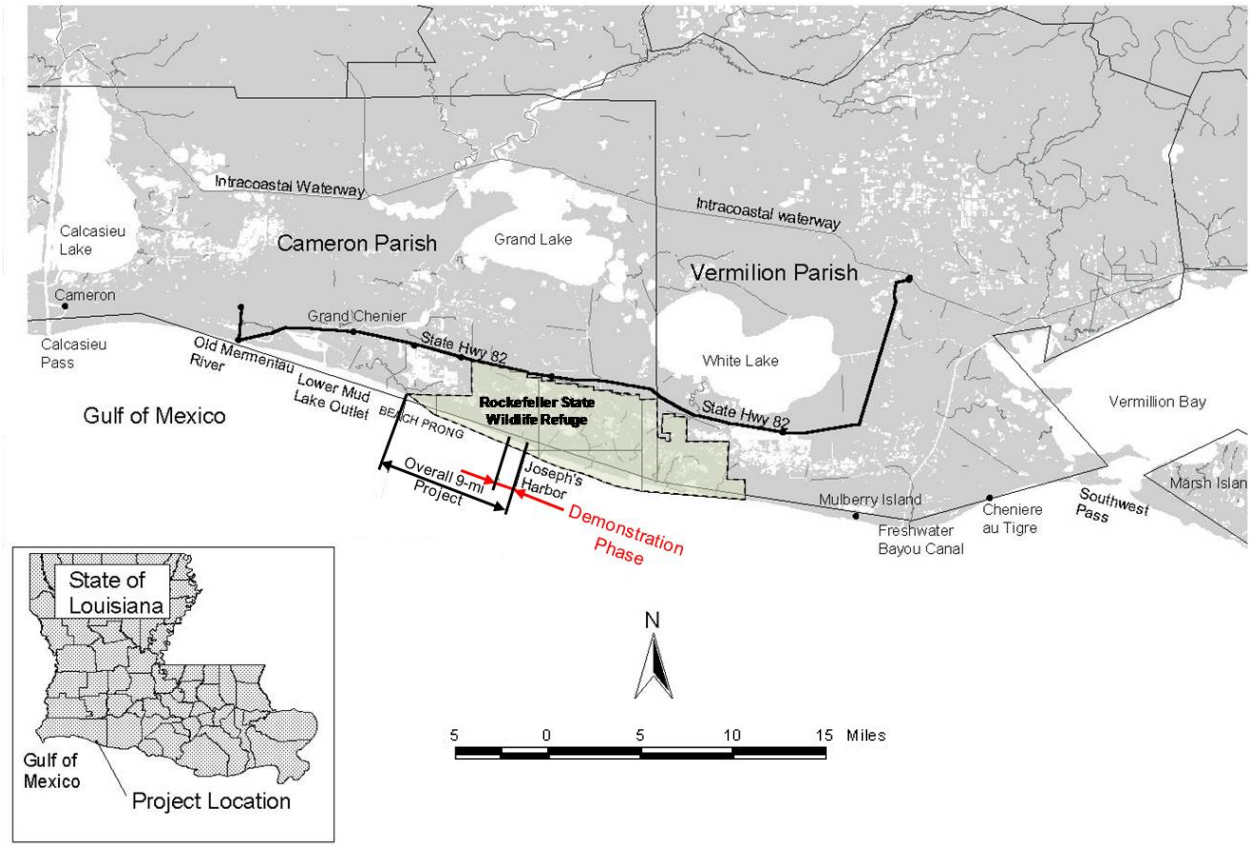

Figure 1. Project Location.

1.2 Authorization and Funding

HDR was given authorization for this work through CPRA\OCPR Contract Nos. 2503-10-22 and 2503-11-64 under the supervision of Mr. Darrell Pontiff, P.E. at CPRA\OCPR. The project was initially funded and authorized in accordance with the Coastal Wetlands Planning Protection and Restoration Act (16 U.S.C.A., Section 3951-3956) as project ME-18, Priority Project List 10. The current demonstration portion was funded with qualified outer continental shelf oil and gas revenues by the Coastal Impact Assistance Program, Bureau of Ocean Energy Management, Regulation and Enforcement, U.S. Department of the Interior. The views and conclusions contained in this document are those of the authors and should not be interpreted as representing the opinions or policies of the U.S. Government. Mention of trade names or commercial products does not constitute their endorsement by the U.S. Government.

1.3 Site Conditions

A detailed characterization of site conditions is provided in Shiner Moseley (2006). A summary of key site characteristics that are of particular relevance to the monitoring program is provided below.

Geotechnical Conditions

Subsurface soil conditions are relatively uniform alongshore and across-shore within the project area. Upper stratigraphy of the soils between approximate elevations[1] +5 feet and 0 loose to medium-dense shell with shell fragments. Below this stratum to an approximate depth of 40 feet is a very-soft to soft under-consolidated, erodible clay that was reported during the geotechnical field investigation to have a consistency "similar to drilling mud." Measurements of undrained shear strength within the upper 20 ft averaged about 85 psf. Below the stratum of very-soft to soft clay to a depth of 100 feet is a stiff to very stiff clay. Figure 2 provides a generalized subsurface profile along the project shoreline from Joseph Harbor (B-20) to the western boundary of the refuge (B-1).

Grab sampling performed during the geotechnical investigation did not reveal any significant surface deposits of sand or shell across the submerged portion of the beach profile. Surface sediments are relatively uniform across-shore and alongshore, although there is an abrupt change across-shore at the waterline where the surface sediments transition from shell particles to silt and clay.

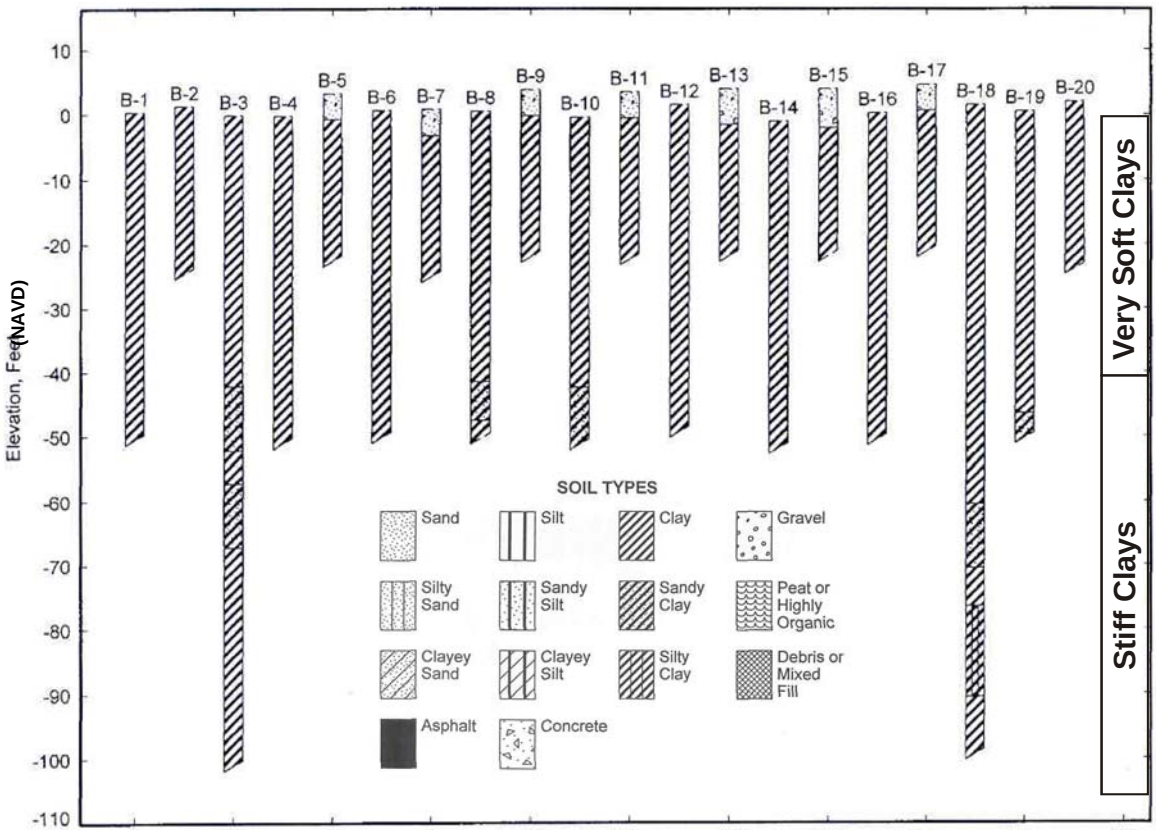

Figure 2. Generalized subsurface profile (modified from Fugro 2002).

[1] All elevations are referenced to the North American Vertical Datum of 1988 (NAVD), unless noted otherwise.

Nearshore Profile

An idealized profile illustrating the relatively shallow, gently-sloping seafloor along the project shoreline is shown in Figure 3. The back beach and upper beach consist of a veneer of shell hash overlying marsh sediment; this shell hash is transported landward by waves during higher tides and storms. The lower berm is predominantly exposed clays with some remnant plant stems and roots; this berm, where it exists, is typically much flatter than the upper and lower beach and forms a terrace. A photograph of a typical lower berm is shown in Figure 4. Note the presence of remnant marsh grass roots in the foreground, indicating how quickly the shoreface has been cut. As shown in Figure 5, when the lower berm is not present or it submerged, waves directly attack the marsh fringe.

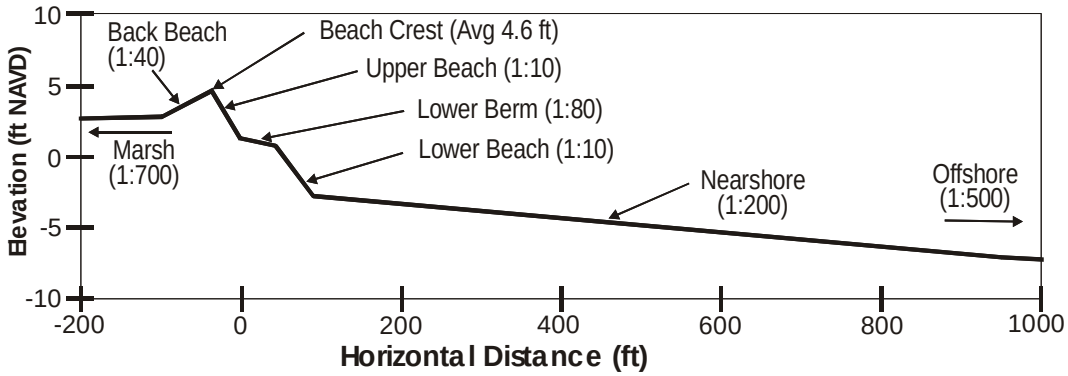

Figure 3. Idealized nearshore profile.

Figure 4. Typical lower berm.

Figure 5. Waves directly attacking marsh fringe.

Geomorphology

Byrnes *et al.* (1995) documented that modern rates of shoreline recession within Louisiana's Chenier Plain are generally increasing with time, and that the long-term average rate along the Rockefeller Refuge is about 35 ft per year. In Figure 6, shoreline change is plotted from March 1998 to July 2002. During this relatively recent and short-term period, shoreline recession averaged about 50 ft/yr, resulting in a net loss of approximately 200 ft. Note from the shoreline changes plotted in Figure 6 that the rate of shoreline recession is somewhat variable alongshore, particularly near Joseph Harbor, where as much as 400 ft of retreat occurred. The higher rates measured near Joseph Harbor may be a short-term anomaly considering that the photographs represented only a 2.5-year period, or it could be related to readjustment of the shoreline associated with the loss of the small island that recently existed at the mouth of the channel (Figure 7). Longer-term records do not appear to indicate a higher erosion rate near Joseph Harbor.

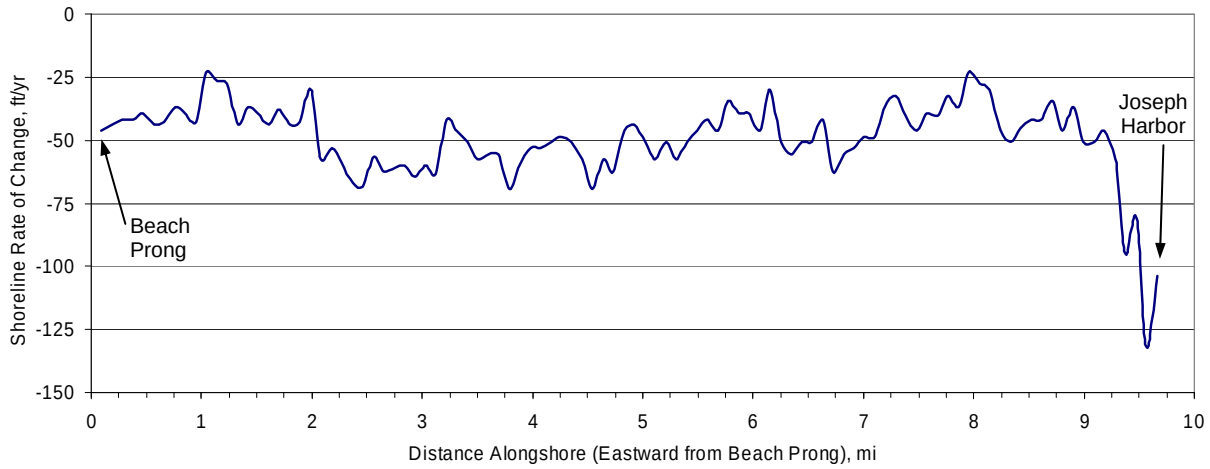

Figure 6. Rates of Shoreline Retreat from 3/2/98 to 7/18/02.

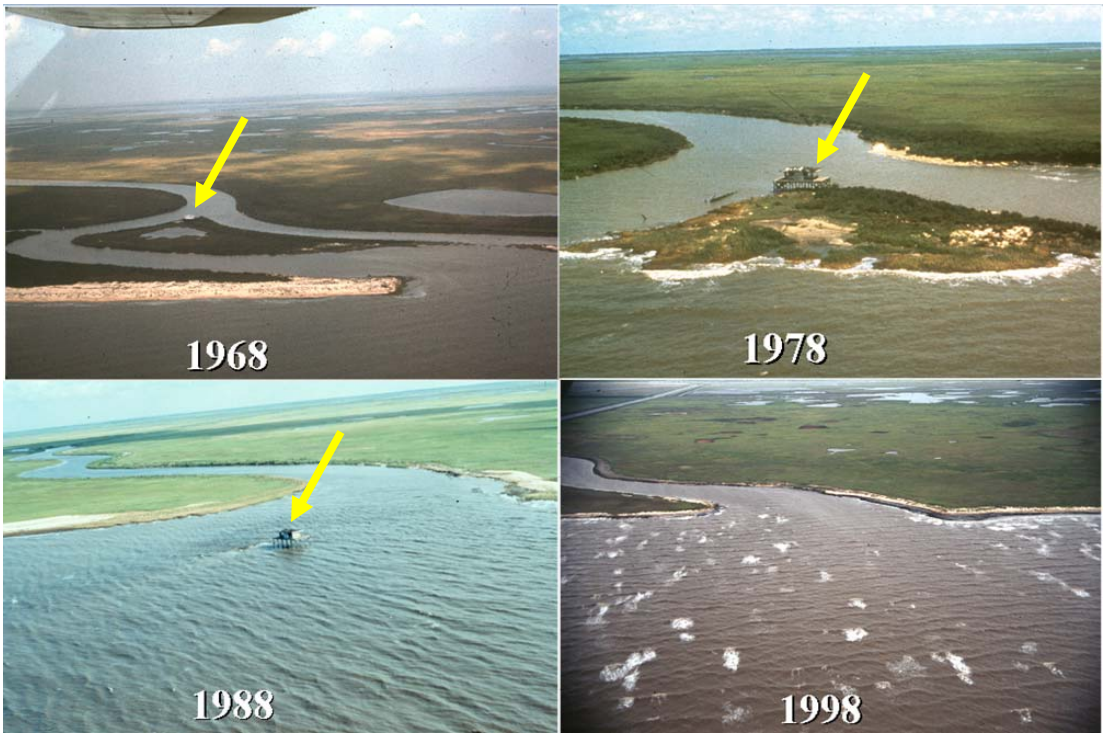

Figure 7. Rapid erosion at Joseph Harbor between 1968 and 1998 (photographs courtesy of Mr. Guthrie Perry, LDWF).

Tropical cyclones play a significant role in erosion along the Rockefeller Refuge shoreline. During storms, deposits of shell that are perched atop the beach along the refuge shoreline are typically transported landward by waves as washover deposits onto the marsh as shown in Figure 4. This process results in an exposed zone of fragmented marsh seaward of the beach to be reworked during and/or after the storm.

Depending on storm duration, stronger storms that generate a large surge may not necessarily produce the most severe erosion since the beachface and marsh become submerged and are somewhat protected by a cushion of water as waves pass overhead. However, storms with a large surge can still produce severe erosion. For example, in September 2005, Hurricane Rita produced a surge of approximately 15 ft at Calcasieu Pass and, based on comparison of pre- and post-storm aerial photographs, caused over 150 ft of erosion along portions of the ME-18 project reach.

Waves

During project design, wave modeling was performed to estimate wave height as a function of position across shore for various storm return periods. Results indicated that, during storms, nearshore wave heights have a much stronger dependency on surge elevation than offshore wave height or windspeed. Storm waves are primarily controlled by the broad, shallow shelf offshore of the refuge that attenuates waves as they travel landward. Therefore, because surge elevation directly controls nearshore wave heights, surge elevation, as opposed to windspeed or offshore

wave height, was the key factor in defining the project design waves. Although attenuation by fluid mud was also recognized as a potentially important factor in controlling nearshore wave heights, methodology for confident estimates of the dampening influence of mud was not available.

Tides

The tide along the Louisiana coast is mixed, meaning that both semi-diurnal and diurnal signals are present, but the diurnal component is typically dominant. Although no long-term tide statistics are available for the immediate project area, long-term records and published tidal statistics are available through the National Oceanic and Atmospheric Administration (NOAA), U.S Geological Survey (USGS), and U.S. Army Corps of Engineers (USACE) for several sites along the southwest coast of Louisiana. The diurnal tide ranges at these stations vary from approximately 1.1 ft to 2.2 ft. This variation is likely associated with gage location with respect to land, channels, and other boundaries affecting daily changes in water level. Note that, because the tide range is defined as the average difference between the daily high and low waters, on the Louisiana coast the tide range does not reflect the much larger range in water elevation that occurs seasonally due to meteorological and other factors.

Subsidence and Sea-Level Rise

The combination of vertical land motion (such as subsidence) and eustatic (i.e., global) sea-level rise is termed “relative sea-level rise.” Relative sea-level rise at Rockefeller Refuge was projected to be five to ten inches over the desired 20-year design life of the project. Future increases in relative sea level rise could gradually decrease the ability of the shoreline protection structure(s) to attenuate waves.

2.0 PROTECTION ALTERNATIVES

The demonstration project was constructed along the Gulf shoreline of the Rockefeller Refuge just west of Joseph Harbor (Figure 8). Two breakwater sections, each having an approximate length of 500 ft, and a beach fill section approximately 650 ft in length were constructed for the demonstration project. A fourth alternative, a concrete panel breakwater, was included as an "additive alternate" item in the construction bidding documents but not constructed due to cost. The breakwater sections consisted of a reef breakwater and a reef breakwater with lightweight aggregate core (LWAC). HDR (2010) provides detailed information on the constructed project features including final dimensions, material quantities, and settlement measurements taken by the contractor during construction. Figure 9 shows the original design layout and Figure 10 shows the constructed location for each alternative.

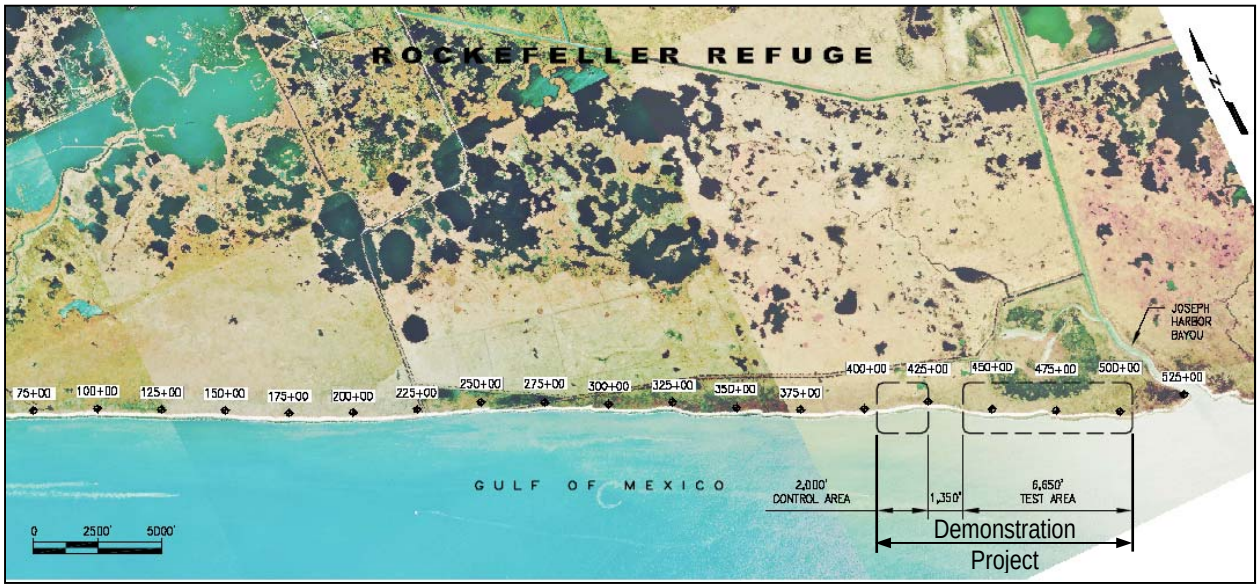

Figure 8. Location of Demonstration Project.

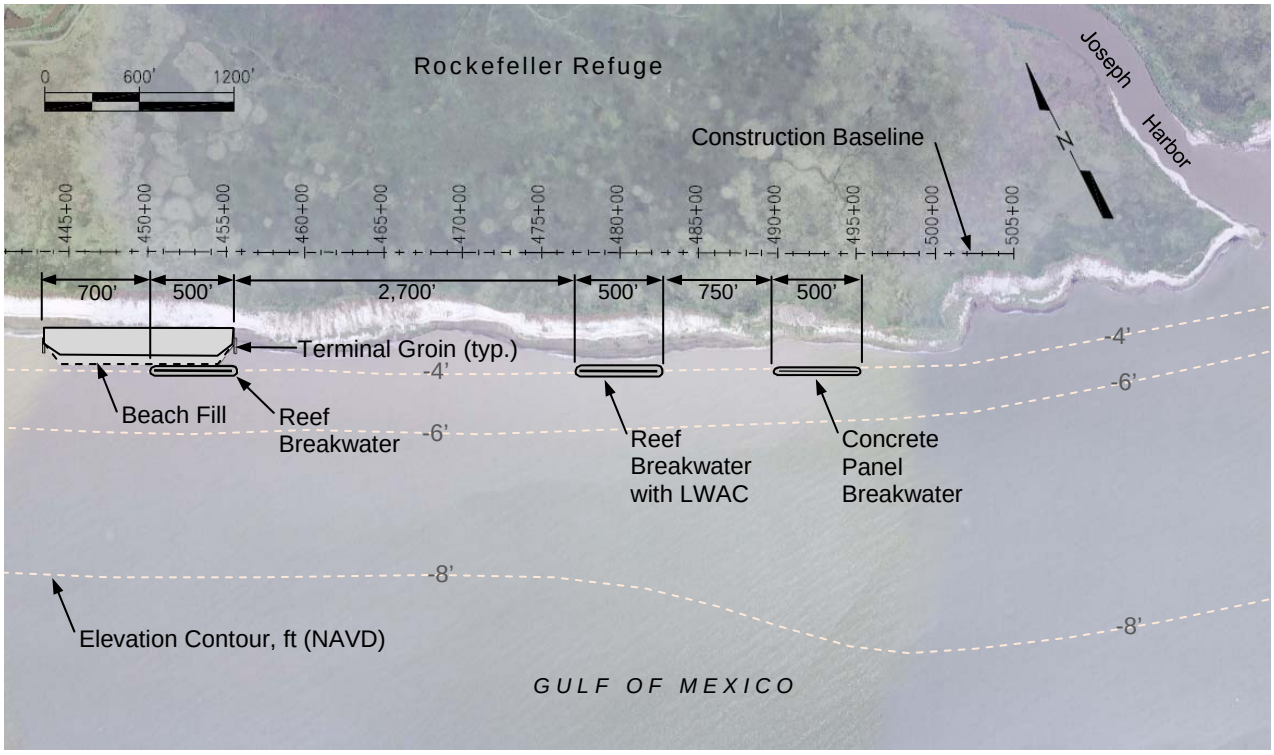

Figure 9. Original design layout for demonstration project.

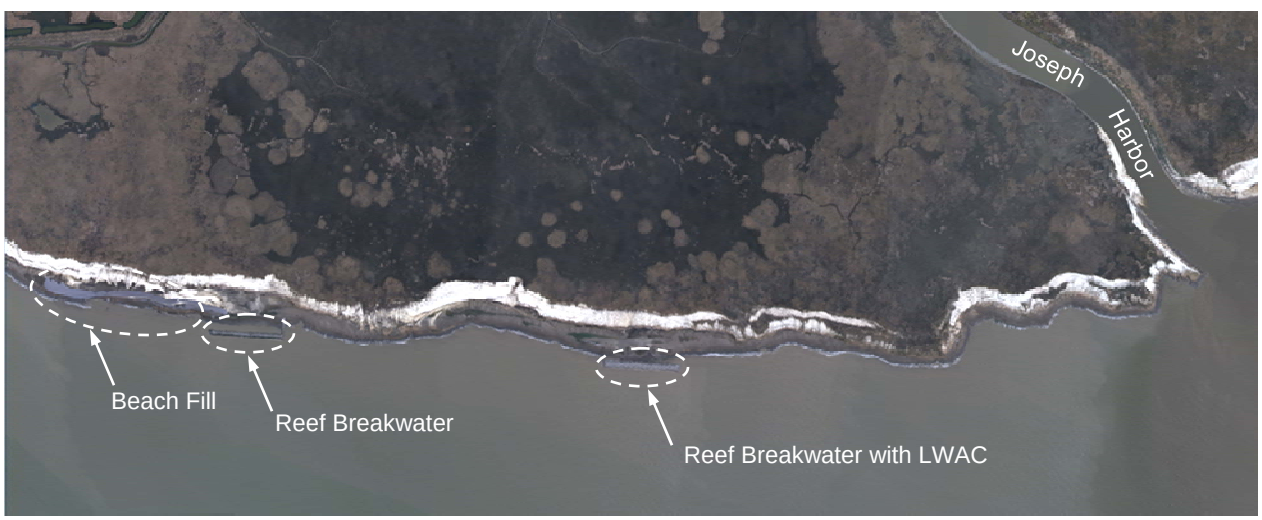

Figure 10. Locations of constructed test sections (photo date: 2/16/10).

2.1 Project Design Criteria

The primary challenge for ME-18 is design and construction of a cost-feasible structure that will be both hydraulically stable under open-coast wave conditions and geotechnically stable despite very soft foundation soils. As summarized in Shiner Moseley (2006), potential project alternatives were evaluated based on their ability to meet the following criteria:

- Prevent beach erosion for up to Category 1 hurricane conditions, which were estimated to have a return interval of about 10 years at the project site.

- Be designed, constructed, monitored, and maintained over a 20-year design life for $42,000,000 with a construction cost of about $38,000,000 or $785/ft.
- Where practicable, the protection should remain stable for more severe storm conditions up to an event having a 100-year return period.
- Modified design criteria were considered for a hypothetical increase of the construction budget by 50% (i.e., from $38,000,000 to $57,000,000) and relaxation of the "no erosion under a Category 1 hurricane" requirement.

For each project alternative, estimates of soil consolidation and settlement by Fugro (2004) were applied to evaluate performance under the lower crest elevations that are expected to exist at completion of a 20-year project life.

2.2 Reef Breakwater

Reef breakwaters are rubble mounds of a rock size similar to that found in the armor and/or first underlayer of conventional breakwaters. Reef breakwaters are typically constructed without underlayers or a core of smaller stone and are broad crested in comparison to conventional breakwaters. By traditional definition, reef breakwaters are designed to adjust in cross-section in response to the waves and currents at the site, a characteristic not considered here. Low, broad crested breakwaters are designed to decrease the wave energy impacting the shoreline, but still allow some wave transmission under day-to-day conditions.

Although reef breakwaters are lower than conventional breakwaters, their much broader crest helps promote breaking and attenuation of waves that may otherwise pass over and through the structure with minimal energy reduction. For ME-18, the key benefit of a lower crested reef breakwater is reduced bearing pressure on the soil. The original design included construction of crushed stone beach fill landward of the breakwater to absorb remaining smaller waves that are transmitted across the breakwater. However, during construction it was decided to exclude the beach fill behind the breakwater for two reasons: (1) measured settlement of the breakwater during construction was substantially less than anticipated, allowing the breakwater crest to be higher than planned and (2) the extremely soft soils prevented efficient operation of land-based construction equipment that was required for placement of the beach fill landward of the breakwater. Figure 11 shows a track hoe on timber mats that sank into the soft soil during construction.

Figure 11. Track hoe sunk in soft soil during construction.

A typical section of the reef breakwater is shown in Figure 12. The design (pre-settlement) crest elevation and width are shown along with the constructed nominal dimensions. Based on the design crest elevation of +1.0 ft, Fugro estimated total settlement to be approximately 1.2 ft. The specified riprap gradation required a minimum W_{50} of 1,000 lb and W_{max} of 5,000 lb. A photograph of the constructed breakwater is shown in Figure 13.

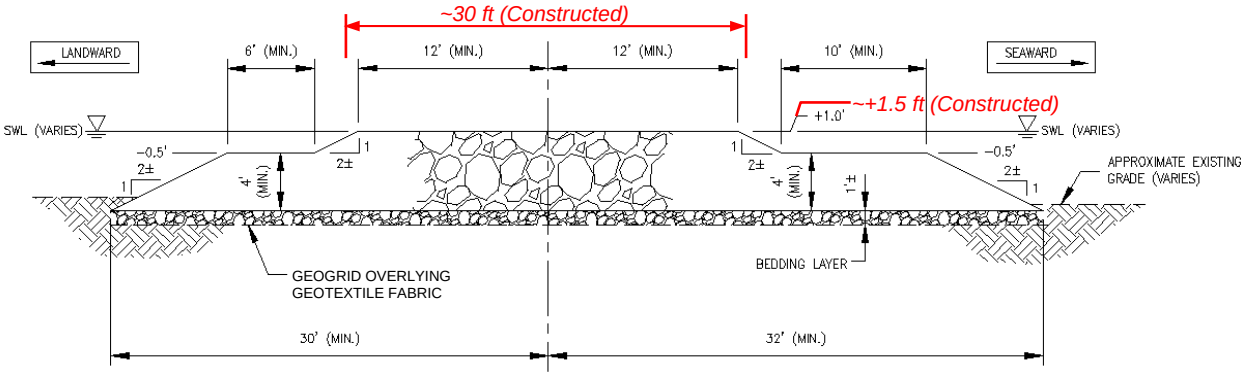

Figure 12. Design Section for Reef Breakwater. Constructed nominal dimensions are shown in red italics.

Figure 13. Constructed Reef Breakwater (7/23/09).

2.3 Reef Breakwater with Lightweight Aggregate Core

For the reef breakwater with LWAC, the stone within the core of the breakwater was replaced with an encapsulated lightweight aggregate that is almost neutrally buoyant, decreasing the bearing pressure and allowing greater crest elevation and increased wave attenuation. A typical section of the reef breakwater with LWAC is shown in Figure 14. The design (pre-settlement) crest elevation and width are shown along with the constructed nominal dimensions. Based on the design crest elevation of +3.25 ft, Fugro estimated total settlement to be approximately 1.3 ft. The specified riprap gradation was the same as for the reef breakwater.

The lightweight aggregate was contained within approximate 7 ft by 10 ft geotextile bags that are shown in Figures 15 and 16. An important element of the monitoring program is to assess the potential for damage to the bags and/or decreased hydraulic stability caused by the LWAC and greater crest elevation. Figure 17 is an oblique aerial photograph of the completed breakwater. In the photograph, note the two LWA containers landward of the breakwater that were dislodged by waves during construction prior to being capped with riprap.

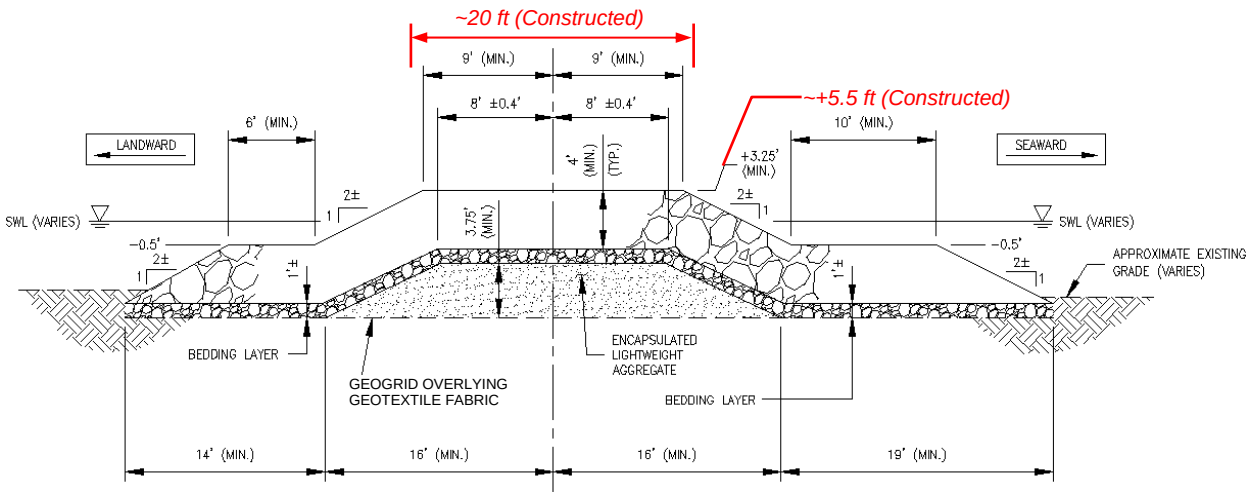

Figure 14. Design Section for Reef Breakwater with LWAC. Constructed nominal dimensions are shown in red italics.

Figure 15. Geotextile containers being filled with Lightweight Aggregate.

Figure 16. Geotextile containers being placed for LWAC.

Figure 17. Constructed Reef Breakwater with LWAC.

2.4 Beach Fill

In addition to the two reef breakwater approaches, the demonstration project included beach fill constructed with crushed stone. Although both stand-alone beach fill and beach fill landward of the reef breakwater were originally planned, the beach fill landward of the breakwater was not constructed for reasons discussed above in Section 2.2.

Although gravel (and cobble) beach fills have been successful at several locations around the world, no previous projects were identified where the material was placed on soft clay. Figure 18 shows a typical section of the beach fill constructed for the demonstration project. Including settlement, the placement density was approximately 20 yd^3/ft.

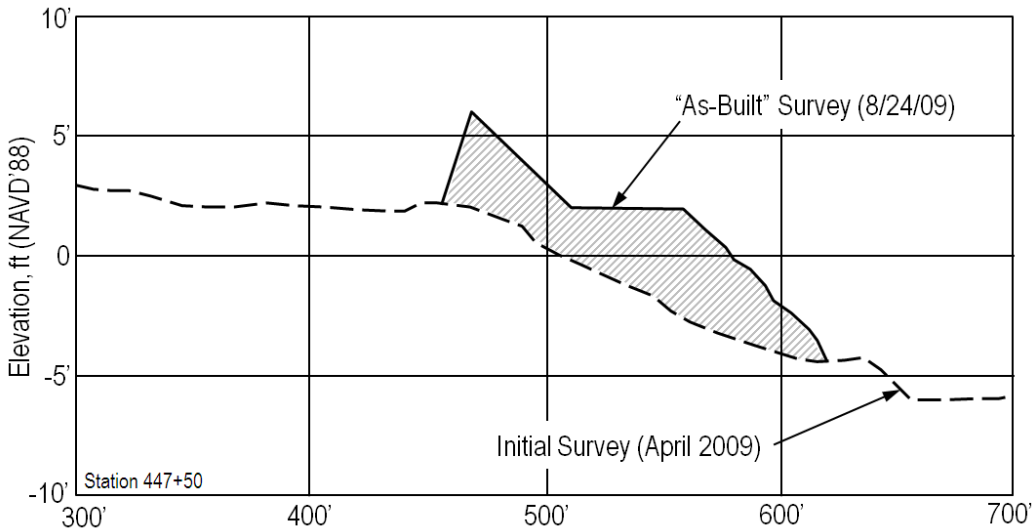

Figure 18. Typical "as-built" section for crushed-stone Beach Fill.

During design, wave-induced longshore transport and spreading, particularly "end losses" at each lateral end of the fill, were identified as forcing mechanisms that would influence the test installation. Although methods of predicting rates of longshore transport for coarse-grained materials are relatively limited, the known dependence of beach fill longevity on initial length was generally expected to present a considerable challenge in representing a full 9.2-mile beach fill with a much shorter test installation. Because performance of the test section was expected to be influenced significantly more by end losses than a longer project, terminal groins were planned at each end of the fill to reduce lateral spreading. The focus of the monitoring program was thus intended to be evaluation of cross-shore, not longshore, performance.

Construction-sequencing requirements resulted in installation of the terminal groins prior to the beach fill. As already discussed, construction of the portion of the beach fill behind the reef breakwater was deducted from the work; this deduction occurred after both groins had already been constructed. Figures 19 through 21 are photographs of the constructed beach fill and terminal groins.

Figure 19. Terminal Groins.

Figure 20. Beach Fill and Terminal Groin.

Figure 21. Beach Fill construction.

3.0 MONITORING RESULTS

3.1 Monitoring Overview

Although formal monitoring of the demonstration project did not commence until completion of construction, survey data and aerial photography collected prior to and during construction were included in this performance assessment. Table 1 provides a summary of the collected data and Figure 22 shows data collection locations. Enlarged views of the data collection locations, including settlement plate locations, at the reef breakwater with LWAC and reef breakwater and beach fill are provided in Figures 23 and 24, respectively. Table 2 provides an overview of the data collection platforms (DCPs) that were installed as part of the monitoring program. Additional information regarding surveys performed during the monitoring period can be found in John Chance Land Surveys, Inc. survey report (JCLS 2011). JCLS (2011) also includes ground photography taken at each survey transect during each survey event. Appendix A provides a more detailed description and photographs of the DCPs.

Table 1. Summary of collected data.

Description	Frequency or Timeframe	Entity that Collected or Provided Data
Topographic and Bathymetric Surveys (Shore-Perpendicular Profiles and Settlement Plates)	1. Start of Construction (April 2009) 2. During Construction 3. End of Construction (February/April 2010)	J. Bordelon and Assoc, LLC
	4. July/August 2010 5. November 2010 6. March 2011	John Chance Land Surveys, Inc.
Oblique Aerial Photographs	Periodic during and after construction	LDWF and Gulf Coast Aerial Mapping
Ground Photographs	Periodic during and after construction	HDR and John Chance Land Surveys, Inc.
Aerial Orthophotographs	1. February 2010	The Atlantic Group, LLC
	2. July 2010 3. November 2010 4. April 2011	Gulf Coast Aerial Mapping
Wave and Tide Data	May 2010 to November 2010	ENCOS, Inc.

Table 2. Overview of data collection platforms.		
Station	**Location**	**Data Collected**
East DCP	Landward of Reef Breakwater with LWAC	Wave Height/Period and Water Level
West DCP	Landward of Reef Breakwater	Wave Height/Period, Water Level, Wind, Precipitation, Barometric Pressure, Relative Humidity, and Air Temperature
Offshore DCP	At Station 468+00 approximately 3,200 ft offshore in 14-ft deep water.	Wave Height/Period/Direction and Water Level
Control DCP	At Station 415+00 approximately 60 ft offshore at -5 ft depth contour.	Wave Height/Period and Water Level

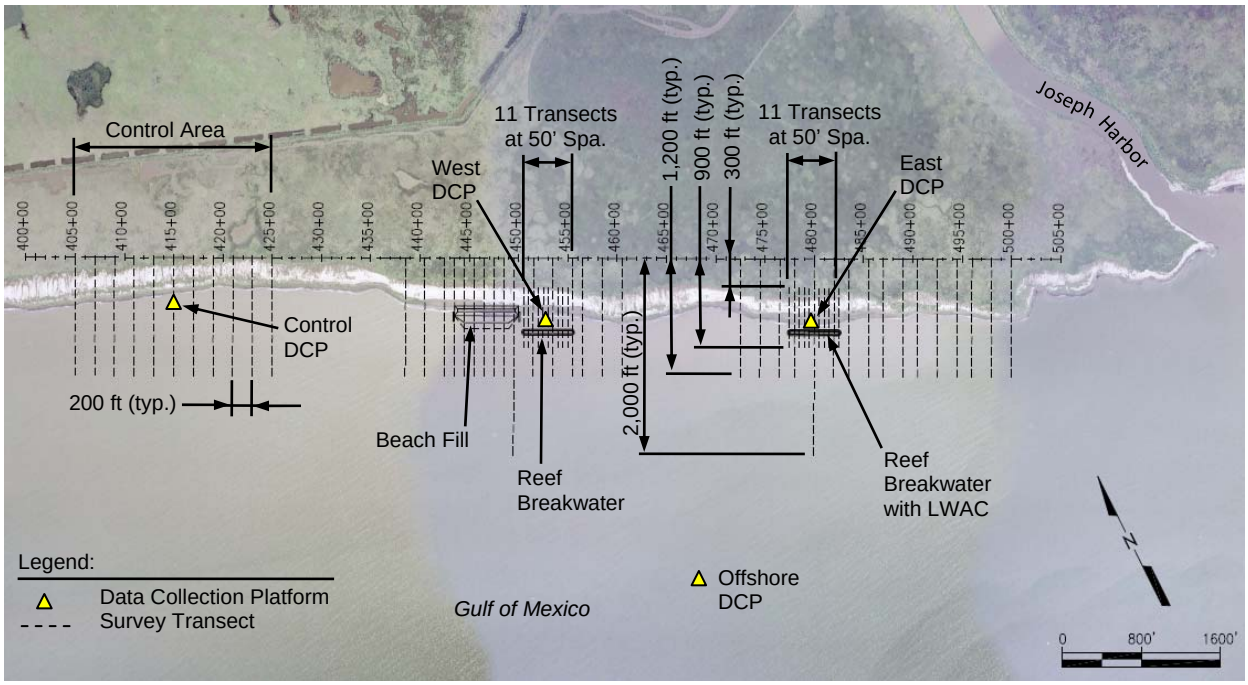

Figure 22. Data collection locations.

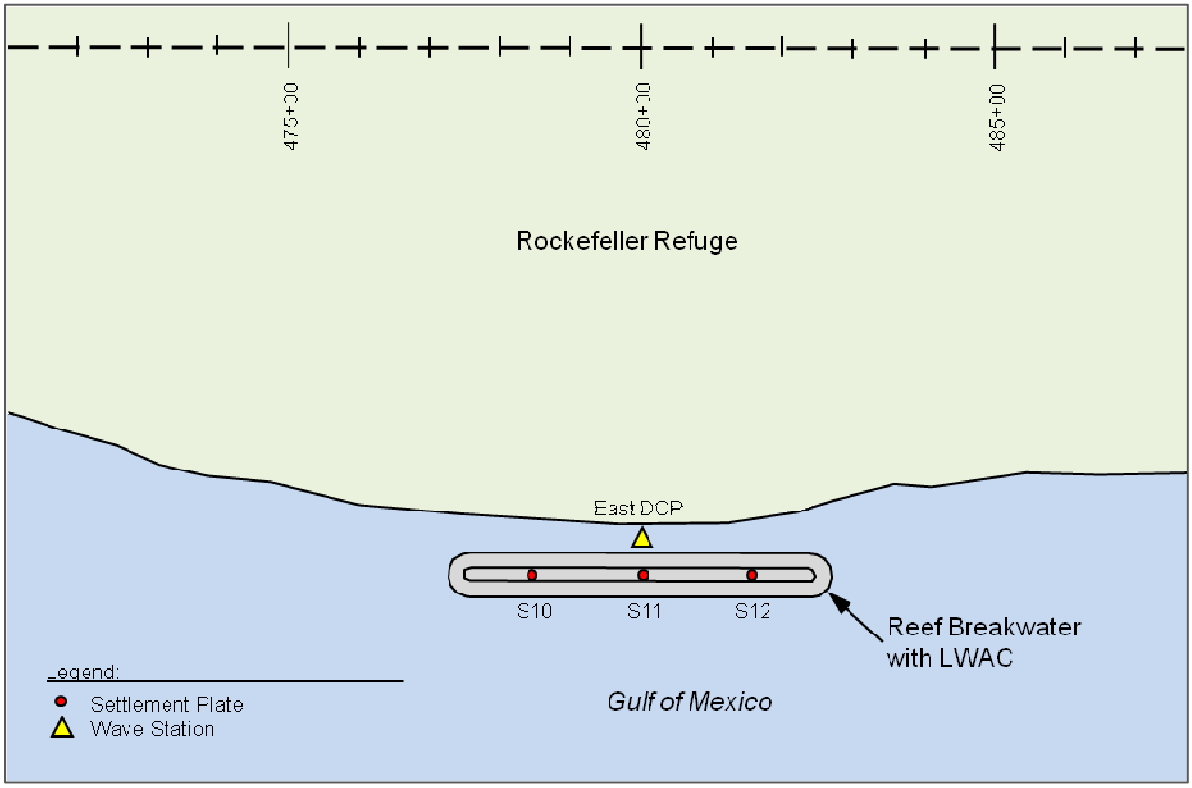

Figure 23. Data collection locations at Reef Breakwater with LWAC.

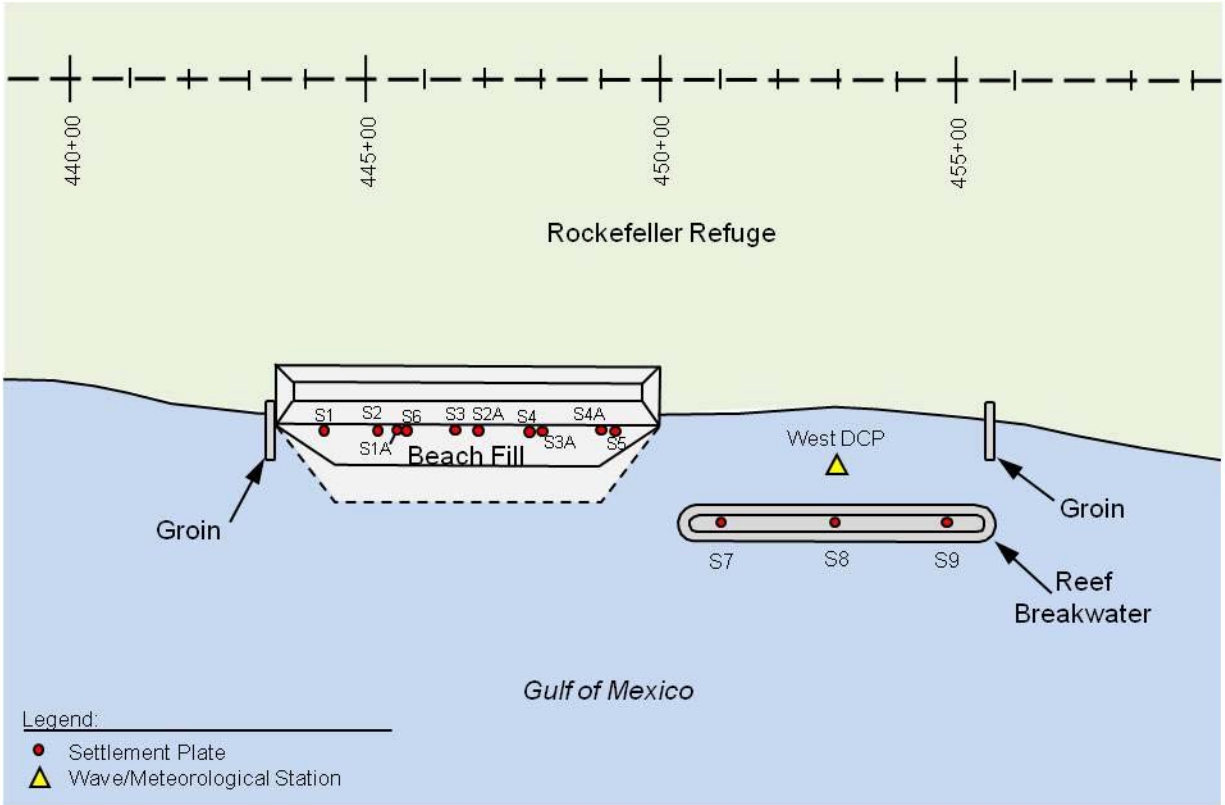

Figure 24. Data collection locations at Reef Breakwater and Beach Fill.

3.2 Settlement and Structural Integrity

The very soft foundation soils along the refuge shoreline present a significant design challenge and warranted considerable attention during development of the demonstration program. In addition to analysis of bearing capacity and slope stability, structure settlement was assessed in detail. If settlement of a particular structure is excessive, increased wave transmission will result in greater erosion of the marsh, and the structure may be deemed unacceptable for implementation in the full 9.2-mile project. Two basic types of settlement were considered during design: (1) elastic settlement (also referred to as "instantaneous" and "construction" settlement) due to soil displacement and (2) consolidation settlement due to soil consolidation. Installation of settlement plates during construction aided measurement of settlement. Figure 25 shows settlement plates installed during construction of the demonstration project.

Figure 25. Installation of settlement plates at Rockefeller Refuge.

Measurements of settlement for the reef breakwater with LWAC and reef breakwater are presented in Figures 26 and 27, respectively. Locations of the individual settlement plates are shown in Figures 23 and 24, and referenced by the settlement plate designation S1, S2, etc. The horizontal axis on these plots represents time after final rock placement, so they exclude elastic settlements that occurred as the rock was being placed. Also shown are the predicted and/or maximum acceptable levels of settlement developed during design. Meaningful measurements of beach fill settlement were impeded by rapid spreading of the fill and are not presented here.

Measured elastic settlements are plotted in Figure 28. As summarized in Table 3, the measured values were generally less than the predicted values. For design, elastic settlement is commonly estimated to be on the order of 20% of the (calculated) total consolidation settlement and/or initial structure height. The lesser values of elastic settlement shown in Figure 28 are a beneficial result of the monitoring program and, if made available to construction contractors, are expected to help reduce bid prices for the full 9.2 mile project.

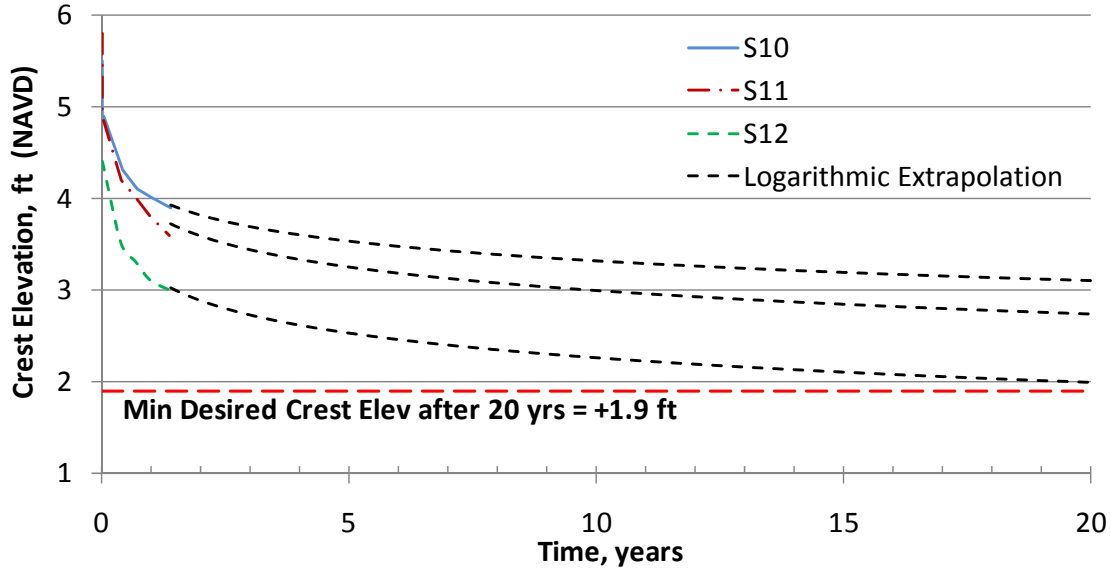

Figure 26. Measured and extrapolated settlement of Reef Breakwater with LWAC.

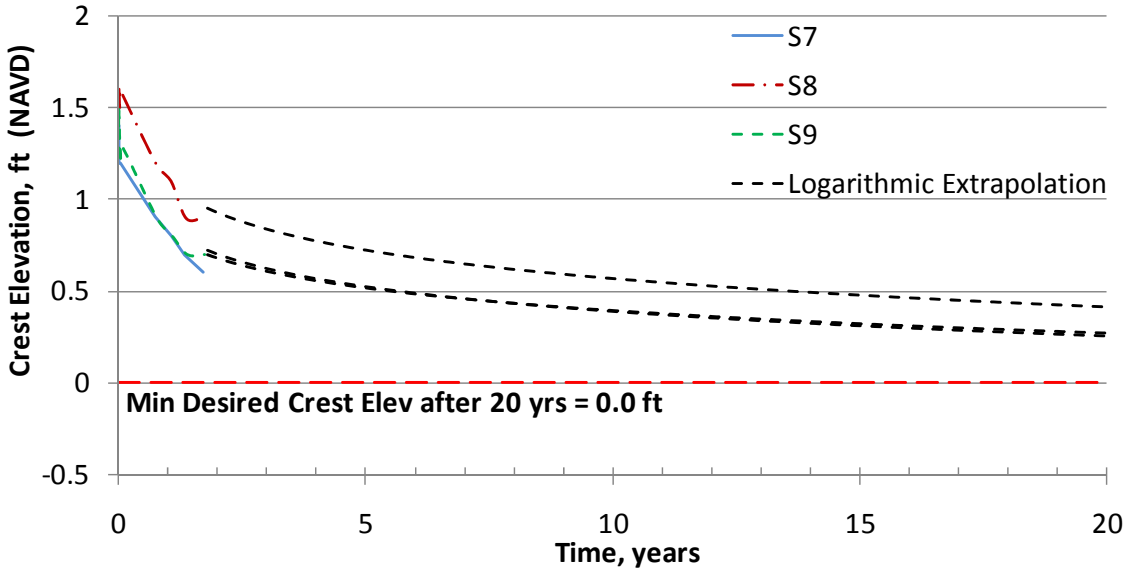

Figure 27. Measured and extrapolated settlement of Reef Breakwater.

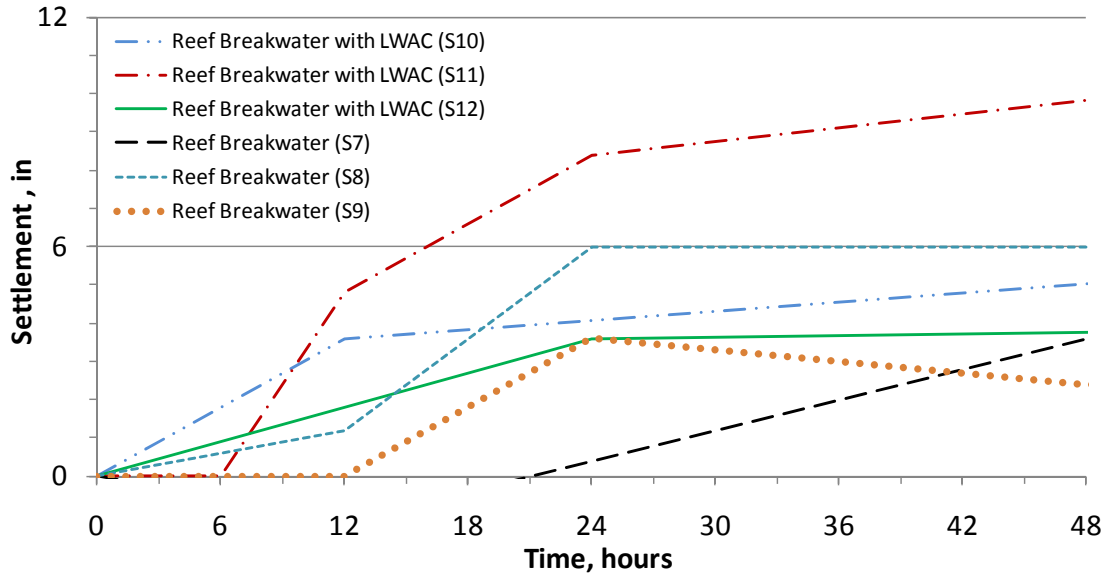

Figure 28. Elastic settlements measured at breakwaters during and immediately after rock placement.

Table 3. Elastic settlement of breakwaters.

Structure	Approximate Elastic Settlement, in	
	Predicted	Measured
Reef Breakwater with LWAC	6 to 24	S10 = 5
		S11 = 9
		S12 = 4
		Average = 6
Reef Breakwater	6 to 24	S7 = 4
		S8 = 6
		S9 = 4
		Average = 5

In addition to geotechnical stability, hydraulic stability of the breakwaters was assessed. With the exception of some minor displacement of armor stone caused by scour near the eastern terminus of the reef breakwater with LWAC (see Section 3.4 for additional discussion on scour); the riprap at both breakwaters appeared relatively stable during the monitoring period. Even though no hurricanes passed through the project area, the demonstration project helped to improve confidence in the stability of the breakwaters against waves because (1) the height of nearshore waves during storms will be limited by the broad, shallow shelf off the refuge coastline and (2) the relatively low crest of the breakwaters will allow energy from storm waves to pass overtop of the structures. The observed hydraulic stability is particularly significant for the reef breakwater with LWAC because available design theory does not account for the potential destabilizing effects of the LWAC.

3.3 Meteorological and Oceanographic Conditions

Wind

Wind data were collected to determine if (1) winds during the monitoring period were representative of those that typically occur on a seasonal and/or annual basis (for example, was the monitoring period unusually windy or calm) and (2) locally-measured wind speeds and directions are similar to those measured at nearby longer-term meteorological station at Calcasieu Pass (NOAA Station 8768094), approximately 45 miles to the west. Wind data collected during the monitoring period can also be used to develop a numerical wave model for design of the full 9.2 mile project.

A wind rose was developed for data measured from June 2006 to June 2011 at Calcasieu Pass (Figure 29). Note the predominant southeast wind direction. Wind speeds were less than 20 mph approximately 91 percent of the time.

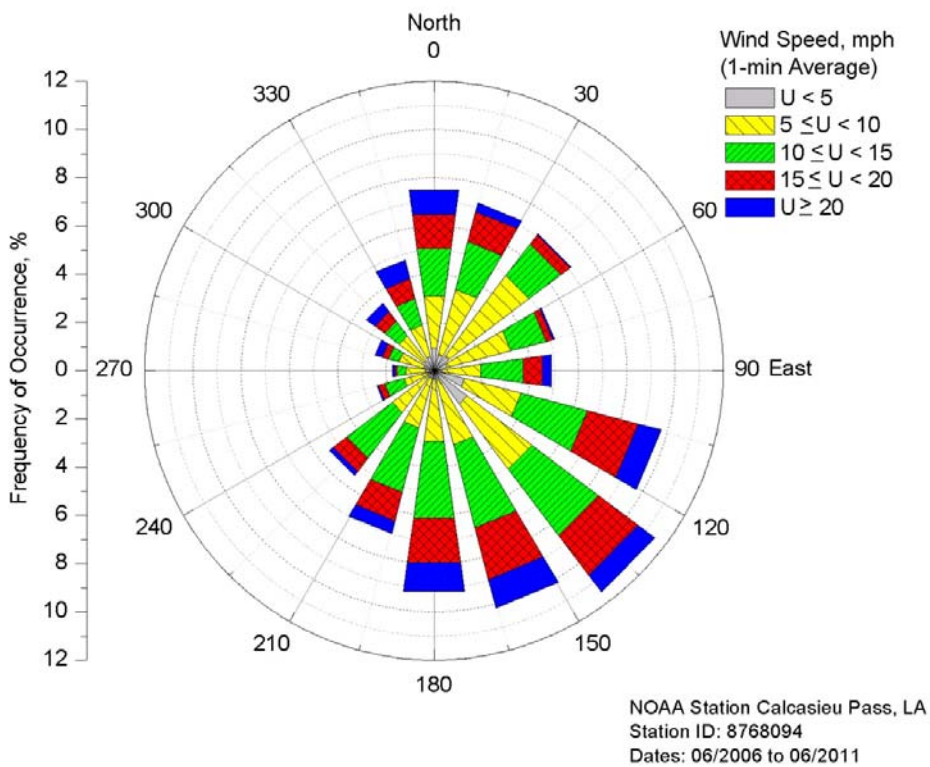

Figure 29. Calcasieu Pass (NOAA) wind rose.

At the project site, wind speed and direction were measured at the West DCP located landward of the reef breakwater. As shown in the wind rose in Figure 30, prevailing winds were predominantly from the south/southeast. Wind speeds during the collection period were less than 20 mph approximately 95 percent of the time with a maximum wind speed of 63.7 mph recorded on October 7, 2010. Figure 31 shows a wind rose at the Calcasieu Pass gage spanning the same months as the West DCP for the years 2006 to 2011. The prominent wind direction at Calcasieu Pass for this period is also from the south/southeast with a maximum wind speed of 65.5 mph.

Wind speeds were less than 20 mph approximately 92 percent of the time. The similarities between the two wind roses show that the wind data gathered during the monitoring period are representative of conditions that typically occur at the site.

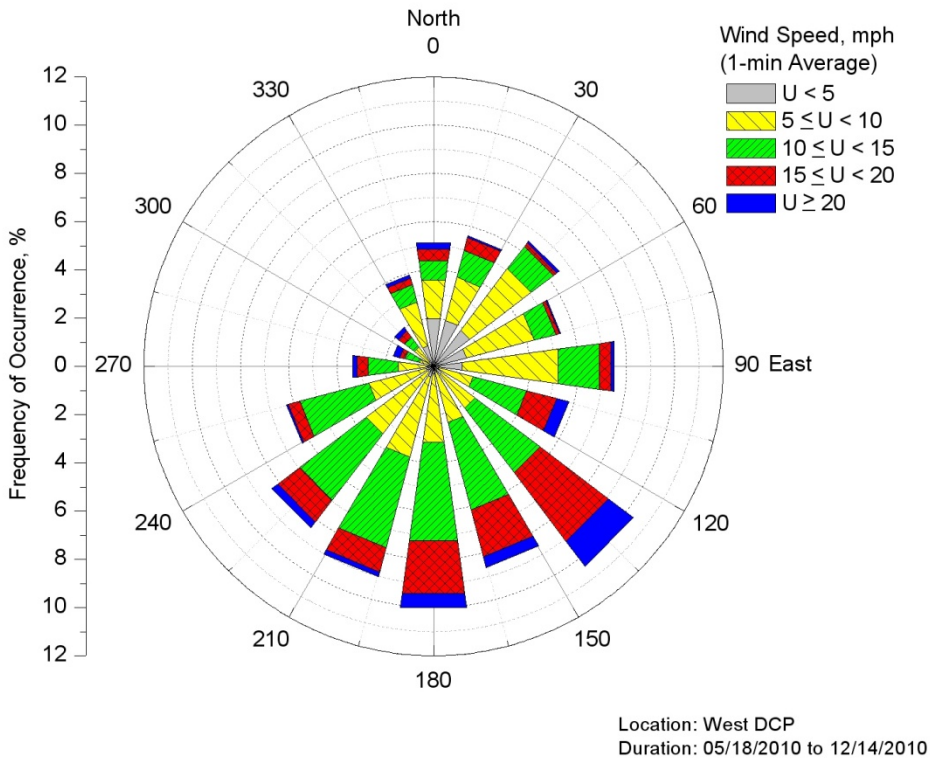

Figure 30. Rockefeller Refuge wind rose.

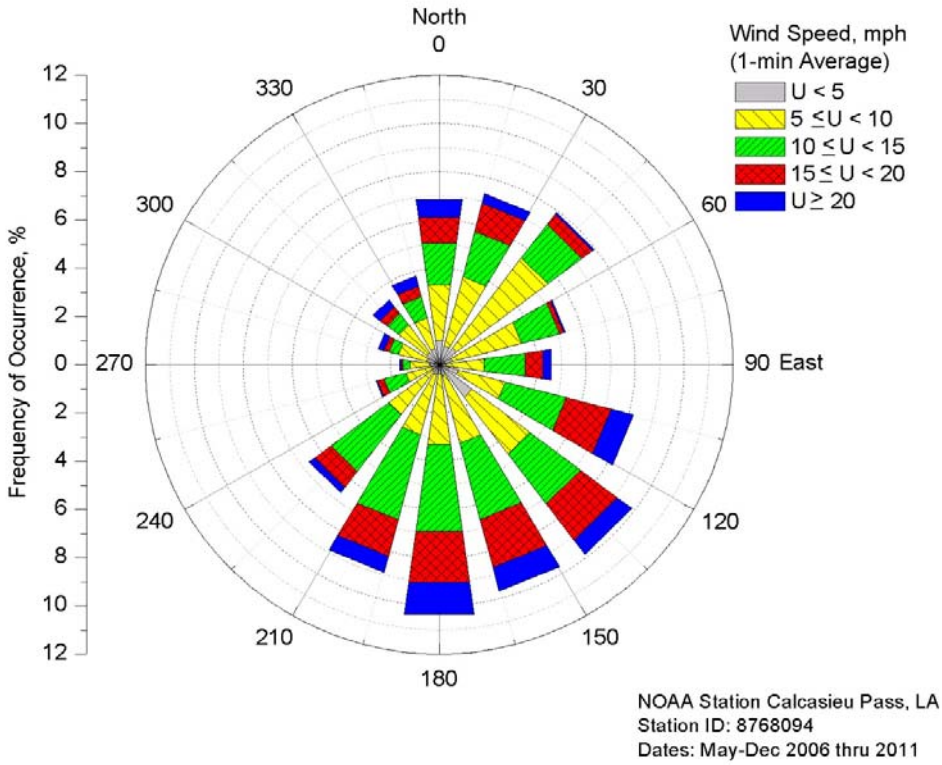

Figure 31. Calcasieu Pass (NOAA) wind rose (May to December).

Water Level

Water level data were recorded at the three nearshore DCP locations and referenced to NAVD ’88 based on surveys performed by John Chance Land Surveys, Inc. (2011). All three sets of water level data were compared to data recorded at Calcasieu Pass by USGS and the Freshwater Bayou South by USACE, shown in Appendix B. Although data recorded from both Calcasieu Pass and Freshwater Bayou South appear to be representative of the water levels at the project site, there are large gaps in the Freshwater Bayou South data during the monitoring period. If long-term water level data are needed for final design, it is recommended that the Calcasieu Pass record be applied.

Based on the comparison and a review of each data set, it was determined that the water level recorded at the West DCP provided the most continuous accuracy. The water level recorded at the West DCP is shown below in Figure 32. Figure 33 illustrates the percent of time the water level exceeded the crest elevation of each breakwater during the 6-month data collection period. Water level exceeded the crest elevation of the reef breakwater and reef breakwater with LWAC approximately 62% and 0% of the time, respectively. The water level percent exceedance curve for data obtained from Calcasieu Pass from January 2006 to March 2011 is also shown in Figure 33 to show the relative comparison for a longer-term water level record.

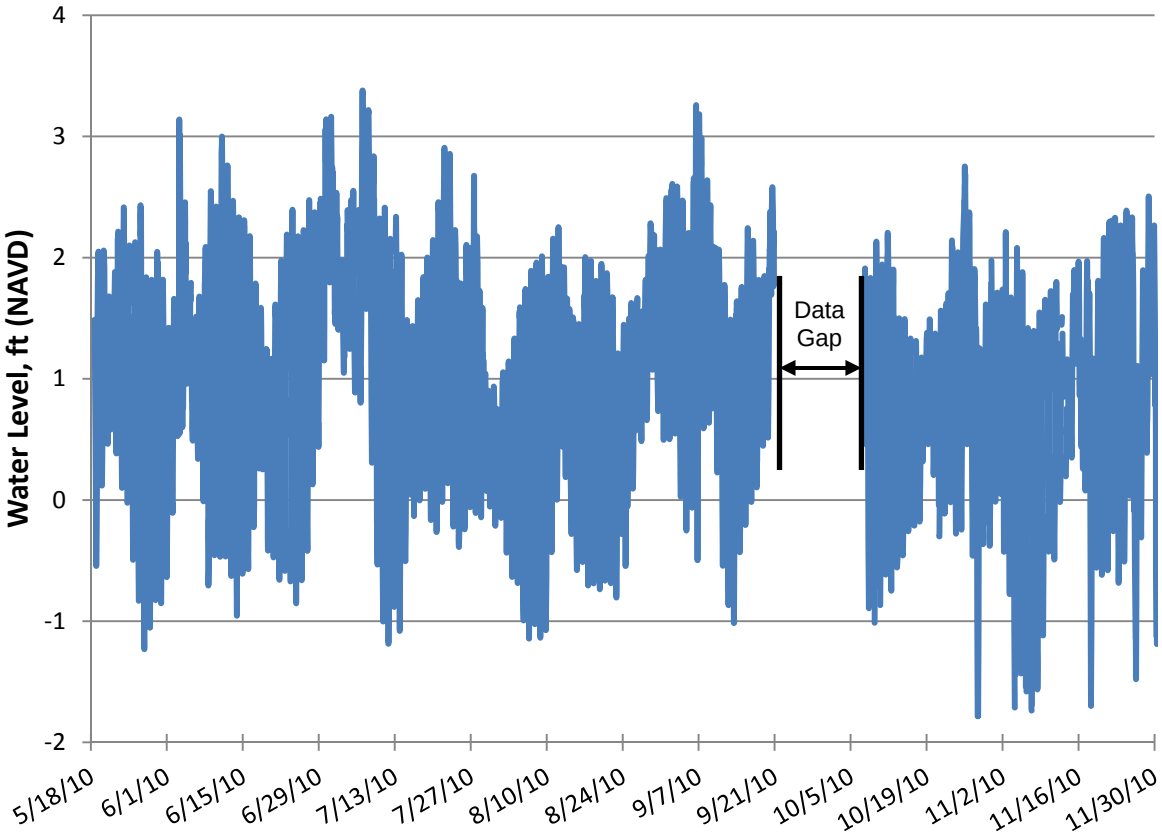

Figure 32. Water level data recorded at West DCP.

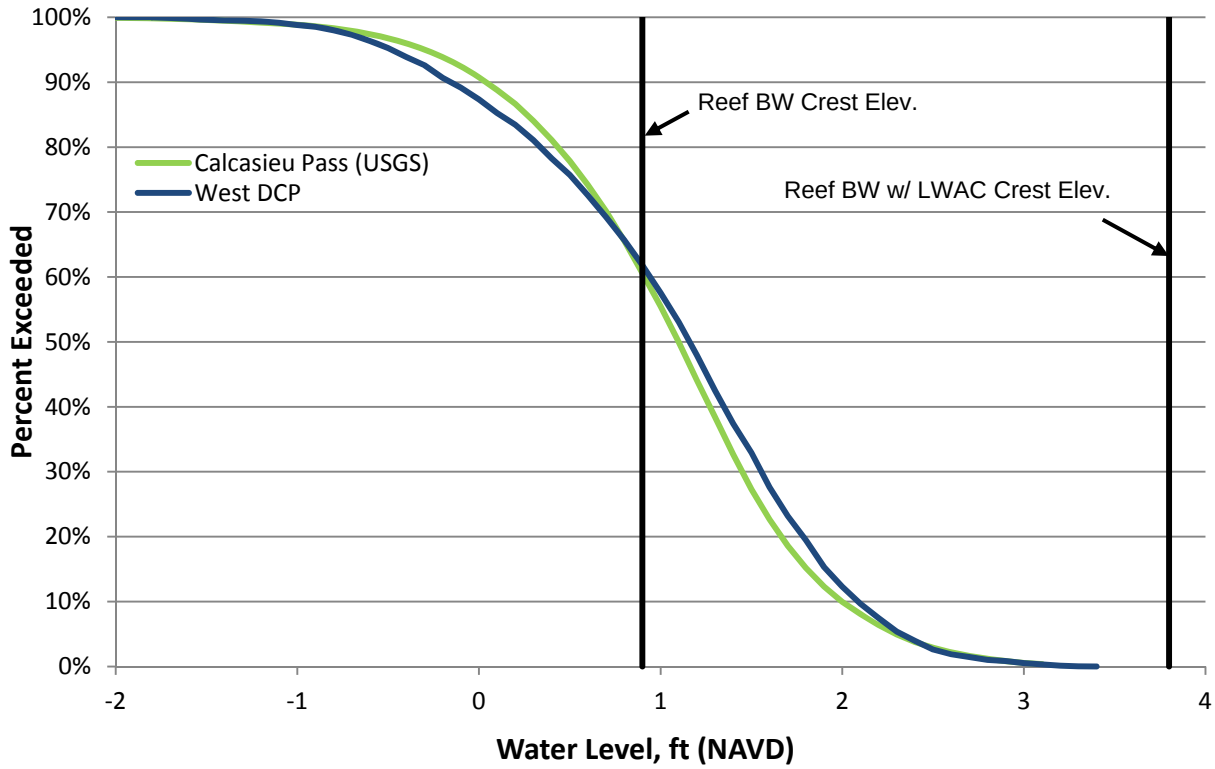

Figure 33. Water level exceedance at West DCP.

Waves

A wave rose for the Offshore DCP (installed in a water depth of approximately 14-ft located 3350 ft offshore) is shown in Figure 34. During the data collection period the predominant direction of wave propagation was from the south, as depicted in Figure 35. The significant wave height (H_s) was less than 1.5 ft approximately 92.5 percent of the time with an average peak wave period of 5.7 sec. The maximum recorded H_s was 3.2 ft with a peak period of 13.4 sec on July 1, 2010.

Tables 4 and 5 list the average and maximum monthly H_s for each of the DCPs. Note, although wave data was collected at each of the four locations during the month of November, the data for the Offshore DCP was deemed unusable due to power issues and is therefore not presented. For waves propagating from offshore to the control site, H_s tended to increase in May and June and decrease during the remaining four months. Average wave heights recorded landward of the reef breakwater and reef breakwater with LWAC were typically 67 and 96 percent smaller, respectively, than those recorded at the control site. Overall, the largest waves were recorded in June, July, and September compared to the other months.

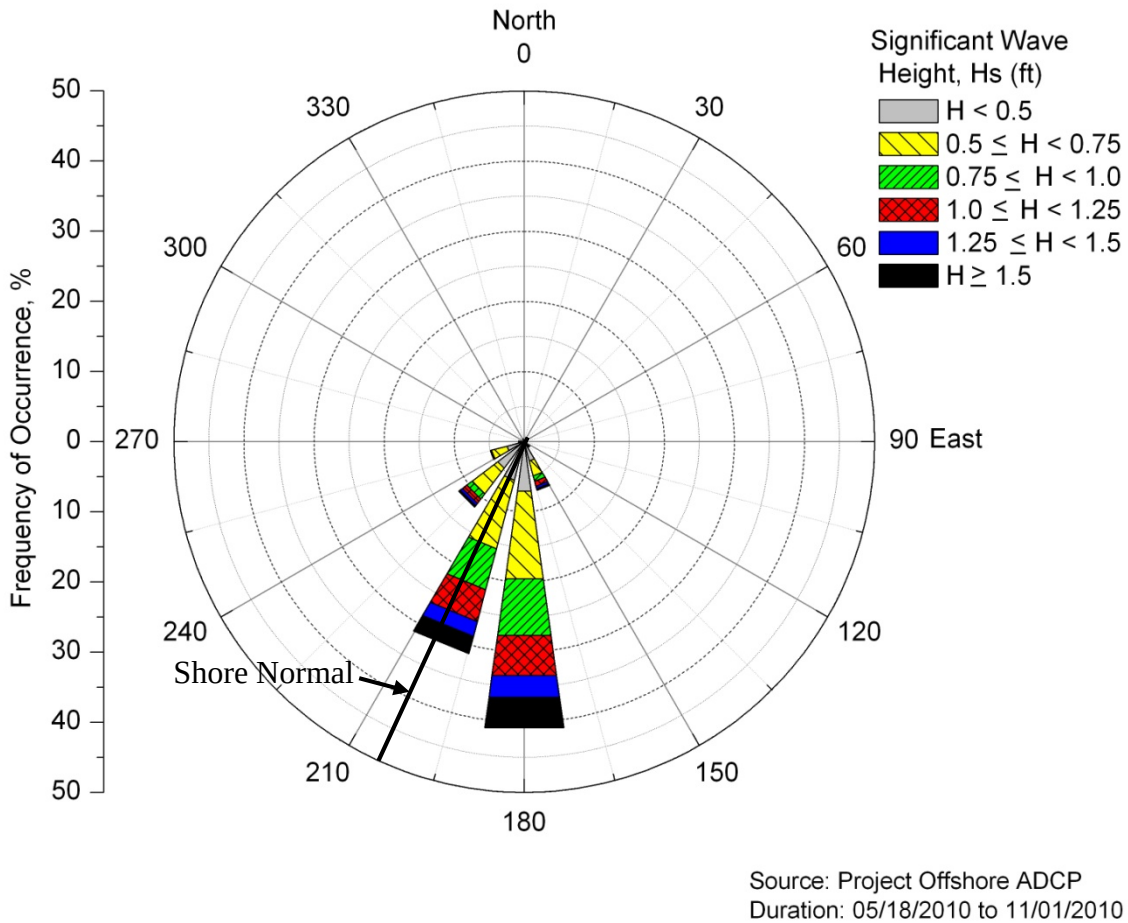

Figure 34. Rockefeller Refuge wave rose.

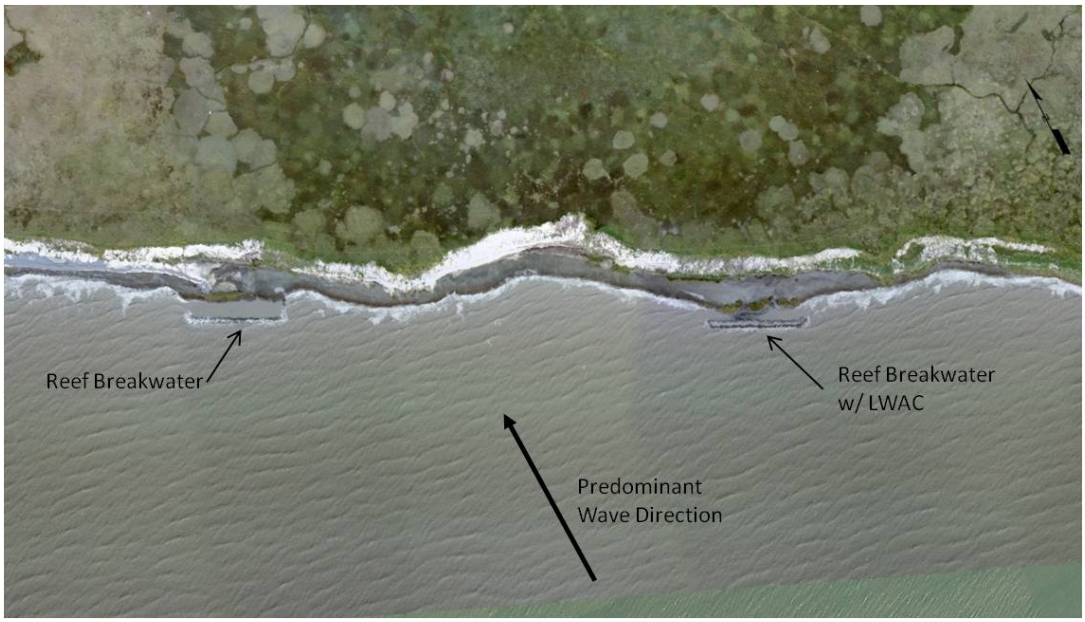

Figure 35. Predominant wave direction.

Table 4. Monthly average Hs, feet.				
Month	**Offshore DCP**	**Control DCP**	**West DCP**	**East DCP**
May 2010	0.7	1.0	0.1	0.03
June 2010	0.9	1.2	0.3	0.04
July 2010	0.9	0.5	0.3	0.03
August 2010	0.7	0.7	0.2	0.02
September 2010	0.9	0.7	0.4	0.04
October 2010	0.7	0.6	0.2	0.02

Analysis of the monthly maximum significant wave heights produced similar trends. As summarized in Table 5, monthly maximum H_s at the control area tended to be similar to, and in some cases larger than the offshore H_s. Monthly maximum H_s at both the reef breakwater and the reef breakwater with LWAC were significantly less than at the Offshore DCP. Monthly maximum H_s measured at the LWAC were typically smaller than those measured at the reef breakwater.

Table 5. Monthly maximum Hs, feet.				
Month	**Offshore DCP**	**Control DCP**	**West DCP**	**East DCP**
May 2010	1.6	2.2	0.5	0.1
June 2010	3.1	2.8	1.6	0.6
July 2010	3.2	3.1	1.9	0.5
August 2010	1.6	1.8	0.8	0.1
September 2010	2.6	2.8	1.5	0.5
October 2010	1.7	2.1	0.9	0.1

Of particular interest for evaluating performance of the breakwaters is their ability to attenuate wave energy. Since the primary cause of shoreline recession along the refuge Gulf shoreline is wave action, reducing the wave energy that impacts the shore will effectively reduce the shoreline recession. The ability of breakwaters to attenuate waves is commonly expressed through the wave transmission coefficient, K_t, which is the ratio of wave heights landward and seaward of the breakwater. Wave transmission coefficients were calculated using the data obtained from the three nearshore DCP's and the one offshore DCP. Offshore wave heights

were transformed (shoaled and refracted) to equivalent wave heights just seaward of the breakwaters using linear wave theory (Dean and Dalrymple, 1991). The equivalent nearshore wave heights were then compared to the corresponding wave heights just landward of the breakwaters. Calculated wave transmission coefficients, averaged over the entire monitoring period, are listed in Table 6. Owing to its higher crest elevation, the reef breakwater with LWAC was much more effective at dissipating waves.

Table 6. Average wave transmission coefficients, K_t.

Reef Breakwater	Reef Breakwater with LWAC
0.27	0.03

The 30% Design Report (Shiner Moseley 2004) lists design values for wave transmission for day-to-day conditions, a 1-year storm, and a Category 1 hurricane. During the monitoring period, waves meeting the day-to-day height of 1.6 ft were measured on 21 occasions. As a comparison to the targeted design values, the transmission coefficients, transmitted wave heights, and energy reduction percentages were calculated and averaged as summarized in Table 7. The comparison of design and measured values shows that the constructed structures met or performed better than the design thresholds, largely attributable to the lesser settlements and higher crest elevations than were assumed during design.

Table 7. Comparison of design and measured wave transmission for approximate "day-to-day" conditions.

	Reef Breakwater		Reef Breakwater with LWAC	
	Design	Actual	Design	Actual
Transmission Coefficient, Kt	0.48	0.37	0.13	0.04
Wave height seaward of structure, Hs (ft)	1.6	1.6	1.6	1.6
Wave height landward of structure, Hs (ft)	0.8	0.6	0.2	0.1
Energy Reduction (%)	75	84	98	99.7

Additionally, the transmission coefficients were compared based on the water level, seaward wave height at the breakwater, and wave period. A summary of these results is provided in Table C-1 in Appendix C. Within these categories, the reef breakwater with LWAC was consistently the most efficient option with less than 10% wave height transmission for all cases when water level was less than +3 ft. Effectiveness of the reef breakwater improved significantly

when water levels were less than +1 ft, which is expected considering its approximate +1.0 ft crest elevation.

The transmission coefficients calculated using the monitoring results were also compared to transmission coefficients calculated empirically based on Wamsley and Ahrens (2003). After initially considering several common empirical methods for calculation wave transmission, Wamsley and Ahrens (2003) was found to be the most applicable for the project breakwater geometry and wave conditions. Representative values for water level, wave height, and wave period were taken from each of the categories shown in Table C-1 discussed above and used in the calculation. The calculated coefficients are listed for the two breakwaters in Tables C-2 and C-3 in Appendix C. The difference between the coefficients from the physical results and the numerical results is listed in Table C-4.

Results from the Wamsley and Ahrens approach were found to vary significantly from the transmission coefficients calculated from the monitoring data. The average difference was approximately 0.3 for the reef breakwater and 0.4 for the reef breakwater with LWAC. The largest differences were for the reef breakwater with LWAC when nearshore H_s seaward of the breakwater was less than 0.5 ft. For the reef breakwater, the Wamsley and Ahrens results varied most when water levels were less than +1 ft, which corresponded to the water levels during which the reef breakwater attenuated waves most effectively. The least variation was found for the LWAC at wave periods less than 3 seconds, when wave transmission coefficients were extremely small. These results indicate that standard, published methodologies for empirical calculation of wave transmission should be applied with caution and/or modifications for design of future phases of ME-18.

3.4 Geomorphology

Shoreline Change

Shoreline change was determined from surveys completed at the end of construction in February 2010 and at three subsequent dates: August 2010, November 2010, and March 2011. To be consistent, the “shoreline” was defined as the +1.0 ft contour, except at Stations 454+00, 454+50, and 455+00 landward of the reef breakwater. The +0.5 ft contour was applied at these locations due to a lower scarp, as illustrated schematically in Figure 36. Shoreline change measured from the survey data was compared with aerial photography obtained throughout the monitoring period for qualitative verification. Aerial photography collected during the monitoring program is provided in Appendix D.

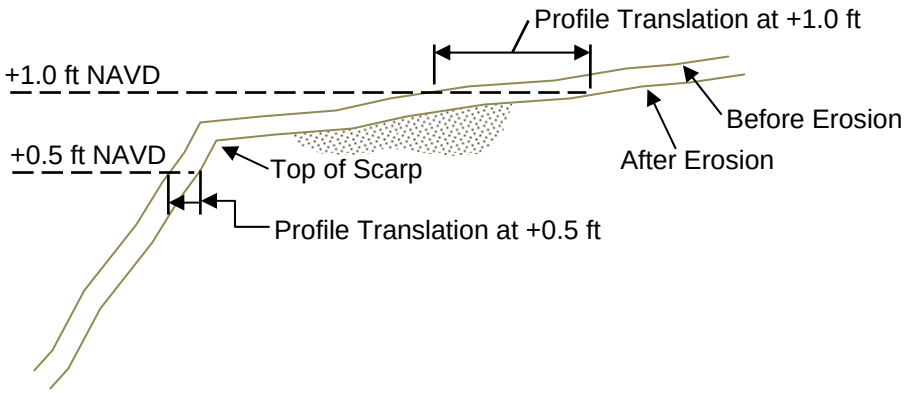

Figure 36. Representative schematic of Profile at Stations 454+00, 454+50, and 455+00.

Shoreline change at each survey transect overlaid on aerial photography is provided in Appendix E. Table 8 summarizes the average shoreline change at the three test installations and control area. The beach fill eroded approximately 85 ft between February 2010 and March 2011, with nearly 60 ft occurring during the first six months. As already discussed, the relatively rapid erosion of the beach fill was likely due to the short length of the test installation and significant lateral spreading (i.e., "end losses"). Anticipated challenges associated with test installations of beach fill were presented in Shiner Moseley (2006).

Erosion landward of the reef breakwater with LWAC was minimal with an average shoreline recession of approximately 3 feet. Shoreline recession landward of the reef breakwater for the same timeframe was 17.8 ft, and recession at the control area was 45.3 ft. Relative trends of shoreline recession at the breakwaters and control area are consistent with those observed for wave transmission. Locations of lesser wave transmission experienced less erosion.

Table 8. Average shoreline change.

	Average Shoreline Change, ft		
	February to August 2010 (6 mos)	**February to November 2010 (9 mos)**	**February 2010 to March 2011 (13 mos)**
Control Area	-26.9	-37.7	-45.3
Beach Fill	-59.5	-61.3	-84.4
Reef Breakwater	-8.4	-10.8	-17.8
Reef Breakwater with LWAC	-1.5	+0.5	-3.0

Nearshore Profile Morphology

An important aspect of the monitoring program was to document potential scour and other localized effects on the nearshore profile caused by each alternative. This analysis was accomplished through review of cross-shore profiles provided by JCLS (2011), observations made during site visits, and aerial and ground photography. Several representative cross-shore profiles are presented in Appendix F. Note that Stations 425+00 and 500+00 included data from the design surveys performed by JCLS in 2002; these data were included to help illustrate the extreme change in profile and shoreline position since design of the original project. The complete set of cross-shore profiles can be found in JCLS (2011).

Key findings of the profile analysis were as follows:

- Some localized scour around the east end of the reef breakwater with LWAC was observed. Although the scour is not evident in the survey data (likely due to the location of the survey transects), field observations revealed some localized scour around the end of the breakwater. The survey data do show that the crest elevation near the east end of the breakwater gradually became slightly lower than other portions of the breakwater, which may have been due to some spreading of the riprap due to the scour.
- In addition to widespread lowering of the nearshore profile, localized scour along the seaward toe of both breakwaters was observed in the survey data. Although the localized scour appears to have been relatively minimal, future monitoring/surveying of the breakwaters would provide valuable information regarding the need for design modifications of the seaward toe apron for the full 9.2 mile project.
- An accumulation of very soft/loose sediments was reported landward of the reef breakwater while surveys were being performed. However, the survey data did not provide evidence of this material.

4.0 ALTERNATIVE COMPARISON

Information and data collected during the construction and subsequent monitoring of the demonstration project produced valuable information which should be utilized to further refine design for the full project. Each alternative was evaluated based on five criteria: 1) ability to accommodate soft soil, 2) ability to attenuate waves, 3) ability to reduce shoreline recession, 4) constructability, and 5) cost. An alternative's ability to attenuate waves, accommodate soft soils, and reduce shoreline recession was assessed during the monitoring program. Information on the constructability and cost of each alternative was obtained from data and observations made during construction.

Ability to Accommodate Soft Soils

An alternative's ability to accommodate soft soils was based on the structure's settlement, measured at settlement plates, both during and after construction. Based on settlement measured during construction, all of the alternatives settled less than anticipated. Settlements measured during the monitoring period revealed that both the reef breakwater and reef breakwater with LWAC performed as expected or better.

Ability to Attenuate Waves

Wave transmission is a measure of the performance of a breakwater's ability to attenuate waves and is commonly expressed as a wave transmission coefficient (K_t), or the ratio of wave heights landward and seaward of the breakwater. The concept of wave transmission is not directly applicable to a beach fill because, under typical water levels, waves break directly on the shoreface and berm and are not transmitted landward of the beach. Therefore, the beach fill's ability to attenuate waves was not assessed other than visually. Wave transmission coefficients for the reef breakwater with LWAC and reef breakwater, as calculated from wave heights measured during the monitoring period, are summarized in Section 3.0. The reef breakwater performed reasonably well with an average K_t of 0.27, a reduction in wave height of 73%. The reef breakwater with LWAC performed the best, with an average K_t of 0.03, a reduction in wave height of 97%.

Ability to Reduce Erosion

Survey data along with aerial photography collected during construction and the monitoring period were applied to assess each of the alternative's ability to reduce erosion along the refuge shoreline. For this evaluation criterion, the alternatives were not only compared to each other but also to the control area. The shoreline landward the reef breakwater with LWAC, reef breakwater, and beach fill receded an average of approximately 3 ft, 18 ft, and 84 ft, respectively, over the 14-month monitoring period. In comparison, the control area experienced average erosion of approximately 45 ft over the 14 month period. While the high rate of erosion of the beach fill may be somewhat misleading due to initial equilibration and subsequent spreading of the fill, it is evident that the beach fill alternative was not effective at reducing erosion, particularly when compared to the breakwaters. Although both breakwaters showed a

significant reduction in shoreline recession compared to the control area, the reef breakwater with LWAC performed the best at reducing erosion.

Constructability

Although the constructability of a structure is not easily quantifiable, a relative comparison of the alternatives was made based on observations during construction and discussions with the construction contractor. The reef breakwater appeared to be the simplest structure to construct, primarily due to its homogeneous cross-section. The reef breakwater with LWAC had a more complex cross-section and required prefabrication of the containers for the LWAC. Although the reef breakwater with LWAC required more steps to construct, once the contractor became familiar with the process, the rate of construction improved.

The beach fill appeared to be the most challenging to construct. The beach fill required a much higher placement density of materials than the breakwaters, requiring more time per linear foot of construction. The location of the landward portion of the breakwater was difficult to reach with conventional equipment, and, as shown in Figure 11, working with large equipment on land was ineffective. All three of the alternatives were constructed without the use of a flotation channel, although the construction contractor was unable to work on the beach fill during periods of low tide; this increased the time required for beach fill construction.

Cost

Unit costs from the demonstration project provide a relative comparison of costs. Based on those unit costs, the reef breakwater was the lowest-cost alternative. The unit price of the reef breakwater with LWAC and beach fill were approximately 22% and 55%, respectively, higher than the unit price of the reef breakwater.

Comparison Overview

Table 9 presents a qualitative comparison of the alternatives. Although this qualitative comparison is somewhat subjective, it provides a general overview and relative performance comparison based on the five criteria. As shown in Table 9, the reef breakwater with LWAC performed the best overall and the beach fill performed the worst. Note the beach fill's ability to attenuate waves was not scored for reasons discussed above, and its ability to accommodate soft soil is only based on observations made during and immediately after construction.

Table 9. Alternative comparison matrix.

Legend 3 = Good 2 = Moderate 1 = Poor	**Reef Breakwater with LWAC**	**Reef Breakwater**	**Beach Fill**
Ability to Accommodate Soft Soils	3	3	2
Ability to Attenuate Waves	3	2	N/A
Ability to Reduce Shoreline Erosion	3	2	1
Constructability	2	3	1
Cost	2	2	1

5.0 OPINION OF PROBABLE CONSTRUCTION COST

Based on results of the demonstration project and monitoring effort, cost was reassessed for the full 9.2 mile project constructed with the reef breakwater with LWAC alternative. Cost was not assessed further for the reef breakwater alternative because it did not reduce shoreline recession to the desired extent.

As shown in more detail in Appendix G, opinions of probable construction cost (cost) were developed for the following three cases:

- Estimate A: Existing Design, Full 9.2 Miles
- Estimate B: Theoretical Design Refinements, Full 9.2 Miles
- Estimate C: Existing Design, $50,000,000 Budget

These estimates are explained in more detail below. For each case, unit costs were reduced from those incurred during the demonstration program to account for economy of scale benefits. In addition, several experienced construction contractors (including the contractor who constructed the demonstration project) were interviewed for projections on cost, schedule, and other considerations for construction. As noted in the itemized cost breakdowns provided in Appendix G, contingencies of 25% (as required for typical CWPPRA cost estimates) and 15% were applied. Based on the significant design effort that has already been put forth on this project as well as knowledge gained from the demonstration program, it is recommended that a 15% contingency be considered (instead of 25%) for project budgeting purposes. Note that all estimates exceed the original target budget of $38,000,000.

An important aspect of the construction cost is schedule. The costs presented herein assume the project will be bid under one contract and will allow a flexible construction period. Exposure to open Gulf weather and wave conditions will limit the time of year a construction contractor would be able to work effectively and efficiently. This would likely extend the construction schedule to between three to four years. It is recommended that flexibility in the contract time be given to the contractor to allow for efficient use of good weather periods. Also, allowing the contractor to demobilize during long periods of expected downtime due to weather may lessen the risk the contractor assumes, potentially reducing overall costs.

Due to the very large volume of stone and other material that would be needed to complete construction, project schedule should be coordinated with potential quarries and suppliers to provide them upfront knowledge of the project and allow them time to prepare for the project.

5.1 Estimate A: Existing Design, Full 9.2 Miles

This cost is for a full 9.2-mile long project based on the existing design for the reef breakwater with LWAC as constructed for the demonstration project. With a contingency of 15%, the total cost is approximately $98,919,550 (or $2,036/LF).

5.2 Estimate B: Theoretical Design Refinements, Full 9.2 Miles

Based on data collected and observations made during the demonstration project and monitoring period, several theoretical design refinements were applied to the reef breakwater with LWAC to assess potential cost savings. Actual design refinements for the full project should be thoroughly assessed during detailed design. For a contingency of 15%, the total cost is approximately $87,419,550 (or $1,800/LF).

5.3 Estimate C: Existing Design, $50,000,000 Budget

This case was included to determine the potential length of a project if the construction budget is limited to $50,000,000. Similar to Estimate A, the breakwater cross section applied for the demonstration project was assumed. For a 15% contingency, the length of the project would be approximately 4.7 miles.

6.0 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

The purpose of this monitoring program was to evaluate performance of three shoreline protection alternatives constructed at the Rockefeller Wildlife Refuge. The test sections consisted of a reef breakwater, a reef breakwater with LWAC, and a gravel/crushed stone beach fill. Project construction was completed December 2009 and monitoring was completed April 2011. Overall objectives of the monitoring program were to:

- Compare the performance of each alternative;
- Determine which alternative is most suitable for a full 9.2-mile project;
- Present data to aid in design optimization for the most suitable alternative.

More specifically, each alternative was compared in terms of ability to accommodate soft soils, ability to attenuate waves, ability to reduce erosion, constructability, and cost. To assist in the evaluation of each alternative, the monitoring program included collection of wave and tide data at three nearshore locations and one offshore location as well as meteorological data at one of the nearshore locations. Topographic/bathymetric survey data and aerial photography were also collected at three separate time intervals during the monitoring period.

6.2 Conclusions and Recommendations

Primary results of the monitoring program are as follows:

- Because of the extremely soft clay foundation soils along the refuge shoreline, the potential for excessive settlements during and subsequent to construction were a significant concern. Actual settlements were less than anticipated, which will help reduce unit cost of the full project.

- Review of measured shoreline recession rates along the demonstration project area revealed that, as expected, the reduction of shoreline recession was the greatest in the center of each breakwater, with recession increasing towards the breakwater ends. It is therefore recommended to construct the full project as continuous as feasible with minimal gaps.

- A comparison of tide data collected at the demonstration project site to nearby long-term tide records indicated that water levels recorded at the USGS Calcasieu Pass gage are most similar to those at the refuge. Long-term data from the Calcasieu Pass gage are therefore recommended for application during design of the full project. The NOAA Calcasieu Pass site is recommended for long-term records of wind.

- Additional wave transmission analysis will be an important aspect of the design of the full 9.2 mile project. Data and results from the wave transmission analysis presented herein should be applied to refine cross-sectional geometry for the full project.

- The reef breakwater with LWAC is recommended for the full 9.2 mile project, primarily due to the breakwater's better ability to attenuate waves and reduce shoreline recession. This alternative also performed well in accommodating the soft foundation soils and displayed adequate hydraulic stability despite potential destabilizing effects of the LWAC.

- The reef breakwater was marginally effective and is recommended for consideration as a lower-cost alternative to the reef breakwater with LWAC should project stakeholders be willing to consider lesser design criteria.

- Performance evaluation of the beach fill was limited by its relatively short length and significant end losses (lateral spreading). However, based on the monitoring results, it is unlikely to provide the same level of shoreline protection as the breakwater options. Also, it is unlikely to be cost feasible and posed the greatest constructability challenges.

- Consideration of design modifications and/or improvements for the full project should include refining breakwater crest elevation and width based on measured wave transmission and refining the construction template to account for measured settlements.

- Preliminary cost projections for the reef breakwater with LWAC have been developed for the full 9.2 mile project. Depending on the level of breakwater cross-section refinement that can be realized through detailed design, the cost for a full project is expected to range from approximately $87,400,000 to $98,900,000. For a hypothetical construction budget of $50,000,000, the project length would be approximately 4.7 miles.

- Construction of the full 9.2 mile project is expected take approximately three to four years to complete, mainly due to the size and remote location of the project as well as the amount of expected downtime due to weather in the exposed Gulf. It is recommended the construction contractor be provided flexibility in the construction schedule to allow for efficient use of good weather days.

- Monitoring of the demonstration project is recommended to continue through design of the full 9.2 mile project. At a minimum, annual surveys and aerial photography should be performed to assist in assessing the alternatives longer-term performance. In addition to benefiting projects along the Rockefeller Refuge shoreline, such monitoring will benefit other similar projects across coastal Louisiana.

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APPENDIX A

Installation Report for Wave and Meteorological Stations

Installation of Wave and Meteorological Stations for the Rockefeller Refuge Gulf Shoreline Stabilization Demonstration Project (ME-18)

Prepared for:

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600 Jefferson Street Suite 504
Lafayette, LA 70501-6987

ENCOS Project No. 09-0154

Prepared by:

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June 2010

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1. Map of project area and DCP locations

APPENDIX

Appendix
A. Report in pdf format

1.0 Introduction

ENCOS, Inc. (ENCOS) was contracted by HDR, Inc. (HDR) in December 2009 to provide data collection for a six-month period for the Rockefeller Refuge Gulf Shoreline Stabilization Demonstration Project (ME-18). ENCOS will provide data collection services associated with weather and wave properties in the project within the Rockefeller Wildlife Management Area (WMA) and Game Refuge, Cameron and Vermillion Parishes, LA. This report documents the installation of the data collection platforms (DCPs).

2.0 Scope of Work

ENCOS installed and will maintain three (3) surf zone DCPs and one (1) offshore DCP. The DCPs will collect wave parameters and weather conditions. The surf zone DCPs are to be surveyed to provide water level corrected to North American Vertical Datum 1988 (NAVD 88), significant wave height, and wave period. All DCPs are visited monthly to clean the sensors and recover data. The collected data will be used to calibrate HDR models for wave transmission of breakwaters. **Figure 1** contains a map of the DCP locations within the project area.

3.0 Equipment

ENCOS installed the NexSens ISIC 3100 data logger interfaced with a NexSens AccuStage water level sensor for the three (3) surf zone DCPs. These systems are capable of collecting wave height, wave period, and tide measurements. The surf zone DCPs are set to Central Standard Time (CST) and collect wave parameters for four (4) minute duration every 30 minutes. The surf-zone DCPs provide raw wave and tidal information in nearly real-time to an offsite database on an hourly basis.

ENCOS installed a Vaisala WXT 500 at the west breakwater. West DCP weather station collects wind speed and direction, daily precipitation, relative humidity, barometric pressure, and air temperature. The weather station transmits data to an off-site database in nearly real-time on an hourly basis.

ENCOS installed the Teledyne RD Instrument Workhorse Monitor acoustic doppler current profiler (ADCP) as the Offshore DCP. The ADCP collects wave direction, wave height, wave period, and unreferenced water level. The ADCP is set to CST and collects wave parameters and velocity profiles once every 60 minutes.

4.0 Data Collection Platforms

ENCOS constructed West DCP from four stainless steel 2-inch diameter well casings driven to refusal. A 3 ft x 3 ft piece of fiberglass grating was secured to the well casing pilings as a platform (**Table 1**). The field crew attached the wave sensor to West DCP in the same manner as East and Control DCPs mentioned above. ENCOS personnel mounted the weather station on top of a 20 ft length of galvanized 2 inch pipe driven into the seabed adjacent to one of the DCP support pilings. The weather station was mounted above all structures on the DCP to ensure free flow of air around the weather sensor.

ENCOS used existing 12-inch pilings installed by HDR for three (3) of the four (4) DCPs. East DCP and Control DCP are constructed of 1-inch diameter schedule 40 aluminum pipe 8 ft long mounted to a 6-inch by 8 ft piece of aluminum channel. ENCOS placed the wave sensor inside the 1-inch pipe for protection against tampering and boat traffic. The wave sensor extends below the pole to allow for free exchange of water to the instrument. ENCOS secured the DCPs to the wooden piling with three to four 6-inch lag screws and several bands of ¾ inch stainless steel banding wrapped around the DCP and the wooden piling (**Tables 2 and 3**).

ENCOS used the remaining 12 inch piling installed by HDR as the Offshore DCP. The ADCP is enclosed in a 4 ft x 5 ft x 2 ft trawl frame which sits on the seafloor. ENCOS tethered the trawl frame to the 12 inch piling with a 20 ft length of chain (**Table 4**).

5.0 Field Activities

Below is a chronological summary of ENCOS field activity installing and servicing the DCPs.

March 23, 2010 – ENCOS performed site reconnaissance to assess the project area including: access routes and vessels needed, subsurface conditions, proximity to boating traffic, water depths, site conditions, and type of equipment suitable for project conditions.

April 21, 2010 – ENCOS installed West DCP structure.

April 28, 2010 – ENCOS personnel deployed Offshore DCP and ran a one month preliminary data test on the ADCP unit.

May 17, 2010 & May 18, 2010 – ENCOS field crew completed installation of all wave DCP instruments; all sites are operational.

6.0 Access Rights and Rights of Entry

ENCOS received access rights to the Rockefeller WMA Refuge from Refuge Managers Guthrie Perry and Tom Hess to conduct initial site visit, site installation, and site maintenance throughout the duration of the shoreline stabilization demonstration project.

7.0 Surveys

ENCOS will coordinate with the survey contractor on the requirements needed to adjust water levels to NAVD88.

8.0 Conclusion

ENCOS installed four (4) DCPs within the project area for the Rockefeller Refuge Gulf Shoreline Stabilization Demonstration Project (BE-15) from March – May 2010. The stations require cleaning, calibration, and data recovery by ENCOS monthly through November 2010. The stations are scheduled for decommissioning in December 2010.

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Imagery: Landsat Thematic Mapper Satellite Image 2005,
UTM Zone 15 NAD83, LOSCO

Figure 1. Map of Data Collection Platforms
Rockefeller Refuge Gulf Shoreline
Stabilization Demonstration Project (ME-18)
ENCOS Project No. 09-0154

May 2010

N

Map Scale 1 in = 1,750 feet 1:21,000

0 0.15 0.3 0.6 0.9 1.2 Miles

TABLE 1 - Specifications and conditions at the time of install for West DCP

STATION NAME:	West DCP
LOCATION:	South of Rockefeller NWR in Gulf near beach. Data Logger iSIC ID #10 and #13.
DESCRIPTION:	Wave and Meteorological Station
INSTALLATION DATE:	5/17/2010
INSTALLED BY:	G.Badon, J.Chaky, A. Trahan
STATION TYPE:	stainless steel pile structure
STOP PLATE TO PRESSURE SENSOR DISTANCE:	0.065'
SURVEY ELEVATION:	TBD
WATER LEVEL AT INSTALLATION:	4.12'
LATITUDE:	N29.63685
LONGITUDE:	W92.77813
MEASURED OVER ALL CASING LENGTH:	8.00'
MARK TO SENSOR DISTANCE:	TBD
DISTANCE OF CASING ABOVE WATER LINE (AT INSTALL):	4.21'
DISTANCE OF CASING BELOW WATER LINE (AT INSTALL):	3.79'
DISTANCE BELOW MUD LINE (UNIPOLE):	N/A
BASE PLATE TO TOC (UNIPOLE):	N/A
STOP PLATE TO TOC:	7.56'

TABLE 2 - Specifications and conditions at the time of install for East DCP

STATION NAME:	East DCP
LOCATION:	South of Rockefeller NWR in Gulf near beach. Data Logger iSIC ID #12.
DESCRIPTION:	Wave Station
INSTALLATION DATE:	5/17/2010
INSTALLED BY:	G.Badon, J.Chaky, A. Trahan
STATION TYPE:	1" Schd 80 Al
STOP PLATE TO PRESSURE SENSOR DISTANCE:	0.065'
SURVEY ELEVATION:	TBD
WATER LEVEL AT INSTALLATION:	2.8'
LATITUDE:	N29.64025
LONGITUDE:	W92.78577
MEASURED OVER ALL CASING LENGTH:	8.00'
MARK TO SENSOR DISTANCE:	TBD
DISTANCE OF CASING ABOVE WATER LINE (AT INSTALL):	5.7'
DISTANCE OF CASING BELOW WATER LINE (AT INSTALL):	2.3'
DISTANCE BELOW MUD LINE (UNIPOLE):	N/A
BASE PLATE TO TOC (UNIPOLE):	N/A
STOP PLATE TO TOC:	7.56'

TABLE 3 - Specifications and conditions at the time of install for Control DCP

STATION NAME:	Control DCP
LOCATION:	South of Rockefeller NWR in Gulf near beach. Data Logger iSIC ID #11.
DESCRIPTION:	Wave Station
INSTALLATION DATE:	5/18/2010
INSTALLED BY:	G.Badon, J.Chaky, A. Trahan
STATION TYPE:	1" Schd 80 Al
STOP PLATE TO PRESSURE SENSOR DISTANCE:	0.065'
SURVEY ELEVATION:	TBD
WATER LEVEL AT INSTALLATION:	7.6'
LATITUDE:	N29.64498
LONGITUDE:	W92.79539
MEASURED OVER ALL CASING LENGTH:	8.00'
MARK TO SENSOR DISTANCE:	TBD
DISTANCE OF CASING ABOVE WATER LINE (AT INSTALL):	4.0'
DISTANCE OF CASING BELOW WATER LINE (AT INSTALL):	4.0'
DISTANCE BELOW MUD LINE (UNIPOLE):	N/A
BASE PLATE TO TOC (UNIPOLE):	N/A
STOP PLATE TO TOC:	7.5'

TABLE 4 - Specifications and conditions at the time of install for Offshore DCP

STATION NAME:	Offshore DCP
LOCATION:	South of Rockefeller NWR in Gulf. Workhorse monitor.
DESCRIPTION:	ADCP Wave Station
INSTALLATION DATE:	4/28/2010
INSTALLED BY:	P. Templet, J.Chaky, A. Trahan
STATION TYPE:	Trawl Frame
STOP PLATE TO PRESSURE SENSOR DISTANCE:	N/A
SURVEY ELEVATION:	N/A
WATER LEVEL AT INSTALLATION:	14'
LATITUDE:	N29.62983
LONGITUDE:	W92.78491
MEASURED OVER ALL CASING LENGTH:	N/A
MARK TO SENSOR DISTANCE:	N/A
DISTANCE OF CASING ABOVE WATER LINE (AT INSTALL):	N/A
DISTANCE OF CASING BELOW WATER LINE (AT INSTALL):	N/A
DISTANCE BELOW MUD LINE (UNIPOLE):	N/A
BASE PLATE TO TOC (UNIPOLE):	N/A
STOP PLATE TO TOC:	N/A

APPENDIX B

Water Level Data

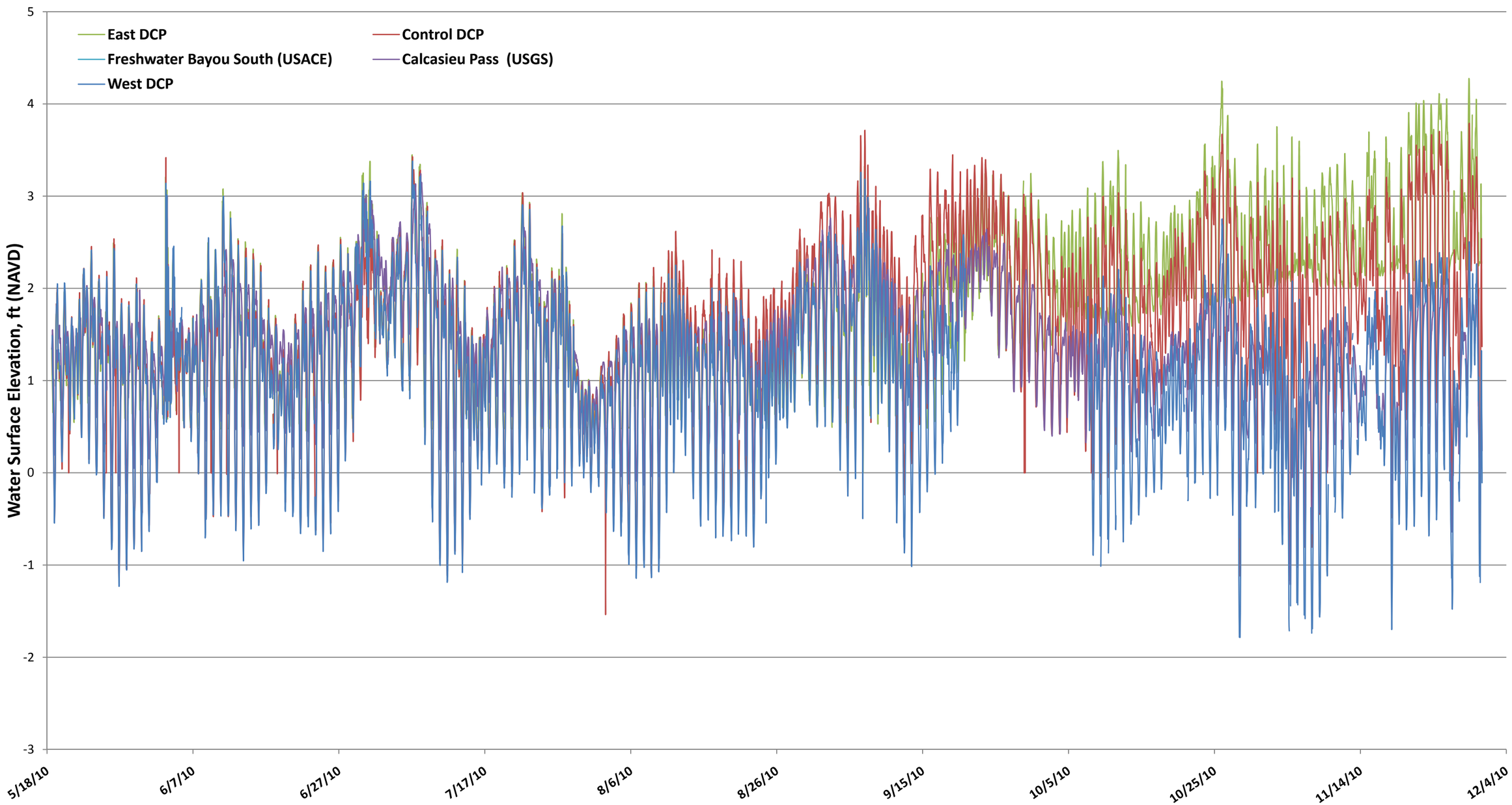

Figure B-1. Water level comparison.

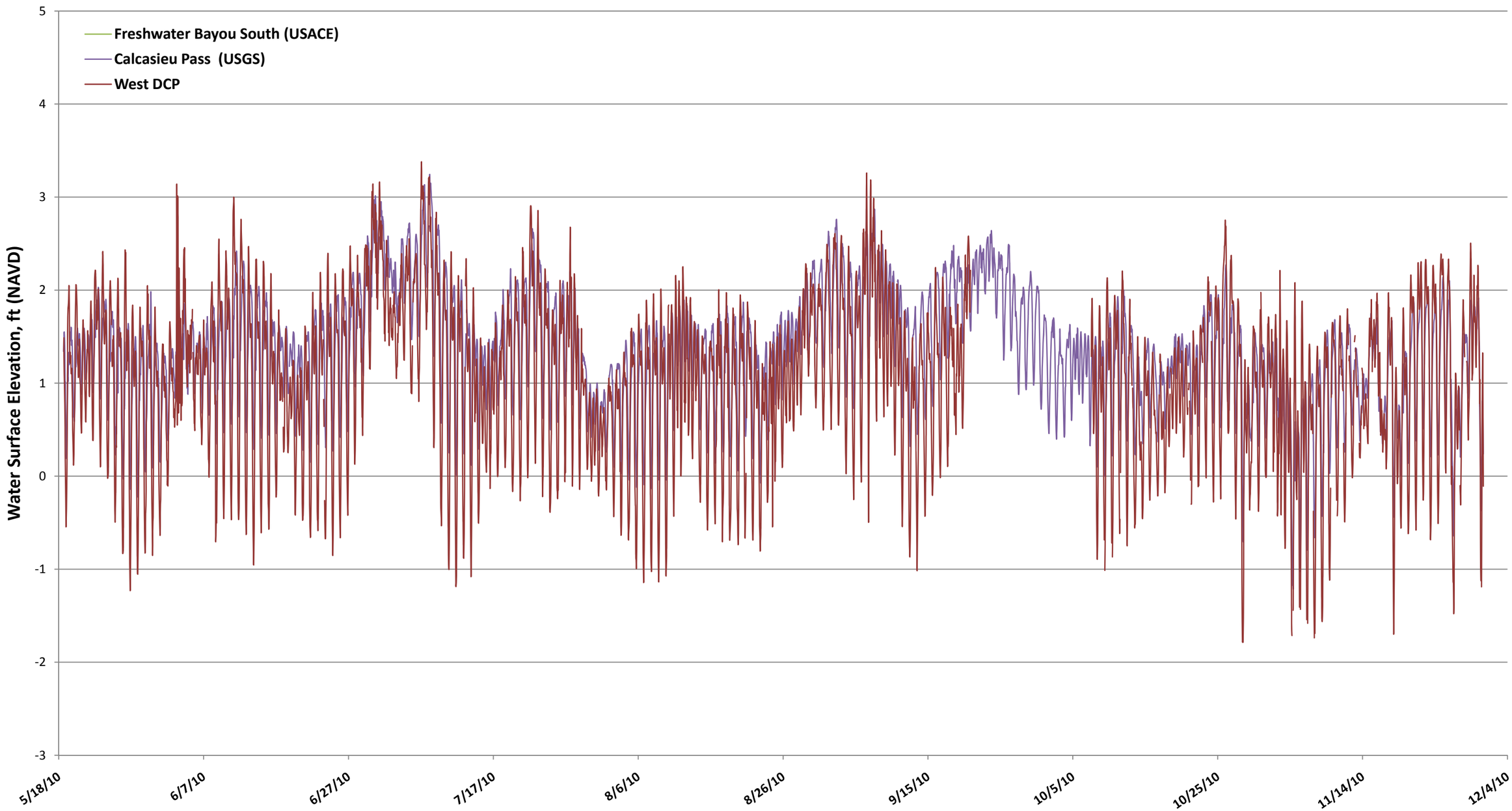

Figure B-2. Water level comparison - West DCP, Calcasieu Pass, and Freshwater Bayou.

APPENDIX C

Wave Transmission Tables

Table C-1. Average measured wave transmission coefficients.																
		Significant Wave Height, Hs (ft)														
		< 0.5			0.5 to 0.75			0.75 to 1			1 to 1.25			> 1.25		
Water Level (ft)	Wave Period, Tp (sec)	Control	Reef	LWA	Control	Reef	LWA	Control	Reef	LWA	Control	Reef	LWA	Control	Reef	LWA
< 0	< 3	0.95	0.10	0.02	0.81	0.02	0.01	--	--	--	0.66	0.28	0.03	--	--	--
	3 to 4.5	0.74	0.11	0.02	0.80	0.13	0.03	0.84	0.25	0.02	0.84	0.12	0.01	0.99	0.46	0.03
	4.5 to 6	0.74	0.14	0.02	0.68	0.11	0.02	0.83	0.07	0.01	0.89	0.14	0.01	0.77	0.37	0.03
	6 to 7.5	0.68	0.16	0.03	0.55	0.30	0.02	0.67	0.33	0.03	0.78	0.26	0.02	0.75	0.35	0.03
	> 7.5	0.66	0.18	0.04	0.55	0.33	0.03	0.58	0.41	0.04	0.75	0.36	0.02	0.72	0.36	0.03
0 to 1	< 3	0.76	0.17	0.02	0.65	0.13	0.04	0.56	0.17	0.02	--	--	--	--	--	--
	3 to 4.5	0.80	0.12	0.02	0.84	0.10	0.03	0.82	0.08	0.02	0.68	0.19	0.02	1.01	0.04	0.01
	4.5 to 6	0.80	0.12	0.03	0.82	0.09	0.02	0.74	0.09	0.01	0.98	0.12	0.02	1.16	0.15	0.03
	6 to 7.5	0.69	0.13	0.02	0.73	0.09	0.02	0.79	0.16	0.02	0.69	0.14	0.02	0.91	0.19	0.02
	> 7.5	0.72	0.18	0.03	0.68	0.11	0.02	0.67	0.16	0.02	0.53	0.17	0.02	0.89	0.21	0.02
1 to 2	< 3	0.88	0.34	0.04	0.86	0.35	0.03	0.31	0.22	0.02	0.51	0.17	0.01	--	--	--
	3 to 4.5	1.01	0.31	0.04	0.95	0.27	0.03	0.89	0.25	0.03	0.97	0.30	0.03	0.86	0.23	0.02
	4.5 to 6	0.82	0.30	0.04	0.88	0.25	0.03	0.97	0.25	0.03	1.02	0.29	0.02	1.06	0.27	0.03
	6 to 7.5	0.82	0.33	0.05	0.92	0.28	0.04	0.93	0.33	0.03	1.03	0.33	0.03	0.89	0.30	0.03
	> 7.5	1.22	0.42	0.06	0.92	0.36	0.03	0.84	0.36	0.03	0.84	0.42	0.04	0.97	0.40	0.03
2 to 3	< 3	0.48	0.38	0.04	0.67	0.45	0.04	0.58	0.44	0.03	--	--	--	1.18	0.34	0.03
	3 to 4.5	1.28	0.51	0.06	0.99	0.41	0.04	0.93	0.46	0.06	0.84	0.35	0.04	1.23	0.44	0.04
	4.5 to 6	0.74	0.40	0.05	0.73	0.41	0.05	0.86	0.44	0.05	0.80	0.42	0.04	1.05	0.40	0.04
	6 to 7.5	0.82	0.58	0.08	0.66	0.37	0.04	0.72	0.44	0.05	0.94	0.44	0.04	0.92	0.42	0.05
	> 7.5	1.04	0.53	0.07	0.80	0.50	0.05	0.95	0.55	0.06	0.72	0.43	0.04	0.97	0.47	0.08
> 3	3 to 4.5	--	--	--	--	--	--	0.50	0.43	0.14	--	--	--	--	--	--
	4.5 to 6	--	--	--	--	--	--	--	--	--	0.47	0.47	0.09	0.90	0.42	0.06
	6 to 7.5	--	--	--	--	--	--	--	--	--	--	--	--	0.87	0.56	0.12
	> 7.5	--	--	--	--	--	--	--	--	--	--	--	--	0.88	0.55	0.13

Table C-2. Empirically calculated wave transmission coefficients for Reef Breakwater (Based on Wamsley and Ahrens, 2003).						
		Significant Wave Height, Hs (ft)				
Water Level (ft)	Wave Period, Tp (sec)	0.4	0.625	0.875	1.125	1.35
-0.5	2	0.11	0.07	0.05	0.04	0.04
	3.75	0.56	0.45	0.37	0.32	0.28
	5.25	0.67	0.57	0.49	0.43	0.38
	6.75	0.71	0.61	0.53	0.47	0.43
	8	0.73	0.63	0.55	0.49	0.45
0.5	2	0.11	0.07	0.05	0.04	0.04
	3.75	0.56	0.45	0.37	0.34	0.31
	5.25	0.67	0.57	0.49	0.47	0.45
	6.75	0.71	0.61	0.53	0.54	0.52
	8	0.73	0.63	0.55	0.58	0.56
1.5	2	0.11	0.09	0.11	0.14	0.17
	3.75	0.56	0.51	0.47	0.46	0.46
	5.25	0.67	0.65	0.61	0.59	0.58
	6.75	0.71	0.7	0.67	0.65	0.64
	8	0.73	0.72	0.69	0.67	0.66
2.5	2	0.69	0.69	0.69	0.69	0.69
	3.75	0.85	0.8	0.77	0.74	0.73
	5.25	0.9	0.85	0.81	0.78	0.77
	6.75	0.89	0.85	0.82	0.79	0.77
	8	0.86	0.84	0.81	0.79	0.77
3.5	2	0.9	0.83	0.79	0.76	0.74
	3.75	1.06	0.94	0.87	0.82	0.79
	5.25	1.12	1	0.92	0.87	0.83
	6.75	1.14	1.03	0.95	0.89	0.85
	8	1.15	1.04	0.96	0.9	0.86

Table C-3. Empirically calculated wave transmission coefficients for Reef Breakwater with LWAC (Based on Wamsley and Ahrens, 2003).						
		Significant Wave Height, Hs (ft)				
Water Level (ft)	**Wave Period, Tp (sec)**	**0.4**	**0.625**	**0.875**	**1.125**	**1.35**
-0.5	**2**	0.09	0.06	0.04	0.03	0.03
	3.75	0.55	0.44	0.36	0.3	0.27
	5.25	0.66	0.56	0.48	0.42	0.37
	6.75	0.71	0.61	0.53	0.47	0.42
	8	0.73	0.63	0.55	0.49	0.45
0.5	**2**	0.09	0.06	0.04	0.03	0.03
	3.75	0.55	0.44	0.36	0.3	0.27
	5.25	0.66	0.56	0.48	0.42	0.37
	6.75	0.71	0.61	0.53	0.47	0.42
	8	0.73	0.63	0.55	0.49	0.45
1.5	**2**	0.09	0.06	0.04	0.03	0.03
	3.75	0.55	0.44	0.36	0.3	0.27
	5.25	0.66	0.56	0.48	0.42	0.37
	6.75	0.71	0.61	0.53	0.47	0.42
	8	0.73	0.63	0.55	0.49	0.45
2.5	**2**	0.09	0.06	0.04	0.03	0.03
	3.75	0.55	0.44	0.36	0.32	0.3
	5.25	0.66	0.56	0.48	0.46	0.44
	6.75	0.71	0.61	0.53	0.53	0.51
	8	0.73	0.63	0.55	0.56	0.55
3.5	**2**	0.09	0.08	0.1	0.13	0.16
	3.75	0.55	0.49	0.45	0.44	0.44
	5.25	0.66	0.63	0.59	0.57	0.56
	6.75	0.71	0.69	0.65	0.63	0.62
	8	0.73	0.7	0.67	0.65	0.64

Table C-4. Absolute difference between empirically calculated and measured wave transmission coefficients.											
		Significant Wave Height, Hs (ft)									
		< 0.5		0.5 to 0.75		0.75 to 1		1 to 1.25		> 1.25	
Water Level (ft)	Wave Period, Tp (sec)	Reef	LWA	Reef	LWA	Reef	LWA	Reef	LWA	Reef	LWA
< 0	< 3	0.01	0.07	0.05	0.05	0.05	0.04	0.24	0.00	--	--
	3 to 4.5	0.45	0.53	0.32	0.41	0.12	0.34	0.20	0.29	0.18	0.24
	4.5 to 6	0.53	0.64	0.46	0.54	0.42	0.47	0.29	0.41	0.01	0.34
	6 to 7.5	0.55	0.68	0.31	0.59	0.20	0.50	0.21	0.45	0.08	0.39
	> 7.5	0.55	0.69	0.30	0.60	0.14	0.51	0.13	0.47	0.09	0.42
0 to 1	< 3	0.06	0.07	0.06	0.02	0.12	0.02	--	--	--	--
	3 to 4.5	0.44	0.53	0.35	0.41	0.29	0.34	0.15	0.28	0.27	0.26
	4.5 to 6	0.55	0.63	0.48	0.54	0.40	0.47	0.35	0.40	0.30	0.34
	6 to 7.5	0.58	0.69	0.52	0.59	0.37	0.51	0.40	0.45	0.33	0.40
	> 7.5	0.55	0.70	0.52	0.61	0.39	0.53	0.41	0.47	0.35	0.43
1 to 2	< 3	0.23	0.05	0.26	0.03	0.11	0.02	0.03	0.02	--	--
	3 to 4.5	0.25	0.51	0.24	0.41	0.22	0.33	0.16	0.27	0.23	0.25
	4.5 to 6	0.37	0.62	0.40	0.53	0.36	0.45	0.30	0.40	0.31	0.34
	6 to 7.5	0.38	0.66	0.42	0.57	0.34	0.50	0.32	0.44	0.34	0.39
	> 7.5	0.31	0.67	0.36	0.60	0.33	0.52	0.25	0.45	0.26	0.42
2 to 3	< 3	0.31	0.05	0.24	0.02	0.25	0.01	--	--	0.35	0.00
	3 to 4.5	0.34	0.49	0.39	0.40	0.31	0.30	0.39	0.28	0.29	0.26
	4.5 to 6	0.50	0.61	0.44	0.51	0.37	0.43	0.36	0.42	0.37	0.40
	6 to 7.5	0.31	0.63	0.48	0.57	0.38	0.48	0.35	0.49	0.35	0.46
	> 7.5	0.33	0.66	0.34	0.58	0.26	0.49	0.36	0.52	0.30	0.47
> 3	3 to 4.5	--	--	--	--	0.36	0.04	--	--	--	--
	4.5 to 6	--	--	--	--	--	--	0.35	0.35	0.37	0.38
	6 to 7.5	--	--	--	--	--	--	--	--	0.41	0.50
	> 7.5	--	--	--	--	--	--	--	--	0.43	0.56

APPENDIX D

Aerial Photography

Control Site
July 2010 Aerial Photograph

1
- D1

SCALE: **0** **100** **200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

		DATE
DRW	EAR	
DES	EAR	
CHK	BLG	
DWG SCALE		
DWG No.		
TASK No.	163377(PW69815)	

ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION
DEMONSTRATION PROJECT
(ME-18)

HDR
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Control Site
July 2010 Aerial Photograph

SHEET
D1
OF **9** REV **A**

Beach Fill and Reef Breakwater
July 2010 Aerial Photograph

1 - D2

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRW EAR	DATE	
CHK EAR	DATE	
APP BLG	DATE	
CADD SCALE		
MWH No.		
PROJ 163377(PW69815)		

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Beach Fill and Reef Breakwater
July 2010 Aerial Photograph

SHEET
D2
OF **9** REV **A**

Reef Breakwater with LWA Core
July 2010 Aerial Photograph

1 - D3

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRN	EAR	DATE
DES	EAR	DATE
CHK	BLG	DATE
DWG SCALE		
DWG No.		
JOB No.	163377(PW69815)	

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Reef Breakwater with LWA Core
July 2010 Aerial Photograph

SHEET
D3
OF **9** REV **A**

1
- D4

Control Site
November 2010 Aerial Photograph

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No
PW No 163377(PW69815)

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Control Site
November 2010 Aerial Photograph

SHEET
D4
OF **9** REV **A**

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

		DATE
DWN	EAR	
DES	EAR	
CHK	BLG	
DWG No.		
SCALE		
PROJ No.	163377(PW69815)	

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Beach Fill and Reef Breakwater
November 2010 Aerial Photograph

1
D5

Beach Fill and Reef Breakwater
November 2010 Aerial Photograph

SCALE: **0 100 200**
Feet

SHEET
D5
OF **9** REV **A**

1
D6

Reef Breakwater with LWA Core
November 2010 Aerial Photograph

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRAWN	EAR	DATE
DES'D	EAR	DATE
CHK'D	BLG	DATE
SCALE		
PROJ. No.		
DWG. No.	163377(PW69815)	

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Reef Breakwater with LWA Core
November 2010 Aerial Photograph

SHEET
D6
OF **9** REV **A**

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

		DATE
DWN	EAR	
DES	EAR	
CHK	BLG	
SCALE		
DWG No.	163377(PW69815)	

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Control Site
April 2011 Aerial Photograph

1 - D7

Control Site
April 2011 Aerial Photograph

SCALE: **0 100 200**
Feet

SHEET
D7
OF **9** REV **A**

Beach Fill and Reef Breakwater
April 2011 Aerial Photograph

1 - D8

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRAWN	EAR	DATE
DES.	EAR	DATE
CHK.	BLG	DATE
SCALE		
WMP No.		
PROJ. No.	163377(PW69815)	

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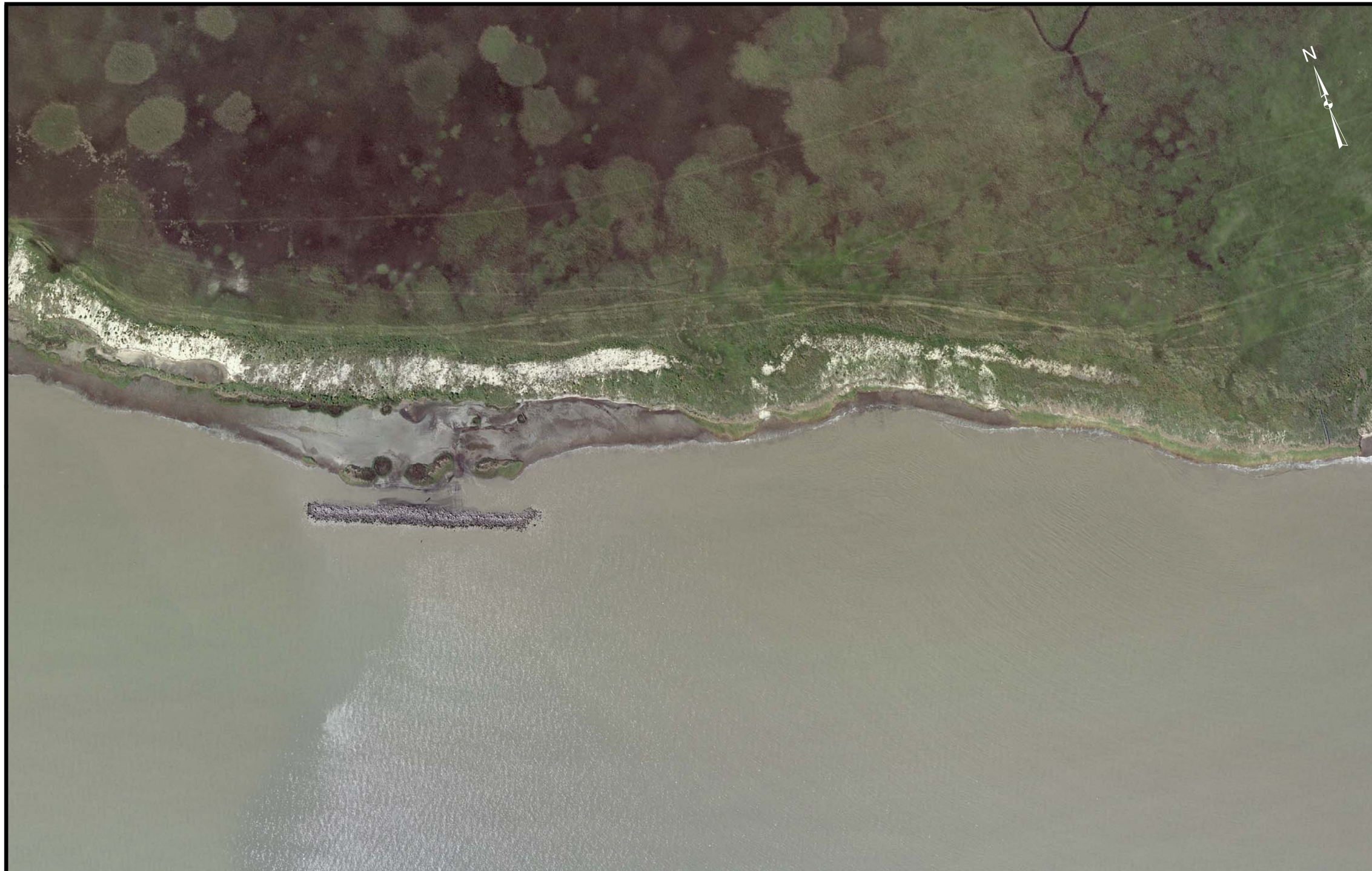
Beach Fill and Reef Breakwater
April 2011 Aerial Photograph

SHEET
D8
OF **9** REV **A**



Reef Breakwater with LWA Core
April 2011 Aerial Photograph

1 - D9

SCALE: **0 100 200**
Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DSN	EAR	DATE
DWN	EAR	DATE
CHK	BLG	DATE
SCALE		
DWG No.	163377(PW69815)	

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Reef Breakwater with LWA Core
April 2011 Aerial Photograph

SHEET
D9
OF **9** REV **A**

APPENDIX E

Shoreline Change

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No. 163377(PW69815)

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Control Site
Shoreline Change Between February 2010 and August 2010

1
- E1

Control Site
Shoreline Change Between February 2010 and August 2010

SCALE: **0 100 200**
Feet

SHEET
E1
OF **9** REV **A**

Beach Fill and Reef Breakwater
Shoreline Change Between February 2010 and August 2010

1 - E2

SCALE: 0 100 200 Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRWN EAR DATE
DES EAR DATE
CHK BLG DATE
SCALE
DWG No.
PW No. 163377(PW69815)

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Beach Fill and Reef Breakwater
Shoreline Change Between
February 2010 and August 2010

SHEET
E2
OF 9 REV A

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No.
PROJ No. 163377(PW69815)

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Reef Breakwater with LWA Core
Shoreline Change Between
February 2010 and August 2010

1
- E3

Reef Breakwater with LWA Core
Shoreline Change Between February 2010 and August 2010

SCALE: **0 100 200**
Feet

SHEET
E3
OF **9** REV **A**

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRAWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No.
TASK 163377(PW69815)

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Control Site
Shoreline Change Between
February 2010 and November 2010

1 - E4

Control Site
Shoreline Change Between February 2010 and November 2010

SCALE: **0 100 200**
Feet

SHEET
E4
OF **9** REV **A**

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No.
PROJ 163377(PW69815)

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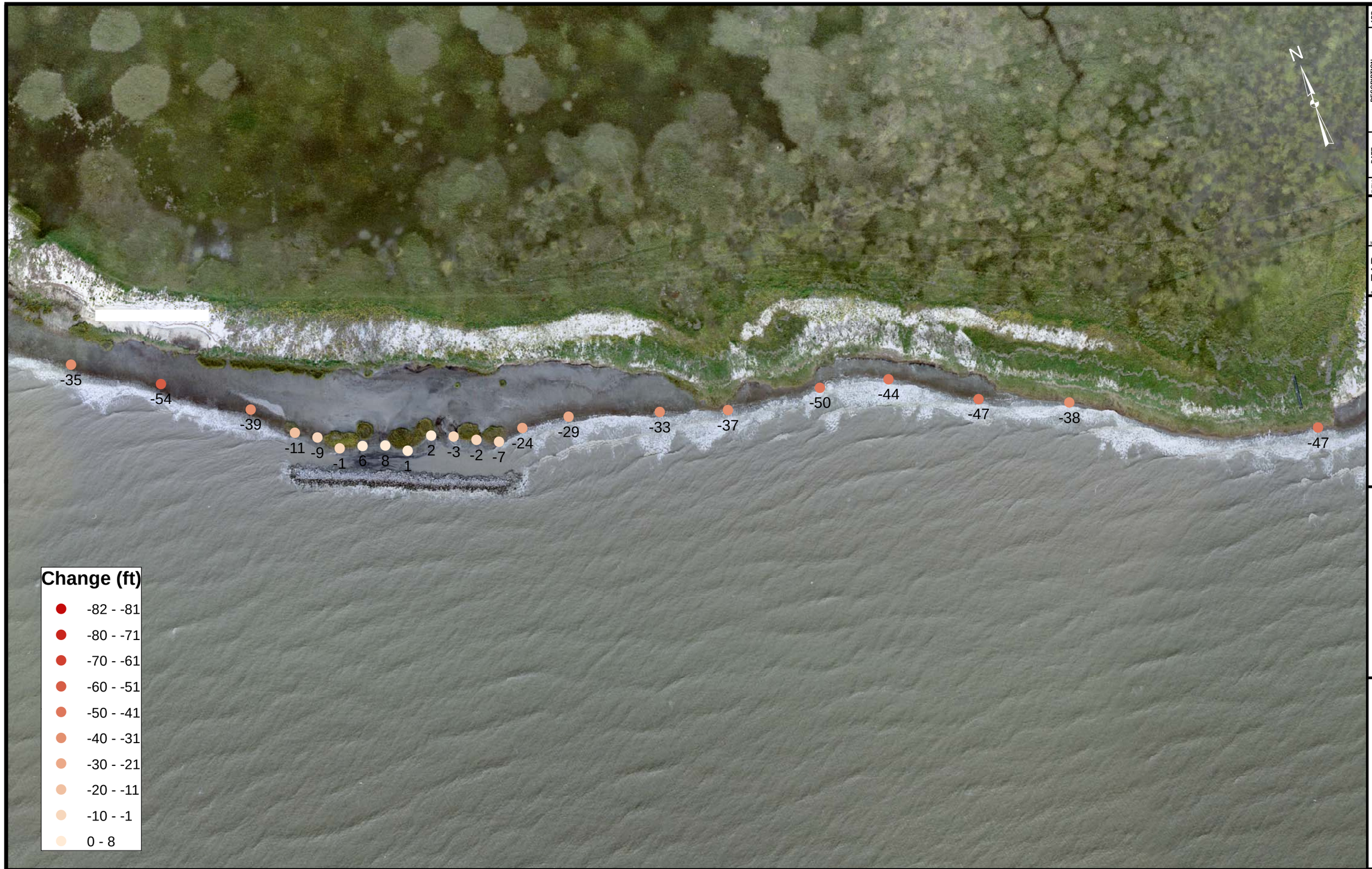
Beach Fill and Reef Breakwater
Shoreline Change Between
February 2010 and November 2010

1
- E5

Beach Fill and Reef Breakwater
Shoreline Change Between February 2010 and November 2010

SCALE: **0 100 200**
Feet

SHEET
E5
OF **9** REV **A**



REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRW	EAR	DATE
DES	EAR	DATE
CHK	BLG	DATE
LOAD SCALE		
LOAD WMS No.		
PROJ No.	163377(PW69815)	

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Reef Breakwater with LWA Core
Shoreline Change Between
February 2010 and November 2010

1
- E6

Reef Breakwater with LWA Core
Shoreline Change Between February 2010 and November 2010

SCALE: **0 100 200**
Feet

SHEET
E6
OF **9** REV **A**

Control Site
Shoreline Change Between February 2010 and March 2011

1 - E7

SCALE: 0 100 200 Feet

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRAWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No. 163377(PW69815)

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Control Site
Shoreline Change Between
February 2010 and March 2011

SHEET
E7
OF 9 REV A

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRWN EAR DATE
DES EAR DATE
CHK BLG DATE
DWG SCALE
DWG No.
PROJ No. 163377(PW69815)

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Beach Fill and Reef Breakwater
Shoreline Change Between
February 2010 and March 2011

1 - E8

Beach Fill and Reef Breakwater
Shoreline Change Between February 2010 and March 2011

SCALE: **0 100 200**
Feet

SHEET
E8
OF **9** REV **A**

REV	DATE	DESCRIPTION	APP
A	6/11	Monitoring Report	

DRWN	EAR	DATE
DES	EAR	DATE
CHK	BLG	DATE
DWG SCALE		
DWG No.		
PROJ No.	163377(PW69815)	

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DEMONSTRATION PROJECT
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Reef Breakwater with LWA Core
Shoreline Change Between February 2010 and March 2011

1
- E9

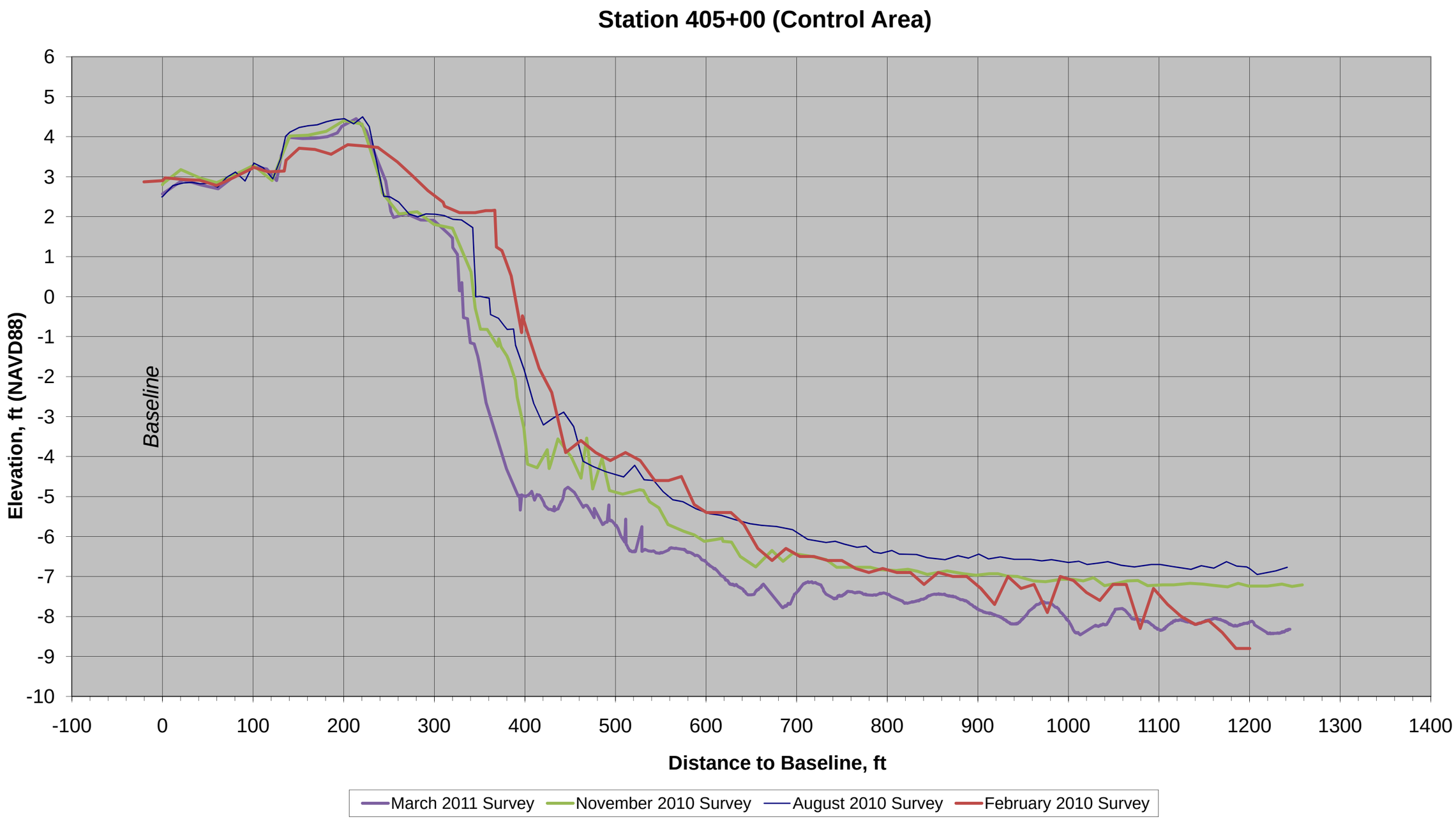
Reef Breakwater with LWA Core
Shoreline Change Between February 2010 and March 2011

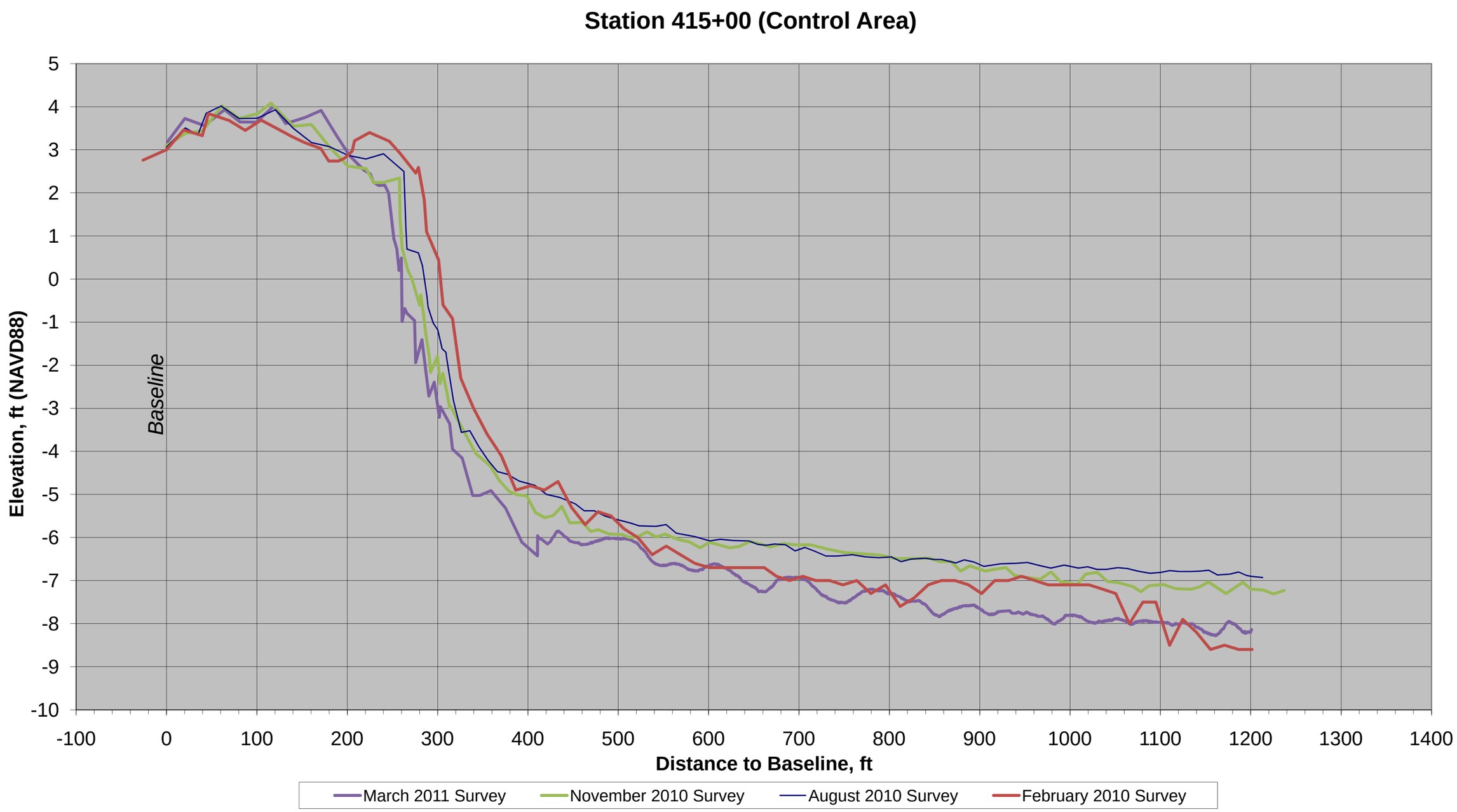
SCALE: **0** **100** **200**
Feet

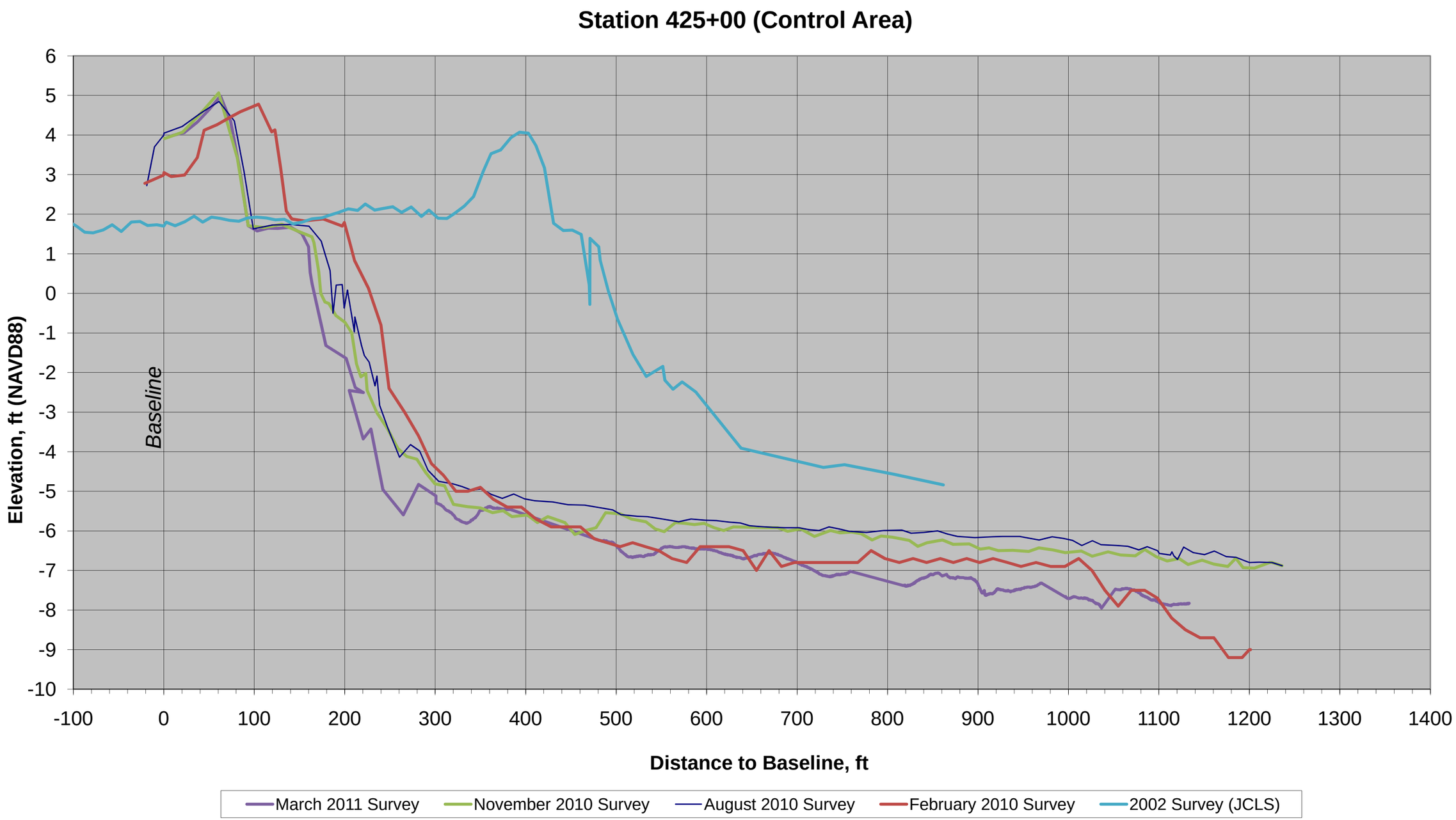
SHEET
E9
OF **9** REV **A**

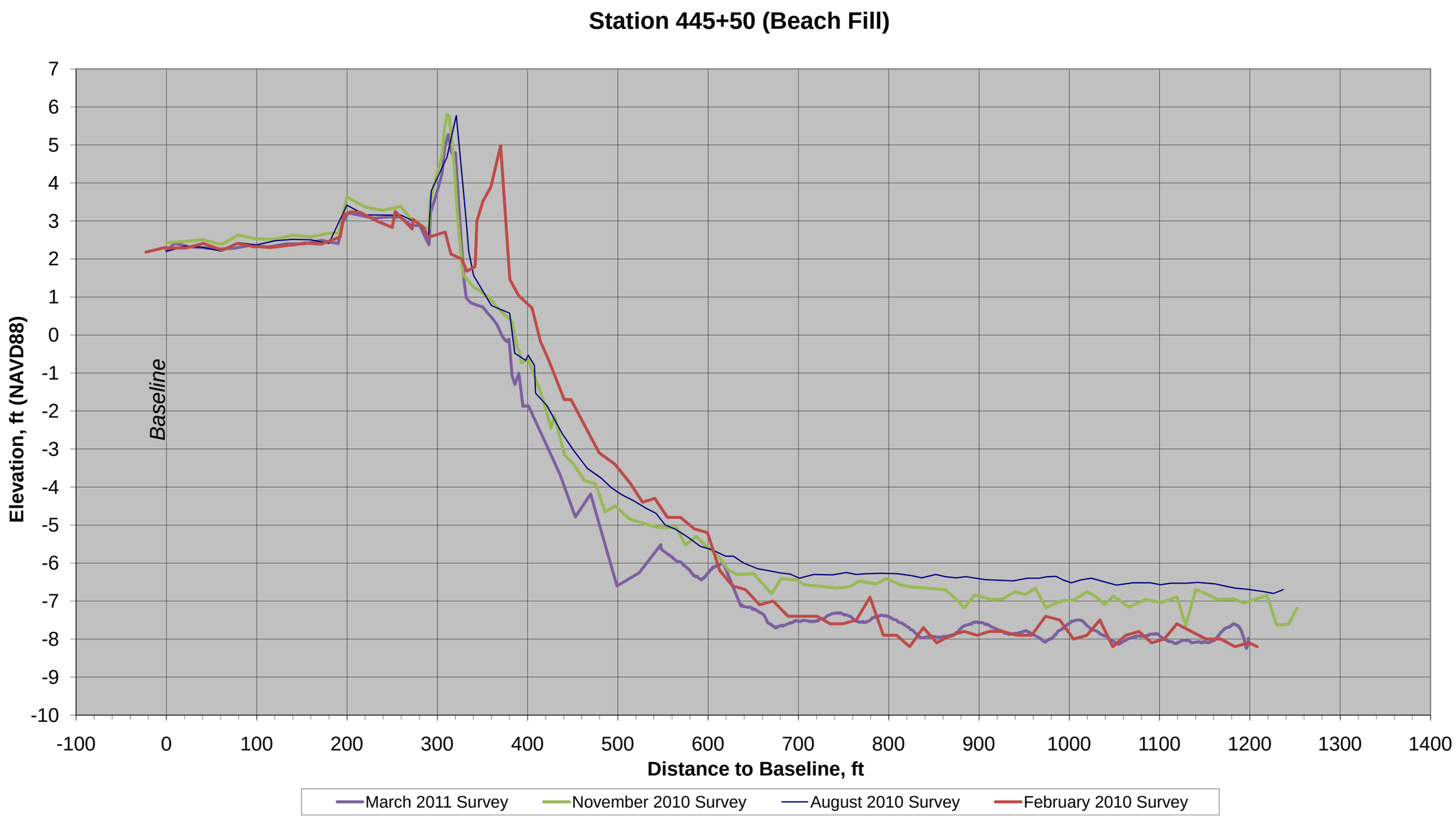
APPENDIX F

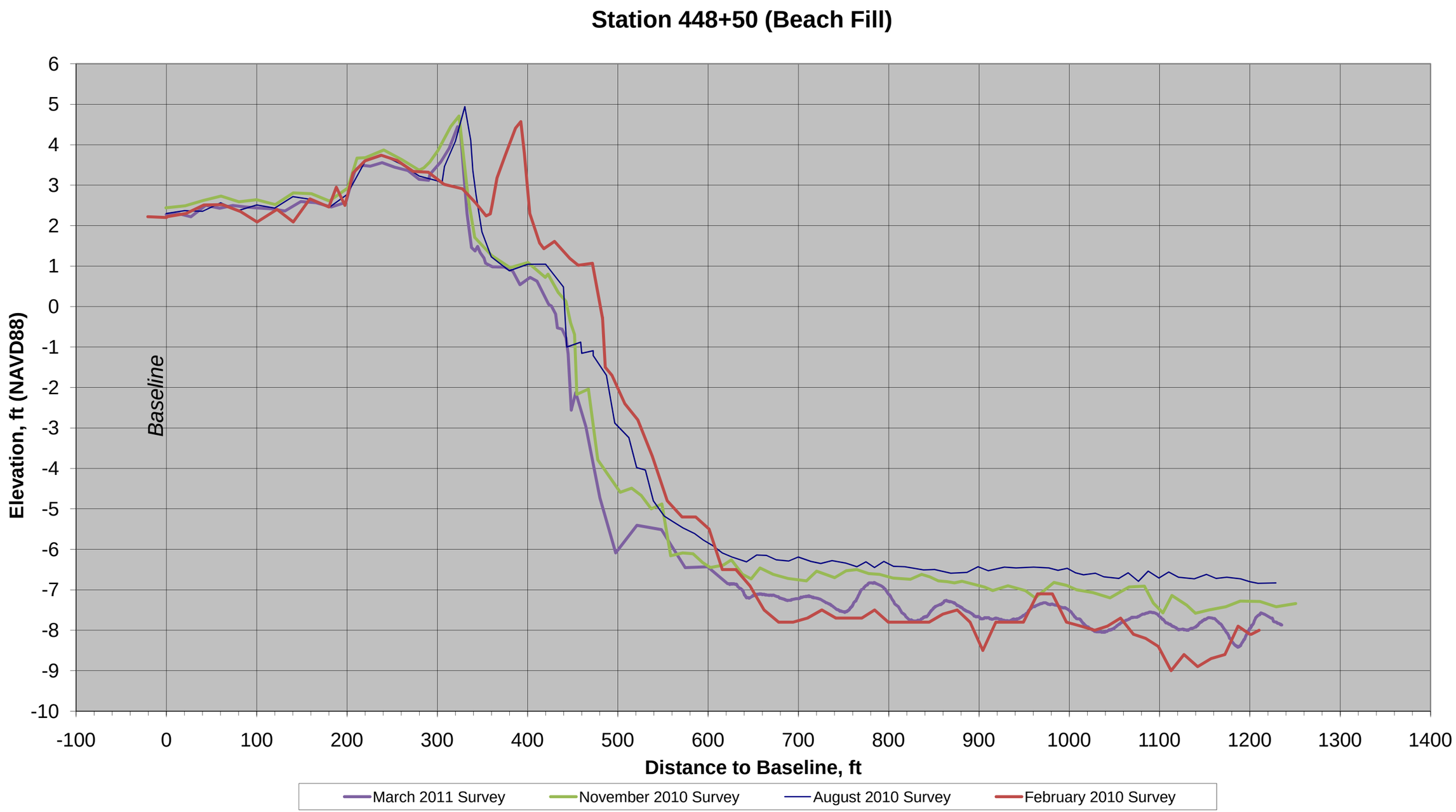
Cross-Shore Profiles

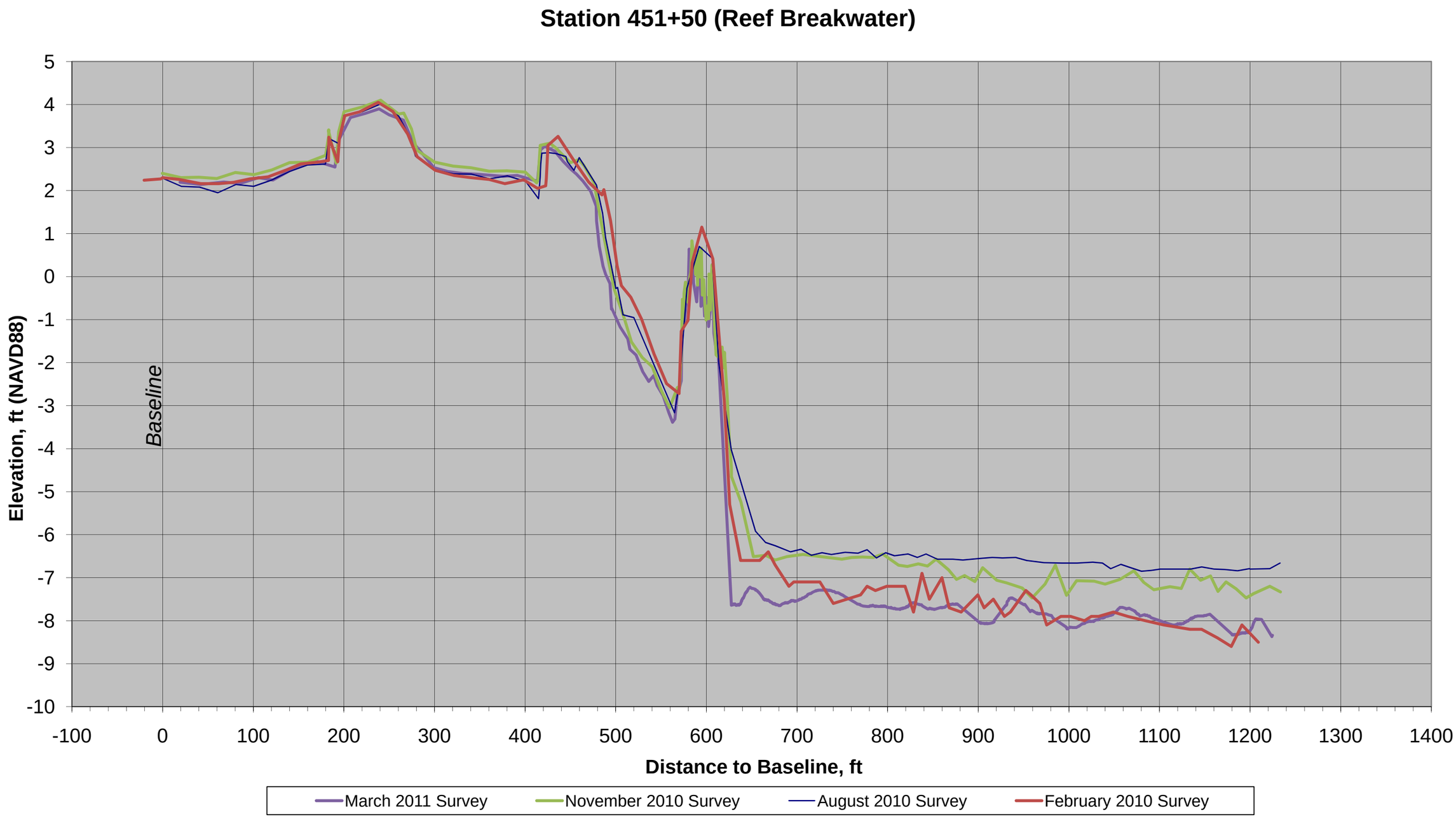

Station 405+00 (Control Area)
Elevation, ft (NAVD88)
Distance to Baseline, ft
Baseline
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey

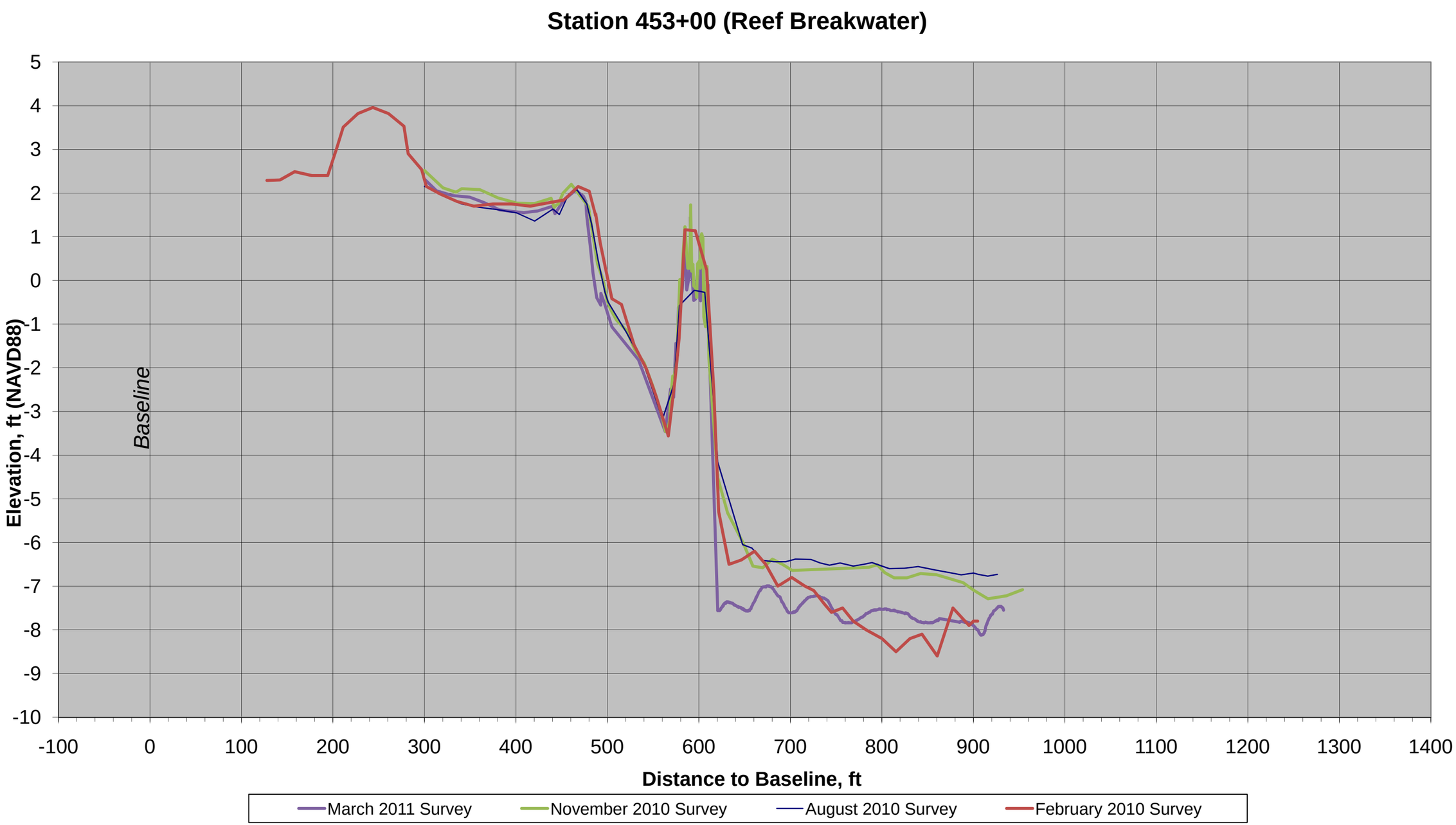

Station 415+00 (Control Area)
Elevation, ft (NAVD88)
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey

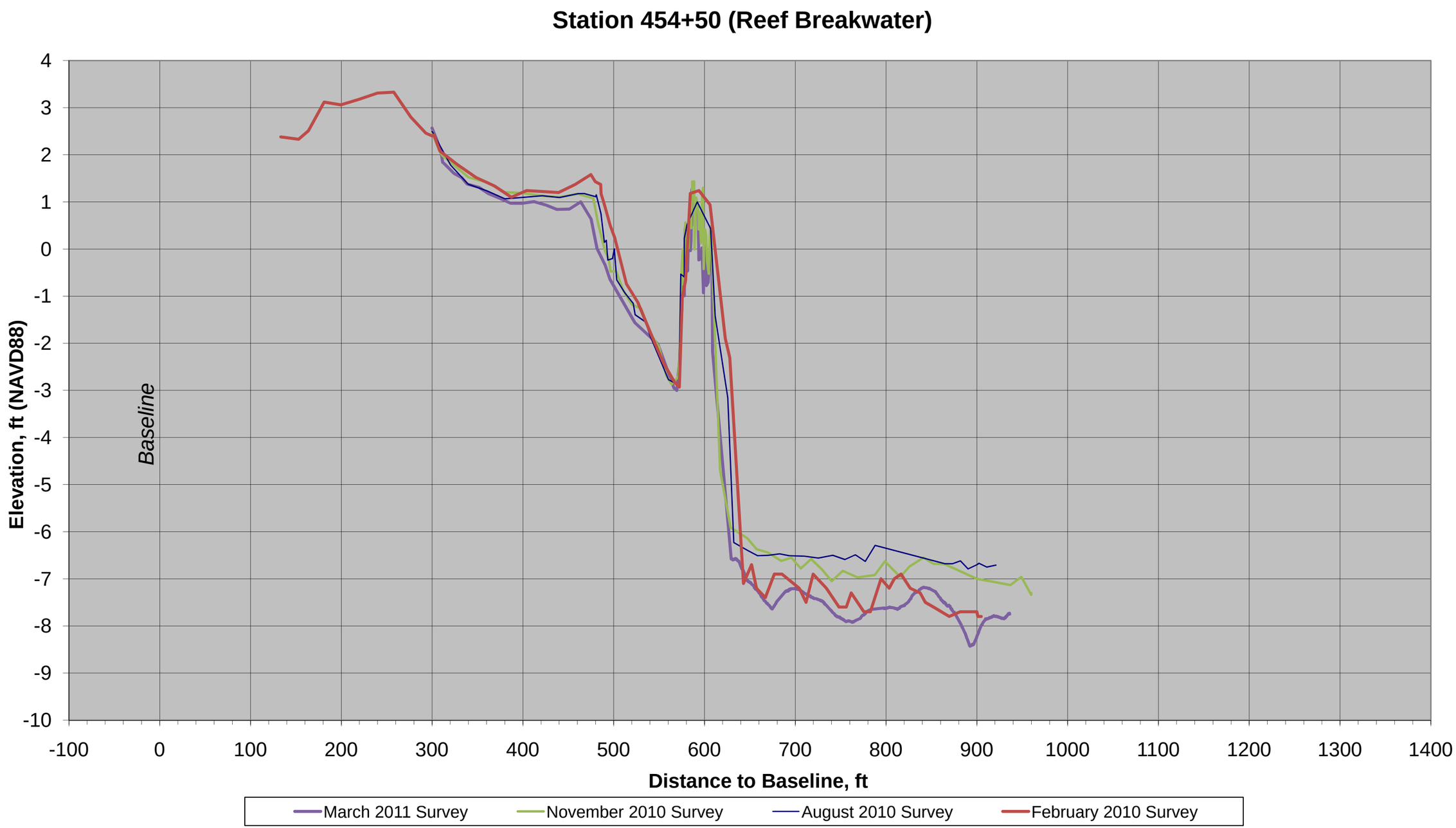

Station 425+00 (Control Area)
Elevation, ft (NAVD88)
Distance to Baseline, ft
Baseline
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey
2002 Survey (JCLS)

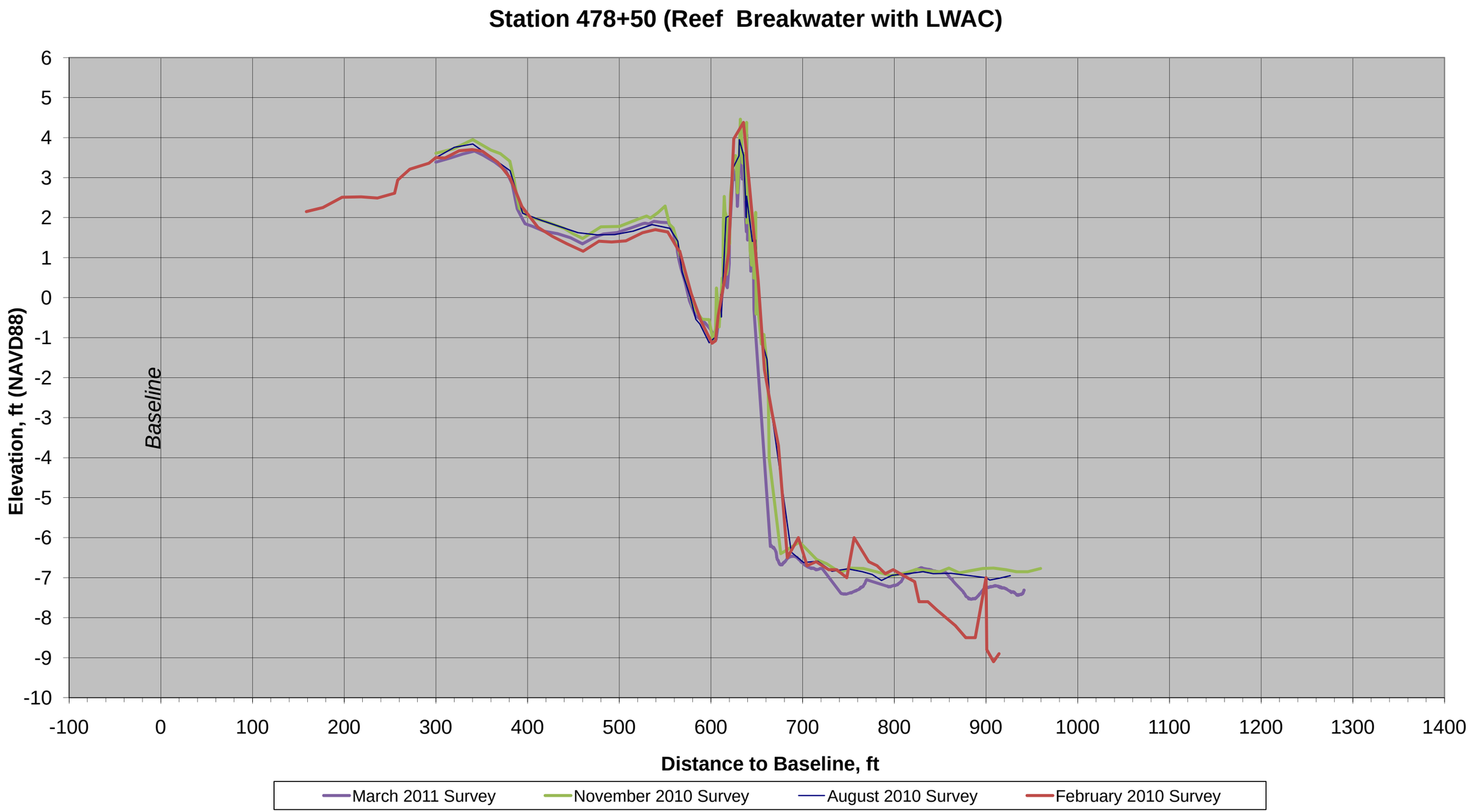

Station 445+50 (Beach Fill)
Elevation, ft (NAVD88)
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey

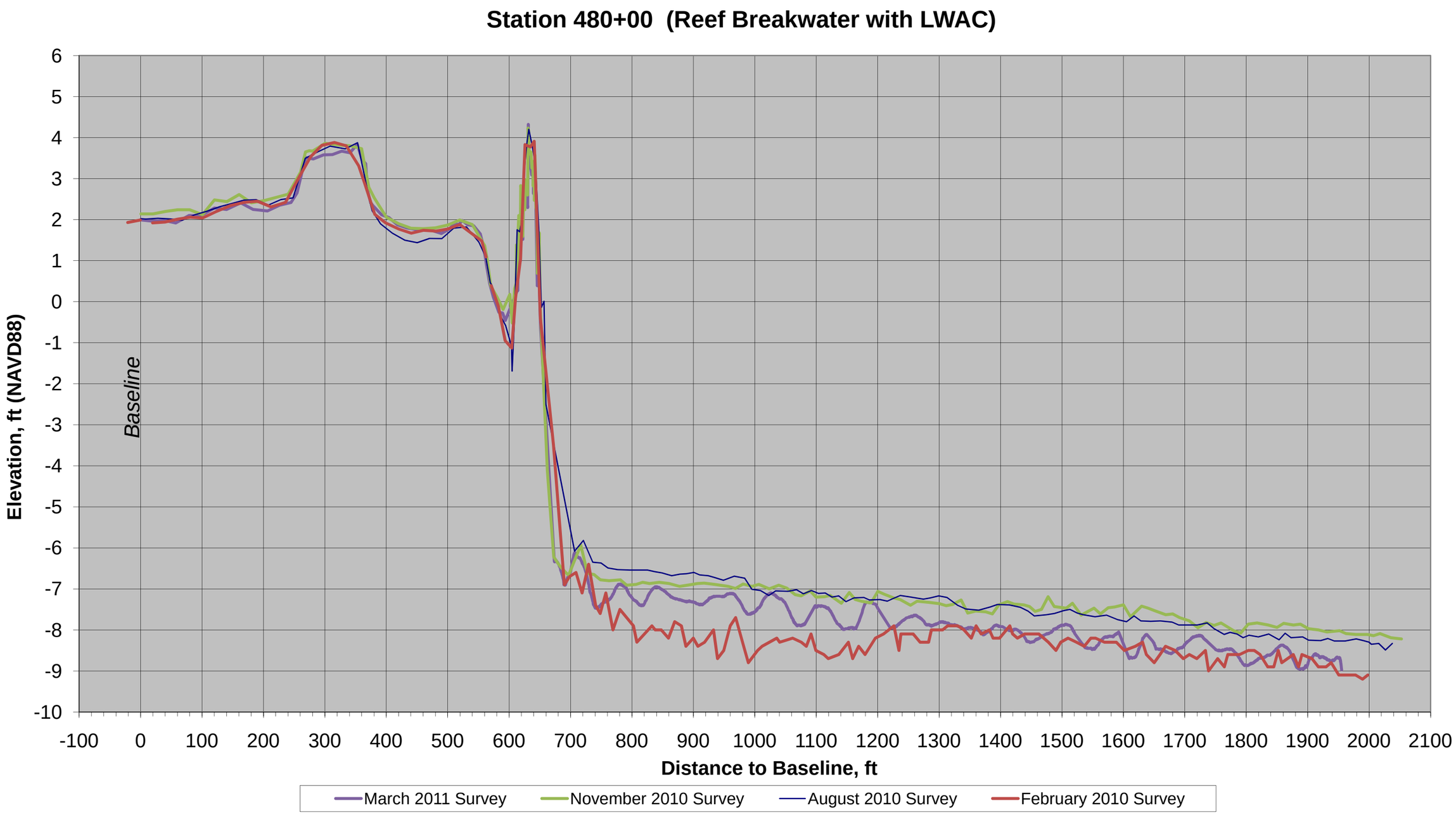

Station 448+50 (Beach Fill)
Elevation, ft (NAVD88)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey

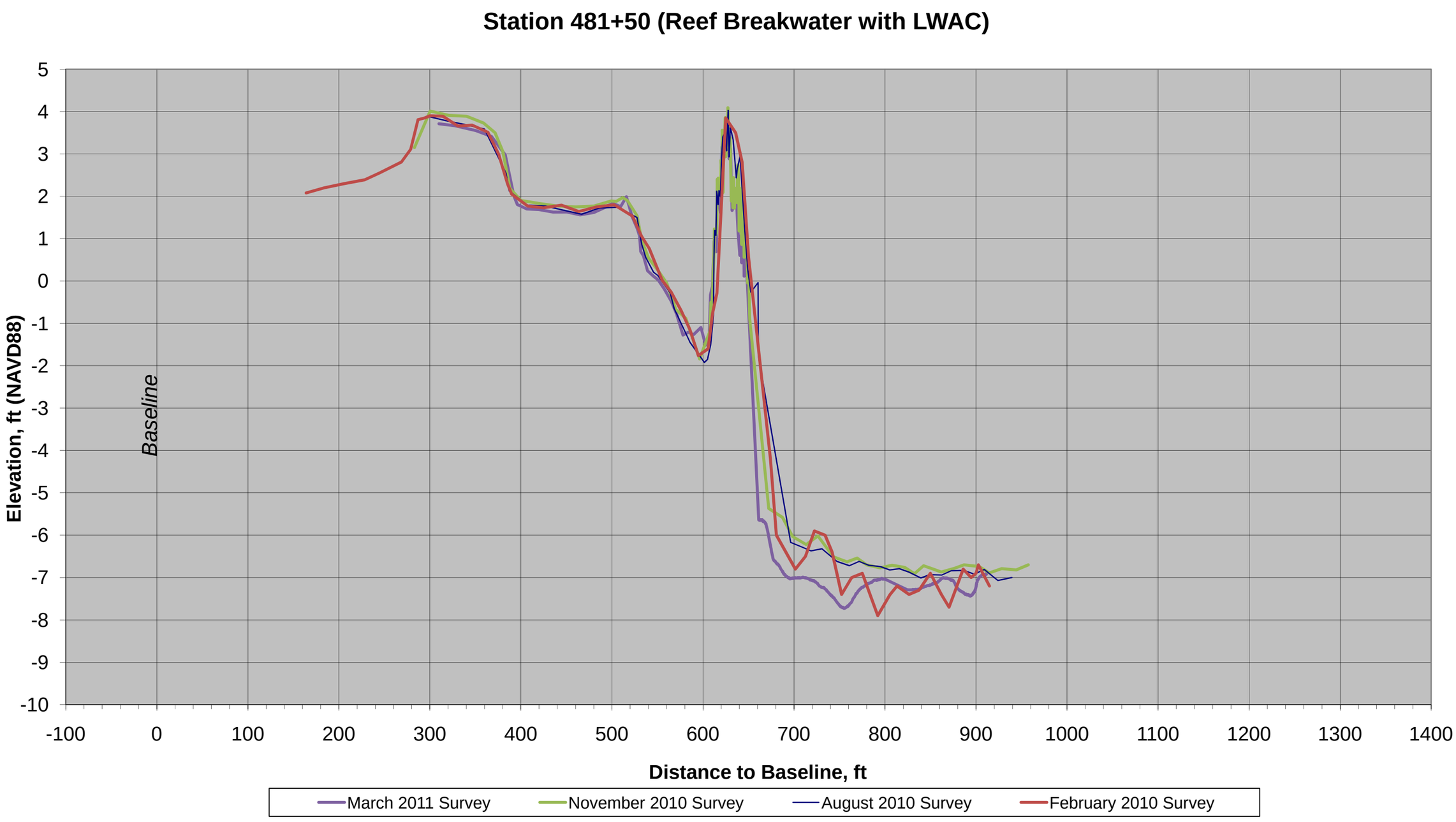

Station 451+50 (Reef Breakwater)
Elevation, ft (NAVD88)
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey

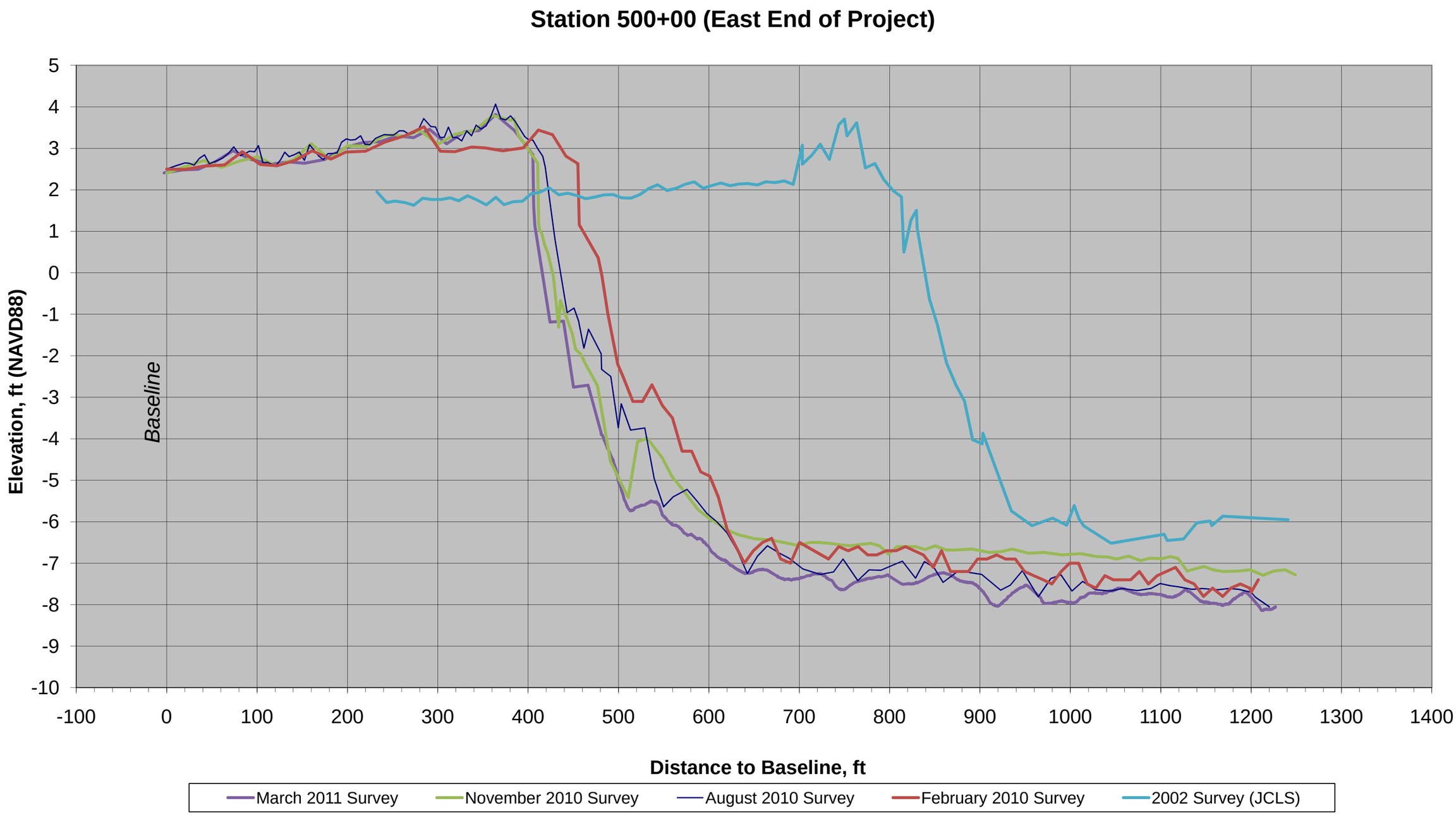

Station 453+00 (Reef Breakwater)
Elevation, ft (NAVD88)
Distance to Baseline, ft
Baseline
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey


Station 454+50 (Reef Breakwater)
Elevation, ft (NAVD88)
Distance to Baseline, ft
Baseline
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey


Station 478+50 (Reef Breakwater with LWAC)
Elevation, ft (NAVD88)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey


Station 480+00 (Reef Breakwater with LWAC)
Elevation, ft (NAVD88)
Distance to Baseline, ft
Baseline
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey


Station 481+50 (Reef Breakwater with LWAC)
Elevation, ft (NAVD88)
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey


Station 500+00 (East End of Project)
Elevation, ft (NAVD88)
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
Baseline
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Distance to Baseline, ft
March 2011 Survey
November 2010 Survey
August 2010 Survey
February 2010 Survey
2002 Survey (JCLS)

APPENDIX G

Opinion of Probable Construction Cost
(Full Project)

STATE OF LOUISIANA
COASTAL PROTECTION AND RESTORATION AUTHORITY
ROCKEFELLER REFUGE GULF SHORELINE STABILIZATION DEMONSTRATION PROJECT (ME-18)
CONCEPTUAL-LEVEL OPINION OF PROBABLE CONSTRUCTION COST

Reef Breakwater w/ LWA Core
Estimate A: Existing Design, Full 9.2 Miles

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
GENERAL				
Mobilization / Demobilization	1	LS	$ 2,000,000	$ 2,000,000
Lights / Daybeacons (Warning Signs)	33	EA	$ 5,000	$ 165,000
Topo / Hydro Surveying	1	LS	$ 500,000	$ 500,000
Hazard Survey	1	LS	$ 100,000	$ 100,000
			CONTINGENCIES (25%):	$ 691,000
			SUBTOTAL:	**$ 3,456,000**
REEF BREAKWATER WITH LWA CORE				
Armor Stone	665,700	TON	$ 80	$ 53,256,000
Bedding Layer	176,300	TON	$ 60	$ 10,578,000
LWA Core	169,800	CY	$ 90	$ 15,282,000
Settlement Plates	49	EA	$ 3,000	$ 147,000
Grid Geocomposite	398,900	SY	$ 10	$ 3,989,000
			CONTINGENCIES (25%):	$ 20,813,000
			SUBTOTAL:	**$ 104,065,000**
			TOTAL:	**$ 107,521,000**
			TOTAL WITH 15% CONTINGENCY[(1)]:	**$ 98,919,550**

Notes:

(1) Based on the significant level of design that has been completed to date for this project and information obtained from the demonstration and monitoring phase, it is recommended that a 15% contingency be considered (instead of 25%) for project budgeting purposes.
(2) Quantities shown include the additional material associated with construction settlements and are based on a typical section overlaid on existing grade.
(3) This preliminary-level opinion of probable cost is based on data available at the date of publication and is not necessarily all inclusive. Actual construction costs may vary based on changes during final detailed design and market conditions at time of bidding.
(4) Quantities used in the preliminary-level opinion of probable cost are conceptual. Further refinement of the cross section design may reduce costs.
(5) It is assumed that there will be no flotation channel used in the construction of the project, based on the demostration phase.
(6) Lights / Daybeacons (Warning Signs) are assumed every 1500 feet.
(7) Settlement Plates are assumed every 1000 feet.
(8) Costs for project engineering, regulatory coordination, and overall administration are not included in this estimate.

STATE OF LOUISIANA
COASTAL PROTECTION AND RESTORATION AUTHORITY
ROCKEFELLER REFUGE GULF SHORELINE STABILIZATION DEMONSTRATION PROJECT (ME-18)
CONCEPTUAL-LEVEL OPINION OF PROBABLE CONSTRUCTION COST

Reef Breakwater w/ LWA Core

Estimate B: Theoretical Design Refinements, Full 9.2 Miles

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
GENERAL				
Mobilization / Demobilization	1	LS	$ 2,000,000	$ 2,000,000
Lights / Daybeacons (Warning Signs)	33	EA	$ 5,000	$ 165,000
Topo / Hydro Surveying	1	LS	$ 500,000	$ 500,000
Hazard Survey	1	LS	$ 100,000	$ 100,000
			CONTINGENCIES (25%):	$ 691,000
			SUBTOTAL:	**$ 3,456,000**
REEF BREAKWATER WITH LWA CORE				
Armor Stone	665,700	TON	$ 80	$ 53,256,000
Bedding Layer	176,300	TON	$ 60	$ 10,578,000
LWA Core	169,800	CY	$ 90	$ 15,282,000
Settlement Plates	49	EA	$ 3,000	$ 147,000
Grid Geocomposite	398,900	SY	$ 10	$ 3,989,000
Allowance for Design Refinement	1	LS	$ (10,000,000)	$ (10,000,000)
			CONTINGENCIES (25%):	$ 18,313,000
			SUBTOTAL:	**$ 91,565,000**
			TOTAL:	**$ 95,021,000**

TOTAL WITH 15% CONTINGENCY[(1)]: $ 87,419,550

Notes:

(1) Based on the significant level of design that has been completed to date for this project and information obtained from the demonstration and monitoring phase, it is recommended that a 15% contingency be considered (instead of 25%) for project budgeting purposes.
(2) Quantities shown include the additional material associated with construction settlements and are based on a typical section overlaid on existing grade.
(3) This preliminary-level opinion of probable cost is based on data available at the date of publication and is not necessarily all inclusive. Actual construction costs may vary based on changes during final detailed design and market conditions at time of bidding.
(4) Quantities used in the preliminary-level opinion of probable cost are conceptual.
(5) Estimate B (shown on this sheet) reflects potential cost savings through refinement of the current cross-section design based on construction and monitoring of the demonstration project. The cost shown is based on several possible design refinements including reducing landward toe apron, reducing crest width, and/or reducing crest height. Actual design refinements, and associated savings, should be based on detailed design of the full project.
(6) It is assumed that there will be no flotation channel used in the construction of the project, based on the demostration phase.
(7) Lights / Daybeacons (Warning Signs) are assumed every 1500 feet.
(8) Settlement Plates are assumed every 1000 feet.
(9) Costs for project engineering, regulatory coordination, and overall administration are not included in this estimate.

STATE OF LOUISIANA
COASTAL PROTECTION AND RESTORATION AUTHORITY
ROCKEFELLER REFUGE GULF SHORELINE STABILIZATION DEMONSTRATION PROJECT (ME-18)
CONCEPTUAL-LEVEL OPINION OF PROBABLE CONSTRUCTION COST

Reef Breakwater w/ LWA Core
Estimate C: Existing Design, $50M Budget

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
GENERAL				
Mobilization / Demobilization	1	LS	$ 1,500,000	$ 1,500,000
Lights / Daybeacons (Warning Signs)	15	EA	$ 5,000	$ 75,000
Topo / Hydro Surveying	1	LS	$ 250,000	$ 250,000
Hazard Survey	1	LS	$ 75,000	$ 75,000
			CONTINGENCIES (25%):	$ 475,000
			SUBTOTAL:	**$ 2,375,000**
REEF BREAKWATER WITH LWA CORE				
Armor Stone	303,907	TON	$ 80	$ 24,313,000
Bedding Layer	80,485	TON	$ 60	$ 4,829,000
LWA Core	77,517	CY	$ 90	$ 6,977,000
Settlement Plates	22	EA	$ 3,000	$ 67,000
Grid Geocomposite	182,107	SY	$ 10	$ 1,821,000
			CONTINGENCIES (25%):	$ 9,502,000
			SUBTOTAL:	**$ 47,509,000**
			TOTAL:	**$ 49,884,000**
			Project Length (miles):	**4.2**
			Project Length (miles) with 15% Contingency[(1)]:	**4.7**

Notes:

(1) Based on the significant level of design that has been completed to date for this project and information obtained from the demonstration and monitoring phase, it is recommended that a 15% contingency be considered (instead of 25%) for project budgeting purposes.
(2) Quantities shown include the additional material associated with construction settlements and are based on a typical section overlaid on existing grade.
(3) This preliminary-level opinion of probable cost is based on data available at the date of publication and is not necessarily all inclusive. Actual construction costs may vary based on changes during final detailed design and market conditions at time of bidding.
(4) Quantities used in the preliminary-level opinion of probable cost are conceptual. Further refinement of the cross section design may reduce costs.
(5) It is assumed that there will be no flotation channel used in the construction of the project, based on the demostration phase.
(6) Lights / Daybeacons (Warning Signs) are assumed every 1500 feet.
(7) Settlement Plates are assumed every 1000 feet.
(8) Costs for project engineering, regulatory coordination, and overall administration are not included in this estimate.