US Army Corps of Engineers®
Jacksonville District

SAN JUAN METRO AREA, PUERTO RICO

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

JULY 2020

APPENDIX A: ENGINEERING

This page was intentionally left blank.

Table of Contents

Table of Figures

Table of Tables

List of Abbreviations

ADCIRC	Advanced Circulation Model	NOAA	National Oceanic And Atmospheric Administration
AEP	Annual Exceedance Probability	NOS	National Ocean Service
ANOVA	Analysis Of Variance	NPCC	New York City Panel On Climate Change
BCR	Benefit-To-Cost Ratio	NRC	National Research Council
BPE	Bank Protection Easement	NRCS	Natural Resources Conservation Service
CFS	Cubic Feet Per Second	NTDE	National Tidal Datum Epoch
CI	Confidence Interval	OMRR&R	Operations, Maintenance, Repair, Replacement, and Rehabilitation
CL	Condado Lagoon	OWI	Ocean Weather Inc.
CSRM	Coastal Storm Risk Management	PARA	Prepare, Absorb, Recover, And Adapt
DEM	Digital Elevation Model	PCX	Planning Center Of Expertise
DNER	Department Of Natural And Environmental Resources	PDT	Project Delivery Team
EA	Each	PED	Pre-Construction Engineering & Design
EC	Engineer Circular	PFMA	Potential Failure Mode Analysis
ECB	Engineering And Construction Bulletin	PLCA	Probabilistic Life Cycle Analysis
EM	Engineer Manual	POR	Period Of Record
EN	Engineering	PR	Puerto Rico
EP	Engineer Pamphlet	PRVD02	Puerto Rico Vertical Datum Of 2002
ER	Engineer Regulation	PSE	Protective System Element
ERDC	Engineer Research And Development Center	QRA	Qualitative Risk Assessment
ESJB	East San Juan Bay	RPN	Rio Puerto Nuevo
ET	Extra-Tropical	RSL	Relative Sea Level
ETL	Engineer Technical Letter	RSLC	Relative Sea Level Change
EWL	Extreme Water Level	SACS	South Atlantic Coastal Study
FE	Flowage Easements	SAJ	Jacksonville District
FEMA	Federal Emergency Management Agency	SCS	Soil Conservation Services
FIS	Flood Insurance Study	SJH	San Juan Harbor
FPLE	Flood Protection Levee Easements	SLC	Sea Level Change
FRM	Flood Risk Management	SLR	Sea Level Rise
FWOP	Future Without Project	SPM	Shore Protection Manual
FWP	Future With Project	SPT	Standard Penetration Test
GEV	Generalized Extreme Value	STWAVE	Steady State Spectral Wave
GIS	Geographic Information System	SWAN	Simulating Waves Nearshore
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System	SWEL	Stillwater Elevation
HSD	Honest Significant Variance	TAW	Technical Advisory Committee For Water Retaining Structures
MA	Model Area	TS	Tropical Storm
MCS	Monte Carlo Simulation	TSP	Tentatively Selected Plan
MHHW	Mean Higher High Water	TWAE	Temporary Work Area Easement
MHW	Mean High Water	USACE	U.S. Army Corps Of Engineers
MLLW	Mean Lower-Low Water	USDA	U.S. Department Of Agriculture
MLW	Mean Low Water	VLM	Vertical Land Movement
MSL	Mean Sea Level	WIS	Wave Information Study
NHC	National Hurricane Center	WJSB	West San Juan Bay
NNBF	Natural And Nature Based Features		

1 Purpose

Section 204 of the Flood Control Act of 1970, Public Law 91-611, authorizes the Secretary of the Army, acting through the Chief of Engineers, to prepare plans for the development, utilization, and conservation of water and related land resources of drainage basins and coastal areas in the Commonwealth of Puerto Rico (PR). The Bipartisan Budget Act of 2018, Public Law 115-123, signed into law February 9, 2018, authorizes the Government to conduct the San Juan Metropolitan (Metro) Coastal Storm Risk Management (CSRM) Study at full Federal expense ($3 million). The three-year study schedule initiated on September 17, 2018.

The U.S. Army Corps of Engineers, Jacksonville District, and the Local Sponsor, Puerto Rico Department of Natural and Environmental Resources (DNER), conducted this CSRM Study to address flooding from coastal storms and hurricanes along the back bay areas within the municipality of the San Juan Metropolitan Area. This appendix is prepared in accordance with ER-1110-2-1619 and ER 1105-2-101 to understand the risks of the project area and resolve them using reliability based designs.

The San Juan Metro CSRM Study will assess potential solutions to storm damage susceptibility and determine Federal interest in the study area and develop a recommended plan for reduction of coastal storm risk by addressing infrastructure damages and public safety hazards along the back bay shoreline and adjacent municipalities. To properly design and develop potential solutions, this study analyzed the impacts of storm surge, tidal influences, wave contributions, and sea level change (SLC) over the life cycle of the proposed project.

2 Project Location

The CSRM study is located on the north end of Puerto Rico and includes San Juan Bay, Condado Lagoon, San José Lagoon, Los Corozos Lagoon, La Torrecilla Lagoon and Piñones Lagoon, as well as the interconnecting Martín Peña and San Antonio Channels and the Suárez Canal. San Juan Bay, Condado Lagoon, and La Torrecilla Lagoon connect with the Atlantic Ocean where effects from storm surge, tidal influences, wave contributions, and SLC cause flooding into the back bay regions of the San Juan Metropolitan Area. These coastal parameters threaten critical infrastructure and contribute to public safety hazards.

The study area was broken into six reaches as seen within **Figure A - 1**, although the Project Delivery Team (PDT) screened out three reaches due to reasons described in the main report and **Section 4.2**. For this study, the San Juan Metro area consists of three study reaches; West San Juan Bay (WSJB), East San Juan Bay (ESJB), and Condado Lagoon (CL). WSJB is located on the west side of the San Juan Bay and includes Toa Baja, Cataño, the Bay side of Guaynabo, and portions of the port on the south side of San Juan Bay. The WSJB reach extends inland to include the neighborhood of Las Vegas and Puente Blanco in Cataño. ESJB consists of areas along the east side of San Juan Bay and includes El Viejo San Juan, Puerta de Tierra, Isla Grande, and Tras Talleres until the Puerto Nuevo Bridge. CL is located just east of ESJB and El Boquerón Inlet connects CL to the Atlantic Ocean. CL includes areas of Miramar and Condado that surround Laguna del Condado, and extends westbound until Calle Taft in Condado. The three study reaches consist of approximately 32 miles of back bay shoreline.

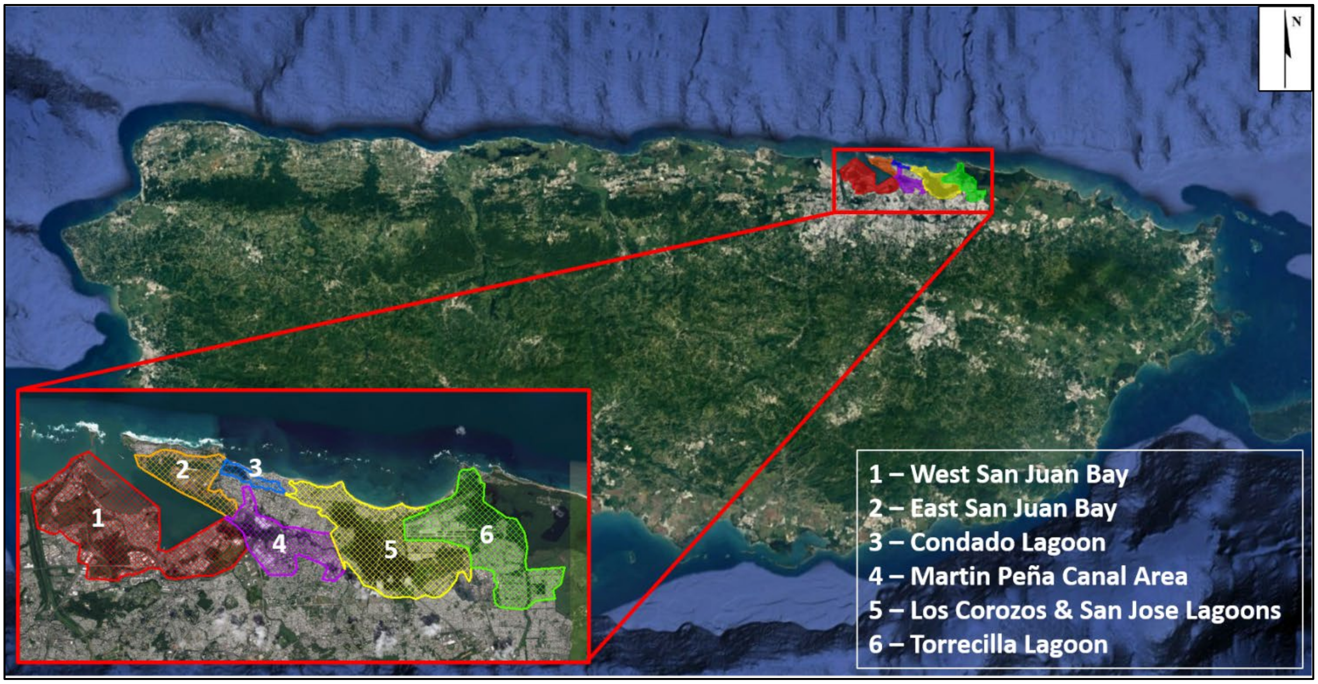

Figure A - 1. Project Location

3 Natural Forces

The natural forces affecting the San Juan Metropolitan area are Winds, Waves, Tides, Storms, and SLC.

3.1 Winds

Local winds can contribute to storm surge and the generation of small-amplitude, short period, waves which are important contributors to infrastructure damage throughout the back bay region. The study area lies within the tropical trade wind zone, resulting in moderate winds from a prevailing easterly direction. Elevated wind speeds from the north-northeast quadrant in winter months occur during passage of northeasters, which can cause extensive storm surge and shorefront damage. “Northeasters” are frontal weather patterns driven by cold Arctic air masses that extend as far south as Puerto Rico. While Northeasters often result in wave conditions that cause extensive erosion and increased wave setup on the north coast of Puerto Rico, the south coast of Puerto Rico experiences little influence from these events. Occasionally tropical storms (TS) impact the area and generate devastating winds, waves, and storm surge, which can cause direct damage to coastal structures and infrastructure.

Wind data offshore of the study area is available from the U.S. Army Corps of Engineers (USACE) Wave Information Study (WIS) Program. WIS hindcast data are generated using the numerical hindcast model WISWAVE (Hubertz, 1992). WISWAVE is driven by wind fields overlaying a bathymetric grid. Model output includes significant wave height, peak and mean wave period, peak and mean wave direction, wind speed, and wind direction. In the Atlantic Ocean, the WIS hindcast database covers a 35-year period of record extending from 1980 to 2014.

There are six WIS stations surrounding Puerto Rico. WIS Station 61019, located at latitude 19.0°and longitude -66.0° (approximately 38.3 miles north of the study area) displayed within **Figure A - 2**, is representative of offshore wind and wave conditions for the study area. **Figure A - 3** shows a summary of wind data from WIS Station 61019 along with a wind rose. This figure contains a summary of average wind speeds and frequency of occurrence broken down into eight 45-degree angle-bands. The annual average winds are predominantly from the east, which is representative of the tropical trade wind zone. The average wind speeds are highest out of the northeast due to the high frequency of northeasters affecting the area. Wind conditions within Puerto Rico are seasonal; this is displayed within the wind rose presented in **Figure A - 3**, which provides an additional breakdown of the seasonal wind conditions in the study area.

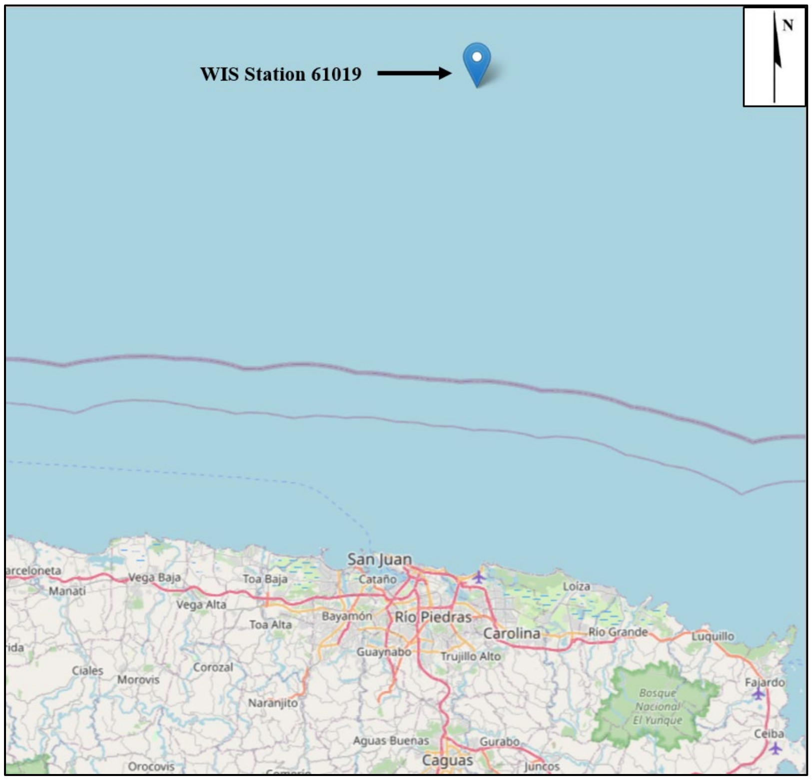

Figure A - 2. WIS Station 61019

Wind Direction (from)	WIS Station #61019 (1980-2014)	
	Percentage Occurrence (%)	Average Wind Speed (mph)
N	1.7	13.0
NE	13.8	16.4
E	66.6	15.9
SE	13.6	12.2
S	2.5	10.8
SW	0.8	10.2
W	0.5	10.6
NW	0.6	11.5

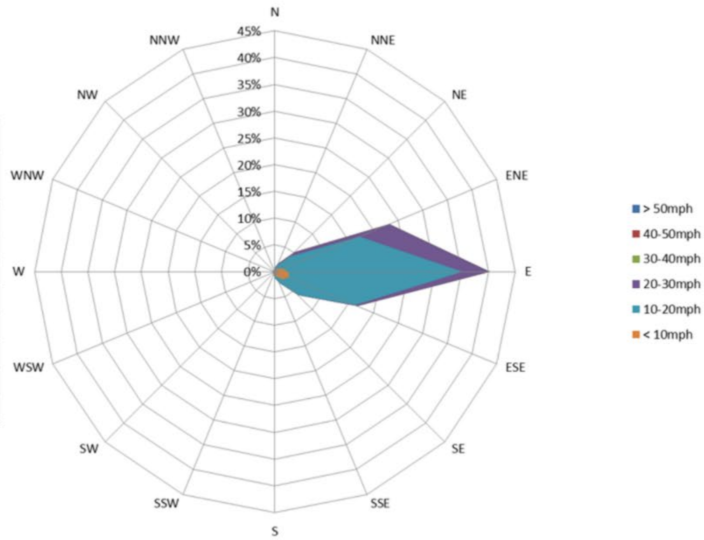

Figure A - 3. Wind Rose – WIS Station 61019

Table A - 1. Seasonal Wind Conditions

Month	WIS Station #61019 (1980-2014)	
	Average Wind Speed (mph)	Predominant Direction (from)
January	16.7	E
February	16.3	E
March	15.1	E
April	14.1	E
May	13.7	E
June	15.0	E
July	16.6	E
August	15.6	E
September	13.7	E
October	13.2	E
November	15.4	E
December	16.7	E

Wind conditions within Puerto Rico are seasonal, displayed within **Table A - 1**, which provides an additional breakdown of the seasonal wind conditions in the study area. During summer and fall months (June through November), tropical atmospheric waves often develop into tropical storms and hurricanes which can generate devastating winds, waves, and storm surge. **Section 3.4** discusses these intense seasonal events in detail.

Near San Juan, winds are predominantly from the east throughout all months, although throughout the winter and spring months the secondary wind direction is generally from the northeast, and throughout the summer and fall months, the secondary wind direction generally is from the southeast.

Additionally, daily breezes onshore and offshore result from differential heating of land and water masses. These diurnal winds typically blow perpendicular to the shoreline and have less magnitude than the trade winds and northeasters. While these breezes play a significant role in local weather patterns, they are not an appreciable cause of nearshore damage and erosion.

3.2 Waves

The wave energy dissipation that occurs as waves directly impact coastal structures is often a principal cause of infrastructure damage. Wave height, period, and direction, in combination with tides and storm surge, are the most important factors influencing the behavior of the shoreline. The San Juan Metro study area is exposed predominantly to short period wind-waves with periodic exposure to longer period storm swells within certain portions of the study area. However, the majority of the back bay study area is protected by Isla de Cabras and Old San Juan land masses fronting the Atlantic Ocean, which dissipate most of the ocean-driven waves. The remaining wind-driven waves within the San Juan Bay and ocean-driven waves, through the San Juan Bay Inlet, are generally depth limited as they approach the shoreline, thus limiting the size and associated period of the waves. Periodic damage to upland development, within specific portions of the back bay shoreline, is partially attributable to large storm waves produced primarily by northeasters during the late fall, winter, and early spring months and tropical disturbances, including hurricanes, during the summer months. Storm passage (northeasters and tropical storms) is frequent for the study area; even without landfall, a storm system passing within several hundred miles may cause an increase in waves that can affect the area.

This study obtained wave data from the USACE WIS hindcast database for the Atlantic Ocean. As previously discussed, this study implemented the WIS station 61019, displayed within **Figure A - 2**. Given the deep depth at this station (11,733 feet), wave conditions at 61019 are representative of the general study area but do not accurately represent nearshore wave conditions within the San Juan Bay.

Figure A - 4 summarizes the percentage of occurrence and average wave height of the WIS waves by direction. Average wave heights range from 5.9 feet to 9.6 feet, indicating a moderate wave climate year round. Wave directions are generally from the east and northeast quadrants as displayed in the wave rose presented in **Figure A - 4**. A seasonal breakdown of wave heights show that higher wave heights are more frequent in the late fall, winter, and early spring months (November through March) and tend to originate from the northeast and east quadrant equally (**Table A - 2**). These larger wave heights can be attributed to the northeasters occurring along the east coast of North America inherently driving larger waves southeast towards the study area. Late spring, summer, and early fall waves (April through October), are smaller and originate predominantly from the east.

Wave Direction (from)	WIS Station #61019 (1980-2014)	
	Percentage Occurrence (%)	Average Wave Height (ft)
N	5.73	7.0
NE	30.12	6.5
E	63.74	6.2
SE	0.20	5.9
S	0.03	8.9
SW	0.01	9.6
W	0.01	6.4
NW	0.16	7.8

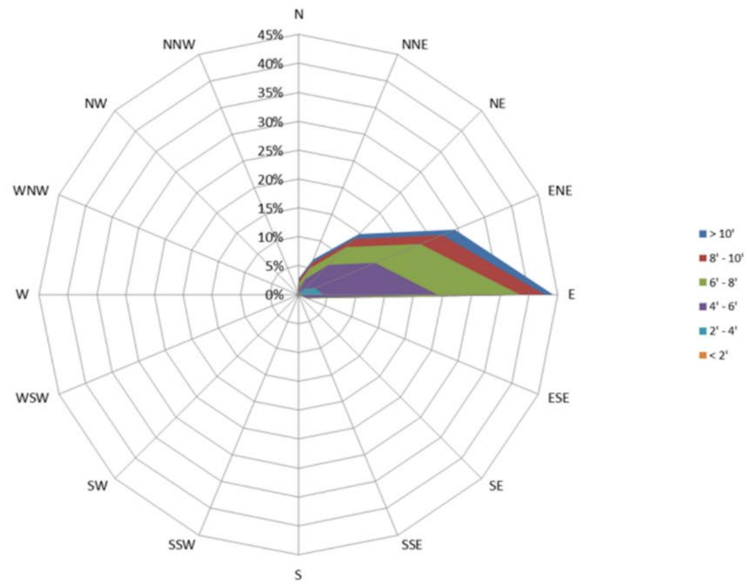

Figure A - 4. Wave Rose - WIS Station 61019

Table A - 2. Seasonal Wave Conditions

Month	WIS Station #61019 (1980-2014)	
	Average Wave Height (ft)	Predominant Direction (from)
January	7.8	E
February	7.5	E
March	6.8	E
April	6.0	E
May	5.3	E
June	5.4	E
July	6.1	E
August	5.6	E
September	5.4	E
October	5.6	NE
November	7.0	NE
December	7.7	NE

Table A - 3 provides a seasonal breakdown of percent occurrence by wave period, which displays that long period, storm-generated swells are common throughout the year. The late fall, winter, and spring months (November to April) have slightly larger periods indicating the influence of Northeasters throughout the months of November through April. The highlighted values show the dominant wave period for each month. None of the dominant periods are less than 8.0 seconds.

Table A - 3. Wave Period – Percent Occurrence

Wave Period (Sec)	Percent Occurrence by Wave Period Band											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
< 4.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.0 - 4.9	0.2	0.0	0.0	0.0	0.0	0.1	0.0	0.2	0.0	0.0	0.0	0.0
5.0 - 5.9	0.2	0.0	0.2	0.3	0.3	0.5	0.2	0.6	0.3	0.3	0.1	0.1
6.0 - 6.9	0.6	1.0	0.8	2.1	2.6	2.9	2.1	6.2	5.6	5.1	1.5	0.8
7.0 - 7.9	2.7	2.5	5.4	7.0	10.3	11.5	14.3	21.1	17.9	12.3	6.7	6.2
8.0 - 8.9	13.4	13.6	16.6	23.6	35.2	44.5	45.4	43.0	30.3	29.4	23.1	14.9
9.0 - 9.9	32.2	31.5	28.9	32.0	32.8	35.8	35.2	21.1	20.0	23.2	28.3	29.1
10.0 - 10.9	28.8	26.4	21.3	14.8	9.7	3.5	2.5	3.5	11.1	10.9	17.3	19.9
11.0 - 11.9	12.2	13.0	10.0	8.3	5.2	0.7	0.1	2.1	6.1	8.7	10.5	12.9
> 12.0	9.9	12.1	16.8	12.0	4.0	0.6	0.2	2.3	8.6	10.2	12.5	16.1

3.3 Tides and Currents

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

Tides in San Juan, Puerto Rico are affected by mixed semidiurnal tidal fluctuations of the Atlantic Ocean, meaning two high and low tides occur at different elevations per tidal day. The study obtained tidal datums for San Juan, La Puntilla from NOAA tide station 9755371 in San Juan Bay, PR. The NOAA gauge contains data from 11/29/1977 to present (12/31/2019). Tidal datums are summarized in **Table A - 4** and are referenced to the Puerto Rico Vertical Datum of 2002 (PRVD02) and Mean Sea Level (MSL). The PRVD02 vertical datum is the official vertical datum of Puerto Rico and is referenced to the MSL of NOAA tide station at San Juan (9755371). The datums presented in **Table A - 4** are based on a tidal analysis periods of 01/01/1983 to 12/31/1987 and 01/01/1990 to 12/31/2001. The mean tide range, the difference between Mean High Water (MHW) and Mean Low Water (MLW), is 1.11 feet and the spring tide range, the difference between Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW) is 1.58 feet.

Table A - 4. Tidal Datums for San Juan, La Puntilla (9755371)

Tidal Datum	Elevation (ft-PRVD02)
Mean Higher-High Water (MHHW)	0.81
Mean High Water (MHW)	0.54
Puerto Rico Vertical Datum of 2002 (PRVD02)	0.00
Mean Sea Level (MSL)	0.00
Mean Low Water (MLW)	-0.56
Mean Lower-Low Water (MLLW)	-0.77

Along the Atlantic (northern) and Caribbean (southern) coasts of Puerto Rico, the trade winds greatly influence the currents. In general, there is a west drift caused by prevailing east trade winds; the velocity averages about 0.23 miles per hour and is strongest near the island. With variable winds or light trade winds, it is probable that tidal currents are felt at times along the Atlantic and Caribbean coasts of Puerto Rico (NOAA, 2019).

3.4 Storm Effects

The back bay shoreline of San Juan Metro is generally influenced by tropical systems during the summer and fall months (hurricane season) and by northeasters during the late fall, winter, and spring months. Although hurricanes typically generate larger waves and storm surge, northeasters can have a greater cumulative effect on the area due to longer storm duration and greater frequency of event occurrence. Periodic and unpredictable hurricanes and coastal storms, with their intense breaking waves and elevated water levels, can cause significant damage to the shoreline and back bay infrastructure.

San Juan Metro is located in an area of significant storm activity. **Figure A - 5** shows historical trajectories of hurricanes and tropical storms from 1851 to 2019 as recorded by the National Hurricane Center (NHC). These hurricane data are available from NOAA (*https://oceanservice.noaa.gov/news/historical-hurricanes/*). The shaded circle in the center of this figure indicates a 100-nautical mile radius drawn from the center of the study area (San Juan). Based on NHC records, 119 tropical storms have passed within this 100-mile radius over the 169-year period of record. The 100-mile radius was chosen because a tropical disturbance passing within this radial area would likely produce damages along the shoreline. Stronger storms are capable of producing significant damage to the coastline from far greater distances.

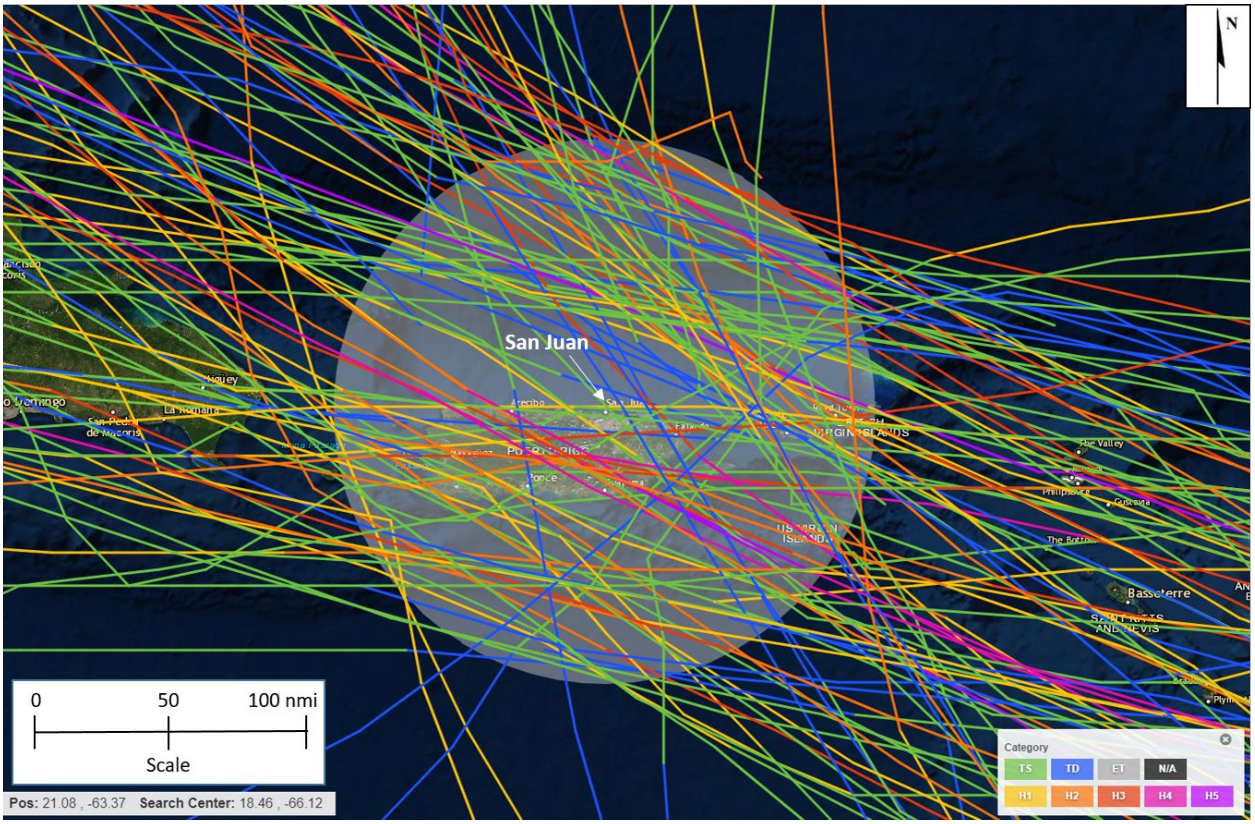

Figure A - 5. Historic Tropical Storm Tracks (1851 - 2019, 100-nautical mile radius)

In recent years, a number of named storms have significantly affected the study area including Hugo (1989), Georges (1998), Irma (2017), Maria (2017), and extra-tropical storm Riley (2018). Along with these named storms, damages from more distant storms have caused indirect impacts, including damage from winds, waves, and elevated water levels.

3.4.1 Storm Surge

The definition of storm surge is the rise of the ocean surface above its astronomical tide level due to storm forces. Surges occur primarily due to atmospheric pressure gradients and surface stresses created by wind blowing over a water surface. Strong onshore winds pile up water near the shoreline, resulting in super-elevated water levels along the coastal region and within inland waterways. In addition, the lower atmospheric pressure, which accompanies storms, also contributes to a rise in water surface elevation. Extremely high wind velocities coupled with low barometric pressures, such as those experienced in tropical storms, hurricanes, and very strong northeasters, can produce high damaging water levels. Water level (with storm surge) time series are critical for input into coastal storm risk modeling applications. In addition to wind speed, wind direction, and wind duration, storm surge is also influenced by the water depth, length of fetch (distance over water), wave setup, and frictional characteristics of the nearshore sea floor. An increase in water depth may increase the potential for coastal flooding and allow larger storm waves to attack the shore. The influence of water depth on wave heights can be attributed to the term depth-limited waves. As a wave approaches a shoreline, the front of the wave slows due to bed friction, which allows the remaining energy on the backside of the wave to build up and eventually reach a breaking point. A deeper water depth allows the wave more room to build up prior to breaking due to bed friction.

The annual exceedance probability (AEP) is the probability of occurrence of an event within any given year. The AEP for storm surge events can provide insight into the vulnerabilities of a given location through the comparison of flooding caused by the event with the existing topography of an area. **Table A - 5** provides the peak storm surge heights of standard AEP events for the San Juan Metro area. The engineering team obtained storm surge levels corresponding to each AEP event presented in **Table A - 5** from the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) conducted in 2009 (FEMA, 2009) and the NOAA gauge 9755371 (San Juan, PR). The storm surge elevations presented include the effects of astronomical high tide and wave setup. **Table A - 5** displays AEP events from FEMA, which illustrates significantly larger elevations, for events equal to or greater than the 2% AEP, compared to the same NOAA events. The NOAA gauge at San Juan, PR (9755371) shows lower elevations for events equal to or above a 2% AEP event because the period of record (approximately 42 years) is too small; indicating additional recorded data is needed to accurately represent the larger events. The historical NOAA water levels could be used to predict future events, although the short historical record of the NOAA gauge may underestimate the future risk. The FEMA results provide an opportunity to evaluate impacts of stronger synthetic storms that may not have occurred on record, but could occur in the future.

Table A - 5. FEMA and NOAA Peak Storm Tide Elevations

Annual Exceedance Probability (AEP)	NOAA Peak Storm Surge Height (ft-PRVD02)*	FEMA Peak Storm Surge Height (ft-PRVD02)
0.200	1.79	0.85
0.100	1.90	1.87
0.020	2.29	3.94
0.010	2.49	4.92
0.004	-	6.43
0.002	-	7.71

*AEP developed from 1983 to 2001

3.5 Sea Level Change

3.5.1 Relative Sea Level Change Guidance and Tools

Relative Sea Level (RSL) refers to local elevation of the sea with respect to land, including the lowering or rising of land through geologic processes such as subsidence and glacial rebound. It is anticipated that sea level will rise within the next 100 years. The climate assessment for Relative Sea Level Change (RSLC) follows the USACE guidance of Engineer Regulation (ER) 1100-2-8162 (USACE 2013) and Engineer

Technical Letter (EP) 1100-2-1 (USACE, 2019). ER 1100-2-8162 and EP 1100-2-1 provide guidance for incorporating the direct and indirect physical effects of projected future SLC across the project life cycle in managing, planning, engineering, designing, constructing, operating, and maintaining the USACE projects and systems of projects. Planning studies and engineering designs over the project life cycle, for both existing and proposed projects, will consider alternatives formulated and evaluated for the entire range of possible future rates of SLC.

ER 1100-2-8162 provides both a methodology and a procedure for determining a range of sea level change estimates based on global sea level change rates, the local historic sea level change rate, the construction (base) year of the project, and the design life of the project. Three estimates are required by the guidance, a Low (Baseline) estimate representing the minimum expected sea level change, an Intermediate estimate (NRC I), and a High estimate (NRC III) representing the maximum expected sea level change. These estimates reference the midpoint of the latest National Tidal Datum epoch, 1992. Please refer to ER 1100-2-8162 for a detailed explanation of the procedure, equations employed, and variables included to account for the eustatic change as well as site-specific uplift or subsidence to develop corrected rates.

To better empower data-driven and risk-informed decision-making, the USACE has developed two web-based SLC tools: Sea Level Change Curve Calculator and the Sea Level Tracker. Both tools provide a consistent and repeatable method to visualize the dynamic nature and variability of coastal water levels at tide gauges, allow comparison to the USACE projected SLC scenarios, and support simple exploration of how SLC has or will intersect with local elevation thresholds related to infrastructure (e.g., roads, power generating facilities, dunes, and buildings). Taken together, decision-makers can align various SLC scenarios with existing and planned engineering efforts, estimating when and how the sea level may impact critical infrastructure and planned development activities (USACE, 2018).

The USACE Sea Level Change Curve Calculator was used to compute the RSLC, and is available at: http://corpsmapu.usace.army.mil/rccinfo/slc/slcc_calc.html. This Calculator uses the methodology described in ER 1100-2-8162, which supersedes EC 1165-2-212. The USACE Sea Level Change Curve Calculator provides comparisons to the following:

- the NOAA Technical Report OAR CPO-1 "Global Sea Level Rise Scenarios for the United States National Climate Assessment" (NOAA, 2012);
- the National Research Council's (NRC) "Sea Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future report" (NRC, 2012); and
- the "NPCC2 Climate Risk Information 2013: Climate Methods Memorandum" drafted by the New York City Panel On Climate Change (NPCC, 2013).

Additionally, extreme water levels (EWL) incorporated into the Sea Level Change Curve Calculator are based on statistical probabilities using recorded historic monthly extreme water level values. NOAA Technical Report National Ocean Service (NOS) CO-OPS 067 – "Extreme Water Levels of the United States 1893-2010" describes the methods and data used in the calculation of the exceedance probability levels using a generalized extreme value (GEV) statistical function (NOAA, 2013). The USACE method uses the same NOAA recorded monthly extreme values in a percentile statistical function. Both methods use data recorded and validated by NOAA at the long-term, established tide gauges. The extreme values at the gauge can be significantly different from what may occur at the study site, due to differences in site characteristics and complex interactions of physical forces that vary between the locations. The level of confidence in the exceedance probability decreases with longer return periods. Additional information is available at the CO-OPS website at: http://tidesandcurrents.noaa.gov/est.

The USACE Sea Level Tracker (USACE, 2018c) tool is used to analyze historic sea level behavior and the measured trends at selected gauges close to the project site that contain a recent period of record (POR). This tool is utilized to show historical data, such as the MSL monthly 5-yr and 19-yr moving averages at the gauge along with the USACE Low, Intermediate, and High SLC Prediction Curves.

3.5.2 Sea Level Rise Assessment

Based on historical sea level measurements taken from NOAA gauge 9755371 San Juan Bay, PR, the historic sea level change rate *(e+M)* was determined using the updated published SLC from *http://www.corpsclimate.us/ccaceslcurves.cfm*. At gauge 9755371, the MSL trend updated for 2018 is 2.04 mm/year (0.0066929 feet/year), with a 95% confidence interval of +/- 0.39 mm/year (0.0012795 feet/year). This is based on monthly MSL data from 1962 to 2018, which is equivalent to a change of 0.67 ft in 100 years; reference **Figure A - 6** and **Figure A - 7**. The PDT applied the SLC rate from NOAA gauge 9755371, listed above, within the Sea Level Rise Curve Calculator and Sea Level Tracker.

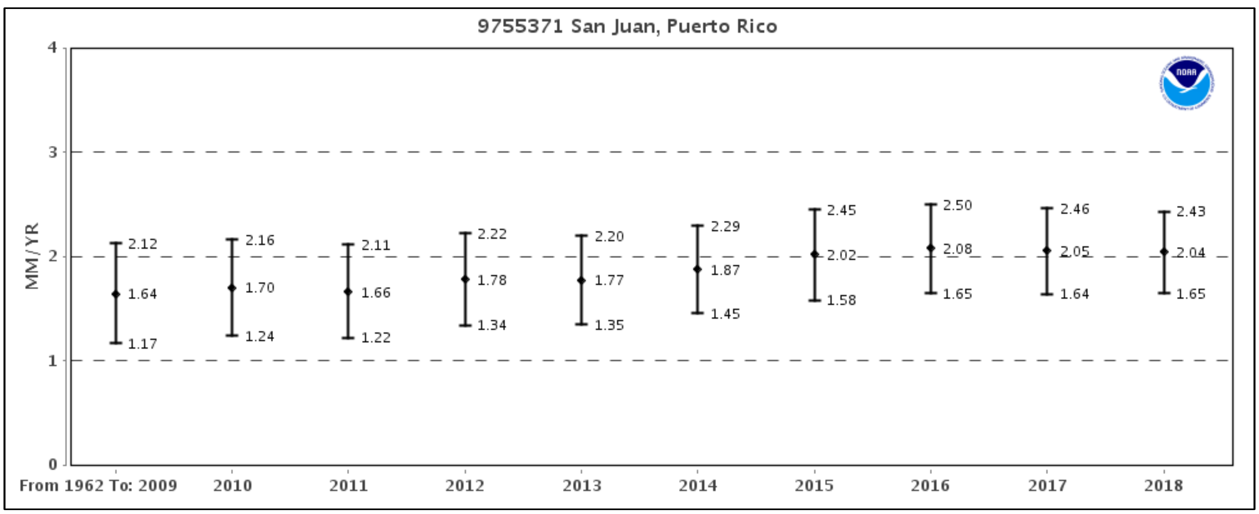

Figure A - 6. Updated SLC in mm/yr Based On Monthly Water Level Data from 1962 to 2018

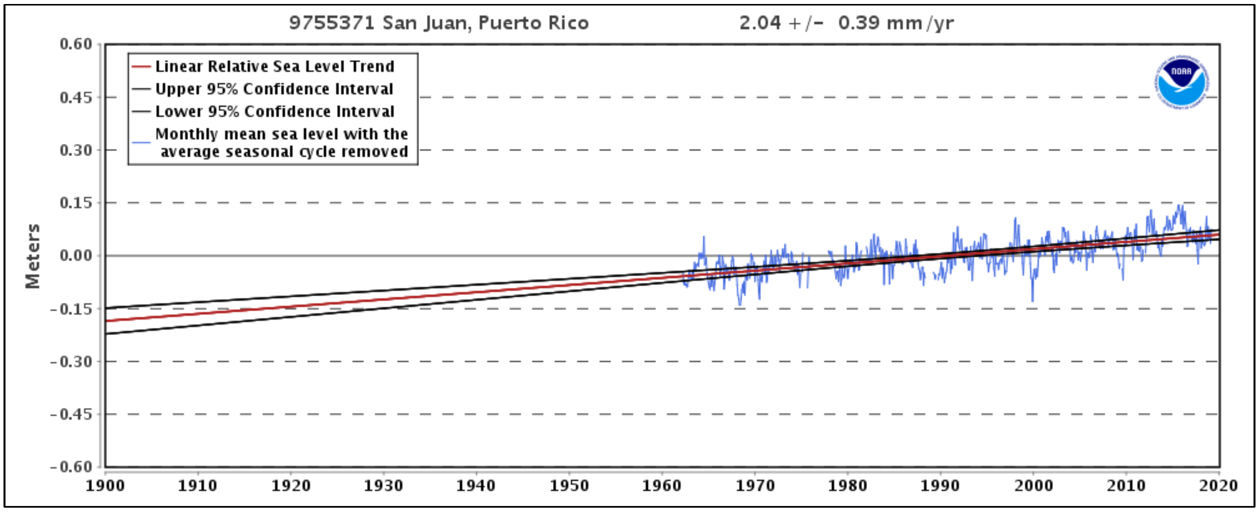

Figure A - 7. Mean Sea Level Trend Gauge 9755371 San Juan, PR

The following analysis evaluates potential effects on the San Juan Metro study. For the purpose of this analysis, the following years are evaluated:

- 2029 (Assumed construction completion)
- 2079 (50 years following construction completion, the end of Federal participation)
- 2129 (100 years following construction completion, representing the end of the San Juan Metro CSRM project life cycle)

Climate for which the project is designed can change over the planning life cycle of that project and may affect its performance, or impact operation and maintenance activities. Given these factors, the USACE guidance from ECB 2018-14, suggests that the project life cycle should be up to 100 years. For most projects, the project life cycle starts when construction is complete, which typically corresponds to the time when the project starts accruing benefits. For the San Juan Metro CSRM study, the economic analysis period for this study begins with a Generation II Coastal Risk Model (G2CRM) start year of 2020 (economic base year of 2029), extends to the end of Federal participation in 2079, and the end of the project life cycle in 2129.

Figure A - 8 displays the relative SLC projections from the center of the National Tidal Datum Epoch (NTDE), 1992, to the project base year (2029) with MSL monthly 5-yr and 19-yr moving averages. Both the 5- and 19-yr MSL moving averages generally follow the intermediate SLC curve for the study area. Therefore, the intermediate SLC curve was utilized for plan formulation processes, utilizing the 50-year and 100-year SLC to assess project damages and benefits as well as inform the design of project features. Following TSP, the PDT will run the low and high SLC curves within G2CRM to compare damages to the proposed design for the evaluation of risk and potential adaptation of project features.

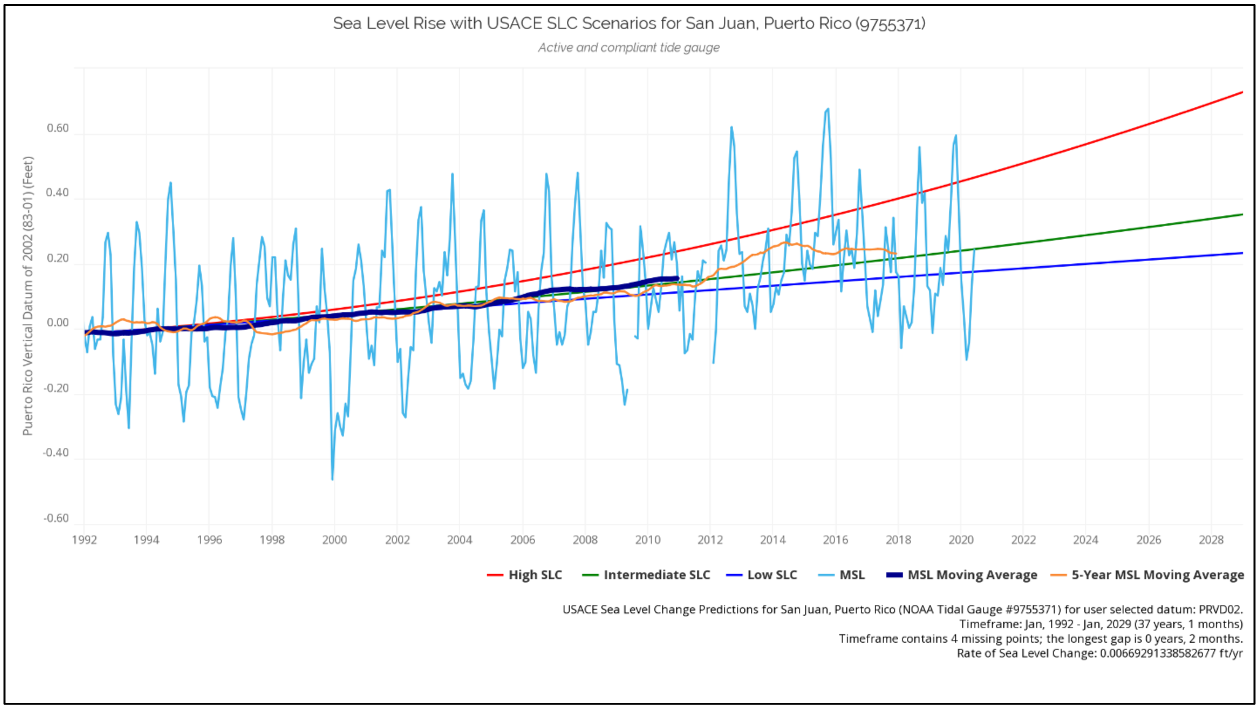

Figure A - 8. Relative SLC (1992 - 2029) with Monthly, 5-yr, and 19-yr MSL Moving Averages

Table A - 6 shows the relative SLC projections from 2020 to 2129 for the low, intermediate, and high curves. **Figure A - 9** displays the relative SLC projections from 2020 to 2129 for three levels of projected future SLC over the life of the project, as well as the NOAA 1% AEP above the intermediate curve and average elevations for Cataño (3.22 ft) and Condado Lagoon (2.38 ft). Note, the EWL (NOAA 1% AEP) above the intermediate curve shown within **Figure A - 9** is different than the FEMA AEP events used within the economic analysis. **Figure A - 10** displays the relative SLC projections from 2020 to 2129 for the three SLC curves overlaid with the average design elevation (8 ft-PRVD02) discussed within **Section 4.5**. The engineering team selected the average design elevation with resilience in mind and will follow recently published guidance from ECB 2020-6 (USACE, 2020) and use the principle of resilience: Prepare, Absorb, Recover, and Adapt (PARA) prior to the final Feasibility Report to appropriately design. The average design elevation will protect the study area out to the end of the project life cycle for the low, intermediate, and high SLC scenarios. The design elevation will also protect the study area from a 1.33% AEP event in addition to intermediate SLC out to the end of the project life cycle and MHHW.

The low elevation project area has a high probability of coastal flooding, and the communities are vulnerable to the effects of relative sea level change. **Figure A - 11** displays an example of this susceptibility, which plots the tidal datums and extreme water levels for NOAA gauge 9755371 against the average shoreline elevations at Condado Lagoon and Cataño. The average elevation for Condado Lagoon is above the NOAA 25-yr event, although portions of Condado Lagoon are as low as 1 ft-PRVD02 (below the NOAA 1-yr event). Additionally, as previously stated, the extreme values at the NOAA gauge can be significantly different than what was analyzed from FEMA due to differences in site characteristics and complex interactions of physical forces. For a full comparison to the FEMA AEP events, refer to **Table A - 5**. This comparison illustrates the vulnerability of much of the project area to elevated water levels due to even minor amounts of storm surge or even elevated high tides.

The total regional sea level rise predicted by the three scenarios (low, intermediate, and high) will have a significant impact to the San Juan Metro area. Potential impacts of rising sea level include overtopping of waterside structures, increased shoreline erosion, and flooding of low-lying areas. SLC will further exacerbate the problem of inundation due to storm surge and tidal impacts to the study area for the Future Without Project condition. As stated in Section 3.6.2, the engineering team selected the intermediate SLC scenario as a basis for the feasibility level of design. Therefore, the team designed all proposed project features to account for 50 years of potential sea level rise (approximately 1.17 ft). This increase in water elevation due to sea level rise is reflected as an increase in the design elevation of hard structures such as levees, floodwall, and seawalls. Additionally, the team incorporated measures, such as pump stations and culverts, into the project to help facilitate the proper discharge of the inland hydrologic runoff during elevated sea levels due to storm surge and sea level rise.

The optimization of the design elevation for the project features will continue over the upcoming months, including incorporation of recent guidance (ECB 2020-6). As this optimization occurs, the crest elevation, invert elevation, or other critical threshold elevations will be defined and compared against the sea level rise curves. This comparison will allow the appropriate design refinements to be recommended as well as proposition of potential methodologies that may trigger sea level change adaptations for project features. Additional analysis is forthcoming to determine the additional project modifications that would be needed for the federal project up to and beyond the 50-year planning horizon necessary to provide the same level of protection.

Table A - 6. Relative SLC Projections - Gauge: 9755371, San Juan, PR

Year	Low (Baseline)	Intermediate (NRC I)	High (NRC III)
	ft-PRVD02	ft-PRVD02	ft-PRVD02
2020	0.19	0.26	0.48
2029	0.25	0.37	0.76
2079	0.58	1.26	3.39
2129	0.92	2.59	7.88

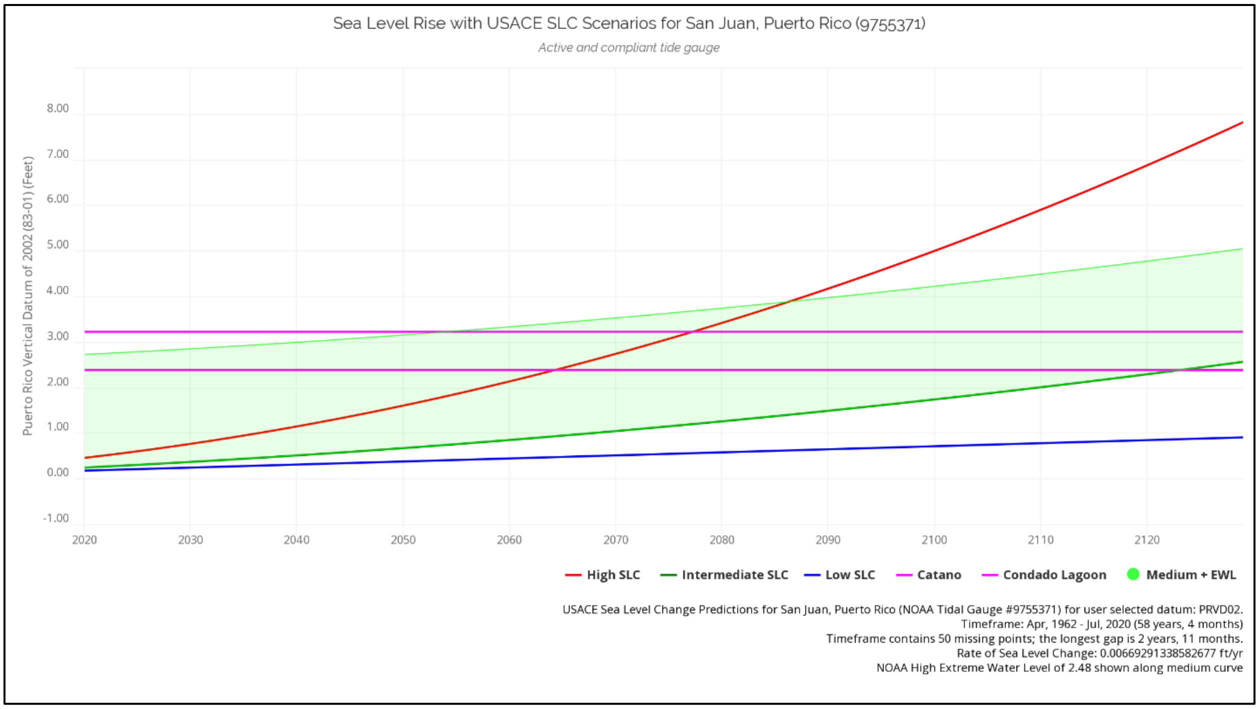

Figure A - 9. Relative SLC (2020 – 2129) with Average Elevations for Cataño (3.22 ft) and Condado Lagoon (2.38 ft)

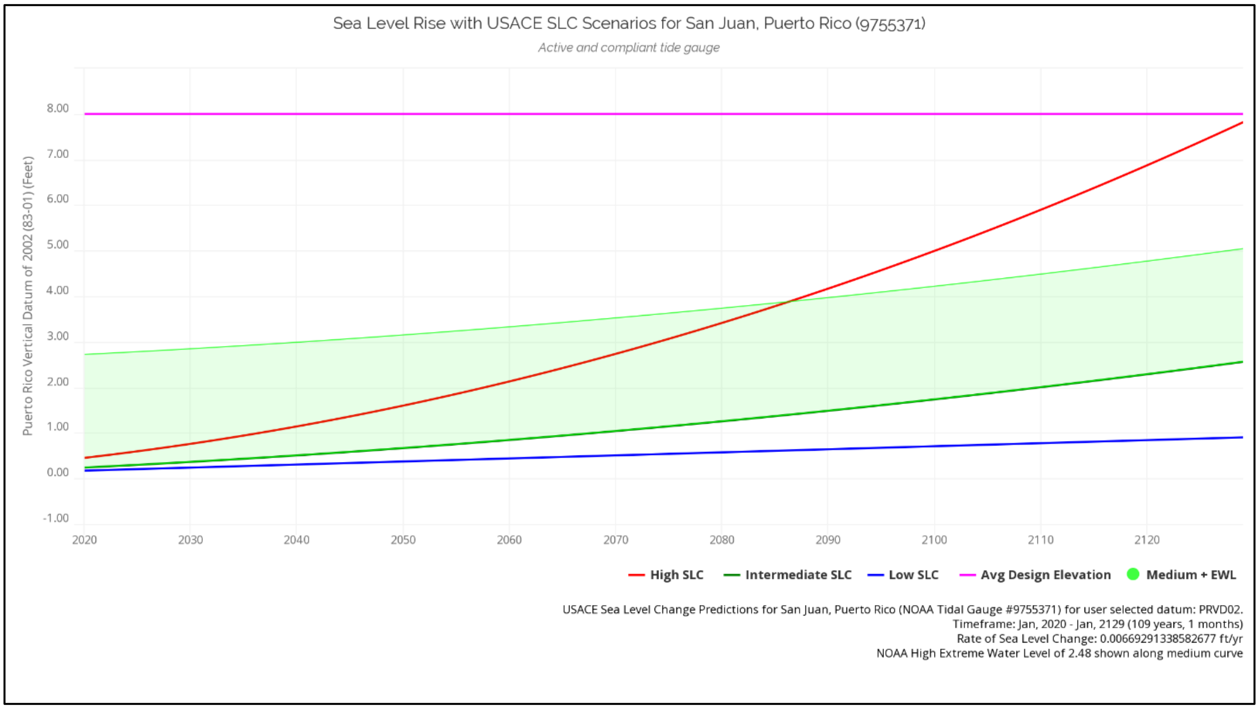

Figure A - 10. Relative SLC (2020 – 2129) with the Average Design Elevation (8 ft-PRVD02)

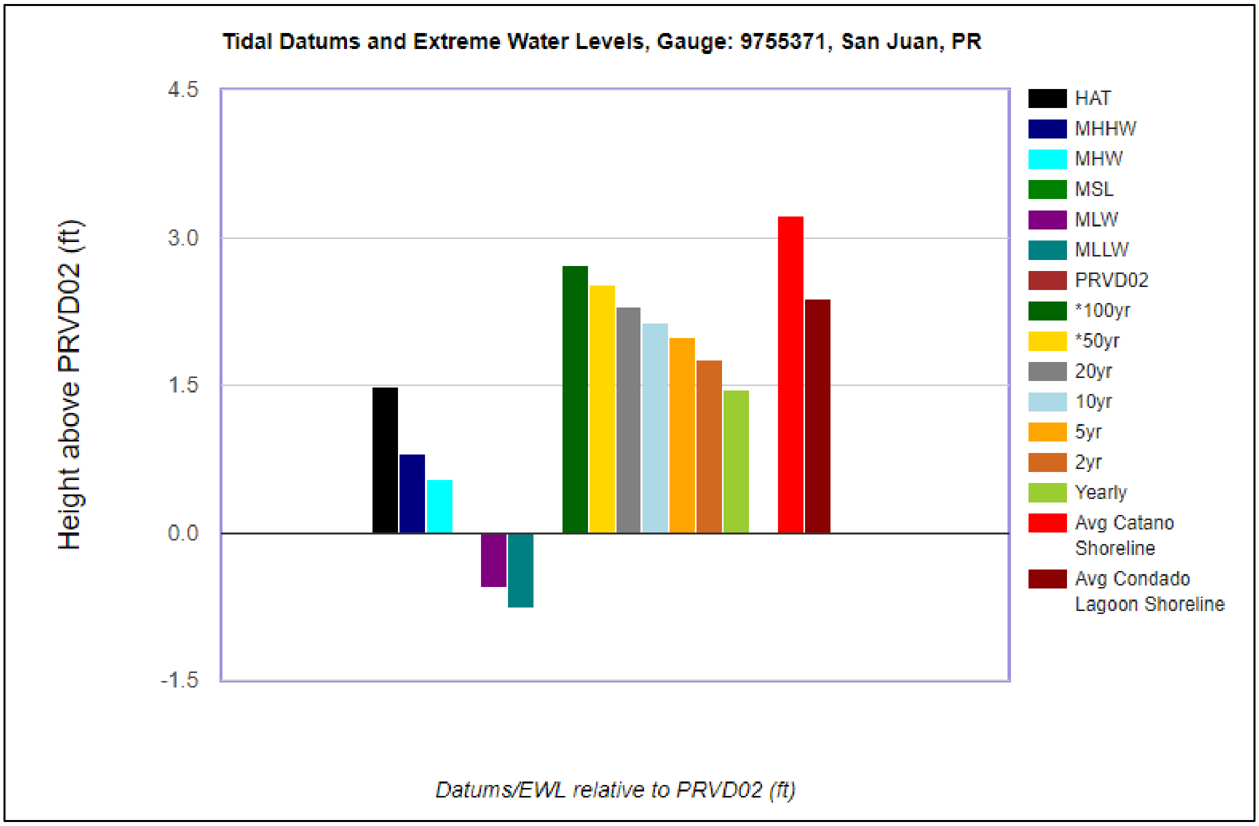

Figure A - 11. 2018 Tidal Datums and EWL for Gauge 9755371, San Juan, PR

3.5.3 Vertical Land Movement

Vertical land movement (VLM) is the change of a land surface in reference to a vertical datum over time. Positive trends in VLM indicate that the land is rising and negative trends in VLM indicate that the land is subsiding (Zervas et al., 2013). Zervas et al. (2013) found the VLM at NOAA tide station 9755371, San Juan Bay, to be 0.0008 in/yr (0.02 mm/yr). Indicating the study area does not experience significant VLM.

4 Engineering Evaluation

This chapter discusses the engineering analysis and assumptions implemented within the San Juan Metro study to assist the PDT in the selection of the recommended plan. Various engineering disciplines contributed to the completion of this study and the selection of a TSP.

4.1 Study Reach Delineation

To determine the coastal flooding extent within the San Juan Metropolitan area, the engineering team utilized three data sources. These were the 2018 FEMA advisory 0.2% AEP, the NOAA Seal Level Rise (SLR) viewer with 6-ft above MHHW, and the Advanced Circulation Model (ADCIRC) and Simulating Waves Nearshore (SWAN) modeling results of a Category 5 Maximum of Maximum event with approximately 3.3 feet (1 meter) of SLR. The combination of these data sets resulted in the delineation of six study reaches displayed within **Figure A - 12**.

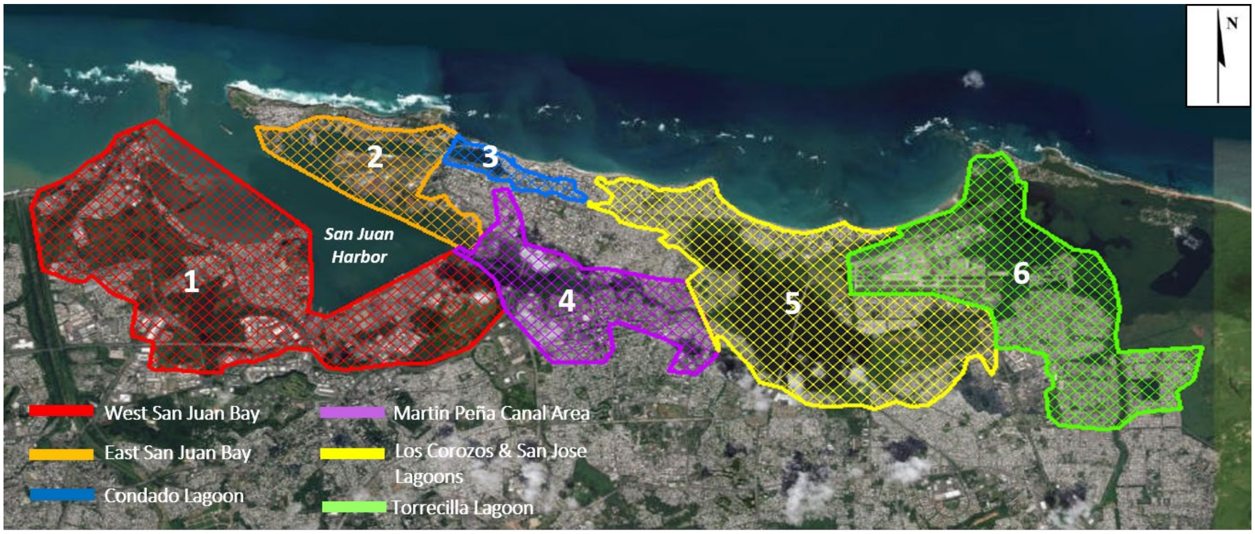

Figure A - 12. Study Reach Delineation

4.2 Study Reach Screening

Through the USACE SMART planning process, the PDT screened out study reaches 2, 4, 5, and 6. Initially, the PDT removed reaches 4 – 6 from the study after the team discovered that inland hydrology in combination with storm surge heavily influences flooding within these reaches. The engineering team analyzed existing water level data, rainfall data and reviewed various ADCIRC/STWAVE storm events to understand the inland hydrology and storm surge relationship within reaches 4 - 6. To quantify the difference in damages from both storm surge and inland hydrology the PDT would require additional time and budget outside the scope of this CSRM study. The PDT acknowledges flooding problems due to both storm surge and precipitation within reaches 4-6 and recommends a future study to assess the damages within these reaches. The PDT later removed Reach 2 after completion of Future Without Project (FWOP) G2CRM model runs, which resulted in insufficient damages to support an economically justified plan. Refer to the main report for additional description of the screening of reaches.

4.3 Generation II Coastal Risk Model

The PDT received approval from the Planning Center of Expertise (PCX) to implement G2CRM within the San Juan Metro study. G2CRM is a computer model that implements an object-oriented Probabilistic Life Cycle Analysis (PLCA) model using event-driven Monte Carlo Simulation (MCS). This allows for incorporation of time-dependent and stochastic event-dependent behaviors such as sea level change, tide, and structure raising and removal. The model is based on driving forces (storms) that affect a coastal region (study area). The study area is comprised of individual sub-areas of different types that may interact hydraulically and may be protected by coastal defense measures that serve to shield the areas and the assets they contain from storm damage (USACE, 2018b). To determine the damages for a specific event and time G2CRM compares the total water level (sum of the storm surge, SLC, tide, and potential wave inputs) to asset first floor elevations within FWOP or Protective System Element (PSE) elevations and then first floor elevations within Future With Project (FWP). G2CRM consists of multiple engineering inputs to accurately represent the study area which are described in the below sections. See the Economics Appendix for more information regarding the development of the G2CRM economic inputs.

4.3.1 Digital Elevation Model

A Digital Elevation Model (DEM) consists of arrays of regularly spaced land surface elevation values referenced to a horizontal reference datum. The engineering team created a 3m x 3m DEM to develop necessary inputs into G2CRM and assist in the design of measures within the study area. The engineering team compiled surface elevations from various data sources throughout the study area. **Table A - 7** lists the data sources in order of priority, with the higher priority surveys labeled from top to bottom.

Table A - 7. Data Incorporated Into the DEM

Priority Order	Data Name	Year	Data Type	Exported Resolution (m)
1	19-146 San Juan Harbor	2019	Hydrographic Survey	3
2	18-024 RPN Margarita Channel	2018	Hydrographic Survey	3
3	2018 USACE FEMA Topobathy Lidar DEM: Main Island, Culebra, and Vieques, Puerto Rico	2018	DEM	1
4	2016 USACE NCMP Topobathy Lidar DEM: Puerto Rico	2016	DEM	1
5	2016 NOAA NGS Topobathy Lidar DEM: Puerto Rico	2016	DEM	1
6	17-180 Caño Martin Peña	2017	Hydrographic Survey	1
7	14-020 Condado Lagoon	2014	Hydrographic Survey	3
8	ERDC Mosaic	2018	DEM	3

4.3.2 Model Areas

Model areas (MAs) are areas that comprise the overall study area. The water level in the modeled area is used to determine consequences to the assets contained within the area (USACE, 2018b). After the screening of study reaches 4 through 6, the engineering team divided the remaining reaches into MAs based on the separability from possible sources of coastal flooding for input into G2CRM. The engineering team used the DEM and the NOAA SLR Viewer to determine model separability based on the location of various flood sources. **Figure A - 13** displayed below, shows the location of the eight MAs developed. The engineering team originally designated the MAs titled WSJB-1A and WSJB-1B as one MA, WSJB-1. The engineering team later separated WSJB-1 into two MAs due the separability of the two areas from the Aguas Frias Canal.

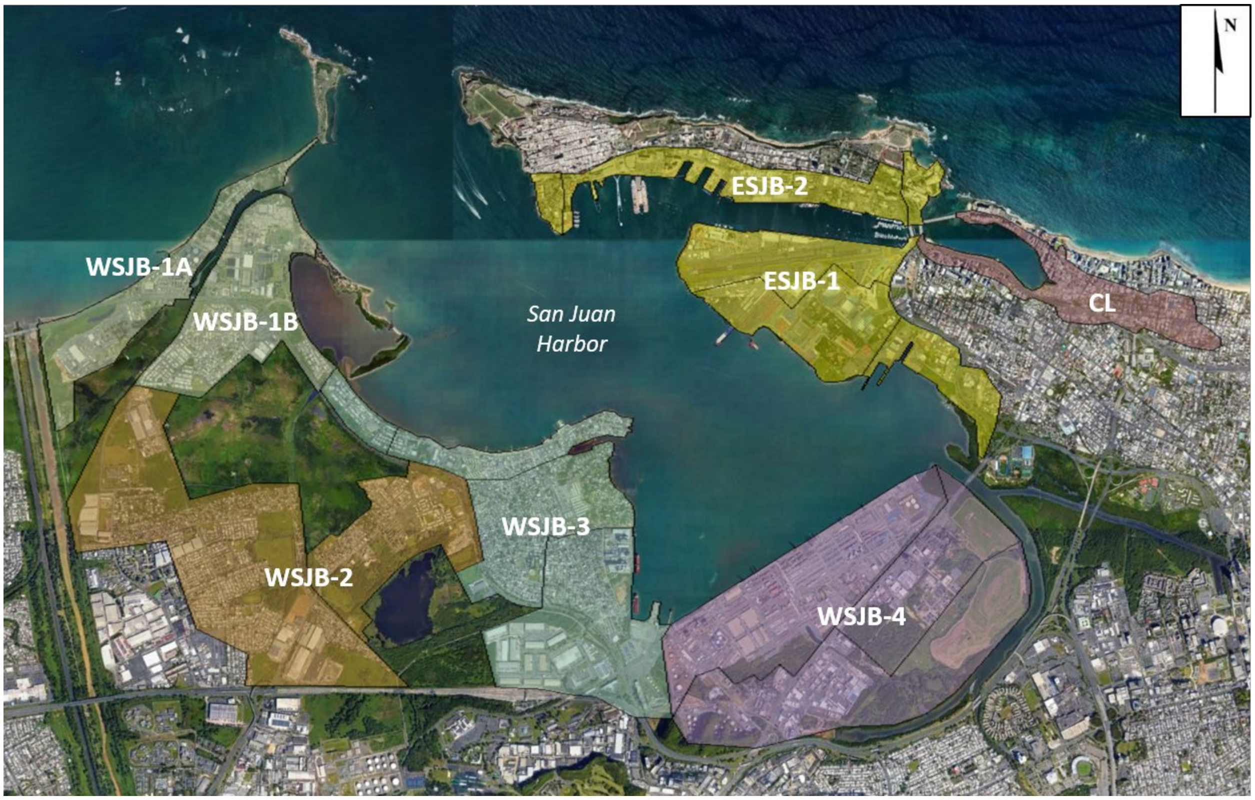

Figure A - 13. Model Area Locations

4.3.3 Protective System Elements

A protective system element (PSE) is the infrastructure that defines the coastal boundary; be it a coastal defense system that protects the modeled areas from coastal flooding (levees, pumps, closure structures, etc.) or a locally developed coastal boundary comprised of bulkheads and/or hardened shoreline (USACE, 2018b). Initially, the engineering team designated the PSEs to encompass the entire extent of the MAs adjacent to the flood source, based on the DEM and NOAA SLR Viewer, as a worst-case scenario. The engineering team later refined the exact locations and PSE lengths to remove high elevation areas following the development of the design elevations and completion of FWOP. Within FWOP, the engineering team chose the top elevation of the PSE to be equal to the lowest ground elevation along the PSE, this will best represent the FWOP conditions. Within FWP, the top elevation of the PSE will depend on the alternative and correspond to the correct design elevation.

4.3.4 Meteorological Driving Forces

Meteorological driving forces are storm hydrographs (surge and waves) at locations, as generated externally from high fidelity storm surge and nearshore wave models such as ADCIRC and STWAVE (USACE, 2018b). Additionally, the number of storms per year and relative storm probability are incorporated into G2CRM and further described below.

4.3.4.1 <u>Storm Hydrographs</u>

To develop tropical storm hydrographs, the Engineer Research and Development Center (ERDC) coupled ADCIRC and STWAVE. ADCIRC is a two-dimensional hydrodynamic model that conducts short- and long-term simulations of tide and storm surge elevations and velocities in deep-ocean, continental shelves, coastal seas, and small-scale estuarine systems. ADCIRC uses the finite element method to solve the reformulated, depth-averaged shallow water equations. The model runs on a triangulated mesh with elevations derived from a seamless bathymetric/topographic DEM that includes both offshore and overland areas. The triangulated format of the mesh allows variation in the element size, so the study area can have a high concentration of nodes while fewer nodes (with higher element areas) can be placed farther away to make the mesh size more efficient without compromising accuracy (FEMA 2015). STWAVE is a steady-state, finite difference, spectral model based on the wave action balance equation. ERDC clipped the ADCIRC mesh used in the South Atlantic Coastal Study (SACS) to cover the area of responsibility of this study, ranging from the approximate location of the municipality of Dorado to the municipality of Loíza. Similarly, ERDC also used the SACS STWAVE grid from the San Juan region. ERDC interpolated the DEM onto the grids and included savepoints provided by the Jacksonville District (SAJ) in addition to the existing SACS savepoints. SAJ selected 19 tropical storms from the SACS storm suite based on the water level based AEP events from FEMA, referenced in **Table A - 5**. The angle of incidence and landfall locations varied for these tropical storm events around Puerto Rico. Of the 19 tropical events ran in ADCIRC/STWAVE the engineering team selected 12 to represent the storm suite for G2CRM input. These 12 selected tropical events ranged from 33.3% to 0.2% AEP events in relation to the FEMA AEP events.

To reduce computational analysis of G2CRM, which bogged down the model and exponentially increased computational time, the engineering team performed an Analysis of Variance (ANOVA) with post-hoc Tukey Honest Significant Variance (HSD) statistical analysis on the initial storm suite per savepoint to determine whether each savepoint was statistically significantly different from one another. Then, the engineering team compared the volume of water generated by each storm per savepoint using the Trapezoidal Integration Rule. Lastly, the team compared the maximum water levels per storm for each savepoint. Through these analyzes, the engineering team determined that water levels from multiple savepoints could adequately be represented by the hydrograph at a single savepoint.

To represent higher frequency and longer duration extra-tropical (ET) events, the engineering team selected two events from Ocean Weather Inc.'s (OWI) operational (historical) storms from 1979 through 2017. Since the operational storms from OWI did not include events after 2017, the team added an additional ET storm (Riley) to the storm suite. **Table A - 8** displays the 15 storms described above. **Figure A - 14.** Storm Hydrographs for WSJB-3 displays an example input for model area WSJB-3.

Table A - 8. G2CRM Storm Suite

Storm	Storm Type	Stillwater Elevation, SWEL (ft)	Annual Exceedance Probability
3187	TS	0.24	33.0%
3/11/2013	ET	0.41	32.4%
12/11/2007	ET	0.51	28.5%
3/5/2018	ET	0.83	20.5%
3205	TS	0.86	18.0%
3192	TS	1.27	13.5%
3114	TS	1.77	10.0%
3167	TS	2.33	6.5%
3021	TS	2.73	4.3%
3295	TS	3.25	2.9%
3171	TS	3.86	2.0%
3229	TS	4.29	1.4%
3281	TS	4.92	1.0%
3045	TS	5.72	0.5%
3036	TS	6.84	0.2%

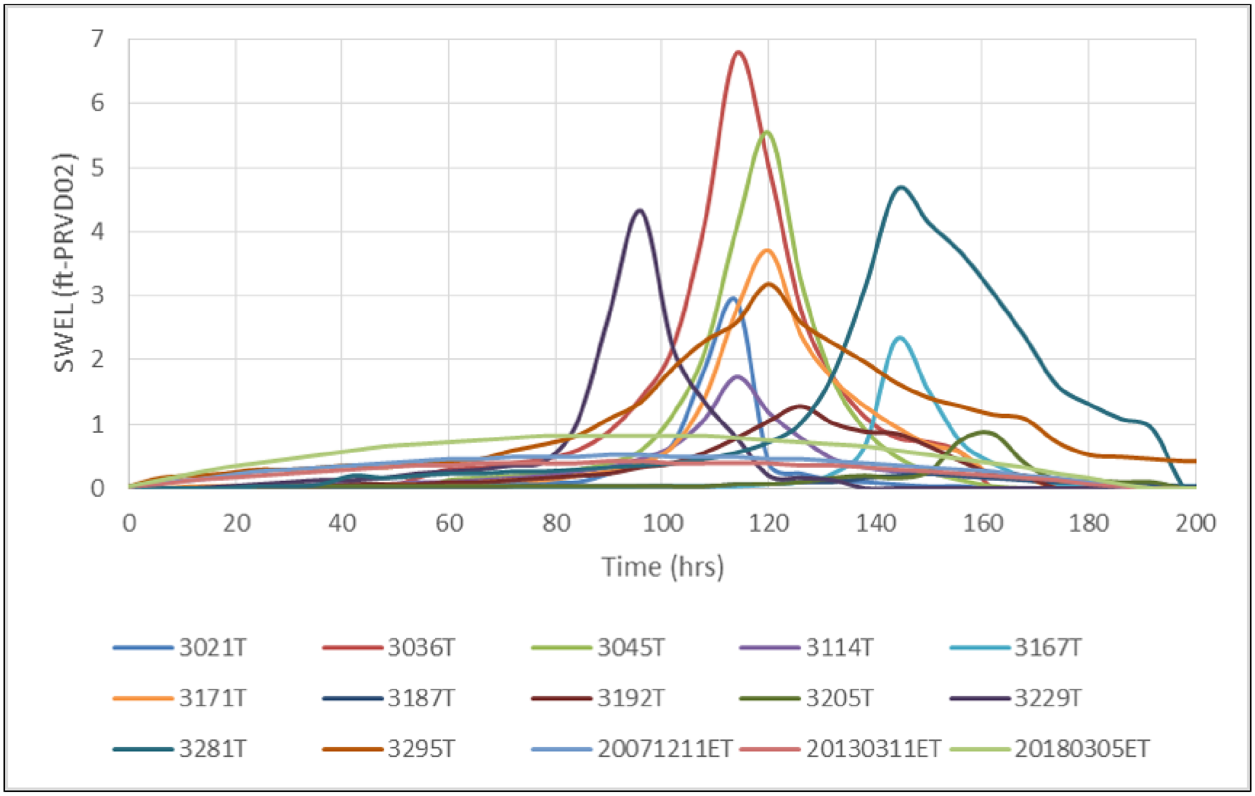

Figure A - 14. Storm Hydrographs for WSJB-3

4.3.4.2 Storms per Season

To determine the storm event generation G2CRM first selects the tropical and extra-tropical events to occur through each season within the year. This study implemented three storm seasons within each year: January through May as an extra-tropical season, June through November as a tropical season, and December as an extra-tropical season. The team chose three storm seasons because G2CRM will not allow a storm season to contain a break in time within it. G2CRM then uses the Poisson distribution to randomly select the number of storms that occur within each season based on the predetermined average number of storms in a season input. To determine the number of tropical and extra-tropical storm occurrences, this study analyzed wave data from WIS station 61019. To more accurately define what is classified as a storm impacting the study area the engineering team used wave heights instead of storm distance from the study area to ensure all events influencing the study area, regardless of distance, were accounted for. The analysis classified a "storm" as the average of the entire dataset plus two standard deviations (10.6 ft). The analysis applied a decluster time of five days to eliminate any duplicate events and then applied a peak-over-threshold of the wave height classified as a "storm" (10.6 ft). In order to classify which events were tropical or extra-tropical, the analysis used the NOAA Historical Hurricane Tracks (North Atlantic Basin) to filter through the data. **Table A - 9** displays the storm occurrences per year for extra-tropical and tropical events.

Table A - 9. Storms per Season

Span of Season	Season Type	Average Storms per Season
January through May	Extra-Tropical	6.8
June through November	Tropical	1.6
December	Extra-Tropical	0.6

4.3.4.3 Relative Storm Probability

After G2CRM selects the number of storms occurring in each season the model then chooses which storms will occur in each season by randomly selecting storms out of the available storm suite using bootstrap sampling with replacement (higher probability storms are chosen more often). To determine the relative storm probability based on surge, the study analyzed water level data from the NOAA gauge (9755371) at San Juan and compared them to the FEMA 2009 AEP curve, see **Table A - 5**. As described within **Section 3.4.1**, data from FEMA provides a wider range of synthetic storms, which include more intense storms that may not have occurred on record, but could occur in the future. Therefore, the PDT decided to use the FEMA AEP event elevations since they are both more conservative as well as more applicable for a study assessing 50 to 100 years into the future.

4.3.5 Tides

The engineering team selected the NOAA tide station 9755371 (San Juan Bay, PR) described within **Section 3.3**.

4.3.6 Sea Level Change Rate & Curve

The study implemented a sea level change rate of 2.04 mm/year (0.0066929 feet/year) based on the MSL trend at San Juan Bay, PR gauge 9755371. As mentioned within **Section 3.5.3**, the study area does not experience VLM; therefore, the sea level change rate excludes an adjustment for VLM. G2CRM also requires the selection of a SLC curve: the low, intermediate, or high USACE SLC curves. G2CRM then internally applies the appropriate SLC equation dependent upon the SLC curve selection. The study selected the intermediate curve because it followed the 5-yr and 19-yr mean sea level moving average trends referenced within **Figure A - 8**. Following TSP, the PDT will run the low and high SLC curves within G2CRM to compare damages to the proposed design for the evaluation of risk and potential adaptation of project features. For additional details on sea level change, refer to **Section 3.5**.

4.3.7 Stage Volume Input

G2CRM has an optional data import tool for the stage-volume relationship, which is used to represent internal ponding within the model area. If a stage-volume is not employed G2CRM will instantaneously transmit the stage, above the input PSE top elevation, into the model area. To more accurately represent the coastal flooding into a model area, G2CRM uses weir equation to calculate a time-dependent volume transmitted into the model area until the storage capacity within the model area is filled; after which G2CRM transitions back to transmitting the stage unmediated into the model area.

Geomatics created stage-volume curves using the DEM to determine the volume within each model area in relation to various stage elevations.

4.4 Geotechnical Engineering

Geotechnical engineering provided support to the study by reviewing existing geotechnical and geological data and providing design support and recommendations to assist in the selection of the TSP. To assist in design the average seaward elevation, average landward elevation, and design elevations (within **Section 4.5**) were provided to the geotechnical engineer. Currently, Jacksonville District's EN-GG Section is in the process of acquiring Standard Penetration Test (SPT) borings along the alignment of alternative locations to obtain the top of rock elevation for the potential design refinement of alternatives. See Geotechnical Engineering Appendix (Appendix D) for additional description of geotechnical-related tasks.

4.5 Design Elevations

To produce risk-based design elevations for the desired measures the engineering team followed ECB 2019-15 and ER 1105-2-101. ER 1105-2-101 states the assurance, also known as conditional non-exceedance probability, is based on the uncertainty in the flow and stages associated with a given exceedance probability event. This study utilized the 90% Confidence Interval (CI) from FEMA to incorporate the total water level uncertainty.

Initially, the PDT planned to utilize various design elevations to represent each measure, although due to insufficient time within the CSRM study process, the engineering team chose a single design elevation that would protect from the larger AEP events. After selection of the TSP, the PDT will analyze additional design elevations to ensure the most economically feasible elevation for the selected plan. To represent the design elevation, the study employed the 90% CI of the 1% AEP event with MHHW and the intermediate SLC out to the end of the assumed federal participation (2079). This design methodology will allow the engineering team to account for resilience, following ECB 2020-6, as mentioned within **Section 3.5.2**. **Table A - 10** displays the design elevations for each event within each model area, with the preliminary high design elevation corresponding to the 100-year event (shaded). The engineering team analyzed the stage-damage output from the FWOP G2CRM model runs to confirm the design elevations would provide sufficient damage reduction to each model area. The study used the average design elevation between the model areas for a representative cost estimate; following TSP the study will use specific design elevations relative to each model area.

Table A - 10. Design Elevations

Storms	AEP	15212 (WSJB1/2)	15214 (WSJB3)	15228 (WSJB4)	14932 (CL)
3187	0.330	2.40	2.37	2.45	2.54
3/11/2013	0.324	2.55	2.55	2.55	2.55
12/11/2007	0.285	2.66	2.66	2.66	2.66
3/5/2018	0.205	3.05	3.05	3.05	3.05
3205	0.180	3.13	3.12	3.17	3.13
3192	0.135	3.52	3.56	3.89	3.95
3114	0.100	4.16	4.06	4.17	4.25
3167	0.065	4.24	4.71	6.08	5.00
3021	0.043	5.11	5.35	5.64	5.13
3295	0.029	5.72	5.69	6.24	5.67
3171	0.020	6.46	6.31	7.13	6.46
3229	0.014	6.83	7.09	8.09	7.16
3281	0.010	7.72	7.58	8.93	7.86
3045	0.005	8.49	9.11	9.69	9.11
3036	0.002	10.55	10.66	11.13	10.41

Within the study area, some model areas are more susceptible to waves than others. At these model areas, the engineering team increased the design elevation to account for the 2% design wave runup using the Eurotop, Shore Protection Manual (SPM), and the Technical Advisory Committee for Water Retaining Structures (TAW) methodologies. The 2% design wave runup is the runup level exceeded by 2% of the incoming waves.

4.6 Study Measures

The Future Without Project (FWOP) results indicate that coastal storm events, along with tides, will continue to cause socioeconomic impacts within the study area. These impacts are expected to increase in frequency because of continued sea level change; therefore, the PDT formulated a set of measures to reduce these impacts by identifying measures; by scale; and combinability of measures with sound engineering design and practices. The PDT considered structural, non-structural, and natural and nature based features (NNBF) to reduce impacts from coastal flooding and wave attack. Reference the main report for additional details on the various measures prior to engineering design. A summary of the various designed measures are listed within **Table A - 11**. Each measure is marked as applicable per study area and/or the entire study area. This section provides a broad overview of the general assumptions for the study measures, while **Section 4.7** contains additional design information and details for each measure within each alternative. Prior to final Feasibility Report release, SAJ will perform a Potential Failure Mode Analysis (PFMA) and Qualitative Risk Assessment (QRA), to satisfy the requirements in Chapter 21 of ER 1110-2-1156, ECB 2019-15, and ER 1110-2-101 for floodwalls/seawalls, levees, and elevated living shorelines.

Table A - 11. Study Measures

Alternative	CL	WSJB_1A	WSJB_1B	WSJB_2	WSJB_3	WSJB_4	Study Wide
Floodwall/Seawall	X	X	X	X	X	X	X
Recreational Floodwall/Seawall	X				X		
King Pile Seawall	X						
T-Wall					X		
Toe Protection for Floodwall/Seawalls (Minimum)	X				X		
Toe Protection for Floodwall/Seawalls (Maximum)					X		X
Fill & Vegetation	X						
Elevated Living Shoreline	X		X		X		
Small Sluice Gate				X			
Medium Sluice Gate		X	X				
Levee		X	X	X		X	
Tiered Levee				X			
0m Breakwater					X		
1.83m Breakwater					X		
Emergent Island/Breakwater 0m					X		
Emergent Island/Breakwater 1.83m					X		
Large Storm Surge Barrier							X
Medium Storm Surge Barrier							X
Culverts (Inland Hydrology)	X	X	X	X	X	X	X
Pumps (Inland Hydrology)	X	X	X	X	X		

4.6.1 Floodwalls & Seawalls

Based on the design elevations and local geology, the engineering team determined the implementation of either floodwalls or seawalls at various locations within each model area. Seawalls are vertical structures, constructed parallel to the shoreline that separate land and water areas, and are primarily designed to prevent erosion and other damage due to wave action and additionally flooding. Floodwalls are vertical structures usually built of concrete, masonry, or a combination of both and are reinforced and anchored to withstand a flood load. The type and size of floodwalls is dependent on the difference between the back bay structure floor elevation and the design flood event. There are three types: T-Walls, L-Walls, and I-Walls. ETL 1110-2-575 and ECB 2017-03 advises for the use of I-walls where the height of wall is to be 6 feet or less. L-Walls, sometimes referred to as kicker pile walls, can withstand an 8-ft flood event. T-Walls have no height limit, but typically are constructed when the height of the wall is greater than 4-ft. **Figure A - 15** through **Figure A - 19** display examples of both floodwalls and seawalls.

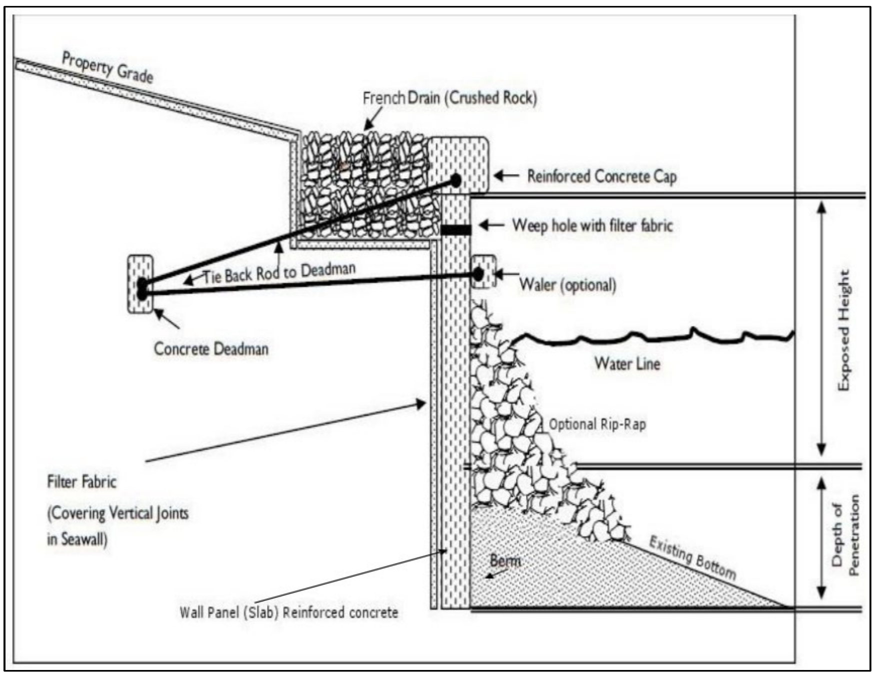

Figure A - 15. Typical Seawall (City of Marco Island, 2018)

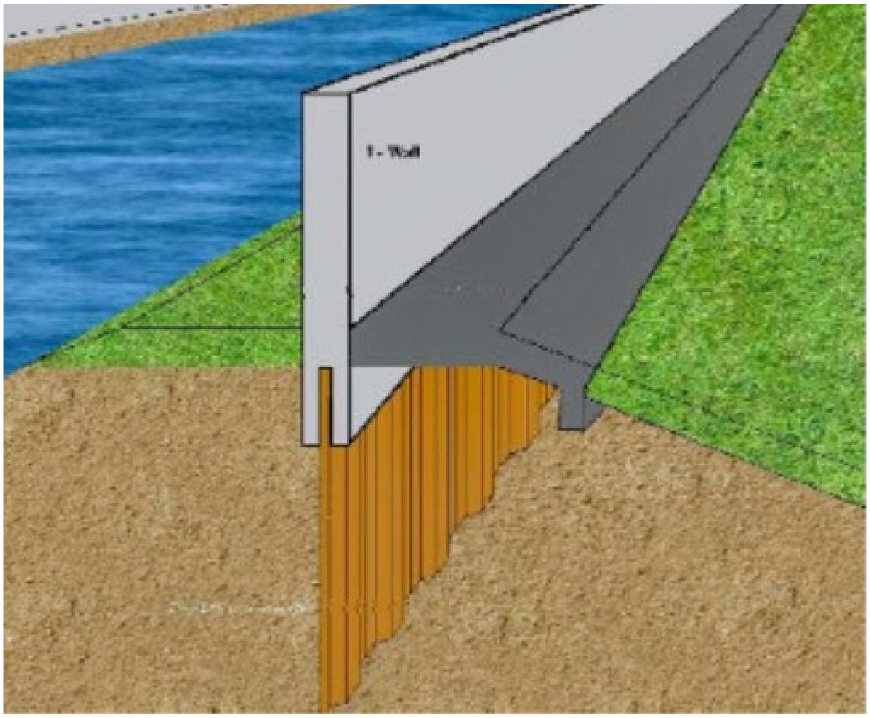

Figure A - 16. Typical Sheet-pile based I-Wall (Image courtesy of Tulane University)

Figure A - 17. Existing I-Wall Located In the City of Norfolk

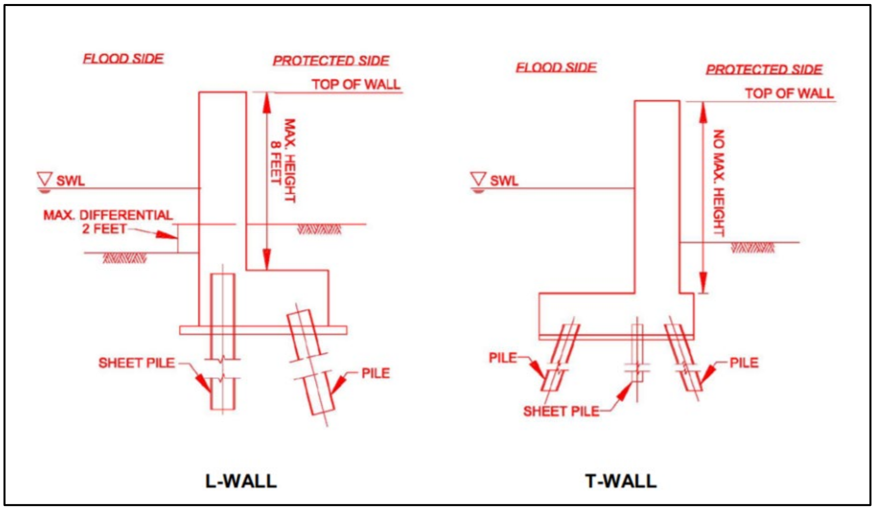

Figure A - 18. Typical T-Wall and L-Wall Configuration (Image courtesy of New Orleans District)

Figure A - 19. Existing T-Wall Located at the Richmond, VA Flood Control Project

The study team assigned various seawall designs to study locations subject to additional wave and current impacts. A steel cantilever sheet pile seawall is considered appropriate at study locations with sufficient space for backfill behind the seawall. The steel sheet piles will be driven into the ground two times the difference between the design elevation and the seaward elevation. Depending on the geology and depth of rock within each area, the engineering team chose king piles to provide additional support to the steel cantilever sheet piles when the depth driven is restricted by rock. All steel cantilever sheet pile seawalls will consist of a concrete cap that varies in dimensions depending on whether or not the seawall will provide recreational use. The design elevations within all model areas are higher than 6-ft, therefore the engineering team did not consider I-Walls as an appropriate measure. The team proposed T-Walls in locations where there is uncertainty in the depth of rock and there is insufficient room for backfill behind large king pile seawalls. Areas influenced by waves and currents required toe protection seaward of the chosen structure; the engineering team followed EM 1110-2-1614 to determine the apron width and height. The engineering team is currently refining locations where a smaller T-Wall or L-Wall could supplement a cantilever seawall at locations with limited wave and current action or little room for backfill; the team anticipates that this refinement will not change the TSP. The engineering team is planning to complete this analysis prior to the final Feasibility Report. As mentioned within **Section 3.5.2**, all floodwalls and seawalls will be designed with consideration for future adaptation for climate change.

4.6.2 Levees

As defined by ECB 2019-8, a levee system is comprised of one or more components, which collectively provide flood risk reduction to a defined area, referred to as a leveed area. The engineering team followed EM 1110-2-1913 for levee design; in which the guidance recommends a 1V:5H slope for sand levees and 1V:2H is the steepest slope recommended. The exact sediment type is unknown at the levee locations and therefore the engineering team assumed a slope of 1V:3.5H, which has been successfully designed for other projects in PR. Additionally, the study assumed a top width of 12-ft to maintain vehicle access. The engineering team also considered horizontal levees, also known as tiered levees, which are wider levees that either provide additional berms or have a more gradual slope. The team designed the tiered levees to consist of three berms, with the lower two berms at specified elevations to support marsh grass and mangrove plantings, which will help stabilize the levee by providing scour protection at the levee toe. To ensure the levee does not promote additional development into the floodplain the engineering team chose locations adjacent to existing structures. **Figure A - 20** and **Figure A - 21** displays examples of the different levee types. Further refinements of the design, including compliance of guidance ECB 2019-8, will be analyzed prior to the final Feasibility Report.

The engineering team plans to perform a QRA for any alternatives that include a levee (and potentially a floodwall or seawall), prior to the final Feasibility Report. To address life safety and the qualification of the tolerable risk guidelines the engineering team will follow Planning Bulletin 2019-04, which states levee risk, which is sometimes considered as incremental risk, is used to describe the additional risk imposed by non-performance of the levee. The incremental risk may occur from one or more of four scenarios: 1) breach prior to overtopping, 2) overtopping with breach, 3) malfunction or improper operation of levee system components, and 4) levee overtopping without breach. Floodwaters would inundate the community protected by the levee in the event of non-performance, posing a risk to life loss. The engineering team will also follow ECB 2019-15, which states a goal of the proposed levee is to achieve all four Tolerable Risk Guidelines: 1) Understanding the Risk, 2) Building Risk Awareness, 3) Fulfilling Daily Responsibilities, and 4) Actions to Reduce Risk. The team anticipates that during the QRA, multiple data sources such as inundation maps, first floor elevations, and potential breach models may assist in understanding the flooding that could result in the failure of a project feature.

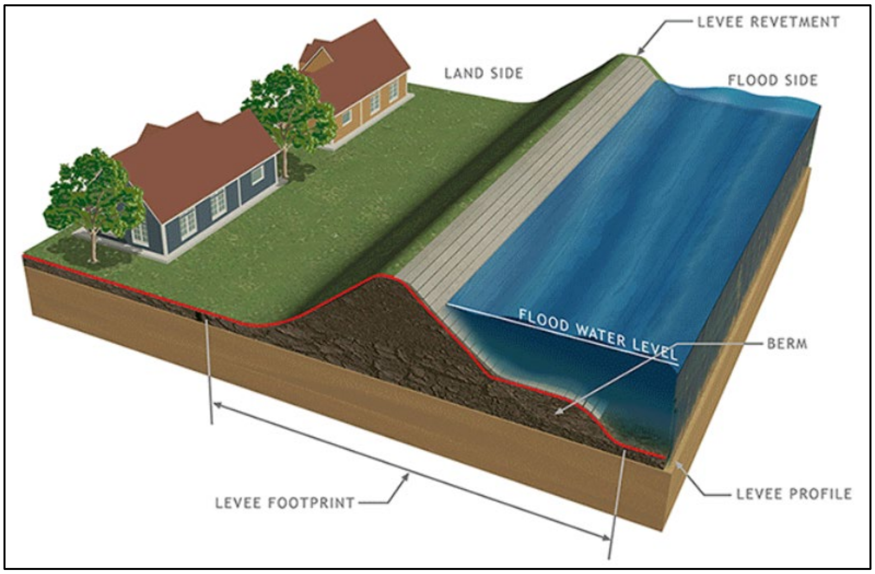

Figure A - 20. Typical Levee (North Atlantic Coast Comprehensive Study Report)

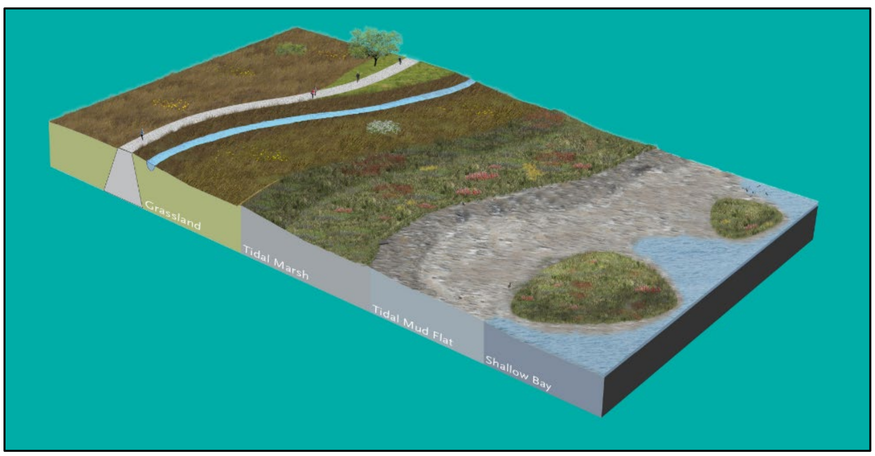

Figure A - 21. Example of Tiered Levee (Aquarium of the Bay)

4.6.3 Elevated Living Shoreline

The elevated living shoreline is similar to the tiered levee since it also consists of three berms; with the top berm set to the design elevation and the lower two berms set to specific elevations to support marsh grass and mangrove plantings which will help protect the structure toe from scour and provide additional protection from the smaller surge events. Unlike the tiered levee, the top berm of the elevated living shoreline consists of a concrete stem wall to provide last resort protection from erosion. The exact depth of the stem wall will be refined within Pre-Construction Engineering & Design (PED) for each model area through additional site-specific modeling. The engineering team designed toe protection in compliance with EM 1110-2-1614 to determine the apron width and stone diameter. Within the toe protection a sediment tube, surrounded by filter fabric, will be placed to support mangrove plantings, which provides additional stabilization to the berm. The study followed EM 1110-2-

1913 for slope design; the engineering team assumed sandy sediment at the measure locations and therefore a shallower slope of 1V:4H. **Figure A - 22** displays an example of an elevated living shoreline.

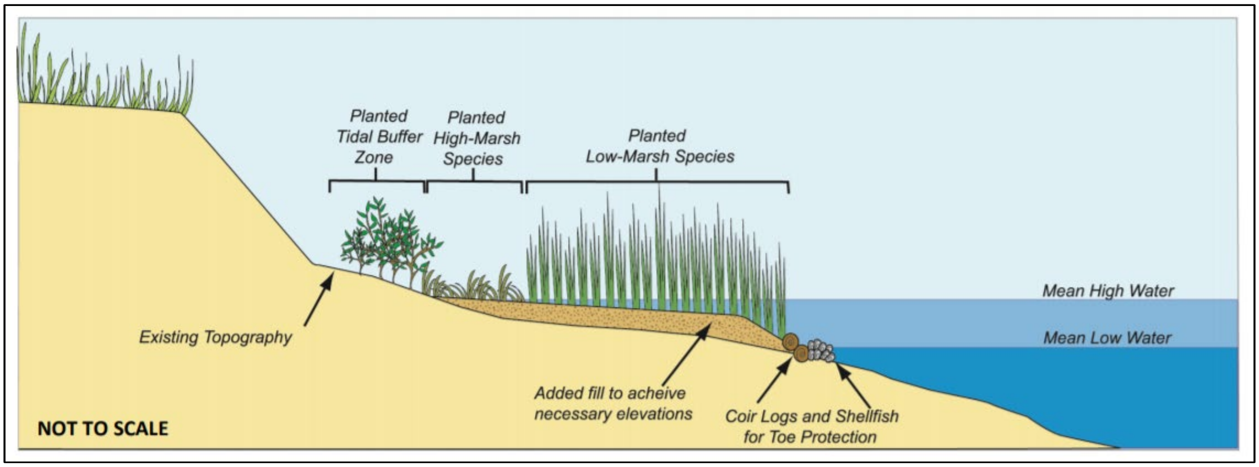

Figure A - 22. Example of Elevated Living Shoreline without a Stem Wall (from Conservation Gateway)

4.6.4 Breakwaters

In addition to storm surge, storm-driven waves also affect portions of the study area. In particular, the northern shoreline of WSJB-3 faces the San Juan Harbor Inlet entrance where waves propagate from offshore and onto the existing shoreline, causing significant damage through wave runup and overtopping. The engineering team designed breakwater measures to reduce damages from wave energy as well as reduce the influence of waves to the total water level due to wave setup and wave runup. The study designed two types of breakwater measures: breakwaters consisting solely of rock and emergent island breakwaters consisting of fill surrounded by rock revetment.

To protect an extended length of shoreline along WSJB-3 (Cataño) the engineering team decided to implement detached segmented breakwaters to cost-effectively reduce the impacts from waves on the shoreline, while maintaining vessel navigation through and around the breakwater system. The designed breakwater maintained space to avoid impacts to the existing ferry terminal track, which travels from the Cataño shoreline to Old San Juan. To reduce impacts of sand accumulation into the federal channel the engineering team maintained a minimum distance of 400 ft (from the closest breakwater to the start of the federal channel). Additionally, the engineering team designed the breakwaters far enough offshore to avoid a congregation of sediment on the landward side; also known as a tombolo. The study team recommends additional analysis within PED to confirm the breakwaters will not negatively affect the federal channel through sand accumulation.

The team determined the desired gap width using diffraction diagrams within the SPM (SPM, 1984) to maintain a diffracted wave height of 20% to 40% of the incident wave height. To avoid the development of a tombolo on the landward side of the designed breakwater, the engineering team followed equations from Dean & Dalrymple, 2004 to determine the suitable offshore distance of the breakwaters. Following EM 1110-2-2904, the engineering team determined the appropriate stone size for the design wave height of a 1% AEP event using Hudson’s equation. The revetment thickness surrounding the emergent island was determined following EM 1110-2-1614. The design included a marine mattress based on, Hughes, 2006, to encompass the entire bottom width of the structure. **Figure A - 23** illustrates an example of a typical breakwater.

To determine the reduction in wave setup caused by the breakwater measure for FWP model runs within G2CRM, ERDC modeled the breakwater measure with two design elevations within ADCIRC/STWAVE. The engineering team selected the design elevations based on the existing conditions (0 ft-PRVD02) and storm surge conditions (6 ft-PRVD02). The engineering team chose the 50-yr flood event with MHHW and intermediate SLC to the end of federal participation to represent the storm surge condition. By comparing both the seaward and landward savepoints of the breakwater, the team calculated the difference in the total water level to determine wave setup contribution to the total water level.

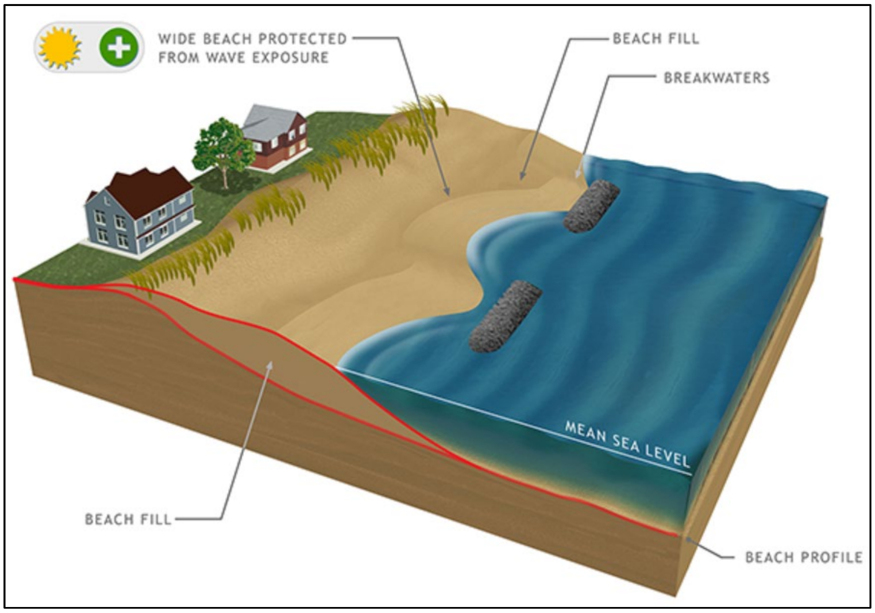

Figure A - 23. Typical Breakwater (North Atlantic Coast Comprehensive Study Report)

4.6.5 Storm Surge Barriers

The PDT considered two types of storm surge barriers, sluice gates and sector gates. The engineering team designed the sluice gates to at locations with small channel entrances, less than 150 feet, as an alternative to prevent flooding from storm surge. The engineering team appropriately sized pumps based on cited outflows from the desired channels or existing pump capacities to dissipate inland drainage during gate closures. The sluice gates considered for this study are vertical rising sluice gates, which are usually metal plates that are typically controlled by machinery. **Figure A - 24** displays an example of an existing sluice gate at the Malaria Canal entrance in Puerto Rico.

Figure A - 24. Typical Vertical Sluice Gate (Malaria Canal)

The engineering team proposed sector gates at locations with large channel entrances, larger than 150 feet. A sector gate consists of two gates in the shape of a quarter circle that rotate around two vertical axes. The gates are generally built with steel frames to distribute the loading. Typically, when the gates are open they are stored along the channel edge (Kerssens et al., 1989). **Figure A - 25** displays an example of a typical sector gate. Sector gates have the ability to quickly open or close with ease. They can span great widths and remain partially open for extended periods if needed. The main disadvantages are the large footprint of the structure itself and the significant cost of construction. For design considerations and general guidance, the engineering team followed Mooyaart et al., 2014. The more robust sector gates would be deployed at the two inlet entrances within San Juan Harbor, El Boquerón Inlet and the San Juan Harbor Inlet. These large sector gates will prevent all model areas from storm surge when closed, but when open they will not protect against tides or SLC.

Figure A - 25. Typical Sector Gate (Bayou Dupre, Sector Gate)

4.6.6 Inland Hydrology Measures

According to the Engineering Manual 1110-2-1413, an interior area is defined as the area protected from direct riverine, lake, or tidal flooding by levees, floodwalls or seawalls, and low depression or natural sinks. Management measures, such as a levee or a wall, associated with an interior area is generally referred to as the project alignment. The project alignment excludes floodwater originating from the exterior, but normally does not directly alleviate flooding that may subsequently occur from interior rainfall runoff. In fact, the project alignment can often aggravate the problem of interior flooding by blocking drainage outlets.

The engineering team computed interior drainage calculations using the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS). To simulate the hydrologic precipitation-runoff relationship in dendritic watersheds the U.S. Army Corps of Engineers developed HEC-HMS version 4.3 software. To simulate the upland watersheds of the San Juan Metro area to estimate runoff volumes and flow hydrographs for use in the feasibility level design of interior drainage structures the engineering team used HEC-HMS. This analysis ensured the identification of the appropriate interior drainage components to handle residual flooding due to the proposed project features. The level of detail provided is commensurate with the study purpose and other technical elements as described in EM 1110-2-1413, Hydrologic Analysis of Interior Areas. The analysis here-in aided in identifying the type, size, and configuration of the components, as well as study measure locations and costs necessary for the economic and real estate evaluation. The engineering team is finalizing operational requirements but is expected to be complete by the final Feasibility Report.

4.6.6.1 HEC-HMS Model Parameters

The engineering team calculated the sub-basins for the HEC-HMS model from the DEM using the ArcMap Geographic Information System (GIS) software, HEC-GeoHMS add-in. The team also compared sub-basins delineations against the 8-digit watershed boundary dataset from Natural Resources

Conservation Service (NRCS) for confirmation of proper delineation. The Loss Method, selected within HEC-HMS, for the sub-basin determines the infiltration calculations used for that sub-basin. The team selected the Soil Conservation Services (SCS) Curve Number Loss as the Loss Method for the HEC-HMS model set-up because of its relative ease of use as well as the availability of land use and soil property data for the watershed. The Soil Conservation Services (now the NRCS) curve number method implements the curve number methodology for incremental losses. The SCS curve number method was used to estimate the amount of runoff potential from the rainfall event based on the relationship between soil type, land use and hydrologic soil conditions. This method is applicable for single storm event modeling.

Since little to no observed hydraulic data such as stage or flow gauge readings are available within the watershed, curve numbers were derived using a previously calibrated model for the Rio Puerto Nuevo (RPN) Basin (RPN H&H Analysis, 2019), which is adjacent to (or includes) many of the watershed within the San Juan Metro Area.

The Transform Method for the sub-basin determines the runoff calculations used for a particular sub-basin. To represent the response of each sub-basin to the rain event, a hydrograph for each sub-basin based on the time of concentration must be calculated. The time of concentration is defined as the time it takes water to travel from the hydraulically furthermost point in the watershed to the outlet. The Transform Method selected to represent the runoff within the watershed was the Clark Unit Hydrograph methodology, which requires the identification of a time of concentration and storage coefficient.

There are several formulas available to estimate the time of concentration. A common formula is the TR-55 Methodology (USDA, 1986). It uses parameters for three different flow characteristics for sheet flow, shallow concentrated flow, and channel flow to compute the time of concentration for each sub-basin. Parameters such as the flow length, slope, and Manning's roughness coefficient are used to determine the adequate time. The engineering team computed these parameters in ArcMap and used them in the computation of the time of concentration per sub-basin, adjusting as necessary as a control parameter to ensure the time was sufficient to describe the hydrologic conditions present.

The team estimated the storage coefficients based on the dimensionless ratio equations found in EM 1110-2-1417, which relates the time of concentration to an estimated storage coefficient based on the watershed characteristics. The ratios are representative of varying basin responses due to slope and runoff potential (i.e. urban versus undeveloped areas) and are computed using the time of concentration. The engineering team estimated initial storage coefficients to be equal to the time of concentration, which was based on the watershed having a minimal to moderate slope.

For the meteorological input, the engineering team developed frequency-based hypothetical design storms using point precipitation from NOAA Atlas 14 Precipitation-Frequency Atlas of the United States, Volume 3 Version 4.0 in conjunction with Chapter 4 of the U.S. Department of Agriculture (USDA) NRCS Part 630 Hydrology National Engineering Handbook. The engineering team selected 100-year, 24-hour storm event for design of interior flood features.

4.6.6.2 HEC-HMS Model Results

The results of the HEC-HMS model helped inform the design of the components necessary to understand the interior facility planning efforts, and proposed measures necessary to realize benefits and mitigate for potential residual damages due to the proposed project features. Depending on the watershed area and feature proposed, the team assumed that either gravity or pumped drainage would be required to mitigate for the residual flooding. The determination was made following a characterization of the basin, for example, a more channelized or concentrated flow regime (which results in higher peak flow values) would likely require a pump, whereas sheet flow from local drainage may require a culvert or gravity discharge structure. For the purpose of the draft Feasibility Report, the team designed the gravity features to provide interior flood relief under low exterior stages (gravity conditions). The engineering team assumed the local storm drainage system would function essentially as it did without a levee or seawall in place for floods up to the selected design capacity of the 100-year, 24-hour event. The gravity drainage structures mainly consist of culverts with measures such a duck-bill or flap gate proposed on the downstream condition to limit bi-directional flow and backwater impacts to the interior drainage areas. **Figure A - 26** displays an example of a culvert with a flap gate. The team performed culvert calculations assuming partial pipe flow under unsubmerged conditions, which is a conservative assumption as it limits peak discharge capacity. It is assumed, at this time, that areas with proposed protection measures including levees and/or floodwalls would have multiple culvert outlets, equally spaced along the length of the proposed feature, to aid in the collection of sheetflow. Culverts ranged between 2-6 ft in diameter, depending on the drainage area and required flow capacity. The pump capacities were computed similarly, however the pump mix is made up of multiple 50-100 cfs pump sizes, necessary to aid in the drainage of multiple design storm scenarios less than the 100-year, 24-hour storm.

Figure A - 26. Example of a Culvert with a Flap Gate

The current analysis did not consider fully the coincidental probability of surge and excessive rainfall-runoff for flood routings and treated the two as independent variables. Following selection of the TSP, additional analysis will occur to ensure the proposed design features do not exacerbate flooding during a surge plus rainfall event. Examples of potential design refinements include addition of a collector drain or excavated detention storage feature on the interior drainage side that could collect the rainfall until the tailwater conditions from surge recede. Areas that are currently susceptible to interior flooding (WSJB-2, WJSB-3, and Condado) currently have pump stations included in the project design that would operate under normal and exacerbated surge conditions. Additionally, development of frequency functions may also be completed after selection of the TSP, to aid in the development of operational constraints and further refined costs.

4.7 Study Alternatives

After developing a better understanding of the storm-induced problems within each model area, the PDT developed and assessed an array of alternatives within G2CRM to determine the alternative that best addresses the primary study objective. Each alternative includes a single or combination of measures to protect each model area. The engineering team designed all alternatives to the 90% CI of the 1% AEP event with MHHW and the intermediate SLC out to the end of the assumed federal participation (2079) as referenced in **Section 4.5**. Note that the images displaying potential flood locations within this section are DEM elevations below the design elevation within each model area and are not exact flood extents in the model area. The engineering team assumed the alternatives would mimic the location of the existing shoreline and the proposed alternatives will be built in front of existing structures unless otherwise noted.

4.7.1 WSJB-1A

WSJB-1A contains two sources of potential coastal flooding: through the Caño Aguas Frias along the south side of WSJB-1A and from the Atlantic Ocean into the north side of the model area. **Figure A - 27** displays the DEM elevations below the specified design elevation for this model area. Coastal flooding will first occur on the south side of WSJB-1A through the Caño Aguas Frias and eventually propagate into the north side of the model area from the Atlantic Ocean.

Figure A - 27. DEM Elevations below Design Elevation for WSJB-1A

To mitigate the projected coastal flooding problem along the backside of WSJB-1A, the PDT delineated the alternative listed within **Table A - 12**. **Figure A - 28** displays the floodwall/seawall and levee in red and yellow, respectively. All measures, unless specified otherwise, will be built to the specified design elevation ranging from 4- to 6-ft PRVD02. Originally, the team chose design elevations to protect up to 7- to 9-ft PRVD02 on the backside of WSJB-1A, although to represent the damage reduction, the alternatives along Caño Aguas Frias could only protect up to a design elevation below 6 ft-PRVD02 (the elevation when flooding typically occurs from the Atlantic Ocean). Protection of the oceanside of WSJB-1A is discussed further below. The engineering team designed the alternative to account for inland drainage using culverts along both the seawall and levee.

Alternative 1 consists of a floodwall/seawall along the eastern side of the Caño Aguas Frias. The engineering team designed the floodwall/seawall to be a steel cantilever sheet pile seawall, although some locations of the seawall may transition into a floodwall depending on available room within the canal and amount of wave action in the area. The sheet piles will be driven approximately 25 ft deep and will contain backfill up to the design elevation. The seawall will contain a 2-foot by 2-foot concrete cap and the team assumed no toe protection due to the limited wave action along the Caño Aguas Frias. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall with a total pile length of approximately 55 feet. A standard levee is proposed further west along the north side of the Caño Aguas Frias. The exact sediment type is unknown at the levee locations and therefore the engineering team assumed a slope of 1V:3.5H. Additionally, the study assumed a top width of 12-ft to maintain vehicle access to the levees.

Following economic analysis, the PDT removed WSJB-1A from the study since its benefit-to-cost ratio (BCR) is below 1.0; refer to the Economics Appendix for additional information.

Table A - 12. Alternatives for WSJB-1A

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	Floodwall/Seawall + Levee	Floodwall/Steel Cantilever Sheet Pile Seawall	4,469	10	-
		Standard Levee	5,632	50	-
	Inland Drainage	24" Culverts (Floodwall/Seawall)	15	-	2
		24" Culverts (Levee)	55	-	2

Figure A - 28. Alternative for WSJB-1A

Due to coastal flooding from higher surge events, the engineering team analyzed a proposed revetment that would protect a portion of WSJB-1A from flooding from the Atlantic Ocean. The proposed revetment extends from the northeastern tip of Palo Seco to the terminal groin located adjacent to existing power plant facilities, as shown within **Figure A - 29**. The engineering team estimated the approximate length of the revetment to be 4,650 feet and designed to approximately 10 ft-PRVD02. The team added the construction cost, without real estate and environmental (mitigation) costs, to the total cost of Alternative 1 (levee and seawall) and compared the total to the FWOP damages. This resulted in a BCR close to 1.0. Since it is highly improbable that all residual damages will be reduced during the 50-year planning horizon, as well as the time before construction completion, and the project costs did not account for real estate and environmental mitigation, the addition of a revetment within Alternative 1 for WSJB-1A is not justified.

Figure A - 29. Proposed Revetment Location

4.7.2 WSJB-1B

WSJB-1B contains three sources of potential coastal flooding: through the Caño Aguas Frias along the north side of WSJB-1B, through La Esperanza Park on the east side of the model area, and through Malaria Canal just south of the model area. **Figure A - 30** displays the DEM elevations below the specified design elevation for this model area. Coastal flooding will first occur on the east side of WSJB-1B and higher surge events will flood into the north side of the model area through the Caño Aguas Frias and the south side of WSJB-1B through the Malaria Canal.

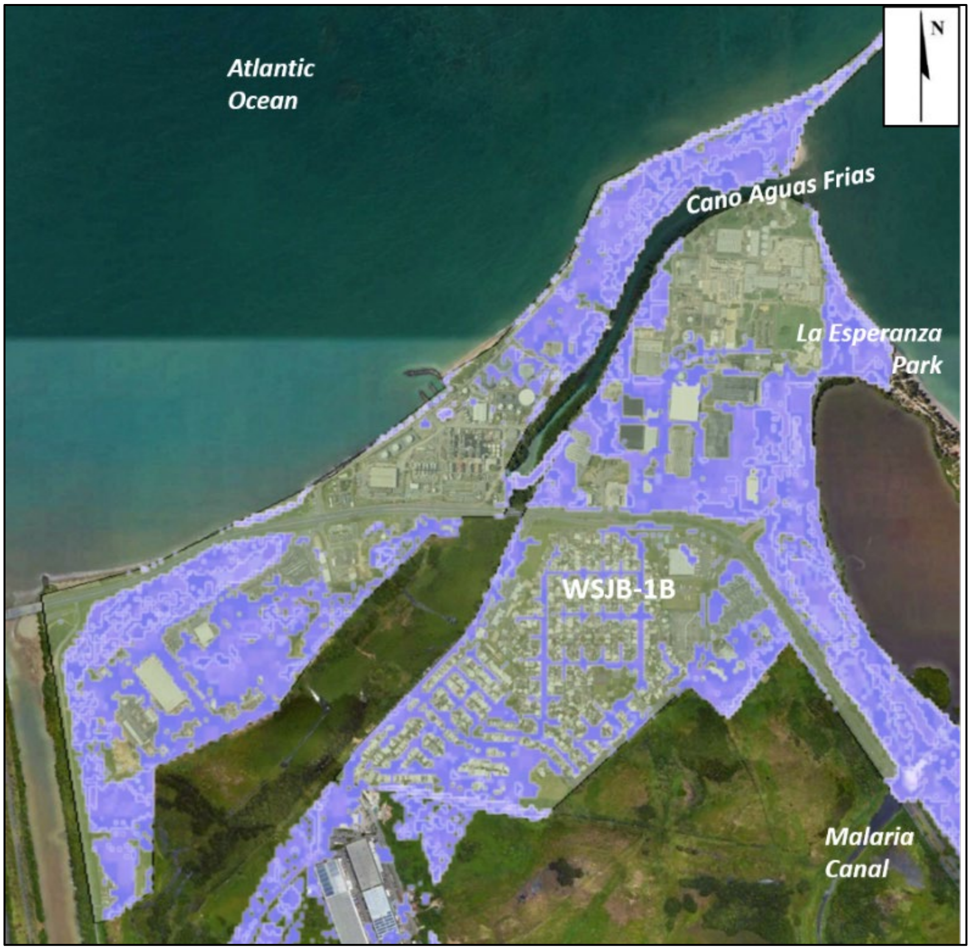

Figure A - 30. DEM Elevations below Design Elevation for WSJB-1B

The PDT developed two alternatives, listed within **Table A - 13**, to reduce the damages from storm surge throughout WSJB-1B. **Figure A - 31** displays Alternative 1**,** with the floodwall/seawall represented in red, the levee in orange, and the elevated living shoreline in green. All measures, unless specified otherwise, will be built to the specified design elevation ranging from 7- to 9-ft PRVD02. The engineering team designed each alternative to account for inland drainage using culverts assuming placement at an even interval along the length of each feature, depending on the measures.

Alternative 1 consists of a floodwall/seawall designed to be a steel cantilever sheet pile seawall, although some locations of the seawall may transition into a floodwall depending on available space to build within the model area and the amount of wave action in the area. The sheet piles will be driven approximately 25 ft deep and contain backfill up to the design elevation. The seawall will contain a 2-foot by 2-foot concrete cap and the team assumed no toe protection at the seawall location due to the limited wave action around WSJB-1B. If it is determined that a floodwall is necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall with a total pile length of approximately 55 feet. The team is proposing standard levees along the western and southern sides of the model area. The exact sediment type is unknown at the levee locations, and therefore the engineering team assumed a slope of 1V:3.5H. Additionally, the study assumed a top width of 12-ft to maintain vehicle access to the levees. The elevated living shoreline located on the eastern side of the model area will consist of three berms, with the first berm set to the specified design elevation and a top width of 5 feet. The slope of the living shoreline will be kept at a 1V:4H on both the landward and seaward sides. A concrete stem wall will be placed within the top berm and will extend from the top of the structure to 2-feet below the existing grade. The second berm will be set to an elevation of 2 ft-PRVD02 and maintain a berm width of 1 foot to support various vegetative species like marsh grass. The third berm will be set to an elevation of -1 ft-PRVD02 and contain a berm width of 3 feet. Toe protection will encompass the entire lower berm, at a width and height of 3 feet with a D_{n50} of 1-foot and unit weight of 147 lb/ft^3. A 1-foot in diameter sediment tube surrounded by filter fabric will be placed within the center of the toe protection to support mangrove plantings to help stabilize the toe of the berm.

Alternative 2 is very similar to Alternative 1, except the elevated living shoreline is replaced with a continuous floodwall/seawall as described in Alternative 1 above.

Following the economic analysis, Alternative 1 provided the highest BCR; refer to the Economics Appendix for additional information.

Table A - 13. Alternatives for WSJB-1B

<table>
<tr><th>Alt #</th><th>Measure</th><th>Description</th><th>Approx. Length (ft)</th><th>Approx. Width (ft)</th><th>Quantity (EA)</th></tr>
<tr><td rowspan="6">1</td><td rowspan="3">Floodwall/Seawall + Levee + Elevated Living Shoreline</td><td>Floodwall/Steel Cantilever Sheet Pile Seawall</td><td>5,584</td><td>10</td><td>-</td></tr>
<tr><td>Standard Levee</td><td>5,764</td><td>50</td><td>-</td></tr>
<tr><td>Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings</td><td>3,422</td><td>80</td><td>-</td></tr>
<tr><td rowspan="3">Inland Drainage</td><td>24" Culverts (Floodwall/Seawall)</td><td>15</td><td>-</td><td>5</td></tr>
<tr><td>24" Culverts (Levee)</td><td>55</td><td>-</td><td>6</td></tr>
<tr><td>24" Culverts (Living Shoreline)</td><td>85</td><td>-</td><td>3</td></tr>
<tr><td rowspan="4">2</td><td rowspan="2">Floodwall/Seawall + Levee</td><td>Floodwall/Steel Cantilever Sheet Pile Seawall</td><td>9,006</td><td>10</td><td>-</td></tr>
<tr><td>Standard Levee</td><td>5,764</td><td>50</td><td>-</td></tr>
<tr><td rowspan="2">Inland Drainage</td><td>24" Culverts (Floodwall/Seawall)</td><td>15</td><td>-</td><td>8</td></tr>
<tr><td>24" Culverts (Levee)</td><td>55</td><td>-</td><td>6</td></tr>
</table>

Figure A - 31. Alternative 1 for WSJB-1A

4.7.3 WSJB-1A & 1B

The PDT developed an additional alternative to protect both WSJB-1A and WSJB-1B listed within **Table A - 14** and displayed within **Figure A - 32**. The alternative consisted of a combination of a sluice gate, floodwall/seawall, levee, and elevated living shoreline displayed in blue, red, orange, and green, respectively. All measures within this alternative will be set to a design elevation ranging from 7- to 9-ft PRVD02. The engineering team designed the alternative to account for inland drainage using culverts along the seawall, levee, and elevated living shoreline measures as well as a pump station, located adjacent to the proposed sluice gate.

The team chose a sluice gate due to the smaller channel entrance at the Caño Aguas Frias with a width of approximately 100 ft. Just upstream along the Caño Aguas Frias, the Palo Seco Power Plant discharges a maximum of 1,011 CFS into the Caño Aguas Frias. The engineering team assumed that the power plant would remain operational during a surge event and the outflow from the power plant would have to be matched in the case of a gate closure. Therefore, the team designed a pump station with a total capacity of 1,100 CFS, in order to exceed the discharge capacity of the Palo Seco Power Plant combined with additional rainfall runoff from a storm event. The team also designed a floodwall/seawall to extend from the sluice gate to tie into higher ground elevation on the north side of the gate. The combination of the sluice gate and floodwall/seawall will prevent coastal flooding into both model areas along the Caño Aguas Frias. The remaining floodwall/seawall, levee, and elevated living shoreline, described in **Section 4.7.2**, will protect the eastern and southern sides of WSJB-1B.

Following economic analysis, the PDT removed this alternative due to a BCR below 1.0; refer to the Economics Appendix for additional information.

Table A - 14. Combined Alternative for WSJB-1A and WSJB-1B

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	Storm Surge Gate + Floodwall/Seawall + Levee + Elevated Living Shoreline	Sluice Gate	100	20	-
		Floodwall/Steel Cantilever Sheet Pile Seawall	2,857	10	-
		Standard Levee	2,825	50	-
		Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings	3,422	80	-
	Inland Drainage	24" Culverts (Floodwall/Seawall)	15	-	3
		24" Culverts (Levee)	55	-	3
		24" Culverts (Living Shoreline)	85	-	4
		100 CFS Pump (Sluice Gate)	-	-	11

Figure A - 32. Alternative 1 for WSJB-1A & WSJB-1B

4.7.4 WSJB-2

The coastal flooding sources into WSJB-2 are through the Malaria Canal and the Caño Aguas Frias. At the entrance of the Malaria Canal, an existing sluice gate remains closed with an approximate top elevation of 2 ft-PRVD02. Storm surge propagates into the model area following the overtopping of the existing sluice gate. **Figure A - 33** displays the DEM elevations below the specified design elevation for this model area. Coastal flooding will occur along the east and west sides of the Malaria Canal and propagate into the model area as the surge increases. Larger storm surge events can also flood through the Caño Aguas Frias and into the northwest side of WSJB-2.

Figure A - 33. DEM Elevations below Design Elevation for WSJB-2

To mitigate the projected problem along the backside of WSJB-2, the PDT delineated the alternatives listed in **Table A - 15**. **Figure A - 34** displays Alternative 1, which consists of a floodwall/seawall and levee shown in red and yellow, respectively. All measures, unless specified otherwise, will be built to the specified design elevation ranging from 7- to 9-ft PRVD02. The engineering team also designed to account for inland drainage using either culverts or pumps depending on the measures proposed within each alternative.

Alternative 1 consists of a floodwall/seawall along the east and west sides of the Malaria Canal. The team designed the floodwall/seawall to be a steel cantilever sheet pile seawall, although some locations of the seawall may transition into a floodwall depending on available room within the canal and the amount of wave action in the area. The sheet piles will be driven approximately 25 ft deep and will contain backfill up to the design elevation. The seawall will contain a 2-foot by 2-foot concrete cap and the team assumed no toe protection since there is no wave action along the model area. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall with a total pile length of approximately 55 feet. The team proposed standard levees at locations around the model area that have sufficient room to support the measure, as shown within **Figure A - 34**. The exact sediment type is unknown at the levee locations and therefore the engineering team assumed a slope of 1V:3.5H. Additionally, the study assumed a top width of 12-ft to maintain vehicle access to the levees.

Alternative 2 is the same as Alternative 1, except the engineering team designed a horizontal (tiered) levee instead of a standard levee. The horizontal (tiered) levees consisted of the same slope and top width as the standard levees and were designed to consist of three berms, with the lower two berms at 2-ft and 0-ft PRVD02 to support marsh grass and mangrove plantings which will help stabilize the levee. Additionally, the lower two berms will consist of a 5-ft top elevation.

Alternatives 3 and 4 consist of a sluice gate that will extend approximately 50-ft across the Malaria Canal. Since the existing sluice gate remains closed, pumps are currently installed to discharge the inland rainfall runoff. The engineering team assumed that the existing FEMA pumping capacity at Malaria Canal will have to be maintained, therefore the design includes three 50 CFS pumps and one 100 CFS pump. A floodwall/seawall will tie into high elevations on either side of PR-165 from the sluice gate. The western side of the model area will be protected by either a standard or a horizontal (tiered) levee, as described above, depending on the alternative.

Lastly, alternative 5 is a buyout of all assets within WSJB-2 below the design elevation.

Following economic analysis, Alternative 3 provided the highest BCR; refer to the Economics Appendix for additional information.

Table A - 15. Alternatives for WSJB-2

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	Floodwall/Seawall + Levee	Floodwall/Steel Cantilever Sheet Pile Seawall	8,456	10	-
		Standard Levee	9,996	50	-
	Inland Drainage	24" culverts (WSJB_2A Floodwall/Seawalls)	15	-	4
		24" culverts (WSJB_2A Levees)	55	-	9
		36" culverts (WSJB_2B Floodwall/Seawalls)	15	-	1
		36" culverts (WSJB_2B Levees)	55	-	1
2	Floodwall/Seawall + Horizontal Levee	Floodwall/Steel Cantilever Sheet Pile Seawall	8,456	10	-
		Horizontal (Tiered) Levee	9,996	70	-
	Inland Drainage	24" culverts (WSJB_2A Floodwall/Seawalls)	15	-	4
		24" culverts (WSJB_2A Levees)	55	-	9
		36" culverts (WSJB_2B Floodwall/Seawalls)	15	-	1
		36" culverts (WSJB_2B Levees)	55	-	1
3	Storm Surge Gate + Floodwall/Seawall + Levee	Sluice Gate	41	20	-
		Floodwall/Steel Cantilever Sheet Pile Seawall	1,043	10	-
		Standard Levee	3,245	50	-
	Inland drainage	100 CFS Pump (Sluice Gate)	-	-	1
		50 CFS Pump (Sluice Gate)	-	-	3
4	Storm Surge Gate + Floodwall/Seawall + Horizontal Levee	Sluice Gate	41	20	-
		Floodwall/Steel Cantilever Sheet Pile Seawall	1,043	10	-
		Horizontal (Tiered) Levee	3,245	70	-
	Inland drainage	100 CFS Pump (Sluice Gate)	-	-	1
		50 CFS Pump (Sluice Gate)	-	-	3
5	Buyout	Buyout of Properties Below Design Elevation	-	-	-

Figure A - 34. Alternative 1 for WSJB-2

4.7.5 WSJB-3

Storm surge along with the influence of waves will cause flooding into WSJB-3 through the San Juan Harbor. **Figure A - 35** displays the DEM elevations below the specified design elevation for this model area. Coastal flooding will initially occur through the San Fernando Canal, which is located behind Cataño and within the northeast region of the model area. As the surge increases, the flooding will propagate further into WSJB-3 through the north and southeast sides of the model area. Storm generated waves will pass through the San Juan Harbor Inlet causing increases in flooding due to wave setup and wave runup.

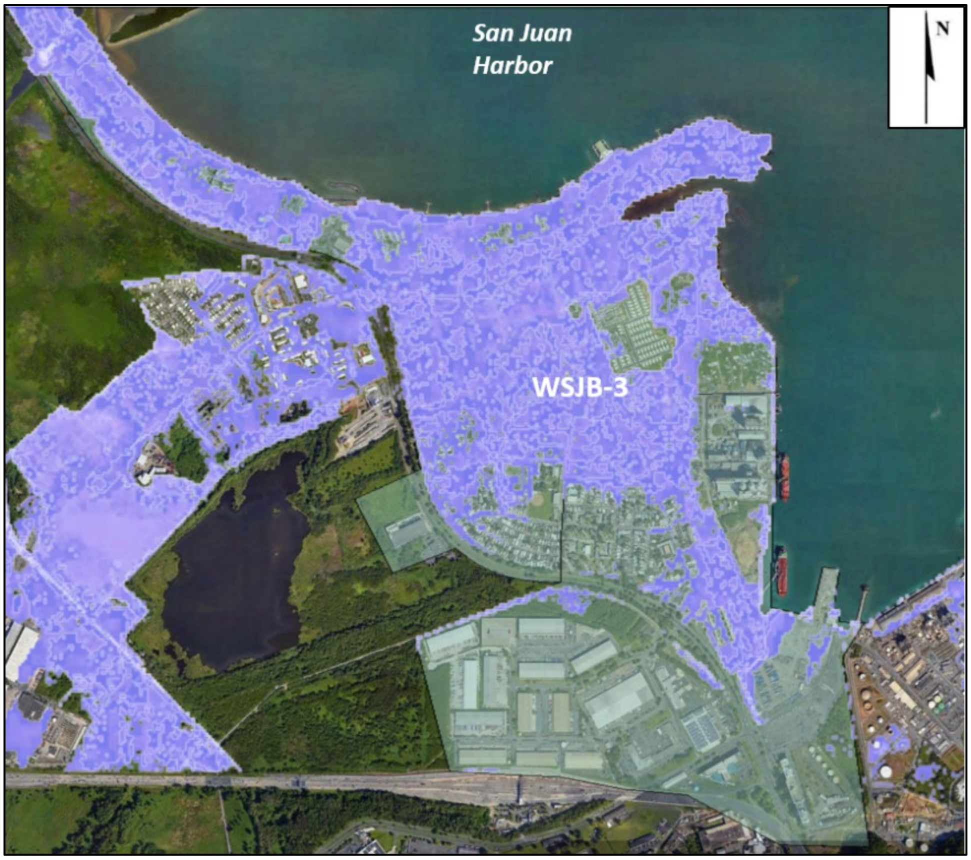

Figure A - 35. DEM Elevations below Design Elevation for WSJB-3

To protect WSJB-3 from both surge and waves the PDT delineated the alternatives listed within **Table A - 16**. **Figure A - 36** displays an example of Alternative 2**,** which consists of a floodwall/seawall and rock breakwater shown in red and purple, respectively. All measures, unless specified otherwise, will be built to the specified design elevation ranging from 7- to 9-ft PRVD02. The engineering team also designed to account for inland drainage within all alternatives using both culverts and pumps. The culverts will be placed in various locations throughout the model area and vary in width depending on the measure type. Pumps will be placed within the northeastern portion of the model area to assist with the outflow of rainfall at the lowest elevation region within the model area.

Alternative 1 consists of a floodwall/seawall protecting WSJB-3A and the east side of WSJB-3B, WSJB-3C, and WSJB-3D. The team designed the floodwall/seawall to be a steel cantilever sheet pile seawall although some locations of the seawall may transition into a floodwall depending on available room within the canal and the amount of wave action in the area. The sheet piles will be driven approximately 25 to 40 feet deep to maintain a depth driven of twice the difference between the design elevation and the seaward elevation. Within WSJB-3A and WSJB-3B the sheet pile will be driven 25 feet deep, within WJSB-3C it will be driven 30 feet deep, and within WSJB-3D it will be driven 40 feet deep. The seawall will contain a 2-foot by 2-foot concrete cap with backfill up to the design elevation as well as toe protection with a D_{n50} of approximately 2-ft with a unit weight of 147 lb/ft^3. The toe protection will have an approximate width of 8-ft and a height of 4-ft. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5

ft along the centerline of the wall with a total pile length of approximately 55 feet. In addition to the floodwall/seawall, Alternative 1 consists of a higher T-Wall along the north side of the model area. The T-Wall is designed to an elevation of 14 to 17 ft-PRVD02 to protect against the 2% wave runup. The T-Wall will consist of two piles spaced approximately 5 ft along the centerline of the wall with a total pile length of approximately 55 feet. The T-Wall will contain backfill up to an elevation of 5 ft-PRVD02 as well as toe protection with a D_{n50} of approximately 2.6 feet with a unit weight of 147 lb/ft^3. The toe protection will have an approximate width of 12 feet and a height of 5 feet.

Instead of a higher T-Wall, Alternative 2 consists of 11 segmented rock breakwaters offshore of WSJB-3B to protect the Cataño shoreline from the incoming wave action. The breakwaters will be placed approximately 656 ft (200 m) offshore with a top elevation of 6 ft-PRVD02. Each breakwater will be 328 ft (100 m) long and between each breakwater there will be a 164-foot (50 m) gap width. The top width of each will be 10 ft with an approximate bottom width of 82 ft. An approximate slope of 1V:2H will be used and a 10- to 12-inch marine mattress will be placed at the bottom of each breakwater. The rock will have a D_{n50} of approximately 2.8- to 3.4-ft with a unit weight of 147 lb/ft^3. A floodwall/seawall, described above within Alternative 1, will protect all areas depicted within **Figure A - 36** from storm surge.

Alternative 3 is essentially the same as Alternative 2, except it uses an emergent island breakwater surrounded by rock revetment to dissipate the waves instead of a rock breakwater. The dimensions of the fill are the same as the rock breakwater except the rock revetment around the outside of the fill will consist of a D_{n50} of approximately 2.8- to 3.4-ft with a unit weight of 147 lb/ft^3 and have a 6-foot layer thickness. A 10- to 12-inch marine mattress will be placed underneath the rock on the 1V:2H slope sides of the fill.

Alternative 4 consists of the rock breakwater with a recreational seawall directly behind the breakwater within a portion of WSJB-3B. The remainder of this alternative will consist of the floodwall/seawall as described above. The recreational seawall is essentially the same, as the standard steel cantilever sheet pile seawall described above, except it will contain additional fill and a 4-foot by 2-foot concrete cap.

Alternative 5 consists of the rock breakwater, described above, and an elevated living shoreline in addition to the floodwall/seawall. The living shoreline will be placed along the northeastern-facing shoreline of WSJB-3A. The remainder of this alternative will consist of the floodwall/seawall as described above. The elevated living shoreline will consist of three berms, with the first berm set to the specified design elevation and a top width of 5 ft and the slope will be kept at a 1V:4H on both the landward and seaward sides of the living shoreline. A concrete stem wall will be placed within the top berm and will extend from the top of the structure to 2-feet below the existing grade. The second berm will be set to an elevation of 2 ft-PRVD02 and maintain a berm width of 1 foot to support various vegetative species like marsh grass. The third berm will be set to an elevation of -1 ft-PRVD02 and contain a berm width of 4 feet. Toe protection will encompass the entire bottom berm width of 4 ft and berm height of 3 ft, with a D_{n50} of 2 ft and a unit weight of 147 lb/ft^3. A 1-ft diameter sediment tube surrounded by filter fabric will be placed within the center of the toe protection to support mangrove plantings to help stabilize the toe of the berm.

Following economic analysis, Alternative 5 provided the highest BCR; refer to the Economics Appendix for additional information. The exact location of the proposed floodwall/seawall along the backside of WSJB-3B (through the canal) may be refined prior to final Feasibility Report.

Table A - 16. Alternatives for WSJB-3

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	Floodwall/Seawall + High T-Wall	Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	14,699	20	-
		High T-Wall with Toe Protection	8,261	62	-
	Inland Drainage	24" Culverts (WSJB_3A Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B T-Wall)	67	-	3
		24" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		72" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		100 CFS Pump (WSJB_3C)	-	-	3
		50 CFS Pump (WSJB_3C)	-	-	1
		24" Culverts (WSJB_3D Floodwall/Seawall)	25	-	3
2	Floodwall/Seawall + Breakwater	Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	22,960	20	-
		6-ft Rock Breakwater	3,608	82	-
	Inland Drainage	24" Culverts (WSJB_3A Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B Floodwall/Seawall)	25	-	4
		24" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		72" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		100 CFS Pump (WSJB_3C)	-	-	3
		50 CFS Pump (WSJB_3C)	-	-	1
		24" Culverts (WSJB_3D Floodwall/Seawall)	25	-	3
3	Floodwall/Seawall + Breakwater	Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	22,960	20	-
		6-ft Emergent Island Breakwater with Rock Revetment	3,608	94	-
	Inland drainage	24" Culverts (WSJB_3A Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B Floodwall/Seawall)	25	-	4
		24" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		72" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		100 CFS Pump (WSJB_3C)	-	-	3
		50 CFS Pump (WSJB_3C)	-	-	1
		24" Culverts (WSJB_3D Floodwall/Seawall)	25	-	3

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
4	Floodwall/Seawall + Rec Seawall + Breakwater	Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	17,677	20	-
		Recreational Seawall with Toe Protection	5,284	75	-
		6-ft Rock Breakwater	3,608	82	-
	Inland drainage	24" Culverts (WSJB_3A Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B Floodwall/Seawall)	25	-	2
		24" Culverts (WSJB_3B Rec Seawall)	80	-	2
		24" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		72" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		100 CFS Pump (WSJB_3C)	-	-	3
		50 CFS Pump (WSJB_3C)	-	-	1
		24" Culverts (WSJB_3D Floodwall/Seawall)	25	-	3
5	Floodwall/Seawall + Elevated Living Shoreline + Breakwater	Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	20,998	20	-
		Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings	1,962	80	-
		6-ft Rock Breakwater	3,608	82	-
	Inland drainage	24" Culverts (WSJB_3A Floodwall/Seawall)	25	-	1
		24" Culverts (WSJB_3B Floodwall/Seawall)	25	-	4
		24" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		72" Culverts (WSJB_3C Floodwall/Seawall)	25	-	2
		100 CFS Pump (WSJB_3C)	-	-	3
		50 CFS Pump (WSJB_3C)	-	-	1
		24" Culverts (WSJB_3D Floodwall/Seawall)	25	-	3

Figure A - 36. Alternative 2 for WSJB-3

4.7.6 WSJB-4

The sources of coastal flooding into WSJB-4 are from the Bechara Channel, the San Juan Harbor, and the Margarita Canal. **Figure A - 37** displays the DEM elevations below the specified design elevation for this model area and **Figure A - 38** displays the model areas within WSJB-4 for reference. The model area will initially flood through the Bechara Channel, which is a tidally influenced channel that goes through the center of the model area from the San Juan Harbor on the north side, underneath the port, and out the south side of the model area. As the storm surge increases, the coastal flooding will propagate further into the interior of WSJB-4A and WSJB-4B through the Bechara Channel. The San Juan Harbor will produce coastal flooding into the northeast region into WSJB-4B and the Margarita Canal will flood the south side of the model area into WSJB-4A and WSJB-4B.

Figure A - 37. DEM Elevations below Design Elevation for WSJB-4

To mitigate the storm surge flooding into WSJB-4, the PDT delineated the alternatives listed within **Table A - 17**. **Figure A - 38** displays an example of Alternative 2**,** which consists of a floodwall/seawall and levee shown in red and orange, respectively. All measures, unless otherwise specified, will be built to the specified design elevation ranging from 8- to 10-ft PRVD02. The engineering team also designed to account for inland drainage within all alternatives using culverts. The culverts will be placed in various locations throughout the model area and vary in width, depending on the measure type.

Alternative 1 consists of a floodwall/seawall at all measure locations depicted in **Figure A - 38**. The team designed the floodwall/seawall to be a steel cantilever sheet pile seawall, although some locations of the seawall may transition into a floodwall depending on available room within the canal and the amount of wave action in the area. The sheet piles will be driven approximately 25 deep to maintain a depth driven of twice the difference between the design elevation and the seaward elevation. The seawall will contain a 2-foot by 2-foot concrete cap and the team assumed no toe protection since there is no wave action along the model area. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall, with a total pile length of approximately 60 feet.

In addition to the floodwall/seawall, Alternative 2 consists of a levee along the south side of WSJB-4A. The exact sediment type is unknown at the levee location, and therefore the engineering team assumed a slope of 1V:3.5H. Additionally, the study assumed a top width of 12-ft to maintain vehicle access to the levee. The team designed the floodwall/seawall as previously discussed in Alternative 1.

Following economic analysis, Alternative 2 provided the highest BCR; refer to the Economics Appendix for additional information.

Within WSJB-4, the engineering team assumed all past and present Rio Puerto Nuevo (RPN) Flood Risk Management (FRM) project components are completed and in-service. There are two RPN FRM project components located within the model area: the Rio Puerto Nuevo Margarita Channel improvements (currently under construction) and the Bechara Industrial Area (previously constructed). The RPN FRM project was designed as an inland flood-risk management project and was not designed to address storm surge inundation. The Bechara Industrial Area was designed in the early 2000s and constructed in the 2012 timeframe. The current feasibility study assesses risk related to storm surge and has identified vulnerability within the Bechara Channel and therefore proposed seawall/floodwall alternatives along the Bechara Channel. The Margarita Channel improvements are currently ongoing, however the team confirmed the design elevations are comparable to the design elevation for WSJB-4 and determined that no measure is required at the RPN Margarita Channel location.

Table A - 17. Alternatives for WSJB-4

<table>
<tr><th>Alt #</th><th>Measure</th><th>Description</th><th>Approx. Length (ft)</th><th>Approx. Width (ft)</th><th>Quantity (EA)</th></tr>
<tr><td rowspan="3">1</td><td>Floodwall/Seawall</td><td>Floodwall/Steel Cantilever Sheet Pile Seawall</td><td>7,697</td><td>10</td><td>-</td></tr>
<tr><td rowspan="2">Inland Drainage</td><td>24" culverts (WSJB_4A)</td><td>15</td><td>-</td><td>7</td></tr>
<tr><td>24" culverts (WSJB_4B)</td><td>15</td><td>-</td><td>4</td></tr>
<tr><td rowspan="5">2</td><td rowspan="2">Floodwall/Seawall + Levee</td><td>Floodwall/Steel Cantilever Sheet Pile Seawall</td><td>6,178</td><td>10</td><td>-</td></tr>
<tr><td>Standard Levee</td><td>1,519</td><td>50</td><td>-</td></tr>
<tr><td rowspan="3">Inland Drainage</td><td>24" culverts (WSJB_4A Floodwall/Seawall)</td><td>15</td><td>-</td><td>6</td></tr>
<tr><td>24" culverts (WSJB_4A Levee)</td><td>55</td><td>-</td><td>1</td></tr>
<tr><td>24" culverts (WSJB_4B Floodwall/Seawall)</td><td>15</td><td>-</td><td>4</td></tr>
</table>

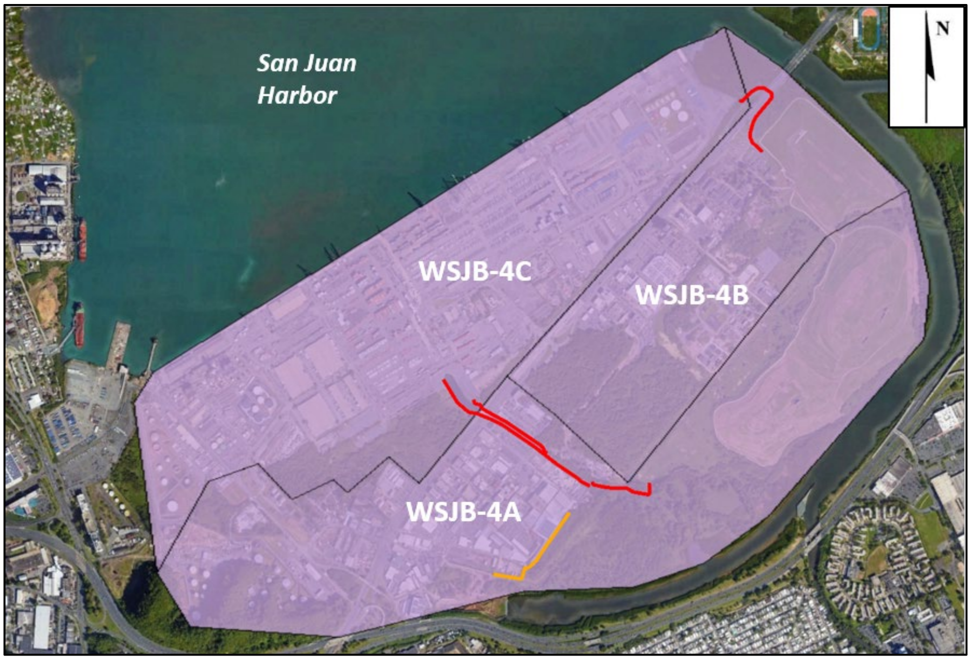

Figure A - 38. Alternative 2 for WSJB-4

4.7.7 CL

The sources of coastal flooding into Condado Lagoon are from the Atlantic Ocean. **Figure A - 39** displays the DEM elevations below the specified design elevation for this model area. The model area will initially flood through the northeastern side, and as the storm surge increases, the flooding will propagate further east as well as towards the north side of the model area. The south side of Condado Lagoon will flood during the higher surge events.

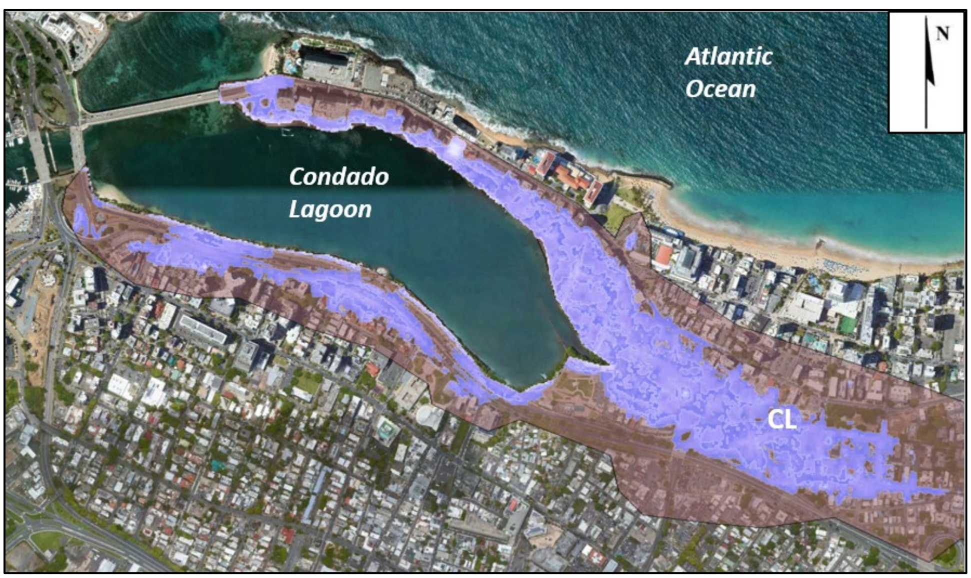

Figure A - 39. DEM Elevations below Design Elevation for CL

To protect Condado Lagoon, the PDT delineated the alternatives listed within **Table A - 18**. **Figure A - 40** displays an example of Alternative 1, which consists of a floodwall/seawall shown in red. All measures, unless specified otherwise, will be built to the specified design elevation ranging from 7- to 9-ft PRVD02. The engineering team designed each alternative to account for inland drainage using both culverts and pumps. The culverts will be placed in various locations throughout the model area and vary in width depending on the measure type. Pumps will be placed within the eastern side of the model area, to assist with the outflow of rainfall at the lowest elevation region within the model area. If necessary, a retention basin could be implemented to support the storage of rainfall runoff on the eastern side of Condado Lagoon. The alternative proposed within Condado Lagoon will provide risk reduction due to storm surge and inherently protect from nuisance tidal flooding, although no benefits related to tidally influenced nuisance flooding were calculated.

Alternative 1 consists of a floodwall/seawall and is depicted within **Figure A - 40**. The team designed the floodwall/seawall to be a steel cantilever sheet pile seawall with king piles, although some locations of the seawall may transition into a floodwall depending on available room within the canal and the amount of wave action in the area. The engineering team chose king piles in this area due to the uncertainty in the depth of rock. The closest geotechnical information indicates rock could be between 25 to 35 feet below the surface, and a steel cantilever seawall would have to be driven down 30 feet below the surface. The sheet piles will be driven approximately 25 deep and additionally supported by the king piles, which will be driven to an elevation of -46 ft-PRVD02. The seawall will contain a 2-foot by 2-foot concrete cap with backfill up to the design elevation, as well as toe protection with a D_{n50} of approximately 2-ft and a unit weight of 147 lb/ft^3. The toe protection will have an approximate width of 8-ft and a height of 4-ft. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall with a total pile length of approximately 55 feet. The same toe protection described above for the seawall would be used if a T-Wall is necessary.

Alternative 2 contains the same floodwall/seawall described above within Alternative 1, with additional fill and a 4-foot by 2-foot concrete cap to support recreational use.

Alternative 3 contains the same floodwall/seawall described above within Alternative 2, with additional fill on the seaward side to support mangrove plantings at the base of the structure.

Alternative 4 consists of an elevated living shoreline at the locations designated within **Figure A - 40**. The elevated living shoreline will consist of three berms, with the first berm set to the specified design elevation and a top width of 5 ft and the slope will be kept at a 1V:4H on both the landward and seaward sides of the living shoreline. A concrete stem wall will be placed within the top berm and will extend from the top of the structure to 2-feet below existing grade. The second berm will be set to an elevation of 2 ft-PRVD02 and maintain a berm width of 1-foot to support various vegetative species like marsh grass. The third berm will be set to an elevation of -1 ft-PRVD02 and contain a berm width of 3 feet. Toe protection will encompass the entire bottom berm width of 3 feet and berm height of 3 feet with a D_{n50} of 1-ft and a unit weight of 147 lb/ft^3. A 1-ft diameter sediment tube surrounded by filter fabric will be placed within the center of the toe protection to support mangrove plantings to help stabilize the toe of the berm.

Alternative 5 is a combination of both Alternative 1 and Alternative 4. With the floodwall/seawall described in Alternative 1 on the northern side of the model area and the elevated living shoreline described in Alternative 4 on the southern side of the model area.

Alternative 6 is a combination of both Alternative 2 and Alternative 4. With the recreational floodwall/seawall described in Alternative 2 on the northern side of the model area and the elevated living shoreline described in Alternative 4 on the southern side of the model area.

Following economic analysis, Alternative 4 provided the highest BCR; refer to the Economics Appendix for additional information. Exact refinement of the structure centerline will be adjusted prior to the final Feasibility Report to optimize cut/fill volumes as well as recreational and environment considerations. Examples of this refinement could include the southern portion of Condado Lagoon being further setback, which will allow views from the Jaime Benítez National Park to not be blocked.

Table A - 18. Alternatives for Condado Lagoon

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	Floodwall/Seawall	Floodwall/Steel Cantilever Sheet Pile Seawall with King Piles & Toe Protection	6,628	10	-
	Inland Drainage	24" Culverts (Northern Shoreline)	15	-	3
		24" Culverts (Southern Shoreline)	15	-	7
		48" Culverts (East Shoreline)	15	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2
2	Recreational Floodwall/Seawall	Recreational Floodwall/Steel Cantilever Sheet Pile Seawall with King Piles & Toe Protection	6,628	55	-
	Inland Drainage	24" Culverts (Northern Shoreline)	60	-	3
		24" Culverts (Southern Shoreline)	60	-	7
		48" Culverts (East Shoreline)	60	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2
3	Recreational Floodwall/Seawall + Vegetation	Recreational Floodwall/Steel Cantilever Sheet Pile Seawall with King Piles, Toe Protection, & Fill at Structure Base for Mangroves	6,628	70	-
	Inland Drainage	24" Culverts (Northern Shoreline)	75	-	3
		24" Culverts (Southern Shoreline)	75	-	7
		48" Culverts (East Shoreline)	75	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
4	Elevated Living Shoreline	Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings	6,628	80	-
	Inland Drainage	24" Culverts (Northern Shoreline)	85	-	3
		24" Culverts (Southern Shoreline)	85	-	7
		48" Culverts (East Shoreline)	85	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2
5	Floodwall/Seawall + Elevated Living Shoreline	Floodwall/Steel Cantilever Sheet Pile Seawall with King Piles & Toe Protection	4,274	10	-
		Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings	2,354	80	-
	Inland Drainage	24" Culverts (Northern Shoreline)	15	-	3
		24" Culverts (Southern Shoreline)	85	-	7
		48" Culverts (East Shoreline)	15	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2
6	Recreational Floodwall/Seawall + Elevated Living Shoreline	Recreational Floodwall/Steel Cantilever Sheet Pile Seawall with King Piles & Toe Protection	4,274	70	-
		Elevated Living Shoreline With Toe Protection, Stem Wall, and Sediment Tube for Plantings	2,354	80	-
	Inland Drainage	24" Culverts (Northern Shoreline)	75	-	3
		24" Culverts (Southern Shoreline)	85	-	7
		48" Culverts (East Shoreline)	75	-	3
		100 CFS Pump (East Shoreline)	-	-	1
		50 CFS Pump (East Shoreline)	-	-	2

Figure A - 40. Alternative 1 for CL

4.7.8 Study-Wide Area

To protect the entire study area, the PDT delineated the alternative listed within **Table A - 19** and displayed within **Figure A - 41**. The alternative consists of two sector gates shown in blue and a floodwall/seawall shown in within **Figure A - 41**. The team assumed the San Juan Harbor sector gate to span the width of the entrance channel, approximately 1,000 feet wide. The remainder of the structure will span an additional 4,000 feet from the actual gate location. The team assumed a structure width of 150 feet. A floodwall/seawall will continue from the western side of the sector gate and tie into a high elevation area within WSJB-1A. The team designed the floodwall/seawall to be a steel cantilever sheet pile seawall, although some locations of the seawall may transition into a floodwall depending on available room within the canal and the amount of wave action in the area. The sheet piles will be driven approximately 25 ft deep and will contain backfill up to the design elevation. The seawall will contain a 2-foot by 2-foot concrete cap with backfill up to the design elevation as well as toe protection with a D_{n50} of approximately 2-ft and a unit weight of 147 lb/ft^3. The toe protection will have an approximate width of 8-ft and a height of 4-ft. If a floodwall is determined to be necessary due to a restriction of space, a T-Wall will be used. The T-Wall will consist of two piles spaced approximately 7.5 ft along the centerline of the wall with a total pile length of approximately 55 feet. The same toe protection described above for the seawall would be used if a T-Wall is determined to be used instead. The team assumed the Condado Lagoon sector gate to be 300 feet wide and the remainder of the structure will be an additional 700 feet wide. The team also assumed the width of the structure to be 150 feet.

Following economic analysis, the PDT removed the study-wide alternative since its cost is more than all the other alternatives combined. This alternative will also only protect the model area from storm surge events when the gates are closed, but will not protect the study area from tides and SLC when the gates area open. Refer to the Economics Appendix for additional information.

Table A - 19. Alternative for the Study-Wide Area

Alt #	Measure	Description	Approx. Length (ft)	Approx. Width (ft)	Quantity (EA)
1	SJH Sector Gate + CL Sector Gate + Floodwall/Seawall	San Juan Harbor Sector Gate	4,827	1,000	150
		Condado Lagoon Sector Gate	967	300	150
		Floodwall/Steel Cantilever Sheet Pile Seawall with Toe Protection	1,942	-	20

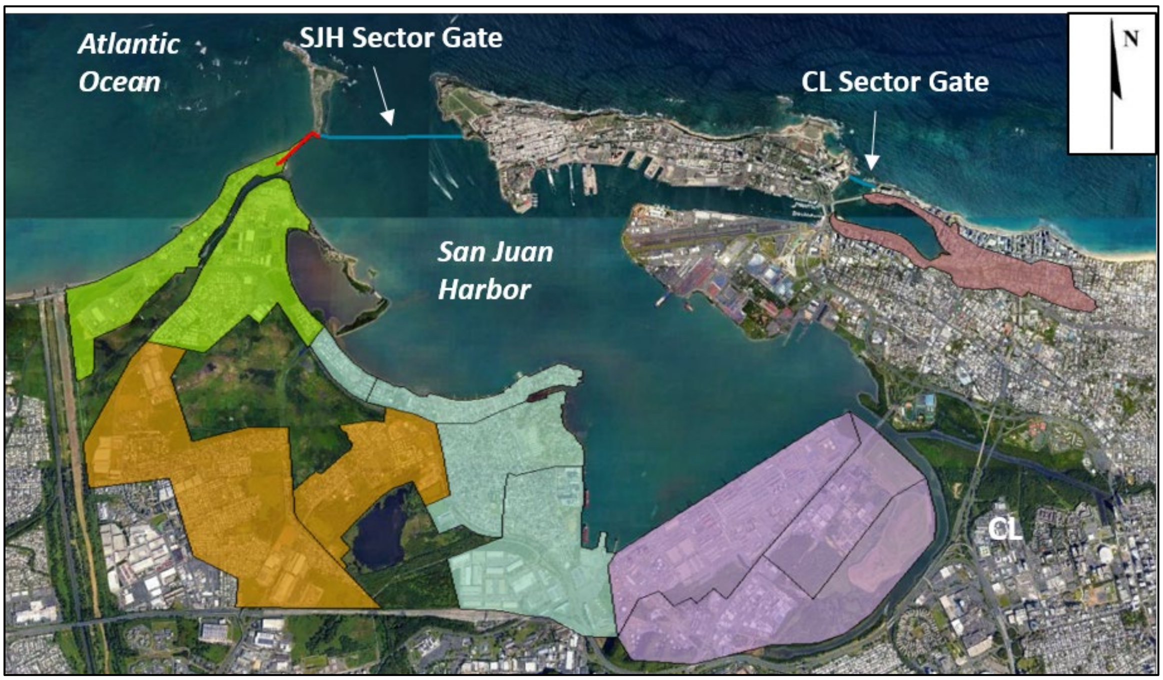

Figure A - 41. Alternative for Study-Wide Area

4.8 Cost Engineering

Cost Engineering input all the study measures noted within **Section 4.6** and formulated costs based on the alternatives discussed within **Section 4.7**. Refer to Appendix D for the Cost Estimating and Risk Analysis done for each selected measure.

4.9 Recommendation

Through the screening of project alternatives within each model area using the G2CRM model, the recommended plan is shown within **Figure A - 42**. As described in detail within **Section 4.7,** the PDT selected Alternative 1 within WSJB-1B, Alternative 3 within WSJB-2, Alternative 5 within WSJB-3, Alternative 2 within WSJB-4, and Alternative 4 within CL.

Figure A - 42. Study Recommendation

4.10 Pre-Construction Engineering & Design (PED) Considerations

During PED, design refinements will be conducted for all planned structural elements based on new field investigations and analyses. This chapter discusses not only what information and field investigations will be needed to achieve a final design, but also, how and what is proposed in the study that may be changed or adjusted.

4.10.1 Updated Surveys

It is recommended that topographic and/or bathymetric surveys be performed during PED in areas where structural measures are proposed. New surveys may determine an adjustment to the proposed height and/or length of structures is necessary. All elevations within the alternative designs are based off elevations from the DEM, which is based off data from 2018 or older. A more recent and comprehensive topographic and hydrographic survey will be required in order to develop plans and specifications.

4.10.2 Floodwall/Seawall Design Refinement

During the PED phase, subsurface explorations will be conducted along floodwall/seawall alignments to supplement the existing information, reference the Geotechnical Appendix (Appendix D) for additional details. Information from all subsurface explorations will be used to develop site-specific subsurface cross sections and refine the floodwall/seawall designs, if necessary. These data will supplement additional design calculations, including but not limited to axial and lateral load capacity, settlement, footing uplift pressure, and depth driven of piles/sheet piles. Findings from these analyses could result in changes to the assumed embedment depth of the piles (shorter or longer). Additionally, it is recommended to further analyze the wave conditions adjacent to each model area to potentially refine the rock size within alternatives that include toe protection and/or breakwaters. The apron width and height may be subject to change following this analysis as well. The wave analysis will also be used to determine if additional rock revetment should be designed to reduce potential wave reflection towards other areas within San Juan Harbor. The engineering team will reevaluate the crest elevation of the system to consider the latest information on the total water level, waves, and SLC per the ER 1105-2-101 guidance on risk based design.

Additionally, locations along the proposed floodwalls/seawalls may need refinement where existing boat ramps and/or marinas are located to maintain public accessibility to the waterfront.

4.10.3 Breakwater Design Refinement

During the PED phase, the study team recommends additional analysis to confirm the breakwaters will not negatively influence the federal channel through sand accumulation. If additional analysis determines the Federal channel will be affected the breakwater locations will be refined.

4.10.4 Inland Hydrology Analysis Refinement

During the PED phase, H&H will refine the interior drainage analysis to more accurately design measures for interior drainage relief. The analysis would entail the use of the HEC-HMS (Hydrologic Engineering Center – Hydrologic Modelling System) software version 4.3 or the latest model available with the guidance of Engineering Manual 1110-2-1413. The engineering team will use rainfall frequencies ranging from the 50% to 0.2% AEP events with 24-hour point rainfall from NOAA Atlas 14 as the input.

4.10.5 Alignment & Easements

During the PED phase, the team will collect more information and data, including real estate information. The collection of new information and data may require adjustments to the proposed alignments, if easements cannot be acquired in certain areas. Real estate requirements for the study area consist of Flowage Easements (FE), Flood Protection Levee Easements (FPLE), Temporary Work Area Easement (TWAE), and Bank Protection Easement (BPE). These easements are necessary to provide adequate construction room to build proposed flood risk management features and secure lands needed for Operations and Maintenance (O&M). Additionally, the engineering team created preliminary staging areas, although additional refinement to the exact acreage and location will be refined in PED. More information on easements and real estate requirements can be found in the Real Estate Appendix (Appendix E) in this study report.

4.10.6 Operations, Maintenance, Repair, Replacement, and Rehabilitation (OMRR&R)

During the PED phase, the monitoring procedures for the project will be written in the OMRR&R Manual for the local sponsor who will have the primary responsibility for operating and maintaining the project. The intent of the document is to provide the local sponsor with some clear and comprehensive guidance on the operation and maintenance of flood control structures that may be included in this study. It will describe how to plan and prepare for high water and storm events, and lays out steps to take during emergencies that will help reduce the threat of flooding. The manual will also explain the types of assistance that the U.S. Army Corps of Engineers can provide to a community before, during, and after a flood. Monitoring and inspections must occur to ensure that the project functions as designed and that the local sponsor confirms to all OMRR&R recommendations and requirements that will assist in functionality of the project.

The local sponsor should be prepared to carry out maintenance activities on all flood control structures every year. Regular maintenance is critical, because many types of problems will escalate exponentially when left unchecked. For example, debris and unwanted growth need to be removed from riprap and the areas adjacent to floodwalls, and from channels and waterways. Metal gates and other components need to be painted and greased periodically. Concrete damage needs to be identified and repaired early or it will get worse. Beyond these examples of ongoing maintenance, there are also more significant repairs that will be necessary from time to time. On occasion, the local sponsor may have to add stone to control an erosion problem, or do some major earthwork to repair an embankment. Pump stations also need to be completely overhauled periodically. Routine maintenance is expected in any project and can be planned for in advanced.

5 References

Dean, R.G. & Dalrymple, R.A. (2004). *Coastal Processes with Engineering Applications*. Cambridge University Press.

EurOtop. (2016). *Manual on wave overtopping of sea defenses and related structures. An overtopping manual largely based on European research, but for worldwide application*. Retrieved from: www.overtopping-manual.com

Federal Emergency Management Agency. (2009). *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update, Final Draft*. Federal Emergency Management Agency. Retrieved from: https://www.fema.gov/media-library-data/1388780453134-c5e577ea3d1da878b40e20b776804736/Atlantic+Ocean+and+Gulf+of+Mexico+Coastal+Guidelines+Update+(Feb+2007).pdf

Federal Emergency Management Agency. (2009). *Flood Insurance Study – Commonwealth of Puerto Rico and Municipalities.* November 18, 2009.

Federal Emergency Management Agency. (2015). *South Florida Storm Surge Study, Final Intermediate Data Submittal #1*, FEMA IDIQ Contract HSFEHQ-09-D-0369. Federal Emergency Management Agency.

Hubertz, J. A. (1992). *Uses for Marine Mattresses in Coastal Engineering*. ERDC/CHL, U.S. Army Engineer, Vicksburg, MS.

Hughes, S. A. (2006). *User's Guide to the Wave Information Studies (WIS) Wave Model, version 2.0*. WIS Report 27. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kerssens, P., Oorschot, J. van, & Pot, B. v.d. (1989). *Storm surge barrier in the Rotterdam Waterway; a new approach*. Proceedings of the 6th Congress of the Asian and Pacific Regional Division of the IAHR, Kyoto, Japan, July 20-22, 1988, Delft Hydraulics Publications 424.

Mooyaart, L.F., Jonkman, S. N., de Vries, P. A.L., van der Toorn, A., & van Ledden, M. (2014) *Storm Surge Barrier: Overview and Design Considerations*. Retrieved from: https://www.researchgate.net/publication/276311759_storm_surge_barrier_overview_and_design_considerations

National Oceanic and Atmospheric Administration. (2008). *NOAA Atlas 14: Precipitation-Frequency Atlas of the United States*. Volume 3, Version 4.0: Puerto Rico and the U.S. Virgin Islands. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Center for Operational Oceanographic Products and Services: Silver Spring, MD. Retrieved from: https://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume3.pdf

National Oceanic and Atmospheric Administration. (2012). *Global Sea Level Rise Scenarios for the US National Climate Assessment*. NOAA Tech Memo OAR CPO-1. 37 pp, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Center for Operational Oceanographic Products and Services: Silver Spring, MD. Retrieved from: https://scenarios.globalchange.gov/sites/default/files/NOAA_SLR_r3_0.pdf

National Oceanic and Atmospheric Administration. (2013). Extreme Water Levels of the United States 1893-2010. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Center for Operational Oceanographic Products and Services: Silver Spring, MD. Retrieved from: https://tidesandcurrents.noaa.gov/publications/NOAA_Technical_Report_NOS_COOPS_067a.pdf

National Oceanic and Atmospheric Administration. (2019). *Coast Pilot 5 - 47th Edition, 2019 covers the Gulf of Mexico from Key West, FL to the Rio Grande, including Puerto Rico and the Virgin Islands*. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Center for Operational Oceanographic Products and Services: Silver Spring, MD. Retrieved from: https://nauticalcharts.noaa.gov/publications/coast-pilot/index.html

National Research Council. (2012). *Sea-level rise for the coasts of California, Oregon, and Washington: past, present, and future*. National Research Council, the National Academies Press: Washington, DC. Retrieved from: https://www.nap.edu/resource/13389/sea-level-rise-brief-final.pdf

New York City Panel on Climate Change. (2013). *Climate Risk Information 2013: Observations, Climate Change Projections, and Maps*. New York City Mayor's Office of Sustainability: New York, NY, Retrieved from: http://www.nyc.gov/html/planyc2030/downloads/pdf/npcc_climate_risk_information_2013_report.pdf

U. S. Army Corps of Engineers. (1984). *Shore Protection Manual: Volume I*. Coastal Engineering Research Center, Department of the Army: Washington, DC. Retrieved from: http://ft-sipil.unila.ac.id/dbooks/S%20P%20M%201984%20volume%201-1.pdf

U. S. Army Corps of Engineers. (1984). *Shore Protection Manual: Volume II*. Coastal Engineering Research Center, Department of the Army: Washington, DC. Retrieved from: http://ft-sipil.unila.ac.id/dbooks/S%20P%20M%201984%20volume%202-1.pdf

U. S. Army Corps of Engineers. (1994). *EM 1110-2-1417: Engineering and Design: Flood-Runoff Analysis.* Washington *DC*. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf

U. S. Army Corps of Engineers. (1995). *EM 1110-2-1614: Engineering and Design: Design of Coastal Revetments, Seawalls, and Bulkheads.* Washington *DC*. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1614.pdf

U. S. Army Corps of Engineers. (1997). *ER 1110-2-1407: Engineering and Design: Hydraulic Design for Coastal Shore Protection Projects.* Washington *DC*. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-1407.pdf

U. S. Army Corps of Engineers. (2000). *EM 1110-2-1913: Engineering and Design: Design and Construction of Levees.* Washington *DC*. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1913.pdf

U. S. Army Corps of Engineers. (2006). *ER 1105-2-101: Risk Analysis for Flood Damage Reduction Studies. Washington DC*. Retrieved from:

https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1105-2-101.pdf

U.S. Army Corps of Engineers. (2011). *EC 1165-2-212: Sea Level Change Considerations for Civil Works Programs.* Circular No. 1165-2-212, Department of the Army: Washington, DC. Retrieved from: http://www.corpsclimate.us/docs/EC_1165-2-212-Final_10_Nov_2011.pdf

U.S. Army Corps of Engineers. (2011). *ETL 1110-2-575:* E*ngineering and Design Evaluation of I-Walls*. Technical Letter No. 1110-2-575, United States Army Corps of Engineers: Washington, DC. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerTechnicalLetters/ETL_1110-2-575.pdf

U.S. Army Corps of Engineers. (2013). *ER 1110-2-8162: Incorporating Sea-level Change in Civil Works Programs*. Washington, DC. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1100-2-8162.pdf

U.S. Army Corps of Engineers. (2017). *ECB 2017-3: Design and Evaluation of I-Walls Including Sheet Pile Walls*. United States Army Corps of Engineers: Washington, DC. Retrieved from: https://wbdg.org/FFC/ARMYCOE/COEECB/ARCHIVES/ecb_2017_3.pdf

U.S. Army Corps of Engineers. (2017). *Planning Risk Assessment for Flood Risk Management Studies.* United States Army Corps of Engineers: Washington, DC. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1105-2-101.pdf

U.S. Army Corps of Engineers. (2018a). *EM 1110-2-1413: Engineering and Design Hydrologic Analysis of Interior Areas*. Washington *DC*. Retrieved from: https://www.publications.usace.army.mil/Portals/76/Users/227/19/2019/EM_1110-2-1413.pdf?ver=2018-09-13-125346-687

U.S. Army Corps of Engineers. (2018b). *Generation II Coastal Risk Model User's Manual*. Army Corps of Engineers. Retrieved from: http://g2crm.com/

U.S. Army Corps of Engineers. (2018c). Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. ECB 2018-14. Army Corps of Engineers. Retrieved from: https://www.wbdg.org/FFC/ARMYCOE/COEECB/ecb_2018_14.pdf

U.S. Army Corps of Engineers. (2018d). *Sea Level Tracker User Guide*. Version 1.0. Army Corps of Engineers. Retrieved from: https://climate.sec.usace.army.mil/slr_app/SeaLevelTracker_User_Guide.pdf

U. S. Army Corps of Engineers. (2019). ECB 2019-8: *Managing Overtopping of Levee Systems.* Retrieved from: https://wbdg.org/FFC/ARMYCOE/COEECB/ecb_2019_8.pdf

U. S. Army Corps of Engineers. (2019). ECB 2019-15: *Interim Approach for Risk-Informed Designs for Dam and Levee Projects.* Retrieved from: https://www.wbdg.org/FFC/ARMYCOE/COEECB/ecb_2019_15.pdf

U.S. Army Corps of Engineers. (2019). *EP 1100-2-1: Procedures To Evaluate Sea Level Change: Impacts, Responses, and Adaptation*. Engineering Pamphlet No. 1100-2-1, United States Army Corps of Engineers: Washington, DC. Retrieved from:

https://www.publications.usace.army.mil/Portals/76/Users/182/86/2486/EP-1100-2-1.pdf?ver=2019-09-13-141310-707

U.S. Army Corps of Engineers. (2020). *ECB 2020-6: Implementation of Resilience Principles in the Engineering & Construction Community of Practice*. Engineering Construction Bulletin No. 2020-6, United States Army Corps of Engineers: Washington, DC.

U.S. Department of Agriculture. (1986). *Urban Hydrology for Small Watersheds: TR-55*. Retrieved from: http://www.ce.utexas.edu/prof/maidment/CE365KSpr16/Docs/TR55Manual.pdf

van der Meer, J.W. 2002. *Wave Run-up and Overtopping at Dikes*. Technical Report, Technical Advisory Committee for Water Retaining Structures (TAW), Delft, Netherlands. Retrieved from: http://www.overtopping-manual.com/assets/downloads/TRRunupOvertopping.pdf