RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY

HURRICANE SANDY LIMITED REEVALUATION REPORT
FOR
HURRICANE AND STORM DAMAGE REDUCTION
UNION BEACH, NEW JERSEY

U.S. Army Corps of Engineers, New York District

ENGINEERING APPENDIX
June 2017

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1.0 INTRODUCTION

1.1 Description of Project Area and Vicinity

Union Beach is located in the northern portion of Monmouth County, New Jersey. It occupies an approximate 1.8 square mile area of land along the coast of Raritan Bay. The area is located in low elevation regions with numerous small creeks providing drainage. Low-lying residential and commercial structures in the area experience flooding caused by coastal storm inundation. This problem has progressively worsened in recent years due to loss of protective beaches and increased urbanization in the area with structures susceptible to flooding from rainfall and coastal storm surges, erosion and wave attack, combined with restrictions to channel flow in the tidal creeks. This area was devastated by Hurricane Sandy on October 29, 2012. In Union Beach, approximately 90 percent of the Borough's land was flooded, ranging from 2 to 10 feet in depth. Union Beach reported that 60 properties were destroyed by Hurricane Sandy and 629 properties faced substantial damage. Approximately 24,500 tons of storm damage debris littered the Borough. Trees and power lines throughout the Borough fell. The Borough also faced total power outages for over two weeks. A more extensive list of Hurricane Sandy's impacts to Union Beach is included in the Main Report.

1.2 Scope of Engineering Analysis for Hurricane Sandy Limited Reevaluation Report

The purpose of a Hurricane Sandy Limited Reevaluation Report (HSLRR) is to determine if the 2007 Authorized Plan is still economically justified, given changes in policy and physical and socioeconomic conditions, since the authorization. Documentation of post-Sandy changes to the landscape included post-Sandy mapping acquired by Light Detection and Ranging (LIDAR) to recalculate earthwork quantities for levees and floodwalls. No new beach profiles were surveyed for the dune and beachfill; however, the onshore portion of the 1997 profiles were compared with sections taken at the same locations using the post-Sandy LIDAR. A new sea level rise analysis was conducted in accordance with Engineering Circular EC-1100-2-8162 (31 December 2013). The interior flooding analysis was reanalyzed using Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and the beach profile was verified using the Storm-induced Beach Change Model (SBEACH). No in-depth analysis was conducted for those project features which did not appear to have been impacted by criteria or physical changes since the feasibility study. However, a new estimate of all project costs was developed. The engineering team also participated in an Abbreviated Cost Risk Analysis to develop appropriate contingencies for the cost estimate in accordance with ER 1110-2-1302.

1.3 The 2007 Authorized Plan

The 2007 Authorized Plan is the recommend plan from the 2003 feasibility study, authorized for construction by Congress in the 2007 Water Resources Development Act (Public Law 110-114). As shown in Figure 1, it consisted of levees, floodwalls, road raising, relocations and closure structures, sector gates, and pumping stations at creeks, and a dune, groins and beachfill with periodic nourishment to form a continuous alignment on the east, north and west boundaries of the project area. The report used National Geodetic Vertical Datum of 1929 (NGVD29) and North American Datum 1927 (NAD 1927) State Plane New Jersey. The 2007 Authorized Plan consisted of a levee and floodwall alignment for the Chingarora Creek element that began at the high ground (+15ft NGVD29) near the intersections of Florence Avenue and Bank Street and ended at the northwestern end of the shorefront element. The shorefront element consisted of a beach and dune incorporating terminal groins with adjoining revetments stretching from the Chingarora Creek levee/floodwall alignment to the southeastern limit of the dune that ties into the levee alignment near Flat Creek. The Flat / East Creeks element consisted of a floodwall and levee alignment that began at the southeastern limit of the Shorefront element and tied into the existing Keansburg levee at the eastern end of the project limits.

Table 1 provides a comparison of the specific features included in the 2007 Authorized Plan and what is being proposed in the HSLRR. A detailed discussion of the HSLRR changes is provided in section 6.0 of this appendix.

Table 1. Pertinent Data Comparison – Feasibility vs. HSLRR- Union Beach

	2007 AUTHORIZED PLAN	**HSLRR**
LEVEE/FLOODWALL ELEMENT		
Levee		
Length (Chingarora: 6,428) (Flat/East: 4,442)	10,870 FT	
Length (Chingarora: 2,243) (Flat/East: 4,560)		6,803 FT
Top Elevation		
NGVD29	15.0 FT	15.0 FT
NAVD88 (NAVD29 minus 0.965 feet)[1]	14.0 FT	14.0 FT
Crest Width	10 FT	10 FT
Slopes	1V:2.5H	1V:2.5H
Fill Volume	156,700 CY	111,378 CY
Interior Levee		
Length	3,388 FT	3,388 FT
Top Elevation	8.0 FT	8.0 FT
NGVD29	8.0 FT	8.0 FT
NAVD88	7.0 FT	7.0 FT
Crest Width	2 FT	2 FT
Slopes	2.0:1	2.0:1
Fill Volume	3,997 CY	3,953CY
Interior Drainage		
Primary Outlet Structures	11	11
Secondary Outlet Structures	37	45
8 @ 18" Concrete Pipe	210 FT	210 FT
23 @ 24" Concrete Pipe	905 FT	
31 @ 24" Concrete Pipe		1,055 FT
7@ 36" Concrete Pipe	270 FT	270 FT

[1] http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl

	2007 AUTHORIZED PLAN	HSLRR
3 @ 48" Concrete Pipe	230 FT	480 FT
1@ 4' x 4' Box Culvert	80 FT	25 FT
6 @ 60" Concrete Pipe	840 FT	840 FT
6ft x 6ft Tide Gate Structures wI Sluice Gates	6	6
Natural Ponding Areas	4.21 Acres	4.21 Acres
Floodwall		
Length - Total	6,885 FT	12,907 FT
Chingarora		
I-wall	4,468 FT	0 FT
T-wall on spread footing	488 FT	0 FT
T-wall on piles	0 FT	10,977 FT
Flat/East		
T-wall on piles	1929 FT	1929 FT
Top Elevation		
NGVD29	15.0 FT	15.0 FT
NAVD88	14.0 FT	14.0 FT
Road Raising	580 FT	580 FT
Road Closure Gate (Miter 50' x7')	1	1
Flat Creek Storm Gate	1	1
Flat Creek Gate Width Opening	35 FT	35 FT
Flat Creek Gate Height	20 FT	20 FT
Flat Creek Pump Station Capacity	250 CFS	250 CFS
East Creek Storm Gate	1	1
East Creek Gate Width Opening	35 FT	35 FT
East Creek Gate Height	20 FT	20 FT
East Creek Pump Station Capacity	100 CFS	100 CFS
Chingarora Creek (CI-3- Cl-5) Pump Station Capacity	40 CFS	40 CFS
SHOREFRONT ELEMENT		
Length of Beach and Dune	3,160 FT	3,160 FT
Width of Dune Crest	50 FT	50 FT
Width of Beach Berm	50- 164 FT	50- 164 FT

	2007 AUTHORIZED PLAN	HSLRR
Elevation of Dune		
NGVD29	17.0 FT	17.0 FT
NAVD88	16.0 FT	16.0 FT
Elevation of Beach Berm		
NGVD29	9.0 FT	9.0 FT
NAVD88	8.0 FT	8.0 FT
Length of Eastern Terminal Groin	228 FT	228 FT
Length of Western Terminal Groin	245 FT	245 FT
Length of North Western Revetment	405 FT	405 FT
Length of South Eastern Revetment	630 FT	630 FT
Dune Slopes		
Landward	1V:5H	1V:5H
Seaward	1V:10H	1V:10H
Beach Berm Slope	1V:15H	1V:15H
Renourishment		
Every 9 years thereafter (by trucking)	21,000 CY	21,000 CY
Total Initial Fill		
Including design, advance, overfill, and tolerance fill	688,000 CY	688,000 CY
REAL ESTATE REQUIREMENTS		
Fee		29.67AC
Permanent Easements	87.30 AC	63.01 AC
Temporary Easements	3.25 AC	15.25 AC
ENVIRONMENTAL CONSIDERATIONS		
Wetland Mitigation	17.5 AC	22 AC
Mitigation Acquisition	17.5 AC	22 AC
ECONOMICS		
Price Level	October 2002	October 2016
Discount Rate	$5^{7/8}$%	$3^{1/8}$%
Initial Project Cost	$96,669,300	$273,005,000
Annual Project Cost	$6,864,000	$13,011,000
PHYSICAL CONDITIONS		
Tides		

	2007 AUTHORIZED PLAN	HSLRR
Semi-Diurnal		
Tide range*	Mean 5.0 FT	Mean 5.0 FT
	Spring 5.6 FT	Spring 5.6 FT
Stage		
Highest Observed Water Level (Keyport, September 12, 1960)		
NGVD29	10.5 FT	
NAVD88	9.5 FT	
Highest Observed Water Level (Battery Park, October 29 , 2012)		
NGVD29		12.0 FT
NAVD88		11.0 FT

Note: Tide data is interpolated from NOAA values at Atlantic Highlands and WayCake Creek.

Figure 1. Authorized Plan Alignment (2007)

2.0 EXISTING CONDITIONS

Presently there is no hurricane and storm damage reduction project in Union Beach, but there is a USACE project in the Borough of Keansburg to the east, completed in 1973. That project will be the eastern tie-out of the proposed Union Beach project.

2.1 Hurricane Sandy Impacts

At approximately 2000 EST on 29 October 2012, Hurricane Sandy made landfall approximately 5 miles south of Atlantic City, NJ, where it collided with a blast of arctic air from the north, creating conditions for an extraordinary historic and Hurricane along the East Coast with the worst coastal impacts on the Atlantic Coast of northern New Jersey and New York. The highest tide ever recorded at the Battery Park within New York City, and which exceeded predicted elevations of the storm by 9+ feet. As noted above, this area was devastated by Hurricane Sandy. The section of the beach northwest of the outfall groin appears to have lost 3'-4' vertically for about 60'. There were damages to the bulkhead that parallels most of the bayfront but the sand landward of the bulkhead was either retained or replaced after the storm. The town is still recovering with a mix of homes rehabilitated, raised, repaired, uninhabitable or demolished.

3.0 PHYSICAL CONDITIONS

3.1 Horizontal and Vertical Datums

As noted previously, the 2003 Feasibility Study was developed using the NGVD29 and NAD 1927 State Plane New Jersey. For this HSLRR all analyses were conducted using these same datums to match the drawings provided in accordance with the expedited schedule requirements of PL 113-2. Post Sandy LIDAR and recent aerial imagery were converted to these same datums. Per EM 1110-2-6065, the current datum recommended for use is the North American Vertical Datum of 1988 (NAVD88) and North American Datum 1983 (NAD83). Preconstruction Engineering and Design and construction will utilize NAVD88 and NAD83 but also be connected and modeled relative to the National Water Level Observation Network (NWLON) tidal datum and the National Spatial Reference System (NSRS) orthometric datum established by the department of Commerce.

3.2 Astronomical Tides

Tides at Union Beach are semi-diurnal and have a mean range of 5 feet and a spring range of 5.6 feet. Until Hurricane Sandy, the maximum recorded storm water elevation in the vicinity of the study area was observed at Keyport during hurricane Donna. The water level reported was +10.5 feet NGVD29 on September 12, 1960. More recent storm water levels in Keyport were +10.1 feet NGVD29 on December 11, 1992. Water levels from the same storm reached approximately +10 feet NGVD29 in Union Beach. On October 29, 2012 - Hurricane Sandy made landfall approximately 25 miles southwest of Atlantic City. According to the USGS's Hurricane Sandy Storm Tide Mapper website, https://water.usgs.gov/floods/events/2012/sandy/sandymapper.html, Hurricane Sandy produced a water level in the location of Keyport harbor of +14.5 ft. NAVD, which is approximately +15.6 ft NGVD29 (14.5 + 1.08 = +15.6 ft NGVD29). This would be the highest tide recorded at this area. The Peak stage at the Sandy Hook NOS tide gage before it was destroyed in Hurricane Sandy was +10.49 ft NAVD which is +11.57 ft NGVD29 (10.49 + 1.08 = +11.57 ft NGVD29), and ranks as the highest mark for this location. The previous highest tide recorded at Sandy Hook was +7.27 ft NAVD which occurred during Hurricane Donna, September

1960. Figure 2 below is the normal tidal signature at Sandy Hook station - SDHN4, which is approximately 10 miles east of the project area.

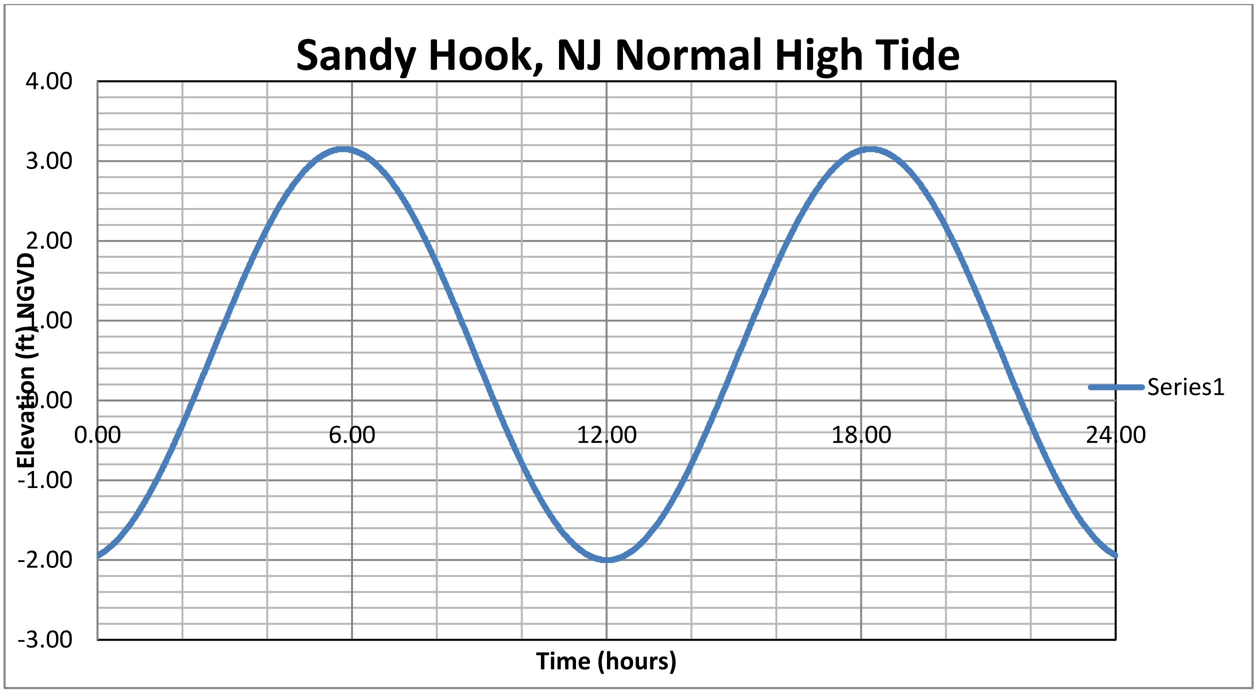


Figure 2. Normal Tide at Sandy Hook

3.3 Sea Level Rise

The Department of the Army Engineering Circular ER 1100-2-8162 (31 Dec 2013) requires that future sea level rise (SLR) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The project team should evaluate structural and nonstructural components of the proposed alternatives in consideration of the "low," "intermediate" and "high" potential rates of future SLR for both "with" and "without project" conditions. This range of potential rates of SLR is based on findings by the National Research Council (NRC, 1987) and the Intergovernmental Panel for Climate Change (IPCC, 2007). The historic rate of future sea-level rise is determined directly from gauge data gathered in the vicinity of the project area. Tide conditions at Sandy Hook (National Oceanic and Atmospheric Administration (NOAA) Station #8531680) best represent the conditions experienced in Union Beach. A 75-year record (1932 to 2006) of tide data gathered at Sandy Hook, NJ indicates a mean sea level trend (eustatic SLR + the local rate of VLM) of +3.9 mm/year. See Sub-Appendix C-Sea Level rise.

SLR considers the effects of (1) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the "regional" rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth's crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids. See Figure 3 for Modified NRC curves for predicting future rates of eustatic SLR.

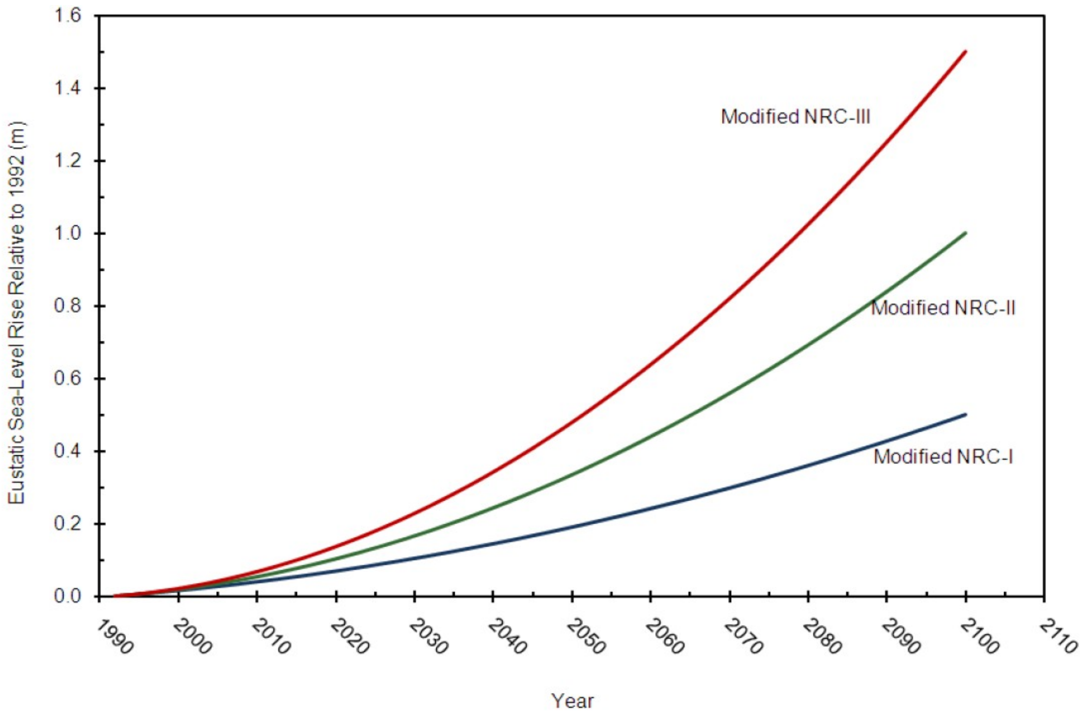


Figure 3. Modified NRC curves for predicting future rates of eustatic SLR.

The Union Beach project design water level stages were derived from Federal Emergency Management Agency (FEMA) modeling efforts in 2013. Using the base year 2013 from which future sea level elevations are estimated, Table 2 shows the projected increase in water surface elevation for the historic, intermediate and high rates of future sea level rise at Union Beach, New Jersey to the year 2100.

Table 2. Increase in Predicted Water Surface Elevations at Union Beach, NJ

Year	USACE Low	USACE Int	USACE High
2013	0.27	0.31	0.44
2018	0.34	0.4	0.59
2023	0.4	0.49	0.76
2028	0.47	0.58	0.95
2033	0.53	0.68	1.16
2038	0.6	0.79	1.38
2043	0.66	0.9	1.63
2048	0.73	1.01	1.89
2053	0.79	1.13	2.17
2058	0.86	1.25	2.47
2063	0.92	1.37	2.79
2068	0.99	1.5	3.13
2073	1.06	1.64	3.49
2078	1.12	1.78	3.86
2083	1.19	1.92	4.26
2088	1.25	2.07	4.67
2093	1.32	2.22	5.1
2098	1.38	2.38	5.55
2100	1.41	2.44	5.73

For the Union Beach project, the low rate of sea level rise has been incorporated into project design, per standard practice. Water surface elevation changes for the intermediate and high rates of SLR are presented. The Union Beach Project consists of components that are adaptable to future increases in sea level due to climate change. The sand dune and berm cross section could include increases in dune crest height, and corresponding increase in berm elevation to compensate for increasing still water levels. The levee and wall systems could also be modified with parapet walls or additional wave baffles pending design analyses to support additional height. If applicable, additional pump station capacity could be added to handle additional overtopping. However, a post-authorization change report would be required to make these changes. Regular renourishment operations are part of the 2007 Authorized Plan. Further details are contained in the Sub-Appendix C, Sea Level Rise Analysis.

3.4 Currents

Based on findings from the September 2003 Feasibility Report, currents in the project area are predominately tidal, with contributions from waves and creek discharges. In the Raritan Bay navigation channel north of Keansburg, the maximum average flood and ebb currents are 0.6 knots and 0.4 knots respectively (NOAA, 1995).

Compared to currents along an open coast or near a river or inlet, the tidal or wave-driven currents at Union Beach are believed to result in minimal influence on the local dynamics. The creeks in the area feature small cross sectional areas, which generally indicate mild flow rates. In addition, observed offshore tidal currents were minimal. The NOAA current data does not include the Union Beach area.

3.5 Bay Storm Stage

When investigating the bay storm stage water surface elevations the original study used a numerical/statistical model for 3 locations in New York Harbor, marked as nodes P1, P2, and I3. These were developed in 1998, by Engineer Research and Development Center's (ERDC) Dr. Norman Sheffner, who performed ADvanced CIRCulation (ADCIRC) modeling of the New York Harbor region for a Dredged Material Management Study. His P1, and P2 nodes are almost identical to new updated FEMA nodes 348054 and 422529, but the nearest node to I3 was Node 53246. Figure 4 is a map indicating node points. Table 3 below is a comparison of Node Locations between the ERDC Nodes and The FEMA Nodes:

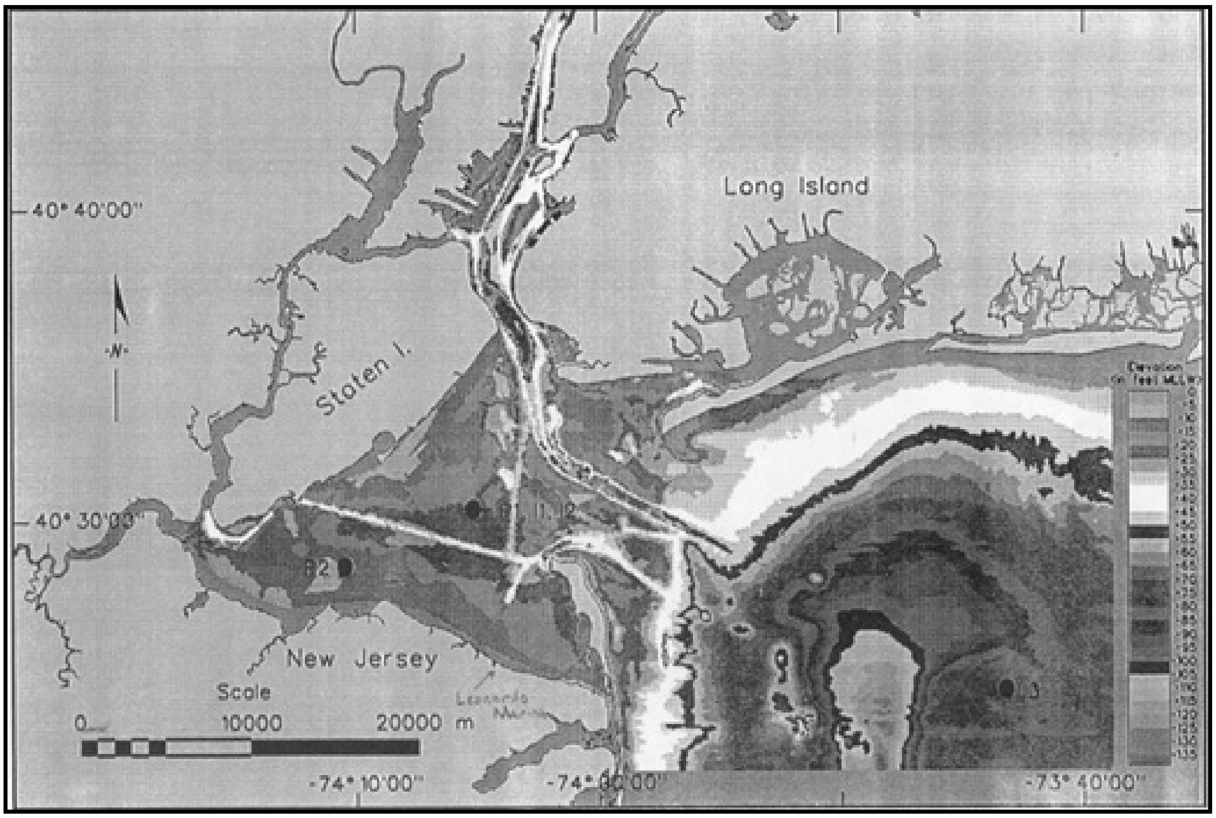


Figure 4. Node Location Map

Table 3. Comparison of ERDC Nodes vs. FEMA Nodes

Comparison Node Geographic Coordinates (NAD83) in degrees					
DMMP Node	Latitude	Longitude	FEMA Node	Latitude	Longitude
P1	40.50270556	-74.08263438	348054	40.50327	-74.0822

Comparison Node Geographic Coordinates (NAD83) in degrees					
DMMP Node	Latitude	Longitude	FEMA Node	Latitude	Longitude
P2	40.47429751	-74.16792754	422529	40.47454	-74.16823
I3	40.46502782	-73.84213115	53246	40.51437	-73.84025

The node selected for the SBEACH (Storm-induced BEAch Change) modeling of Union Beach, NJ was FEMA Node 422529 or Node P2. It is recommended as the Bay Storm Stage, with the note that datum conversion and wave setup subtraction where necessary. Table 4 compares the 1998 and 2013 stage elevations (ft NGVD29) for Node P2.

Table 4. P2 Stage Elevations in feet NGVD29

Return Period in years	1998 Stage Elevation w/o Wave Setup at Node P2 in ft. NGVD29	2013 FEMA Stage Elevation with Wave Setup at Node 422529 in ft. NGVD29
2	5.7	
5	8	7.5
10	9.2	8.9
25	10.6	10.6
50	11.5	11.9
100	12.2	13.3
200	13	14.7
500	13.9	16.6

In comparison with the previous data at node P2, the Node 422529 1 Percent Exceedence water elevation was 1.1 foot higher. This value included wave setup while the P2 value did not. It was considered prudent in light of the uncertainties associated with the limited SBEACH modeling being conducted for this analysis to select the higher water level. The maximum water level determined for the SBEACH modeling was the 1 Percent Exceedence level of +13.3 feet, NGVD29.

When examining the storm hydrograph a review of the Hurricane Sandy tidal records at Sandy Hook, NJ and The Battery, NY found there was excellent correlation between the two hydrographs up until the point at the peak of the storm when the Sandy Hook gage stopped recording. Based on this it was considered appropriate to use the shape of the storm hydrograph at The Battery in the SBEACH Modeling.

The maximum water level at the Battery gage was approximately 1 foot less than the maximum water level discussed earlier in this section. The decision was made to prorate The Battery water levels so the maximum elevation matched the 1 Percent Exceedence level.

A plot of the adopted storm hydrograph is shown below in Figure 5. Further details contained in Sub-Appendix D, SBEACH Modeling.

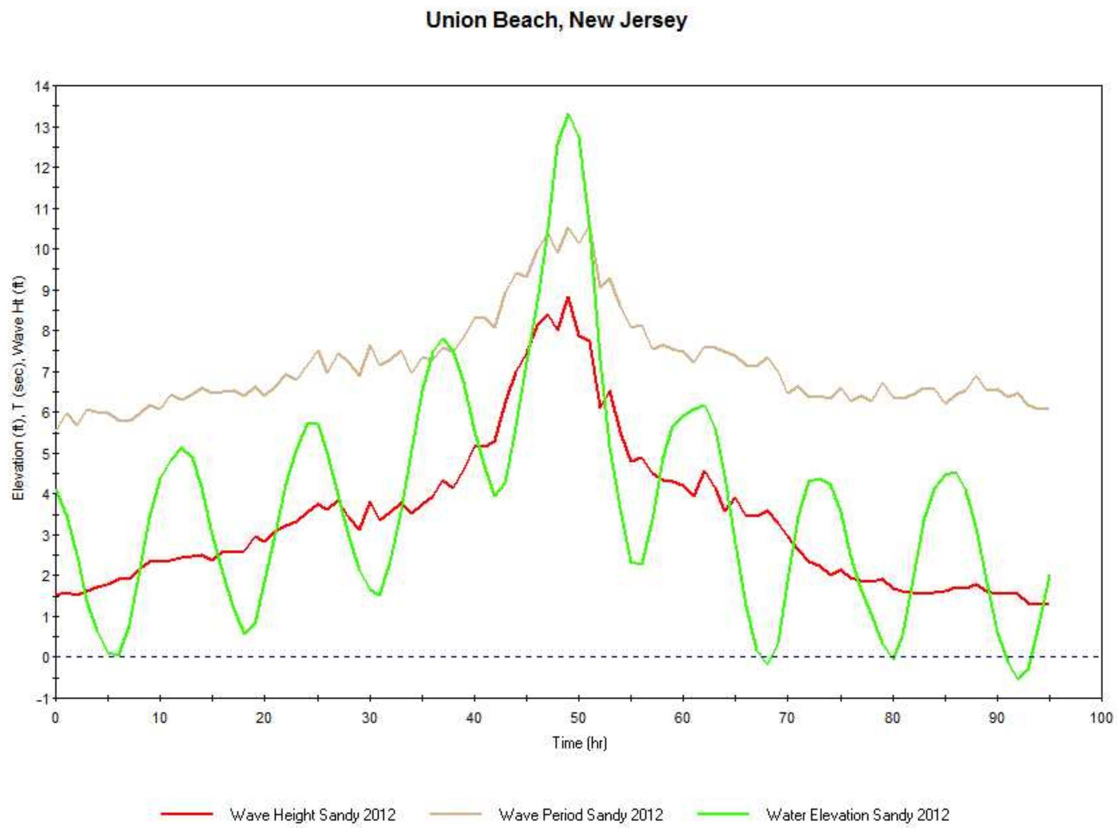


Figure 5. Plot of the adopted storm hydrograph

3.6 Waves

Wave gage data could not be found in the vicinity of the project with the exception of a limited data set from a temporary gage deployed prior to Hurricane Sandy landfall. Wave data for the model runs used surrogate data from the storm wave data that were available from the NOAA buoy station 44065. Wave data were matched to storms having similar maximum water levels as described in the Feasibility Study resulting in a peak wave height of 8.4 feet.

These wave heights match the wave heights from the preliminary wave height estimates prepared by Coastal Hydraulics Lab (CHL) (Ebersole, 1998). Those wave height estimates include extratropical storm generated ocean waves, tropical storm generated ocean waves, and local wind generated, fetch limited waves and accounted for the elevated water levels associated with the storm stage. To determine the 2, 5, 10, 25, 50, 100, 200, and 500-year offshore wave heights, a combined wave height frequency curve was extrapolated from the wave heights provided by CHL for the location closest to Union Beach (node "P2"). Offshore wave heights represent the maximum wave heights at the node P2location (with bottom grade at -17 feet NGVD29) expected for a given return period during the peak storm stage. Offshore wave heights appear in Table 5 below showing the design wave heights previously set as the Design Wave Heights. Further details are contained in Sub-Appendix D, SBEACH Modeling.

Table 5. Original 1998 ERDC Modeled Wave Information

Return Period (Years)	Offshore Wave Height (1) (-17' NGVD) (feet)	σ (2) (feet)	Wave Generation (1)	Wave Period (3) Tp (seconds)	Storm Stage (ft NGVD)	Design Wave Height (feet) (4)			Type of Wave (4)			Breaking Depth or Depth at Structure (feet below Storm Stage)		
						Reach 1	Reach 2	Reach 3	Reach 1	Reach 2	Reach 3	Reach 1	Reach 2	Reach 3
2	1.6	1.1	ocean	4.6	4.5	2.6	2.8	2.6	broken	broken	broken	2.5	2.0	2.7
5	1.9	0.9	ocean	5.2	7.9	3.2	2.9	3.1	broken	unbroken	broken	2.9	3.2	3.3
10	2.5	0.7	fetch	5.5	9.4	4.0	2.9	4.0	broken	unbroken	broken	3.9	4.7	4.2
25	4.0	0.5	fetch	5.8	11.2	5.9	4.5	5.8	broken	unbroken	broken	6.1	6.4	6.4
50	5.0	0.7	fetch	6.0	12.1	7.1	5.6	7.0	broken	unbroken	broken	7.5	7.4	7.8
100	6.2	1.0	fetch	6.2	12.5	8.4	6.9	8.3	broken	unbroken	broken	9.1	7.8	9.3
200	7.4	1.5	fetch	6.4	12.8	9.8	8.4	9.6	broken	broken	broken	10.9	8.1	11.0
500	8.5	2.4	fetch	6.7	15.5	11.1	9.3	10.9	broken	broken	broken	12.6	10.8	12.6

NOTES:

(1) Extrapolation of CHL wave predictions; predictions include extra-tropical storm generated ocean waves, tropical storm generated ocean waves, and local wind generated, fetch limited waves. Offshore depth = 17' relative to NGVD.

(2) Uncertainty for breaking wave heights assumed to be equal to uncertainty for offshore waves. Uncertainty values are the maximum of the extra-tropical storm generated ocean waves, tropical storm generated ocean waves, and local wind generated, fetch limited waves.

(3) Peak periods correspond to wave periods for Port Monmouth.

(4) See Figure A-2 for the location of Reach 1, Reach 2, and Reach 3.

3.7 Winds

Based on findings from the September 2003 Feasibility Report, wind data is not adequately available within the project limits, but is available for the Sandy Hook area, east of the project area. A wind rose was constructed based on data covering a 10-year period between 1924 and 1934. The prevailing winds are from the northwest, occurring 19 percent of the time. Winds from the north, northeast, south, and west occur more than 15 percent of the time. Winds from the east and southeast occur approximately 10 percent of the time. The northeast accounts for most occurrences greater than 50 mph.

Wind information is also available in the Wave Information Study database (WIS, 1993). This database provides hindcast winds at 3-hour intervals for the 1956-1975 period. The wind data represents the 10-minute averaged wind at 10 meters above open water. The maximum wind velocity in WIS database is 56 mph. Winds of such velocity are primarily oriented from the north. This maximum velocity and general direction is similar to the Sandy Hook 1924-34 wind rose.

3.8 Bayshore Characteristics

Based on findings from the September 2003 Feasibility Report, existing characteristic dimensions of the beach at Union Beach were taken from the October 1997 onshore and offshore survey; see Figure 6, Shoreline Reach Delineation. The characteristics vary considerably from one profile line to the next. However, at profiles where a beach is present, some characteristics are consistent, including a dramatic slope change from the steep beach to the near flat offshore slope. This slope change, identified at the slope break point, is generally between elevations -1.1 and +2.9 feet NGVD29.

4.0 STORMS

4.1 General

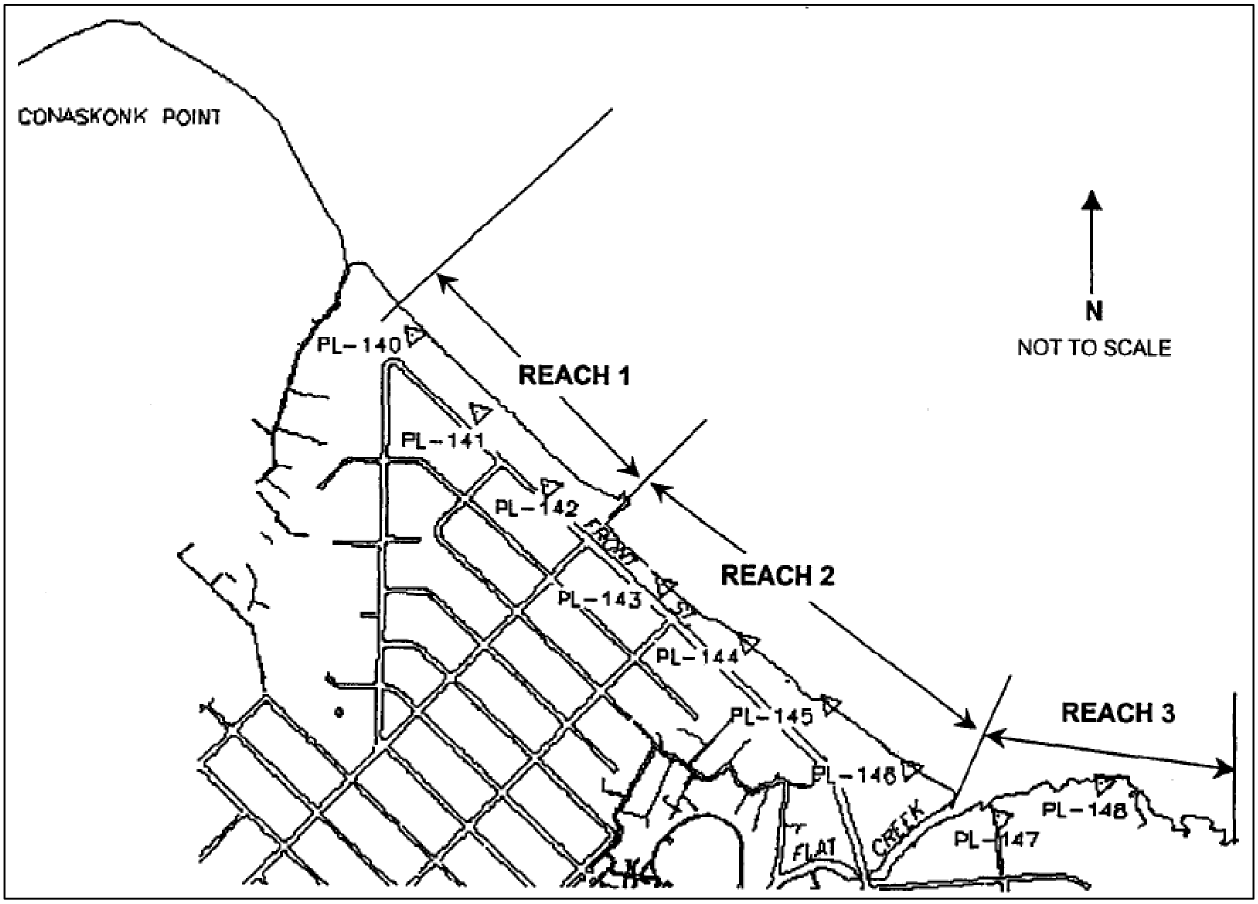


Figure 6. Shoreline Reach Delineation – Union Beach, NJ

Two distinct classes of storms that affect the study area are nor'easters (extratropical) and hurricanes (tropical). Nor'easters, named after the predominant direction of winds, are large-scale low pressure disturbances which usually occur from November through March. The severity of a nor'easter is not as great as that of a hurricane. Although wind gusts can reach hurricane strength in a very severe nor'easter, sustained wind speeds are rarely greater than 50 knots. The flood damage caused by the typical nor'easter is often more a function of its duration rather than its intensity, as the longer storms have more opportunity to destroy both natural and engineered flood protection features. Also, as nor'easters typically last two to three days, it is possible for the storm to impact the study area during several periods of high astronomical tide. Hurricanes are a rarer occurrence in the study area than nor'easters. By the time hurricanes approach the latitudes of the north New Jersey coast, they are usually in a state of energy loss and are beginning to decay into the category of tropical storm. Despite their infrequency and short duration, hurricanes have the potential to be devastating in the study area because of their high wind speed and high surge. Please refer to the 2003 Feasibility Report for a complete listing of storms prior to Hurricane Sandy.

4.2 Hurricane Sandy – October 29, 2012

On October 22, 2012, Hurricane Sandy originated in the Caribbean and strengthened as it crossed over eastern Cuba and the Bahamas on October 25, 2012. At that point, it was a Category 2 storm with winds in excess of 125 mph. Upon its approach to the coastline a trio of weather factors combined to create Hurricane Sandy: (1) an intense Category 1 hurricane, (2) a trough of low pressure dipping down from the Arctic feeding the hurricane and (3) a block of high pressure in the northeastern Atlantic Ocean pushing Sandy toward the east coast. According to the National Hurricane Center's operational advisories, Hurricane Sandy transitioned to a post-tropical storm at approximately 1700 EST on October 29, 2012, about 1-hour before landfall approximately 25 miles southwest of Atlantic City with sustained winds of 90 miles per hour. After Hurricane Sandy made landfall in New Jersey, sustained winds increased as an effect of an additional storm approaching from the west. The combination of storms, timed with the full-moon high-tide on October 29, exacerbated storm-tide flooding along the New Jersey, New York, and Connecticut coastlines, and caused significant backwater to occur far inland along the Delaware and Hudson Rivers. Storm effects along the Hudson River were measured as far inland as Albany, New York.

In the days leading up to the storm making landfall, the United States Geological Survey (USGS) deployed storm-tide monitoring instruments to characterize the height, extent, and timing of storm tides better than could be accomplished by existing USGS or NOAA observational fixed-place networks. A temporary monitoring network of water-level and barometric pressure sensors was deployed at 224 locations along the Atlantic coast from Virginia to Maine to continuously record the timing, areal extent, and magnitude of hurricane storm tide and coastal flooding generated by Hurricane Sandy. There were a total of 62 barometric pressure sensors, plus, 162 water-level and wave-height sensors that were deployed at 147 locations during October 26 –29 prior to landfall. The records these sensors created were greatly supplemented by an extensive post-flood high-water mark (HWM) flagging and surveying campaign from November to December 2012 involving more than 950 HWMs. This survey resulted in a database of 950 HWMs following Sandy, and was the single largest HWM recovery effort in recent USGS history. Figure 7 shows an inundation map with high water marks for the Union Beach area. Details can be found on the USGS website: https://water.usgs.gov/floods/events/2012/sandy/

In detailing the strength of Hurricane Sandy and looking back to historical meteorological records dating back to 1851, the lowest barometric pressure of 945.6 mb in Atlantic City's history was recorded as Hurricane Sandy made landfall, replacing the previous record of 961 mb set in 1932. Hurricane Sandy also had the highest ever observed wave heights recorded at two National Data Buoy Center (NDBC) buoys as well as the highest ever recorded total water elevations.

- NDBC Station 44065 (New York Harbor Entrance) recorded an offshore wave height of 32.5 ft.
- NDBC Station 44025 (Long Island) recorded an offshore wave height of 31.5 ft.
- Peak stage at the nearby Battery Park, NY National Oceanic and Atmospheric Administration's National Ocean Service's (NOS) tide gauge was +11.1 ft NAVD which is approximately +12.0 ft NGVD29 (11.1 + 0.896 = +12.0 ft NGVD29). At the Battery Park gauge, three high tides exceeded predicted tide by 2 ft; of those three high tides, one was above predicted tide by 3 ft, and another exceeded predicted tide by 9 ft. It was the highest tide ever recorded at the Battery Park gauge.

Peak stage at the Sandy Hook NOS tide gage before it was destroyed in the storm was +10.49 ft NAVD which is +11.57 ft NGVD29 (10.49 + 1.08 = +11.57 ft NGVD29). At Sandy Hook before the gauge was destroyed, two high tides exceeded predicted tide by more than 2ft, of those two high tides, one was above predicted by more than 3 ft. At the nearby Battery Park gage, three high tides exceeded predicted tide by more than 2ft, of those three high tides, one was above predicted by more than 3 ft, and another exceeded predicted by more than 9ft. It was the highest recorded tide ever at the Battery Park gage. At Sandy Hook, the peak stage before it was destroyed now ranks 1st on the tidal record. The previous highest tide recorded at Sandy Hook was +7.27 ft NAVD which is +8.35 ft NGVD29 (7.27 + 1.08 = +8.35 ft NGVD29) and occurred during the September 1960 storm.

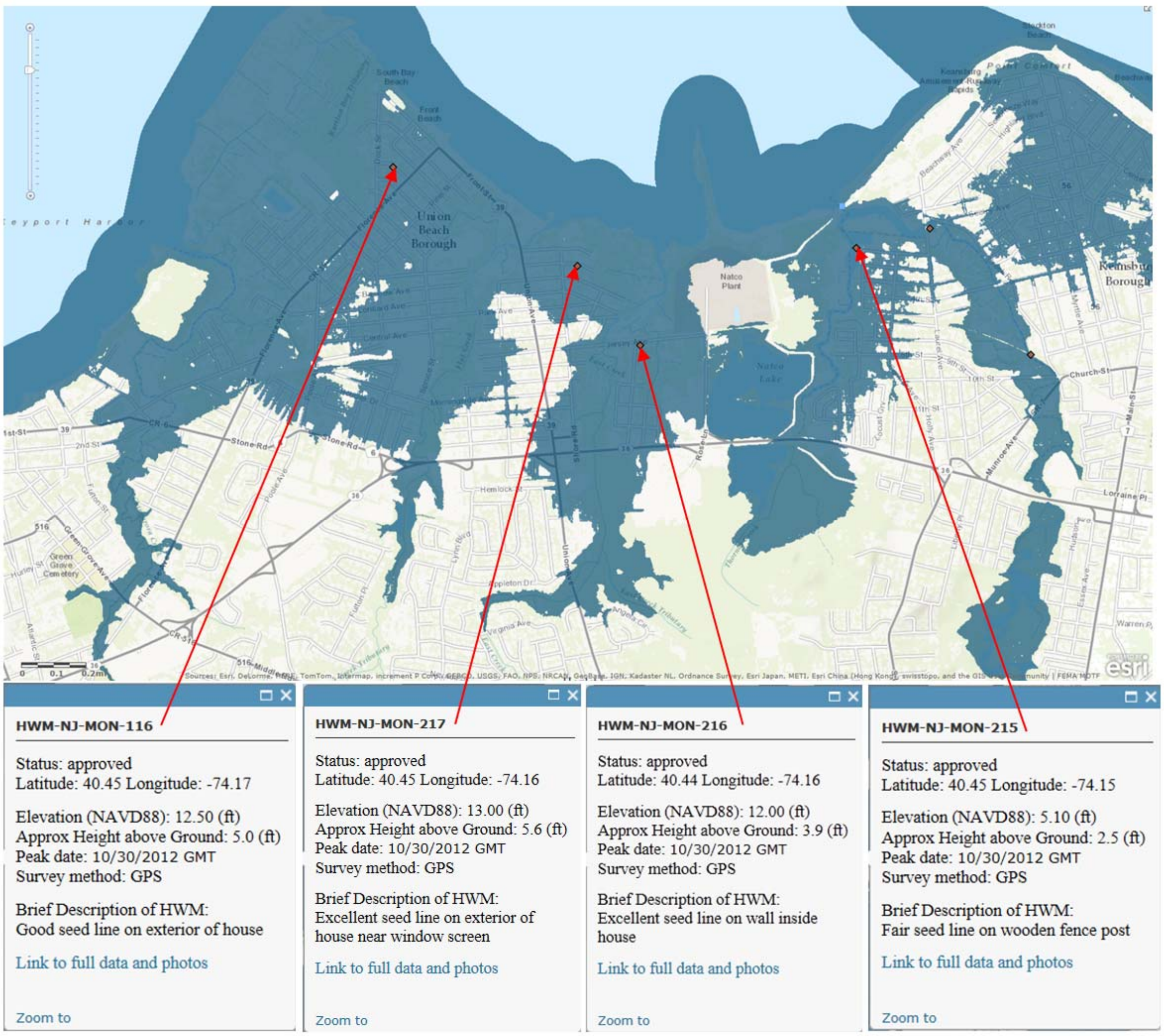


Figure 7. Inundation Map with High Water Marks[2]

[2] Source: https://water.usgs.gov/floods/events/2012/sandy/sandymapper.html

5.0 REEVALUATION OF THE 2007 AUTHORIZED PLAN

5.1 General

For the HSLRR, the alignment and project components were reviewed for any issues due to changes in criteria, physical conditions resulting from Hurricane Sandy, cost savings or subsequent development. The following items were reevaluated:

- Coastal Barrier Resources Act (CBRA) realignment by the US Fish and Wildlife Service
- Easements and property development
- Floodwall design
- Embankment design
- Constructability
- Broadway closure gate
- Flat and East Creek Gates
- Conditions along East Creek
- Interior flooding analysis
- Dune and beach analysis
- Overtopping & failure analysis
- Quantity estimates
- Beachfill borrow

5.2 CBRA Boundary

Designed to prevent development within the natural coastal barrier systems, the Coastal Barrier Resources Act (CBRA) prohibits new federal expenditures or financial assistance within System Units of the Coastal Barrier Resources System (CBRS). Units are identified and managed by the US Fish and Wildlife Service. The original Union Beach Project alignment impacted s portions of the CBRS System Unit NJ-04. In the 2003 EIS, compliance with the Coastal Barrier Resources Act (CBRA) was pending. The 2008 Record of Decision was signed without no mention if compliance with the CBRA was completed. The US Fish and Wildlife Service has no records of compliance either. As a part of assessing impacts of Hurricane Sandy to the CBRS, the US Fish and Wildlife Service re-examined CBRS units affected by the storm for potential boundary realignment. As a result, NJ-04 was realigned to include more wetlands and exclude the Bayshore Regional Sewerage Authority (BSRA) treatment plant. The Environmental Analysis Branch (EAB) of the District, consulted with the US Fish and Wildlife Service regarding the proposed revisions to the NJ-04 alignment. As a result, the project alignment was changed from levee to floodwall along the northern segment of the project alignment to accommodate the new NJ-04 unit alignment as well as limit the amount of real estate affected to the maximum extent practicable (see draft SEA Appendix D). The revised NJ-04 alignment has been included on plan sheets 1-5.

5.3 Easements and Property Development

The project easements were reviewed for compliance with the USACE's vegetation management policy, ETL 1110-2-571, 10 April 2009, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams and Appurtenant Structures. The guidance requires 15 feet from levee toes, drains or structural features and 15 feet from the faces of floodwalls and a minimum of 8 feet beyond the footing. The 2007 Authorized Plan did not include temporary easements of 10 feet for construction beyond the required perpetual easement or fee acquisition on the landside of the alignment. In addition, the change of the proposed I-Walls to T-Walls increased the perpetual easement or fee acquisition from 10 feet from both faces of the I-Walls to 21 feet from each face of the T-Walls (footing plus 8 feet). This increased real estate cost as discussed in the Main Report. Details of the updated easements are included in Section 6.7.

Since the completion of the 2003 Feasibility Study, a condominium development adjacent to Flat Creek, a storage/treatment tank at the Bayshore Regional Sewage Treatment Authority plant, swimming pools, fences, outbuildings and other structures have been constructed along the alignment or within the required easements. Since the condominiums are approximately 25 feet from the levee centerline and the levee height is almost 9 feet with 1V:2.5H side slopes, the levee toe is literally at the doorstep of the easternmost condominiums (See Figure 8). Certainly this would concern the property owners due to obstruction of their existing views of the waterfront and reduced open space. In addition, the USACE required vegetation management easement of 15 feet would not be fully available. For this HSLRR update, no attempt was made to avoid these features and the added or reduced costs were not calculated. This issue should be further analyzed in PED and is discussed again in Section 7.5.

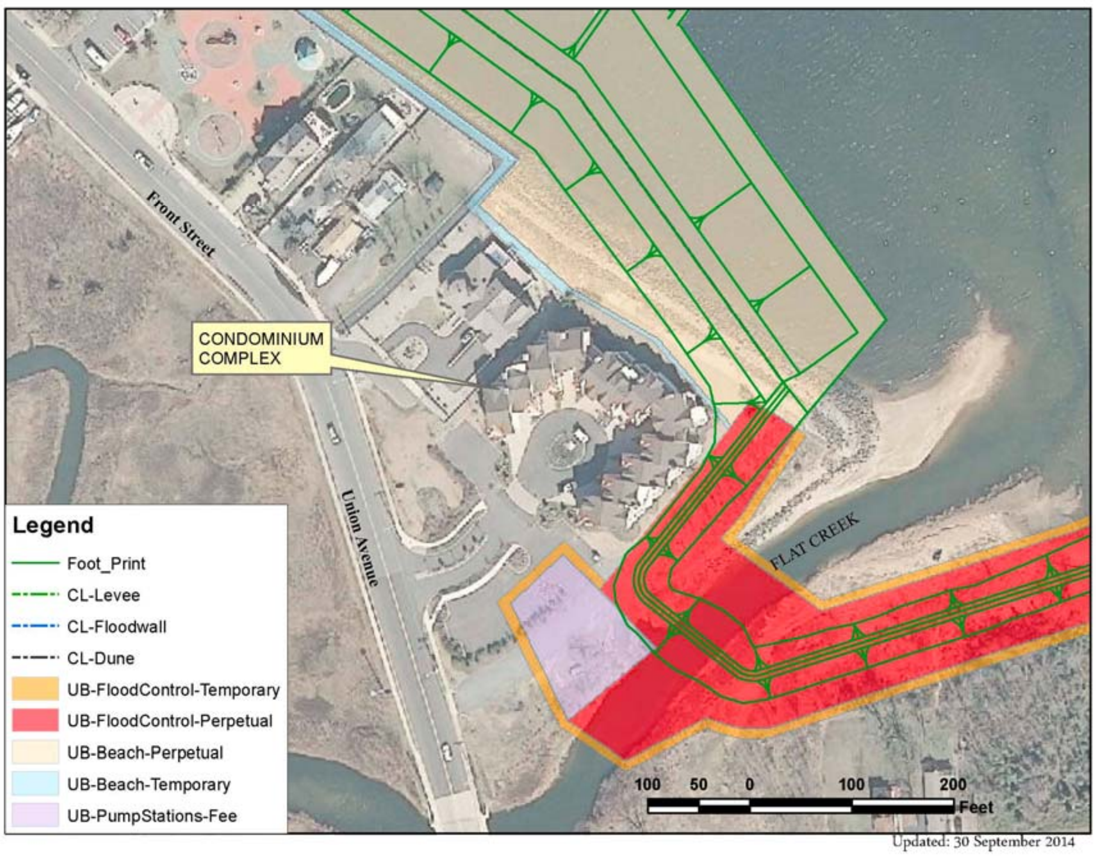
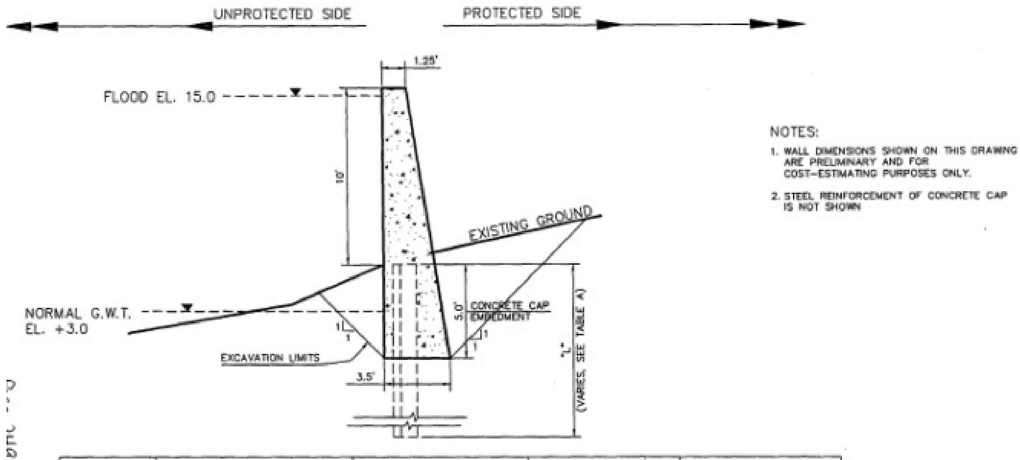


Figure 8. Easement Impacts at New Condominium Development

5.4 Floodwall

As stated above, the proposed floodwalls in the 2007 Authorized Plan were a combination of I-Walls, T-Walls on spread footings and T-Walls on piles. The locations of each type of floodwall in the 2007 Authorized Plan are shown on Figure 1. See Figure 9, Figure 10, and Figure 11 below for typical sections proposed in the 2007 Authorized Plan. The T-Wall was placed on piles for the two reaches east of the beach due to foundation concerns. However, the majority of the Union Beach floodwalls were originally designed as I-Walls, a slender cantilever wall embedded into the base soil and stabilized by reactive lateral earth pressure.



NO.	STA. TO STA. LIMITS	WALL HEIGHT ABOVE EXISTING GROUND	TOTAL LENGTH "L"	SHEET PILE SECTION
1	23+50 – 27+50	5' – 7.5'	18	PZ27
2	27+50 – 38+50	7.5' – 10'	25	PZ35
3	45+50 – 58+00	7.5' – 10'	25	PZ35
4	74+70 – 80+74	5' – 7.5'	18	PZ27
5	106+86 – 118+00	7.5' – 10'	25	PZ35

TABLE A

Figure 9. Feasibility I-Wall Section (elevations in ft. NGVD).

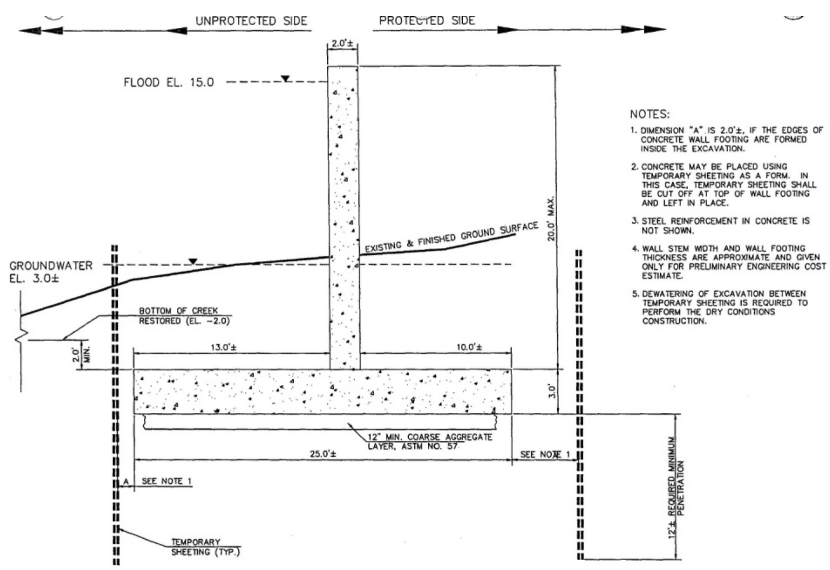


Figure 10. Feasibility T-Wall (elevations in ft. NGVD)

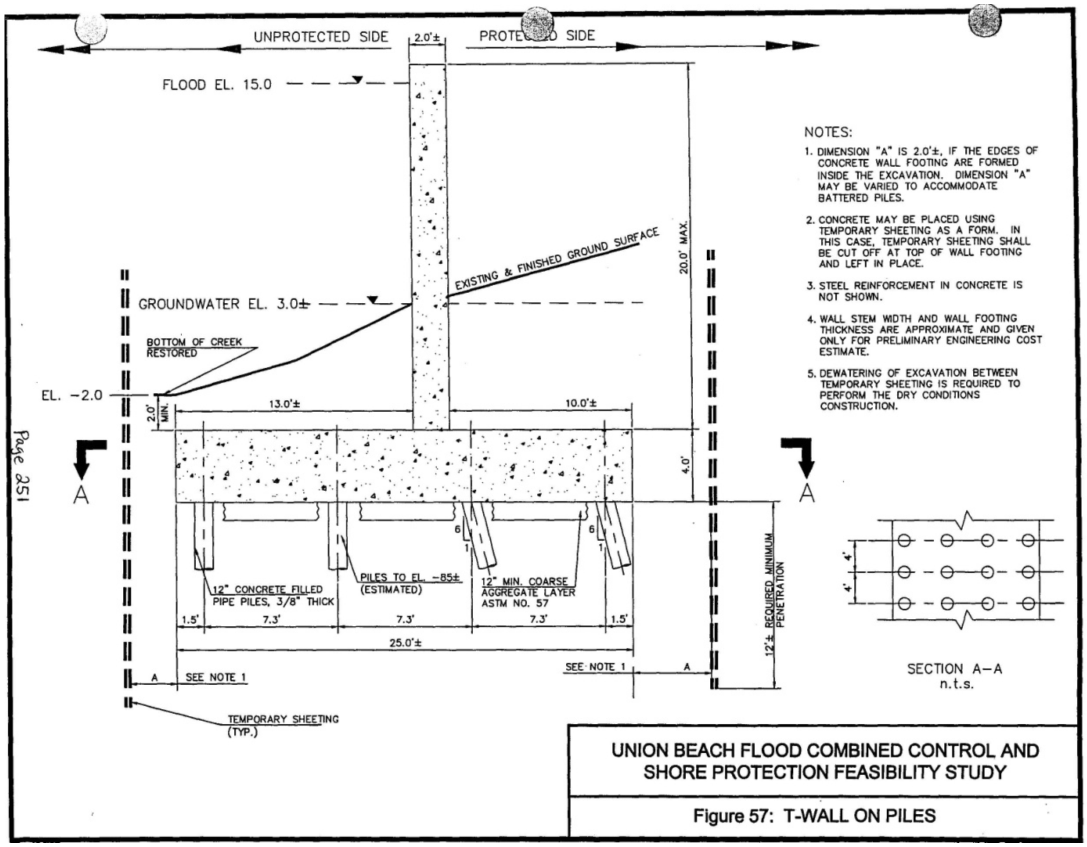


Figure 11. Feasibility T-Wall on Piles (elevations in ft. NGVD)

Due to concerns on performance of I-Walls in major storm events especially in coastal regions, EC 1110-2-6066 “Design of I-Walls” was issued on 1 April 2011 by consolidating the findings and lessons learned from studies performed after Hurricane Katrina and other storms. EC 1110-2-6066 paragraph 2-2e(9)states: "While overtopping of the I-walls led to significant scour and damage in many cases, overtopping of T-walls did not lead to extensive scour and erosion, because the base of the inverted T-wall sections extended over the protected side. T-walls performed well during Katrina. Because of their pile foundations, they are better able to transfer high lateral water loads into stronger underlying foundation materials." Since the EC has expired, and a replacement has not been completed, Engineering Construction Bulletin (ECB) No. 2014-18 has been issued to provide the following interim guidance. For the design of I-walls, use EC 1110-2-6066. For the evaluation of I-walls, use ETL 1110-2-575. For the design of cantilever and single anchored earth retaining sheet pile walls, use EM 1110-2-2504.

Based on these criteria changes, it was necessary to reevaluate the floodwall design for Union Beach, particularly for the I-walls. Of the total 6885 ft of floodwall in the 2007 Authorized Plan, nearly 85% is greater than 6 ft in height. However, Erosion control along the I-Wall is also a major concern. Significant changes on the protected side of I-wall would be required to prevent loss of material due to overtopping. The cost to construct erosion control along the unprotected side of I-Wall and overtopping protection on the landside was assumed to be more than a T-Wall system. Additionally, the T-Wall floodwall provides a more stable floodwall system and has better performance when erosion is a main concern for coastal flood mitigation project.

After consideration of new criteria and limited foundation information, the decision was made to replace all floodwall with T-wall on piles. This was deemed to be most prudent for obtaining a conservative updated cost for the HSLRR. Paragraph 6.4 details the preliminary design for all T-Wall that was incorporated into this HSLRR. For design details, see Sub-Appendix A, Floodwall Analysis, and Sub-Appendix B-1, Floodwall Pile Analysis West Walls and B-2, Floodwall Pile Analysis East Walls.

5.5 Levee

Levee Design is accomplished in accordance with EM 1110-2-1913 Design and Construction of Levees. The 1978 edition was utilized by the New York District for the preliminary design conducted as part of the 2003 Feasibility Report. During that study it was determined that a single levee embankment section could be used for both the east and west alignments. The levee section from the 2007 Authorized Plan is shown in Figure 12.

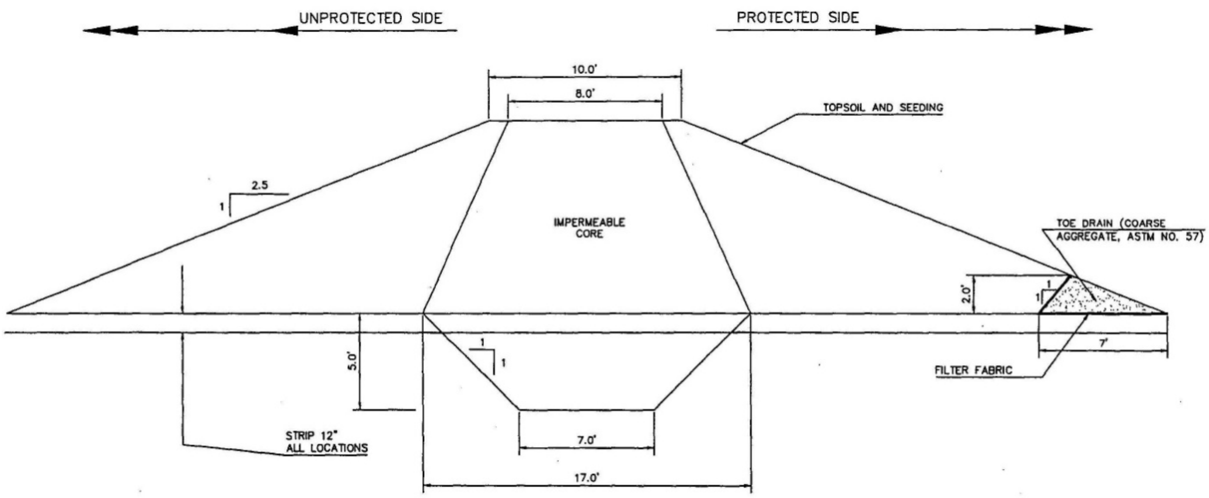


Figure 12. Feasibility Levee

Two selected representative levee cross sections were utilized for the preliminary design, one at Sta. 63+64 and the other at Sta. 51+50. Previously completed flood control feasibility studies for nearby sites, (i.e. Port Monmouth a community within Middletown Township), with similar geologic and hydraulic conditions, were used to select initial side slopes and embankment material components, which were applied in stability and seepage analyses of the proposed levees. Conditions that controlled the design along with the results of all analyses supported the recommended slopes and material components.

The recommended levee had a crest width of 10 feet and 1V:2.5H side slopes. Levee heights varied from 4.5 to 12.5 feet to support the design flood elevation of +15.5 feet NGVD29. The design at the time (2003) recommended using commercially available embankment materials from known suppliers. The 2007 Authorized Plan indicated a toe drain that theoretically would meet the standards determined in the seepage analyses; however it did not include any penetration of the toe drain into the foundation which by current state of the practice is recommended. In addition, no overtopping protection was provided. As part of the HSLRR, the levee design was updated and details are included in Section 6.2.

5.6 Constructability

After review of aerial photography and an initial site visit, it was clear that construction may be difficult for several reasons; 1) limited access points, 2) alignment adjacent to and across wetlands, 3) dewatering required for floodwall and possibly the levee inspection trench in multiple locations, 4) proximity to residences and 5) multiple levee wall transitions. These issues were considered in the development of the HSLRR cost estimate.

5.7 Broadway Closure Gate

In the 2007 Authorized Plan, a gate was provided at Broadway because the roadway could not be elevated to the design height due to site constraints. The Broadway closure was specified as a miter gate with a 40 ft wide opening and a 7 ft height. The support structure would be set back from the roadway five feet on either side, which will reduce the potential for impact by vehicles and provide space for pedestrian passage. A miter gate would require an extensive pile foundation. The closure type and size were reevaluated as part of the HSLRR and details are included in Section 6.4.

5.8 Flat and East Creek Gates

The 2007 Authorized Plan included the selection of sector gates for the closures on Flat and East Creeks. This decision was primarily based on the fact that sector gates can operate in areas with channel sedimentation more reliably than sluice gates. The sector gates, referred to as “storm gates” within the 2007 Authorized Plan, were sized using a UNET model to maintain tidal interchange of the wetland areas behind the line of protection. Each sector gate facility was proposed to be 35-feet wide to allow normal tidal flushing. For Flat Creek, the existing bridge over Union Avenue/Front Street is 25 feet wide and restricts the flow more than the proposed 35-foot wide downstream sector gate. For East Creek, a 35 foot wide sector gate was recommended just downstream of the existing Henry Hudson Trail bridge. Since the existing Jersey Avenue Bridge over East Creek is only 15 feet wide, this upstream bridge constricts the existing tidal flows. The downstream bridge for the Henry Hudson Trail is 34 feet wide. The proposed height of the sector gates is +15’ NGVD29. This alternative would require two sector gates, each about 17’ to 18’ wide to meet the necessary 35’ wide opening.

However, based on information from the nearby Keansburg project, the cost to maintain Sector gates is reported to be quite excessive. The construction costs are also quite high when compared to other possible options, such as sluice gates. Therefore, alternatives to sector gates were evaluated and the sector gates were replaced with a sluice gate and box culvert design in this HSLRR. Details are included in Section 6.5.

5.9 Conditions along East Creek

In the 2007 Authorized Plan, T-Wall on piles was aligned immediately along the top of the bank of East Creek from Sta 39+50 to Sta 43+50. However, a review of cross sections along this reach indicates that a portion of the unprotected side of the wall footing is in the creek. Since we do not have depths in the creek, there is concern that footing depth may be greater than planned resulting in taller stem and redesign. This issue requires further investigation in PED and is discussed in Section 7.6.

5.10 Interior Flooding Analysis

The interior flooding analysis was reevaluated and updated to utilize new computer models developed since the 2007 Authorized Plan. Both HEC-1 and HEC-IFH are now classified as legacy programs and are no longer supported. As such these models have been superseded by HEC-HMS (Hydrologic Modeling System) also developed by the USACE-HEC. The updated HEC-HMS models were built in order to assess the changes to the ponding elevations at the LOP due to: revisions in methodology for determining hypothetical rainfall data, the occurrence of additional storm events that changed the tailwater tide marigrams, and recalculation of ponding storage. The HEC-HMS models were built using the drainage structures, pump stations and ponding areas specified in the NED plan of the feasibility report. Tide marigrams used for the tailwater conditions of the outlet structures were updated taking into account additional storm events that have occurred since the feasibility study. The hypothetical precipitation developed in the feasibility study used the NWS Technical Paper No. 40, Rainfall Frequency Atlas of the United States and Hydro-Meteorological Report No. 35. These documents have been superseded by the NWS Atlas 14. The hypothetical precipitation was updated using Atlas 14. Ponding storage curves for East Creek and Flat Branch ponds were recalculated using the latest available GIS LIDAR mapping. The analyses done to reevaluate the interior flooding analysis are documented in Sub-Appendix F.

After HSLRR reevaluation, no immediate changes to the 2007 Authorized Plan were recommended. All drainage structures, ponding areas, and pump stations included in the 2007 Authorized Plan were incorporated into the updated HSLRR cost estimate. Recommendations for further reevaluation and analysis in PED are detailed in Section 7.7.

5.11 Dune and Beach Analysis

The 2007 Authorized Plan included a dune height and width of +17 ft (NGVD29) and 50 ft, respectively, as well as a berm at +9 ft (NGVD29). This template was reanalyzed using the computer model SBEACH with updated (post-Sandy) FEMA water surface elevations. SBEACH was applied to relate profile characteristics to levels of protection from storm damage. When a storm erodes a beach, the sand is usually not lost from the system. Rather, it is moved offshore, frequently into one or more bars. Low wave conditions after the storm will slowly move this material back onshore, rebuilding the berm. The analysis addresses the question of how much sand must be placed in a berm and dune to provide adequate protection from storms. The procedure applied in the HSLRR was to verify the target beach profile along the shoreline that would provide an appropriate level of erosion, flooding, and storm damage reduction to the structures at Union Beach, New Jersey, including the 2003 authorized advanced nourishment so that, at a minimum, the target profile would be maintained.

The SBEACH model was not calibrated for the Union Beach project site prior to data runs being made, because pre- and post-storm profiles were not available for the site. Profile data from September 1997 long profile surveys and NOAA coastal charts were used to depict the offshore bathymetry. Post Sandy onshore LIDAR was also compared to the 1997 profiles. A summary of this analysis is presented in Section 6.4 and the complete Dune and Beach Analysis is presented in Sub-Appendix D, SBEACH Modeling. The SBEACH analysis confirmed that the design dune and berm template provide sufficient elevation and volume of material to withstand a simulated 1 Percent Exceedance event and no changes are presently recommended. The originally authorized dune and berm template was incorporated into the HSLRR and all associated costs were updated.

A reanalysis of the dune and beach template utilizing updated profiles is recommended for PED (see Section 7.4).

Additionally, a sensitivity analysis was performed for the design dune and berm template utilizing the Sea Level Rise scenarios discussed in Sub-Appendix C, Sea Level Rise Analysis.

5.12 Overtopping & Failure Analysis

The 2003 Feasibility Report states that the Union Beach levee/floodwall system would provide "protection against the 100 year (1 % annual chance) storm with 92 % reliability...", and economic analyses of the 2007 Authorized Plan accrued benefits up to the levee/floodwall elevation of +15 feet NGVD29. This HSLRR incorporates lessons learned from Katrina regarding the susceptibility of levees and floodwalls when still water elevations allow waves to interact with the levee/floodwall system. As such, economic benefits calculations have been updated, and now incorporate levee/floodwall failure analyses for storms resulting in water surface elevations lower than +15 feet NGVD29.

As part of this HSLRR, five overtopping models were used to develop the mean overtopping flow rates for the different return intervals, and overtopping calculations were performed for stage elevations both with and without 0.7 feet of sea level rise over the period of analysis. Results of this analysis and the impacts on Project Design Performance are summarized in Section 6.10, and full details are included in Sub-Appendix E, Overtopping & Failure Analysis.

Overtopping was analyzed using 2013 FEMA Stage Elevations. The overtopping analysis, for the designed vertical wall and levee, used the interactive computer-based design and analysis system, Automated Coastal Engineering System, ACES, (which is based on equations found in the Corps of Engineers Coastal Engineering Manual (CEM)) and the online version of the European Overtopping Manual (EurOtop) (which is based off equations that can be found in the EurOtop Manual) for comparison. The analysis included the development of peak overtopping rates for various idealized return periods (2 year, 5 year, 10 year, 25 year, 50 year, 100 year, 200 year, and 500 year). ACES and EurOtop used both the Probabilistic and Deterministic approach to analyze the overtopping for the design of the vertical wall and levee. The project conditions utilized the vertical wall, levee, and dune/beach dimensions and elevations formulated in the previous Feasibility Report and were referenced to NGVD vertical datum.

For the project area the waves were determined to be impulsive and this was due to the relatively shallow water and breaking wave conditions in front of and at the wall. The same analysis that was done for the flood wall was used for the levee; however, the appropriate slope (angle) was used for the levee in lieu of a vertical wall. The full overtopping analysis is presented in Sub-Appendix E, Overtopping & Failure Analysis.

5.13 Quantity Estimates

As part of the HSLRR, the quantities were recalculated for the levees and floodwalls, including earthwork, concrete, reinforcing steel and pilings. Post-Sandy LIDAR was utilized to develop a topographic model and run profiles and cross sections for earthwork calculations. Reevaluation of the number, size, and location of drainage structures and pumping stations were not within the scope of the HSLRR. No new profiles were run or bathymetry surveyed offshore for the beach post-Sandy; however, the Feasibility beach profiles (1997) were translated to match the post Sandy onshore LIDAR. The total area required for mitigation was updated to account for additional wetland impacts due to easements requirements for the T-wall in place of I-Wall. The new mitigation areas were assumed to be adjacent to or an extension of the original areas.

Table 1 compares the 2007 Authorized Plan and the HSLRR, and significant quantity changes are discussed in Section 6.8.

5.14 Beachfill Borrow

In the 2007 Authorized Plan, one distinct borrow area was specified for beachfill, The area is located east of Sandy Hook, in the southwest corner of the Seabright '88 footprint (See green outlined area in Figure 13).This borrow site is approximately 18 miles from Union Beach (15.5 miles haul distance and 2.1 miles pumping distance). The total volume available within the area designated for Union Beach is 1.3 mcy. This computation discounts side slope volumes. The assumption was made that 33% of the material found will be deemed unsuitable by the dredger, i.e., a total of 1.3 mcy was delineated, of which 0.9-mcy is considered usable. The number of Vibracore is four. This factor of 33% is to account for uncertainty due to the limited data available. The material varies in grain size from fine sand (0.17mm median grain size) to coarse gravel (~32mm). The material is expected to contain negligible amounts of silt and/or clay. In a couple of instances the lithologic layers will contain 100% gravel, but on average not more than 25% of the material per area is expected to consist of gravel. In spite of the age of the information, we assumed the subsurface data remained accurate. The costs were updated for the HSLRR using unit costs from recent projects. Recommendations for additional investigation of the borrow area during PED are included in Section 7.8.

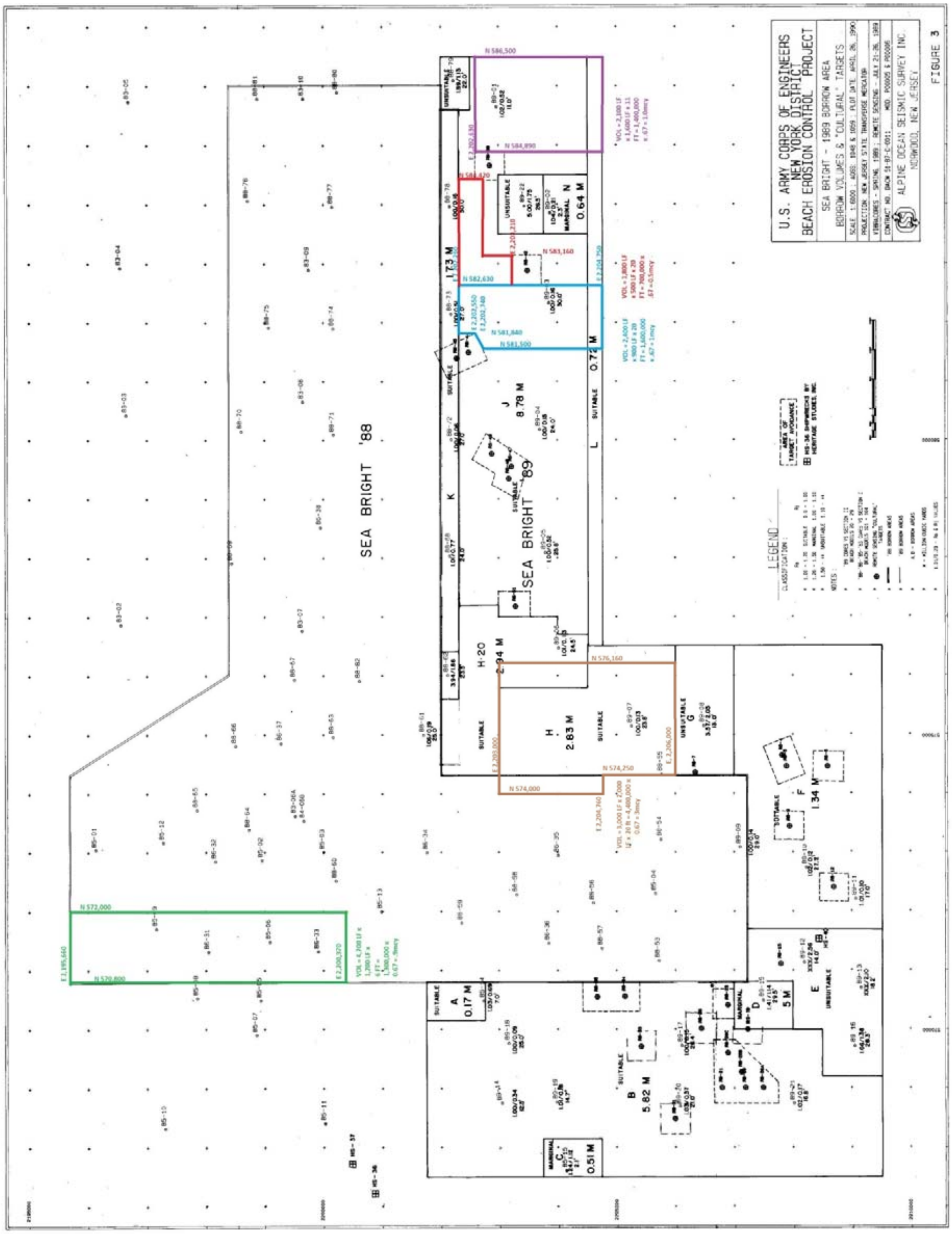


Figure 13. Sand Borrow Area at Sea Bright 88 Borrow Area (Green Outline)

6.0 CHANGES INCORPORATED INTO THE HSLRR

6.1 General

After reevaluation of the 2007 Authorized Plan, several changes were recommended and incorporated into the HSLRR. Additional changes recommended for the PED phase are discussed in Section 7. Below is a summary of the significant changes incorporated into the HSLRR:

- Limited alignment change for CBRA realignment by the US Fish and Wildlife Service
- Levee design revised.
- All floodwall changed from I- Wall to T-wall on piles.
- Broadway closure changed to roller gate.
- Flat and East Creek Sector Gates changed to Sluice Gates
- Updated easements.
- Updated quantities.
- Modified construction phasing.
- Project design performance – Overtopping & failure analysis.

6.2 Alignment Change for CBRAS Unit NJ-04

As stated in Section 5.2, the entire alignment for the 2007 Authorized Plan within the CBRS Unit NJ-04 was realigned to account for revisions made by the US Fish and Wildlife Service as a result of Hurricane Sandy. The revised unit alignment is included with the project alignment on project plan sheets 1-5.

6.3 Levee

The levee section in the 2007 Authorized Plan was updated to better address potential seepage risks in accordance with current design practices. Specifically, a blanket drain and a more robust toe drain extending into the foundation were included to assure adequate seepage control. The levee side slopes and footprint of the levee have not been changed. In addition, soil cement was added to the landside slope for overtopping protection. The updated levee cross section is shown in Figure 14. This section was used to update quantities and all associated costs in the HSLRR.

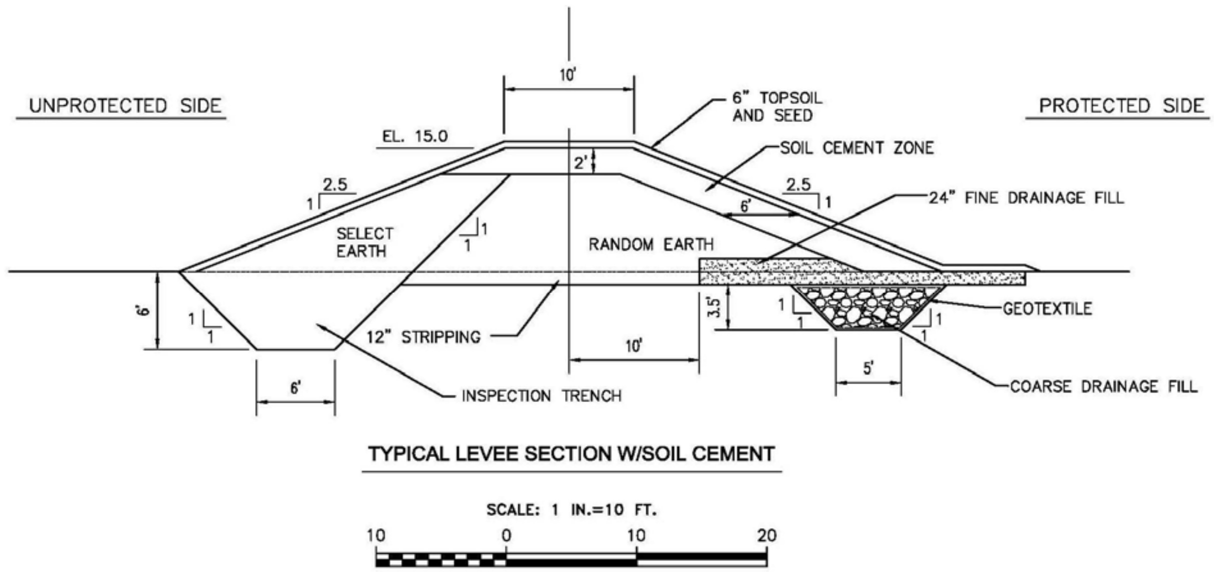


Figure 14. Proposed Levee Section (elevations in ft. NGVD)

For the HSLRR, it is assumed that the embankment will utilize a zone of select earth (impervious) consisting of more impervious material with a plasticity index (PI) greater than 5 and at least 25% fines. The random earth zone would consist of materials classified as GW, GM, GC, SW, SM, SC, ML, or CL or combinations thereof. The final design shall be based on best utilization of available materials. Materials for the blanket and toe drain shown in Figure 14 shall be designed utilizing New Jersey or AASHTO aggregate standards. The soil cement will be designed during PED based on the materials available. For the main levee, the typical section fill quantities increased about 13% due to the revised toe drain. The cross section for the interior berm presented in the 2007 Authorized Plan also had a central core of impervious and a toe drain with side slopes at 1V:2H. The purpose of this embankment is to prevent spring tides from inundating the low lying area along Harris Avenue. The interior levee geometry was unchanged for this study. However, the interior levee embankment composition was changed to all random earth and the toe drain eliminated. A typical section is shown in Figure 15.

For embankment materials, most of the suppliers listed in the 2003 study appear to be either sand and gravel suppliers or general contractors and no test reports were furnished indicating availability of supplying impervious levee fill. The design based material requirements primarily focused on desired permeability (hydraulic conductivity) parameters. The final design for all levees will be accomplished in PED phase after additional investigations are accomplished, as recommended in Section 7.2.

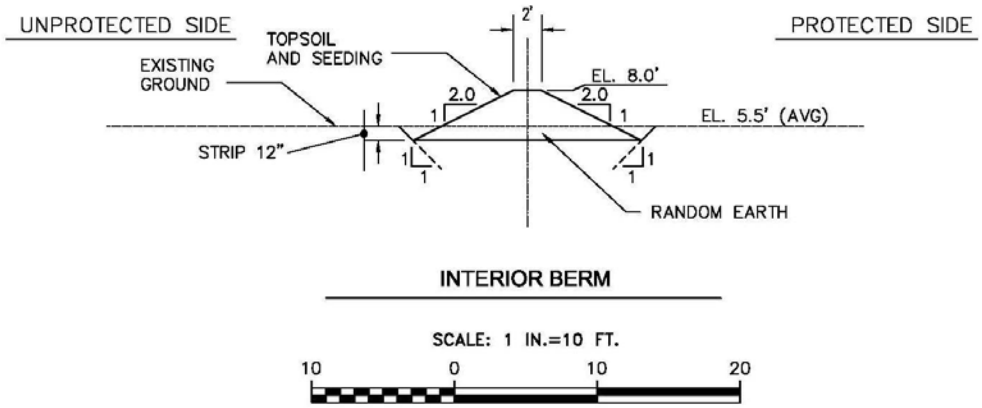


Figure 15. Revised Interior Berm (elevations in ft. NGVD)

6.4 Floodwall

As stated previously, updated criteria resulted in replacing 4,472 linear feet of I-Wall with T-Wall on piles along Chingarora Creek. This reach also included 496 linear feet of T-Wall on spread footings, which was also replaced with T-wall on piles. The 2007 Authorized Plan also included 1,929 linear feet of T-Wall on piles along Flat/East Creek due to foundation conditions. This T-wall on piles design was reviewed and used as the basis for the limited redesign for this reevaluation study. An analysis of pile capacity was also conducted. Due to the limited subsurface investigations, conservative assumptions were made for the wall and pile capacity design. It was decided that all the walls for the project should be T-Walls on piles. Further analysis of the existing ground elevations revealed that where a 20-foot stem was necessary a row of 4 piles repeating every 4 feet would be required (Figure 16). Where the stem height averages 14 feet, a row of 3 piles repeating every 4 feet would be required (Figure 17). These two typical revised wall sections were incorporated into the HSLRR. The preliminary design analysis for the wall and piles are detailed in the Sub-Appendix A, Floodwall Analysis, and Sub-Appendix B-1, Floodwall Pile Analysis West Walls and B-2, Floodwall Pile Analysis East Walls.. The T-wall design will be further analyzed in PED after additional subsurface explorations are completed as recommended in Section 7.3.

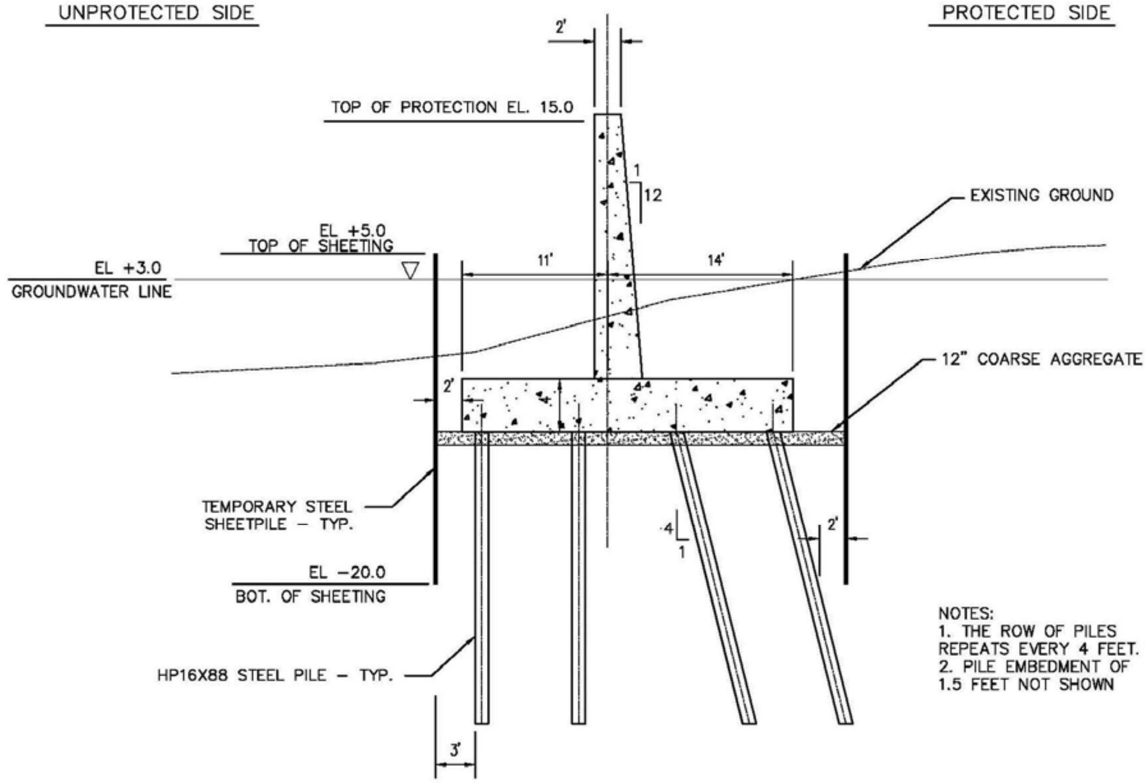


Figure 16. Revised Floodwall Section for 20-Foot Stem (elevations in ft. NGVD)

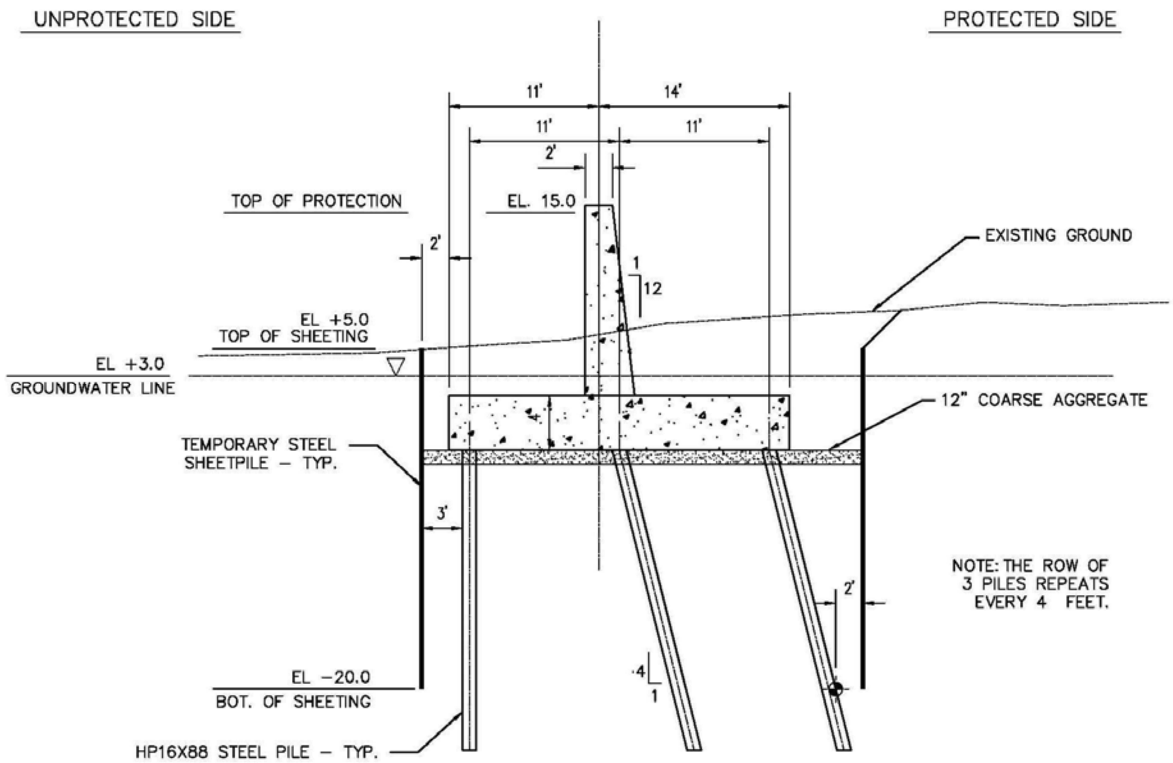


Figure 17. Revised Floodwall Section for 14-foot Stem (elevations in ft. NGVD)

6.5 Broadway Closure Gate

During reevaluation, alternatives to the authorized miter gate at Broadway were considered. Miter gates require an extensive pile foundation due to the swinging of the gate through a minimum of 90 degrees from open to close position. Review of the 2003 estimate revealed that the miter gate cost was based on a width of 40 feet, not the 50 ft specified in the selected plan. In addition, the overall feasibility miter gate cost appeared low when compared to miter gates constructed within the past 12 years for flood risk management projects on the Lackawanna River in Baltimore District.

Based on professional judgment and experience, a less expensive option is a horizontal roller gate, because it would have a simpler foundation but still could be closed just as quickly. The roller gate would need only a limited number of piles. The location and purpose of the Broadway closure remain the same, and therefore the change to a roller gate does not contradict the authorization.

Similar to the miter gate, the roller gate would have a 40-foot wide opening with a total length of 50 feet and be approximately 7 feet in height. The support structure will be set back from the roadway five feet on either side, which will reduce the potential for impact by vehicles and provide space for pedestrian passage. The roller gate in Bound Brook, NJ, shown in Figure 18, is 58 feet wide and 8 feet high. This type gate would require an abutment wall on one end and a section of floodwall behind where the gate is stored in the open position that would complete protection. A limited pile foundation may be required and this will be refined in PED. The change to roller gate was incorporated into the HSLRR.

Figure 18. Roller Gate Road Closure

6.6 Flat and East Creek Gates

The use of sector gates was determined to not be the most cost effective solution for this project.

Sector gates will always be partially submerged due to the normal water depths and the fluctuation of the tide. This constant wetting and drying requires ongoing maintenance including occasional repainting. In addition, when sector gates are closed during a storm, sediment and debris get trapped in the gate pockets requiring considerable effort before the gates can be reopened. If debris is not cleared, the gears that operate the gates could be damaged or stripped. The proposed sector gates have an opening of 35 feet and a height of 20 feet and will require a costly pile foundation.

Box culverts and sluice gates were determined to be less expensive and less maintenance intensive alternatives. The gates and culverts would be sized to provide equivalent tidal exchange and meet any other environmental and recreation requirements. As noted previously, Flat Creek is already restricted by the existing Union Avenue/Front Street Bridge to a width of 35 feet. East Creek is restricted by the 15-foot wide Jersey Avenue Bridge.

Additional foundation information from geotechnical investigations will also be utilized to refine the design in the PED phase. A possible configuration of sluice gates with box culverts is shown in Figure 19. The change to sluice gates with box culverts was incorporated into this HSLRR.

Figure 19. Sluice Gate Closure Example

6.7 Alignment and Easements

The project easements were reviewed for compliance with the USACE's vegetation management policy, ETL 1110-2-571, 10 April 2009, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams and Appurtenant Structures. The guidance requires 15 feet from levee toes, drains or structural features and 15 feet from the faces of floodwalls and a minimum of 8 feet beyond the footing. Details are shown in Figure 20 and Figure 21. This allows for operation and maintenance, surveillance, and access during high-water events. It is also recognized that unwanted vegetation has potential to impact the operations and performance of the system. Impact of vegetation to the flood damage risk management system apart from access and operation impediment includes compromising of foundation integrity if potential seepage paths are created by root penetration and/or root decay. In addition, significant levee damage and creation of points of concentrated seepage discharge can be created by the uprooting of large trees during a flood event. The root-free zone provides a margin of safety between the greatest expected extent of plant roots critical to the performance and reliability of the flood damage risk management system. The typical configuration for a levee, as set forth under USACE's vegetation management policy, is shown in Figure 20.

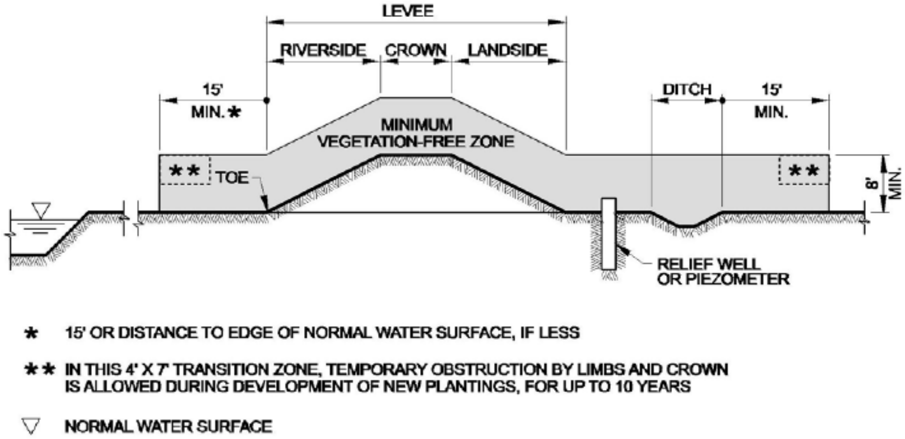


Figure 20. Typical Vegetation Free zone Configuration at Levee

Where the alignment component is a T-Wall, the vegetation-free zone extends horizontally 15-feet from the face of the wall and 8-feet minimum from the footing or any of the FRMP features. Just as in the case with the levee sections, the vertical extent of the vegetation-free zone is 8-feet minimum. The typical configuration for a T-Wall, as set forth under USACE's vegetation management policy, is shown in Figure 21.

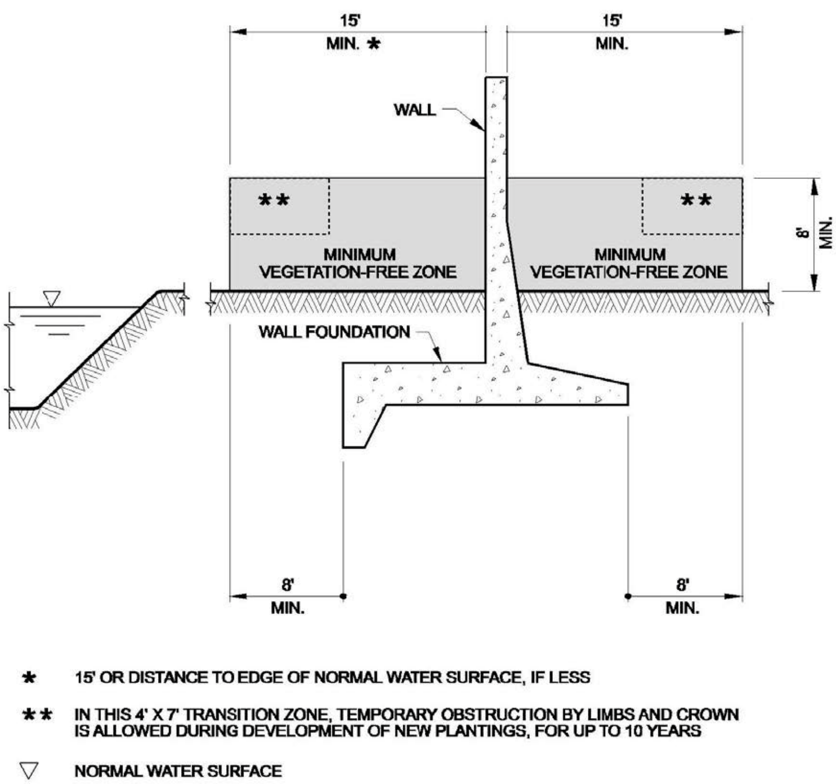


Figure 21. Typical vegetation free zone configuration at wall

The easements in the 2007 Authorized Plan did not include temporary easements on the unprotected side of the levees. This easement is required to enable construction. Therefore, 10 feet of temporary easement is included on the unprotected side. In addition, because T-Walls are required instead of I-Walls, additional perpetual easements to provide 21 feet (8 ft beyond footing) from the wall faces are required beyond the 10 ft provided originally. Due to time constraints, the real estate appraisal required an early decision on the wall easements before the wall design was fully refined. Therefore, a conservative assumed T-wall footing width of 30 ft was used to determine the easements and wetland impacts incorporated into the HSLRR. Final design refinements resulted in a T-Wall footing width of 25 feet. The increased easements due to criteria updates resulted in slightly greater impacts to properties during the detailed real estate analysis. Details on the real estate impacts are included in the Main Report Appendix D, Real Estate, and these issues will be addressed again during PED (See Section 7.5).

6.8 Quantity Estimates

HSLRR quantity estimates were calculated and results for the major project features are summarized in Table 1. Overall, the most significant change was the replacement of all I-wall with T-wall on piles due to criteria changes discussed previously in Section 5.4. On the Chingarora segment, 4,468 ft of I-wall and 488 ft of T-wall on spread footing was replaced with replaced with T-wall on piles in the HSLRR. Alignment changes associated with the CBRA (discussed in section 6.2) resulted in an increase in overall flood wall quantities. This change in floodwall type combined with updated criteria on the easements required for flood risk management projects (discussed in Sections 5.3 and 6.7) resulted in greater impacts to both real estate and wetlands, as seen in Table 1. Another small modification was an increase in levee fill volume for typical sections of 13%, as a result of the refined levee section which includes a blanket filter and a larger toe drain (See Section 6.3), and an overall reduction in total levee fill due to changing much of the Chingarora levee to floodwall to avoid CBRA impacts. As a result of realigning the line of protection to avoid CBRA impacts, there is an increase in the number of drainage outlets. All other quantities for the major project features in Union Beach, including: shorefront elements, drainage structures, storm gates, and pump stations, were unchanged from the 2007 Authorized Plan to the HSLRR.

6.9 Construction Phasing

The feasibility study proposed that there would be 8 phases of construction. This was partially due to the ability of the Sponsor, New Jersey Department of Environmental Protection (NJDEP), to provide its cost share while allowing time for plans and specifications preparation. However, under Hurricane Sandy funding, the 35% NJDEP share is paid upfront by the Federal Government. The NJDEP repays its share over 30 years. Upon completion of an initial estimate for this reevaluation study, it was decided that the project could be broken into 5 phases to properly account for escalation, multiple mob/demob, etc. yet move forward as quickly as possible. The anticipated phasing is as follows:[3]

Phase 1: Beachfront

The entire beachfront alignment would be constructed under one contract. Since this feature would be outflanked by a large storm event, consideration will be given to including the portion of levee parallel to Flat Creek to minimize wave damage to the condominium complex before phase 2 or 4 are completed.

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[3] Compliance with Clean Air Act is unknown at this point. Please note that the M2 reflects the previous phasing sequence. Current phasing is subject to change.

1) Account 02 – Relocations
   Outfall Extension, Dune Overwalk, and Dune Walkway

2) Account 10 – Breakwaters and Seawalls
   Terminal Groins

3) Account 17 – Total Beach Replenishment
   Entire Beachfill and Dune

Phase 2: Flat Creek to East Creek Levee and Floodwall and Interior Levee

This contract begins at the eastern terminal groin at the beachfront and extends along Flat Creek to Front Street before extending Oceanside of Brook Avenue toward East Creek. The levee/floodwall then parallels East Creek before turning east along the Henry Hudson Bike Trail and tying into Phase 3.

1) Account 02 – Relocations
   Raise Jersey Avenue and Harris Avenue
   Reconstruct Henry Hudson Bike Trail

2) Account 11 - Levees and Floodwalls
   Approximately 1,930 LF of concrete T-Wall on piles
   Approximately 1,640 LF of levee embankment
   3,390 LF of Interior Levee

3) Account 13 – Pumping Plant
   Flat Creek 250 CFS Pumping Station
   East Creek 100 CFS Pumping Station

4) Account 15 - Roadway Closure Structure
   7' x 50' Closure Gate (Broadway)

5) Account 15 – Flood Control Diversion Structures
   Flat Creek Sector Gate 20'H x 35' W
   East Creek Sector Gate 20'H x 35' W

6) Account 15 - Interior Drainage
   Interior drainage facilities for this segment includes 22 culverts with sluice and flap gates

Phase 3: East Creek Levee East of East Creek only

This contract represents the initial levee construction by beginning just east of East Creek and extending to the eastern tie-out with the existing Keansburg levee. The existing bikeway will be rebuilt on top of the new levee embankment. Drainage facilities include 3- 6'x6' tide gates with sluice gates and 4 – 60" culverts with sluice gates and flap gates

1) Account 02 – Relocations
   Raise Rose Lane access to IFF facility

Reconstruct bikeway

2) Account 11 - Levees and Floodwalls

3) Account 15 – Interior Drainage
Drainage structures for the East Creek tributary and the Natco Lake outlet

Phase 4: Chingarora Levee and Floodwall

Under this major contract the entire western reach of the levee and floodwall alignment would be constructed from the beginning near Bank Street across the Henry Hudson Bike Trail to the Broadway Closure Gate. Levee and floodwall continue along the rear of properties past Ash Street, along Bay Avenue and Chingarora Street, around the Regional Treatment facility, then parallel to Dock Street before tying into the western terminal groin and dune. Construction of drainage facilities will need to be closely coordinated with the levee and floodwall construction.

1) Account 02 – Relocations
Reconstruct Henry Hudson Bike Trail to ramp over levee

2) Account 11 - Levees and Floodwalls
Approximately 10,977 LF of concrete T-Wall on piles
Approximately 2,243 LF of levee embankment

3) Account 13 – Pumping Plant
Chingarora Creek 40 CFS Pumping Station

4) Account 15 - Roadway Closure Structure
7’ x 50’ Closure Gate (Broadway)

5) Account 15 - Interior Drainage
Interior drainage facilities for this segment include 40 culverts with sluice gates and flap gates ranging in size from 18” to 48” concrete pipe and a 4x4’ Box culvert.

Phase 5: Mitigation

All required mitigation would be constructed during this phase.

1) Account 06 - Fish and Wildlife Facilities
Wetland Mitigation

All drainage structures, pump stations, road raisings and pump stations would be constructed in their respective phases. Phase 1 construction would start in January 2018 and all phases would be completed by early 2022.

6.10 Overtopping & Failure Analysis

The Overtopping and Failure Analysis was also reevaluated for Union Beach. Although the Economic analyses of the 2007 Authorized Plan documented in the 2003 Feasibility Report accrued benefits up to the levee/floodwall elevation of +15 feet NGVD29, this HSLRR incorporates lessons learned from Katrina regarding the susceptibility of levees and floodwalls when still water elevations allow waves to interact with the levee/floodwall system.

Union Beach levees and floodwalls are subject to wave action during more severe events on the northeast and west-facing alignments. When the still-water elevation is significantly lower than the top of the levee/floodwall system at +15 feet NGVD29, small waves may break on the levee/floodwall system, but the freeboard (defined as the vertical distance between the top of the levee/floodwall system and flood waters) prevents waves from overtopping the system. When the still-water elevation approaches +15 feet NGVD29 – yet still below this elevation – less freeboard exists, and waves impacting the levee/floodwall system are more likely to result in overtopping.

Policy Guidance Letter No. 26, Benefit Determination Involving Existing Levees of 23 Dec 1991 defines the highest vertical elevation on the levee such that it is likely that the levee would not fail if the water surface elevation were to reach this level as the Probable Non-failure Point (PNP). It defines the lowest vertical elevation on the levee such that it is highly likely that the levee would fail as the Probable Failure Point (PFP). Highly likely is 85% confidence or greater. Using post-Katrina levee studies, and assuming soil cement reinforcing on the landward slopes of the levees, the non-failure point of the Union Beach levee/floodwall system would be +13.1 feet NGVD29 and the failure point of the system would be +13.6 feet NGVD29.

For this HSLRR, it was assumed that water accumulations behind the levee/floodwall system up to the failure point are negligible, and that the interior water elevations at the failure event are assumed to equal the bay stage elevations. It was likewise assumed that the interior water levels rise linearly between the non-failure point of +13.1 feet NGVD29 and the failure point of +13.6 feet NGVD. Both mean water surface elevations and 90% confidence water surface elevations were utilized in the overtopping models.

The 90% confidence interval results are as follows. At the beginning of the project life in 2022, the non-failure-point elevation of +13.1 feet NGVD29 corresponds to an event with a 26-year exceedance interval, and the failure-point elevation of +13.6 feet NVGD29 corresponds to an event with a 32-year exceedance interval. At the end of the project life, when 0.7 feet of sea level rise is assumed to occur, the non-failure-point elevation of +13.1 feet NGVD29 corresponds to an event with a 19-year exceedance interval, and the failure-point elevation of +13.6 feet NVGD29 corresponds to an event with a 32-year exceedance interval.

7.0 PRECONSTRUCTION ENGINEERING AND DESIGN (PED) CONSIDERATIONS

After the reevaluation of the 2007 Authorized Plan, a number of issues were determined to be beyond the scope of the HSLRR and will require further analysis in the Preconstruction Engineering and Design (PED) Phase. Below is a summary of the items to be considered during the PED Phase:

7.1 General

- Refine Levee Design.
- Refine Floodwall Design.
- Rerun SBEACH with updated profiles.
- Update easements.
- Update design and easements in vicinity of condos.
- Reexamine gate type for Flat and East Creeks.
- Reexamine conditions along East Creek, specifically erosion of stream bank and channel invert.

- Re-analyze Interior Flooding.
- Confirm quality of Beachfill Borrow.

7.2 Levee Design Refinement

The levee design will be refined during the PED phase. Additional investigations are recommended to1) more adequately determine the foundation properties for stability and seepage analyses 2) determine availability of embankment materials and 3) refine the overtopping protection design. An investigation to determine if levee armoring is necessary is also recommended, particularly at areas exposed to waves or at levee/floodwall transitions.

7.3 Floodwall Design Refinement

During the PED phase, additional subsurface investigations will be necessary to refine the floodwall design. A tentative drill plan has been prepared. This supplemental foundation data will help to determine the exact wall design necessary for different reaches as well as the pile type for the T-wall on piles. There is also concern regarding the number of wall bends that occur close to transitions from wall to levee embankment. These bends could result in wave diffraction and increase turbulence and cause erosion. Adjustments in the alignment to soften wall angles and minimize transitions to levee adjacent to wall bends along with the need for limited slope protection can be investigated in PED.

7.4 Beach, Dune, and Groins

Since no post-storm profiles were available for this study, a new analysis should be performed using new profiles which allow model calibration. Application of medium and high sea level rise scenarios could however cause inundation behind the primary dune line. Adaptations to the protective section could include an increase in dune crest height, and corresponding increase in berm elevation to compensate for increasing still water levels. However, application of these adaptability measures would require a post-authorization change report. Regular renourishment operations are part of the 2007 Authorized Plan. Also, the groin design should be refined during PED.

7.5 Alignment and Easements

As mentioned previously, the floodwall easements for the HSLRR were based on a conservative assumption of a T-wall footer width of 25 ft. After the floodwall design is refined in PED, the easements would be updated to get exact impacts to real estate and wetlands. Easements may also need to be adjusted in the vicinity of new property development One area in particular that will require further adjustment in PED is the condo area along Raritan Bay at the eastern dune/levee transition. After initial investigation, it appears that approximately 200 feet of the rock faced levee may require being replaced by floodwall in addition to adjustments in the alignment during PED. Additionally, some of the swimming pools, fences, and outbuildings that have been constructed along the alignment (since the 2007 Authorized Plan was approved) or that fall within the updated easements may be avoided during PED with adjustments to the alignment. A number of movable structures may be also need to be relocated as part of the real estate requirements.

7.6 Conditions along East Creek

In the 2007 Authorized Plan, T-Wall on piles was aligned immediately along the top of the bank of East Creek from Sta 39+50 to Sta 43+50. However, a review of cross sections along this reach indicates that a portion of the unprotected side of the wall footing is in the creek. Since we do not know exact depths in the creek, there is concern that footing depth may be greater than planned resulting in taller stem and redesign. Adjustments to the alignment to avoid erosion along the East Creek bank may be possible for limited reaches but the impact to residential properties is also a concern. It is recommended this issue be further investigated in PED.

7.7 Interior Flooding Analysis

During the PED phase, further study will be needed to refine the pumping requirements for the Chingarora, East Creek, and Flat Creek drainage areas to maximize ponding reductions provided by the pump stations. For the HSLRR pumping analysis, only the 100-yr with 100-yr tailwater scenario was computed. To better model this scenario and the other scenarios, actual low head pump curves should be selected to better portray real world conditions. In addition, peak pond with normal tide and peak pond elevation with the 2-year tailwater should also be modeled during the PED phase to provide better operational guidance under multiple head scenarios.

7.8 Beachfill Borrow

The information on the beachfill borrow area is 28 years old and will need to be reevaluated during the PED phase. Specifically, it is recommended that additional vibracores be accomplished during PED as well as a new survey. Since the sand for renourishment is proposed to come from upland sources per 2007 Authorized Plan, the sources need to be identified and impacts assessed.

8.0 CONCLUSIONS

Hurricane Sandy devastated the Borough of Union Beach and confirmed the need for the 2007 authorized USACE project. This HSLRR reevaluated the 2007 Authorized Plan for Union Beach. The alignment for the levee and floodwall along the Chingarora Creek side of the project were realigned to account for changes to the CBRS alignment.

The revision of the CBRS Unit NJ-04 by the US Fish and Wildlife Service required a revision to the alignment of the 2007 Authorized Plan. The replacement of nearly 5,000 ft of I-Wall with T-wall on piles increased costs considerably. There could be a reduction in the wall design, including footings, based on subsurface data to be collected during PED. Updated criteria requiring larger easements for flood risk management projects also increased the impacts to both real estate and wetlands in the area.

Four areas of the 2007 Authorized Plan are within the CBRS Unit NJ-04, and the team determined that a limited alignment change was warranted. The extent of realignment was constrained by the desire to minimize the effects to private real estate. The alignment change resulted in reducing the levee by 4,229 linear feet and increasing the T-Wall on piles by 5,979 linear feet for an overall increase in levee/floodwall length of 1,983 linear feet.

Additionally, three other modifications were incorporated into the HSLRR based on professional judgment: 1) the levee design was refined to assure adequate seepage protection, resulting in a minor increase in levee fill quantities, 2) the Broadway closure gate was switched from a miter gate to roller gate, resulting in a less extensive foundation and lower costs, and 3) the Flat and East Creek gates were switched from sector gates to sluice gates and box culverts. Updates to the cost estimate to account for changes in both materials and labor costs were also incorporated into the HSLRR.

The Project Design Performance was also reevaluated for Union Beach, incorporating lessons learned from Katrina regarding the susceptibility of levees and floodwalls when still water elevations allow waves to interact with the levee/floodwall system. Based on 90% confidence water surface elevations, at the beginning of the project life in 2022, the non-failure-point elevation of +13.1 feet NGVD29 corresponds to an event with a 26-year exceedance interval, and the failure-point elevation of +13.6 feet NVGD29 corresponds to an event with a 32-year exceedance interval.

The Union Beach Project sand dune and berm are sustainable up to the to the one percent chance event. The expected sacrificial erosion would occur which could then be repaired by placement of new sand. The levees are sustainable up to the overtopping elevation. An outer layer of soil cement was included on the crest and landside slope to limit erosion. The floodwalls do not have overtopping protection although the pile foundation should allow a limited amount of overtopping before sufficient erosion could occur resulting in excessive underseepage. The placement of rock on the landside and oceanside should be considered in final design.

The project consists of components that are adaptable to future increases in sea level due to climate change. The sand dune and berm cross section could include increases in dune crest height, and corresponding increase in berm elevation to compensate for increasing still water levels. The levee and wall systems could also be modified with parapet walls or additional wave baffles pending design analyses to support additional height. If applicable, additional pump station capacity could be added to handle additional overtopping. However, a post-authorization change report would be required to make these changes. Regular renourishment operations are part of the 2007 Authorized Plan.

Several items are recommended for further consideration during the PED phase. The most significant include: levee and floodwall design refinements, beach design with updated profiles, gate type for Flat and East Creeks, new development along the project alignment and real estate impacts.

HURRICANE SANDY LIMITED REEVALUATION REPORT
UNION BEACH, NEW JERSEY

DRAFT ENGINEERING APPENDIX

SUB APPENDIX A

PRELIMINARY FLOODWALL DESIGN

March 2014Revised March 2015

UNITED STATES ARMY CORPS OF ENGINEERS

UNION BEACH FLOOD PROTECTION PROJECT PRELIMINARY FLOODWALL DESIGN REPORT

14 March 2014

Leonard Mulé, PE
USACE Norfolk District
Chief Structural Engineering Section

March 14, 2014
Date

Wayne Miller, PE
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Structural Engineer

March 14, 2014
Date

EXECUTIVE SUMMARY

At the request of and working with U.S. Army Corps of Engineers (USACE) Baltimore District (NAB) in conjunction with the USACE New York District (NAN), the USACE Norfolk District (NAO), conducted a limited reevaluation of the proposed flood protection measures outlined in the "Union Beach, New Jersey Final Feasibility Report - Volumes I, II and III" (FFR) dated September 2003. The FFR listed several flood protection measures including T-type walls and I-walls. The intent was to develop a flood protection measure to be part of the larger hurricane and storm flood protection for a portion of the borough of Union Beach, New Jersey along the Raritan Bay. The primary goals of this analysis were to:

(1) Review and assess the flood wall designs provided in the FFR with regard to current USACE design methodology.
(2) Provide revised preliminary flood wall designs as required based on current USACE design documentation and project objectives.

The T-type wall modeled after Figure 57 in the FFR (see Appendix 6 page A18) was the basis of design. The findings of the load analyses based on "worst case" load scenarios on this cross section indicate Load Case 2 – Waves is the controlling compressive pile load case (55.2 kips) and Load Case 1 – Surge Stillwater Loading is the controlling tensile pile load case (17.7 kips).

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ABBREVIATIONS

ASCE	American Society of Civil Engineers
FFR	Union Beach, New Jersey Final Feasibility Report - Volumes I, II and III" dated September 2003
H&H	Hydrology and Hydraulics
NAB	Baltimore District
NAN	New York District
NAO	Norfolk District
SPM	Shore Protection Manual
USACE	U.S. Army Corps of Engineers

INTRODUCTION

The Union Beach project area is located in the northern portion of Monmouth County, New Jersey. It occupies approximately 1.8 square miles of land area along the coast of the Raritan Bay. The area has been subject to tidal inundation during storms, causing damage to structures in the low-lying residential community. Most of the flooding has been the result of storm surges from the Raritan Bay with backwater flow into the Chingarora, Flat and East Creeks, all of which pass through the project area. After considering the T-type walls and I-type walls proposed in the FFR, this report evaluates the T-type wall section founded on piles as depicted in Figure 57 of the FFR (see Appendix 6 page A18) using the design criteria outlined in USACE design documentation EM 1110-2-2104 for a “worst case” scenario. “Worst case” is defined as the most severe loading conditions on the wall section in accordance with USACE design standards that may be experienced during a 100 year storm event in this locality. The T-type walls founded on spread footings originally considered in the FFR have not been fully evaluated due to Real Estate concerns about the required footing width under “worst case” design loadings and limited soil information provided in the FFR. The proposed I-type walls in the FFR have not been evaluated per post Katrina USACE guidance in EC 1110-2-6066 and due to final grade uncertainties.

BACKGROUND

The most common types of flood walls are the cantilever T-type and cantilever I-type walls. Most flood walls are of the inverted T-type. The proposed flood wall section in this report is a T-type wall. The cross bar of the T serves as a base and the stem serves as the water barrier. The wall is to be supported on piles as indicated in Figure 57 of the FFR. A sheet pile cutoff can be included to control underseepage or provide scour protection for the foundation. At this time neither the sheet pile cutoff nor the scour protection has been included in this report. These items may be included in the analysis at the next stage of the project. In addition, the geometry of the wall section will be refined to more accurately reflect the topography as it relates to wall footing elevation and wall stem height.

I-type flood walls consist of driven sheet piles capped by a concrete wall. I-walls are most often used in connection with levee and T-walls junctions or for protection in narrow restricted areas where the wall height is not over 8 – 10 feet, depending on soil properties and geometry. Given the uncertainties with respect to the topography, soil properties and post Katrina USACE guidance, I-type walls have not been considered in this report.

Since flood walls are usually a primary feature of a local protection project, they must be designed for the most economical cross section per unit length of wall, because they often extend for great distances. Added to this need for an economical cross section is the requirement for safety. The consequences of failure for a flood wall are normally very great since it protects valuable property and human life. Thus, the design of a flood wall is a complex process including safety and economy factors. The design must be executed in a logical, conservative manner based on the function of the wall and the consequences of failure.

An adequate assessment of stability must be include a rational assessment of loads and must account for the basic structural behavior, the mechanism of transmitting compressive and shearing loads to the foundation, the reaction of the foundation to such loads, and the secondary effects of the foundation behavior on the structure.

Some of the critical aspects of design include:

- Preliminary estimates of geotechnical and hydraulic data, subsurface conditions, and the type of structures suitable for the foundation.
- Selection of design parameters, loading conditions, loading effects, potential failure mechanisms, and other related features of the analytical model.
- Evaluation of the technical and economic feasibility of alternative structures.
- Constructability reviews in accordance with ER 1110-1-803
- Refinements of the preliminary structure configuration to reflect the results of detailed site exploration, material availability studies, laboratory testing, and numerical analysis.

- Modification to the structure configuration during construction due to unexpected variations in the foundation conditions.

Flood walls accommodate a difference in soil and/or water elevation over a typically short horizontal distance. On one side of the wall, the driving side, lateral forces exceed those on the opposite side, the resisting side; the force difference and resulting moment are accommodated by forces and pressures developed along the base. Lateral forces may be related to gravity, water seepage, waves, wind and earthquakes.

Earth pressures and forces will be present below grade on both the unprotected and protected sides of the flood wall. NAB Geotechnical Section will provide design values for soil parameters at the next stage of the design. The soil parameters used for this report are based on the soil values contained in the FFR. Earth pressures and forces have been developed using the procedures outlined in Chapter 3 of EM 1110-2-2502.

Water pressures and forces will be present below the groundwater level on both the unprotected and protected sides of the flood wall. In addition, water pressures and forces will be present below the flood level on both the unprotected side of the flood wall. NAB Geotechnical Section should provide seepage analysis. The seepage analysis in this report is based on the information contained in the FFR. Water pressures and forces will be developed using the procedures outlined in Chapter 3 of EM 1110-2-2502.

The wave forces on the flood wall have been calculated by NAO H&H using the Goda Method referenced in Chapter 7 of the Shore Protection Manual (SPM).

Wind pressures and forces have been considered for flood walls during construction. Wind loads can act at any time in the life of the floodwall. In locations subject to hurricanes, a wind load of 50 psf can be used conservatively for walls 20 feet or less in height for wind speeds up to 100 mph. For more severe conditions, the wind loads should be computed in accordance with American Society of Civil Engineers (ASCE) 7-05. For this report a wind load of 50 psf has been used.

The calculation of seismic loads induced by earthquakes has been calculated using ASCE 7-05.

DESIGN NARRATIVE

The proposed T-type flood wall founded on piles in the FFR was designed by URS Greiner Woodward Clyde. It appears the URS analysis:

- assumes cohesionless soils in determining the pile tip capacity. This assumption is to be reviewed by NAB Geotechnical Section as part of the pile foundation design. The design analysis may be revised based on updated soil parameters to be provided by NAB Geotechnical.

- uses the dimensions of the spread footing T-type wall for calculating the design forces and moments of the pile founded T-type wall. The calculations in this NAO preliminary report use the dimensions as indicated in Figure 57 of the Feasibility Report for a pile founded T-type wall.

- assumes cohesionless soils in determining the design forces and moments for the pile founded T-Type wall. The proper soils information to be used in the final design should be provided by the NAB Geotechnical Section. However for this stage of the design the soils information as recorded in the FFR is used for the basis of the design calculations in this NAO preliminary report. Possible revision(s) may include the effect of cohesive soils at -3.0' elevation.

- uses active and passive lateral soil coefficients, K_a and K_p. The design guidance provided by USACE EM 1110-2-2502 indicates the use of at-rest lateral soil coefficient K_o for determining design forces and moments and neglecting the resisting forces provided by the passive soil. The calculations in this NAO preliminary report follow the guidance provided by USACE EM 1110-2-2502 and conservatively neglect the effect of passive soil resistance.

- uses uplift forces calculated with the Line-of-Creep method. The Line-of-Creep method outlined in EM 1110-2-2502 was used in the calculations for the analysis in this NAO preliminary report.

- does not include wave forces in their analysis. The appropriate wave force values for load case C2 - Wave Forces (load case C2a, non-breaking waves and load case C2b, breaking waves) have been calculated by NAO Hydrology and Hydraulics (H&H). NAO H&H used the Goda Method to calculate the wave force magnitudes based on a 100 year storm event in this analysis. The force for load case C2c, broken waves is not included in the analysis per directions from NAO H&H.

The T-type reinforced concrete flood wall section depicted in Figure 57 of the FFR is used as the basis of this review and analysis included in this preliminary report. The analysis was performed using the methods indicated in EM 1110-2-2502.

- An initial overturning analysis of a spread footing T-type wall indicated a larger footing than depicted in Figure 57 is required for stability under “worst case” loadings. The minimum footing width increased from 25 feet to 44 feet. This increase in footing width had potential Real Estate concerns; therefore, this spread footing T-type wall option was no longer considered. The remainder of the design emphasis concentrated on the pile founded option shown in Figure 57 of the FFR. This wall section indicates a 25 foot wide footing which is used in this analysis.

- A 1:12 batter was added to the protected side of the vertical stem in an effort to improve the efficiency of the stem as a bending member.

- Figure 57 of the FFR depicts (4) piles per row spaced at 7'-4" on center. In order to obtain a similar design with regard to pile spacing, the resultant horizontal and vertical loads were supplied to NAB Geotechnical Section. The number of required piles, their spacing and size has been determined with the assistance NAB Geotechnical Section. Based upon the preliminary pile capacities from NAB the spacing as shown in Figure 57 is sufficient. The type and size of pile will be revisited based on updated soil parameters upon approval to proceed with the design.

- The 6:1 pile batter was changed to 4:1 in an effort to more effectively engage the battered piles in resisting the lateral loads. This change may be altered depending on the updated soil information.

- Prediction of breaking wave forces on walls is required for the design of wall structures in coastal waters. The standard procedure utilized the Minikin method documented in the SPM. As the Minikin method is based on the shock pressure caused by breaking waves, the resulting forces and structure designs analyzed by using this procedure are generally considered to be very conservative. The SPM cautions the user about the extremely high wave forces associated with the Minikin method. The Goda Method is an alternative to the Minikin method of the determination of non-breaking and breaking wave forces.

  The rationale of the Goda method (1985) for design analysis is the duration of the impulsive breaking wave force is relatively brief, on the order of a tenth of one hundredth of a second, and the effect of the force on the stability of massive concrete structures, particularly those with rubble mound bases, may be rather insignificant.

  NAO H&H has determined the broken wave is not a viable wave condition for the wall alignment, therefore, these calculation do not include Case C2c. The wave forces have been calculated using the appropriate frequency curve data for a 100 year storm event for the associated wave forces using the Goda method for non-breaking and breaking wave conditions (Cases C2a & C2b).

The following pages contain a summary of the design calculations based on the guidance in EM 1110-2-2502 Coastal Flood Wall Design Load Case Criteria. Each summary of calculations has a sketch of the associated horizontal and vertical loads and the associated load distribution to the pile foundation. The applied loads in the sketches are “keyed” to the calculation worksheets found in the appendices. In addition, the first three load case calculations are also summarized for scour conditions. This is a conservative assumption that yields the maximum uplift tensile pile load of 17.7kips for Load Case 1 and a maximum compressive pile load of 55.2 kips for Load Case 2.

Case C1 - Surge Stillwater Loading

- Backfill is in place to the final elevation
- Water is at the surge stillwater level on the unprotected side
- Wave forces are excluded
- Uplift is acting

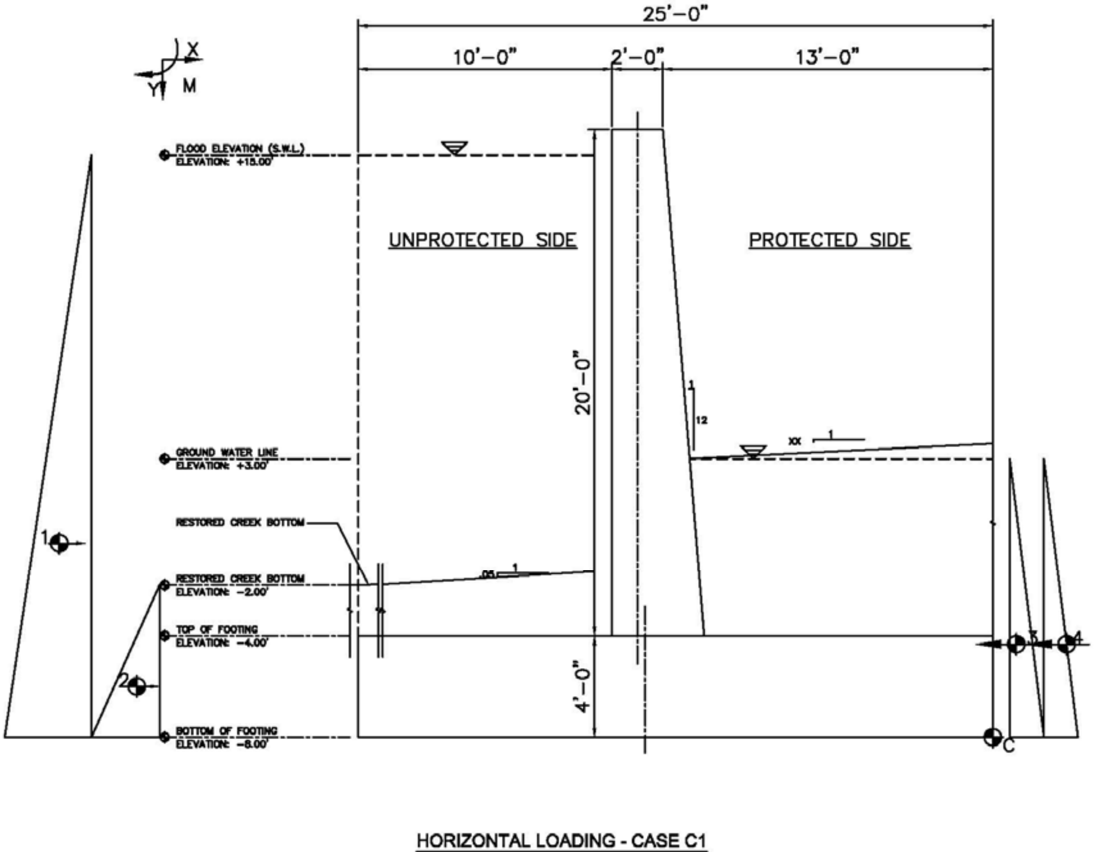


Figure 1a
Case C1 - Surge Stillwater Loading – Horizontal Loads

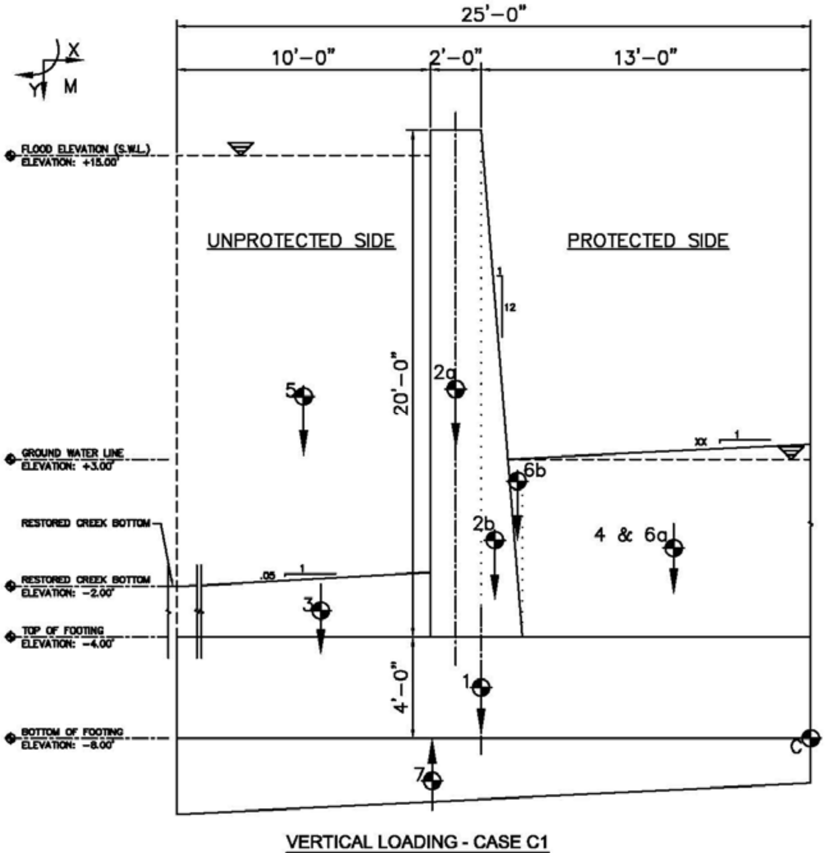


Figure 1b
Case C1 - Surge Stillwater Loading – Vertical Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, ɣ [lb/ft^3]	125.00	110.00
Water Unit Weight, $ɣ_w$ [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, $ɣ_{eff}$ [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	16504.80	7.67	126536.80
Soil (Unprotected Side)	451.67	2.00	903.34
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1518.11	3.67	-5566.41
Wave Force 1	0.00	0.00	0.00
Wave Force 2	0.00	0.00	0.00
	11663.16	**-9.26**	**108031.33**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-29380.00	**-13.42**	**394333.33**

Figure 1c
Case C1 - Surge Stillwater Loading – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	1102.00	20.00	22040.00
Soil Above Toe (Protected Side)*	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	11856.00	20.00	237120.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**45907.07**	**-13.42**	**-616062.53**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	11663.16
Resultant Vertical Magnitude [lb/ft of wall]	16527.07
Resultant Magnitude [lb/ft of wall]	20228.03
Resultant Orientation [°]	54.79
Overturning Moment [lb-ft/ft of wall]	502364.67
Righting Moment [lb-ft/ft of wall]	-616062.53
Net Moment @ Toe [lb-ft/ft of wall]	-113697.86
Resultant Location (Measured From C) [ft]	-6.88
Eccentricity (Measured From Footing CL) [ft]	5.62

Figure 1d
Case C1 - Surge Stillwater Loading – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	9039.07	-24390.08	-15351.01
Pile Load, Q2 [lb]	9039.07	-8130.03	909.04
Pile Load, Q3 [lb]	9039.07	8130.03	17169.09
Pile Load, Q4 [lb]	9039.07	24390.08	33429.14

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	9039.07	-24390.08	-15351.01
Pile Load, Q2 [lb]	9039.07	-8130.03	909.04
Pile Load, Q3 [lb]	14426.65	7887.28	22313.93
Pile Load, Q4 [lb]	14426.65	23661.85	38088.50

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 1 - Stillwater Loading:

Maximum Tensile Load [lb]: -15351.01

Maximum Compressive Load [lb]: 38088.50

Figure 1e
Case C1 - Surge Stillwater Loading – Design Calculations Summary – Pile Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	16504.80	7.67	126536.80
Soil (Unprotected Side)	0.00	3.67	0.00
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1518.11	3.67	-5566.41
Wave Force 1	0.00	0.00	0.00
Wave Force 2	0.00	0.00	0.00
	11211.49	**-9.56**	**107127.99**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-29380.00	**-13.42**	**394333.33**

Figure 1f

Case C1 - Surge Stillwater Loading with Scour – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**37317.07**	**-11.91**	**-444262.53**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	11211.49
Resultant Vertical Magnitude [lb/ft of wall]	7937.07
Resultant Magnitude [lb/ft of wall]	13736.61
Resultant Orientation [°]	35.30
Overturning Moment [lb-ft/ft of wall]	501461.33
Righting Moment [lb-ft/ft of wall]	-444262.53
Net Moment @ Toe [lb-ft/ft of wall]	57198.80
Resultant Location (Measured From C) [ft]	7.21
Eccentricity (Measured From Footing CL) [ft]	19.71

Figure 1g
Case C1 - Surge Stillwater Loading with Scour – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	7937.07	-25594.71	-17657.65
Pile Load, Q2 [lb]	7937.07	-8531.57	-594.50
Pile Load, Q3 [lb]	7937.07	8531.57	16468.64
Pile Load, Q4 [lb]	7937.07	25594.71	33531.78

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	7937.07	-25594.71	-17657.65
Pile Load, Q2 [lb]	7937.07	-8531.57	-594.50
Pile Load, Q3 [lb]	13138.46	8276.84	21415.30
Pile Load, Q4 [lb]	13138.46	24830.52	37968.98

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

Maximum Total Load due to the Case 1 - Stillwater Loading with Scour:

Maximum Tensile Load [lb]: -17657.65

Maximum Compressive Load [lb]: 37968.98

Figure 1h

Case C1 - Surge Stillwater Loading with Scour – Design Calculations Summary – Pile Loads

Case C2 – Breaking & Non-breaking Wave Loading

- Same as Case C1
- Breaking & Non-breaking wave loading added (Goda Method)
- Uplift is acting

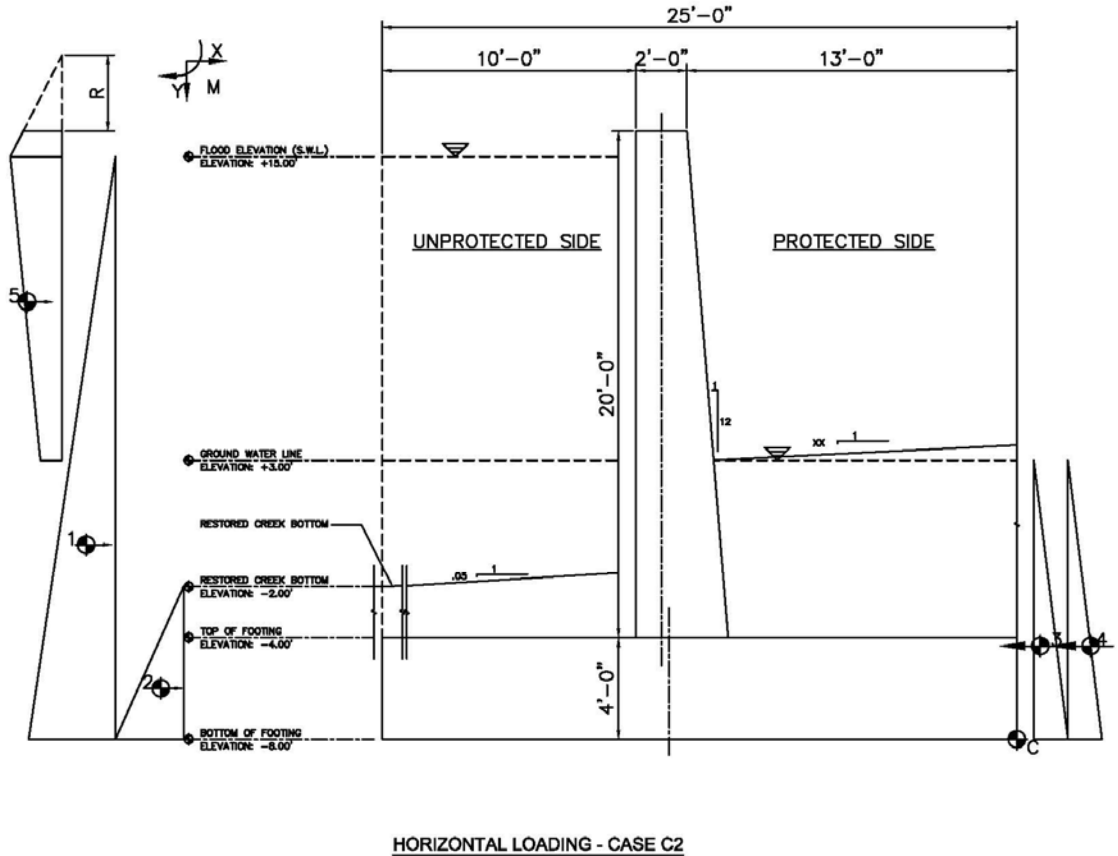


Figure 2a

Case C2 – Breaking & Non-breaking Wave Loading – Horizontal Loads

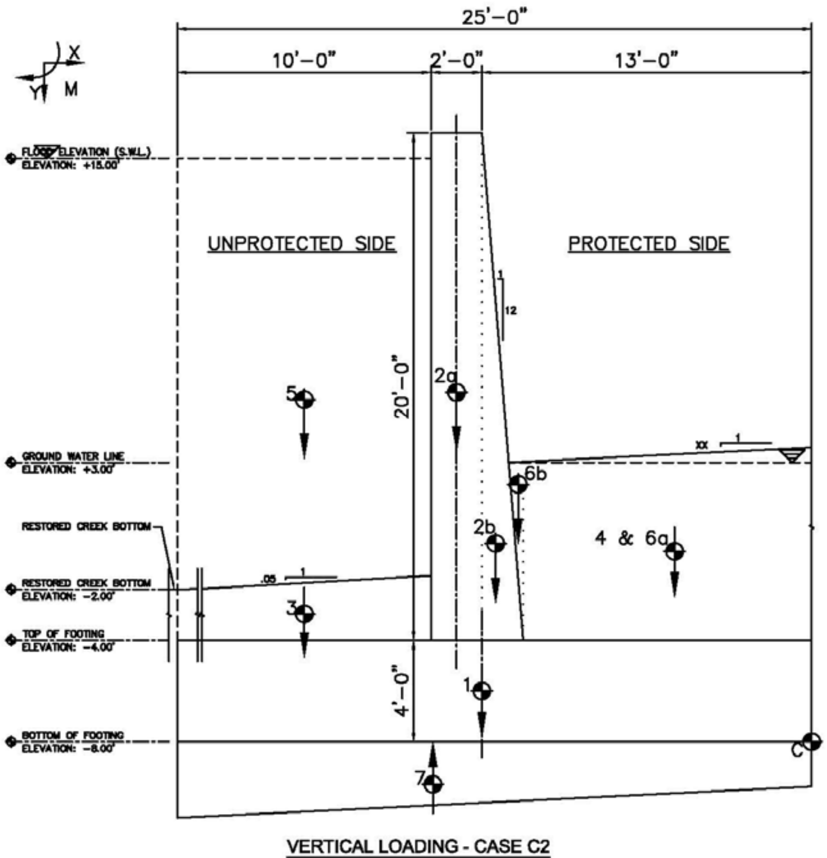


Figure 2b
Case C2 – Breaking & Non-breaking Wave Loading – Vertical Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, $γ_w$ [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	16504.80	7.67	126536.80
Soil (Unprotected Side)	451.67	2.00	903.34
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1518.11	3.67	-5566.41
Wave Force (100 Year Event)	6144.45	16.21	99609.15
Wave Force 2	0.00	0.00	0.00
	17807.61	**-11.66**	**207640.48**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-29380.00	**-13.42**	**394333.33**

Figure 2c
Case C2 – Breaking & Non-breaking Wave Loading – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	1102.00	20.00	22040.00
Soil Above Toe (Protected Side)*	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	11856.00	20.00	237120.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**45907.07**	**-13.42**	**-616062.53**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	17807.61
Resultant Vertical Magnitude [lb/ft of wall]	16527.07
Resultant Magnitude [lb/ft of wall]	24295.16
Resultant Orientation [°]	42.86
Overturning Moment [lb-ft/ft of wall]	601973.82
Righting Moment [lb-ft/ft of wall]	-616062.53
Net Moment @ Toe [lb-ft/ft of wall]	-14088.71
Resultant Location (Measured From C) [ft]	-0.85
Eccentricity (Measured From Footing CL) [ft]	11.65

Figure 2d
Case C2 – Breaking & Non-breaking Wave Loading – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	16527.07	-31499.94	-14972.87
Pile Load, Q2 [lb]	16527.07	-10499.98	6027.09
Pile Load, Q3 [lb]	16527.07	10499.98	27027.05
Pile Load, Q4 [lb]	16527.07	31499.94	48027.00

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	16527.07	-31499.94	-14972.87
Pile Load, Q2 [lb]	16527.07	-10499.98	6027.09
Pile Load, Q3 [lb]	24671.57	10186.48	34858.05
Pile Load, Q4 [lb]	24671.57	30559.43	55231.00

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 2 - Wave Loading:

Maximum Tensile Load [lb]:	-14972.87
Maximum Compressive Load [lb]:	55231.00

Figure 2e

Case C2 – Breaking & Non-breaking Wave Loading – Design Calculations Summary – Pile Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	16504.80	7.67	126536.80
Soil (Unprotected Side)	0.00	3.67	0.00
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1518.11	3.67	-5566.41
Wave Force 1	0.00	0.00	0.00
Wave Force 2	0.00	0.00	0.00
	11211.49	**-9.56**	**107127.99**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-29380.00	**-13.42**	**394333.33**

Figure 2f
Case C2 – Breaking & Non-breaking Wave Loading with Scour – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**37317.07**	**-11.91**	**-444262.53**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	11211.49
Resultant Vertical Magnitude [lb/ft of wall]	7937.07
Resultant Magnitude [lb/ft of wall]	13736.61
Resultant Orientation [°]	35.30
Overturning Moment [lb-ft/ft of wall]	501461.33
Righting Moment [lb-ft/ft of wall]	-444262.53
Net Moment @ Toe [lb-ft/ft of wall]	57198.80
Resultant Location (Measured From C) [ft]	7.21
Eccentricity (Measured From Footing CL) [ft]	19.71

Figure 2g
Case C2 – Breaking & Non-breaking Wave Loading with Scour – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	11053.80	-28727.57	-17673.77
Pile Load, Q2 [lb]	11053.80	-9575.86	1477.94
Pile Load, Q3 [lb]	11053.80	9575.86	20629.66
Pile Load, Q4 [lb]	11053.80	28727.57	39781.37

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	11053.80	-28727.57	-17673.77
Pile Load, Q2 [lb]	11053.80	-9575.86	1477.94
Pile Load, Q3 [lb]	19879.02	9289.95	29168.97
Pile Load, Q4 [lb]	19879.02	27869.84	47748.86

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 2 - Wave Loading:

Maximum Tensile Load [lb]: -17673.77

Maximum Compressive Load [lb]: 47748.86

Figure 2h
Case C2 – Breaking & Non-breaking Wave Loading with Scour – Design Calculations Summary – Pile Loads

Case C3 - Seismic Loading

- Backfill is in place to the final elevation
- Water is at the usual (non-storm) level
- Earthquake forces are included
- Uplift is acting

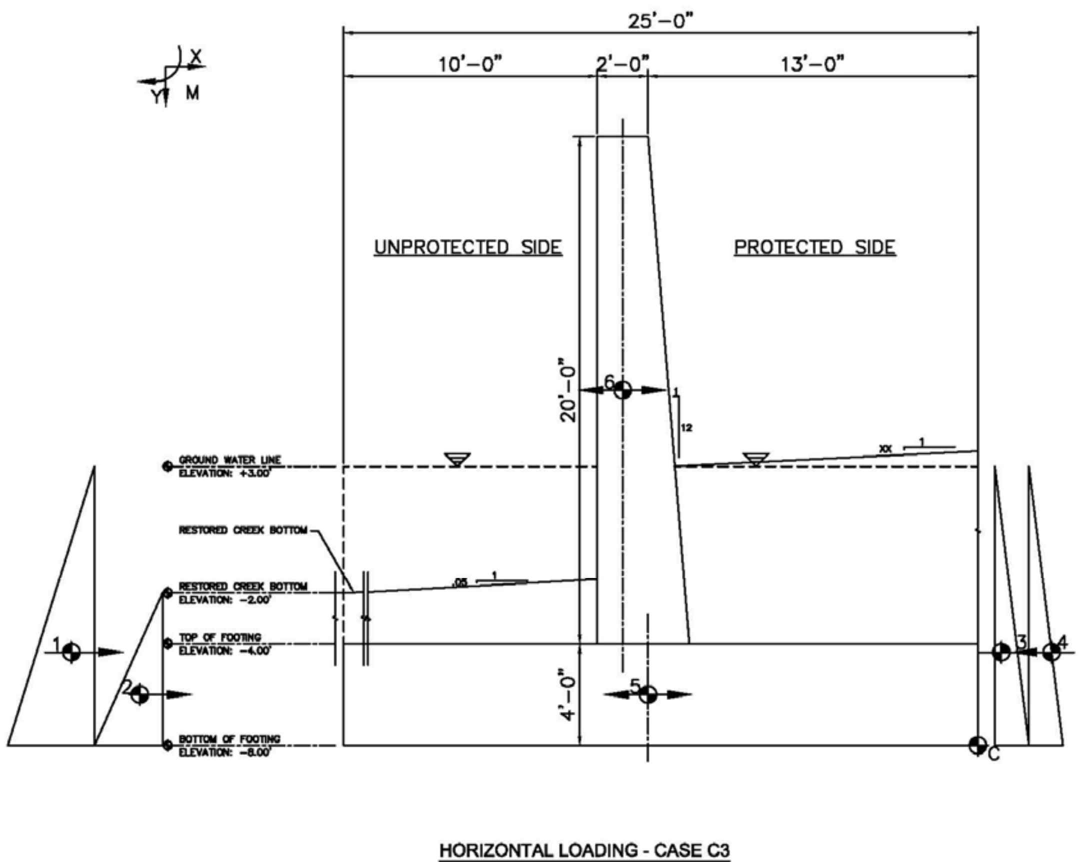


Figure 3a
Case C3 – Seismic Loading – Horizontal Loads

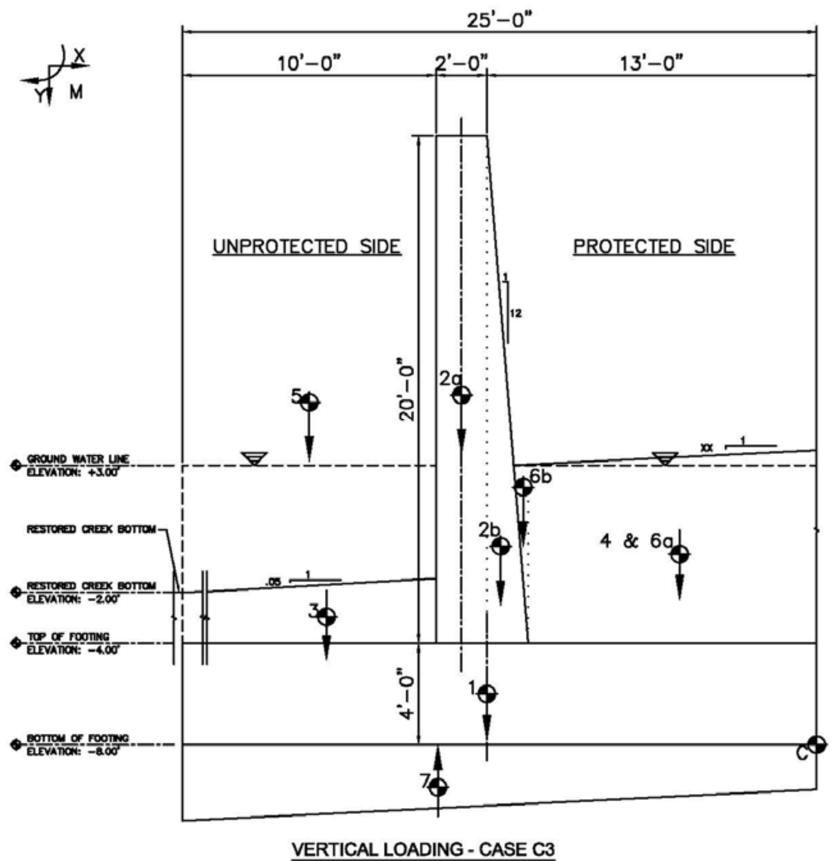


Figure 3b
Case C3 – Seismic Loading – Vertical Loads

Seismic Design Parameters	
S_{DS}	0.27
I_e	1.50
C_s	0.03
V	4152.13

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	532.68	2.33	1242.91
Soil (Unprotected Side)	134.39	0.67	89.59
Ground Water (Protected Side)	-603.70	2.33	-1408.64
Soil (Protected Side)	0.00	0.00	0.00
Wall Footing	1829.25	2.00	3658.50
Wall Stem with Batter	1036.58	14.00	14512.05
	2929.19	**-6.18**	**18094.42**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-17160.00	**-12.50**	**214500.00**

Figure 3c
Case C3 – Seismic Loading – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	1102.00	20.00	22040.00
Soil Above Toe (Protected Side)*	0.00	5.67	0.00
Water Above Heel (Unprotected Side)	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**34047.80**	**-12.97**	**-441532.02**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	2929.19
Resultant Vertical Magnitude [lb/ft of wall]	16887.80
Resultant Magnitude [lb/ft of wall]	17139.95
Resultant Orientation [°]	80.16
Overturning Moment [lb-ft/ft of wall]	232594.42
Righting Moment [lb-ft/ft of wall]	-441532.02
Net Moment @ Toe [lb-ft/ft of wall]	-208937.60
Resultant Location (Measured From C) [ft]	-12.37
Eccentricity (Measured From Footing CL) [ft]	0.13

Figure 3d
Case C3 – Seismic Loading – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	21259.07	-5241.31	16017.76
Pile Load, Q2 [lb]	21259.07	-1747.10	19511.96
Pile Load, Q3 [lb]	21259.07	1747.10	23006.17
Pile Load, Q4 [lb]	21259.07	5241.31	26500.38

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	21259.07	-5241.31	16017.76
Pile Load, Q2 [lb]	21259.07	-1747.10	19511.96
Pile Load, Q3 [lb]	22045.19	1694.94	23740.13
Pile Load, Q4 [lb]	22045.19	5084.82	27130.01

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 3 - Seismic Loading:

Maximum Tensile Load [lb]: N/A

Maximum Compressive Load [lb]: 27130.01

Figure 3e
Case C3 – Seismic Loading – Design Calculations Summary – Pile Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	16504.80	7.67	126536.80
Soil (Unprotected Side)	0.00	3.67	0.00
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1518.11	3.67	-5566.41
Wave Force 1	0.00	0.00	0.00
Wave Force 2	0.00	0.00	0.00
	11211.49	**-9.56**	**107127.99**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-29380.00	**-13.42**	**394333.33**

Figure 3f
Case C3 – Seismic Loading with Scour – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	127.40	11.53	1468.64
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**37317.07**	**-11.91**	**-444262.53**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	11211.49
Resultant Vertical Magnitude [lb/ft of wall]	7937.07
Resultant Magnitude [lb/ft of wall]	13736.61
Resultant Orientation [°]	35.30
Overturning Moment [lb-ft/ft of wall]	501461.33
Righting Moment [lb-ft/ft of wall]	-444262.53
Net Moment @ Toe [lb-ft/ft of wall]	57198.80
Resultant Location (Measured From C) [ft]	7.21
Eccentricity (Measured From Footing CL) [ft]	19.71

Figure 3g
Case C3 – Seismic Loading with Scour – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	20157.07	-8135.63	12021.44
Pile Load, Q2 [lb]	20157.07	-2711.88	17445.19
Pile Load, Q3 [lb]	20157.07	2711.88	22868.94
Pile Load, Q4 [lb]	20157.07	8135.63	28292.70

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	20157.07	-8135.63	12021.44
Pile Load, Q2 [lb]	20157.07	-2711.88	17445.19
Pile Load, Q3 [lb]	20910.90	2630.91	23541.81
Pile Load, Q4 [lb]	20910.90	7892.72	28803.62

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

Maximum Total Load due to the Case 3- Seismic with Scour:

Maximum Tensile Load [lb]:	N/A
Maximum Compressive Load [lb]:	28803.62

Figure 3h

Case C3 – Seismic Loading with Scour – Design Calculations Summary – Pile Loads

Case C4 - Construction Loading

- Floodwall is in place
- Applicable loads (short duration) which are possible during construction period are applied to the wall
- Loads may include strong winds or construction equipment

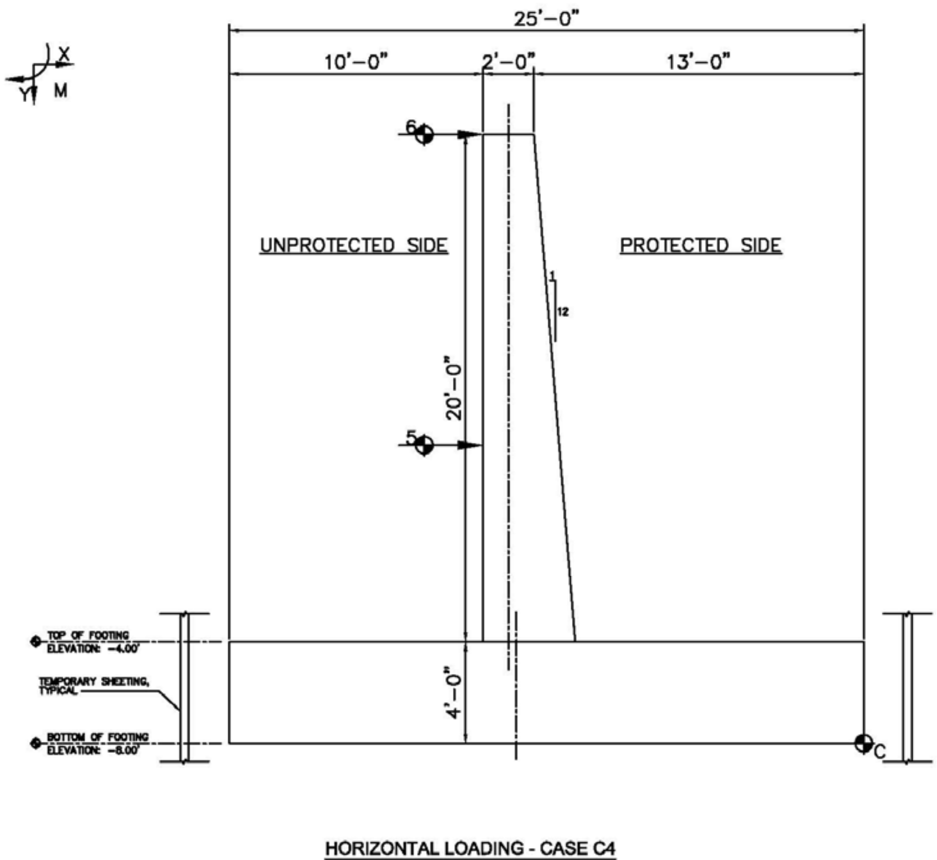


Figure 4a
Case C4 - Construction Loading – Horizontal Loads

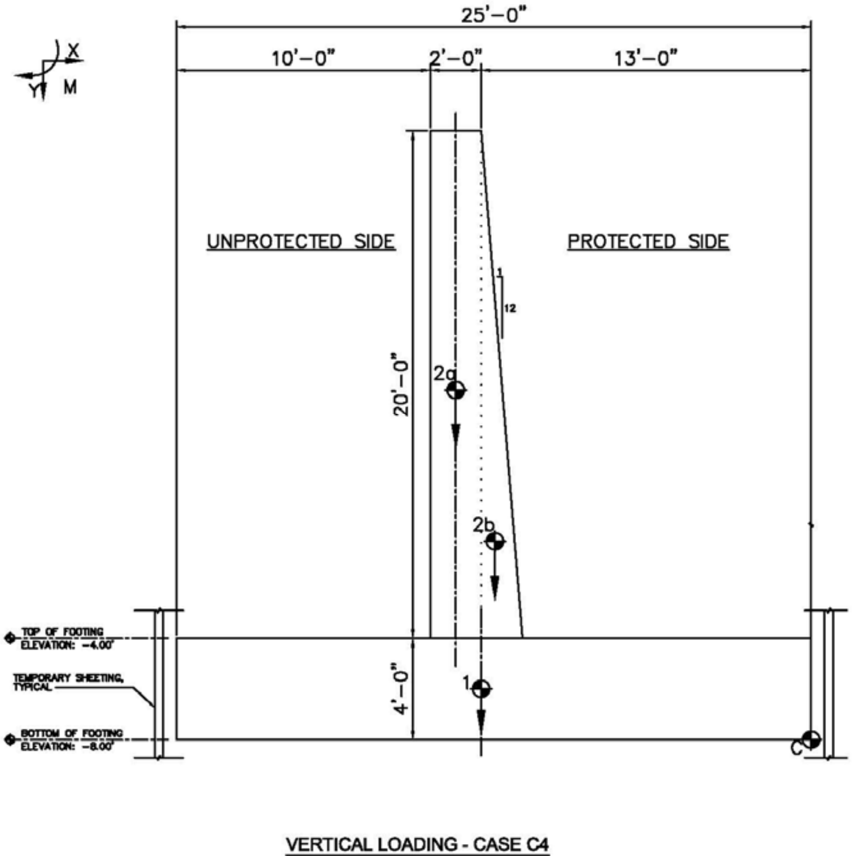


Figure 4b
Case C4 - Construction Loading – Vertical Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	0.00	0.00	0.00
Soil (Unprotected Side)	0.00	0.00	0.00
Ground Water (Protected Side)	0.00	0.00	0.00
Soil (Protected Side)	0.00	0.00	0.00
Wind Load	1200.00	12.00	14400.00
Construction Load	600.00	24.00	14400.00
	1800.00	**-16.00**	**28800.00**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	0.00	**0.00**	**0.00**

Figure 4c
Case C4 – Construction Short-Duration Loading – Design Calculations Summary

Design Parameters	Sand	Clay
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60

Righting Moment (Vertical Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	15000.00	12.50	187500.00
Wall Stem	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	0.00	5.67	0.00
Water Above Heel (Unprotected Side)	0.00	20.00	0.00
Water Above Toe (Protected Side)	0.00	5.67	0.00
Water Above Toe (Protected Side - 12:1 Batter)	0.00	11.53	0.00
* Using East Alignment Subsurface Section Boring C-18 (E-61)	**23500.00**	**-12.88**	**-302611.11**

Resultant Location	
Resultant Horizontal Magnitude [lb/ft of wall]	1800.00
Resultant Vertical Magnitude [lb/ft of wall]	23500.00
Resultant Magnitude [lb/ft of wall]	23568.84
Resultant Orientation [°]	85.62
Overturning Moment [lb-ft/ft of wall]	28800.00
Righting Moment [lb-ft/ft of wall]	-302611.11
Net Moment @ Toe [lb-ft/ft of wall]	-273811.11
Resultant Location (Measured From C) [ft]	-11.65
Eccentricity (Measured From Footing CL) [ft]	0.85

Figure 4d
Case C4 – Construction Short-Duration Loading – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	23500.00	-3262.73	20237.27
Pile Load, Q2 [lb]	23500.00	-1087.58	22412.42
Pile Load, Q3 [lb]	23500.00	1087.58	24587.58
Pile Load, Q4 [lb]	23500.00	3262.73	26762.73

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	23500.00	-3262.73	20237.27
Pile Load, Q2 [lb]	23500.00	-1087.58	22412.42
Pile Load, Q3 [lb]	23671.48	1055.10	24726.58
Pile Load, Q4 [lb]	23671.48	3165.31	26836.79

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 4 - Construction Loading:

Maximum Tensile Load [lb]: N/A

Maximum Compressive Load [lb]: 26836.79

Figure 4e
Case C4 – Construction Short-Duration Loading – Design Calculations Summary – Pile Loads

Case C5 - Wind Loading

- Backfill is in place to the final elevation
- Water is at the usual (non-storm) level
- Wind Load of 50 psf on the protected side of the wall is included
- Uplift is acting

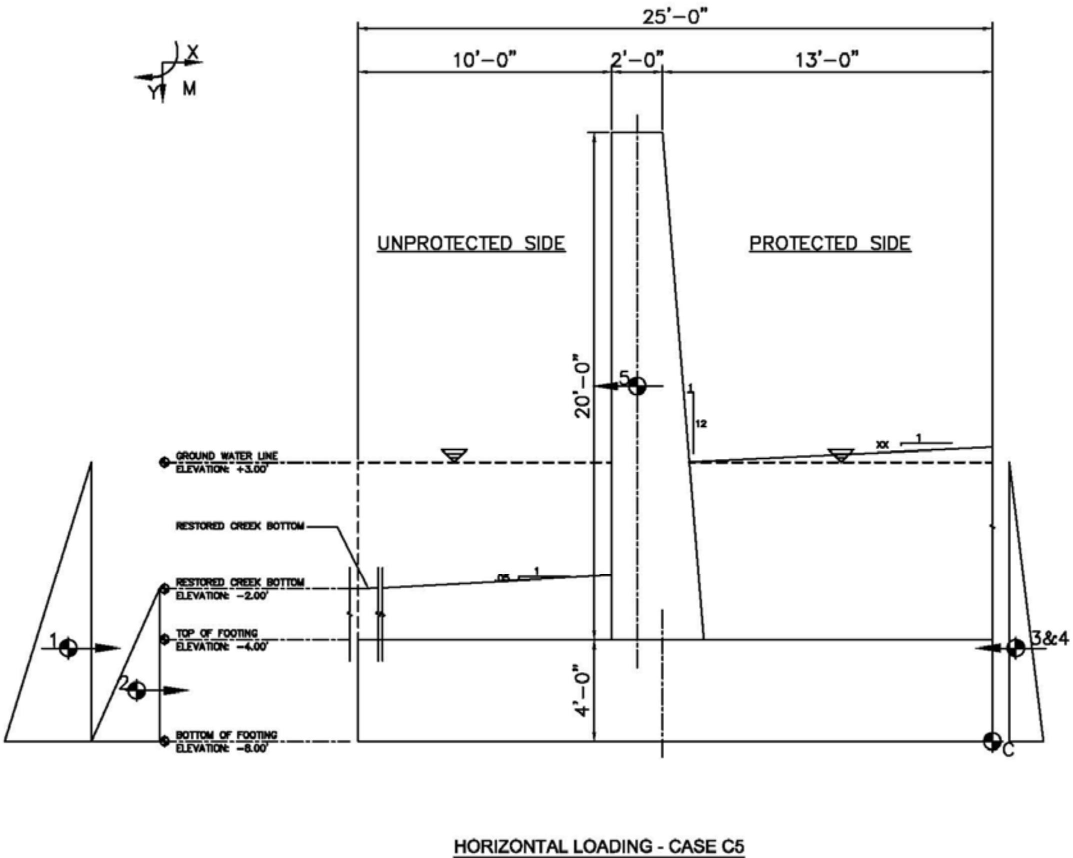


Figure 5a
Case C5 - Wind Loading – Horizontal Loads

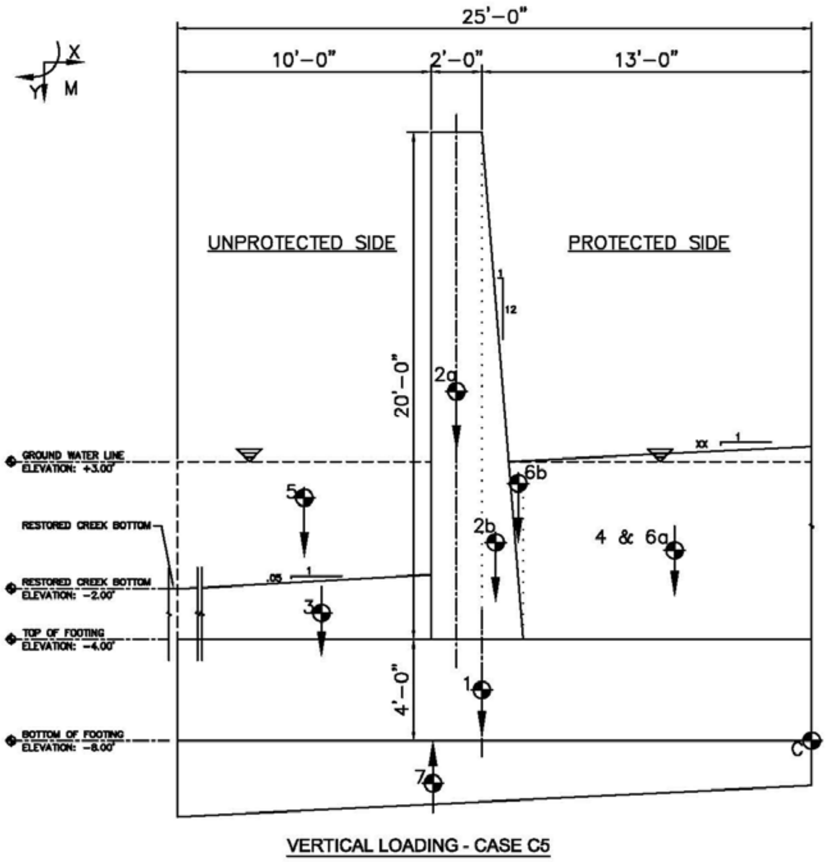


Figure 5b
Case C5 - Wind Loading – Vertical Loads

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, ɣ [lb/ft^3]	125.00	110.00
Water Unit Weight, $ɣ_w$ [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, $ɣ_{eff}$ [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	3775.20	3.67	13842.40
Soil (Unprotected Side)	0.00	1.33	0.00
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1353.51	3.67	-4962.86
Wind Load	650.00	13.50	-8775.00
Wind Load	0.00	0.00	0.00
	-2003.51	**-6.86**	**-13737.86**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-17160.00	**-12.50**	**214500.00**

Figure 5c
Case C5 – Wind Loading – Design Calculations Summary

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44	0.00
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o	0.00	0.47

Overturning Moment (Horizontal Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	3775.20	3.67	13842.40
Soil (Unprotected Side)	0.00	1.33	0.00
Ground Water (Protected Side)	-3775.20	3.67	-13842.40
Soil (Protected Side)	-1353.51	3.67	-4962.86
Wind Load	650.00	13.50	-8775.00
Wind Load	0.00	0.00	0.00
	-2003.51	**-6.86**	**-13737.86**

Overturning Moment (Vertical Loading Forces)			
Item	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift is calculated using the line of creep method outlined in the EM			
	-17160.00	**-12.50**	**214500.00**

Figure 5d

Case C5 – Wind Loading – Design Calculations Summary

Pile Loads (V Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	20157.07	-2798.60	17358.47
Pile Load, Q2 [lb]	20157.07	-932.87	19224.20
Pile Load, Q3 [lb]	20157.07	932.87	21089.93
Pile Load, Q4 [lb]	20157.07	2798.60	22955.66

(-) Indicates Tension

Pile Loads (V Component & H Component)	Axial Load [lb]	My Load [lb]	Total Load [lb]
Pile Load, Q1 [lb]	20157.07	-2798.60	17358.47
Pile Load, Q2 [lb]	20157.07	-932.87	19224.20
Pile Load, Q3 [lb]	18583.38	905.01	19488.40
Pile Load, Q4 [lb]	18583.38	2715.04	21298.42

(-) Indicates Tension

Load Results for a 25 foot wide footing using (2) vertical piles and (2) battered piles at 4:1.

The results above indicate the Maximum Total Load due to the Case 5 - Wind Loading:

Maximum Tensile Load [lb]: N/A

Maximum Compressive Load [lb]: 21298.42

Figure 5e
Case C5 – Wind Loading – Design Calculations Summary – Pile Loads

CONCLUSION

The T-type wall as shown in Figure 57 of the FFR modified with a 1:12 batter on the stem and a 1:4 batter on the piles appears to be capable of withstanding the “worst case” loadings derived from EM 1110-2-2502.

Each load case was analyzed under lateral and gravity loads and a resultant force and moment was included in a table along with the horizontal and vertical components of the resultant force (for example, see the "Resultant Location" table on page 18 of 42). The resultant horizontal and vertical force components and moment are given on a per foot basis. These magnitudes are then multiplied by the tributary width of 4'-0" (per Fig 57) for the pile group loading. The last column on table "Pile Loads (V Component)" (for example, see page 19) displays the total vertical load due to distribution of the gravity load and net moment for each pile. Finally, the "Pile Loads (V Component & H Component)" table (for example, see page 19) takes the data from the table above and includes the effect of horizontal load on the batter piles (Q3 & Q4). The horizontal loads in this analysis were assumed to be equally distributed to the two battered piles within the pile group. The vertical loads in this analysis were assumed to be distributed uniformly to each pile as were the moment effects based on pile location. The total pile axial load is listed in the last column of the "Pile Loads (V Component & H Component)" table. The results are further summarized in the paragraph below the table indicating the maximum axial loads on the piles.

The maximum compressive pile load is 55.2 kips is calculated from Load Case 2. The maximum tensile pile load is 17.7 kips is calculated from Load Case 1 considering scour effects on the unprotected side of the wall.

Overtopping of the wall and scouring effects were not considered in the FFR. Based on the calculated wave heights and the top of wall elevation shown in Figure 57 of the FFR, overtopping is more than likely going to occur. In addition, the occurrence of scouring is within the realm of reason at the base of the wall on both the unprotected and protected sides. Per the USACE guidance in EC 1110-2-6066, the effects of overtopping and scour should be considered and further refined in the design of the wall as there can be serious adverse effects on wall performance during storm events. It is recommended that protection measures against overtopping and scour be included the final wall design.

Also, impact forces were not included in the FFR. These forces should be considered in the final wall design per EC 1110-2-6066.

REFERENCES

The Structural Review and Design is in accordance with the following references:

- American Society of Civil Engineers 7-05, "Minimum Design Loads for Buildings and Other Structures"
- US Army Corps of Engineers EC 1110-2-6066, "Design of I-Walls", 1 April 2011
- US Army Corps of Engineers EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures", 30 June 1992
- US Army Corps of Engineers EM 1110-2-2105, "Design of Hydraulic Steel Structures", 31 March 1993
- US Army Corps of Engineers EM 1110-2-2502, "Retaining and Flood Walls", 29 September 1989
- US Army Corps of Engineers EM 1110-2-2906, "Design of Pile Foundations", 15 January 1991
- US Army Corps of Engineers CETN-III-38, "Breaking Wave Forces On Walls", March 1988
- US Army Corps of Engineers, "Shore Protection Manual - Volumes I and II", 1984
- USGS Website: **http://earthquake.usgs.gov/hazards/designmaps/**
- US Army Corps of Engineers, "Union Beach, New Jersey, Final Feasibility Report - Volumes I, II and III", September 2003

APPENDICES

Appendix 1a
Calculation Worksheet - Case C1 - Surge Stillwater Loading

CASE C1 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Angle of Friction, ϕ [°]	34.00	0.00	
Cohesion, c_{sat}	0.00	800.00	
Soil Unit Weight, γ [lb/ft³]	125.00	110.00	
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40	
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft³]	62.60	47.60	Averaged weight used for soil calculations
Angle of Incline, α [°]	0.00	0.00	
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.44	
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o		0.47	Averaged K factors used for soil calculations

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Item Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	1	62.40	23.00	1435.20	1	16504.80	7.67	126536.80
Soil (Unprotected Side)	2	55.10	6.00	150.56	1	451.67	2.00	903.34
Ground Water (Protected Side)	3	62.40	11.00	686.40	-1	-3775.20	3.67	-13842.40
Soil (Protected Side)	4	55.10	11.00	276.02	-1	-1518.11	3.67	-5566.41
Wave Force 1	5							
Wave Force 2	6							
					$ΣH_O$	11663.16	-9.26	108031.33 $ΣM_{O1}$

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift Pressure 1	7	62.40	23.00	1435.20	25.00			
Uplift Pressure 2	7	62.40	23.00	915.20	25.00			
					$ΣV_O$	-29380.00	-13.42	394333.33 $ΣM_{O2}$

	0
$ΣM_{O1}$	108031.33
$ΣM_{O2}$	394333.33
$ΣM_O$	502364.67

CASE C1 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Concrete Unit Weight, γ [lb/ft³]	150.00	150.00	
Soil Unit Weight, γ [lb/ft³]	125.00	110.00	
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40	
Effective Soil Unit Weight, [lb/ft³]	62.60	47.60	Averaged weight used for soil calculations

Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft²]	Unit Weight, γ [lb/ft³]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	3	10.00	2.00	20.00	55.10	1102.00	20.00	22040.00
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	5	10.00	7.00	70.00	62.40	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64
					$ΣV_R$	38419.07	-12.1373	-466302.53 $ΣM_R$

* Using East Alignment Subsurface Section Boring C-18 (E-61)

Resultant Location			
Resultant Horizontal Magnitude [lb/ft of wall]	11663.16	@	-9.26 From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	9039.07	@	-7.96 From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	14755.81		
Resultant Orientation [°]	37.78		
Overturning Moment [lb-ft/ft of wall]	502364.67		
Righting Moment [lb-ft/ft of wall]	-466302.53		
Net Moment @ Toe [lb-ft/ft of wall]	36062.14		
Resultant Location (Measured From C) [ft]	3.99		

Appendix 1b
Calculation Worksheet - Case C1 - Surge Stillwater Loading (continued)

Appendix 1c
Calculation Worksheet - Case C1 - Surge Stillwater Loading with Scour

CASE C1 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT UNPROTECTED SIDE OF STEM.

CASE C1 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT UNPROTECTED SIDE OF STEM.

Design Parameters	Sand	Clay	
Angle of Friction, ϕ [°]	34.00	0.00	
Cohesion, c_{sat}	0.00	800.00	
Soil Unit Weight, γ [lb/ft³]	125.00	110.00	
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40	
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft³]	62.60	47.60	Averaged weight used for soil calculations
Angle of Incline, α [°]	0.00	0.00	
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.47	Averaged K factors used for soil calculations

Design Parameters	Sand	Clay
Concrete Unit Weight, γ [lb/ft³]	150.00	150.00
Soil Unit Weight, γ [lb/ft³]	125.00	110.00
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft³]	62.60	47.60

Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft²]	Unit Weight, γ [lb/ft³]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	3	10.00	0.00	0.00	55.10	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51
Water Above Heel (Unprotected Side)	5	10.00	7.00	70.00	62.40	4368.00	20.00	87360.00
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64
					$ΣV_R$	37317.07	-11.9051	-444262.53 ($ΣM_R$)

* Using East Alignment Subsurface Section Boring C-18 (E-61)

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Item Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	1	62.40	23.00	1435.20	1	16504.80	7.67	126536.80
Soil (Unprotected Side)	2	55.10	6.00	150.56	0	0.00	3.67	0.00
Ground Water (Protected Side)	3	62.40	11.00	686.40	1	-3775.20	3.67	-13842.40
Soil (Protected Side)	4	55.10	11.00	276.02	1	-1518.11	3.67	-5566.41
Wave Force 1	5							
Wave Force 2	6							
					$ΣH_C$	11211.49	-9.56	107127.99 ($ΣM_{O1}$)

Resultant Location			
Resultant Horizontal Magnitude [lb/ft of wall]	11211.49	@	-9.56 From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	7937.07	@	6.29 From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	13736.61		
Resultant Orientation [°]	35.30		
Overturning Moment [lb-ft/ft of wall]	501461.33		
Righting Moment [lb-ft/ft of wall]	-444262.53		
Net Moment @ Toe [lb-ft/ft of wall]	57198.80		
Resultant Location (Measured From C) [ft]	7.21		

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift Pressure 1	7	62.40	23.00	1435.20	25.00			
Uplift Pressure 2	7	62.40	23.00	915.20	25.00			
					$ΣV_U$	-29380.00	-13.42	394333.33 ($ΣM_{O2}$)

	0
$ΣM_{O1}$	107127.99
$ΣM_{O2}$	394333.33
$ΣM_O$	501461.33

Appendix 1d

Calculation Worksheet - Case C1 - Surge Stillwater Loading with Scour (continued)

Calculation Worksheet - Case C2 - Case C2 – Breaking & Non-breaking Wave Loading

CASE C2 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Angle of Friction, ϕ [°]	34.00	0.00	
Cohesion, c_{sat}	0.00	800.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations
Angle of Incline, α [°]	0.00	0.00	
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44		
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.47	Averaged K factors used for soil calculations

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Water (Unprotected Side)	1	62.40	23.00	1435.20	1	16504.80	7.67	126536.80	
Soil (Unprotected Side)	2	55.10	6.00	150.56	1	451.67	2.00	903.34	
Ground Water (Protected Side)	3	62.40	11.00	686.40	-1	-3775.20	3.67	-13842.40	
Soil (Protected Side)	4	55.10	11.00	276.02	-1	-1518.11	3.67	-5566.41	
Wave Force (100 Year Event)	5				1	6144.45	16.21	99609.15	
Wave Force 2	6				0	0.00	0.00	0.00	
					ΣH_O	**17807.61**	**-11.66**	**207640.48**	ΣM_{O1}

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Uplift Pressure 1	7	62.40	23.00	1435.20	25.00				
Uplift Pressure 2	7	62.40	23.00	915.20	25.00				
					ΣV_O	**-29380.00**	**-13.42**	**394333.33**	ΣM_{O2}
								0	
							ΣM_{O1}	207640.48	
							ΣM_{O2}	394333.33	
							ΣM_O	**601973.82**	

Design Parameters	Sand	Clay							
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00		CASE C2 LOAD CASE					
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00		25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.					
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40							
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations						
Righting Moment (Vertical Forces)									
Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft^2]	Unit Weight, γ [lb/ft^3]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00	
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00	
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11	
Soil Above Heel (Unprotected Side)*	3	10.00	2.00	20.00	55.10	1102.00	20.00	22040.00	
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51	
Water Above Heel (Unprotected Side)	5	10.00	19.00	190.00	62.40	11856.00	20.00	237120.00	
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27	
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64	
* Using East Alignment Subsurface Section Boring C-18 (E-61)					ΣV_R	**45907.07**	**-13.4198**	**-616062.53**	ΣM_R
Resultant Location									
Resultant Horizontal Magnitude [lb/ft of wall]	17807.61	@	-11.66	From Protected Side Toe					
Resultant Vertical Magnitude [lb/ft of wall]	16527.07	@	-13.42	From Protected Side Toe					
Resultant Magnitude [lb/ft of wall]	24295.16								
Resultant Orientation [°]	42.86								
Overturning Moment [lb-ft/ft of wall]	601973.82								
Righting Moment [lb-ft/ft of wall]	-616062.53								
Net Moment @ Toe [lb-ft/ft of wall]	-14088.71								
Resultant Location (Measured From C) [ft]	-0.85								

Appendix 2b

Calculation Worksheet - Case C2 - Case C2 – Breaking & Non-breaking Wave Loading (continued)

CASE C2 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT STEM.

Design Parameters	Sand	Clay
Angle of Friction, ϕ [°]	34.00	0.00
Cohesion, c_{sat}	0.00	800.00
Soil Unit Weight, γ [lb/ft³]	125.00	110.00
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft³]	62.60	47.60
Angle of Incline, α [°]	0.00	0.00
Coefficient of AT-REST Soil Pressure, K_o		0.44
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.47

Averaged weight used for soil calculations
Averaged K factors used for soil calculations

CASE C2 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT STEM.

Design Parameters	Sand	Clay
Concrete Unit Weight, $γ_c$ [lb/ft³]	150.00	150.00
Soil Unit Weight, γ [lb/ft³]	125.00	110.00
Water Unit Weight, $γ_w$ [lb/ft³]	62.40	62.40
Effective Soil Unit Weight, $γ_{eff}$ [lb/ft³]	62.60	47.60

Averaged weight used for soil calculations

Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft²]	Unit Weight, γ [lb/ft³]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11
Soil Above Heel (Unprotected Side)*	3	10.00	0.00	0.00	55.10	0.00	20.00	0.00
Soil Above Toe (Protected Side)*	4	11.33	0.00	0.00	55.10	0.00	5.67	0.00
Water Above Heel (Unprotected Side)	5	10.00	19.00	190.00	62.40	11856.00	20.00	237120.00
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64
					$ΣV_R$	40433.80	$ΣM_R$	569252.02

* Using East Alignment Subsurface Section Boring C-18 (E-61)

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Water (Unprotected Side)	1	62.40	23.00	1435.20	1	16504.80	7.67	126536.80
Soil (Unprotected Side)	2	55.10	0.00	0.00	0	0.00	3.67	0.00
Ground Water (Protected Side)	3	62.40	11.00	686.40	1	3775.20	3.67	-13842.40
Soil (Protected Side)	4	55.10	0.00	0.00	0	0.00	3.67	0.00
Wave Force 1	5				1	6144.45	16.21	99609.15
Wave Force 2	6				0	0.00	0.00	0.00
					-14.0786	$ΣH_O$ 18874.05	-11.25	212303.55 $ΣM_{O1}$

Resultant Location			
Resultant Horizontal Magnitude [lb/ft of wall]	18874.05	@	-11.25 From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	11053.80	@	-15.82 From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	21872.73		
Resultant Orientation [°]	30.36		
Overturning Moment [lb-ft/ft of wall]	606636.88		
Righting Moment [lb-ft/ft of wall]	-569252.02		
Net Moment @ Toe [lb-ft/ft of wall]	37384.87		
Resultant Location (Measured From C) [ft]	3.38		

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft³]	Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]
Uplift Pressure 1	7	62.40	23.00	1435.20	25.00			
Uplift Pressure 2	7	62.40	23.00	915.20	25.00			
					$ΣV_O$	29380.00	13.42	394333.33 $ΣM_{O2}$

	0
$ΣM_{O1}$	212303.55
$ΣM_{O2}$	394333.33
$ΣM_O$	**606636.88**

Appendix 2d

Calculation Worksheet - Case C2 - Case C2 – Breaking & Non-breaking Wave Loading with Scour (continued)

Seismic Design Parameters	
S_{DS}	0.27
I_e	1.50
C_s	0.03
V	4685.21

CASE C3 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Weight [lb/ft]	V/W	Direction		Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Water (Unprotected Side)	1	4368.00	0.12	1		532.68	2.33	1242.91	
Soil (Unprotected Side)	2	1102.00	0.12	1		134.39	0.67	89.59	
Ground Water (Protected Side)	3	4950.40	0.12	-1		-603.70	2.33	-1408.64	
Soil (Protected Side)	4	4371.27	0.12	0		0.00	2.33	0.00	
Wall Footing	5	15000.00	0.12	1		1829.25	2.00	3658.50	
Wall Stem with Batter	6	8500.00	0.12	1		1036.58	14.00	14512.05	
					ΣH_O	**2929.19**	**-6.18**	**18094.42**	**ΣM_{O1}**

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Uplift Pressure 1	7	62.40	11.00	686.40	25.00				
Uplift Pressure 2	7	62.40	11.00	686.40	25.00				
					ΣV_O	**-17160.00**	**-12.50**	**214500.00**	**ΣM_{O2}**
								0	
							ΣM_{O1}	18094.42	
							ΣM_{O2}	214500.00	
							ΣM_O	**232594.42**	

Appendix 3a

Calculation Worksheet - Case C3 - Seismic Loading

Design Parameters	Sand	Clay	
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations

CASE C3 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft^2]	Unit Weight, γ [lb/ft^3]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00	
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00	
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11	
Soil Above Heel (Unprotected Side)*	3	10.00	2.00	20.00	55.10	1102.00	20.00	22040.00	
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51	
Water Above Heel (Unprotected Side)	5	10.00	7.00	70.00	62.40	4368.00	20.00	87360.00	
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27	
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64	
* Using East Alignment Subsurface Section Boring C-18 (E-61)					**ΣV_R**	**38419.07**	**-12.1373**	**-466302.53**	**ΣM_R**

Resultant Location				
Resultant Horizontal Magnitude [lb/ft of wall]	2929.19	@	-6.18	From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	21259.07	@	-11.84	From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	21459.92			
Resultant Orientation [°]	82.15			
Overturning Moment [lb-ft/ft of wall]	232594.42			
Righting Moment [lb-ft/ft of wall]	-466302.53			
Net Moment @ Toe [lb-ft/ft of wall]	-233708.11			
Resultant Location (Measured From C) [ft]	-10.99			

Appendix 3b

Calculation Worksheet - Case C3 - Seismic Loading (continued)

Seismic Design Parameters	
S_{DS}	0.27
I_e	1.50
C_s	0.03
V	4685.21

CASE C3 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT UNPROTECTED SIDE OF STEM.

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Weight [lb/ft]	V/W	Direction		Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Water (Unprotected Side)	1	4368.00	0.12	1.00		532.68	2.33	1242.91	
Soil (Unprotected Side)	2	1102.00	0.12	0.00		0.00	0.00	0.00	
Ground Water (Protected Side)	3	4950.40	0.12	-1.00		-603.70	2.33	-1408.64	
Soil (Protected Side)	4	4371.27	0.12	0.00		0.00	2.33	0.00	
Wall Stem with Batter	5	8500.00	0.12	1.00		1036.58	2.00	2073.15	
Wall Footing	6	15000.00	0.12	1.00		1829.25	14.00	25609.50	
					ΣH_O	**2794.80**	**-9.85**	**27516.93**	**ΣM_{O1}**

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, ɣ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Uplift Pressure 1	7	62.40	11.00	686.40	25.00				
Uplift Pressure 2	7	62.40	11.00	686.40	25.00				
					ΣV_O	**-17160.00**	**-12.50**	**214500.00**	**ΣM_{O2}**
								0	
							ΣM_{O1}	27516.93	
							ΣM_{O2}	214500.00	
							ΣM_O	**242016.93**	

Appendix 3c
Calculation Worksheet - Case C3 - Seismic Loading with Scour

CASE C3 LOAD CASE - SCOUR
25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.
SCOUR IS CONSIDERED TO REMOVE ALL SOILS AT UNPROTECTED SIDE OF STEM.

Design Parameters	Sand	Clay	
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations

Righting Moment (Vertical Forces)									
Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft^2]	Unit Weight, γ [lb/ft^3]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00	
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00	
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11	
Soil Above Heel (Unprotected Side)*	3	10.00	0.00	0.00	55.10	0.00	20.00	0.00	
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51	
Water Above Heel (Unprotected Side)	5	10.00	7.00	70.00	62.40	4368.00	20.00	87360.00	
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27	
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64	
* Using East Alignment Subsurface Section Boring C-18 (E-61)					ΣV_R	**37317.07**	**-11.9051**	**-444262.53**	ΣM_R

Resultant Location				
Resultant Horizontal Magnitude [lb/ft of wall]	2794.80	@	-9.85	From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	20157.07	@	-11.40	From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	20349.90			
Resultant Orientation [°]	82.11			
Overturning Moment [lb-ft/ft of wall]	242016.93			
Righting Moment [lb-ft/ft of wall]	-444262.53			
Net Moment @ Toe [lb-ft/ft of wall]	-202245.60			
Resultant Location (Measured From C) [ft]	-10.03			

Appendix 3d
Calculation Worksheet - Case C3 - Seismic Loading with Scour (continued)

CASE C4 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Angle of Friction, ϕ [°]	34.00	0.00	
Cohesion, c_{sat}	0.00	800.00	
Soil Unit Weight, ɣ [lb/ft^3]	125.00	110.00	
Water Unit Weight, $ɣ_w$ [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, $ɣ_{eff}$ [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations
Angle of Incline, α [°]	0.00	0.00	
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44		
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.47	Averaged K factors used for soil calculations

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, ɣ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Water (Unprotected Side)	1	62.40	0.00	0.00	0	0.00	0.00	0.00	
Soil (Unprotected Side)	2	55.10	0.00	0.00	0	0.00	0.00	0.00	
Ground Water (Protected Side)	3	62.40	0.00	0.00	0	0.00	0.00	0.00	
Soil (Protected Side)	4	55.10	0.00	0.00	0	0.00	0.00	0.00	
Wind Load	5			50.00	1	1200.00	12.00	14400.00	
Construction Load	6			25.00	1	600.00	24.00	14400.00	
					ΣH_O	**1800.00**	**-16.00**	**28800.00**	**ΣM_{O1}**

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, ɣ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Uplift Pressure 1	7	62.40	0.00	0.00	25.00				
Uplift Pressure 2	7	62.40	0.00	0.00	25.00				
					ΣV_O	**0.00**	**0.00**	**0.00**	**ΣM_{O2}**
								0	
							ΣM_{O1}	28800.00	
							ΣM_{O2}	0.00	
							ΣM_O	**28800.00**	

Appendix 4a

Calculation Worksheet - Case C4 - Construction Loading

CASE C4 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations

Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft^2]	Unit Weight, γ [lb/ft^3]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00	
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00	
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11	
Soil Above Heel (Unprotected Side)*	3	10.00	0.00	0.00	55.10	0.00	20.00	0.00	
Soil Above Toe (Protected Side)*	4	11.33	0.00	0.00	55.10	0.00	5.67	0.00	
Water Above Heel (Unprotected Side)	5	10.00	0.00	0.00	62.40	0.00	20.00	0.00	
Water Above Toe (Protected Side)	6a	11.33	0.00	0.00	62.40	0.00	5.67	0.00	
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	0.00	0.00	62.40	0.00	11.53	0.00	
* Using East Alignment Subsurface Section Boring C-18 (E-61)					ΣV_R	**23500.00**	**-12.8771**	**-302611.11**	ΣM_R

Resultant Location				
Resultant Horizontal Magnitude [lb/ft of wall]	1800.00	@	-16.00	From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	23500.00	@	-12.88	From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	23568.84			
Resultant Orientation [°]	85.62			
Overturning Moment [lb-ft/ft of wall]	28800.00			
Righting Moment [lb-ft/ft of wall]	-302611.11			
Net Moment @ Toe [lb-ft/ft of wall]	-273811.11			
Resultant Location (Measured From C) [ft]	-11.65			

Appendix 4b

Calculation Worksheet - Case C4 - Construction Loading (continued)

Appendix 5a
Calculation Worksheet - Case C5 - Wind Loading

CASE C5 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Angle of Friction, ϕ [°]	34.00	0.00	
Cohesion, c_{sat}	0.00	800.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ_{eff} [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations
Angle of Incline, α [°]	0.00	0.00	
Cohesionless Soil Coefficient of AT-REST Soil Pressure, K_o	0.44		
Cohesive Soil Coefficient of AT-REST Soil Pressure, K_o		0.47	Averaged K factors used for soil calculations

Overturning Moment (Horizontal Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Direction	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Water (Unprotected Side)	1	62.40	11.00	686.40	1	3775.20	3.67	13842.40	
Soil (Unprotected Side)	2	47.60	4.00	89.49	0	0.00	1.33	0.00	
Ground Water (Protected Side)	3	62.40	11.00	686.40	-1	-3775.20	3.67	-13842.40	
Soil (Protected Side)	4	47.60	11.00	246.09	-1	-1353.51	3.67	-4962.86	
Wind Load	5			50.00	-1	650.00	13.50	-8775.00	
Wind Load	6								
					ΣH_O	**-2003.51**	**-6.86**	**-13737.86**	**ΣM_{O1}**

Overturning Moment (Vertical Loading Forces)

Item	Item Number	Unit Weight, γ [lb/ft^3]	Item Depth [ft]	Pressure [psf]	Item Width [ft]	Force [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Uplift Pressure 1	7	62.40	11.00	686.40	25.00				
Uplift Pressure 2	7	62.40	11.00	686.40	25.00				
					ΣV_O	**-17160.00**	**-12.50**	**214500.00**	**ΣM_{O2}**
								0	
							ΣM_{O1}	-13737.86	
							ΣM_{O2}	214500.00	
							ΣM_O	**200762.14**	

CASE C5 LOAD CASE

25 FOOT FOOTING WIDTH ANALYZED USING THIS LOAD CASE.

Design Parameters	Sand	Clay	
Concrete Unit Weight, γ_c [lb/ft^3]	150.00	150.00	
Soil Unit Weight, γ [lb/ft^3]	125.00	110.00	
Water Unit Weight, γ_w [lb/ft^3]	62.40	62.40	
Effective Soil Unit Weight, γ' [lb/ft^3]	62.60	47.60	Averaged weight used for soil calculations

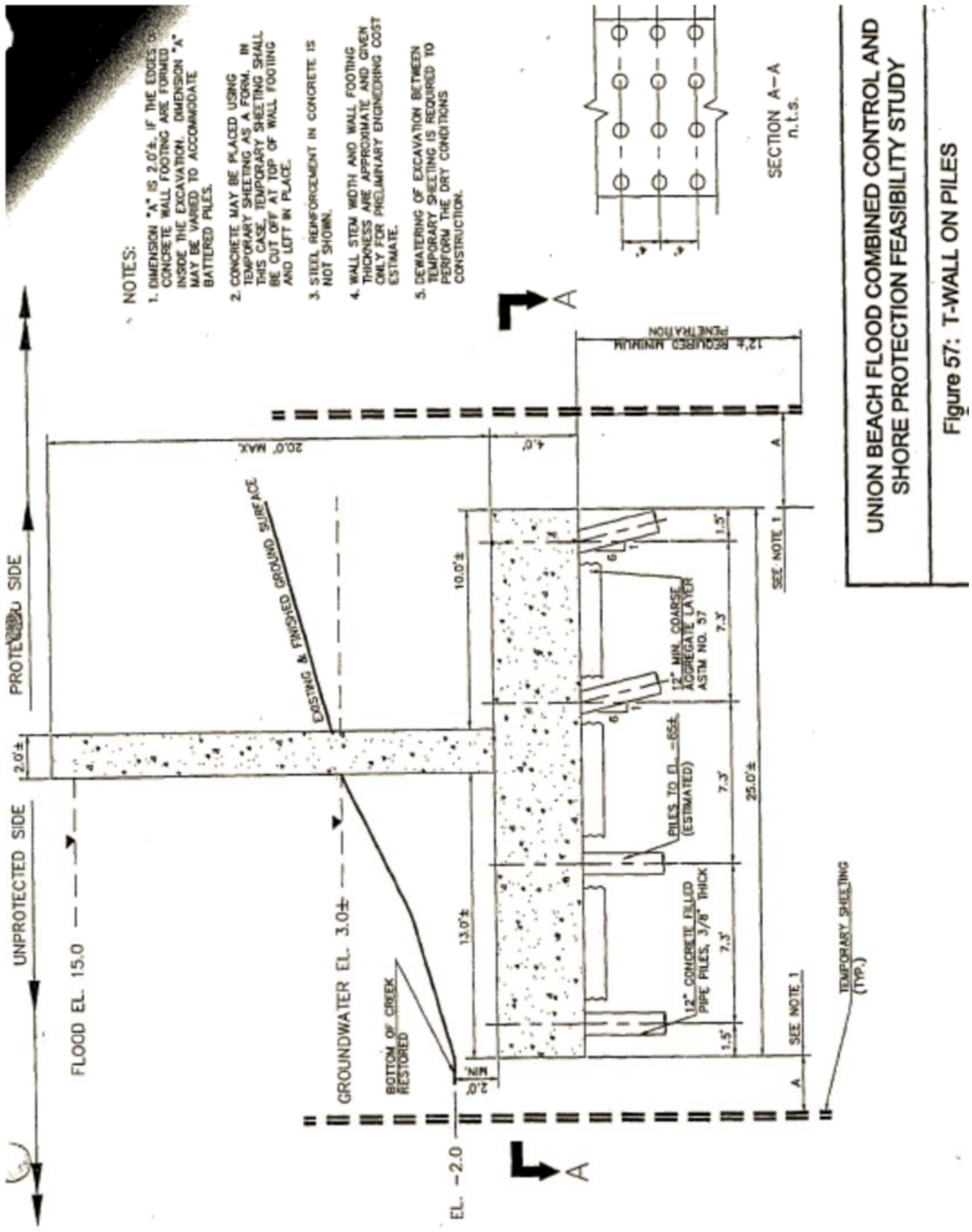
Righting Moment (Vertical Forces)

Item	Item Number	Item Width [ft]	Item Depth [ft]	Area [ft^2]	Unit Weight, γ [lb/ft^3]	Weight [lb/ft]	Moment Arm [ft]	Moment [lb-ft/ft]	
Wall Footing	1	25.00	4.00	100.00	150.00	15000.00	12.50	187500.00	
Wall Stem	2a	2.00	20.00	40.00	150.00	6000.00	14.00	84000.00	
Wall Stem (12:1 Batter)	2b	1.67	20.00	16.67	150.00	2500.00	12.44	31111.11	
Soil Above Heel (Unprotected Side)*	3	10.00	0.00	0.00	55.10	0.00	20.00	0.00	
Soil Above Toe (Protected Side)*	4	11.33	7.00	79.33	55.10	4371.27	5.67	24770.51	
Water Above Heel (Unprotected Side)	5	10.00	7.00	70.00	62.40	4368.00	20.00	87360.00	
Water Above Toe (Protected Side)	6a	11.33	7.00	79.33	62.40	4950.40	5.67	28052.27	
Water Above Toe (Protected Side - 12:1 Batter)	6b	0.58	7.00	2.04	62.40	127.40	11.53	1468.64	
* Using East Alignment Subsurface Section Boring C-18 (E-61)					ΣV_R	**37317.07**	**-11.9051**	**-444262.53**	ΣM_R

Resultant Location

Resultant Horizontal Magnitude [lb/ft of wall]	-2003.51	@	-6.86	From Protected Side Toe
Resultant Vertical Magnitude [lb/ft of wall]	20157.07	@	-11.40	From Protected Side Toe
Resultant Magnitude [lb/ft of wall]	20256.39			
Resultant Orientation [°]	-84.32			
Overturning Moment [lb-ft/ft of wall]	200762.14			
Righting Moment [lb-ft/ft of wall]	-444262.53			
Net Moment @ Toe [lb-ft/ft of wall]	-243500.38			
Resultant Location (Measured From C) [ft]	-10.72			

Appendix 5b
Calculation Worksheet - Case C5 - Wind Loading (continued)



Appendix 6
Union Beach, New Jersey, Final Feasibility Report - Volumes I, II and III", September 2003 Figure 57

HURRICANE SANDY LIMITED REEVALUATION REPORT
UNION BEACH, NEW JERSEY

DRAFT ENGINEERING APPENDIX

SUB APPENDIX B-1

FLOODWALL PILE ANALYSES
WEST WALLS

March 2014Revised March 2015

Preliminary Flood Wall Pile Analysis – West Alignment
Union Beach, New Jersey
(revised March 2015)

1. GENERAL: Preliminary Axial compressive, uplift and lateral load capacities are provided below to the Structural Engineer to determine the preliminary dimension of the pile caps for the pile supported T-walls along the West Alignment. Pile capacities for the East Flood Wall Alignment were previously submitted under a different document.

2. SOIL PROFILE: Drill holes C-1, 2, 4, 8, 9, and 10 obtained from the 2003 Feasibility Report dated September 2003 were utilized to develop a "Generalized Soil Profile" which was utilized for this preliminary pile analysis. Based upon observations and interpretations of the limited laboratory test results and borings logs, it is anticipated that the soils will behave as a sand (drained condition).

   With the exception of DH C-2, N-values ranged from 50 blows per foot to over 100 blows per foot at a depth of approximately 35 feet below grade. Therefore, a maximum pile length of 30 feet was analyzed as it is anticipated that a pile will not be drivable beyond this depth.

3. MINIMUM PILE SPACING: Piles shall be spaced no closer than a minimum center- to- center spacing of 3 diameters. The lateral load analysis utilized P multipliers (Pm) to account for group effects.

4. AXIAL PILE ANALYSIS: APILE plus 5.0 software developed by ENSOFT was utilized to compute the axial pile capacities. The software evaluated the axial capacity for four different methods (FHWA, USACE, Lambda and API); the capacities provided in the following table are computed from the USACE method. Estimated axial Pile loadings as provided by the Structural Engineer range from 24 kips to 234 kips in axial compression and 15 kips in axial tension. Five different pile types,(1) 12-inch pipe pile with 0.312-inch wall thickness driven open ended, (2) HP10 X 42, (3) HP10 X 57, (4) HP12 X 53, and (5) HP16 X 88, were analyzed and their allowable axial capacities provided for a pile length of 30 feet. The following assumptions were utilized in the analysis:

   a. Based upon the 2003 Feasibility report, it was assumed that the bottom of the pile cap is 6 feet below grade.
   b. Existing and finished grades are unknown and therefore down drag was not considered in this preliminary analysis. This shall be revisited during the final design.
   c. Since a static load test will be required during construction, a factor of safety of 2 was implemented for the allowable bearing capacity.

Preliminary Allowable Axial Pile Capacities (with factor of safety of 2)

	12" Diameter Pipe w/0.312" wall thickness (kips)	HP10 X 42 (kips)	HP10 X 57 (kips)	HP12 X 53 (kips)	HP16 X 88
Q_{all} for 30' length	55	75	80	120	240
T_{all} for 30' length	15	20	20	25	32

Q_{all} = Allowable axial capacity in compression; T_{all} = Allowable axial capacity in Tension (Uplift)

5. LATERAL PILE ANALYSIS: LPILE plus 2012.6 version software developed by ENSOFT was utilized for the lateral pile analysis. Four different pile types,(1) 12-inch pipe pile with a 0.312-inch wall thickness driven open ended, (2) HP10 X 42, (3) HP10 X 57, (4) HP12 X 53, and (5) HP16 X 88 for a length of 30 feet, were analyzed and the results tabulated below for each section. The following assumptions were made in the analysis:

   a. Since the head restraint was unknown, maximum moments and lateral deflections for both free head and fixed head are provided. Since it is anticipated that a "partially fixed" restraint will be utilized, a maximum "free head" deflection of 0.75 inches was assumed as the governing deflection.
   b. Axial loading was unknown and therefore not implemented in the lateral analysis.
   c. Top of pile was conservatively assumed at 3 feet below grade in lieu of 6 feet as indicated in the axial capacity analysis.
   d. A cyclic loading of 500 cycles was utilized. This is an estimate and will need to be further investigated during the final design.
   e. As indicated above, piles shall be spaced a minimum center- to- center of 3 diameters. To account for group effects, P-multiplier (Pm) values of 0.8 (for lead row), 0.4 (for second row) and 0.3 (for third and fourth rows) were utilized to determine the average lateral loading for a free head deflection of 0.75 inches and maximum moments. ***The lateral pile loading (service load) provided in the row entitled "Group Effect" illustrated in the below tables shall be used for the preliminary design to take into account the group effects***.

Union Beach – East Wall - Preliminary Lateral Pile Capacities

1. **<u>12-inch Diameter Pipe Pile with 0.312" wall thickness</u>**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	16	640 @ 6.0'	0.77	-660 @ 0	0.23
0.4	10	480 @ 7.0	0.72	-480 @ 0	0.21
0.3	8.5	430 @ 7.5	0.73	-430 @ 0	0.22
GROUP EFFECTS	***11.5***	***520***	***0.75***	***-520***	***0.22***

2. **<u>HP10 X 42</u>**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	16	650 @ 6.0'	0.73	-650 @ 0	0.21
0.4	10.5	500 @ 7.0'	0.73	-500 @ 0	0.21
0.3	9	460 @ 7.0'	0.76	-460 @ 0	0.22
GROUP EFFECTS	***12.0***	***540***	***0.75***	***-540***	***0.21***

3. **<u>HP10 X 57</u>**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	18.5	820 @ 6.0'	0.75	-820 @ 0	0.22
0.4	12	620 @ 7.0'	0.74	-620 @ 0	0.22
0.3	10	540 @ 7.0'	0.73	-540 @ 0	0.21
GROUP EFFECTS	***13.5***	***660***	***0.75***	***-660***	***0.21***

4. **HP12 X 53**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	20	940 @ 7.0'	0.73	-940 @ 0	0.21
0.4	13.5	740 @ 7.0'	0.76	-740 @ 0	0.22
0.3	11	640 @ 8.0'	0.72	-640 @ 0	0.21
GROUP EFFECTS	***16.5***	***775***	***0.75***	***-775***	***0.21***

5. **HP16 X 88**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8 (*battered)	21.5	1440 @ 9.0'	0.74	-1420 @ 0	0.21
0.6 (*battered)	18.0	1260 @ 10.0'	0.74	-1260 @ 0	0.21
0.4	19.5	1320 @ 9.0'	0.75	-1320 @ 0	0.22
0.3	16.5	1180 @ 10.0'	0.75	-1600 @ 0	0.22
GROUP EFFECTS	***20.0***	***1300***	***0.75***	***-1400***	***0.22***

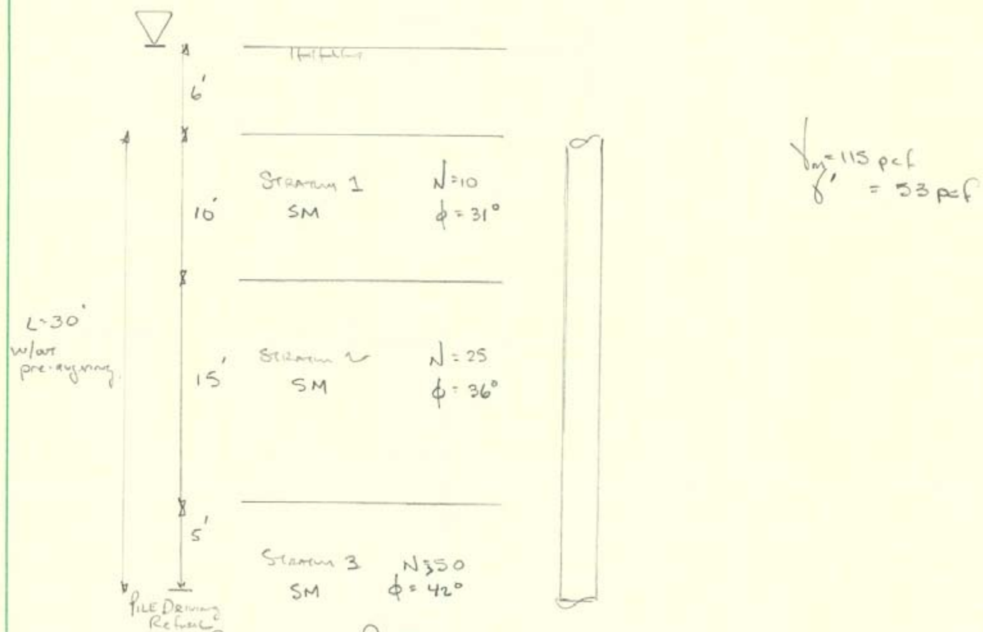
*Last two rows (piles 3 and 4) were evaluated with a 4 to 1 pile batter per "25-Foot Four Pile Arrangement" drawing provided by Structural Engineer.

6. PRELIMINARY PILE RECOMMENDATIONS: Per Sub Appendix A, PRELIMINARY FLOOD WALL DESIGN, the maximum axial compressive loading per pile is 55.2 kips and the maximum axial tensile loading per pile is 17.7 kips. Per conversations with the Structural Engineer, the maximum lateral pile load is 35.6 kips per pile when distributed to only the two battered piles and 17.8 kips when distributed uniformly to all four piles. Based upon these maximum axial pile loadings and for a lateral loading of 17.8 kips per pile uniformly distributed to all four piles, an HP16 X 88 driven to a minimum elevation of -38 feet (30' pile length below the bottom of pile cap) is recommended.

7. POINT OF CONTACT: Contact David Tucker, P.E. at 4140-962-6823 or david.l.tucker@usace.army.mil for any further analysis, questions or comments.

Computed By: DCT | | Jan 14

Wall Along Western Alignment



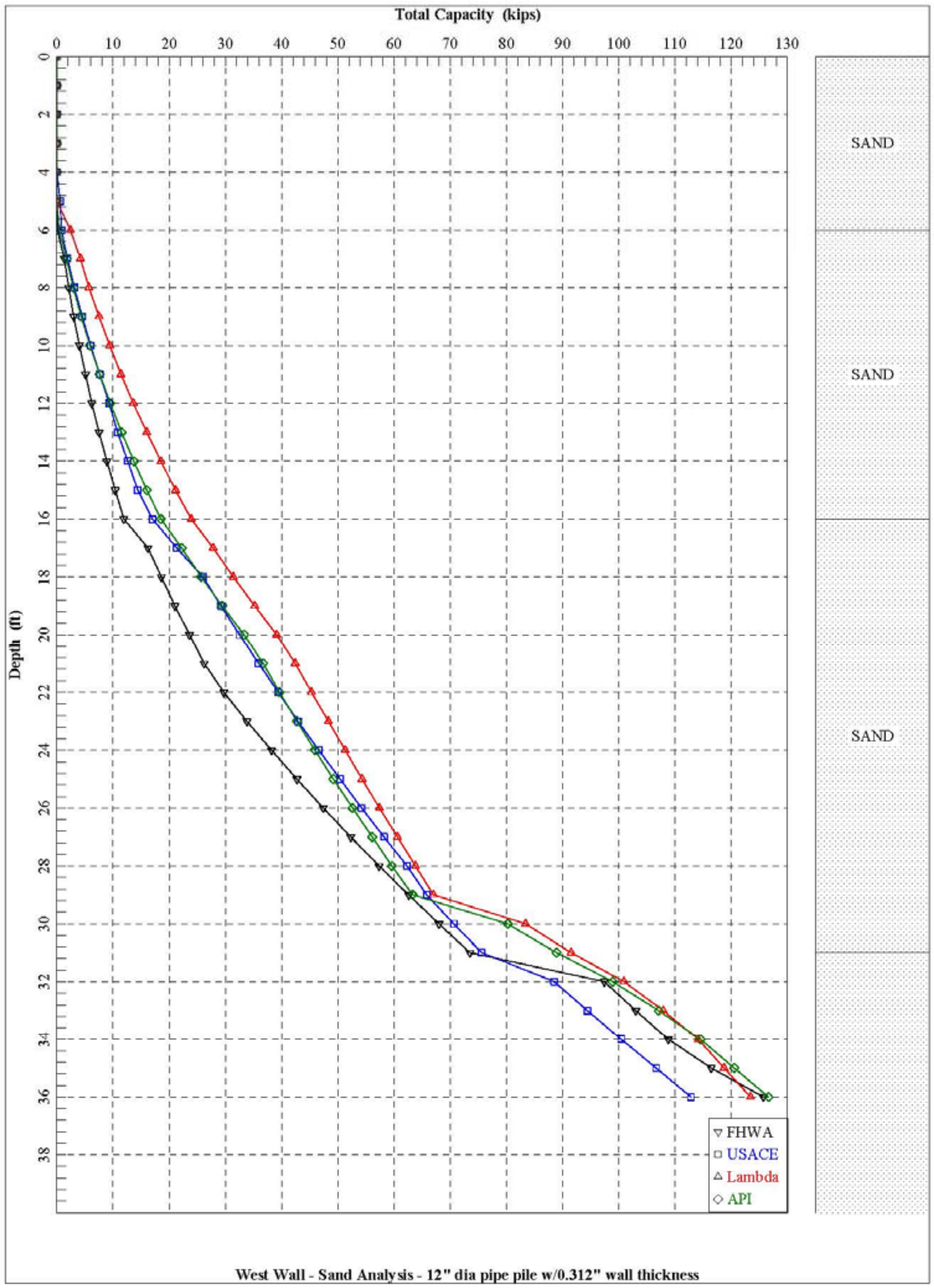
Generalized Profile

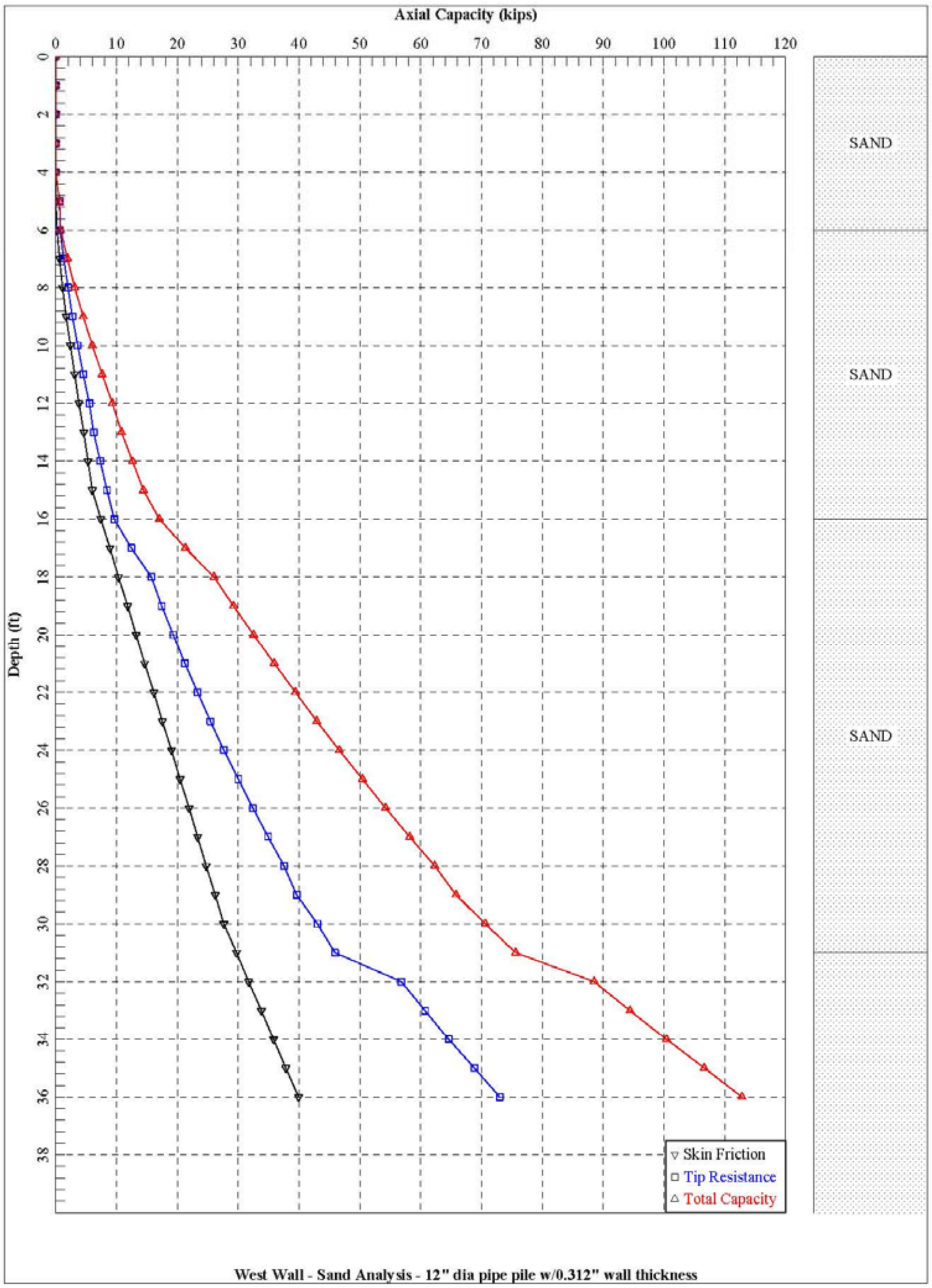
* NSF not Accounted for Since existing & proposed grades Unknown

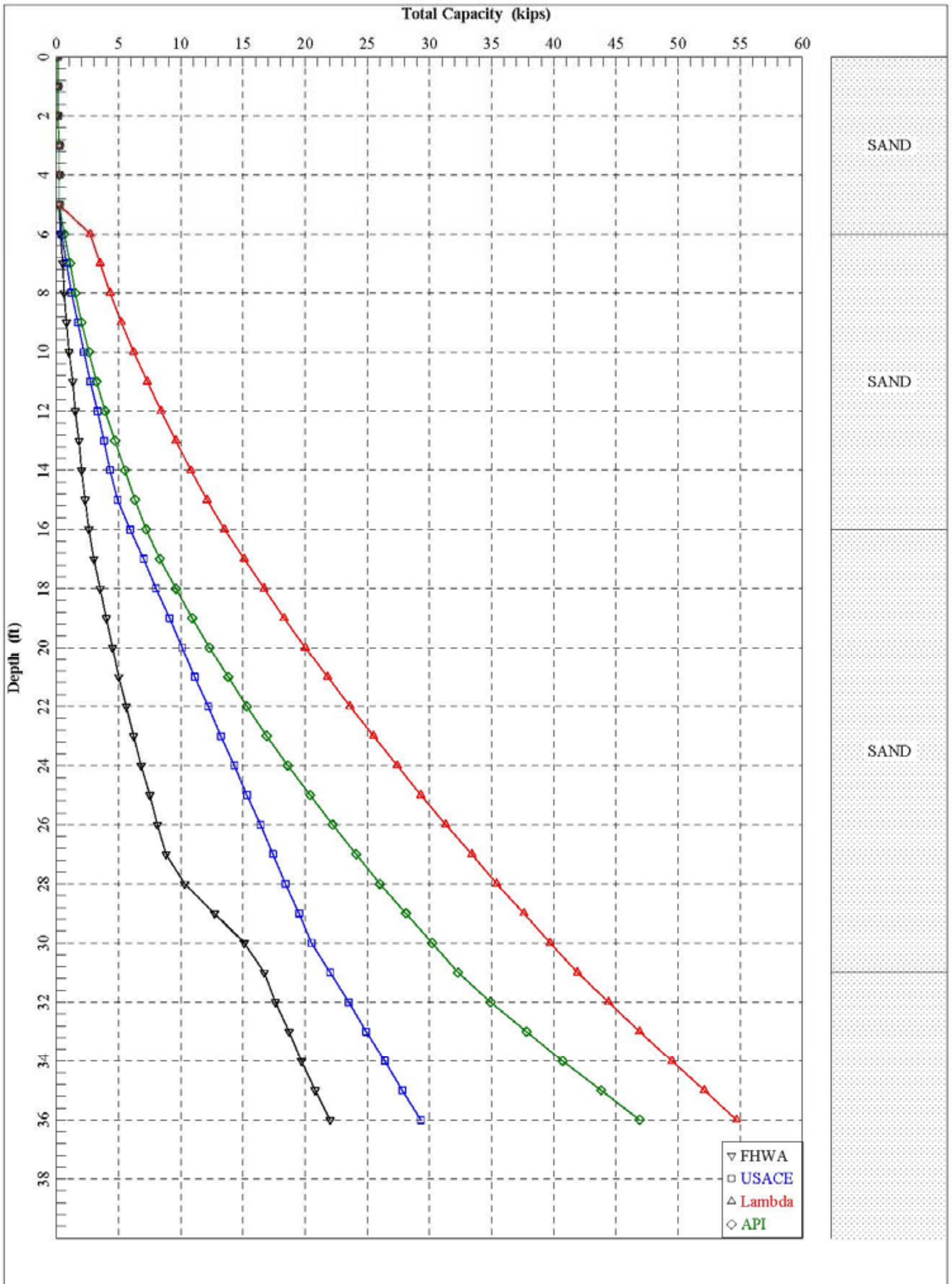
** Soil Profiles Along Western Alignment Vary. The Above Generalized Profile is preliminary & will be better defined following more Specific Drilling.

*** Assume Pile Cap @ 6' below existing Drill hole elevations.

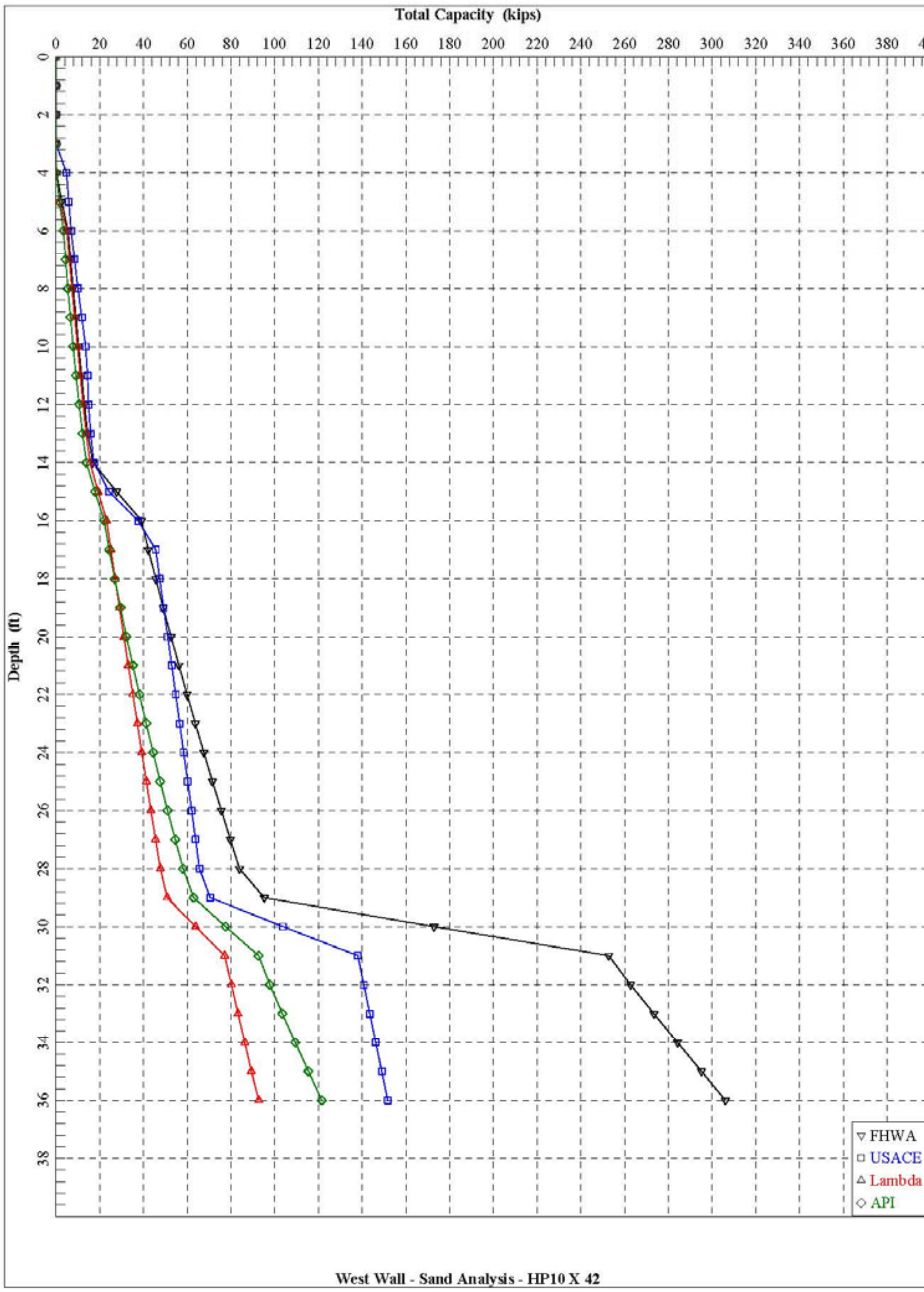
TOPS 35500

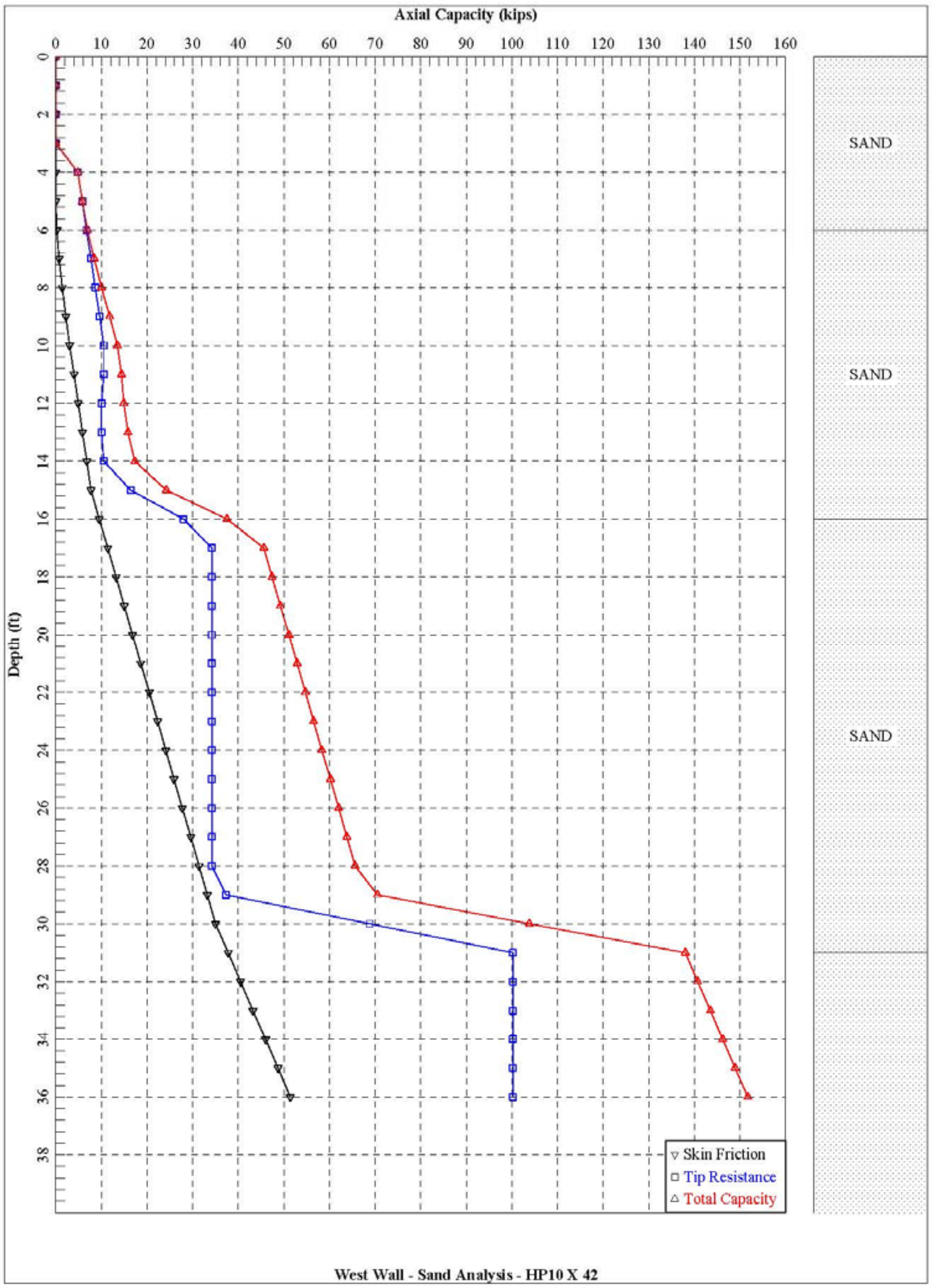

Total Capacity (kips)
0 10 20 30 40 50 60 70 80 90 100 110 120 130
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
SAND
SAND
SAND
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - 12" dia pipe pile w/0.312" wall thickness

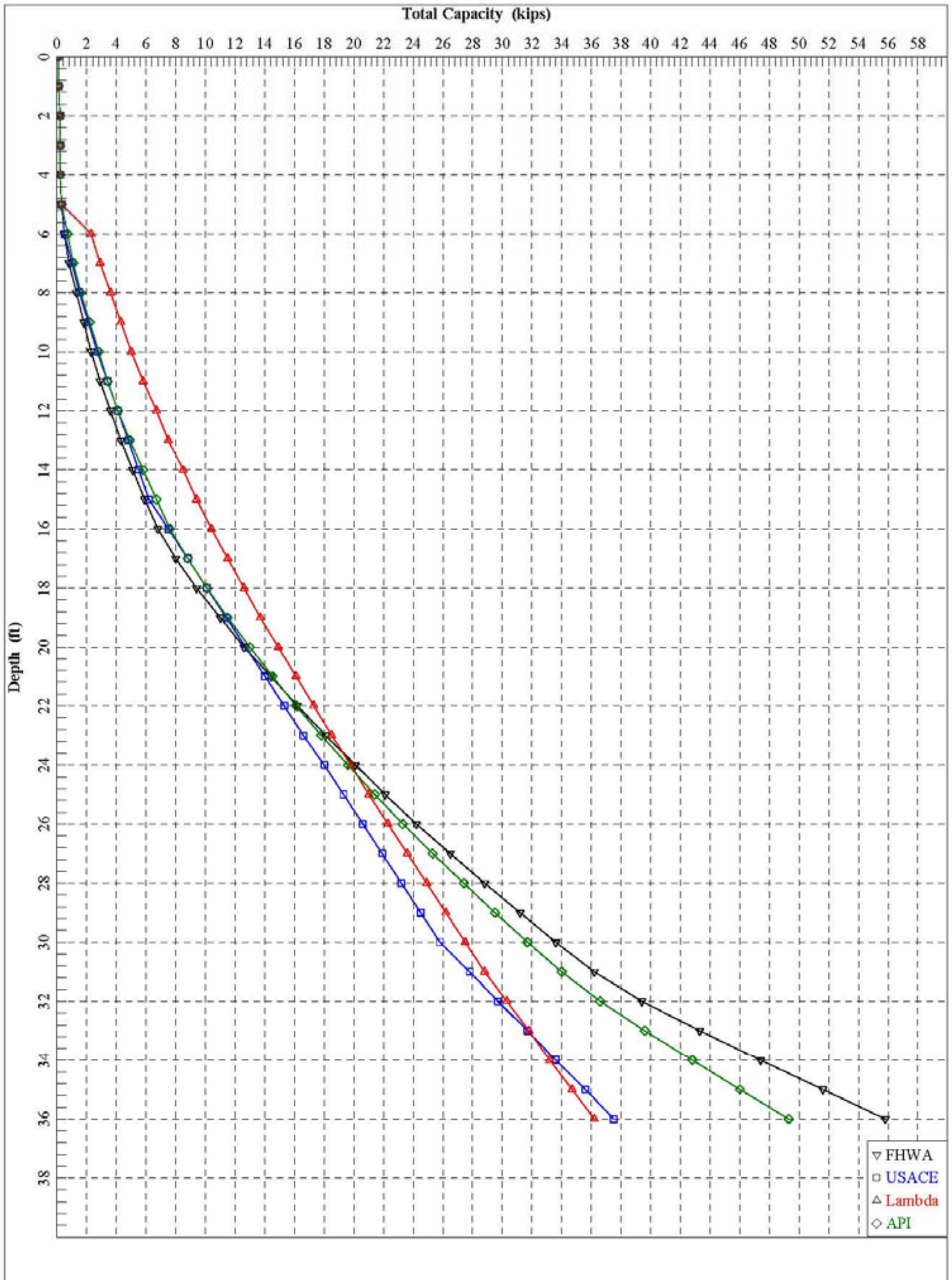

Axial Capacity (kips)
0
10
20
30
40
50
60
70
80
90
100
110
120
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
West Wall - Sand Analysis - 12" dia pipe pile w/0.312" wall thickness



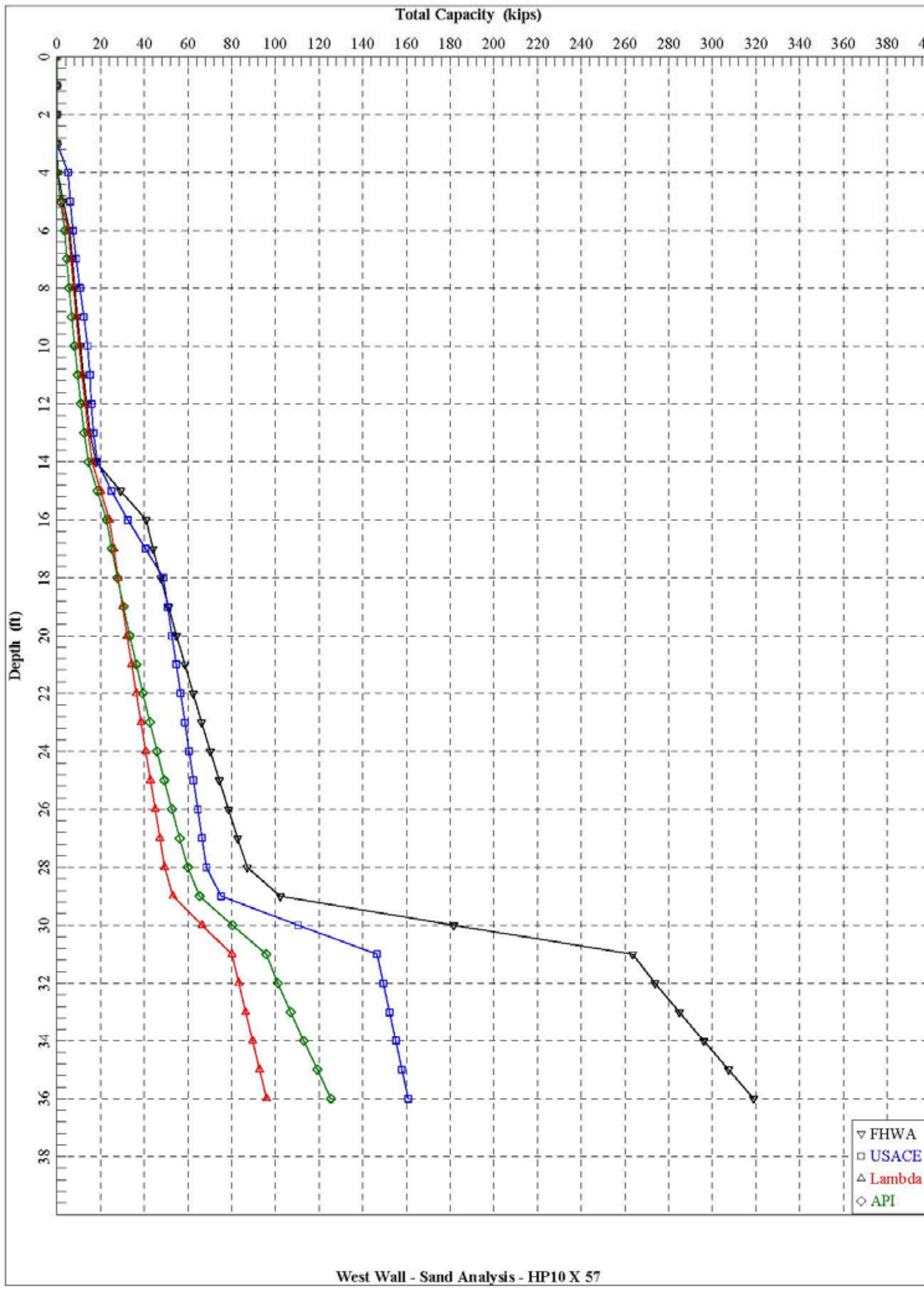
West Wall - Sand Analysis - 12" dia pipe pile w/0.312" wall thickness - Uplift


Total Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - HP10 X 42


Axial Capacity (kips)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
West Wall - Sand Analysis - HP10 X 42



West Wall - Sand Analysis - HP10 X 42 - Uplift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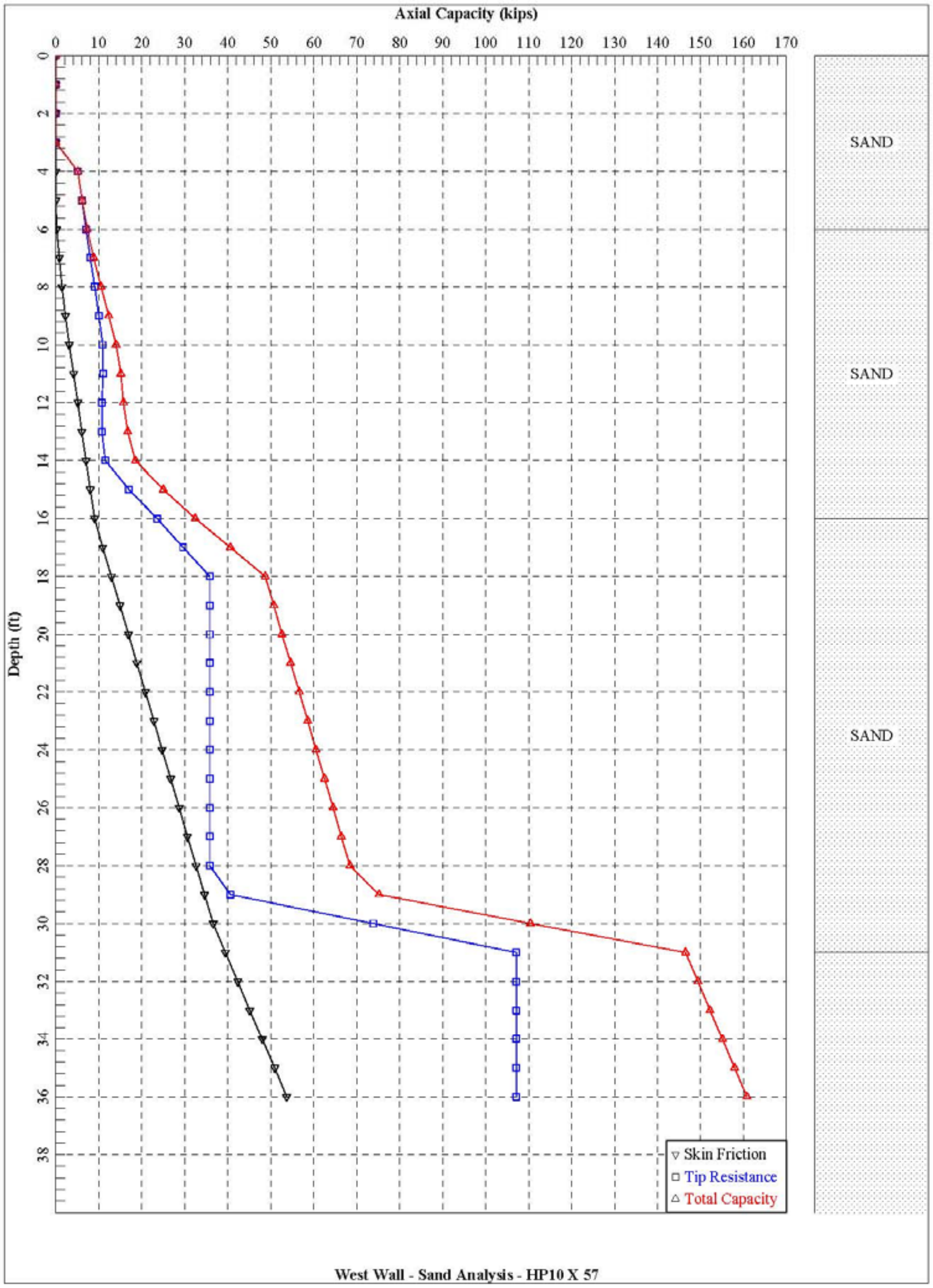
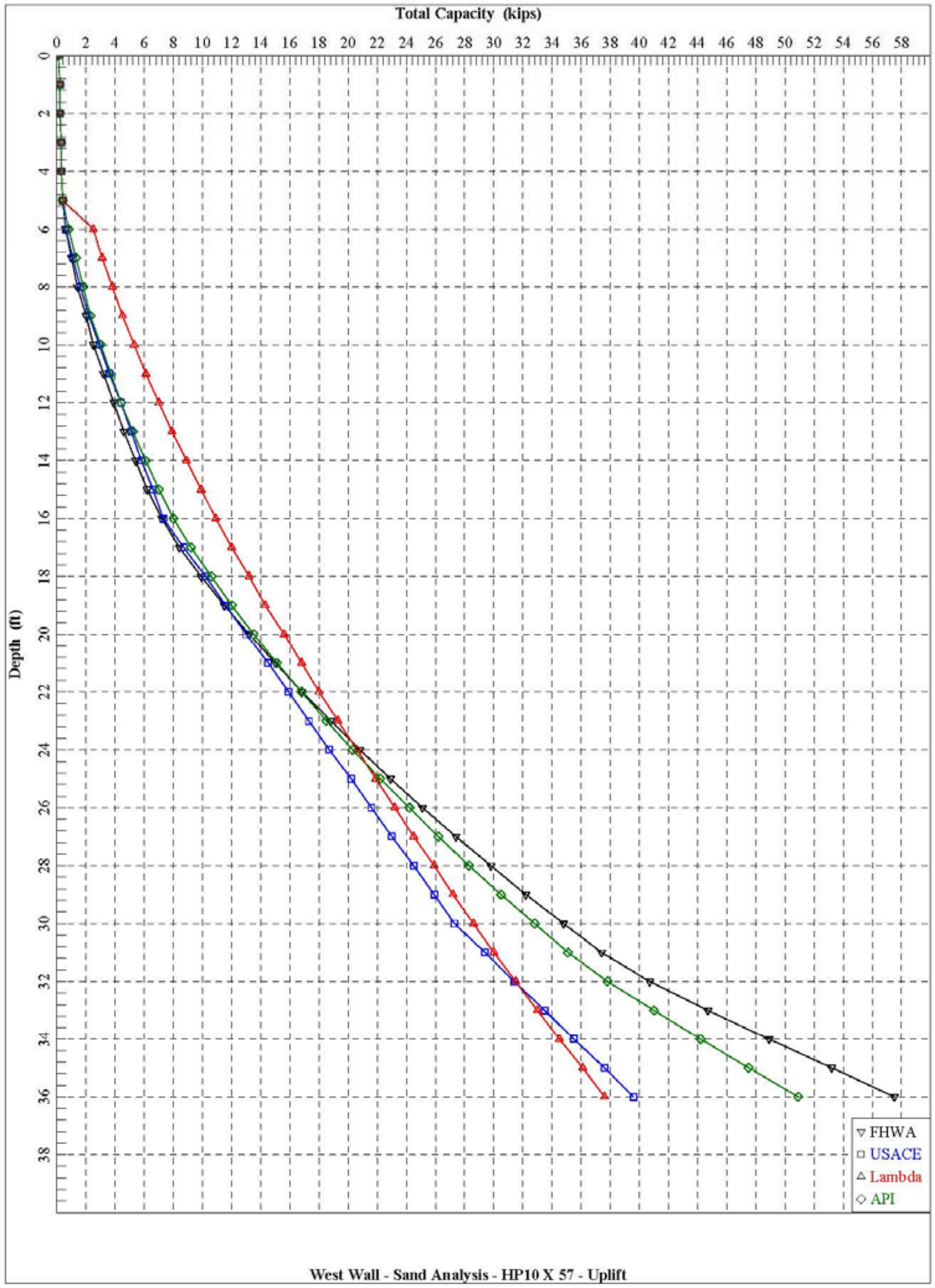
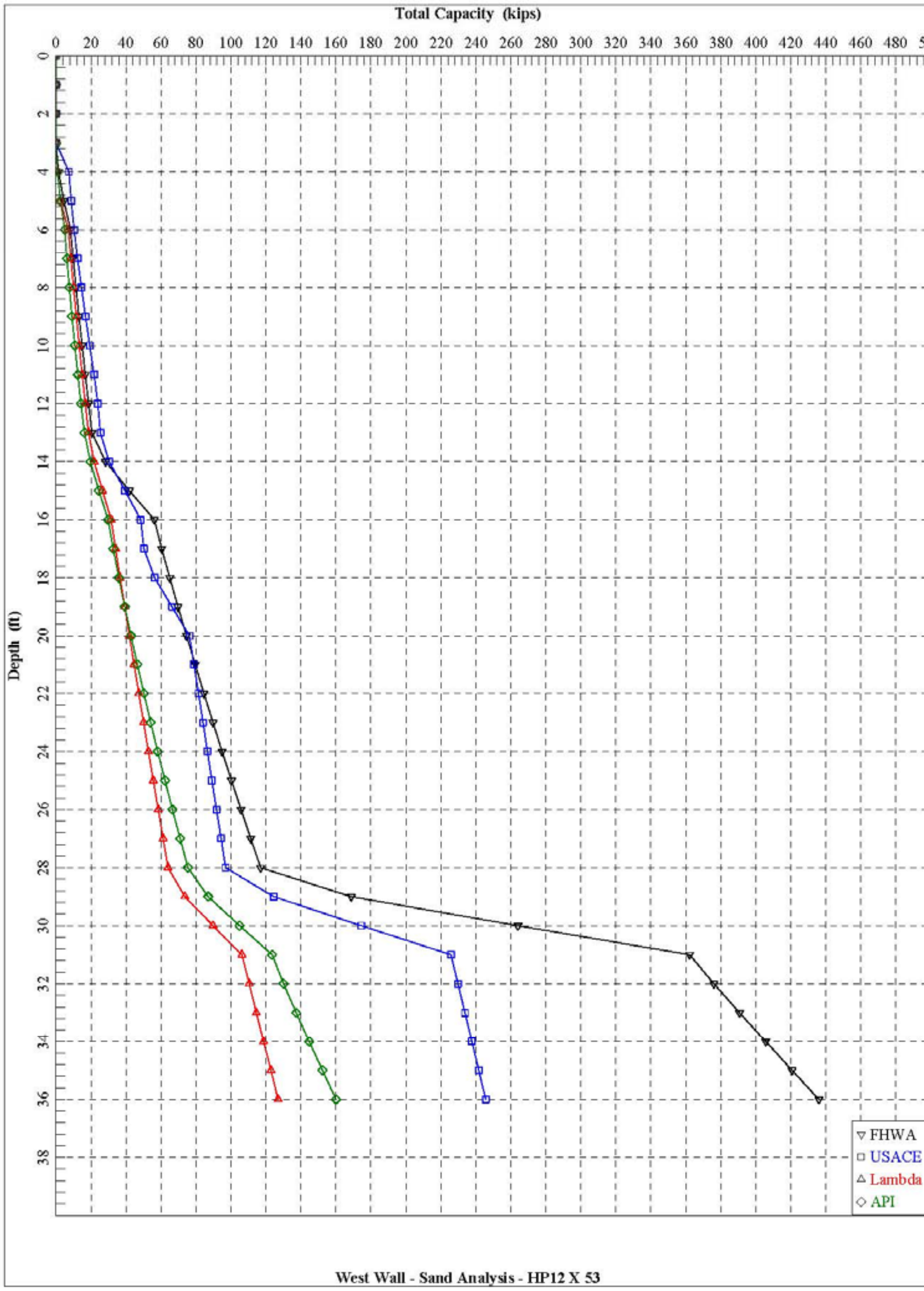
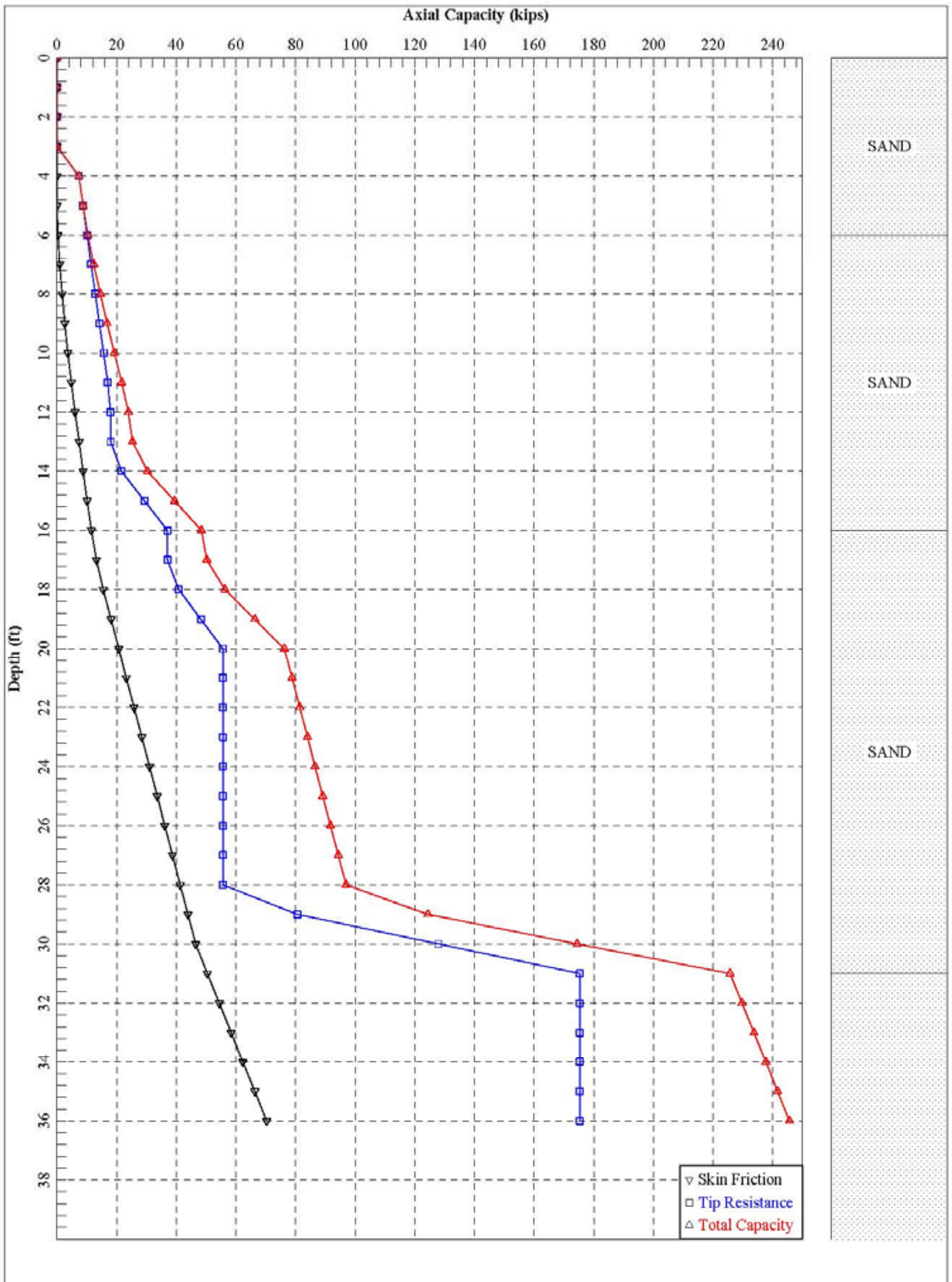
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - HP10 X 57

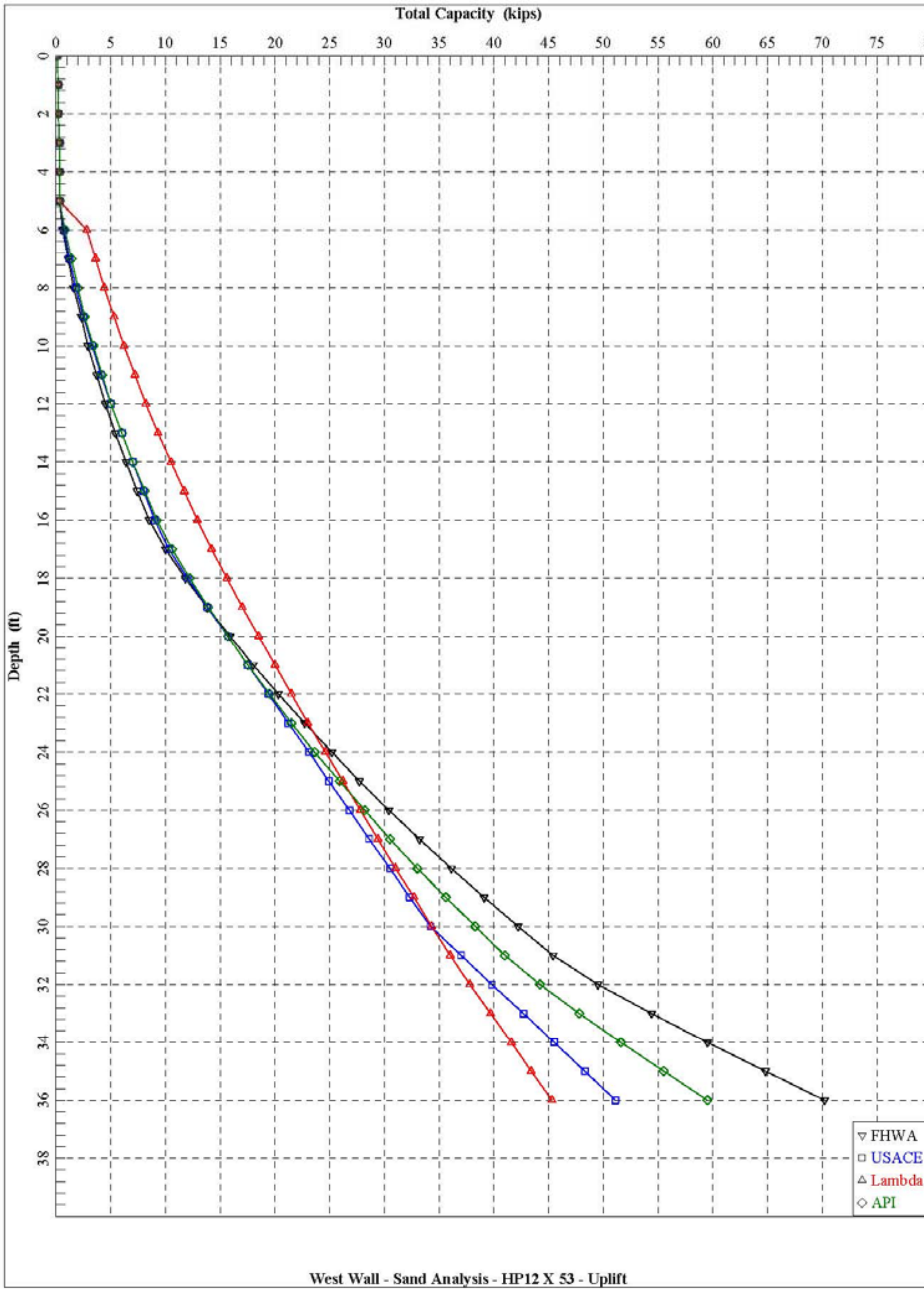

Axial Capacity (kips)
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
West Wall - Sand Analysis - HP10 X 57

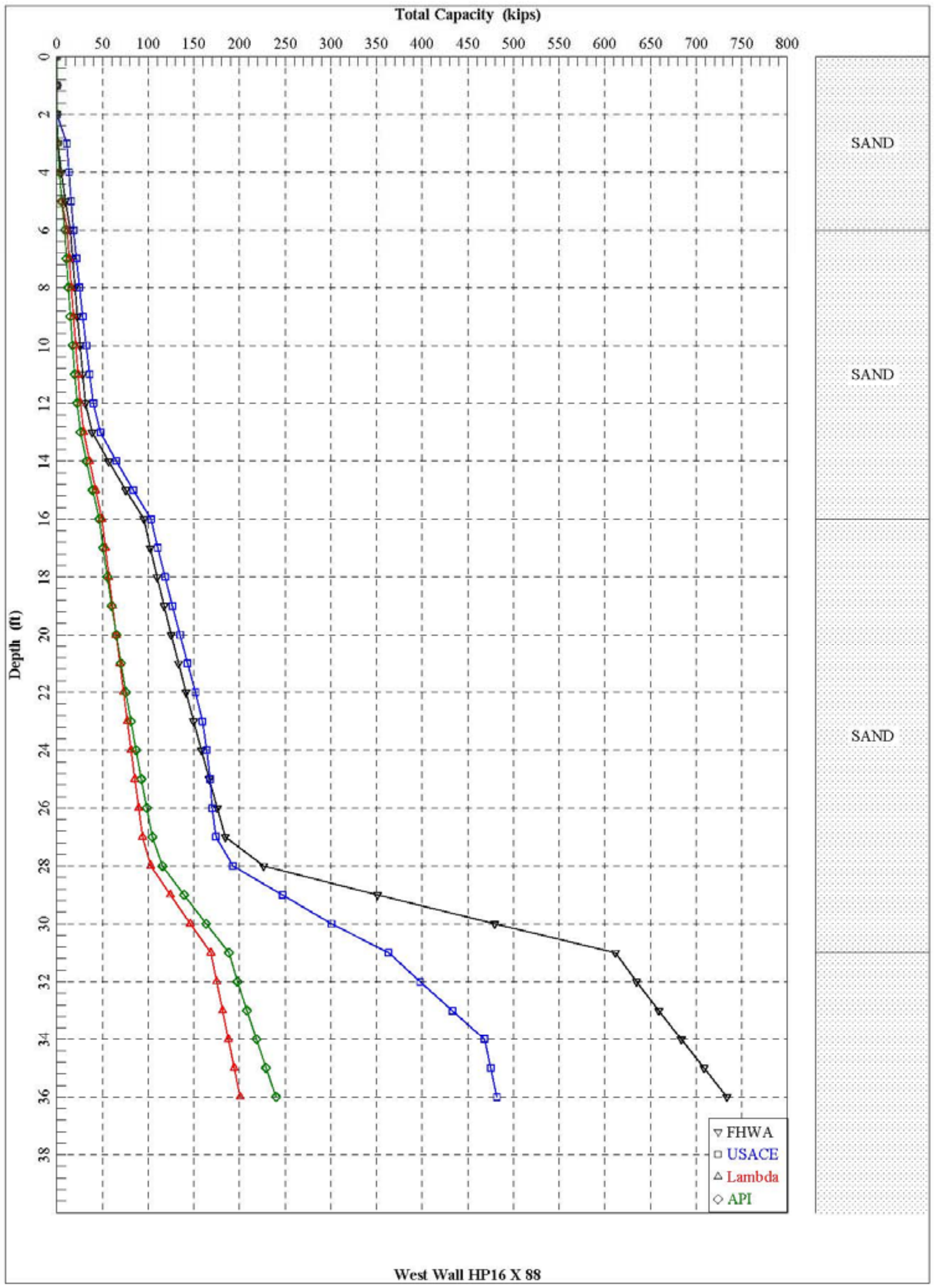

Total Capacity (kips)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - HP10 X 57 - Uplift

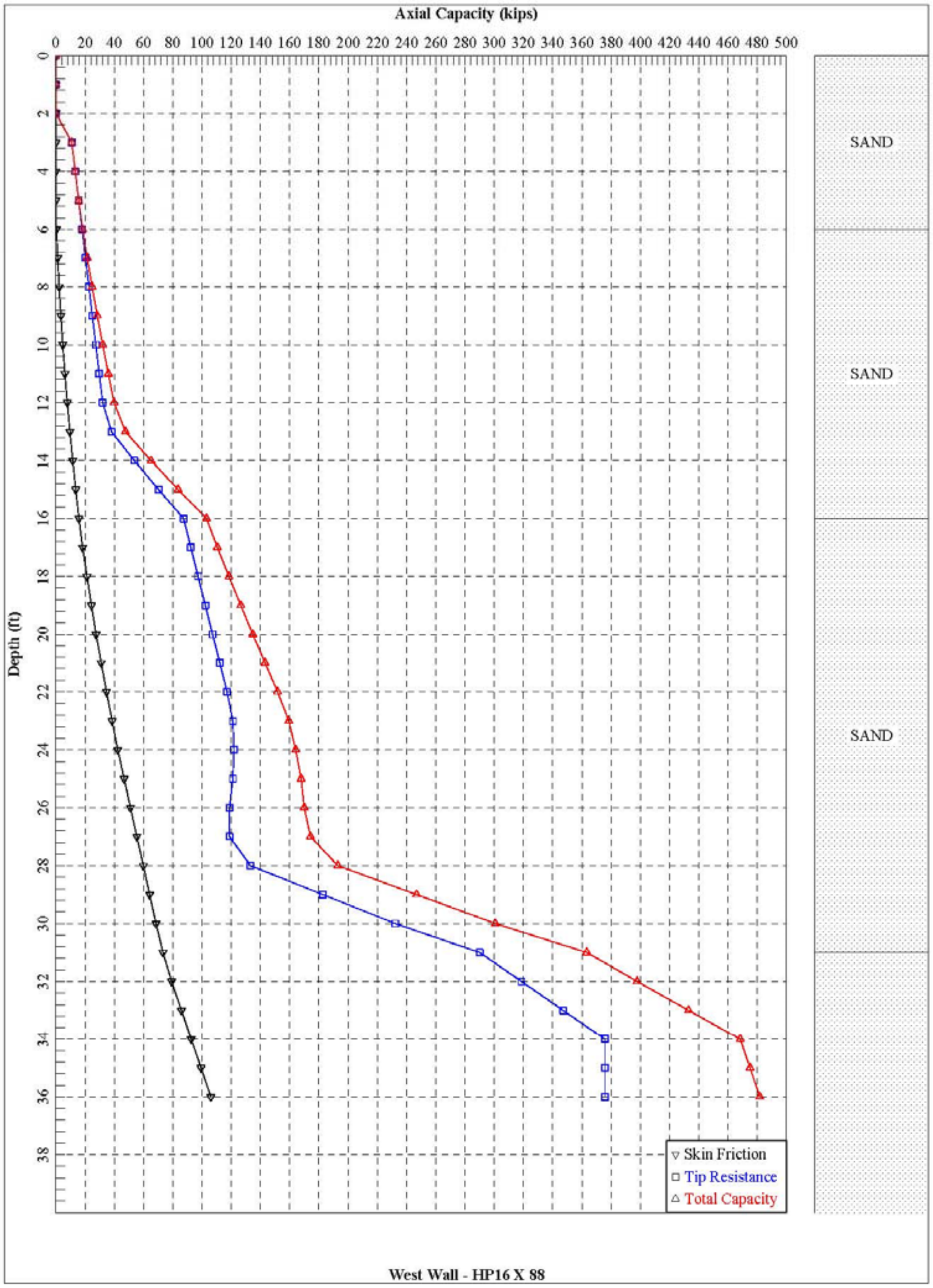

Total Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - HP12 X 53

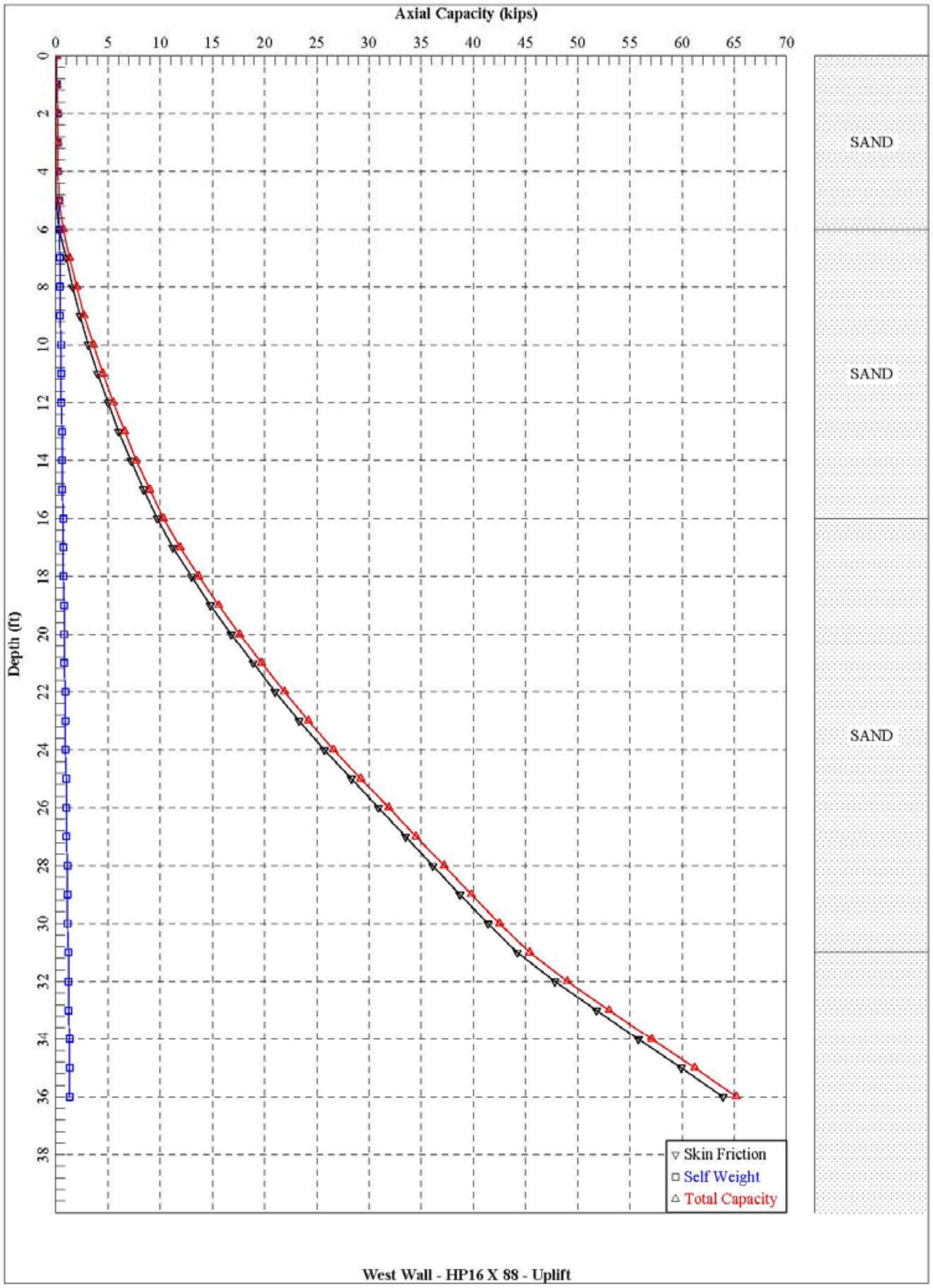


West Wall - Sand Analysis - HP12 X 53


Total Capacity (kips)
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
FHWA
USACE
Lambda
API
West Wall - Sand Analysis - HP12 X 53 - Uplift


Total Capacity (kips)
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 800
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
SAND
SAND
SAND
FHWA
USACE
Lambda
API
West Wall HP16 X 88


Axial Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
West Wall - HP16 X 88


Axial Capacity (kips)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
SAND
SAND
SAND
Skin Friction
Self Weight
Total Capacity
West Wall - HP16 X 88 - Uplift

Computed By: BLT | | Jan 14

Lateral Analysis

Assume $f_{y\,steel}$ = 45 ksi (pipes) & 50 ksi (H-piles)

Structural Acceptability - Determine Ultimate Bending Moment (Yield Moment) (i.e. loading to produce Plastic Hinge)

$M_y = f_y \cdot Z_x$ ← about Strong Axis

Plastic Section Modulus; $M_y = 50^{ksi} \cdot Z_{xx}$

HP 10×42 $d = 9.70''$ $b = 10.10''$ $Z_{xx} = 48.3\ in^3$ ∴ $M_y = 2{,}415$ in-kips

HP 10×57 $d = 9.99''$ $b = 10.20''$ $Z_{xx} = 66.5\ in^3$ $M_y = 3{,}325$ in-kips

HP 12×53 $d = 11.80''$ $b = 12.0''$ $Z_{xx} = 74.0\ in^3$ $M_y = 3{,}700$ in-kips

PP 12" dia w/ 0.312" wall Thickness

$S = \frac{\pi(D^4 - d^4)}{32D}$ & $Z_p = \frac{D^3 - d^3}{6}$ ∴ $Z_p = \frac{12^3 - 11.38^3}{6} = 42.4 in^3$ & $S = \frac{\pi(12^4 - 11.38^4)}{32 \cdot 12} = 32.5 in^3$

∴ $M_y = f_y Z_p = 45^{ksi} \cdot 42.4 in^3 = 1{,}908$ in-kips

HP 16×88 $d = 15.30$ $b = 15.70$ $Z_{xx} = 161\ in^3$ ∴ $M_y = 8{,}050$ in-kips

& $P = 2(b+d) = 2(15.3 + 15.7) = 62.0''$
$A = b \times d = 15.3 \times 15.7 = 240.2\ \square''$
$A_{steel} = 25.8\ \square''$

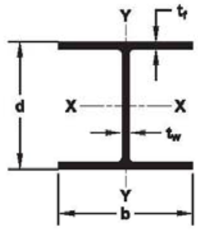
∴ Allowable Design Stress $f_a = 0.33 \cdot 50\ ksi \cdot 25.8\square'' = 426$ kips > 200 kips ok ✓

TOPS 35500

skylinesteel
a NUCOR company

HP

Steel H-Pile



SECTION	Weight lb/ft (kg/m)	Area in² (cm²)	Depth d in (mm)	Flange Width b in (mm)	THICKNESS Flange (t_f) in (mm)	THICKNESS Web (t_w) in (mm)	Coating Area ft²/ft (m²/m)	ELASTIC PROPERTIES AXIS X-X I in⁴ (cm⁴)	AXIS X-X S in³ (cm³)	AXIS X-X Z in³ (cm³)	AXIS X-X r in (cm)	AXIS Y-Y I in⁴ (cm⁴)	AXIS Y-Y S in³ (cm³)	AXIS Y-Y Z in³ (cm³)	AXIS Y-Y r in (cm)
HP 8 HP 200	36 54	10.6 68.4	8.02 204	8.16 207	0.445 11.3	0.445 11.3	3.92 1.19	119 4953	29.8 488	33.6 550.6	3.36 8.53	40.3 1677	9.88 162	15.2 249.1	1.95 4.95
HP 10 HP 250	42 63	12.4 80.0	9.70 246	10.10 257	0.420 10.7	0.415 10.5	4.83 1.47	210 8741	43.4 711	48.3 791.5	4.13 10.5	71.7 2984	14.2 233	21.8 357.2	2.41 6.12
	57 85	16.7 108	9.99 254	10.20 259	0.565 14.4	0.565 14.4	4.91 1.50	294 12237	58.8 964	66.5 1089.7	4.18 10.6	101 4204	19.7 323	30.3 496.5	2.45 6.22
HP 12 HP 310	53 79	15.5 100	11.80 300	12.00 305	0.435 11.0	0.435 11.0	5.82 1.77	393 16358	66.7 1093	74.0 1212.6	5.03 12.8	127 5286	21.1 346	32.2 527.7	2.86 7.26
	63 94	18.4 119	11.90 302	12.10 307	0.515 13.1	0.515 13.1	5.86 1.79	472 19646	79.1 1296	88.3 1447.0	5.06 12.9	153 6368	25.3 415	38.7 634.2	2.88 7.32
	74 110	21.8 141	12.10 307	12.20 310	0.610 15.5	0.605 15.4	5.91 1.80	569 23683	93.8 1537	105 1720.6	5.11 13.0	186 7742	30.4 498	46.6 763.6	2.92 7.42
	84 125	24.6 159	12.30 312	12.30 312	0.685 17.4	0.685 17.4	5.97 1.82	650 27055	106 1737	120 1966.4	5.14 13.1	213 8866	34.6 567	53.2 871.8	2.94 7.47
HP 14 HP 360	73 109	21.4 138	13.60 345	14.60 371	0.505 12.8	0.505 12.8	6.96 2.12	729 30343	107 1753	118 1933.7	5.84 14.8	261 10864	35.8 587	54.6 894.7	3.49 8.86
	89 132	26.1 168	13.80 351	14.70 373	0.615 15.6	0.615 15.6	7.02 2.14	904 37627	131 2147	146 2392.5	5.88 14.9	326 13569	44.3 726	67.7 1109.4	3.53 8.97
	102 152	30.1 194	14.00 356	14.80 376	0.705 17.9	0.705 17.9	7.06 2.15	1050 43704	150 2458	169 2769.4	5.92 15.0	380 15817	51.4 842	78.8 1291.3	3.56 9.04
	117 174	34.4 222	14.20 361	14.90 378	0.805 20.4	0.805 20.4	7.12 2.34	1220 50780	172 2819	194 3179.1	5.96 15.1	443 18439	59.5 975	91.4 1497.8	3.59 9.12
HP 16 HP 410	88 131	25.8 167	15.30 389	15.70 399	0.540 13.7	0.540 13.7	7.52 2.29	1110 46201	145 2376	161 2638.3	6.56 16.7	349 14526	44.5 729	68.2 1117.6	3.68 9.35
	101 150	29.9 193	15.50 394	15.80 401	0.625 15.9	0.625 15.9	7.56 2.30	1300 54110	168 2753	187 3064.4	6.59 16.7	412 17149	52.2 855	80.1 1312.6	3.71 9.42
	121 180	35.8 231	15.80 401	15.90 404	0.750 19.1	0.750 19.1	7.62 2.32	1590 66180	201 3294	226 3703.5	6.66 16.9	504 20978	63.4 1039	97.6 1599.4	3.75 9.53
	141 210	41.7 269	16.00 406	16.00 406	0.875 22.2	0.875 22.2	7.69 2.34	1870 77835	234 3835	264 4326.2	6.70 17.0	599 24932	74.9 1227	116 1900.9	3.79 9.63
	162 241	47.7 308	16.30 414	16.10 409	1.000 25.4	1.000 25.4	7.75 2.36	2190 91154	269 4408	306 5014.4	6.78 17.2	697 29011	86.6 1419	134 2195.9	3.82 9.70
	183 272	54.1 349	16.50 419	16.30 414	1.130 28.7	1.130 28.7	7.81 2.38	2510 104473	304 4982	349 5719.1	6.81 17.3	818 34047	100.0 1639	156 2556.4	3.89 9.88
HP 18 HP 460	135 201	39.9 257	17.50 445	17.80 452	0.750 19.1	0.750 19.1	8.54 2.60	2200 91570	251 4113	281 4604.7	7.43 18.9	706 29386	79.3 1299	122 1999.2	4.21 10.7
	157 234	46.2 298	17.70 450	17.90 455	0.870 22.1	0.870 22.1	8.60 2.62	2570 106971	290 4752	327 5358.5	7.46 18.9	833 34672	93.1 1526	143 2343.3	4.25 10.8
	181 269	53.2 343	18.00 457	18.00 457	1.000 25.4	1.000 25.4	8.66 2.64	3020 125701	336 5506	379 6210.7	7.53 19.1	974 40541	108.0 1770	167 2736.6	4.28 10.9
	204 304	60.2 388	18.30 465	18.10 460	1.130 28.7	1.130 28.7	8.73 2.66	3480 144847	380 6227	433 7095.6	7.60 19.3	1120 46618	124.0 2032	191 3129.9	4.31 11.0

skylinesteel a NUCOR company

HP

Steel H-Pile

Available Steel Grades								
AMERICAN			CANADIAN			EUROPEAN**		
ASTM	YIELD STRENGTH		CSA G40.21	YIELD STRENGTH		EN 10034	YIELD STRENGTH	
	(ksi)	(MPa)		(ksi)	(MPa)		(ksi)	(MPa)
A 36	36	250	Grade 300 W	44	300	HISTAR 355	51	355
A 572 Grade 50*	50	345	Grade 350 W	50	350	HISTAR 420	61	420
A 588	50	345				HISTAR 460	67	460
A 690	50	345						

* Standard grade for H-Pile.

**HISTAR only available in some sizes.

Splicer and H-Pile Point

Splicer

H-Pile Point

Delivery Conditions & Tolerances

	ASTM A 6	
Mass	± 2.5%	
Length§		
30 Feet and Under	± 0.375 inches	
Over 30 Feet	+ (0.375 inches + (length - 30)/80)	– 0.375 inches
Depth	± 0.125 inches	– 0.1875 inches
Flange Width	+ 0.25 inches	
Flanges out of Square		
HP 8 x 42 - HP 12 x 84	≤ 0.25 inches	
HP 14 x 73 - HP 14 x 117	≤ 0.3125 inches	
Web off Center	≤ 0.1875 inches	
Greatest Depth over Theoretical	≤ 0.25 inches	
Camber and Sweep***		
45 Feet and Under	(0.125")(Length in feet/10) but not over 0.375"	
Over 45 Feet	(0.375") + (0.125" (Length in feet - 45)/10)	

§ For HP ordered as bearing piles, length tolerances are +5 in. and -0 in.

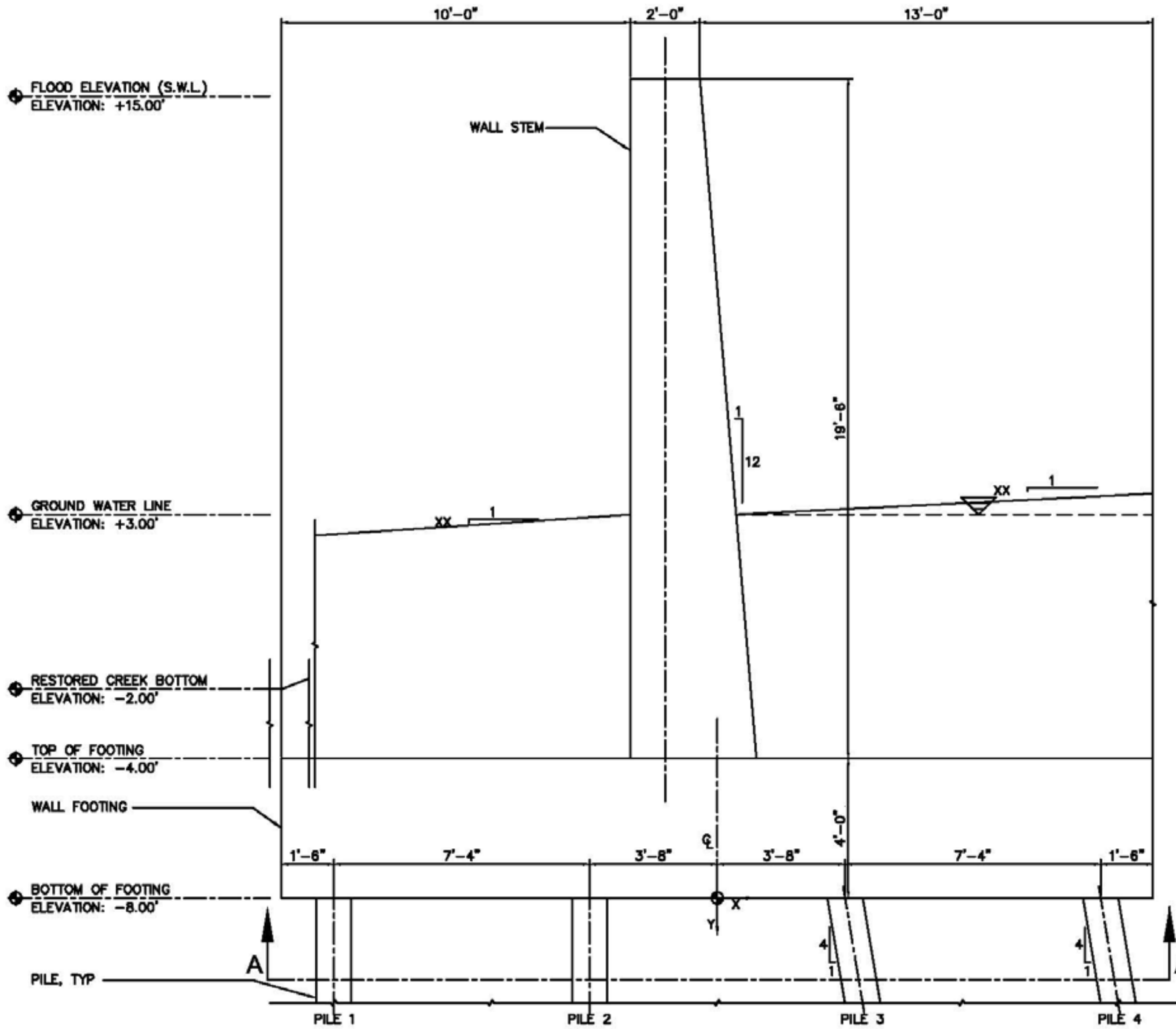
***For the HP 10 x 42, 12 x 53, 12 x 63, 14 x 73, and 14 x 89 ordered as columns, tolerances are subject to negotiation with manufacturer.

Maximum Rolled Lengths'

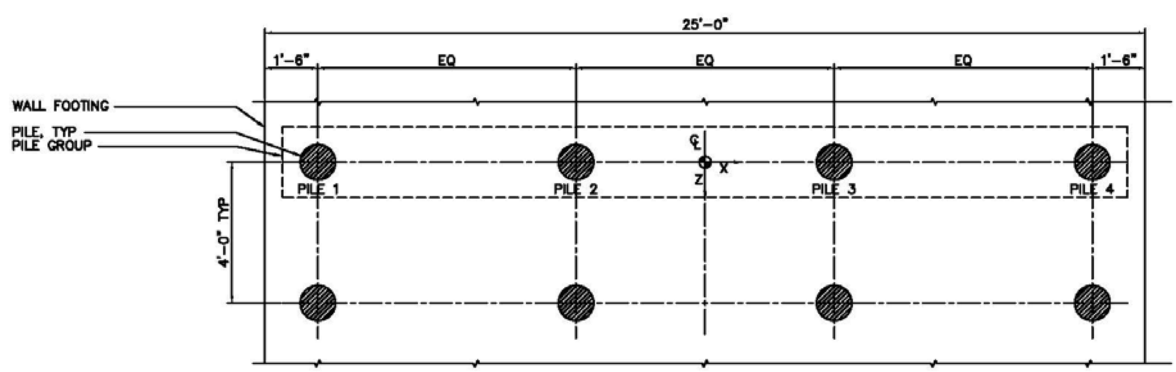
HPs	100 '	30.5 m

' Longer lengths may be possible upon request.

Technical Hotline: 1-866-875-9546 | engineering@skylinesteel.com www.skylinesteel.com



TYPICAL T-TYPE WALL SECTION - FOUR PILE ARRANGEMENT



VIEW A-A - 25 FOOT FOUR PILE ARRANGEMENT

HURRICANE SANDY LIMITED REEVALUATION REPORT

UNION BEACH, NEW JERSEY

DRAFT ENGINEERING APPENDIX

SUB APPENDIX B-2

FLOODWALL PILE ANALYSES
EAST WALLS

Revised March 2015

Preliminary Flood Wall Pile Analysis – East Alignment
Union Beach, New Jersey
February 2014(revised March 2015)

1. GENERAL: Preliminary Axial compressive, uplift and lateral load capacities are provided below to the Structural Engineer to determine the preliminary dimension of the pile caps for the pile supported T-walls along the East Alignment. Pile capacities for the West Flood Wall Alignment will be submitted under a different document.

2. SOIL PROFILE: Drill holes C-17, C-18, and C-20 obtained from the 2003 Feasibility Report dated September 2003 were utilized to develop a "Generalized Soil Profile" which was utilized for the preliminary pile analysis. Based upon observations and interpretations of the limited laboratory test results and borings logs, it is unclear whether the clayey silts and silty sands would behave as a clay (undrained condition) or a sand (drained condition). Therefore, analysis for both sand and clay were conducted for both the axial and lateral pile analysis and the most conservative results utilized. Based upon these analysis, the sand analysis governed for the axial capacity and the clay analysis governed for the lateral analysis. Specific assumptions for each analysis are discussed below.

3. MINIMUM PILE SPACING: Piles shall be spaced no closer than a minimum center- to- center spacing of 3 diameters. The lateral load analysis utilized P multipliers (Pm) to account for group effects.

4. AXIAL PILE ANALYSIS: APILE plus 5.0 software developed by ENSOFT was utilized to compute the axial pile capacities. The software evaluated the axial capacity for four different methods (FHWA, USACE, Lambda and API); the capacities provided in the following table are computed from the USACE method. Estimated axial Pile loadings as provided by the Structural Engineer range from 24 kips to 234 kips in axial compression and 15 kips in axial tension. Five different pile types,(1) 12-inch pipe pile with 0.312-inch wall thickness driven open ended, (2) HP10 X 42, (3) HP10 X 57, (4) HP12 X 53, and (5) HP16 X 88 were analyzed and their allowable axial capacities provided for three different pile lengths (30, 40 and 50 feet). The following assumptions were utilized in the analysis:

    a. Based upon the 2003 Feasibility report, it was assumed that the bottom of the pile cap is 6 feet below grade.
    b. Existing and finished grades are unknown and therefore down drag was not considered in this preliminary analysis. This shall be revisited during the final design.
    c. Since a static load test will be required during construction, a factor of safety of 2 was implemented for the allowable bearing capacity.

Preliminary Allowable Axial Pile Capacities (with factor of safety of 2)

	12" Diameter Pipe w/0.312" wall thickness (kips)	HP10 X 42 (kips)	HP10 X 57 (kips)	HP12 X 53 (kips)	HP16 X 88
Q_{all} for 30' length	30	30	35	50	NA
T_{all} for 30' length	15	15	20	25	NA
Q_{all} for 40' length	55	60	60	90	NA
T_{all} for 40' length	20	30	30	40	NA
Q_{all} for 50' length	85	90	95	145	NA
T_{all} for 50' length	30	40	40	60	NA
Q_{all} for 58' length	NA	NA	NA	NA	200
T_{all} for 58' length	NA	NA	NA	NA	170

Q_{all} = Allowable axial capacity in compression; T_{all} = Allowable axial capacity in Tension (Uplift); NA = Not analyzed

5. LATERAL PILE ANALYSIS: LPILE plus 2012.6 version software developed by ENSOFT was utilized for the lateral pile analysis. Four different pile types,(1) 12-inch pipe pile with a 0.312-inch wall thickness driven open ended, (2) HP10 X 42, (3) HP10 X 57, (4) HP12 X 53, and (5) HP16 X 88 at lengths of 30 feet were analyzed and the results tabulated below for each section. The following assumptions were made in the analysis:

   a. Since the head restraint was unknown, maximum moments and lateral deflections for both free head and fixed head are provided. Since it is anticipated that a "partially fixed" restraint will be utilized, a maximum "free head" deflection of 0.75 inches was assumed as the governing deflection.
   b. Axial loading was unknown at the time of analysis and therefore not implemented in the lateral analysis.
   c. Top of pile was conservatively assumed at 3 feet below grade in lieu of 6 feet as indicated in the axial capacity analysis.
   d. A cyclic loading of 500 cycles was utilized. This is an estimate and will need to be further investigated during the final design.
   e. As indicated above, the clay analysis governed. In addition, p-y curves for "soft clay" were utilized in lieu of "stiff clay in the presence of free water" as it is assumed that any voids that may develop around the pile due to dynamic loadings would be filled with sands or silts making the "stiff clay in the presence of free water" analysis to conservative. This is an assumption that will need to be further analyzed during the final design once the dynamic loadings and soil conditions are better defined.
   f. As indicated above, piles shall be spaced a minimum center- to- center of 3 diameters. To account for group effects, P-multiplier (Pm) values of 0.8 (for lead row), 0.4 (for second row) and 0.3 (for third and fourth rows) were utilized to determine the average

lateral loading for a free head deflection of 0.75 inches and maximum moments. ***The lateral pile loadings (service load) provided in the row entitled "Group Effect" illustrated in the below tables shall be used for the preliminary design to take into account the group effects***.

Union Beach – East Wall - Preliminary Lateral Pile Capacities

1. **12-inch Diameter Pipe Pile with 0.312" wall thickness & HP10 X 42**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	16	570 @ 6.0'	0.76	-600 @ 0	0.21
0.4	9.5	395 @ 7.0	0.72	-420 @ 0	0.20
0.3	8.0	360 @ 7.5	0.76	-385 @ 0	0.22
GROUP EFFECTS	***11.0***	***442***	***0.75***	***-468***	***0.21***

2. **HP10 X 57**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	17	670 @ 6.0'	0.72	-700 @ 0	0.20
0.4	10.5	490 @ 7.0'	0.73	-520 @ 0	0.20
0.3	8.5	420 @ 8.0'	0.73	-450 @ 0	0.20
GROUP EFFECTS	***12.0***	***527***	***0.75***	***-556***	***0.20***

3. **HP12 X 53**

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8	22	975 @ 6.0'	0.72	-1000 @ 0	0.21
0.4	15	790 @ 7.0'	0.76	-800 @ 0	0.22
0.3	12.5	700 @ 8.0'	0.75	-700 @ 0	0.22
GROUP EFFECTS	***16.5***	***822***	***0.75***	***-833***	***0.22***

4. HP16 X 88

Pm	Lateral Loading (kips)	Max Moment (Free Head) (in-kips)	Max Deflection (Free Head) (inches)	Max Moment @ top of pile (Fixed Head) (in-kips)	Max Deflection (Fixed Head) (inches)
0.8 (*battered)	23.5	1320 @ 9.0’	0.76	-1400 @ 0	0.21
0.6 (*battered)	19.0	1140 @ 9.0’	0.75	-1200 @ 0	0.20
0.4	16.5	1020 @ 10.0’	0.74	-1100 @ 0	0.20
0.3	13.5	890 @ 11.0’	0.74	-950 @ 0	0.20
GROUP EFFECTS	***20.0***	***1095***	***0.75***	***-1160***	***0.20***

*Last two rows (piles 3 and 4) were evaluated with a 4 to 1 pile per “25-Foot Four Pile Arrangement” drawing provided by Structural Engineer.

6. PRELIMINARY PILE RECOMMENDATIONS: Per Sub Appendix A, PRELIMINARY FLOOD WALL DESIGN, the maximum axial compressive loading per pile is 55.2 kips and the maximum axial tensile loading per pile is 17.7 kips. Per conversations with the Structural Engineer, the maximum lateral pile load is 35.6 kips per pile when distributed to only the two battered piles and 17.8 kips when distributed uniformly to all four piles. Based upon these maximum axial pile loadings and for a lateral loading of 17.8 kips per pile uniformly distributed to all four piles, an HP16 X 88 driven to a minimum elevation of -38 feet (30’ pile length below the bottom of pile cap) is recommended.

7. POINT OF CONTACT: Contact David Tucker, P.E. at 4140-962-6823 or david.l.tucker@usace.army.mil for any further analysis, questions or comments

Checked By: DLT | Jn 2014

C-17 + 18 · Wall Along Eastern Alignment

+ NSF Not Accounted for Since existing & proposed grades Unknown.
++ Assume Pile Cap 6' Below existing Drill hole grade
+++ Undrained shear strength Values from UU & Unconfined Compression Tests Appear Low based upon N-Values. This is likely due to the presence of Sand in Test Specimen.
++++ Due to the Limited Test Results & Classification of ML materials, a Cohesive & Cohesionless Analysis will be conducted on Generalized Profile

SPT Conducted w/ Manual Hammer

$\gamma_m \approx \gamma_{sat} = 115 pcf \therefore \gamma' = 53 pcf$

3'

6'

STRATUM 1 (24') ML

$\bar{N} = 13 bpf$

$\bar{\sigma}_o' = 0.6 tsf$

$C_N = 0.77 \log \frac{20}{0.6} = 1.17$

$\therefore \bar{N}_{160} = 13 \cdot 1.17 = 15 bpf$

$\therefore \phi = 33°$ (med. Dense Sand) fig 34

OR

$C_u = 0.5 tsf$ (Stiff Silt) fig 40

+ Correlates w/ UU & UC Test in C-17

46'

Stratum 2 (13') ML

$\bar{N} = 21 bpf$ & $\bar{\sigma}_o' = 1.06 tsf$

$C_N = 0.77 \log \frac{20}{1.06} = 0.98$

$\therefore \bar{N}_{160} = 21 bpf$

$\phi = 35°$ (med Dense Sand) fig 34

OR

$C_u = 1.0 tsf$ fig 40

STRATUM 3 (9') ML

$\bar{N} = 38 bpf$ + $\bar{\sigma}_o' = 1.35 tsf$

$C_N = 0.77 \log \frac{20}{1.35} = 0.90$

$N_{160} = 34 bpf$ $\therefore \phi = 39°$ fig 34

OR

$C_u = 1.5 tsf$ fig 40

STRATUM 4 ML

$\bar{N} = 62 bpf$

$\phi = 42°$ fig 34

OR

$C_u = 2.0 tsf$ fig 40

Generalized Profile

· Piles shall be spaced a minimum C to C spacing of 3 diameters

TOPS 35500

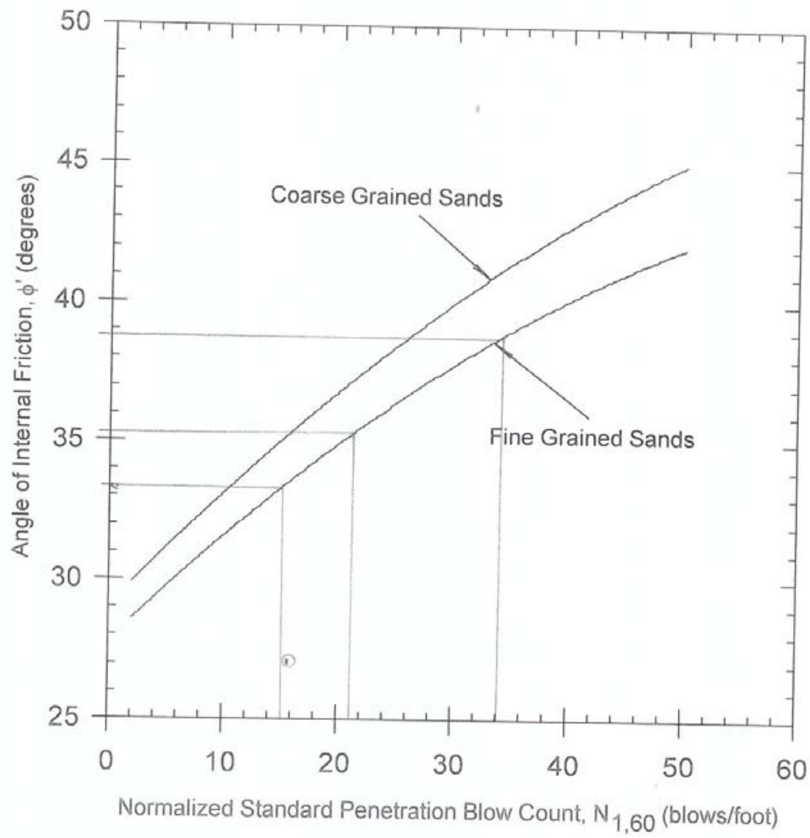
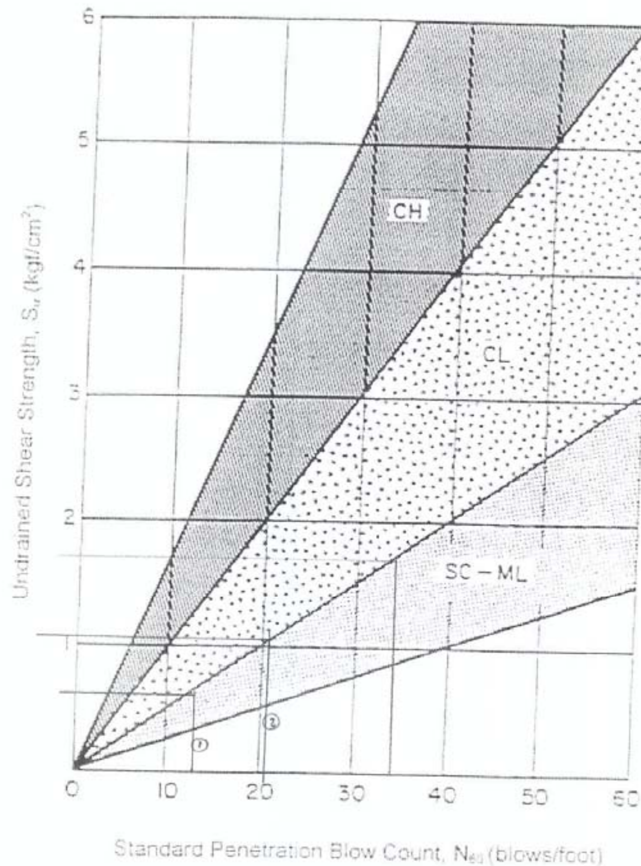


Figure 34. Empirical correlation between friction angle of sands and normalized standard penetration blow count (after Terzaghi et al., 1996)

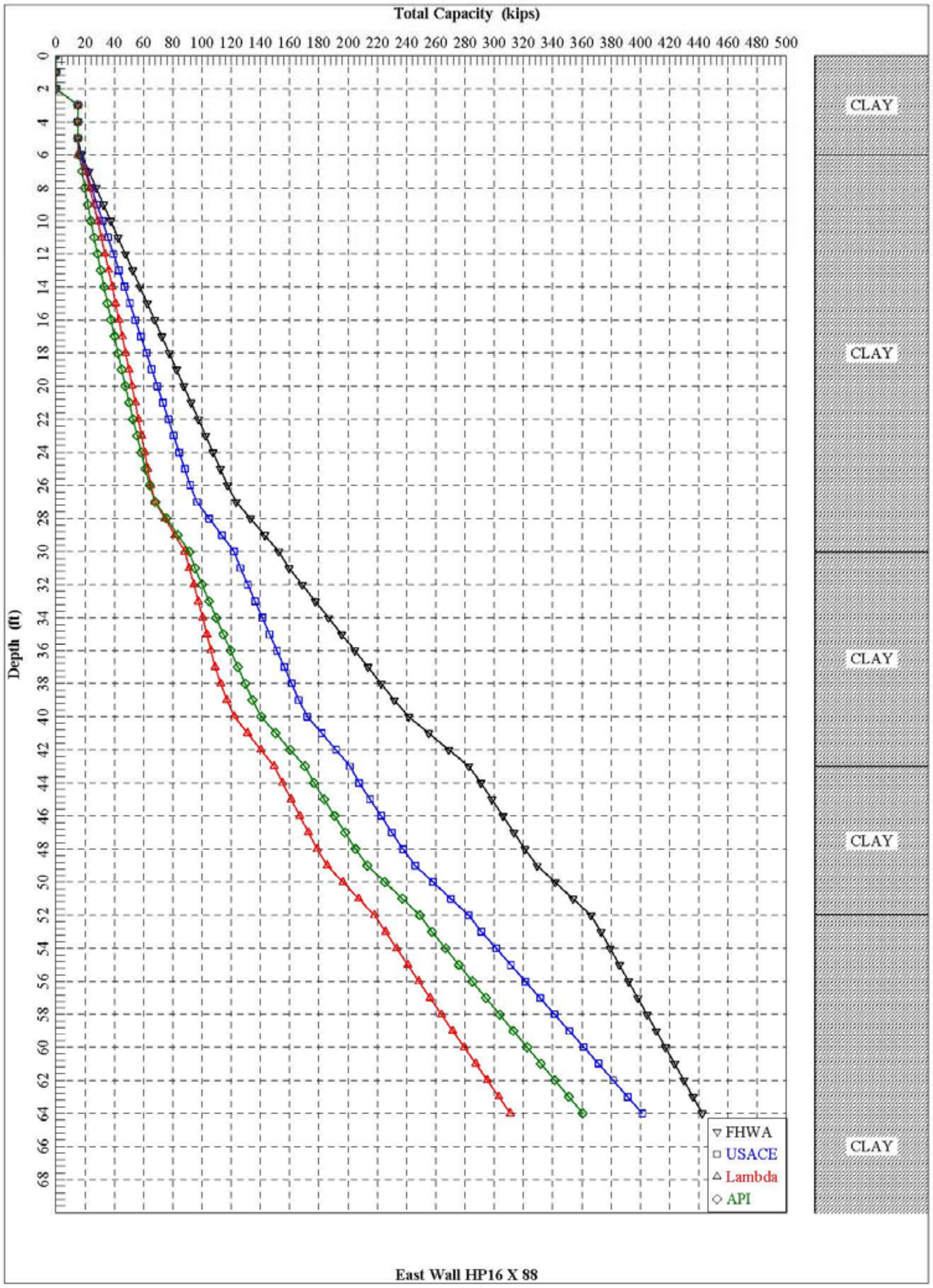
71



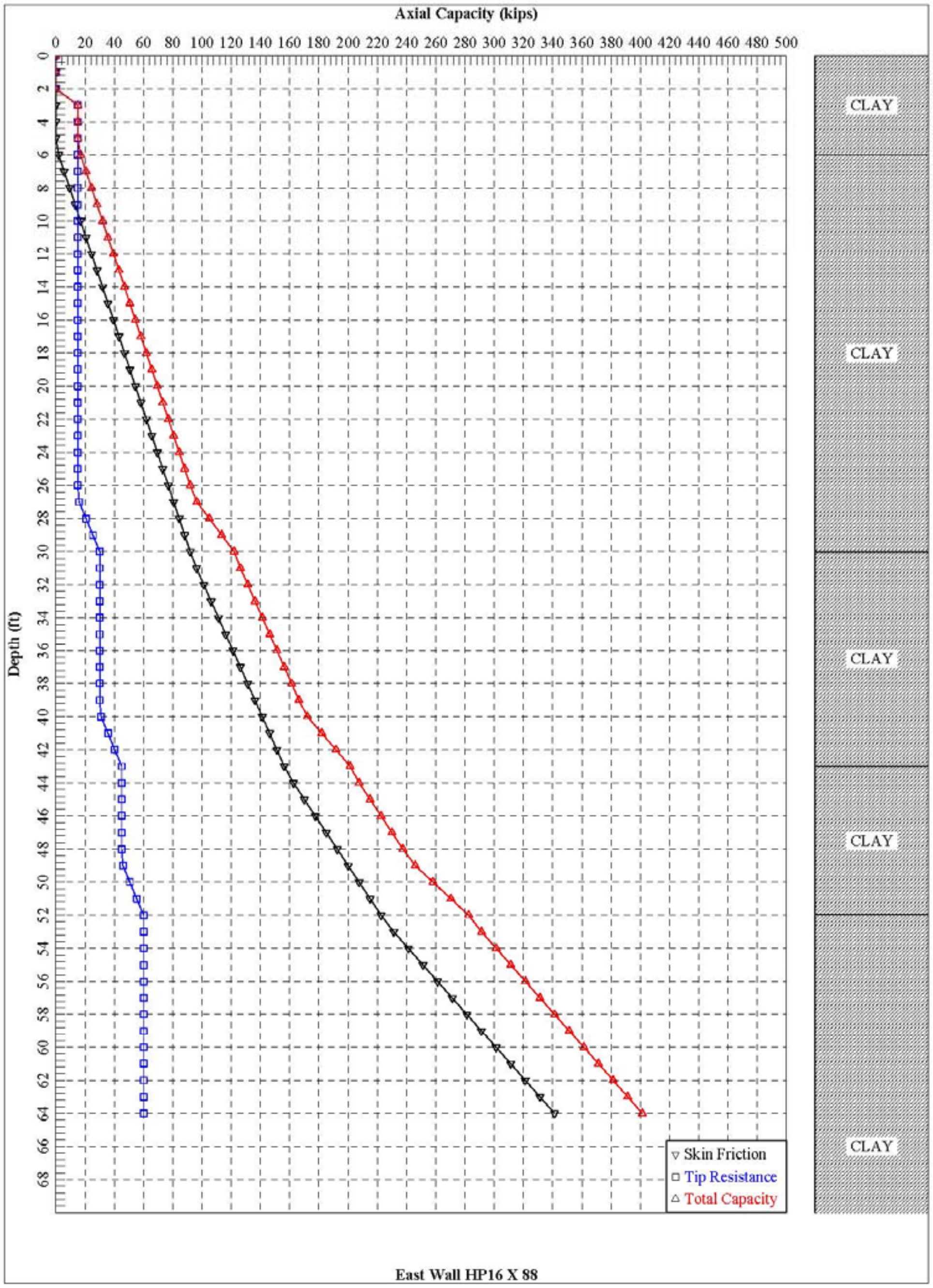
Note: As originally proposed, this correlation used the uncorrected SPT blowcount, N. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which the correlation was based was obtained primarily from tests with such hammers. It therefore seems logical to use N_{60} with this correlation, and it is the recommendation of this report that this be done.

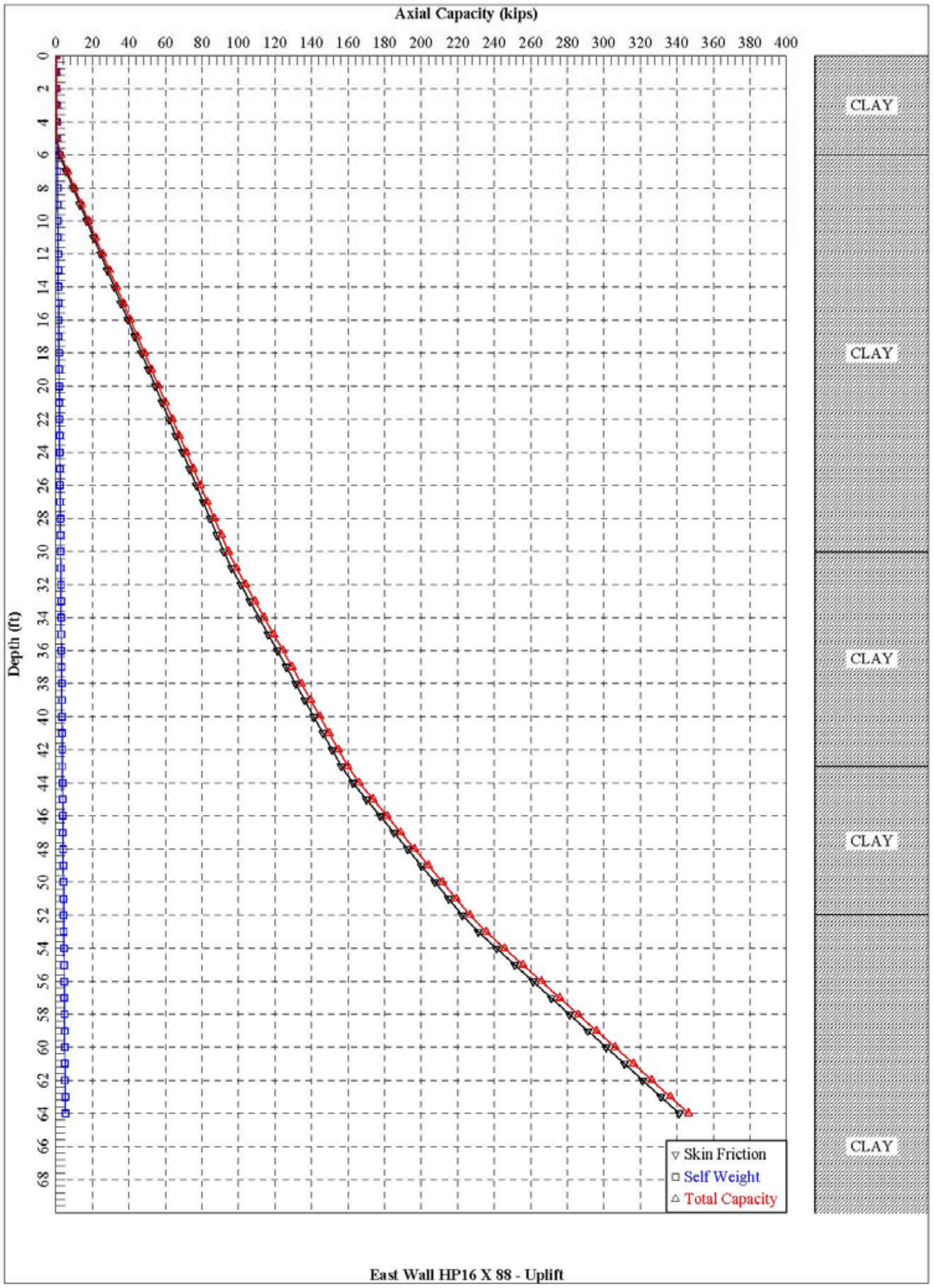
Figure 40. Relationship between standard penetration blow count, N and undrained shear strength, S_u (after Sowers, 1979)

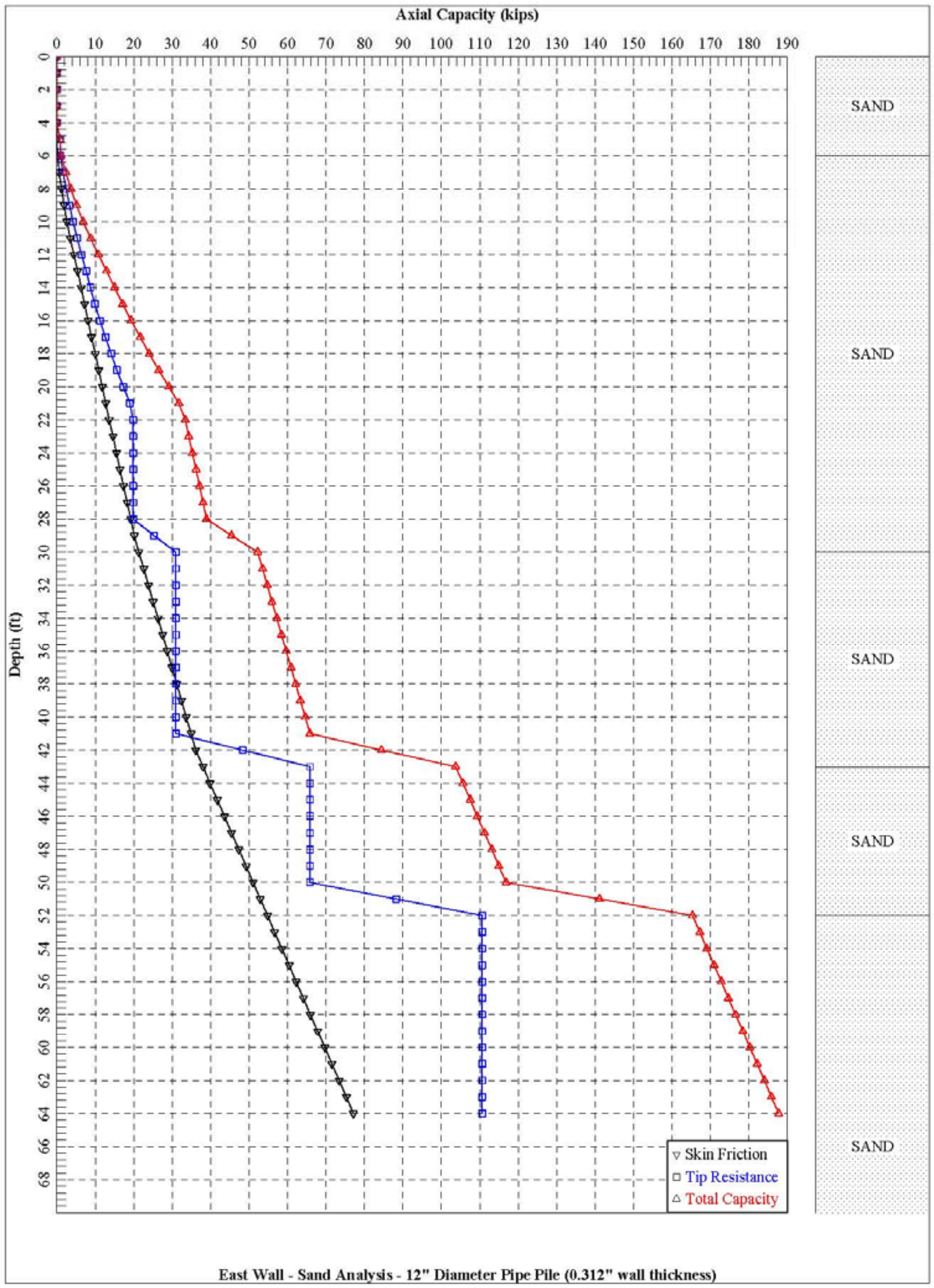
80

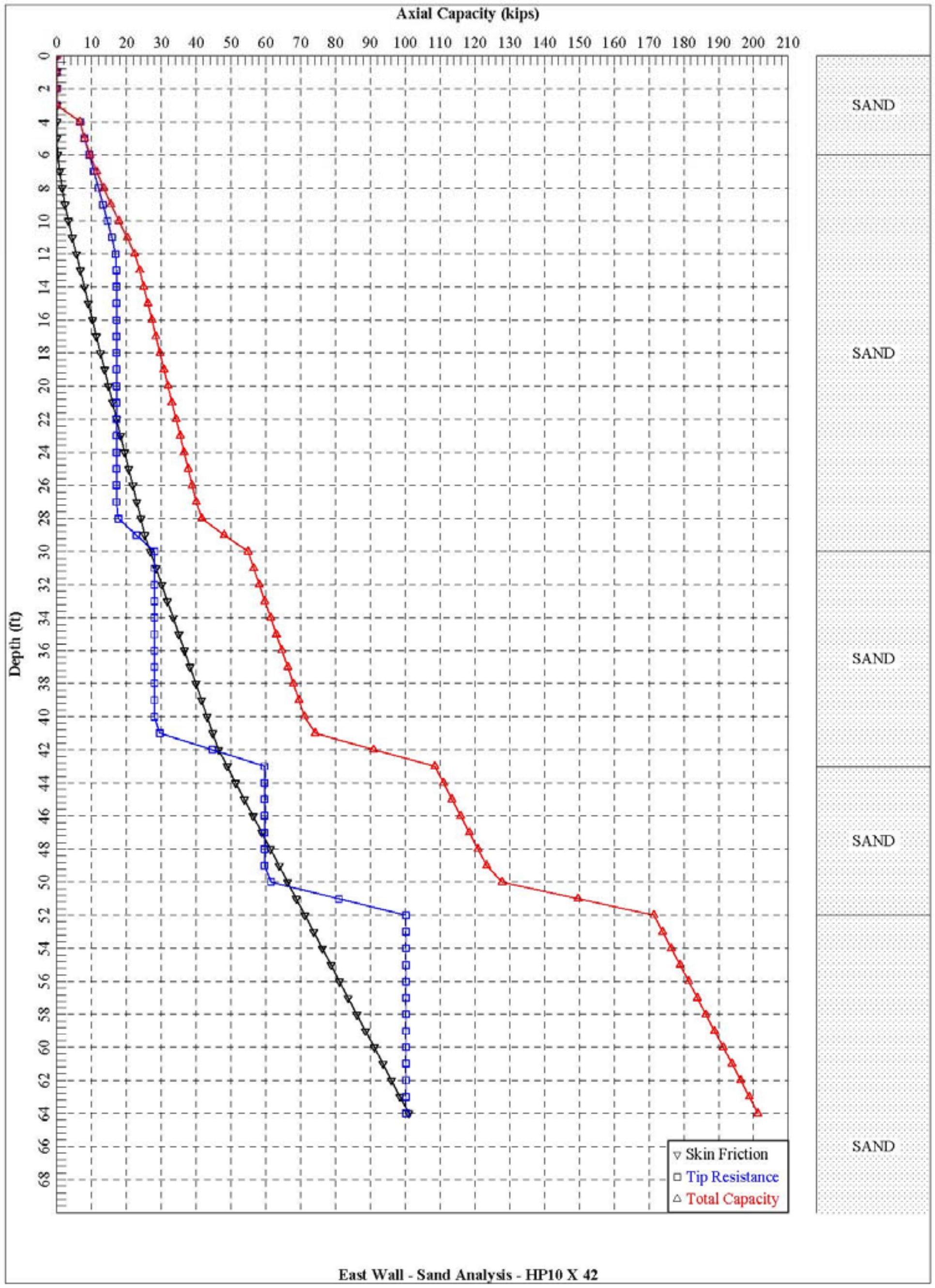


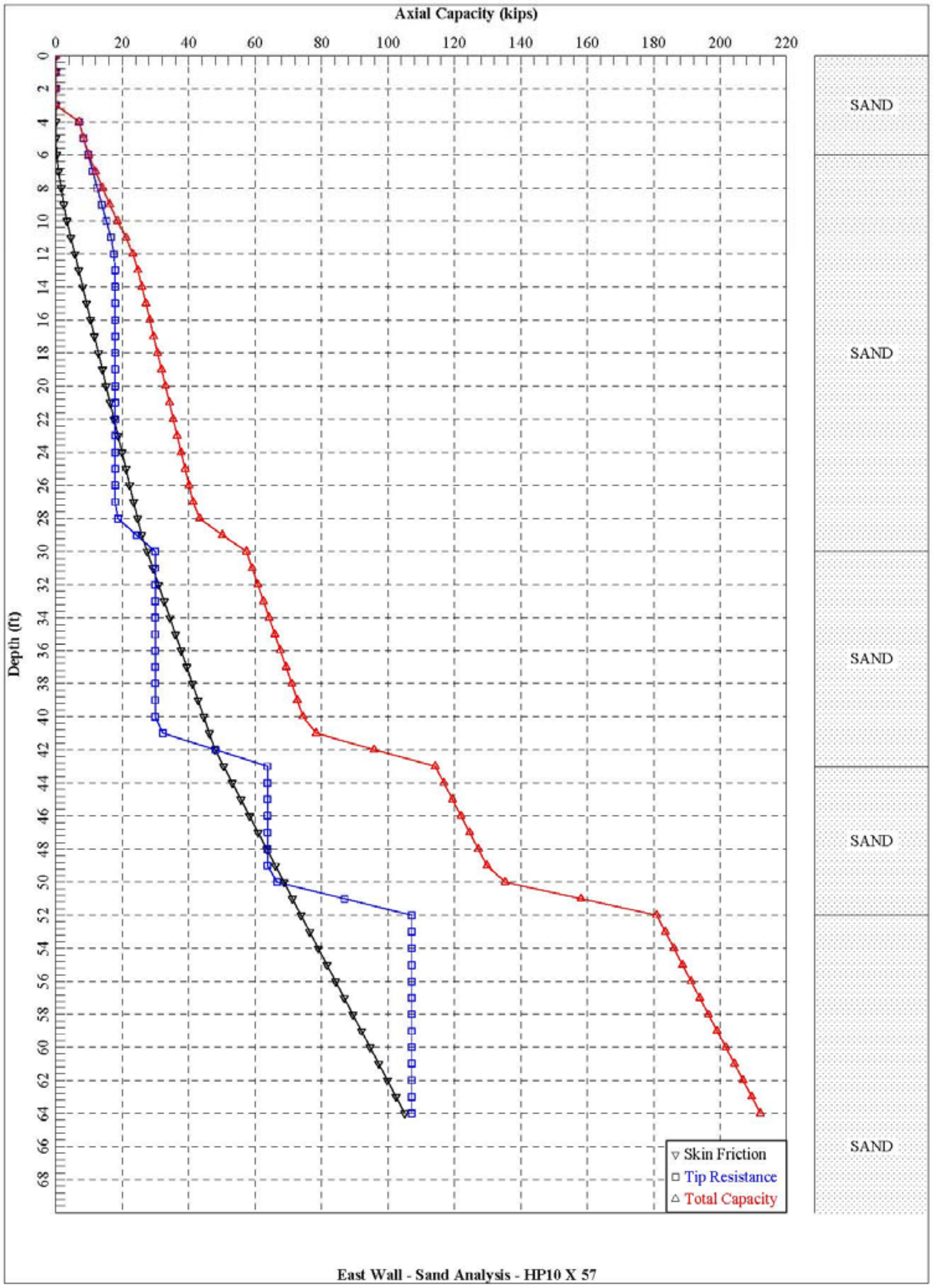
Total Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
CLAY
CLAY
CLAY
CLAY
CLAY
FHWA
USACE
Lambda
API
East Wall HP16 X 88

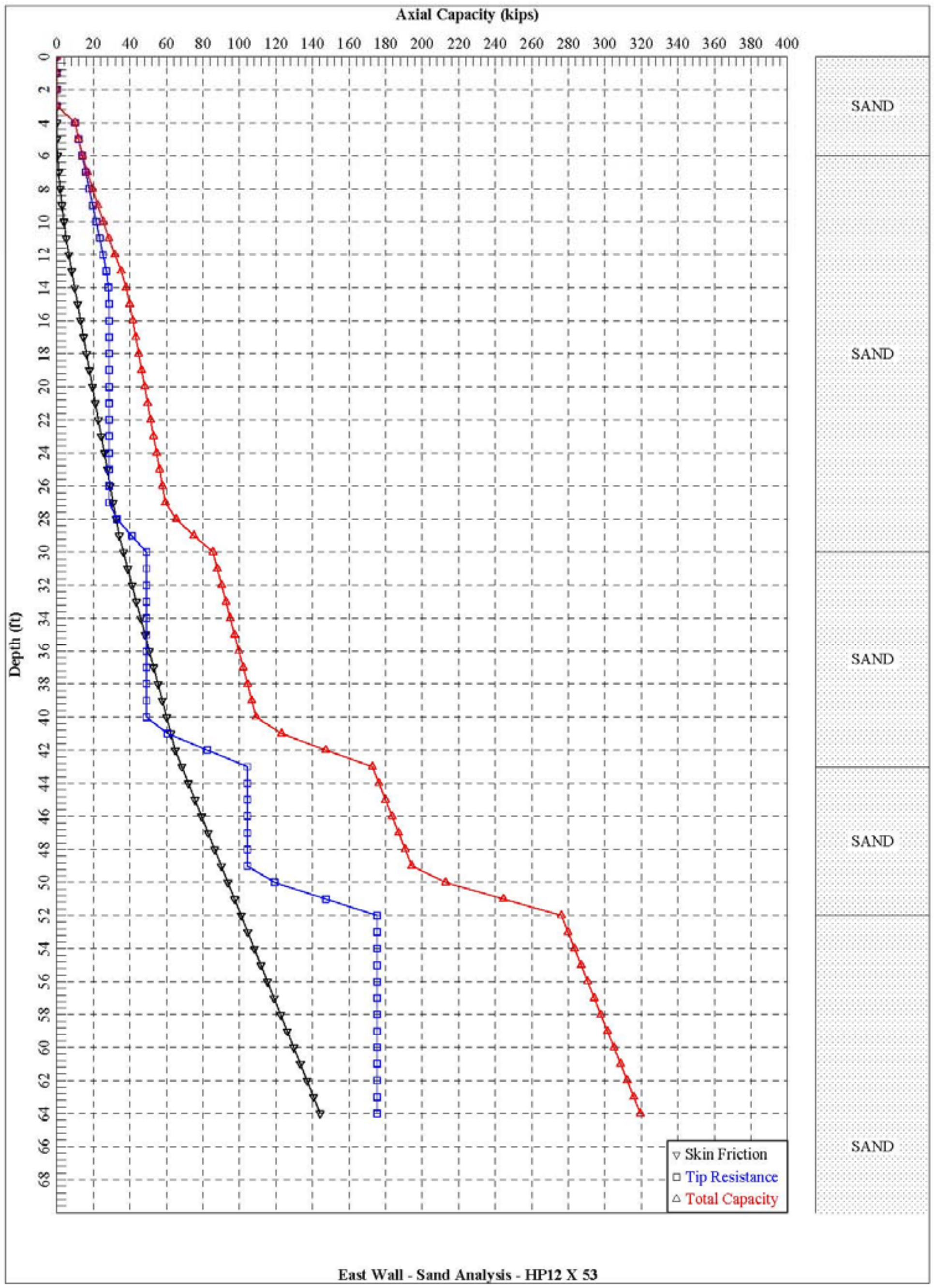

Axial Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
CLAY
CLAY
CLAY
CLAY
CLAY
Skin Friction
Tip Resistance
Total Capacity
East Wall HP16 X 88


Axial Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
CLAY
CLAY
CLAY
CLAY
CLAY
Skin Friction
Self Weight
Total Capacity
East Wall HP16 X 88 - Uplift


Axial Capacity (kips)
Depth (ft)
SAND
SAND
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
East Wall - Sand Analysis - 12" Diameter Pipe Pile (0.312" wall thickness)


Axial Capacity (kips)
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
SAND
SAND
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
East Wall - Sand Analysis - HP10 X 42


Axial Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
SAND
SAND
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
East Wall - Sand Analysis - HP10 X 57


Axial Capacity (kips)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
SAND
SAND
SAND
SAND
SAND
Skin Friction
Tip Resistance
Total Capacity
East Wall - Sand Analysis - HP12 X 53

Computed by: BLT | | | Jan 14

Lateral Analysis

Assume $f_{y\,steel}$ = 45 ksi (pipes) & 50 ksi (H-piles)

Structural Acceptability - Determine Ultimate Bending Moment (Yield Moment) (i.e. loading to produce Plastic Hinge)

$M_y = f_y \cdot Z_x$ ← about Strong Axis

			Plastic Section Modulus	$M_y = 50^{ksi} \cdot Z_{xx}$
HP 10×42	d = 9.70"	b = 10.10"	Z_{xx} = 48.3 in³	∴ M_y = 2,415 in-kips
HP 10×57	d = 9.99"	b = 10.20"	Z_{xx} = 66.5 in³	M_y = 3,325 in-kips
HP 12×53	d = 11.80"	b = 12.0"	Z_{xx} = 74.0 in³	M_y = 3,700 in-kips

PP 12" dia w/ 0.312" wall Thickness

$S = \frac{\pi(D^4 - d^4)}{32D}$ & $Z_p = \frac{D^3 - d^3}{6}$ ∴ $Z_p = \frac{12^3 - 11.38^3}{6} = 42.4\,in^3$ & $S = \frac{\pi(12^4 - 11.38^4)}{32 \cdot 12} = 32.5\,in^3$

∴ $M_y = f_y Z_p = 45^{ksi} \cdot 42.4\,in^3 = 1{,}908$ in-kips

HP 16×88 d = 15.30 b = 15.70 Z_{xx} = 161 in³ ∴ M_y = 8,050 in-kips

& $P = 2(b+d) = 2(15.3 + 15.7) = 62.0"$
$A = b \times d = 15.3 \times 15.7 = 240.2\ \square"$
$A_{s\,steel}$ = 25.8 □"

∴ Allowable Design Stress f_a = 0.33 · 50 ksi · 25.8 □" = 426 kips > 200 kips ok ✓

TOPS 35500

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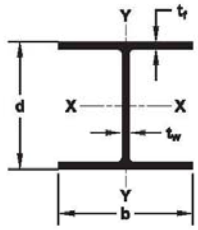
HP

Steel H-Pile



SECTION	Weight	Area	Depth d	Flange Width b	THICKNESS		Coating Area	ELASTIC PROPERTIES							
								AXIS X-X				AXIS Y-Y			
					Flange (t_f)	Web (t_w)		I	S	Z	r	I	S	Z	r
	lb/ft (kg/m)	in² (cm²)	in (mm)	in (mm)	in (mm)	in (mm)	ft²/ft (m²/m)	in⁴ (cm⁴)	in³ (cm³)	in³ (cm³)	in (cm)	in⁴ (cm⁴)	in³ (cm³)	in³ (cm³)	in (cm)
HP 8 HP 200	36 54	10.6 68.4	8.02 204	8.16 207	0.445 11.3	0.445 11.3	3.92 1.19	119 4953	29.8 488	33.6 550.6	3.36 8.53	40.3 1677	9.88 162	15.2 249.1	1.95 4.95
HP 10 HP 250	42 63	12.4 80.0	9.70 246	10.10 257	0.420 10.7	0.415 10.5	4.83 1.47	210 8741	43.4 711	48.3 791.5	4.13 10.5	71.7 2984	14.2 233	21.8 357.2	2.41 6.12
	57 85	16.7 108	9.99 254	10.20 259	0.565 14.4	0.565 14.4	4.91 1.50	294 12237	58.8 964	66.5 1089.7	4.18 10.6	101 4204	19.7 323	30.3 496.5	2.45 6.22
HP 12 HP 310	53 79	15.5 100	11.80 300	12.00 305	0.435 11.0	0.435 11.0	5.82 1.77	393 16358	66.7 1093	74.0 1212.6	5.03 12.8	127 5286	21.1 346	32.2 527.7	2.86 7.26
	63 94	18.4 119	11.90 302	12.10 307	0.515 13.1	0.515 13.1	5.86 1.79	472 19646	79.1 1296	88.3 1447.0	5.06 12.9	153 6368	25.3 415	38.7 634.2	2.88 7.32
	74 110	21.8 141	12.10 307	12.20 310	0.610 15.5	0.605 15.4	5.91 1.80	569 23683	93.8 1537	105 1720.6	5.11 13.0	186 7742	30.4 498	46.6 763.6	2.92 7.42
	84 125	24.6 159	12.30 312	12.30 312	0.685 17.4	0.685 17.4	5.97 1.82	650 27055	106 1737	120 1966.4	5.14 13.1	213 8866	34.6 567	53.2 871.8	2.94 7.47
HP 14 HP 360	73 109	21.4 138	13.60 345	14.60 371	0.505 12.8	0.505 12.8	6.96 2.12	729 30343	107 1753	118 1933.7	5.84 14.8	261 10864	35.8 587	54.6 894.7	3.49 8.86
	89 132	26.1 168	13.80 351	14.70 373	0.615 15.6	0.615 15.6	7.02 2.14	904 37627	131 2147	146 2392.5	5.88 14.9	326 13569	44.3 726	67.7 1109.4	3.53 8.97
	102 152	30.1 194	14.00 356	14.80 376	0.705 17.9	0.705 17.9	7.06 2.15	1050 43704	150 2458	169 2769.4	5.92 15.0	380 15817	51.4 842	78.8 1291.3	3.56 9.04
	117 174	34.4 222	14.20 361	14.90 378	0.805 20.4	0.805 20.4	7.12 2.34	1220 50780	172 2819	194 3179.1	5.96 15.1	443 18439	59.5 975	91.4 1497.8	3.59 9.12
HP 16 HP 410	88 131	25.8 167	15.30 389	15.70 399	0.540 13.7	0.540 13.7	7.52 2.29	1110 46201	145 2376	161 2638.3	6.56 16.7	349 14526	44.5 729	68.2 1117.6	3.68 9.35
	101 150	29.9 193	15.50 394	15.80 401	0.625 15.9	0.625 15.9	7.56 2.30	1300 54110	168 2753	187 3064.4	6.59 16.7	412 17149	52.2 855	80.1 1312.6	3.71 9.42
	121 180	35.8 231	15.80 401	15.90 404	0.750 19.1	0.750 19.1	7.62 2.32	1590 66180	201 3294	226 3703.5	6.66 16.9	504 20978	63.4 1039	97.6 1599.4	3.75 9.53
	141 210	41.7 269	16.00 406	16.00 406	0.875 22.2	0.875 22.2	7.69 2.34	1870 77835	234 3835	264 4326.2	6.70 17.0	599 24932	74.9 1227	116 1900.9	3.79 9.63
	162 241	47.7 308	16.30 414	16.10 409	1.000 25.4	1.000 25.4	7.75 2.36	2190 91154	269 4408	306 5014.4	6.78 17.2	697 29011	86.6 1419	134 2195.9	3.82 9.70
	183 272	54.1 349	16.50 419	16.30 414	1.130 28.7	1.130 28.7	7.81 2.38	2510 104473	304 4982	349 5719.1	6.81 17.3	818 34047	100.0 1639	156 2556.4	3.89 9.88
HP 18 HP 460	135 201	39.9 257	17.50 445	17.80 452	0.750 19.1	0.750 19.1	8.54 2.60	2200 91570	251 4113	281 4604.7	7.43 18.9	706 29386	79.3 1299	122 1999.2	4.21 10.7
	157 234	46.2 298	17.70 450	17.90 455	0.870 22.1	0.870 22.1	8.60 2.62	2570 106971	290 4752	327 5358.5	7.46 18.9	833 34672	93.1 1526	143 2343.3	4.25 10.8
	181 269	53.2 343	18.00 457	18.00 457	1.000 25.4	1.000 25.4	8.66 2.64	3020 125701	336 5506	379 6210.7	7.53 19.1	974 40541	108.0 1770	167 2736.6	4.28 10.9
	204 304	60.2 388	18.30 465	18.10 460	1.130 28.7	1.130 28.7	8.73 2.66	3480 144847	380 6227	433 7095.6	7.60 19.3	1120 46618	124.0 2032	191 3129.9	4.31 11.0

Technical Hotline: 1-866-875-9546 | engineering@skylinesteel.com www.skylinesteel.com skylinesteel a NUCOR company

HP

Steel H-Pile

Available Steel Grades								
AMERICAN			CANADIAN			EUROPEAN**		
ASTM	YIELD STRENGTH		CSA G40.21	YIELD STRENGTH		EN 10034	YIELD STRENGTH	
	(ksi)	(MPa)		(ksi)	(MPa)		(ksi)	(MPa)
A 36	36	250	Grade 300 W	44	300	HISTAR 355	51	355
A 572 Grade 50*	50	345	Grade 350 W	50	350	HISTAR 420	61	420
A 588	50	345				HISTAR 460	67	460
A 690	50	345						

* Standard grade for H-Pile.

**HISTAR only available in some sizes.

Splicer and H-Pile Point

Splicer

H-Pile Point

Delivery Conditions & Tolerances

	ASTM A 6	
Mass	± 2.5%	
Length§		
30 Feet and Under	± 0.375 inches	
Over 30 Feet	+ (0.375 inches + (length - 30)/80)	– 0.375 inches
Depth	± 0.125 inches	– 0.1875 inches
Flange Width	+ 0.25 inches	
Flanges out of Square		
HP 8 x 42 - HP 12 x 84	≤ 0.25 inches	
HP 14 x 73 - HP 14 x 117	≤ 0.3125 inches	
Web off Center	≤ 0.1875 inches	
Greatest Depth over Theoretical	≤ 0.25 inches	
Camber and Sweep***		
45 Feet and Under	(0.125")(Length in feet/10) but not over 0.375"	
Over 45 Feet	(0.375") + (0.125" (Length in feet - 45)/10)	

§ For HP ordered as bearing piles, length tolerances are +5 in. and -0 in.

***For the HP 10 x 42, 12 x 53, 12 x 63, 14 x 73, and 14 x 89 ordered as columns, tolerances are subject to negotiation with manufacturer.

Maximum Rolled Lengths'

HPs	100 '	30.5 m

' Longer lengths may be possible upon request.

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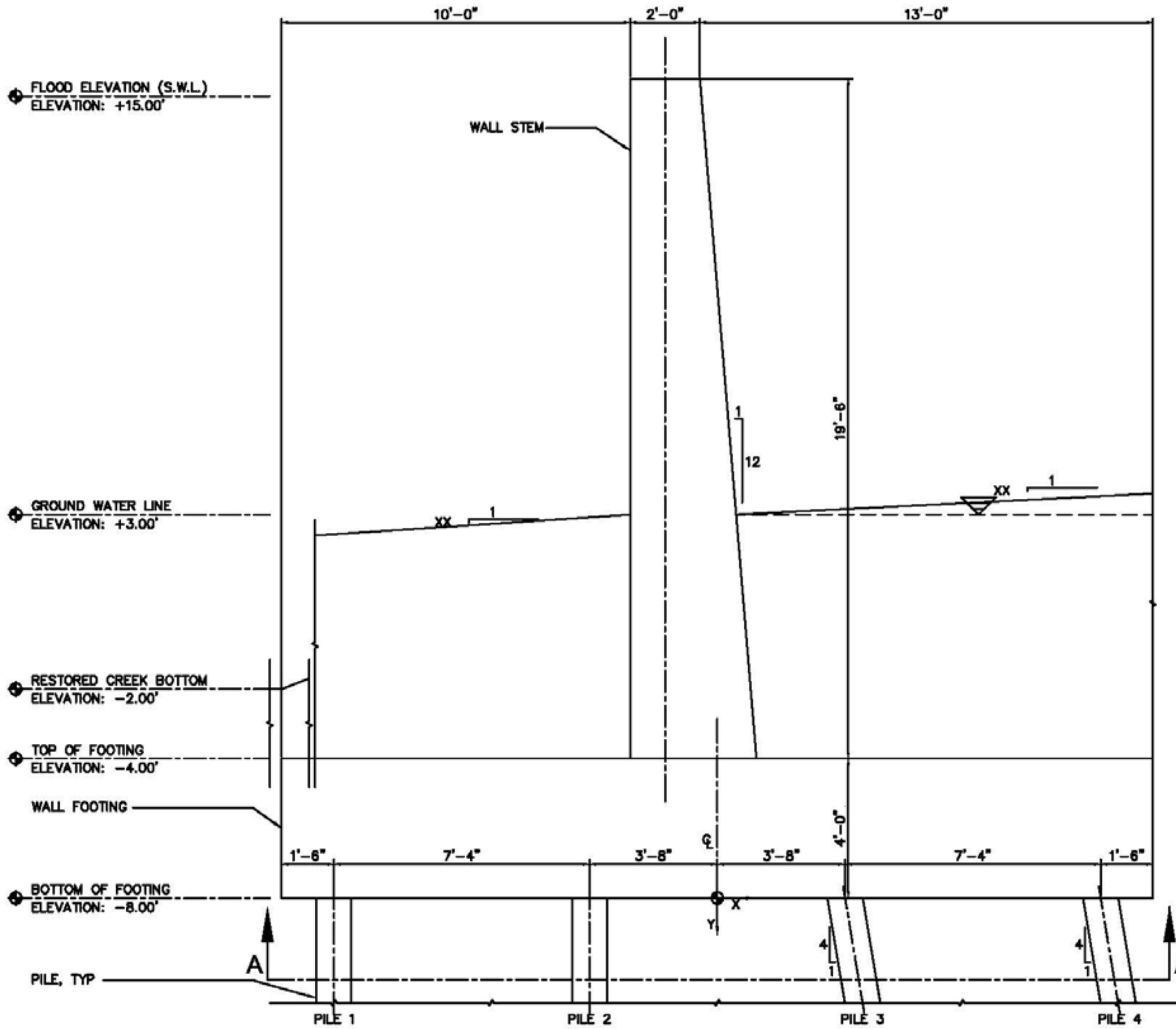
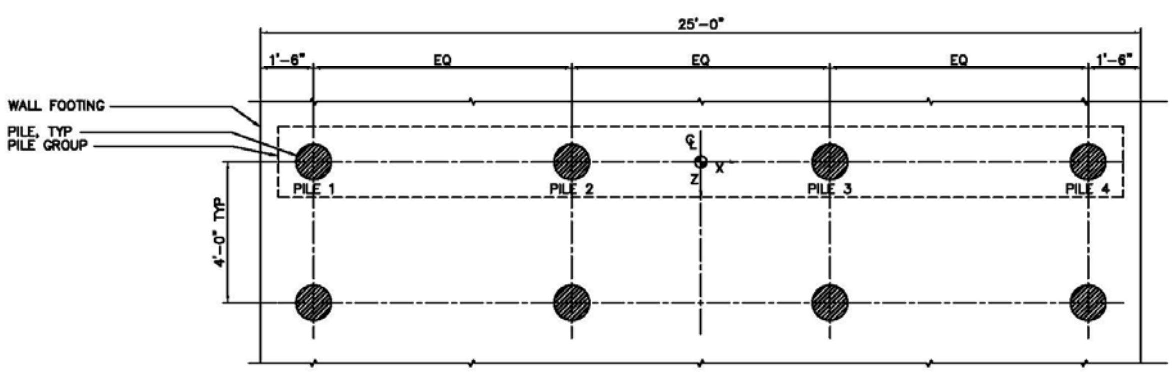


TYPICAL T-TYPE WALL SECTION - FOUR PILE ARRANGEMENT



VIEW A-A - 25 FOOT FOUR PILE ARRANGEMENT

HURRICANE SANDY LIMITED REEVALUATION REPORT
UNION BEACH, NEW JERSEY

DRAFT ENGINEERING APPENDIX

SUB APPENDIX C

SEA LEVEL RISE ANALYSIS

Rev. 18
Feb 2015

1.0 Introduction

1.1 Guidance

The Department of the Army Engineering Circular **ER1100-2-8162 (31 Dec 2013)** requires that future sea level rise (SLR) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The project team should evaluate structural and non-structural components of the proposed alternatives in consideration of the "low," "intermediate" and "high" potential rates of future SLR for both "with" and "without project" conditions. This range of potential rates of SLR is based on findings by the National Research Council (NRC, 1987) and the Intergovernmental Panel for Climate Change (IPCC, 2007).

1.2 Components of Sea Level Rise

SLR considers the effects of (1) the *eustatic*, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the "regional" rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth's crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids.

Union Beach, New Jersey is located in an area that experiences positive land subsistence due to geological processes; therefore, the net relative sea level rise at Union Beach is greater than the eustatic SLR. Said differently, when land in Union Beach subsides as water surface elevation increases, the net local SLR is greater in Union Beach than at a location experiencing an increase in water surface elevation only.

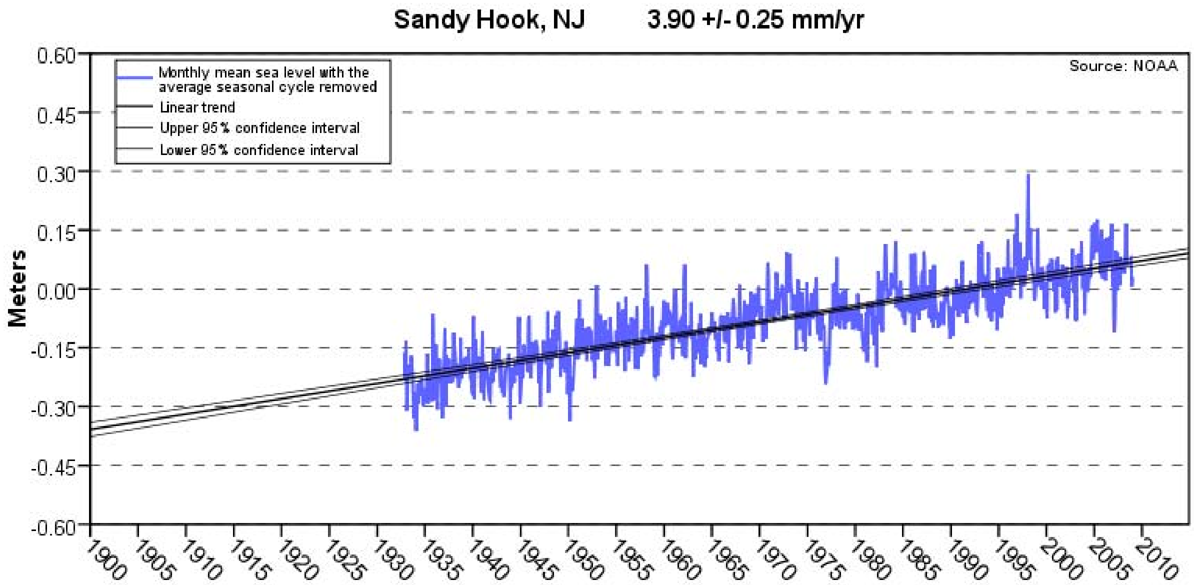
2.0 Rates of Sea Level Rise

When calculating the intermediate and high rates of sea level rise, the local rate of VLM must first be determined. An example calculation for Sandy Hook is provided in Section 2.3.1, Determining Local VLM.

2.1 Historic Rate of Sea Level Rise

The historic rate of future sea-level rise is determined directly from gauge data gathered in the vicinity of the project area. The nearest NOAA tide gauges from which tide data can be evaluated include: The Battery and Montauk Point gauges in New York, and the Sandy Hook gauge in New Jersey. Of these three locations, tide conditions at Sandy Hook (NOAA Station #8531680) best represent the conditions experienced in Union Beach. A 75-year record (1932 to 2006) of tide data gathered at Sandy Hook, NJ indicates a mean sea level trend (eustatic SLR + the local rate of VLM) of <u>+3.9 mm/year</u>.

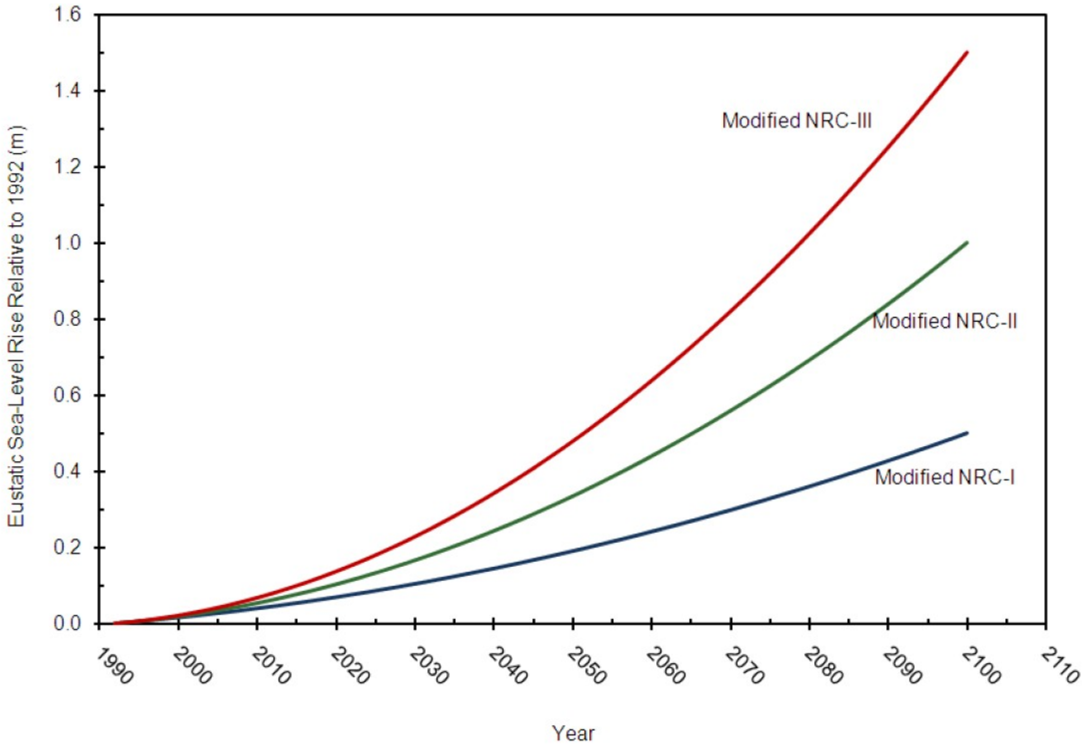
Figure 1: Mean Sea Level Trend at Sandy Hook, NJ (NOAA Station # 8531680)



2.2 Intermediate Rate of Sea Level Rise

The intermediate rate of local mean SLR is estimated by considering the modified NRC projections and adding the appropriate value to the local rate of vertical land movement. The intermediate rate of local sea level rise is based on the modified NRC Curve I since its value is comparable to that of the IPCC projection.

Figure 2: Modified NRC curves for predicting future rates of eustatic SLR.



NRC Curve I is based on the general equation $E(t) = 0.0017t + bt^2$, where

the constant 0.0017 = the IPCC 2007 annual rate of eustatic SLR in meters;

t = time in years (relative to the year 1986 when the curves were developed) and;

$b = 2.71E^{-5}$

2.3 High Rate of SLR

The high rate of local mean SLR is estimated by determining the modified NRC Curve III value and adding it to the local rate of vertical land movement. This high rate scenario exceeds the 2001 and 2007 IPCC projections and considers the potential rapid loss of ice from Antarctica and Greenland.

NRC Curve III is also based on the general equation $E(t) = 0.0017t + bt^2$; however, the constant b changes to $b = 1.13E^{-4}$.

For both the intermediate and high rates of SLR, the NRC curves accelerate upward over time beginning in the year 1992 when the curves were developed; therefore, it is necessary to estimate SLR for a particular time horizon relative to 1992.

2.3.1 Determining Local VLM

The local rate of VLM, which is considered to be constant through time, is determined by subtracting the NRC/IPCC eustatic SLR value (1.7 mm/yr) from the local mean sea level trend. Recall from Section 2.1 above that the two components figuring into the local mean sea level include the eustatic SLR value and the local rate of VLM. The mean rate of SLR at the Sandy Hook station is +3.9 mm/year (7.7 inches in 50 years).

The local rate of VLM at Sandy Hook is calculated from the relationship:

$VLM_{\text{Sandy Hook}}$ = [local rate of SLR] – [eustatic rate of SLR], or

$VLM_{\text{Sandy Hook}}$ = 3.9 mm/yr – 1.7 mm/yr = 2.2 mm/yr (0.087 in/yr)

At Sandy Hook, the local rate of VLM accounts for a total of 4.35 inches (0.087 in/yr x 50 yrs) at a 50-year time horizon.

This local rate of VLM is added back into the sea level rise computations after the eustatic portion has been determined from NRC curves I and III.

3.0 Calculating Sea Level Rise

3.0.1 Historic Rate

The historic rate of sea level rise is determined by extrapolating the mean local SLR trend and multiplying it by the desired time horizon. The local SLR trend at Sandy Hook is 3.90 mm/yr.

Based on the historic rate of SLR it can be expected that sea level will rise 7.67 inches over a 50-year time horizon.

3.0.2 Intermediate Rate

The intermediate rate of sea level rise is computed using the equation

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2) + \text{local VLM}$$

where t_1 and t_2 represent the start and end dates of the projected time horizon in years, relative to 1992.

$$SLR = ((0.0017 \text{ m/yr } (69-19)\text{yr} + .0000271\text{m/yr } (69^2-19^2)\text{yr}) \times (3.281 \text{ ft/m}))$$

$$+ (0.087 \text{ in/yr} \times 50 \text{ yrs})/12 \text{ in/ft}$$

$$= \underline{1.03 \text{ ft}}$$

3.0.3 High Rate

The high rate of sea level rise is computed using the equation

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2) + \text{local VLM}$$

4.0 Projected Water Surface Elevation Increases

The Union Beach project design water level stages were derived from FEMA modeling efforts in 2013. Using the base year 2013 from which future sea level elevations are estimated, Table 1 shows the projected increase in water surface elevation for the historic, intermediate and high rates of future sea level rise at Union Beach, New Jersey to the year 2100.

Table 1: Increase in predicted water surface elevations at Union Beach, NJ under the historic, intermediate and high rates of future sea level rise (from sea level rise base year 2013)

USACE Curves computed using criteria in USACE ER1100-2-8162

Gauge NJ, Sandy Hook: 75 yrs All values are in feet			
Year	USACE Low	USACE Int	USACE High
2013	0.27	0.31	0.44
2018	0.34	0.40	0.59
2023	0.40	0.49	0.76
2028	0.47	0.58	0.95
2033	0.53	0.68	1.16
2038	0.60	0.79	1.38
2043	0.66	0.90	1.63
2048	0.73	1.01	1.89
2053	0.79	1.13	2.17
2058	0.86	1.25	2.47
2063	0.92	1.37	2.79
2068	0.99	1.50	3.13
2073	1.06	1.64	3.49
2078	1.12	1.78	3.86
2083	1.19	1.92	4.26
2088	1.25	2.07	4.67
2093	1.32	2.22	5.10
2098	1.38	2.38	5.55
2100	1.41	2.44	5.73>

5.0 Adaptation of the Shore Protection Beach Component of the Recommended Plan for Increased Sea Levels

The Union Beach Project consists of components that are adaptable to future increases in sea level due to climate change. The sand dune and berm cross section could include increases in dune crest height, and corresponding increase in berm elevation to compensate for increasing still water levels. The levee and wall systems could also be modified with parapet walls or additional wave baffles pending design analyses to support additional height. If applicable, additional pump station capacity could be added to handle additional overtopping. However, for this authorized project, a post-authorization change report would be required to make these changes. Regular renourishment operations are part of the Authorized Project.

HURRICANE SANDY LIMITED REEVALUATION REPORT
UNION BEACH, NEW JERSEY

DRAFT ENGINEERING APPENDIX

SUB APPENDIX D

SBEACH MODELING

Rev. 18 Feb 2015

SBEACH Modeling

1.0 Introduction

Following the methodology described in the Coastal Engineering Manual (CEM, Part 5, Chapter 4), the procedure applied in this project has been to verify the target beach profile along the shoreline that would provide an appropriate level of erosion, flooding, and storm damage reduction to the structures at Union Beach, New Jersey as well as augment this profile with sufficient advanced nourishment so that, at a minimum, the target profile would be maintained. The computer model SBEACH (Storm-induced BEAch CHange) was applied to relate profile characteristics to levels of protection from storm damage. All SBEACH modeling work was performed on PCs using the CEDAS (Coastal Engineering Design and Analysis System, version 4.03) package of models.

SBEACH setup

1.1 Model description and approach

SBEACH is an empirically based numerical model for simulating two-dimensional cross-shore beach change. The model's intended purpose is for predicting short-term profile response to storms. A fundamental assumption of SBEACH is that profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no lateral gain or loss of material by longshore transport.

When a storm erodes a beach, the sand is usually not lost from the system. Rather, it is moved offshore, frequently into one or more bars. Low wave conditions after the storm will slowly move this material back onshore, rebuilding the berm. The discussion in this chapter addresses the question of how much sand must be placed in a berm and dune to provide adequate protection from storms.

Prior to running the model, input data in the form of representative nourished beach profiles and time series of storm waves and water levels were developed. Other input data included sediment grain size and default model (reach and storm) configuration parameters. The primary SBEACH output was a final (post-storm) profile for the input profile for hypothetical storm variants. Analysis of the final profile allowed a determination of whether the design profile was sufficient to withstand the simulated storm events.

1.2 Storm events

The selection of an appropriate storm event involved the water levels associated with a 1 Percent Exceedence Event (100-Year) and the waves developed from the recorded wave data from Super Storm Sandy. This approach was considered appropriate since the purpose of this analysis was to determine the adequacy of the authorized design dune and berm profile to withstand a 1 Percent Exceedence Event.

1.3 Characterization of storm water levels

The storm water levels can be considered by two measures. These are the maximum water level expected and the appropriate storm hydrograph. The appropriate maximum water level was determined to be the 1 Percent Exceedence (100 Year) water level.

Draft FEMA modeling total water surface elevations which include wave setup at Latitude 40.47454 and Longitude -74.16823 (referred to as location 422529) were selected for use in this HSLRR. This location is shown in Figure 1. (This location is the same location used in the Union Beach Feasibility Study, and is referred to as Node P2.) The FEMA water surface elevation frequency values at Node P2 are shown in Table 1 and Figure 2. A direct comparison cannot be made between the Feasibility Study stage and the FEMA stage as the FEMA stage includes wave setup, and the Feasibility Study doesn't.

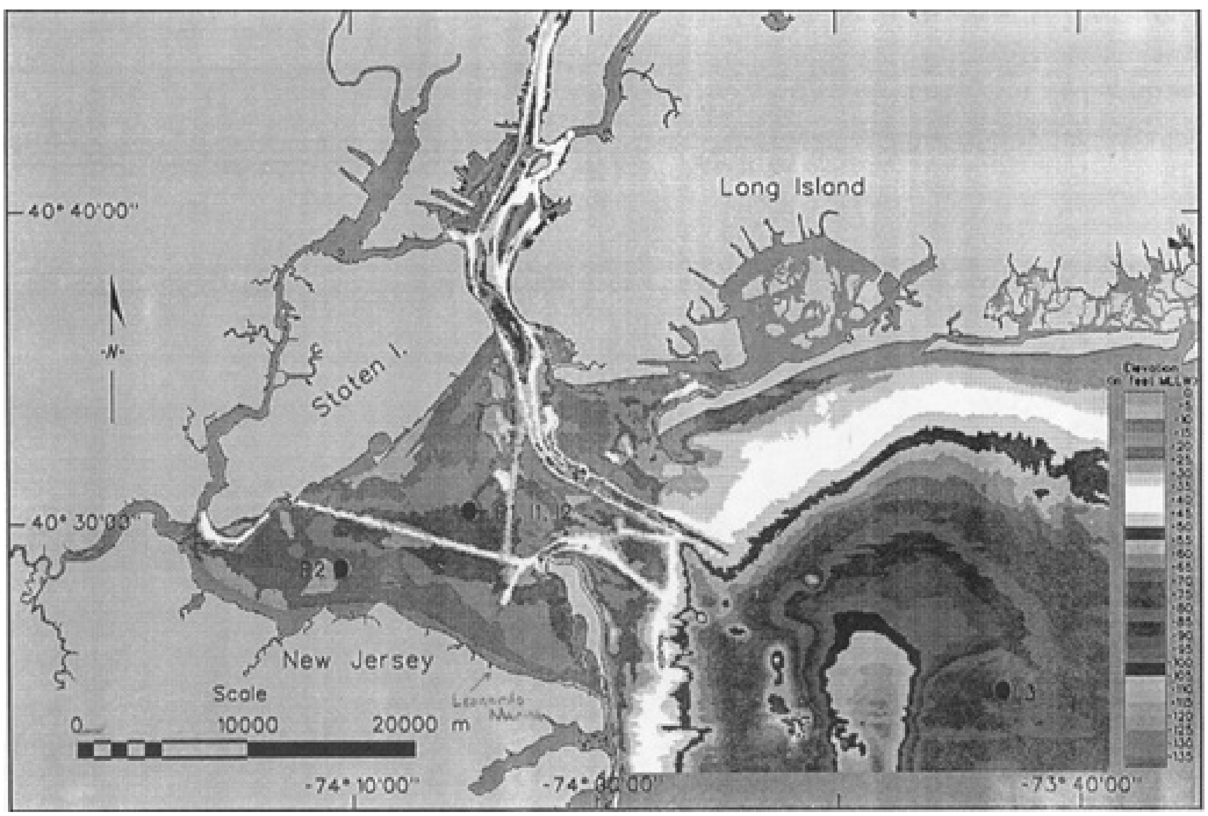


Figure 1 Node Location

Table 1: P2 Stage Elevations in ft. NGVD		
Return Period in years	1998 Stage Elevation w/o Wave Setup at Node P2 in ft. NGVD	2013 FEMA Stage Elevation with Wave Setup at Node 422529 in ft. NGVD
2	5.7	
5	8.0	7.5
10	9.2	8.9
25	10.6	10.6
50	11.5	11.9
100	12.2	13.3
200	13.0	14.7
500	13.9	16.6

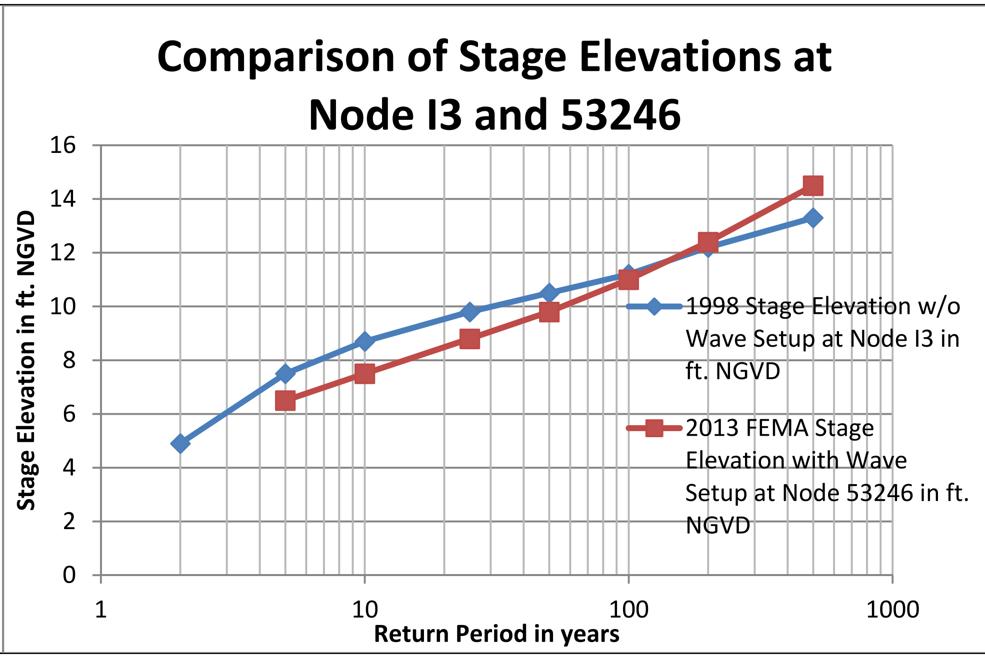
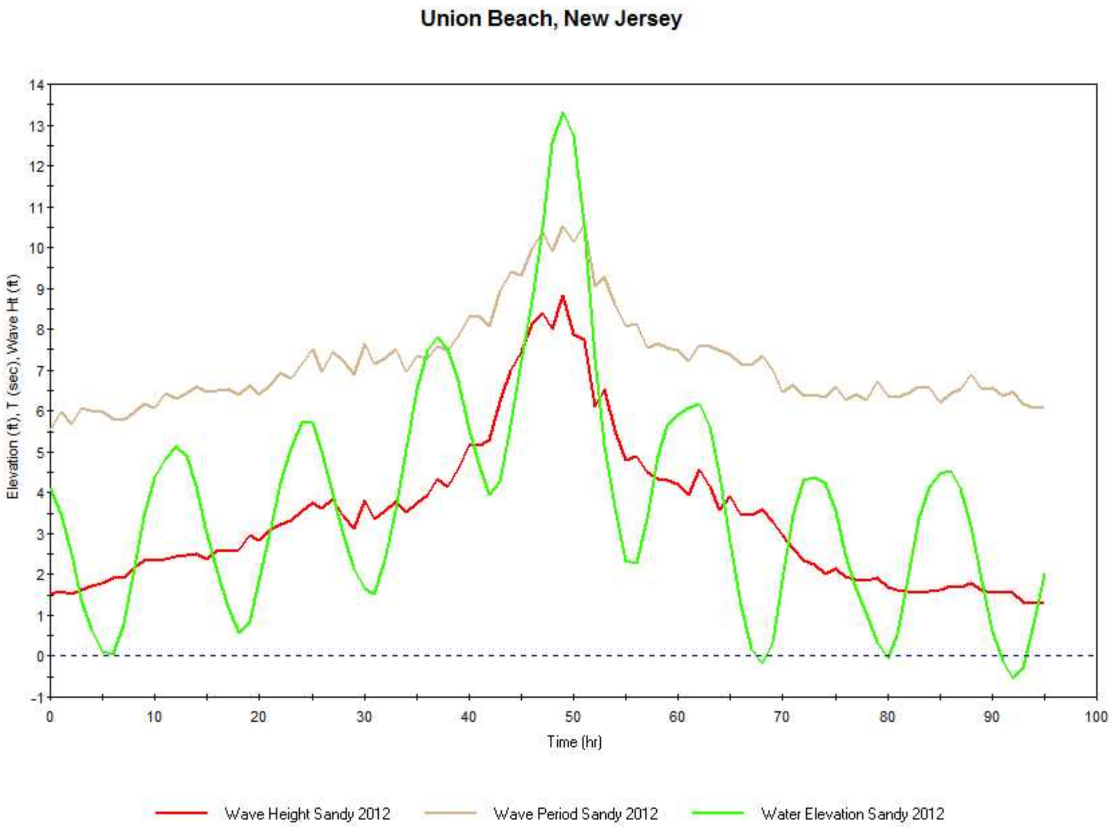


Figure 2

A review of the Hurricane Sandy tidal records at Sandy Hook, NJ and The Battery, NY found there was excellent correlation between the two hydrographs up until the point at the peak of the storm when the Sandy Hook gage stopped recording. Based on this it was considered appropriate to use the shape of the storm hydrograph at The Battery in the SBEACH Modeling.

The maximum water level at The Battery gage was slightly more than 1 foot less than the maximum water level discussed earlier in this section. The decision was made to prorate The Battery water levels during the significant portion of the hydrograph so the maximum elevation matched the 1 Percent Exceedence level.

A plot of the adopted storm hydrograph is shown below.



Note: All Elevations NGVD

1.4 Characterization of storm waves

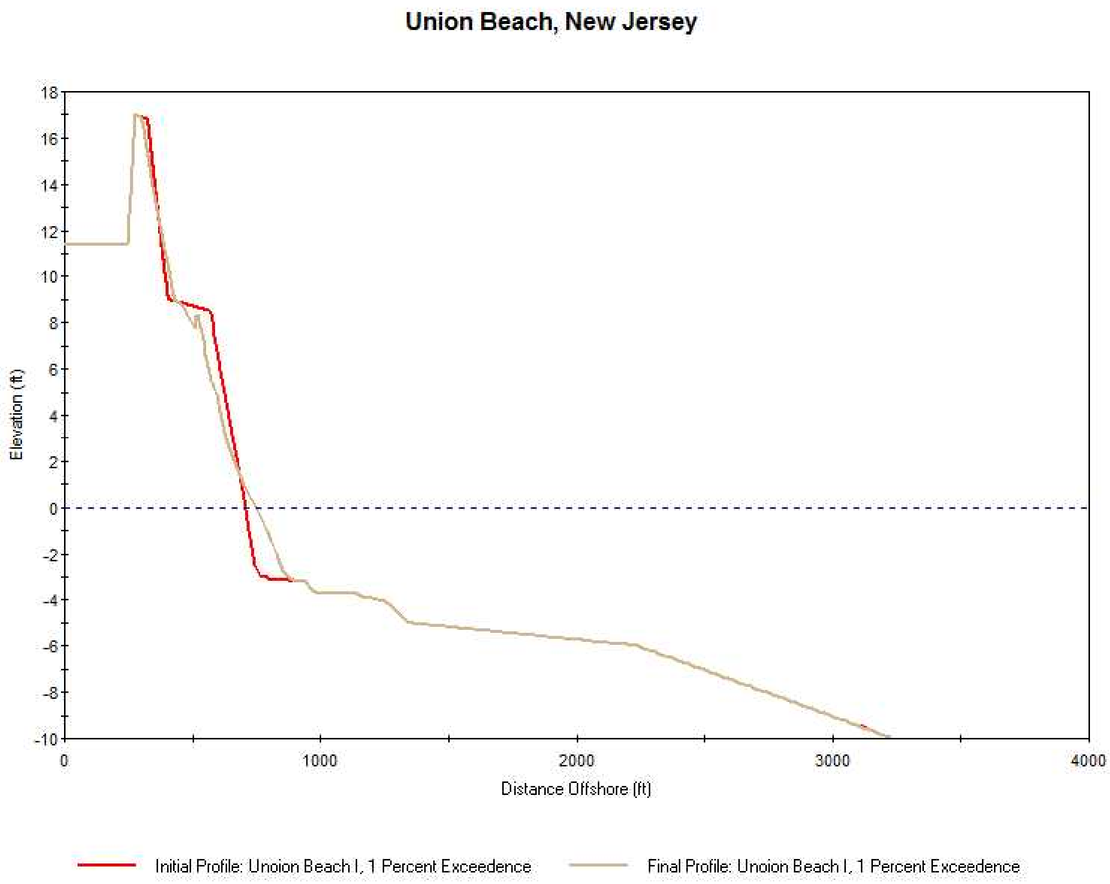
*Wave gage data could not be found in the v*icinity of the project with the exception of a limited data set from a temporary gage deployed prior to Hurricane Sandy landfall. Wave data for the model runs used surrogate data from the storm wave data that were available from the NOAA buoy station 44065. Wave data from the gage were 'ratioed' to match storms having similar maximum water levels as described in the 2003 Feasibility Study resulting in a peak wave height of 8.4 feet.

1.5 Characterization of the beach profile

There is virtually no exposed beach along the eastern half of the shoreline. This is a 'no dry beach width' section of the Union Beach's coastline, which has a non-engineered revetment. The western half of the shoreline currently has an average or approximately 75 feet of dry beach. In comparing profiles from the 2003 feasibility study (primary comparison parameters were berm height, foreshore beach slope, sub-aerial profile volume, and subaqueous profile shape), it was determined that a single idealized profile could represent the nourished profile along the length of the project, since modeling the non-revetment profile would provide a more conservative result given the uncertainties of the revetment design.

The beach profiles were analyzed along with initial SBEACH modeling results to develop the idealized “potential” nourished profile that was used in the SBEACH modeling effort as shown below. All profile elevations are referenced to NGVD 29. The rock revetment (non-eroding surface) was not modeled.

For the idealized profiles, the upland elevation (based upon profile data) is set at 11.4 ft. The dune elevation and width of 17 ft and 50 ft. respectively as well as the berm dimensions of are based upon modeling results in the Feasibility Report. The landward dune slope (1V:5H) and seaward slope of (1V:10H) was determined by the 2003 Feasibility Study storm erosion modeling to be the most stable slopes. The berm height (+9 ft) and foreshore slope (1V:15H) were also deemed appropriate for this analysis. Profile data from September 1997 long profile surveys and NOAA coastal charts were used to depict the offshore bathometry.



1.6 SBEACH model runs

The SBEACH model was not calibrated for the Union Beach project site prior to data runs being made, because the appropriate pre- and post-storm profiles were not available for the site. The SBEACH parameters utilized in the simulations are shown in the following table.

Where no site specific information was available the default model parameters were selected.

Table of SBEACH Parameters

Landward Surf Zone Depth	1.6
Effective Grain Size	0.29 mm
Maximim Slope Prior to Avalanching	30
Sand Remains on Grid	Yes
Transport Rate Coefficent	1.5e-006 (m^4/N)
Overwash Transport Parameter	0.005
Coefficent for Slope-Dependent Term	0.002 (m^2/S)
Transport Rate Decay Coefficent Multiplier	0.5
Water Temperature (Degrees C)	16

1.7 SBEACH Results

As illustrated in the previous graph the design dune and berm template provides sufficient elevation and volume of material to withstand a simulated 1 Percent Exceedence storm event.

1.8 Sea Level Rise Analysis

A sensitivity analysis was performed for the design dune and berm template utilizing the Sea Level Rise scenarios discussed in the Sea Level Change portion of the Appendix. The three scenarios can be characterized as low, medium and high. Adjustments to the 1 Percent Exceedence storm water levels for anticipated Sea Level Rise in the year 2030 and the year 2069 to assess the performance of the design dune and berm template under these increased water levels. A summary of those simulations is discussed in the following paragraphs.

In the 2030 simulations for both the low and intermediate scenarios the dune crest was reduced in with but maintained the design crest elevation. Under the high scenario, the dune crest elevation was reduced by approximately 2 feet. In all three cases the berm was eroded landward approximately 40 feet. Under all three scenarios no significant levels of inundation would be experienced behind the primary dune line based on the simulation results.

In the 2069 simulations for the low scenario the dune crest elevation was reduced by approximately 1.3 feet. The medium and high scenarios reduced the dune crest elevation by approximately 2.5 feet. In all three cases the berm was eroded back to approximately 40 feet in front of the original position of the toe of the design berm. Under the medium and high scenarios significant levels of inundation would be experienced behind the primary dune line.

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SUB APPENDIX E
OVERTOPPING & FAILURE ANALYSIS

Revised 18 Feb 2015

OVERTOPPING & FAILURE ANALYSIS

1.0 Introduction

Overtopping was analyzed using 2013 FEMA Stage Elevations. The overtopping analysis, for the designed vertical wall and levee, used the interactive computer-based design and analysis system, ACES (which is based on equations found in the Corps' of Engineers Coastal Engineering Manual (CEM)) and the online version of EurOtop (which is based off equations that can be found in the EurOtop Overtopping Manual) for comparison. The analysis included the development of peak overtopping rates for various idealized return periods (2 year, 5 year, 10 year, 25 year, 50 year, 100 year, 200 year, and 500 year). ACES and EurOtop used both the Probabilistic and Deterministic approach to analyze the overtopping for the design of the vertical wall and levee. Overtopping Analysis of the Dune and Beach areas will be explained briefly in this sub-appendix. More information of the Overtopping Analysis of the Dune and Beach areas can be read in Sub-Appendix D SBEACH. The project conditions utilized the vertical wall, levee, and dune/beach dimensions and elevations formulated in the previous Feasibility Report and were referenced to NGVD 29 vertical datum.

In addition, the Union Beach Exceedance Interval was analyzed from Waves Overtopping Levees using HEC FDA at 90% Confidence. The risk-reducing capability of the Union Beach project during hurricanes and northeasters is dependent upon the bay-fronting levees and floodwalls ability to resist against wave overtopping and still water overtopping flowrate forces. Wave breaking may result in water splashing over the crest onto the landward side of the protection when the still water surface elevation is lower that the crest elevation of the levee or floodwall. These wave flowrates have the potential of causing scour and possibly failure of the protective ability of the feature. Still water overtopping occurs when the still water surface elevation exceeds the crest elevation and water simply flows over the crest. The vertical distance between the elevation of the still water surface and the crest elevation of the protection feature is called freeboard; large freeboard results in smaller overtopping flowrates than small freeboard. The elevation of the structures evaluated is +15 ft. NGVD.

2.0 Probabilistic Approach versus Deterministic Approach

As mentioned in the introduction, ACES and EurOtop used both the Probabilistic and Deterministic approach to analyze the overtopping for the design of the vertical wall and levee. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties. Imposed forces, as well as the strengths and interactions of the various components are usually not clearly understood and the design process itself is ill defined. In the past, designs were strictly based on deterministic expressions. A deterministic model is one in which every set of variable states is uniquely determined by parameters in the model and by sets of previous states of these variables. Therefore, deterministic models perform the same way for a given set of initial conditions. More recently, probabilistic design methods have been introduced, in which randomness is present and variable states are not described by unique values, but rather by probability distributions. Both approaches are typically used and compared.

3.0 <u>Overtopping –Flood Wall Analysis</u>

The overtopping rates were calculated for the project flood wall using the overtopping formulations provided in the EurOtop software and ACES. The equations, formulations used, and results from each method are shown and explained throughout this section and sub-sections.

3.1 <u>ACES</u>

ACES is an interactive computer-based design and analysis system in the field of coastal engineering containing six functional areas: wave prediction, wave theory, wave transformation, structural design, wave run-up, and littoral processes. For the purpose of this analysis ACES was used to calculate wave-overtopping for the project flood wall conditions. Below (Figure 3.1) is an image of the ACES interactive interface for solving for wave-overtopping:

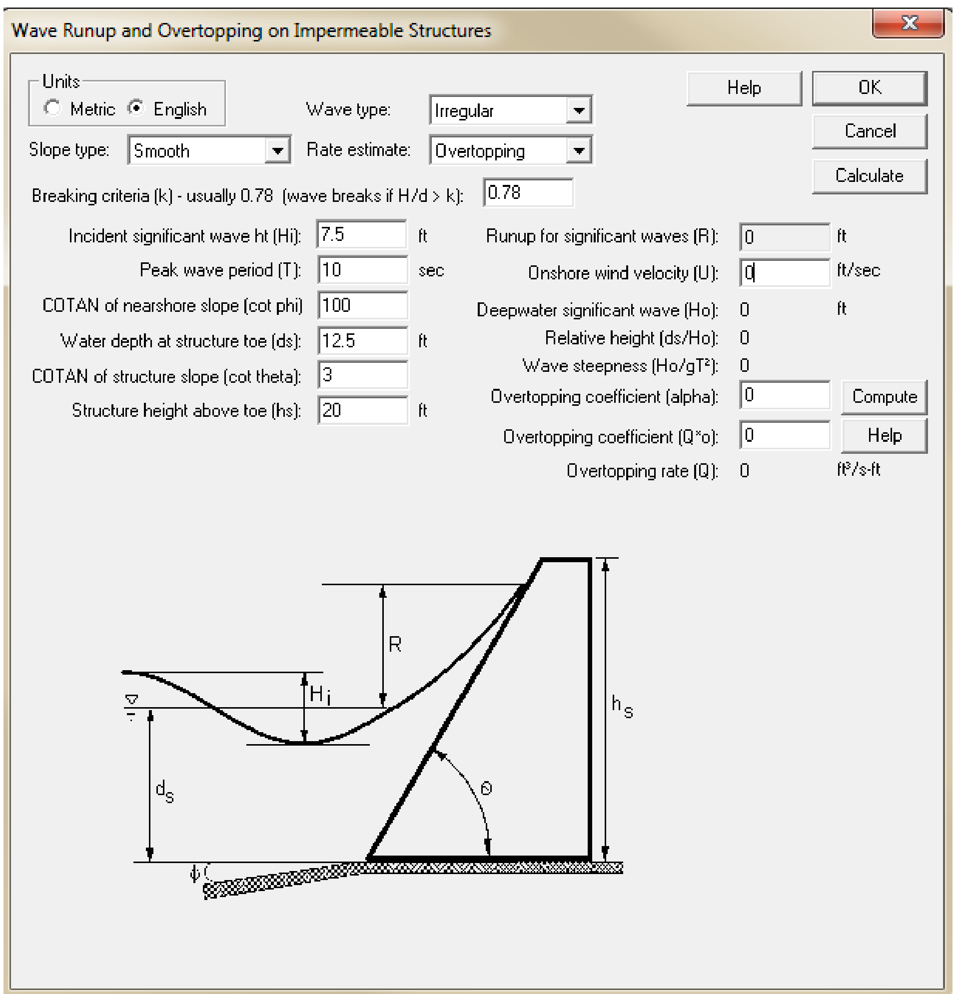

Figure 3.1 – ACES interface

Note: Values shown in the interface were not used in this analysis.

The incident significant wave height (H_i), peak wave period (T), COTAN of nearshore slope (cot phi), water depth at structure toe (d_s), COTAN of structure slope (cot theta), structure height above toe (h_s), and onshore wind velocity (U) are input into the ACES interface. The Overtopping coefficient (alpha) is computed in ACES based off of the COTAN of structure slope value. The Run-up for significant waves (R), Deepwater significant wave (Ho), Relative height (d_s/H_o), and the Wave steepness (H_o/gT^2) are all calculated from the equations programmed in ACES. The Overtopping Coefficient (Q*o) can be found by using Figures 7-24, in the Shore Protection Manual (SPM).

Results are shown in Section 3.4 Individual Model Results of Flood Wall Overtopping Analysis.

3.2 <u>EurOtop</u>

The overtopping rates were calculated for flood wall conditions using the more recent overtopping formulations provided in the online EurOtop software. Below (Figure 3.2) is an image of the EurOtop interactive interface for solving for wave-overtopping for a vertical wall:

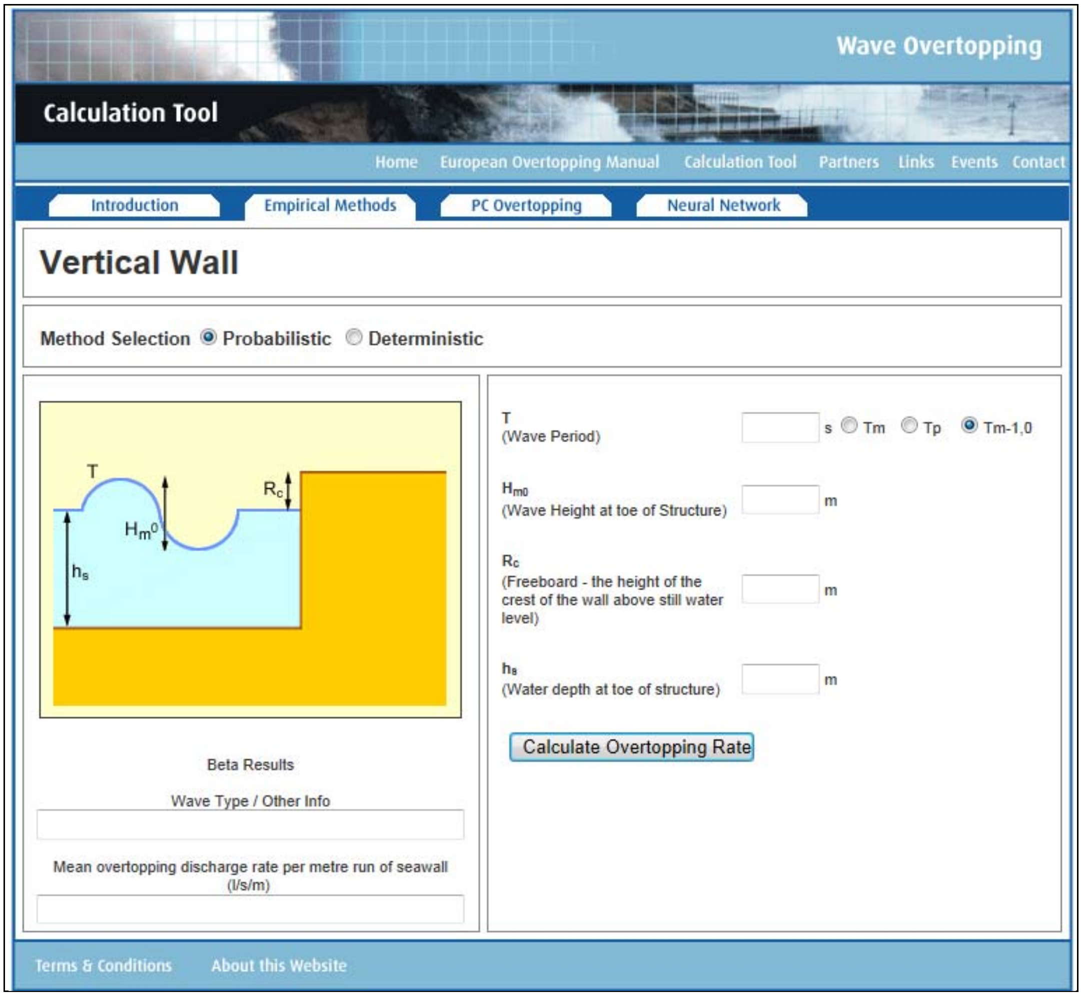

Figure 3.2 – EurOtop interface

The wave period (T), wave height at toe of the structure (H_{mo}), freeboard (R_c) and the water depth at toe of structure (hs) are all input into the calculation tool and the mean overtopping is solved for, as well as the wave type.

A critical determination for the overtopping analysis was to determine if the waves breaking against the wall were Non-Impulsive or Impulsive. This criteria was important in deciding which specific overtopping formula should be used for a vertical seawall overtopping situation. The formula for calculating Non-Impulsive vs. Impulsive waves has been provided below as Figure 3.3, and once again this is a snap shot taken directly from the EurOtop Manual.

This method is for distinguishing between impulsive and non-impulsive conditions at a vertical wall where the toe of the wall is submerged ($h_s > 0$; Figure 7.6). When the toe of the wall is emergent ($h_s < 0$) only broken waves reach the wall.

For submerged toes ($h_s > 0$), a wave breaking or "impulsiveness" parameter, h_* is defined based on depth at the toe of the wall, h_s, and incident wave conditions inshore:

$$h_* = 1.35 \frac{h_s}{H_{m0}} \frac{2\pi h_s}{g T_{m-1,0}^2} \qquad 7.1$$

Non-impulsive (pulsating) conditions dominate at the wall when $h_* > 0.3$, and impulsive conditions occur when $h_* < 0.2$. The transition between conditions for which the overtopping response is dominated by breaking and non-breaking waves lies over $0.2 \leq h_* \leq 0.3$. In this region, overtopping should be predicted for both non-impulsive and impulsive conditions, and the larger value assumed.

Figure 3.3 - Formula for calculating Impulsive vs. Non-Impulsive waves (EurOtop Manual)

Once the wave breaking/overtopping conditions were determined using the above calculation the actual overtopping rate was calculated. For the project area the waves were determined to be impulsive and this was due to the relatively shallow water and breaking wave conditions in front of and at the wall. The formula used for calculating the overtopping rates has been provided below as Figure 3.4 and it was taken as a snapshot from the EurOtop Manual.

Deterministic design or safety assessment, impulsive conditions ($h_* \leq 0.2$): For deterministic design or safety assessment, the following equation incorporates a factor of safety of one standard deviation above the mean prediction:

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 2.8 \times 10^{-4} \left(h_* \frac{R_c}{H_{m0}} \right)^{-3.1} \quad \text{valid over } 0.03 < h_* \frac{R_c}{H_{m0}} < 1.0 \qquad 7.7$$

Figure 3.4 - Overtopping formula for Impulsive wave conditions (EurOtop Manual)

The probabilistic and deterministic methods were both solved for in the EurOtop software. Results are shown in Section 3.4 Individual Model Results of Flood Wall Overtopping Analysis.

3.3 Spreadsheet (check)

An excel spreadsheet (based on the Franco and Franco (1999)) was used as tool to compare with other methods. Below are the equations used to solve for Wave Overtopping. One can see that equations are similar to the ones used in the EurOtop software.

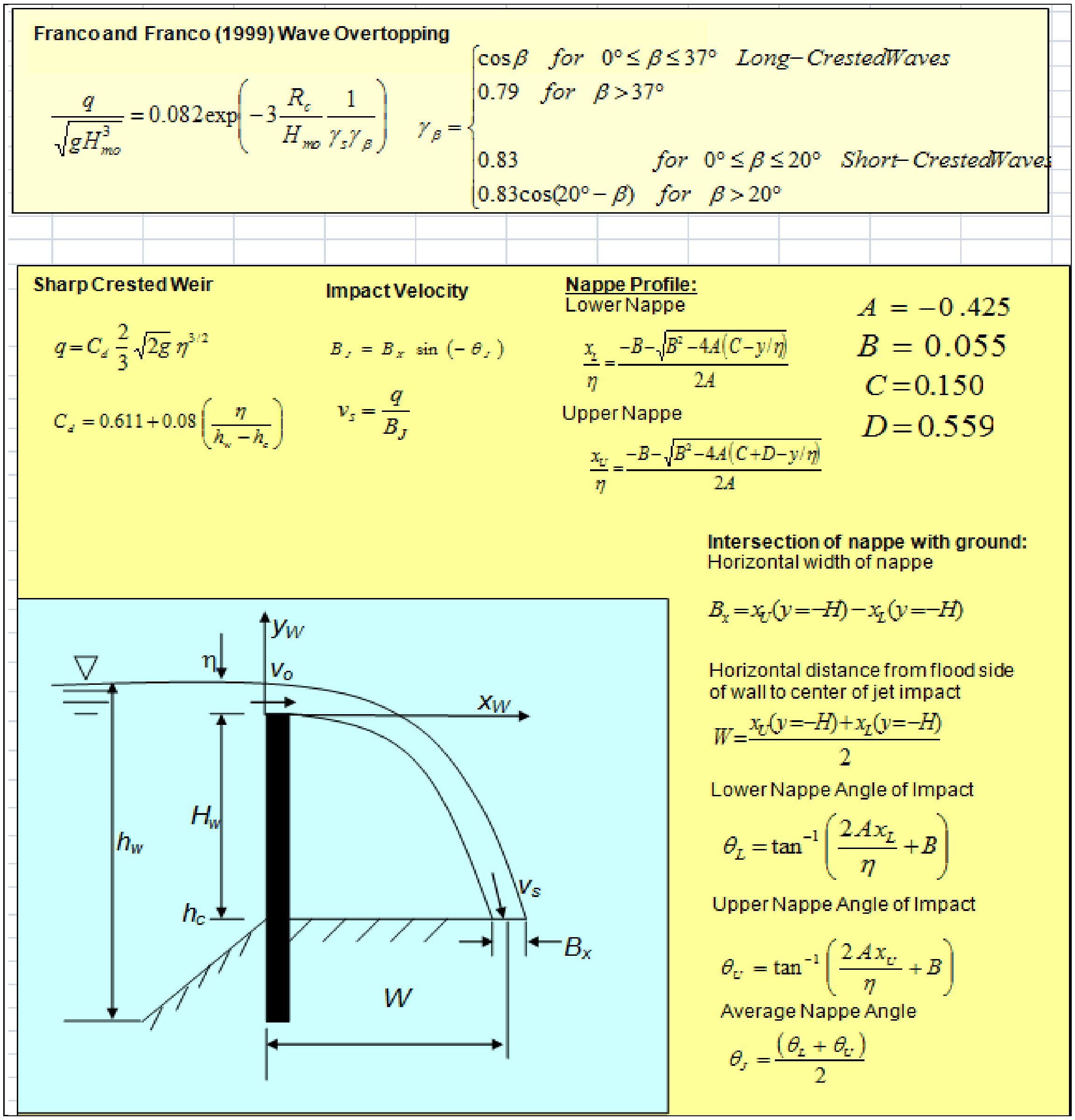

Figure 3.5 –Print Screen of equations used in Spreadsheet for Wall

The probabilistic and deterministic methods were both solved for in the spreadsheet Results are shown in Section 3.4 Individual Model Results of Flood Wall Overtopping Analysis.

3.4– Individual Model Results of Flood Wall Overtopping Analysis

Table 3.1 and Figure 3.6 show a comparison of the results of each method used.

Overtopping of Wall (ft^3/s/ft)						
Return Period (yr)	**Stillwater Level in ft. NGVD**	**ACES**	**EurOTOP - Probabilistic**	**EurOTOP - Deterministic**	**Spreadsheet - Probabilistic**	**Spreadsheet - Deterministic**
2	5.7	0.0000	0.0000	0.0000	0.0000	0.0000
5	7.5	0.0000	0.0000	0.0000	0.0000	0.0000
10	8.9	0.0002	0.0008	0.0066	0.0033	0.0033
25	10.6	0.0196	0.0076	0.0299	0.0325	0.0325
50	11.9	0.5476	0.1001	0.1962	0.4640	0.4640
100	13.3	4.6590	0.6425	0.7727	3.4480	3.4480
200	14.7	Structure Submerged	n/a	n/a	11.0754	11.0754
500	16.6	Structure Submerged	n/a	n/a	26.4825	24.5176

Table 3.1 – Summary of each method (for Wall)

Note that ACES would not solve Overtopping if Stillwater elevation is above the structure. It states that the "Structure is Submerged. Also, EurOTOP would not solve overtopping for the 200 year and 500 year water levels, due to the negative freeboard.

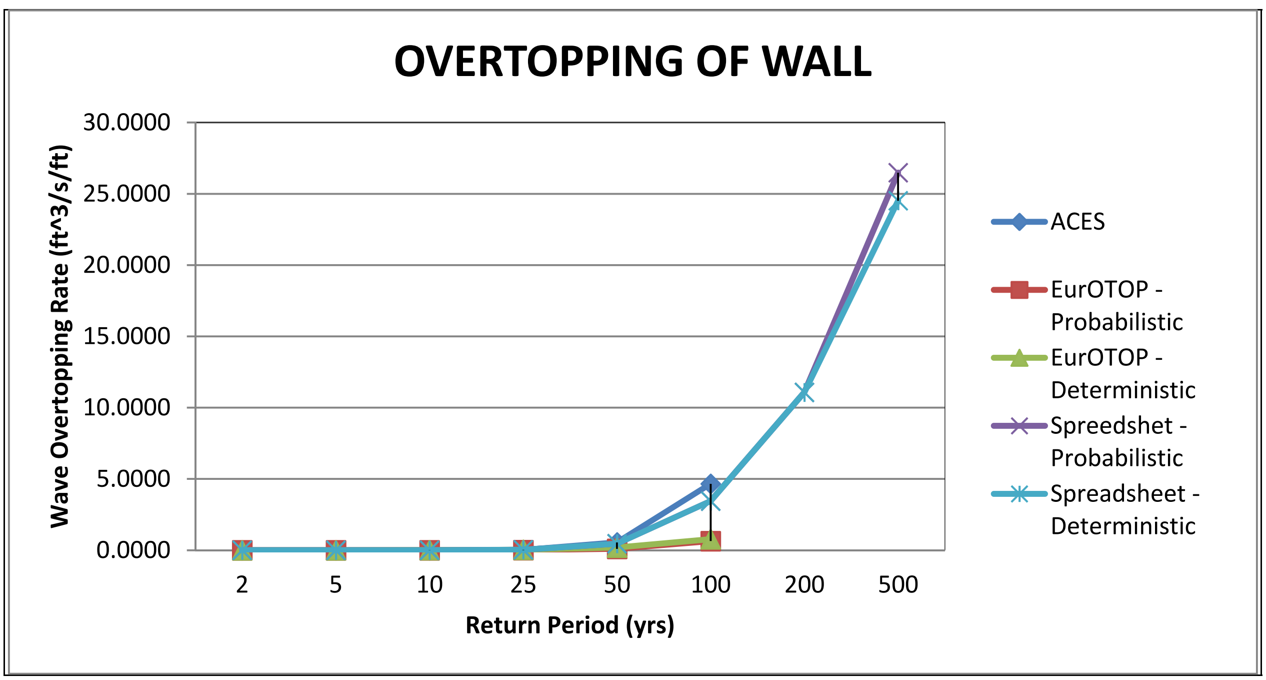

Figure 3.6 – Comparison of each method used (for Wall)

4.0 <u>Overtopping – Levee Analysis</u>

The overtopping rates were calculated for the project levee conditions using the overtopping formulations provided in the EurOtop software and ACES. The equations, formulations used, and results from each method are shown below.

4.1 – <u>ACES</u>

The same analysis that was done for the flood wall was used for the levee; however, the only difference was that the appropriate slope (angle) was used for the levee. For the wall, 90 degrees (or COTAN of one) was used for the structure slope value to represent the slope of a vertical wall. Also, the overtopping coefficient (Q*o) can be found by using Figures 7-25 thru 7-34 (whichever one is appropriate for this project), in the Shore Protection Manual (SPM). Results are shown in Section 4.4 Individual Model Results of Levee Overtopping Analysis.

4.2 – <u>EurOtop</u>

The overtopping rates were calculated for levee conditions using the more recent overtopping formulations provided in the online EurOtop software, which is based off equations that can be found in the EurOtop Overtopping Manual. To simulate a levee the simple slope tool was used.

Below (Figure 4.1) is an image of the EurOtop interactive interface for solving for wave-overtopping for a simple slope (used to simulate a levee):

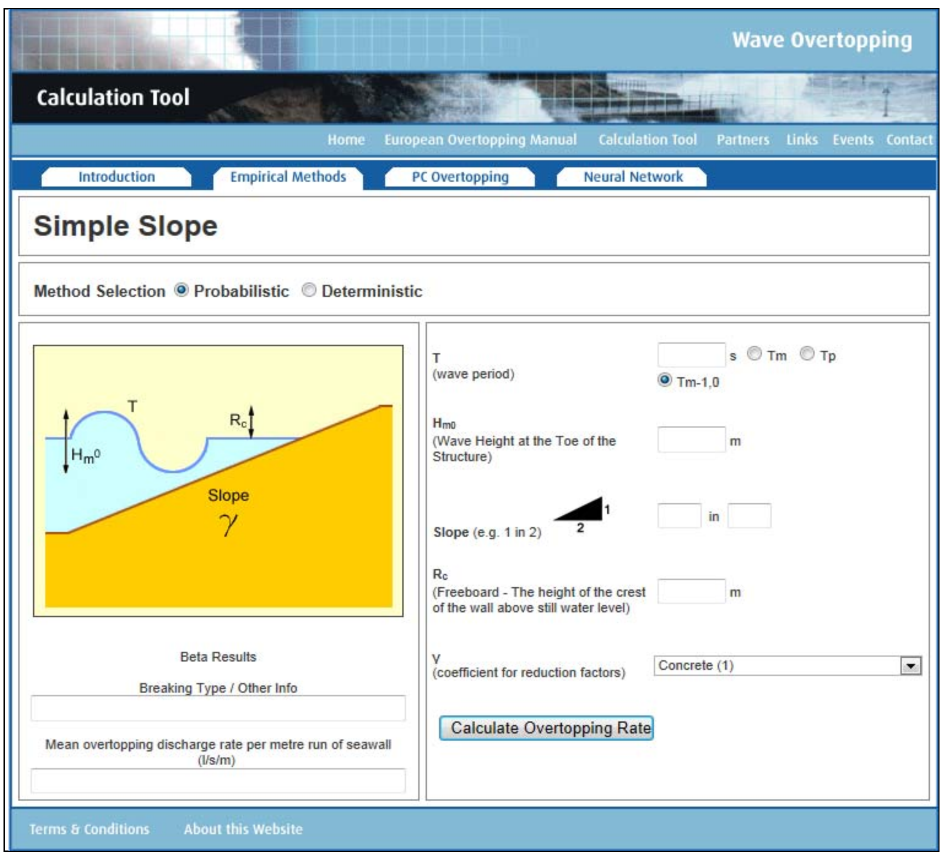

Figure 4.1 – EurOtop interface

The wave period (T), wave height at toe of the structure (H_{mo}), slope, freeboard (R_c) and material the levee will be composed of are all inputted into the calculation tool and the mean overtopping is solved for, as well as the wave breaking type. Results are shown in Section 4.4 Results of Overtopping of Levee Analysis.

4.3 <u>Spreadsheet (check)</u>

The spreadsheet was also used to solve for overtopping of a levee. Below (Figure 4.2) is an image from the spreadsheet.

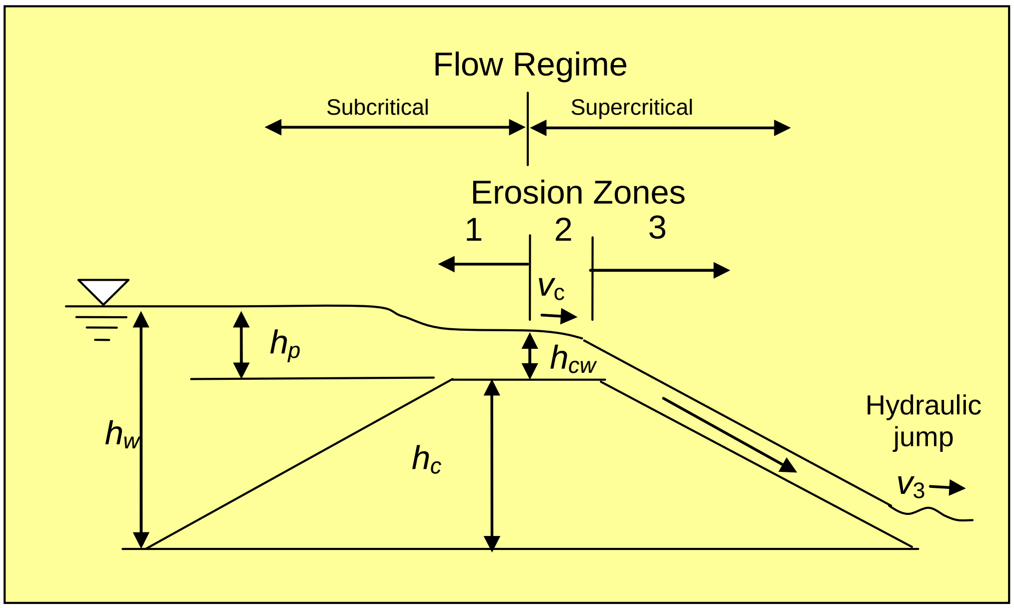

Figure 4.2 – Print Screen of equations used in Spreadsheet for Levee

Results are shown in Section 4.4 Individual Model Results of Levee Overtopping Analysis.

4.4 Individual Model Results of Levee Overtopping Analysis

Table 4.1 and Figure 4.3 show a comparison of the results of each method used.

Overtopping of Levee (ft^3/s/ft)						
Return Period (yr)	**Stillwater Level in ft. NGVD**	**ACES**	**EurOTOP - Probabilistic**	**EurOTOP - Deterministic**	**Spreadsheet - Probabilistic**	**Spreadsheet - Deterministic**
2	5.7	0.000	0.000	0.000	0.000	0.000
5	7.5	0.000	0.000	0.000	0.000	0.000
10	8.9	0.014	0.046	0.072	0.003	0.003
25	10.6	0.180	0.214	0.288	0.033	0.033
50	11.9	1.470	1.295	1.500	0.464	0.464
100	13.3	5.084	5.345	5.561	3.140	3.140
200	14.7	Structure Submerged	12.342	11.862	11.075	11.075
500	16.6	Structure Submerged	39.199	33.769	41.873	38.766

Table 4.1 - Summary of each method (for Levee)

Note that ACES would not solve Overtopping of the levee, if the Stillwater elevation is above the structure. It states that the "Structure is Submerged.

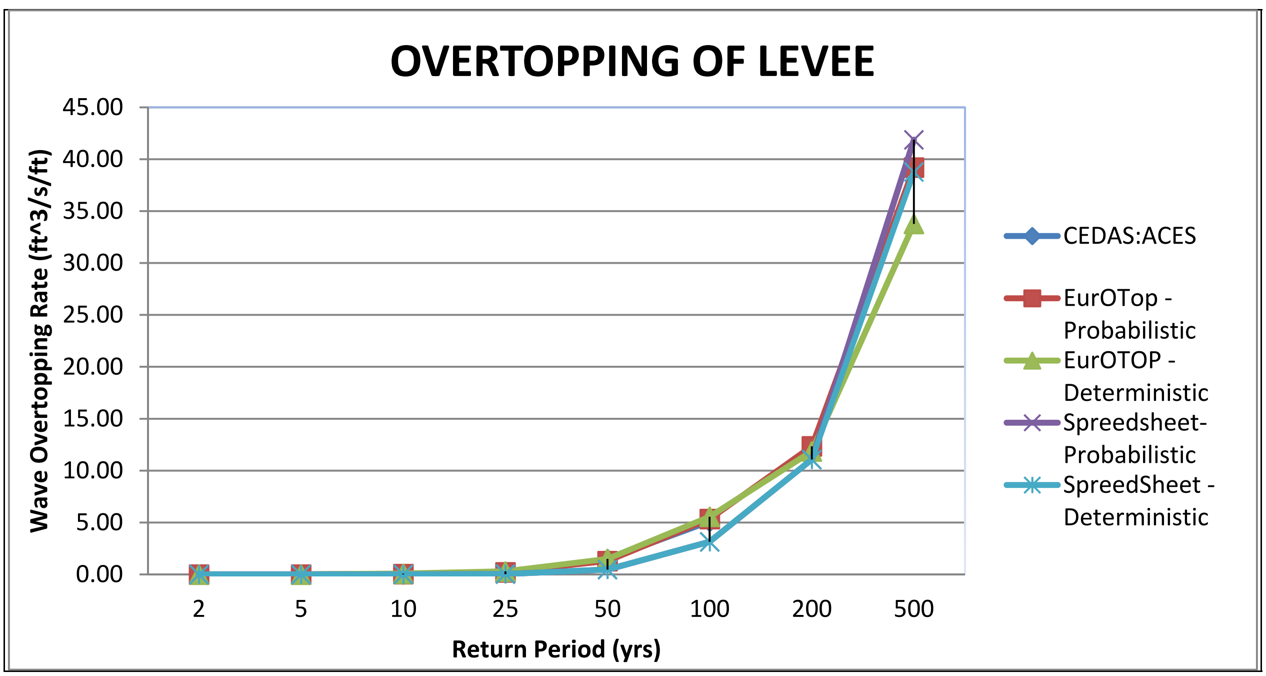

Figure 4.3 - Comparison of each method used (for Levee)

5.0 Overall Model Results of Levee/Floodwall System Overtopping Analysis

To update the overall level of project design performance, studies were examined which have been performed to develop wave flowrate-damage relationship models. Several of these overtopping models were used to develop the overtopping flowrates for the different return intervals; including the Corps Automated Coastal Engineering System (ACES), two Eurotop methods, and a method by Franco and Franco relayed in the Coastal Engineering Manual (EM 1110-2-1100, called the CEM). The results of these models were averaged, and the averages were compared to overtopping thresholds.

A wave overtopping flowrate threshold of 1.99 cfs/ft. was adopted as the Non-Failure point for the soil cement-reinforced levees, based on ERDC lab tests conducted in 2013. A flowrate of 3.00 cfs/ft. was adopted as the Failure point. The Non-Failure and Failure points are used in the economic lifecycle modeling.

Mean water surface elevations were utilized in the overtopping models, and also the 90% confidence water surface elevations. These water surface elevations, valid in Year 0, are shown in **Error! Reference source not found.**, along with the associated freeboards in feet.

Historic sea level rise estimates were evaluated in the models, which added 0.7 ft. to the mean water surface elevations and also to the 90% confidence simulations. These water surface elevations, valid in Year 50 are also shown in **Error! Reference source not found.** along with the associated freeboards.

Wave information, including wave height and wave period, at the base of the structure were developed using ACES fetch-limited analyses, which takes into account the average depth along the wind fetch, and an array of possible wind fetch directions. These wave heights were checked for appropriateness by comparing with depth-limited waves using the depth of water at the toe of the structure (which is equal to the water surface elevation minus the actual grade fronting the structure as determined using LIDAR topography). The fetch limited waves in all cases were found to be lower than the depth limited wave height, and thus appropriate.

Levee Freeboard for Mean and 90% Confidence Water Surface Elevations Year '0' & Year '50'					
Time	Return Period Years	Mean Water Surface Elevation in Ft. NGVD	Freeboard in ft.	90% Confidence Water Surface Elevation in ft. NGVD	Freeboard in ft.
Year '0'	2	6.1	8.9	6.5	8.5
	5	7.5	7.5	8.7	6.3
	10	8.9	6.1	10.6	4.4
	25	10.6	4.4	13.0	2.0
	50	11.9	3.1	14.8	0.2
	100	13.3	1.7	16.7	-1.7
	200	15.2	-0.2	19.3	-4.3
	500	16.6	-1.6	21.3	-6.3
Year '50'	2	6.8	8.2	7.2	7.8
	5	8.2	6.8	9.4	5.6
	10	9.6	5.4	11.3	3.7
	25	11.3	3.7	13.7	1.3
	50	12.6	2.4	15.5	-0.5
	100	14.0	1.0	17.4	-2.4
	200	15.9	-0.9	20.0	-5.0
	500	17.3	-2.3	22.0	-7.0

Table 5.1 – Summary of System Freeboard

The depth of water fronting the structure, called ds, played a more important role than initially thought. In the first series of modeling, the ds was computed as the water surface elevation minus the elevation of the subgrade bottom of the structure. As ds wasn't used for wave estimation, it was only believed to play a negligible role in the wave overtopping flowrate models. So the overly large ds using the subgrade toe was allowed. It was only when the first series of modeling results seemed overly conservative and every other method had been tried to yield more reasonable results that ds adjustments to account for earth elevations fronting the structures were tried as a last resort. Miraculously, this second series of modeling yielded more expected and reasonable results. The field of wave overtopping modeling is still in its infancy, and has a while to go before the process is fully understood. This effect of ds on overtopping is one area needing further research outside of this study.

Results of the individual overtopping models are contained in tables 2.4 and 3.4 above. Results of the average wave overtopping flowrates vs freeboard height are shown below in **Error! Reference source not found.**. Using Freeboard as the ordinate facilitates finding failure and non-failure points for present, future and 90% water surface elevations. Using the plotted curve, the

Non-Failure Point of 1.99 cfs/ft. was found to be caused by freeboard of 1.9 ft., and the Failure Point of 3.01 cfs/ft. was caused by freeboard of 1.4 feet.

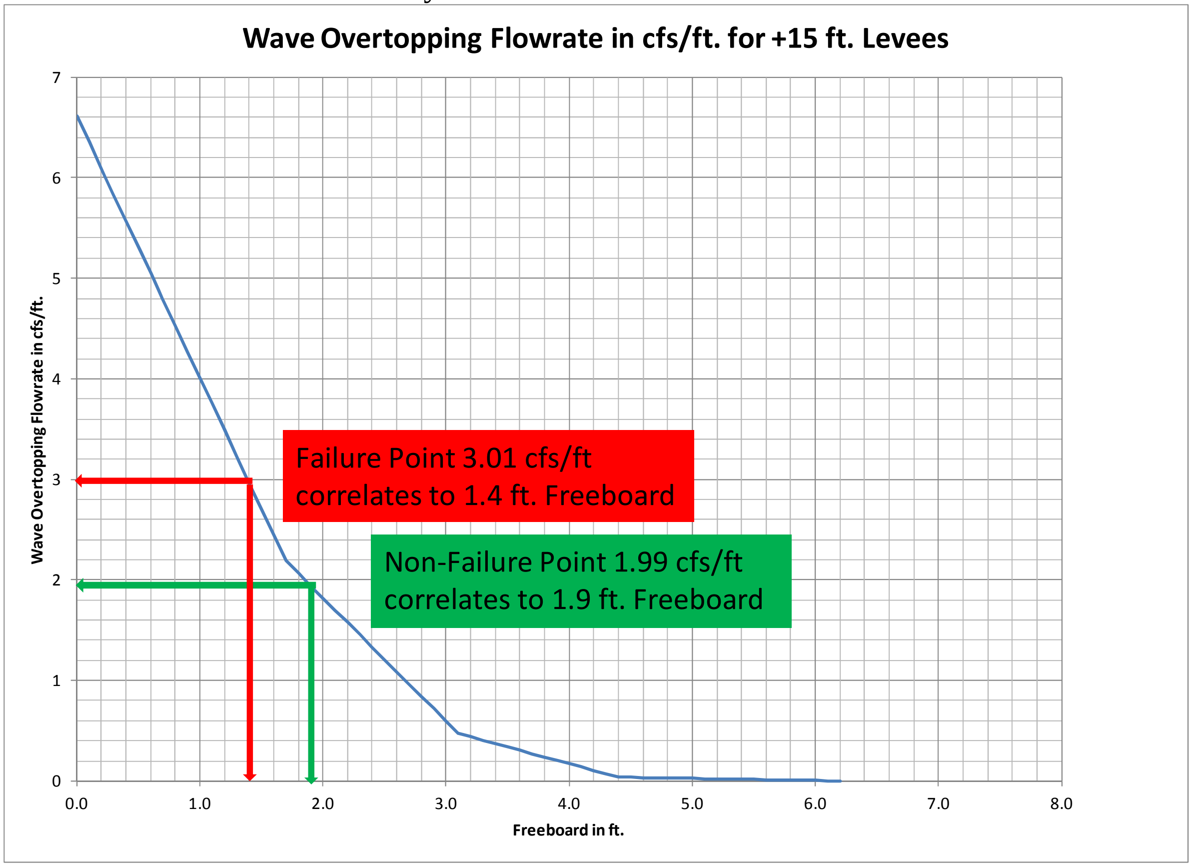

Figure 5.1 - Average Wave Overtopping Flowrates vs Freeboard Height

The Non-Failure Point freeboard of 1.9 ft. correlates to a water surface elevation of 13.1 ft. NGVD, and the Failure Point of 1.4 ft. of freeboard correlates to 13.6 ft. NGVD. Of interest in this analysis is the return intervals correlated with these Non-Failure and Failure Point water surface elevations. The water surface elevation-frequencies shown above in **Error! Reference source not found.** are plotted in **Error! Reference source not found.** for Year ‘0’, and in **Error! Reference source not found.** for Year ‘50’.

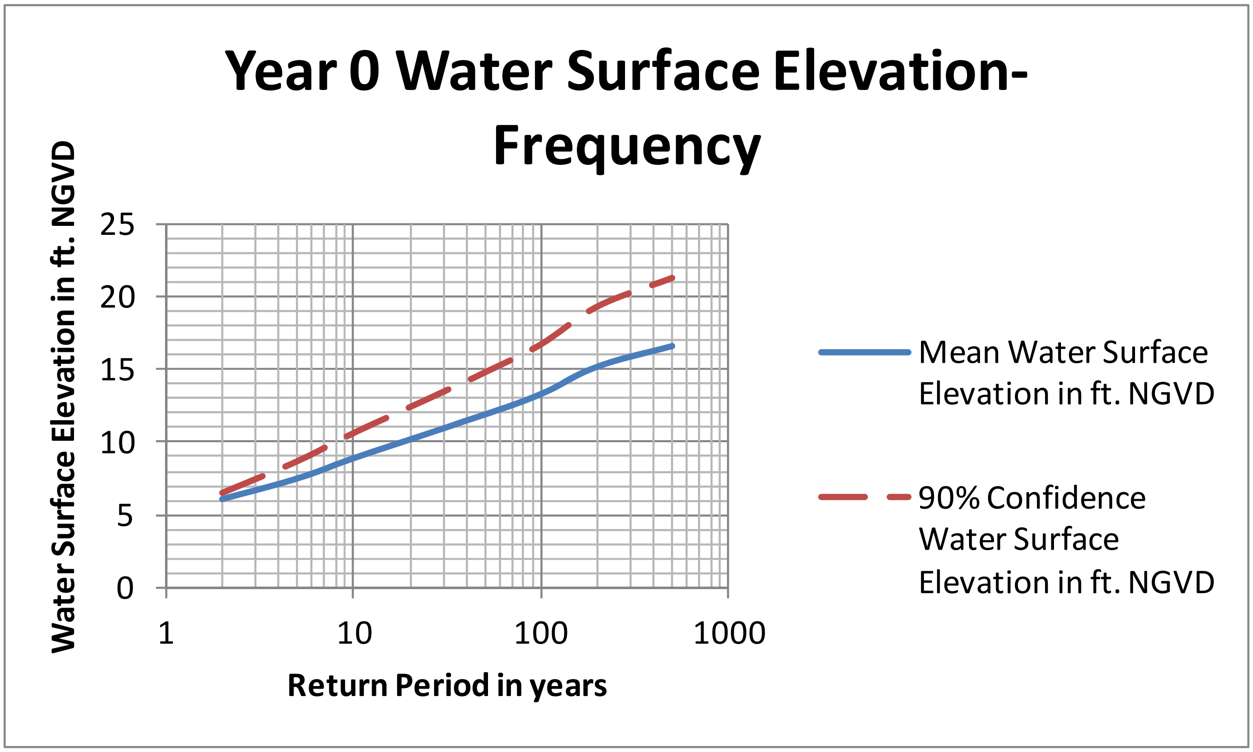

Figure 5.2 – Year '0' Water Surface Elevation Frequency

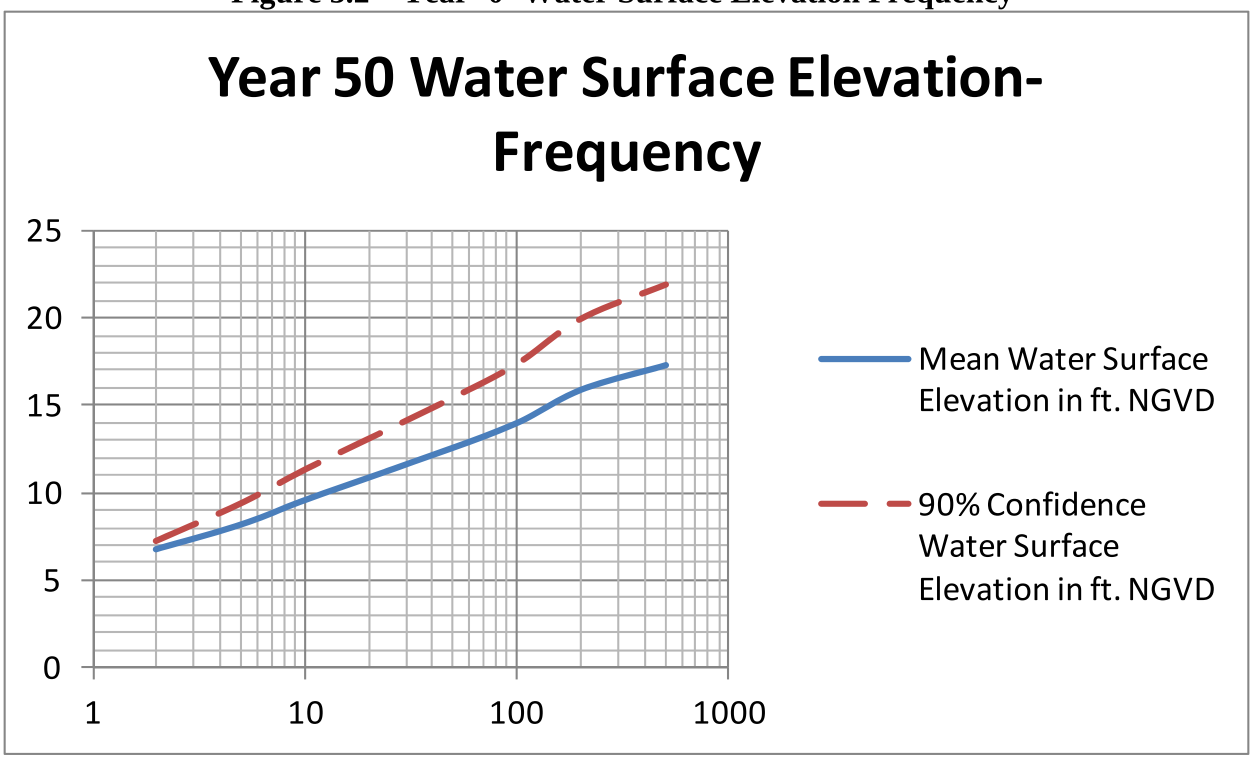

Figure 5.3 – Year '50' Water Surface Elevation Frequency

Results for Year ‘0’ and Year ‘50’ Water Surface Elevation Frequencies for Mean and 90% Confidence Levels follow:

Year 0 Mean Water Surface Elevations: Non-Failure Point (13.1 ft. NGVD)=94-year return period; Failure Point (13.6 ft. NGVD)=116-year return period

Year 0 90% Confidence Water Surface Elevations: Non-Failure Point (13.1 ft. NGVD)=26-year return period; Failure Point (13.6 ft. NGVD)=32-year return period

Year 50 Mean Water Surface Elevations: Non-Failure Point (13.1 ft. NGVD)=62-year return period; Failure Point (13.6 ft. NGVD)=81-year return period

Year 50 90% Confidence Water Surface Elevations: Non-Failure Point (13.1 ft. NGVD)=19-year return period; Failure Point (13.6 ft. NGVD)=32-year return period

HURRICANE SANDY LIMITED REEVALUATION REPORT

UNION BEACH, NEW JERSEY

ENGINEERING APPENDIX

SUB APPENDIX F

INTERIOR FLOODING ANALYSIS

August 2016

Introduction:

An interior flooding analysis was performed for the Borough of Union Beach Feasibility Report dated September 2003. The interior flooding analysis subdivided the protected area into three main watershed areas: Chingarora Creek, Flat Creek, and East Creek. The Chingarora Creek watershed consists of seven subbasins, Cl-1 thru Cl-8 and Chingarora Tributary, located along the Line Of Protection (LOP). The East Creek watershed consists of eight subbasins, E-1 thru E-7, located along the main branch flowing downstream to the line of protection. East Creek Tribuary flows through subbasin E-8 to the LOP. Subbasin EI-1 is an isolated subbasin located along the LOP north of the main branch. Flat Creek was modeled with 12 subbasins, F1-F12, terminating at the LOP. A map of the subbasins is presented in Figure F-1. For comparison, the drainage area map from the Feasibility Study is presented in Figure F-2. The subbasin delineation did not change for this analysis. An update of the interior flooding analysis was required due to changes in the hydrology and hydraulics in the project area and the development of new computer models since the 2003 analysis. This report documents the analyses done to update the interior flooding analysis.

Figure F-1, Drainage Area Map

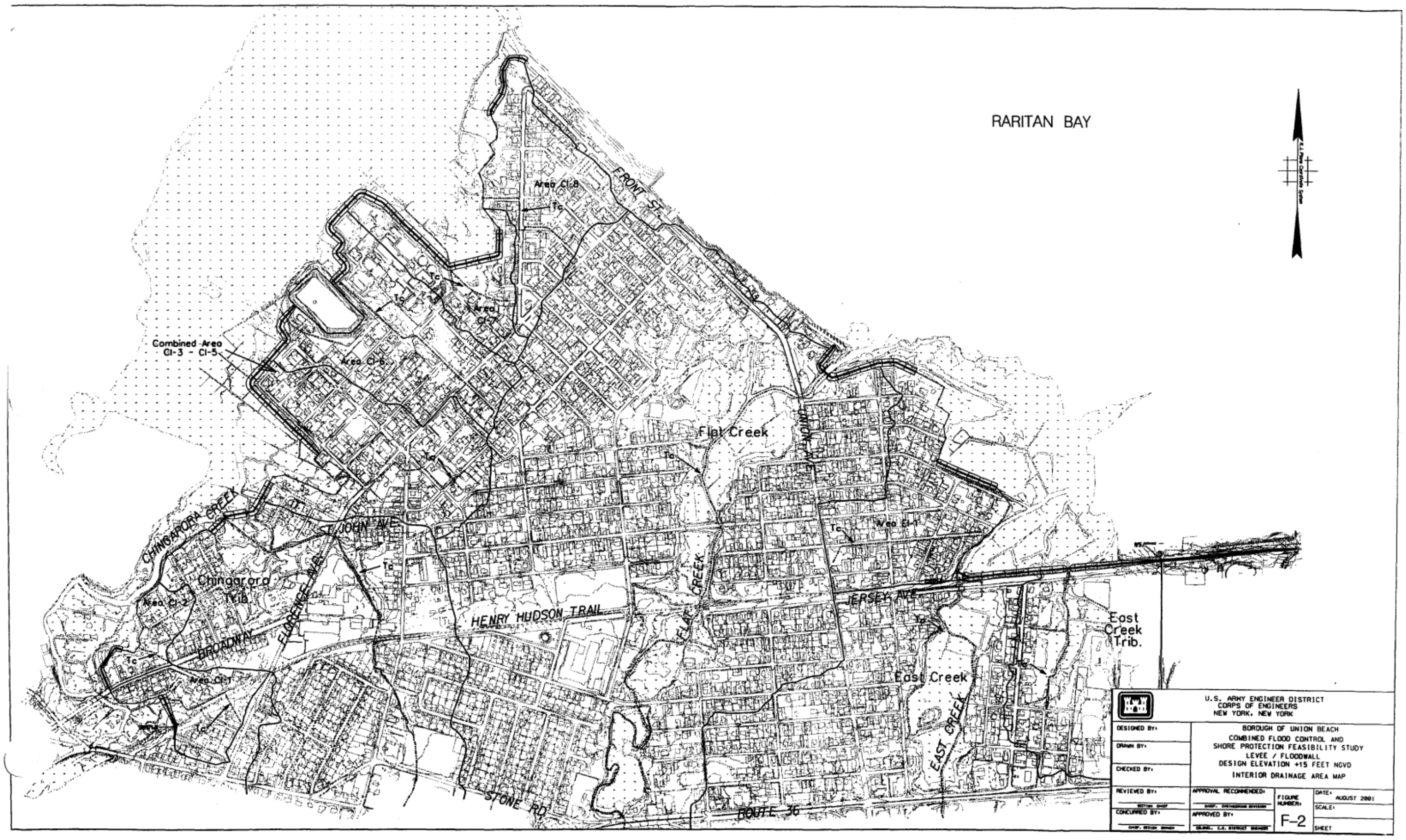

Figure F-2, Drainage Area Map from the Feasibility Report

Model Conversion:

The original interior flooding analysis was conducted using two computer models to simulate the hydrologic response of interior flooding areas and the operation of interior drainage facilities. These models, both developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE-HEC), are the Flood Hydrograph Package (HEC-1) and the Interior Flood Hydrology Package (HEC-IFH). The East Creek main branch and Flat Creek were modeled using HEC-1 to simulate surface runoff from the contributing subareas to the main creeks where they cross the line of protection. The East Creek and Flat Creek subbasins located at the line of protection and all of the Chingarora Creek subbasins were modeled with HEC-IFH to route the runoff against the variable tidal tailwater conditions.

Both HEC-1 and HEC-IFH are now classified as legacy programs and are no longer supported. As such these models have been superceded by HEC-HMS (Hydrologic Modeling System) also developed by the USACE-HEC. The HEC-HMS program computes both the watershed runoff and routes the runoff through the line of protection taking into account the effect of tailwater on drainage facilities therefore, performing the analyses previously performed using both HEC-1 and HEC-IFH. The HEC-HMS model was developed to succeed the aging HEC-1 program and was designed to use advances in engineering and computer science to improve the quality of the simulation results. It uses the same hydrologic and hydraulic computation procedures as the HEC-1 and HEC-IFH models with a few computation differnences that did not affect the models used for this analysis. The legacy HEC-1 and HEC-IFH models developed for the 2003 feasibility design were used to determine the data needed to build the HEC-HMS models.

The HEC-1 models for East Creek and Flat Creek were converted to HEC-HMS using the import capability within HEC-HMS. The resulting HEC-HMS models have the capability of computing the watershed runoff, but did not have the data required to be able to perform a routing through the LOP. The data needed for routing the runoff through the line of protection was determined from the HEC-IFH models.

The HEC-IFH models consisted of many separate files, each consisting of a piece or pieces of data needed to model the subbasins and the drainage facilities accurately. These files were sifted through to determine which contained the most up to date data. The resulting collection of files were analyzed to determine the required information to build the HEC-HMS models for the Chingarora Creek and EI-1 subbasins with additional data needed to finalize the Flat Creek and East Creek HEC-HMS models. The following data was determined from the HEC-IFH files: subbasin characteristics (area, SCS curve number, lag time), drainage structure data (size, length, inlet and outlet elevations, Mannings “n”), pump station capacity and on/off elevations, ponding storage, tailwater stage hydrographs, and hypothetical rainfall data. The hypothetical rainfall data was verified by comparing with Table F-5 in the Interior Drainage Appendix F of the 2003 Feasibility Report. The tailwater stage hydrographs were confirmed by comparing with Figures F-9, F-10 and F-11. Subbasin characteristics were compared with Table F-6. Where data could not be verified using the 2003 Feasibility Report, it was assumed that the HEC-IFH files contained the correct data.

HEC-HMS models were built for all of the subbasins using the 2003 Feasibility Report data and ran to determine the peak pond elevations for the subbasins. As in the 2003 feasibility analysis, the runoff through the line of protection was routed against three different tailwater conditions. These conditions include a normal high tide, a 2-year frequency (50%) condition and a 100-year frequency (1%) condition. The 2-year frequency condition is based on a northeaster with a gradual storm surge occurring over a 48-hour duration. This is considered an expected risk condition. The 100-year frequency condition is considered the high risk condition and is based on a Hurricane with a quicker and more peaked storm surge occurring over approximately a 24-hour. The normal high tide is the low risk condition.

Updating the HEC-HMS models:

Updated models were built in order to assess the changes to the ponding elevations at the LOP due to: revisions in methodology for determining hypothetical rainfall data, the occurrence of additional storm events that changed the tailwater tide marigrams, and recalculation of ponding storage. During the Planning, Engineering and Design (PED) phase of this project, the drainage area characteristics should be reassessed for changes in subbasin area, SCS curve number, and lag time as many structures were damaged or destroyed by Hurricane Sandy. The subbasin characterics used for this analysis are presented in Table F-1.

Table F-1
Subbasin Characteristics

Subbasin	Drainage Area (square miles)	SCS Curve Number	Lag Time (hours)	Lag Time (minutes)
Chingarora Ck Tributary	0.19	77.00	0.62	37.20
Chingarora Creek 1	0.02	76.00	0.20	12.24
Chingarora Creek 2	0.01	75.00	0.26	15.84
Chingarora 3 to Ching 5	0.07	78.00	0.31	18.60
Chingarora Creek 6	0.06	77.00	0.47	28.08
Chingarora Creek 7	0.02	73.00	0.20	12.24

Chingarora Creek 8	0.05	77.00	0.43	25.92
East Creek 1	0.49	69.00	0.60	36.00
East Creek 2	0.05	82.00	0.59	35.40
East Creek 3	0.12	72.00	0.58	34.80
East Creek 4	0.71	72.00	0.86	51.60
East Creek 5	0.29	72.00	0.99	59.40
East Creek 6	0.63	60.00	0.92	55.20
East Creek 7	0.14	79.00	0.48	28.80
East Creek 8 (tributary)	0.07	75.00	0.49	29.40
East Creek I-1	0.05	78.00	0.44	26.28
Flat Creek 1	0.46	66.00	0.83	49.80
Flat Creek 1A	0.17	68.00	0.65	39.00
Flat Creek 2	0.09	62.00	0.40	24.00
Flat Creek 3	0.16	78.00	0.58	34.80
Flat Creek 4	0.67	65.00	1.01	60.60
Flat Creek 5	0.13	73.00	0.55	33.00
Flat Creek 6	0.15	78.00	0.59	35.40
Flat Creek 7	0.32	79.00	0.65	39.00
Flat Creek 8	0.16	86.00	0.68	40.80
Flat Creek 9	0.07	79.00	0.55	33.00
Flat Creek 10	0.09	74.00	0.66	39.60
Flat Creek 11	0.20	77.00	0.49	29.40
Flat Creek 12	0.02	74.00	0.23	13.80

The ponding storage curves were recalculated for the East Creek and Flat Branch ponds at the line of protection using the latest available GIS LIDAR mapping from 2012. The ponding storage curves for the Chingarora Creek and EI-1 subbasins will need to be recalculated during PED.

The data for the hypothetical rainfall storm events for the 2003 feasibility analysis was developed from Technical Paper No. 40, Rainfall Frequency Atlas of the United States. For shorter durations, five minutes through 60 minutes, the NWS Hydro-Meteorological Report No. 35 was utilized. For this update, the rainfall was developed using Atlas 14 from the National Weather Service (which supercedes TP40 and Hydromet 35). The latitude and longitude of the location of the centroid of the drainage area for Union Beach, NJ project was used in Atlas 14 to determine the duration and amounts of the precipitation for the 50% chance (2-yr) thru 0.2% chance (500yr) hypothetical events. Table F-2 presents the updated hypothetical precipitation.

Hypothetical Precipitation (inches)

	Percent Chance Event							
Duration	50% (2 year)	20% (5 year)	10% (10 year)	4% (25 year)	2% (50 year)	1% (100 year)	0.4% (250 year)	0.2% (500 year)
5 min	0.402	0.477	0.532	0.600	0.648	0.696	0.755	0.798
15 min	0.81	0.96	1.07	1.21	1.30	1.40	1.50	1.58
1 hour	1.40	1.75	2.02	2.38	2.65	2.94	3.32	3.61
2 hours	1.72	2.18	2.53	3.02	3.42	3.84	4.42	4.87
3 hours	1.90	2.41	2.81	3.37	3.82	4.30	4.97	5.49
6 hours	2.42	3.07	3.58	4.33	4.95	5.61	6.58	7.34
12 hours	2.94	3.74	4.41	5.39	6.24	7.16	8.55	9.67

24 hours	3.34	4.30	5.12	6.37	7.45	8.66	10.52	12.10
48 hours	3.91	5.02	5.95	7.34	8.53	9.84	11.82	13.40

Table F-2

The tailwater tide marigrams were updated to take into account historical events that have occurred since the 2003 analysis including Hurricane Sandy. The 100yr tide, 2yr tide and normal tide marigrams are presented in Figures F-3 thru F-5.

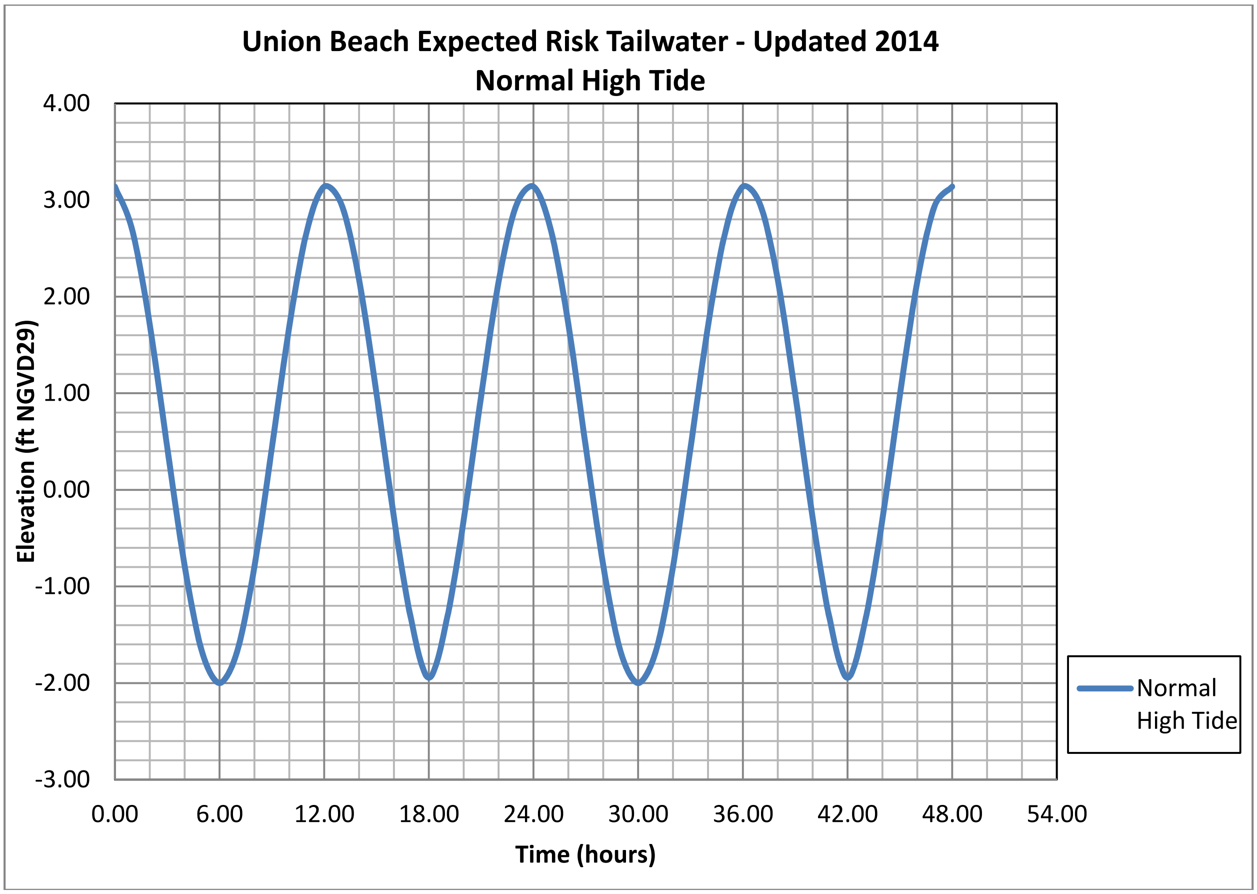

Figure F-3

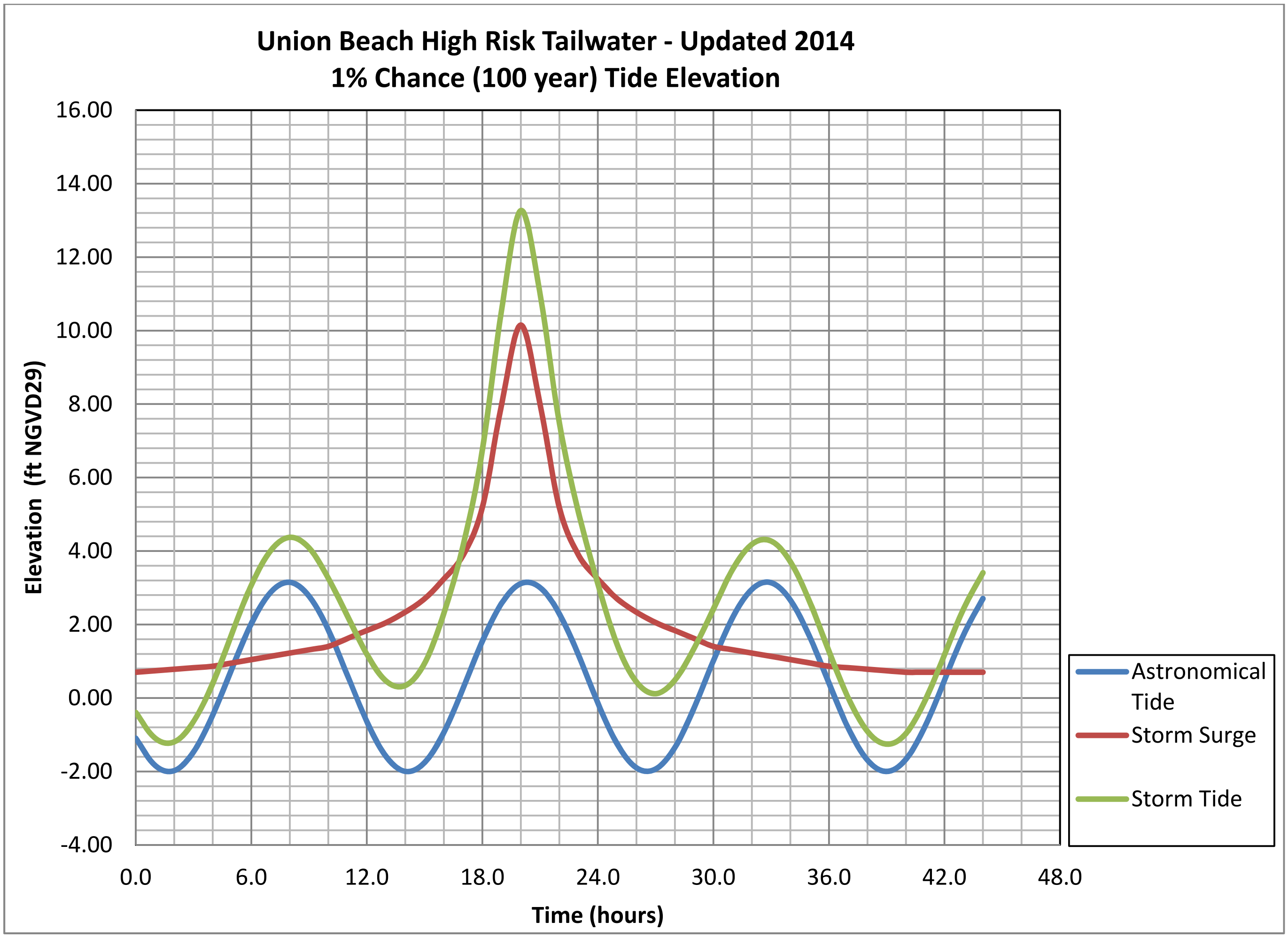
Union Beach High Risk Tailwater - Updated 2014
1% Chance (100 year) Tide Elevation
Elevation (ft NGVD29)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
0.0
6.0
12.0
18.0
24.0
30.0
36.0
42.0
48.0
Time (hours)
Astronomical Tide
Storm Surge
Storm Tide

Figure F-4

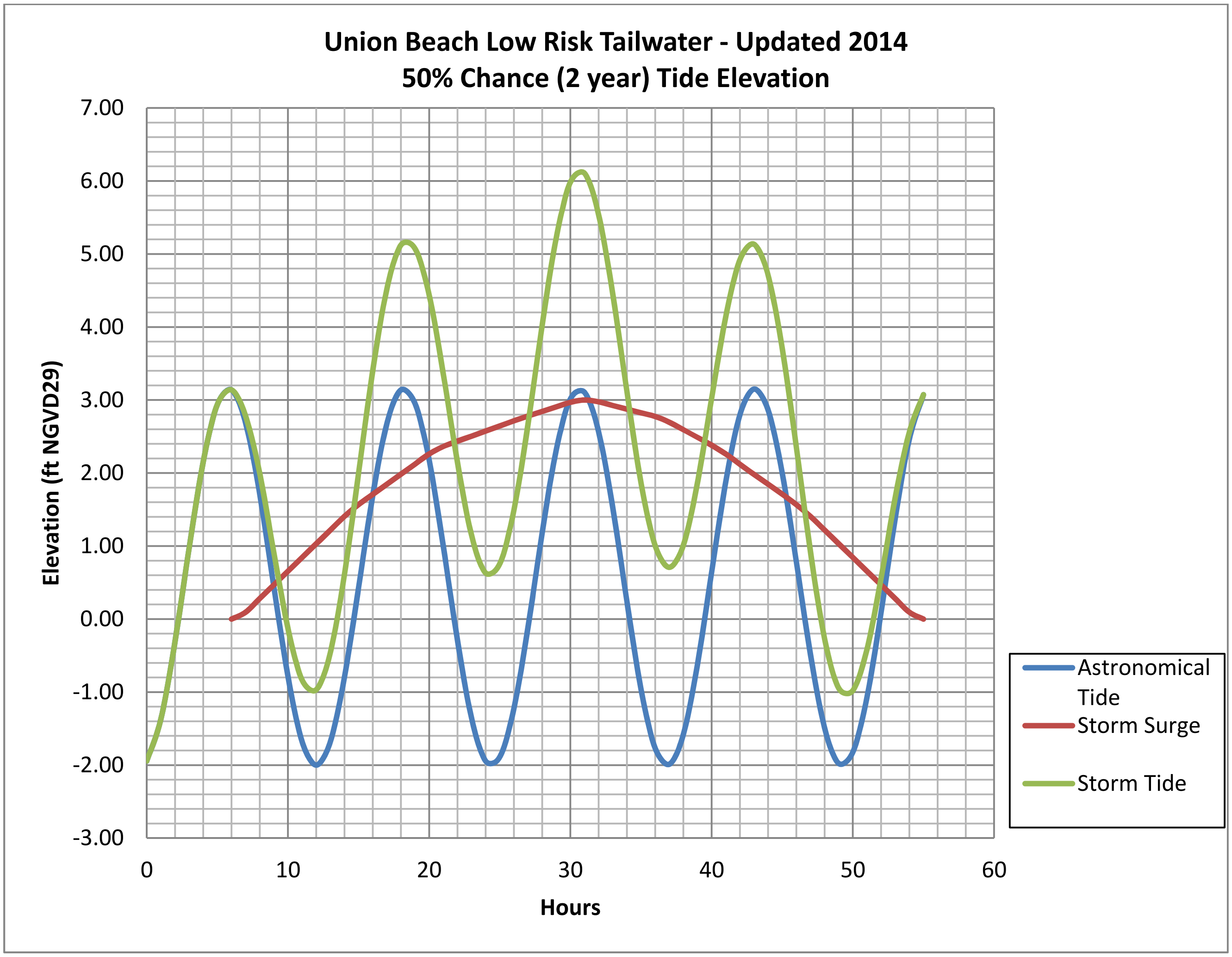

Figure F-5

The physical characteristics of the outlet structures through the line of protection that were designated as minimum facilities in the 2003 feasibility study were used in the updated HEC-HMS models. Pumping alternatives that were determined to be included for the National Economic Development (NED) plan in the feasibility report were added to the updated models and assessed as additional facilities. Additional ponding storage areas were not assessed in this analysis but will be considered during PED. The tide control structures are used to maintain the tidal interchange of the wetlands behind the line of protection and are closed during storm events. The gates are closed in the HEC-HMS model, therefore no discharge from the tide control structures is included. The drainage structure, tide control structure, and pump sizes used in this analysis are presented in Table F-3.

Table F-3
Union Beach Minimum and Additional Facilities

Subbasin	Area (sq mi)	Minimum Facilities: Outlet Structures: Size and Type	Inlet Elevation (ft NGVD29)	Outlet Elevation (ft NGVD29)	Slope (ft/ft)	Tide Control Structures	Additional Facilities: Add Pump Capacity	Increase Pond Storage
Chingarora Tributary	0.19	primary = 2-48" RCP secondary #1 = 24" RCP secondary #2 = 24" RCP	1.0 5.0 4.0	0.88 4.88 3.88	0.0048 0.0048 0.0048	3 - 6' x 6' Sluice Gates	n/a	n/a
CI-1	0.02	primary = 36" RCP secondary = 24" RCP	4.0 4.5	3.35 4.38	0.0100 0.0048		n/a	n/a
CI-2	0.01	primary = 24" RCP secondary #1 = 18" RCP secondary #2 = 18" RCP	5.0 6.0 6.5	4.88 5.50 6.38	0.0048 0.1111 0.0048		n/a	n/a
CI-3 to CI-5	0.07	primary = 36" RCP (existing) secondary #1 = 24" RCP secondary #2 = 2-24" RCP secondary #3 = 24" RCP secondary #4 = 36" RCP(ex) secondary #5 = 24" RCP secondary #6 = 36" RCP(ex) secondary #7 = 24" RCP secondary #8 = 24" RCP	1.9 1.78 2.0 3.0 4.0 3.0 4.0 3.0 4.0	1.75 1.00 1.88 2.35 3.88 2.35 3.88 2.35 3.88	0.0100 0.0104 0.0048 0.0100 0.0048 0.0100 0.0048 0.0100 0.0048		40 cfs	n/a
CI-6	0.06	primary = 2-48" RCP secondary =48" RCP	2.0 2.75	1.7 2.45	0.0050 0.0050		n/a	natural pond expanded to 1.6 acres
CI-7	0.02	primary = 4' x 4' box secondary #1 = 24" RCP secondary #2 = 24" RCP secondary #3 = 24" RCP secondary #4 = 24" RCP	3.6 3.75 5.0 5.0 5.0	2.8 3.0 4.88 4.68 4.65	0.0100 0.0100 0.0048 0.0050 0.0050		n/a	n/a
CI-8	0.05	primary = 2-24" RCP secondary #1 = 24" RCP secondary #2 = 2-24" RCP secondary #3 = 24" RCP	4.0 4.58 4.5 5.0	3.88 4.38 4.38 4.5	0.0048 0.0080 0.0048 0.0100		n/a	natural pond expanded to 0.76 acres
EI-1	0.05	primary = 2-36" RCP secondary #1 = 24" RCP secondary #2 = 24" RCP secondary #3 = 24" RCP secondary #4 = 24" RCP	2.0 4.0 4.0 4.0 4.0	1.88 3.88 3.88 3.88 3.88	0.0048 0.0048 0.0048 0.0048 0.0048		n/a	natural pond expanded to 0.35 acres
East Creek	2.44	primary = 2-60" RCP secondary #1 = 60" RCP secondary #2 = 60" RCP	1.00 1.00 1.00	0.88 0.88 0.88	0.0048 0.0048 0.0048		100 cfs	n/a
East Creek Tributary	0.07	Primary = 60" RCP secondary #1 = 60" RCP	1.00 1.00	0.88 0.88	0.0048 0.0048	3 - 6' x 6' Sluice Gates	n/a	natural pond expanded to 1.0 acres
Flat Creek	2.70	primary = 6-60" RCP secondary = 24" DIP	1.0 1.6	0.88 0.70	0.0015 0.0030	35' Storm Gate	250 cfs	natural pond expanded to 0.50 acres

Separate HEC-HMS models were built with the feasibility input data and with the 2014 updated input data for each subbasin to be able to compare the affect of the changes to the subbasins since the feasibility study analysis. The subbasin runoff was routed through the line of protection against the low, expected and high risk tailwater conditions. The resulting peak runoff is presented in Table F-4. It can be seen from this table that the peak runoff decreased slightly for the all frequency events and all subbasins except for East Creek and Flat Creek where the peak flow increased slightly for less frequent events (2%, 1% and 0.2%).

Table F-4

HEC-HMS Results – Subbasin Runoff

Subbasin	Drainage Area (sq mi)	Peak Runoff (cfs)															
		50% (2 year)		20% (5 year)		10% (10 year)		4% (25 year)		2% (50 year)		1% (100 year)		0.4% (250 year)		0.2% (500 year)	
		Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update	Feasibility Data	2014 Update
CI-1	0.02	20	21	29	28	34	34	n/a	41	49	47	56	53	n/a	60	73	65
CI-2	0.01	9	9	13	12	15	15	n/a	18	22	21	25	23	n/a	27	32	29
CI-3-5	0.07	60	61	87	82	102	99	n/a	122	147	139	168	157	n/a	179	215	196
CI-6	0.06	38	38	55	52	65	63	n/a	79	94	90	108	103	n/a	119	139	132
CI-7	0.02	23	23	33	32	39	38	n/a	47	57	54	65	61	n/a	69	84	76
CI-8	0.05	32	32	46	44	55	53	n/a	66	79	76	91	86	n/a	100	117	110
Chingarora Tributary	0.19	109	109	160	150	189	183	n/a	229	273	265	314	303	n/a	354	404	392
EI-1	0.05	36	36	52	49	62	60	n/a	74	89	85	101	96	n/a	111	130	122
East Cr at LOP	2.44	398	387	644	574	803	766	n/a	1033	1147	1169	1444	1518	n/a	2105	2243	2528
East Cr Tributary at LOP	0.07	35	36	58	54	71	68	n/a	89	108	105	126	121	n/a	143	165	159
East Cr Total at LOP	2.56	404	393	654	583	815	778	n/a	1050	1163	1188	1466	1543	n/a	2143	2282	2576
Flat Cr at LOP	2.7	663	657	981	996	1237	1199	n/a	1439	1604	1618	1794	1814	n/a	2270	2427	2600

The peak pond elevations for subbasins along the line of protection with the three (3) tailwater scenarios are presented in Tables F-6 thru F-25at the end of this sub-appendix. The peak pond elevation, storage, and inundation area results for the high risk event (100-year event with 100-year tailwater) for each subbasin is presented in Table F-26. The data from the tables was used to create pond elevation frequency curves which illustrate the effect of the tailwater conditions on the peak pond elevations. These curves are presented at the end of this sub-appendix as Figures F-6 thru F-25.

It can be seen from the peak pond elevation tables that the updates to the HEC-HMS models did not cause much change in the minimum facilities peak pond elevations. During PED, the effects of changes to subbasin characteristics such as ponding storage should be taken into account for all remaining subbasins (Chinagarora Creek and EI-1). Also, additional drainage structure, pump and ponding alternatives can be refined.

Pumping Alternatives:

Based on the National Economic Development (NED) plan recommended in September 2003 Feasibility Report, there are three subbasins recommended to have a supplemental pumping station to help reduce interior residual flooding. Many alternatives were computed and the ones proposed include: Subbasin Cl-3 to Cl-5 utilizing a 40 cfs pump station; Subbasin E7 or East Creek utilizing a 100 cfs pump station; and F12 or Flat Creek utilizing a 250 cfs pump station. Areas determined for ponding for East Creek and Flat Creek utilized the latest available LIDAR dated 2012. The ponding area for Cl-3 to Cl-5 used the 2003 values.

Each of these recommended pump station alternatives was modeled in the updated HEC-HMS model to determine the influence the pump stations would have on the residual interior water surface elevations. The best available information from the feasibility analysis was used and additional assumptions/revisions are as follows: All pump stations used an inlet elevation set to equal the gravity drain invert; the discharge assumed a pump up and over approach verses a through the line of protection approach which allows the pump to operate at a constant head and can be designed to operate at the pumps higher efficiency point of the curve; because the pumps are operating at a constant head, the curves used considered a relatively flat curve with a quick ramp up to the most efficient point.

As stated in the NED plan, it was assumed that the groundwater elevation (normal high tide) was at 3.0 feet NGVD and pumping would not begin until elevation 4.5 feet NGVD. While this is a conservative approach, it potentially limits the effective time that the pump can be used since the critical elevation for damages to start is at elevation 6.0 feet NGVD. One of the options considered and presented in the table below is revising the start/stop pump elevations. While reducing the elevations improved the results, it was not a significant change and further analysis should be conducted during the PED phase.

Gravity outlets were not considered closed during the event. It is assumed that during the event, water can flow out and will not flow into the system based on the use of flap gates. HEC-HMS accounts for this unlike some of the older models. This allows the maximum amount of flow out of the system and helps reduce the time the pumps operate.

Table F-5

Comparison of Pumping to Non-Pumping Conditions. 1% (100yr) Chance Event, NGVD 29

Sub-Basin	Peak Pond Elevation with 100-year Tailwater, 2003 Feasibility Report data imported to HEC-HMS	Peak Pond Elevation with 100-year Tailwater, 2014 Updated Report	Peak Pond Elevation with 100-year Tailwater, 2014 Updated Report with NED Pumping Station	Peak Pond Elevation with 100-year Tailwater with Pumps, 2014 Updated Report with Revised Pumping Elev.
Chingarora Creek Cl-3 to Cl-5	5.2	5.1	4.9	4.8*
East Creek at the Line of Protection	8.1	8.2	8.0	8.0**
Flat Creek at the Line of Protection	9.1	9.4	8.9	8.7***

* Changed starting condition for Chingarora: Pump 1, On 4.5 OFF 3.5 & Pump 3 On, 4.75 OFF 3.75 to Pumps 1-2, On 3.5 OFF 2.5 & Pump 3 On, 3.75 OFF 2.75

** Changed starting condition for East Creek: Pumps 1, On 4.5 OFF 3.5 & Pump 2 On, 5.0 OFF 4.0 to Pumps 1-2, On 3.5 OFF 2.5 & Pump 3 On, 3.75 OFF 2.75

*** Changed starting condition for Flat Creek: Pumps 1-2, On 4.5 OFF 3.5 & Pump 3 On, 5.0 OFF 4.0 to Pumps 1-2, On 3.5 OFF 2.5 & Pump 3 On, 3.75 OFF 2.75

Based on the results presented in the above table, it appears that the Chingarora pump station may not be required since the water surface stays below the critical elevation of 6.0 feet NGVD (where the stated damages begin) for the 100-year event. Results for East Creek are similar to the original elevations presented in the NED. The Flat Creek results are higher than the original report and could be contributed to the number of larger storm events that have occurred since the report was completed and differences in modeling methods of HEC-1, HEC-IFH and HEC-HMS. During the PED phase of the project, design refinement will be needed to be conducted for both the East Creek and Flat Creek drainage areas to maximize ponding reductions provided by the pump stations.

For this pumping analysis, only the 100-yr with 100-yr tailwater scenario was computed as this is the most critical event. To better model this scenario and the other scenarios, actual low head pump curves should be selected to better portray real world conditions. In addition, peak pond with normal tide and peak pond elevation with the 2-year tailwater will also be modeled during the PED phase to provide better operational guidance under multiple head scenarios.

Table F-6
Chingarora Creek Subbasin CI-1
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	5.7	6.2	6.4
20% (5yr)	6.0	6.3	6.9
10% (10yr)	6.2	6.4	7.2
4% (25yr)	n/a	n/a	n/a
2% (50yr)	6.6	6.7	7.9
1% (100yr)	6.8	6.9	8.2
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	7.3	7.3	8.7

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-7
Chingarora Creek Subbasin CI-1
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	5.7	6.1	6.4
20% (5yr)	6.0	6.2	6.9
10% (10yr)	6.2	6.3	7.1
4% (25yr)	6.4	6.5	7.5
2% (50yr)	6.6	6.7	7.9
1% (100yr)	6.8	6.8	8.1
0.4% (250yr)	7.0	7.0	8.4
0.2% (500yr)	7.2	7.2	8.6

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-8
Chingarora Creek Subbasin CI-2
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	6.5	6.5	7.3
20% (5yr)	6.8	6.8	7.5
10% (10yr)	6.9	6.9	7.7
4% (25yr)	n/a	n/a	n/a
2% (50yr)	7.2	7.2	8.0
1% (100yr)	7.3	7.3	8.1
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	7.6	7.6	8.3

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-9
Chingarora Creek Subbasin CI-2
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	6.5	6.5	7.3
20% (5yr)	6.8	6.8	7.5
10% (10yr)	6.9	6.9	7.7
4% (25yr)	7.1	7.1	7.9
2% (50yr)	7.2	7.2	8.0
1% (100yr)	7.3	7.3	8.1
0.4% (250yr)	7.4	7.4	8.2
0.2% (500yr)	7.5	7.5	8.3

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-10
Chingarora Creek Subbasin CI-3-5
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.9	4.1	4.1
20% (5yr)	4.2	4.5	4.5
10% (10yr)	4.4	4.6	4.6
4% (25yr)	n/a	n/a	n/a
2% (50yr)	4.8	5.0	5.0
1% (100yr)	5.0	5.2	5.2
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	5.2	5.4	5.4

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-11
Chingarora Creek Subbasin CI-3-5
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.9	4.1	4.1
20% (5yr)	4.2	4.4	4.4
10% (10yr)	4.3	4.6	4.6
4% (25yr)	4.6	4.9	4.9
2% (50yr)	4.7	5.0	5.0
1% (100yr)	4.9	5.1	5.1
0.4% (250yr)	5.1	5.2	5.2
0.2% (500yr)	5.1	5.4	5.4

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-12
Chingarora Creek Subbasin CI-6
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.7	4.5	4.5
20% (5yr)	4.1	5.1	5.1
10% (10yr)	4.2	5.2	5.2
4% (25yr)	n/a	n/a	n/a
2% (50yr)	4.6	5.6	5.6
1% (100yr)	4.8	5.8	5.8
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	5.1	6.1	6.1

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-13
Chingarora Creek Subbasin CI-6
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.8	4.8	4.8
20% (5yr)	4.0	5.1	5.1
10% (10yr)	4.2	5.2	5.2
4% (25yr)	4.4	5.4	5.4
2% (50yr)	4.5	5.6	5.6
1% (100yr)	4.7	5.8	5.8
0.4% (250yr)	5.0	6.0	6.0
0.2% (500yr)	5.0	6.1	6.1

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-14
Chingarora Creek Subbasin CI-7
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.9	5.3	5.3
20% (5yr)	5.1	5.5	5.5
10% (10yr)	5.2	5.7	5.7
4% (25yr)	n/a	n/a	n/a
2% (50yr)	5.4	6.0	6.0
1% (100yr)	5.6	6.1	6.1
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	5.8	6.2	6.3

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-15
Chingarora Creek Subbasin CI-7
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.9	5.3	5.3
20% (5yr)	5.1	5.5	5.5
10% (10yr)	5.2	5.7	5.7
4% (25yr)	5.3	5.9	5.9
2% (50yr)	5.4	6.0	6.0
1% (100yr)	5.5	6.1	6.1
0.4% (250yr)	5.7	6.1	6.2
0.2% (500yr)	5.8	6.2	6.3

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-16
Chingarora Creek Subbasin CI-8
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	5.7	6.1	6.1
20% (5yr)	6.0	6.2	6.4
10% (10yr)	6.1	6.3	6.6
4% (25yr)	n/a	n/a	n/a
2% (50yr)	6.4	6.6	7.0
1% (100yr)	6.6	6.8	7.1
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	7.0	7.1	7.4

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-17
Chingarora Creek Subbasin CI-8
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	5.7	6.1	6.1
20% (5yr)	5.9	6.2	6.4
10% (10yr)	6.0	6.3	6.6
4% (25yr)	6.2	6.4	6.8
2% (50yr)	6.4	6.6	7.0
1% (100yr)	6.6	6.7	7.1
0.4% (250yr)	6.8	6.9	7.2
0.2% (500yr)	6.9	7.0	7.4

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-18
Chingarora Tributary Subbasin
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.9	5.1	5.2
20% (5yr)	4.3	5.6	5.8
10% (10yr)	4.5	5.9	6.1
4% (25yr)	n/a	n/a	n/a
2% (50yr)	5.1	6.3	6.8
1% (100yr)	5.3	6.6	7.2
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	5.9	7.0	7.9

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-19
Chingarora Tributary Subbasin
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	3.8	5.1	5.2
20% (5yr)	4.2	5.6	5.7
10% (10yr)	4.4	5.8	6.1
4% (25yr)	4.8	6.1	6.6
2% (50yr)	5.1	6.4	7.0
1% (100yr)	5.3	6.6	7.4
0.4% (250yr)	5.7	6.9	8.0
0.2% (500yr)	6.0	7.2	8.3

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-20
East Creek Subbasin EI-1
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.1	5.5	5.5
20% (5yr)	4.5	5.9	5.9
10% (10yr)	4.7	6.1	6.1
4% (25yr)	n/a	n/a	n/a
2% (50yr)	5.1	6.2	6.3
1% (100yr)	5.3	6.2	6.5
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	5.7	6.4	6.8

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-21
East Creek Subbasin EI-1
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.1	5.5	5.5
20% (5yr)	4.4	5.9	5.9
10% (10yr)	4.7	6.0	6.0
4% (25yr)	4.9	6.1	6.2
2% (50yr)	5.1	6.1	6.3
1% (100yr)	5.2	6.2	6.4
0.4% (250yr)	5.4	6.3	6.6
0.2% (500yr)	5.6	6.3	6.8

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-22
East Creek at the Line of Protection
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.2	4.6	4.6
20% (5yr)	5.2	5.8	5.9
10% (10yr)	5.7	6.2	6.2
4% (25yr)	n/a	n/a	n/a
2% (50yr)	6.5	6.8	7.3
1% (100yr)	7.0	7.3	8.1
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	8.2	8.4	9.3

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-23
East Creek at the Line of Protection
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.2	4.5	4.5
20% (5yr)	4.9	5.5	5.6
10% (10yr)	5.5	6.0	6.2
4% (25yr)	6.3	6.5	7.0
2% (50yr)	6.8	7.0	7.6
1% (100yr)	7.3	7.5	8.2
0.4% (250yr)	8.1	8.2	8.9
0.2% (500yr)	8.6	8.7	9.4

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

Table F-24
Flat Creek at the Line of Protection
HEC-HMS Results with 2003 Feasibility Data

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.6	5.9	6.2
20% (5yr)	5.5	6.6	7.3
10% (10yr)	6.1	6.9	7.8
4% (25yr)	n/a	n/a	n/a
2% (50yr)	6.9	7.5	8.6
1% (100yr)	7.3	7.9	9.1
0.4% (250yr)	n/a	n/a	n/a
0.2% (500yr)	8.0	8.4	9.6

Peak Exterior Tailwater Elevations: Low Risk = 3.2 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 12.5 ft NGVD29)

Table F-25
Flat Creek at the Line of Protection
HEC-HMS Results with Updated Data (2014)

Percent Chance Event	Peak Pond Elevation with Normal Tide Tailwater (Low Risk) (ft NGVD29)	Peak Pond Elevation with 2-year Tailwater (Expected Risk) (ft NGVD29)	Peak Pond Elevation with 100-year Tailwater (High Risk) (ft NGVD29)
50% (2yr)	4.5	5.9	6.3
20% (5yr)	5.4	6.5	7.2
10% (10yr)	6.0	6.9	7.9
4% (25yr)	6.6	7.3	8.5
2% (50yr)	7.0	7.6	9.0
1% (100yr)	7.4	8.0	9.4
0.4% (250yr)	8.0	8.4	9.9
0.2% (500yr)	8.5	8.7	10.3

Peak Exterior Tailwater Elevations: Low Risk = 3.1 ft NGVD29, Expected Risk = 6.1 ft NGVD29, High Risk = 13.3 ft NGVD29)

The 2003 Feasibility Report states that the residual flooding is to be compensated above minimum facilities requirement by the expansion of four (4) ponds as a project element. The remaining flood risk management features are in the form of minimum facilities and pump stations for the interior drainage.

For the 2015 HSLRR Interior Drainage analysis, separate HEC-HMS models were built with the feasibility input data and with the 2014 updated input data for each subbasin to compare the effect of the changes to the subbasins since the feasibility study analysis. The subbasin runoff was routed through the line of protection against the low, expected and high risk tailwater conditions. The resulting peak runoff inidcates that the peak runoff decreased slightly for the all frequency events and all subbasins except for East Creek and Flat Creek where the peak flow increased slightly for less frequent events (2%, 1% and 0.2%).

Results indicate that design modification of the authorized plan is not required at this time. Please refer to the 2003 Feasibility Report, Appendix F.

Pond Expansion:

Pond Cl-8, (33,100 sf) 0.76 ac- upland;
Pond Cl-6, (69,700 sf) 1.60 ac- wetland, stream;
Pond El-1, (15,250 sf) 0.35 ac- wetland;
Pond El-2 (interior), (43,560 sf) 1.00 ac- wetland;
Pond El-3 (interior), (21,780 sf) 0.50 ac- wetland
Total= 4.21 acres

Table F-26
HEC-RAS Results with Updated Data (2014)
100-year Event with 100-year Tailwater (High Risk)

Subbasin	Peak Pond Elevation (ft NGVD29)	Peak Pond Storage (ac-ft)	Peak Pond Innudation Area (acres)
CI-1	8.1	1.5	0.8
CI-2	8.1	0.4	0.5
CI-3-5	5.1	1.8	2.2
CI-6	5.8	5.4	5.9
CI-7	6.1	1.9	2.6
CI-8	7.1	1.7	1.8
Chinagarora Trib	7.4	39.0	12.7
EI-1	6.4	2.5	2.6
East Cr at LOP	8.2	203.2	84.2
Flat Cr at LOP	9.4	418.2	145.2

SUMMARY OF INTEROR DRAINAGE AUTHORIZED PLAN

The determination of required interior facilities in the 2015 HSLRR remains unchanged from the interior facilities in the 2003 Feasibility Report and should provide adequate drainage at least equal to that of the existing infrastructure. This plan represents the minimum interior facilities required to implement the line of protection plan.

The analysis is based on the concepts and guidelines contained in U.S. Army Corps of Engineers' Engineer Manual EM 1110-2-1413 (Hydrologic Analysis of lnterior Areas). The interior areas drain toward three different watersheds, Chingarora Creek, Flat Creek and East Creek. Each of the three main watersheds were divided into sub areas and analyzed separately. Within these three watersheds, a total of ten interior areas were identified and evaluated. For all of the watersheds, numerous outlet structures are required to pass drainage through the line of protection.

Plans include the use of excavated ponds, pump stations and the use of pump stations in conjunction with ponds. The excavated pond plans were developed for interior drainage areas by using a procedure designed to create the largest possible pond for each location. No pond is to be excavated below elevation 3.5 ft. NGVD to minimize standing water conditions. Pond side slopes are set at maximum 3:1 to allow for maintenance and not pose a safety hazard.

As the existing wetlands areas along Chingarora, Flat and East Creeks already provide extensive storage between elevations 3 feet and 5 feet NGVD (10.9 ac ft, 48.9 ac ft, and 26.6 ac ft respectively), no ponding plans were evaluated for the main creeks interior drainage.

Pumping plans incorporate the use of pump stations in conjunction with the Minimum Facility features developed for each interior area. Pump stations are considered as a means of reducing residual flood heights within interior ponds through the mechanical displacement of accumulated surface runoff from the interior watershed. To determine how efficiently a pump station reduces flooding, the resultant reduction in residual flood damages is compared to the initial and annual costs of building and operating the pump station. The ponding/pumping alternatives were analyzed using the storage areas developed for the ponding alternatives and a pump station that would be reasonably cost- effective.

<u>Chingarora Creek Area CI-1</u>

Interior area CI-1, consisting of approximately 13 acres, is located in the southwestern portion of the Chingarora Creek watershed. First roadway flooding in Interior Area Cl-1 is observed at elevation 8.0 ft NGVD, while first significant damage to structures is observed at elevation 11.0 ft NGVD.

Minimum Facility. Minimum facility conditions for Interior Area CI-1 consist of a 36" RCP primary outlet, a 24" RCP secondary outlet, and 90 linear feet of channel excavation. The most cost-effective interior facility for Interior Area CI-1 was identified as being the minimum facility.

Chingarora Creek Area CI-2

Interior area Cl-2 is also located in the southwestern portion of the Chingarora Creek watershed, lying just west of area CI-1. Interior area CI-2 accounts for roughly 7 acres of the Chingarora Creek watershed. First roadway flooding in Interior Area CI-2 is observed at elevation 8.0 ft NGVD, while first significant damage to structures is observed at elevation 10.0 ft NGVD.

Minimum Facility. Minimum facility conditions for Interior Area CI-2 consist of one 24" RCP primary outlet, two 18" RCP secondary outlets, and 380 linear feet of channel excavation. The most cost-effective interior facility for Interior Area CI-2 was identified as being the minimum facility.

Chingarora Creek Tributary

The Chingarora Creek Tributary Interior Area, approximately 125 acres in area, is located northeast of Interior Areas CI-1 and CI-2 in the southern portion of the Chingarora Creek watershed. First roadway flooding in this area is observed at elevation 5.0 ft NGVD, while first significant damage to structures is observed at elevation 8.0 ft NGVD.

Minimum Facility. Minimum facility conditions for the Chingarora Creek Tributary Interior Area incorporate the following: four 48" RCP primary outlet through the line of protection at the flood gate for the main channel, nine 24" RCP secondary outlets, and three 6'x6' sluice gates to maintain the tidal interchange of the wetlands behind the line of protection.

Additional Facilities Considered. Further analyses investigated the need for additional facilities including possible pump stations or ponds. Since, the existing wetlands areas along the Chingarora Tributary already provide extensive storage between elevations 3 and *5* (approximately 10.9 acft), no ponding alternatives were evaluated for the Chingarora Creek Tributary interior drainage. The most cost-effective interior facility for the Chingarora Creek Tributary Interior Area was identified as being the minimum facility.

Combined Chingarora Creek Areas CI-3, CI-4 and CI-5

Interior Area CI3-5 is a combination of three smaller unit areas: CI-3, CI-4, and CI-5.

Together, the combined Interior Area CI3-5 is approximately 40 acres in area, located north of the Chingarora Creek Tributary Interior Area. First roadway flooding in the combined Interior Area CI3-5 is observed at elevation 4.1 ft NGVD, while the elevation of first significant damage to structures is 6.0 ft NGVD.

Minimum Facility. Minimum facility conditions for Interior Area CI3-5 consist of extending an existing 18" outlet with a 36" RCP primary outlet; one 36" RCP secondary outlet, one twin 24" RCP secondary outlet; five 24" RCP secondary outlets, one 36" RCP secondary outlet extension on an existing 24" outlet, and 725 linear feet of channel excavation.

The most cost-effective interior facility for Interior Area CI-3 through CI-5 was identified as the installation of a 40 cfs pump station. As part of the CBRA realignment an additional 10 outlets are recommended to minimize the depth of drainage ditches and pipes.

Chingarora Creek Area CI-6

This area is located near the Monmouth County Outfall Authority settling pond and consists of approximately 36.0 acres. Chingarora Street floods at elevation 4.9 NGVD and the elevation of first significant damage to structures is at 6 feet NGVD.

Minimum Facility. Minimum facility conditions in this interior area consist of a twin 48" RCP primary outlet, and a 48" RCP secondary outlet.

The construction of a ponding area was identified as the most cost-effective interior facility for Interior Area CI-6.

Chingarora Creek Area CI-7

Located just north of Interior Area CI-6, Area Cl-7 comprises an area of approximately 16 acres of the Chingarora Creek watershed. First flooding of roadways in Interior Area CI-7 is observed at elevation 5.5 feet NGVD, while first significant damage to structures is observed at elevation 6.0 ft NGVD.

Minimum Facility. For Interior Area CI-7, minimum facility conditions have been defined as one 4'x4' box culvert primary outlet, four 24" RCP secondary outlets, and 540 liner feet of channel excavation.

The most cost-effective interior facility for Interior Area CI-7 was identified as being the minimum facility.

Chingarora Creek Area CI-8

Interior Area CI-8, comprising an area of approximately 29 acres, is located on the Raritan Bay just north of Interior Area CI-7. While first significant damage to structures is not observed until8.0 ft NGVD, roadway flooding begins when the water reaches 6.4 ft NGVD.

Minimum Facility. For Interior Area CI-8, minimum facility conditions consist of one twin 24" RCP primary outlet, one twin 24" RCP secondary outlet, two 24" RCP secondary outlets, and approximately 230 linear feet of channel excavation.

Construction of a ponding area was identified as the most cost-effective interior facility for Interior Area CI-8.

Interior Facilities for Flat and East Creek

East Creek and East Creek Tributary are treated as one interior drainage area and are hydraulically connected via a ditch that runs along Jersey Avenue. The interior area between East Creek and East Creek Tributary has very low ground elevations (as low as 4-5 feet NGVD) and consequently, floods with frequent tidal storm events. To ensure no disruption of flow through the adjacent wetlands, the storm closure gates need to remain open for the full range of tide levels, including spring tides. The spring tides and other frequent high tide level events, however, inundate the developed area. A small supplemental levee will protect the area between East Creek and East Creek Tributary while allowing flooding of the adjacent wetlands for the full range of non-storm tidal conditions. The levee would be between 1 and 4 feet high with its own drainage outlets.

The interior drainage plan with the highest net benefits on East Creek and East Creek Tributary included a 100 cfs pumping station along with the 8ft NGVD crest elevation interior levee.

Flat Creek

This area consists of approximately 1,734 acres of tributary drainage area. Within this area, Spruce Street floods at elevation 4.6 feet NGVD and the elevation of first significant damage to structures is at 6 feet NGVD.

Minimum Facility. Minimum facilities for Flat Creek consist of six 60" RCP primary outlets through the line of protection at the floodgate for the main channel, and an existing 24" DIP secondary outlet. A 35' floodgate will be used to maintain the tidal interchange of the wetlands behind the line of protection, which will be closed during storm events. Both the primary and secondary outlets are being provided with a flap gate, sluice gate, and trash rack. Drainage ditches will direct runoff along the protected side of the levee to a nearby outfall.

The most cost-effective interior facility for the Flat Creek Interior Area was identified as the installation of a 250-cfs pump station.

East Creek

The East Creek watershed lies in the easternmost portion of the study area, and consists of two smaller interior areas: the Combined East Creek/East Creek Tributary Interior Area, and Interior Area El-l. In total, the watershed comprises an area of approximately 1,636 acres. Interior drainage facility optimization for each interior area is discussed below

This area is located at the northern limits of the East Creek watershed and consists of approximately 34 acres. Bayview Avenue floods at elevation 5.4 feet NGVD and the elevation of first significant damage to structures is at 6 feet NGVD.

Minimum Facility. In this area, minimum facility consists of a twin 36" RCP primary outlet, four 24" RCP secondary outlets, and 275 linear feet of channel excavation.

Additional facilities were evaluated in conjunction with the minimum facility condition. Construction of a ponding area was identified as the most cost-effective interior facility for Interior Area EI-1.

East Creek/ East Creek Tributary

This area consists of approximately 1,601 acres tributary to the East Creek at the line of protection along Jersey Avenue. This area floods at elevation 4.5 feet NGVD and the elevation of first significant damage to structures is at 6 feet NGVD.

Minimum Facility. Minimum facilities for this area consist of a twin 60" RCP primary outlet through the line of protection at the floodgate for the main channel, a 60" RCP outlet through the line of protection at the East Creek Tributary main channel, and three 60" RCP secondary outlets. A 35' flood gate will be used to maintain the tidal interchange of the wetlands behind the line of protection at East Creek, and three 6'x6 sluice gates will be used to maintain the tidal interchange of the wetlands behind the line of protection at the East Creek Tributary. An equalization ditch will be provided along the levee to connect the East Creek and East Creek Tributary interior areas during low intensity, high frequency storm events. Drainage ditches will direct runoff along the protected side of the levee to a nearby outfall. The 100 cfs pump alternative with a levee at 8ft NGVD was selected for the combined East Creek/East Creek Tributary Interior Area.