North Shore of Long Island

Bayville, New York

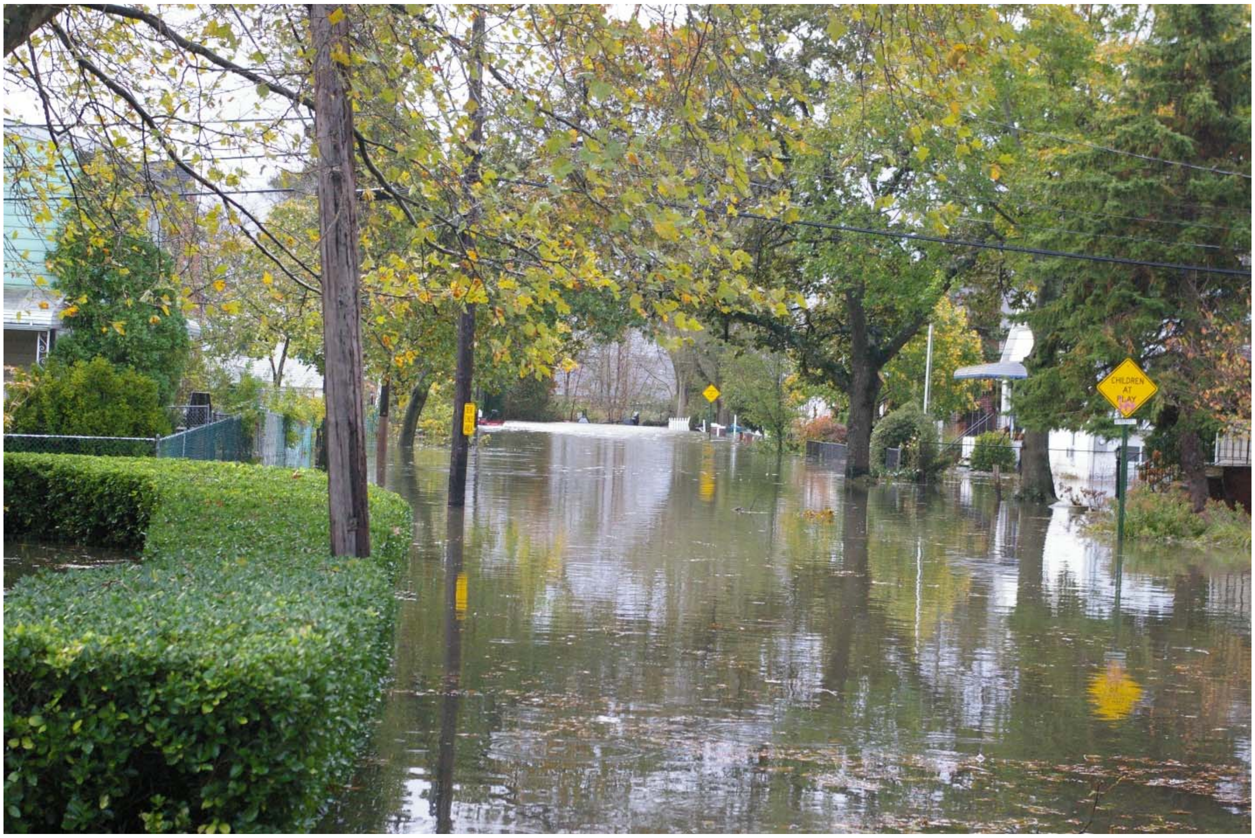
Coastal Storm Risk Management

Feasibility Study

Appendix B: Engineering

February 2016

APPENDIX (B)
Coastal ENGINEERING
Draft Integrated Feasibility Report & Environmental Assessment
February 2016



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1.0 Introduction

1.1 Study Location

The Village of Bayville is located in Nassau County within the Town of Oyster Bay. The study area covers both the north shore of Long Island between Oak Neck Point to the west and Rocky Point to the east on Long Island Sound and the southern shoreline fronting Millneck Bay and Oyster Bay. The study area is shown in Figures 1-1 and 1-2.

1.2 Study Area Overview

Bayville is a low-lying community in the Town of Oyster Bay on the north shore of Long Island. The study area is generally characterized as being a peninsula that runs from west to east and is bounded by Long Island sound to the north and Oyster Bay to the south. Both provide flooding potential to the study area. Long Island Sound produces both high water conditions as well as significant wave energy and Oyster Bay produces similar high water conditions as the Sound but little wave energy. The northern shoreline within the study area is a fully developed, mostly private gated beachfront community. The north shore beachfront is approximately 1.5 miles facing Long Island Sound with beach widths range from 100 to 200 feet. The majority of the beachfront properties are built behind timber bulkhead and the rest are behind dunes. The peninsula's west side ties into high ground and the lowest areas of the study area generally running down the middle of the peninsula. This topography has been described in loose terms as a bowl where once water overtops the water side edges of the peninsula it tends to collect and sit in the bowl. Fresh water from rain events also can cause flooding. The community located between Bayville Avenue and the southern Bay front is generally low-lying residential area.

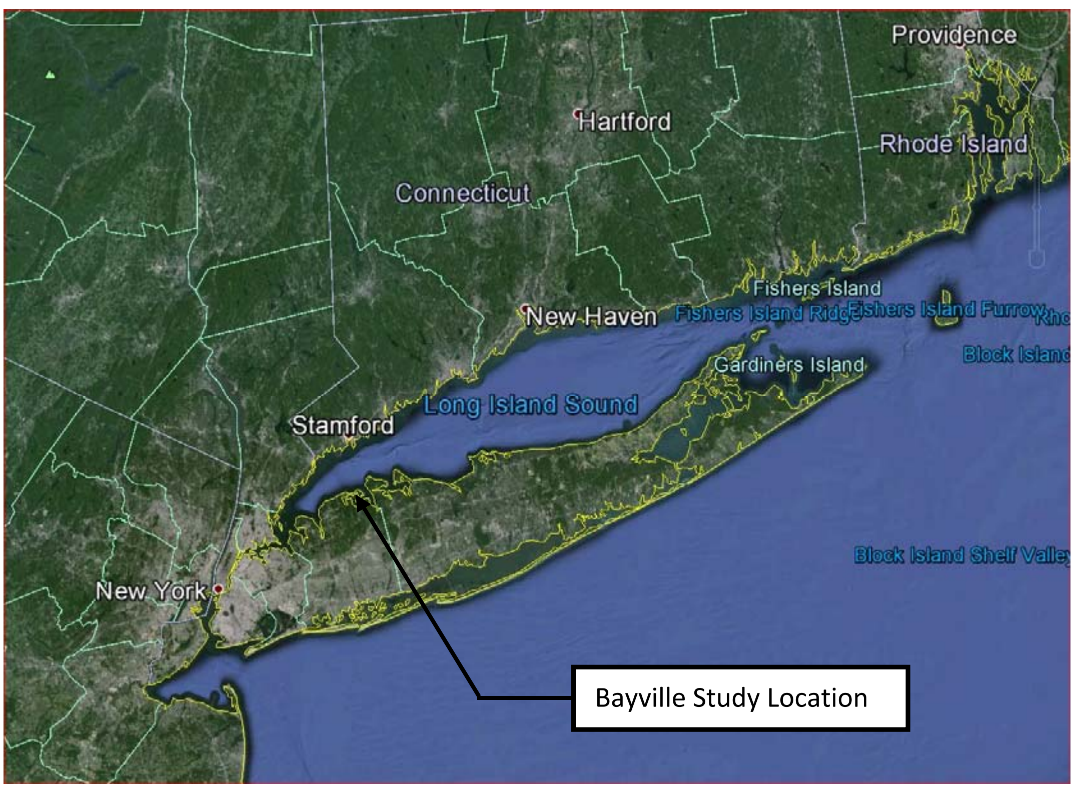


Figure 1-1. Study location

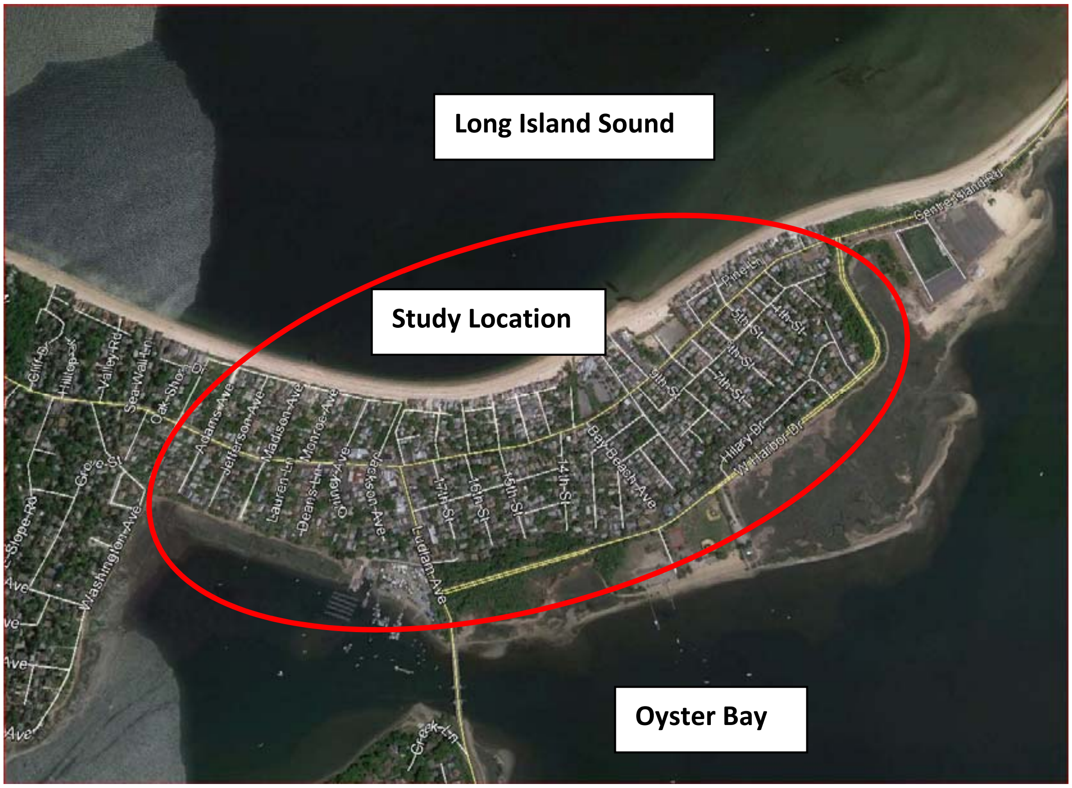


Figure 1-2. Study area aerial image

1.3 Coastal Engineering Analysis Approach

This section includes the coastal forcing conditions and the coastal engineering parameters needed for the design of alternatives and the determination of the Tentatively Selected Plan (TSP). As part of this effort a thorough review and evaluation of draft reports completed by the New York District during the early 2000's for this study area was completed. Significant work had been completed during that effort and in order to meet the accelerated schedule of this study, it was reasoned that it would be very beneficial if the early work was applicable. This study was started before the North Atlantic Comprehensive Coastal Study (NACCS) was completed and therefore the state of the art numerical modeling and resulting return period water levels and wave heights were not available. Waiting for that information was not an option due to the study schedule. The approach taken was to use the existing information provided in the earlier New York District efforts, refine as necessary, develop the coastal engineering forcing parameters and alternatives and when available compare to the NACCS model output. If the NACCS model output was significantly different adjustments would then need to be made to the alternatives leading into the TSP. If the information was similar enough to the TSP then refinements during the optimization of the TSP would be made using the NACCS model output information. As discussed later in this report the New District work was found to be close to the NACCS stage frequency curves and wave heights which verified that the TSP developed with the New York District forcing values is valid.

The New York District work was found in several draft appendices and documents that were well constructed but not finalized nor integrated. Sections from these reports and appendices have been included in this appendix and modified.

2.0 Storm Conditions

Two types of storms are of primary significance along the North Shore: a) tropical storms, which typically impact the New York area from July to October, and b) extra tropical storms, which are primarily winter storms. Extra tropical storms (northeasters) are usually less intense than hurricanes; however, tend to have much longer durations. These storms often cause high water levels and intense wave conditions, and are responsible for significant damages and flooding throughout the coastal region of the project area. Damages incurred by hurricanes and northeasters are highly dependent on storm intensity and duration, however, the relative location of a storm with respect to the north shore of Long Island is particularly important for hurricanes. This dependency on storm location is linked to storm characteristics, which determine where, relative to the storm movement, the most severe conditions exist. Tropical cyclones are characterized as small, fast moving storms consisting of a counter-clockwise spiral about the center of the storm. Winds to the right of the eye are typically most severe because the forward motion of the storm itself reinforces them. This forward storm speed can exceed 20 mph among hurricanes in the North Atlantic. Therefore, considering hurricanes travel in a general northerly direction; the south-facing coastlines are usually subjected to the largest hurricane forces. On the other hand, north-facing coastlines are somewhat protected from the strongest hurricane impacts.

Extra tropical storms are similarly characterized by counter-clockwise spiral directed toward a central low-pressure center. The radius of rotation, however, is typically orders of magnitude greater than the hurricane. Wind direction and velocity at a given coastal location depend on the relative location of the storm track. Most critical to the north shore of Long Island is passage of a northeaster to the east of the Sound, where winds blow initially from the northeast. Wind direction eventually changes with storm movement, to the north/northeast. This storm scenario produces large waves and wind setup along the north shore. Historically, northeasters with northeasterly winds occurring through numerous tidal cycles have caused the worst damages along the study area.

3.0 Winds

The most important forcing mechanism of a storm regarding impacts to a coast is wind. Wind can directly impact coastal structures but typically the most important impacts of wind are the resulting waves that are produced and the storm surge that is developed. Waves and surge typically cause the most damage to coastal development. As such the first analysis that was performed was to parameterize the long term wind field and specific storm induced wind fields.

3.1 Wind Information

Wind data are available at four weather stations located in the vicinity of the study area:

- LaGuardia Airport, Long Island, New York;
- Bridgeport Airport, Connecticut;
- Avery Point, Connecticut;
- Groton, Connecticut

Locations of the four stations are shown in Figure 3-1. Historical wind speed-direction readings were obtained for the period 1973 to 2001, approximately 30 years of record. In addition to the measured wind records, a file of continuous (hindcast) daily wind fields was available for the decade of the 1990's (1990 to 1999). This wind hindcast file includes all available historical marine surface data from buoys, ships, coastal stations, and scatterometer data, all adjusted to effective neutral 10-meter wind speeds. Quantile-quantile and exceedence plot comparisons of wind speed showed excellent agreement between the hindcasted winds and the measurements at NOAA Buoys 44025 and 44028 located south shore of

Long Island and south shore of Rhode Island (Figure 3-1). The comparisons are shown in Attachment 1 of the OCTI Storm Surge Study Report (OCTI, January 2003, referred as OCTI Report throughout this Appendix). The hindcast wind field data was used for daily wave predictions and used as the basis for sediment transport estimates since this set of data reflect more recent weather condition and is representative of over-water condition at the project site.

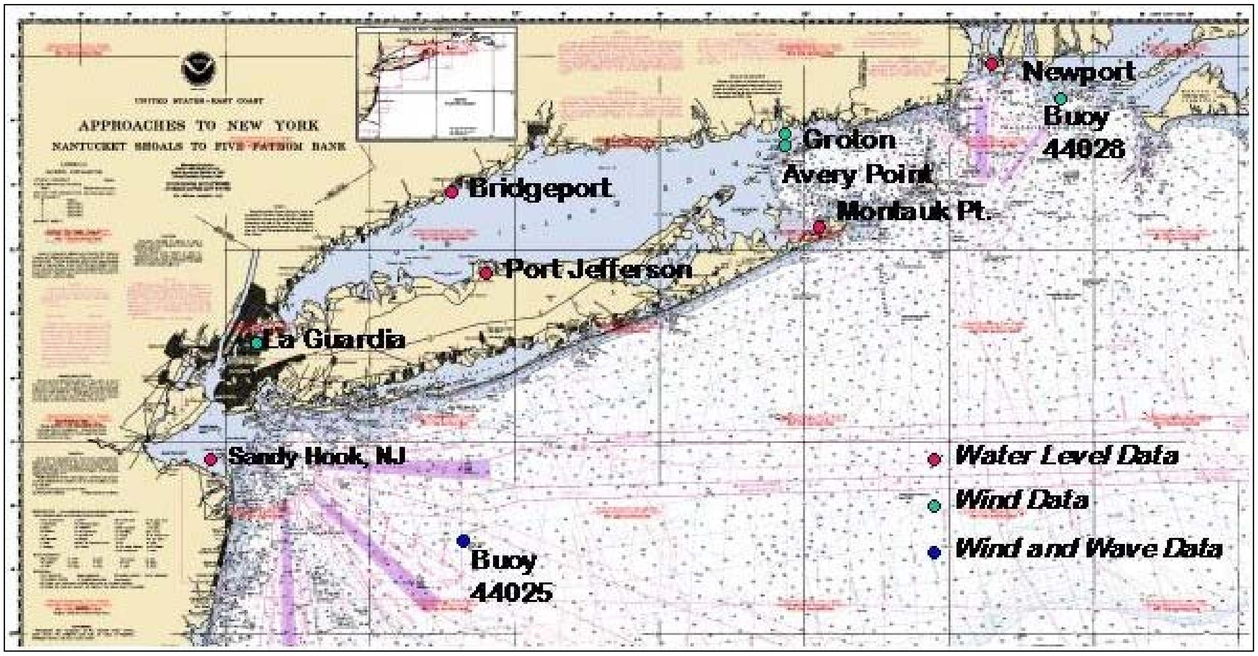


Figure 3-1. Wind Measuring Stations

3.2 Long-Term Wind

The long-term wind speed percent-occurrence vs. directions for the study site was developed based on available data recorded at the four weather stations. A plot of the hindcast wind rose for the project site is shown in Figure 3-2. This figure indicates that the annual winds are predominantly from south and west quadrant. The prevailing wind with speed range from 10 to 20 mph occur approximately 40% of the time.

3.3 Extreme Wind

Extreme wind speed-frequency statistics was developed based on the peak hindcast wind speeds. Note that all peak winds during the storm events were from the east to northeast directions except for two events (the two 1990 events where the peak wind speeds were from the NW and were below 40 mph). The extreme wind speeds (fastest mile) are presented in Table 3-1 below and plots are shown in Figure 3-3. Table 3-1 indicates the 100-year wind speed is approximately 108 mph based on the Gumbel distribution of all historical storms affecting the project site.

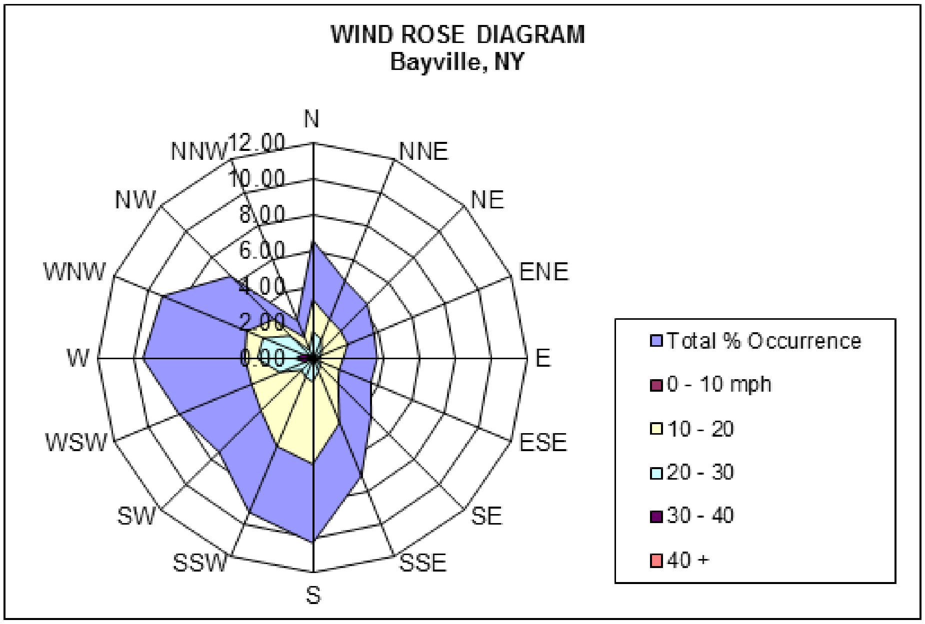


Figure 3-2. Annual wind rose diagram

Table 3-1. Storm Wind - Frequency Relationships Bayville, North Shore of Long Island, New York

Return Period (years)	All Storms (mph)	Extra Tropical (mph)	Tropical (mph)
5			
10	73.7	59.0	64.7
25	88.0	71.5	85.3
50	98.3	80.2	98.7
100	108.0	88.6	111.5
200	118.5	96.9	124.1
500	132.8	107.7	140.6

Note: Wind speed in fastest mile based on Gumble distribution

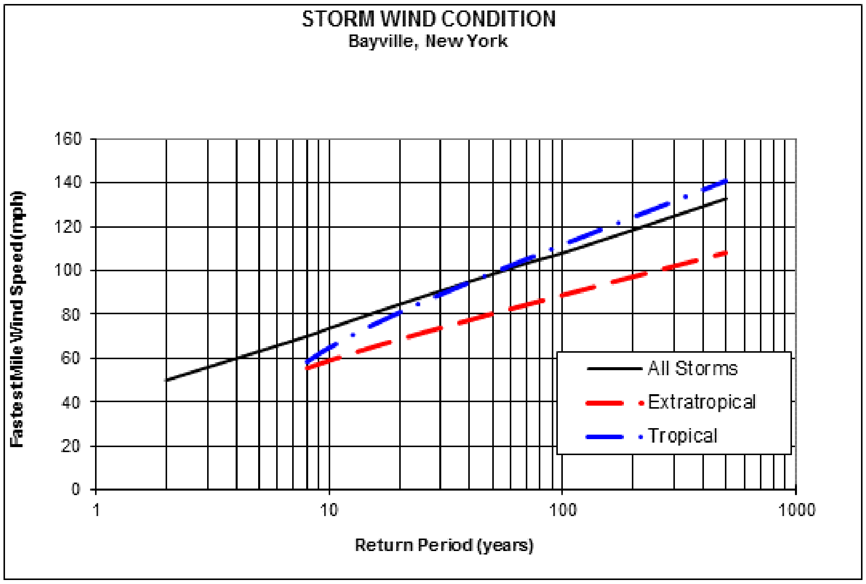


Figure 3-3. Extreme Wind-Frequency

4.0 Water Levels (Tides and Total Storm Water Level)

The water levels for the project area are dependent upon astronomic tides and surge caused by the wind and pressure fields associated with storms. As discussed the area is prone to both tropical storm systems (tropical depressions, storms, and hurricanes), and extra tropical systems which are typically nor'easters which are storms that develop along the East Coast during fall, winter, and early spring. In the below sections astronomical tides, storm surges, and total water levels (tides and storm surges) are discussed and provided.

4.1 Tides and Datums

Tides in the study area are semidiurnal with a mean tidal range of 7.1 ft and a spring range of 8.2 feet. Mean Lower Low Water is -4.28 below NAVD88. Mean Higher High Water is approximately 2.82 feet above NAVD88 based on Tide Tables published by National Oceanic and Atmospheric Administration, National Ocean Service (NOAA, NOS). In the vicinity of the study area, the maximum observed tidal height was about 8.02 ft above NAVD88 as recorded on February 6, 1978 at Port Jefferson tide station. The lowest tide was observed at the same tide station at 7.68 feet below NAVD88 on January 10, 1978. It must be realized that the period of tide recording for the Port Jefferson tide station was not very long and does not include the most recent hurricanes (Irene and Sandy). The astronomical tide elevations are summarized in Table 4-1. The astronomical tidal range at Bayville Back Bay shoreline is approximately 0.2 ft less.

The previous work by the NY District was provided relative to the NGVD29 datum, however, for this effort the datum was NAVD88. Where practical the values have been switched to NAVD88 from NGVD29. There may be instances where this was not practical or was missed. In those cases the conversion from NGVD29 to NAVD88 is NGVD29-1.08 ft=NAVD88.

Table 4-1. Astronomical Tide Elevations

Datum/Event	Elevation		
	ft-MLLW	ft-NGVD29	ft-NAVD88
Highest Observed (6 Feb. 1978)	12.30	9.10	8.02
Mean Higher High Water (MHHW)	7.10	3.90	2.82
NAVD88	4.28	1.08	0.00
Mean Tide Level (MTL)	3.60	0.40	-0.68
NGVD29	3.20	0.00	-1.08
Mean Lower Low Water (MLLW)	0.00	-3.20	-4.28
Lowest Observed (10 Jan. 1978)	-3.40	-6.60	-7.68

Notes:
1. Vertical datums provided are MLLW, NGVD29, and NAVD88.
2. Tide elevations are based on Tide Tables published by NOAA, NOS.

4.2 Storm Surge Modeling and Total Return Period Water Levels

Extreme water elevations due to storm surges and wave setup in the study area can be generated by either large-scale extra tropical storms known as northeasters or tropical storm known as hurricanes. The impact of storm surge on total water level depends on the extent to which the storm coincides with the high astronomical tides. For the Village of Bayville, the storm tide elevations were developed by the consultant firm Offshore and Coastal Technologies, Incorporated (OCTI) utilizing the finite element numerical Advanced Circulation (ADCIRC) model (Luettich, et al., 1992), which is part of the U.S. Army Corps of Engineers Surface water Modeling System (SMS). The extreme water levels were analyzed to estimate surge values corresponding to a range of return periods (2 to 500 years) using the Empirical Simulation Technique, or EST (Sheffner, et al., 1999). The following sections summarize the ADCIRC model computation grid, input parameters, model validation, and estimates of extreme water levels based on EST technique. Details of ADCIRC and EST modeling procedures are presented in the OCTI Report.

4.2.1 Computational Grid

Bathymetric data implemented for the ADCIRC water level modeling and the wave modeling were obtained from several sources. The mesh was adapted from one developed by the Coastal and Hydraulics Laboratory for regional modeling of the Long Island area (Militello and Kraus, 2001). In that mesh, open ocean bathymetry was obtained from NOAA, traditional and SHOALS survey data were obtained for Shinnecock Bay and nearshore areas at Moriches Inlet and Jones Inlet. GEODAS data were obtained for Great Peconic Bay, Great South Bay, and Long Island Sound. For the present study, bathymetry for Long Island Sound was replaced with that from a USGS database (Signell, 1998), except for the local area around Bayville and at Port Jefferson. Bathymetric surveys conducted by the U.S. Army Corps of Engineers, New York District, in 2001 were used to refine the bathymetry in the nearshore of the Bayville study area. To facilitate model validation at a nearshore location National Ocean Survey tide gauge location, bathymetric data for Port Jefferson was taken from NOAA nautical charts 12364-21 and 12362-1. The ADCIRC model mesh and bathymetry in the study area is shown in Figure 4-1. The ADCIRC Long Island Sound mesh with validation gauge locations is illustrated in Figure 4-2.

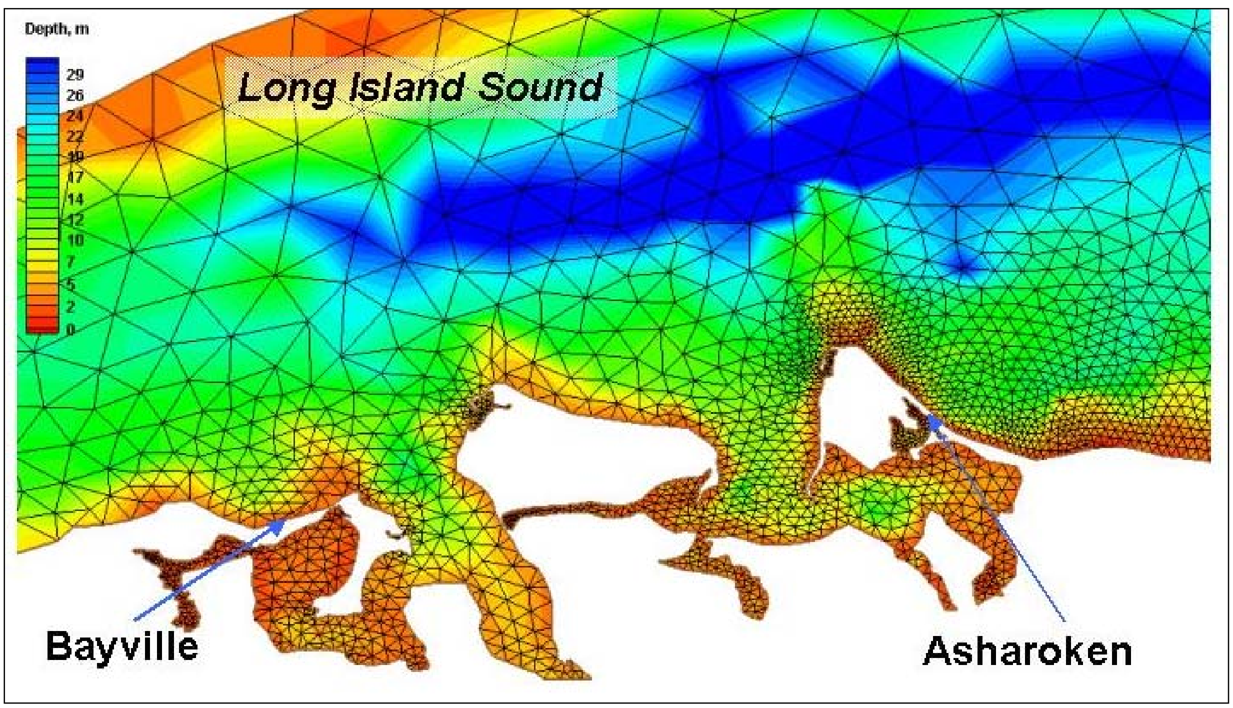


Figure 3-1. Study Area ADCIRC Model Mesh

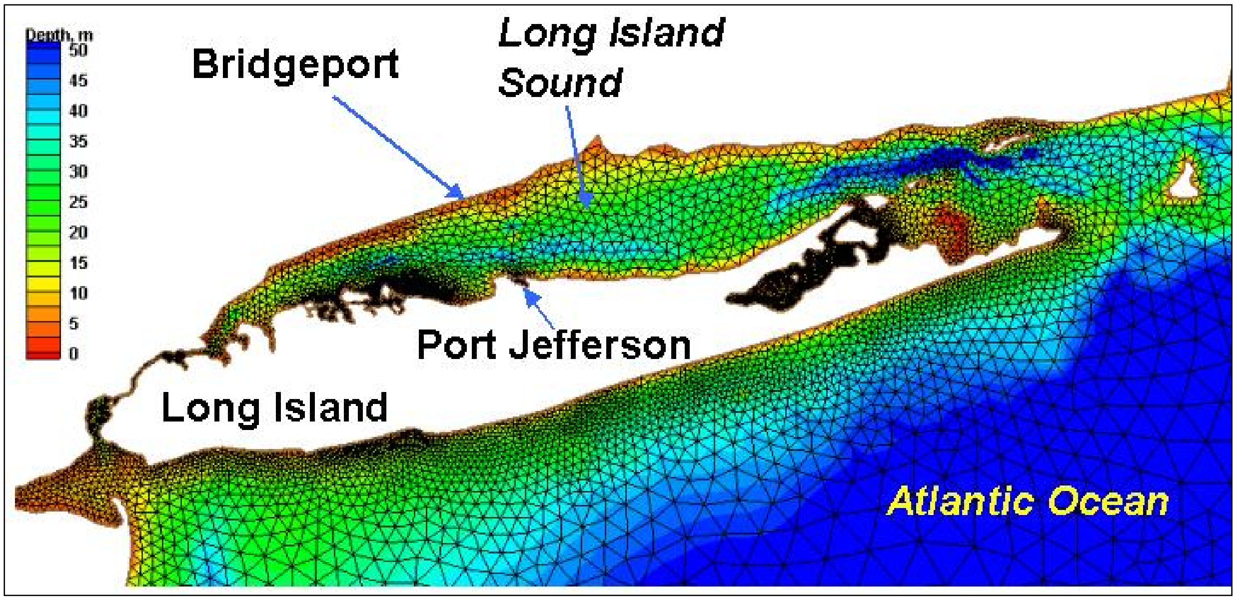


Figure 3-2 ADCIRC Long Island Sound Mesh and Validation Gauge Locations

4.2.1 ADCIRC Model Validation

Model verification was conducted in two parts. The first part was a tidal verification. A month-long run forced by only tidal constituents was conducted for the month of May 1988 and calculated water levels were compared with measurements at Port Jefferson. This location was selected for assessing the model performance because, of available historical gauging locations, it had the closest proximity to the study site. The month of May 1988 was selected after a careful review of historic data to identify a 30-day time period that was not affected significantly by storm surges.

Validation for storms was also performed to establish the accuracy of the model for combined storm surge and astronomical tide. Modification to the wind fields (magnitude of the speed) was the initial method of attempted validation. Four storms were used to calibrate the model: February 1978, December 1990, December 1992, and February 1998. During this process, the small-scale (more accurate) winds were multiplied by one factor, while the large-scale (NCEP) winds were multiplied by another. In addition, the winds over Long Island Sound were scaled separately from the remaining small-scale winds. These scaling were done because the sources of information that went into the wind models were different, with some areas, such as Long Island Sound, having less certainty. With consistent scaling of the wind fields for each of the storms, response to wind forcing was not found to be consistent (i.e. good agreement for one or two storms was not met with good agreement on the remaining storms). Therefore, another approach was taken to validate the surge model.

Including the four calibration storms used for wind field and wind stress examination, a total of nine validation storms were used to assess, or validate, the performance of the model. Four were tropical storms and five were extra tropical events. The results are shown in the validation plots in Attachment 3 of OCTI Report. Measurements were used from NOS tide gauges at Bridgeport or Port Jefferson, depending upon availability. Port Jefferson data were used for this comparison unless no data exists, in which case Bridgeport data were used. The agreement between the numerical model and the measurements is considered acceptable as compared to other published surge model applications (Committee on Tidal Hydraulics, 1980).

4.2.3 Estimates of Extreme Water Levels

Extreme storm water levels were analyzed to estimate values corresponding to a range of return periods using the Empirical Simulation Technique, or EST (Scheffner, et al., 1999). As mentioned earlier, the ADCIRC model was used to develop tropical and extra-tropical storm surge hydrographs for the Long Island Sound area. The storms that were simulated are listed in Table 3-2. The ADCIRC model was also used to simulate a 30-day period of constituent-driven astronomical tide conditions that allowed the computation of tidal constituents at 6 analysis locations within the Bayville project areas are illustrated in Figure 4-3). This astronomical tide information provided a basis for estimating the tidal component of the water level during each storm event and, based on the difference between the storm water level and the tide, the storm surge component was singled out for EST analysis.

Table 3-2. List of Hindcasted Storm Events

Storm Date (Year, Month,Day)	Storm Type	Duration (days)
09/20/1938	Tropical	3.74
09/13/1944	Tropical	3.99
08/30/1954	Tropical	2.99
09/10/1954	Tropical	3.74
09/12/1960	Tropical	3.24
19720621	Tropical	3.99
19760809	Tropical	3.49
19850926	Tropical	3.46
19910819	Tropical	3.47
19960712	Tropical	3.47
19501121	Extra-tropical	7.89
19531102	Extra-tropical	7.89
19551010	Extra-tropical	7.89
19601208	Extra-tropical	7.89
19620302	Extra-tropical	7.89
19670424	Extra-tropical	6.87
19730319	Extra-tropical	7.89
19780204	Extra-tropical	6.99
19800112	Extra-tropical	7.89
19801021	Extra-tropical	6.80
19840323	Extra-tropical	7.99
19901108	Extra-tropical	7.89
19901201	Extra-tropical	6.99
19911027	Extra-tropical	7.89
19921208	Extra-tropical	6.99
19930309	Extra-tropical	5.83
19980201	Extra-tropical	7.89

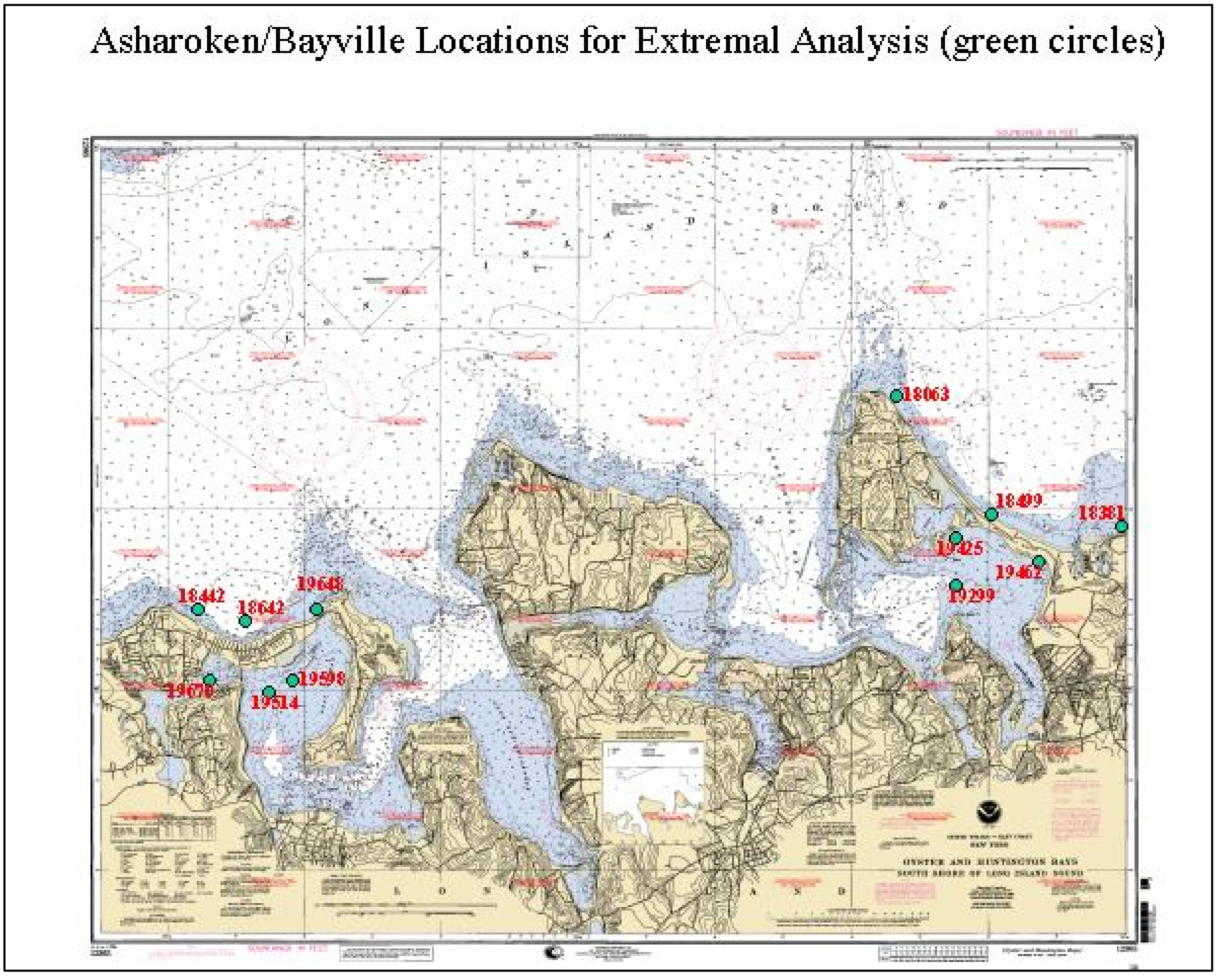


Figure 4-3. Locations for EST Analysis of Extreme Water Levels

At Bayville Long Island Sound side, wave setup was estimated using typical beach profiles and all the permutations of water level required by the EST analysis. Wave setup was estimated using the SBEACH model that employs the Goda method across the beach profile. Wave forcing is discussed in Section 5. The wave setup is used at the location along the profile where the still water depth (surge plus tide) is about 0.5 feet. The input vectors for these surge plus tide plus wave setup analyses were the peak storm offshore wave height (Hmo in feet), the peak surge (ft-NAVD88), and the peak total water level during the storm time history after the astronomical tide was phased with the peak storm surge as described above. The response vectors were the peak storm surge (ft relative to NAVD88), the peak wave setup at the shoreline (in ft), and the total water level (TWL, the sum of tide plus surge plus the wave setup in feet relative to NAVD88). The mean tide permutations were given a relative probably of occurrence of twice that given to the spring and neap tide permutations.

The results of the EST analysis of storm surge plus astronomical tide with and without wave setup are presented in Table 4-3 and Figures 4-4 and 4-5. The average wave setup is approximately 1.5 ft above the storm surge elevation. Two plots are provide showing two return period time ranges with the second figure showing a return period frequency range more likely to be applied to the project.

Table 4-3. Stage-Frequency at Bayville entire storm population included

Return Period	Bayville Long Island Sound Side				Oyster Bay (Bayville Back Bay)	
	(w/o wave setup)		(w/wave setup)			
(years)	ft-NGVD29	ft-NAVD88	ft-NGVD29	ft-NAVD88	ft-NGVD29	ft-NAVD88
2	6	4.92	7.8	6.72	8.2	7.12
5	7.6	6.52	9.2	8.12	8.8	7.72
10	8.9	7.82	10.7	9.62	9.2	8.12
15	9.6	8.52	11.3	10.22	9.9	8.82
20	10.1	9.02	11.6	10.52	10.3	9.22
25	10.4	9.32	11.9	10.82	10.6	9.52
50	11.4	10.32	12.7	11.62	11.6	10.52
100	12.4	11.32	13.4	12.32	12.7	11.62
150	13.1	12.02	13.9	12.82	13.3	12.22
200	13.6	12.52	14.2	13.12	13.7	12.62
500	15.2	14.12	15.3	14.22	15.2	14.12

*NAVD88 = NGVD29-1.08'

*Use w/wave setup elevation for economic benefit analysis

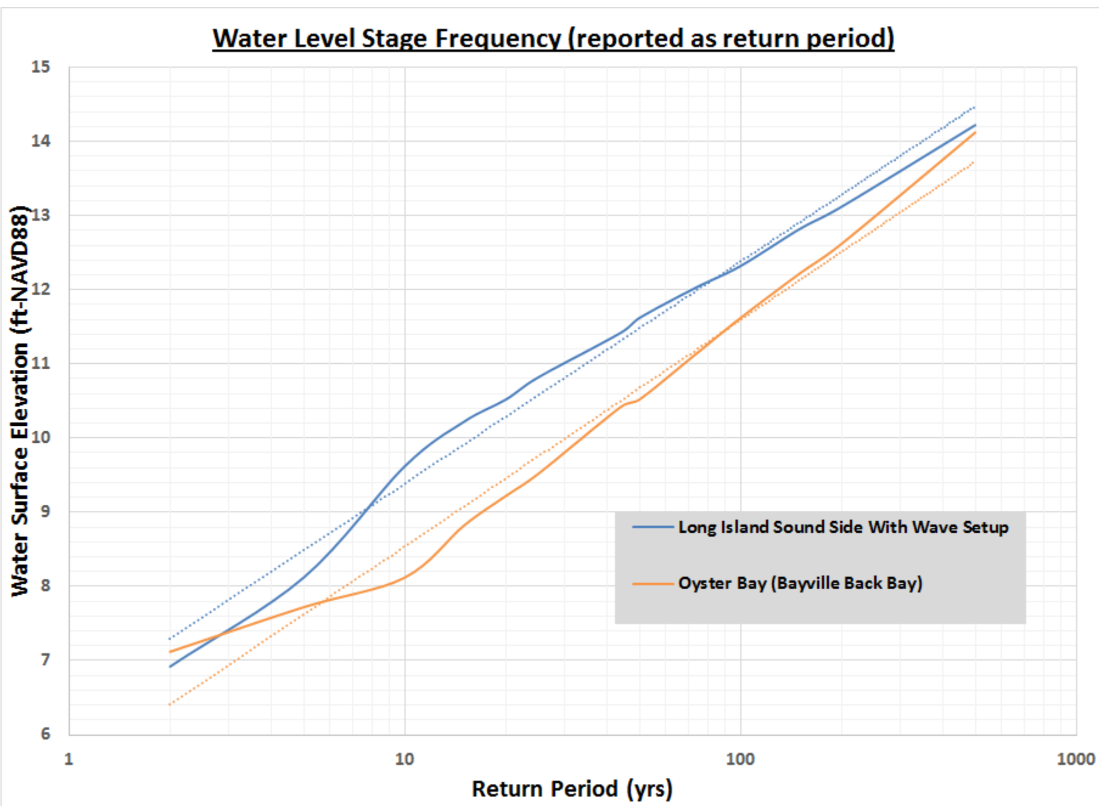


Figure 4-4. Stage frequency plot

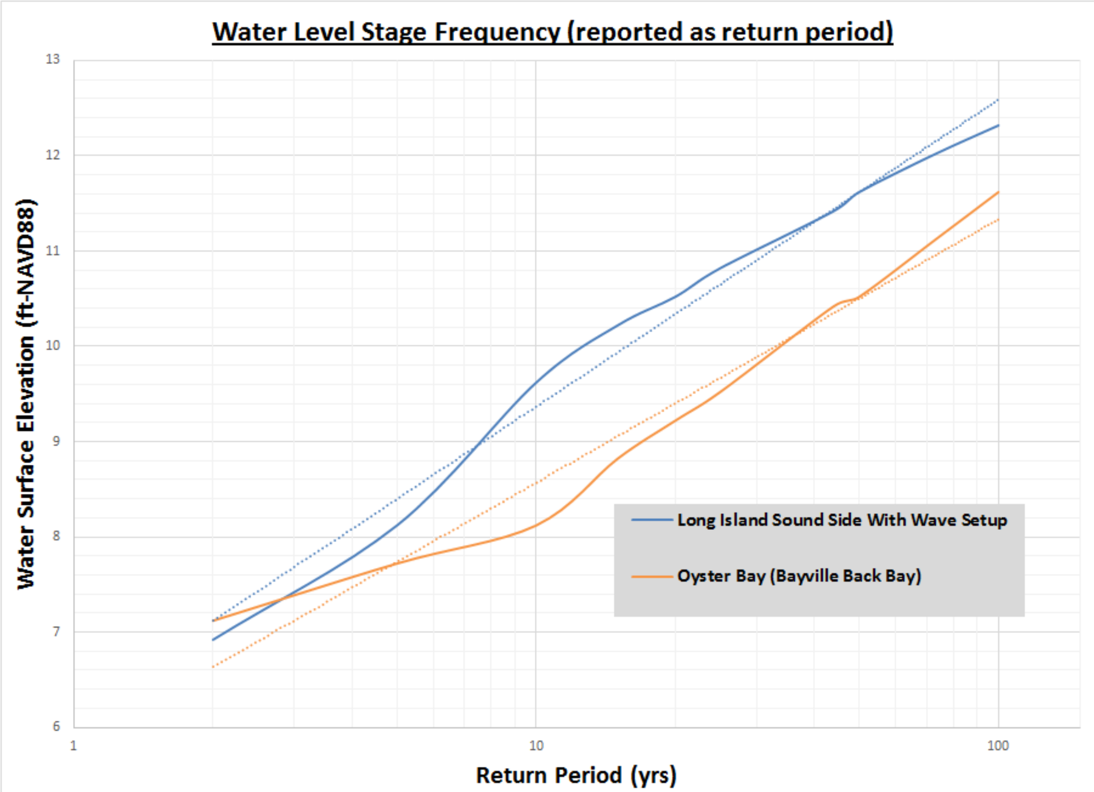


Figure 4-5. Stage Frequency plot (zoomed in to 100 year return period)

5.0 Wave Height Analysis

The North Shore wave regime is dominated by wind-generated waves within Long Island Sound. For Bayville Beach, the general shoreline is east west oriented and the maximum exposure direction for critical wave generation is northeast due to northeaster winds.

A directional spectral time-stepping wave model WAVAD (also known as WISWAVE) was applied to determine extreme storm wave conditions for the same events modeled with ADCIRC, and to characterize long term wave climate by modeling ten-years of continuous waves during the 1990's. In addition, local wind wave hindcast model based on SPM 84 formula was used to predict the deep-water storm wave condition. Nearshore waves were calculated based on depth-limited wave shoaling at controlling depths of 0, 10, and 20 ft NAVD88 contours and compared with SBEACH nearshore wave model output.

5.1 Wave Hindcast Based on WISWAVE Modeling

Bathymetric data implemented for the WISWAVE modeling were obtained from several sources. Open ocean bathymetry was obtained from NOAA nautical charts and bathymetry for Long Island Sound was used from a USGS database (Signell, 1998). Three levels of nesting were used to generate the wave data at a resolution appropriate for the project (see OCTI Report, Figures 11,12,13).

The wave model was driven by the 27 storm wind fields described in the OCTI Report as well as by a 10-year continuous set of hindcasted winds. The storm wind fields were hindcasted on an hourly time increment so as to accurately resolve the time-varying characteristics of rapidly moving storm systems. The 10-years of winds were hindcasted on a 6-hourly time increment that is meant to define the long-term wave distribution in the area of interest. The spatial coverage of the wind input grids is the same as the wind grids described in the OCTI Report.

The results from the storm simulations and the 10-year continuous wave hindcast are archived at output location shown in Figure 5-1. The locations are generally in water depths of 41 feet or greater relative to mean lower low water. During normal tide conditions, the average water depth is about 44 feet, and during storms at these locations the water depth is 49-51 feet deep or greater. These water depths, in combination with the rapidly dropping bottom seaward of the locations, are outside of depth-limited breaking conditions and are within the range of applicability of the wave model.

5.2 Long-term Wave Statistics

The wave model results from the 10-year (WISWAVE) continuous hindcast (1990-1991) were analyzed to estimate long-term wave statistics at the output location offshore of Bayville. The bivariate (wave height-period) and trivariate (wave height-period-direction) frequency-of-occurrence tables for each site were calculated from the output. Wave heights are H_{mo} (zero moment wave height in feet), wave periods are peak spectral peak periods, T_p, in seconds, and wave directions are the direction from which waves travel in degrees clockwise from north. The results are presented in Attachment 5 of the OCTI Report. An annual wave rose diagram is shown in Figure 5-2. Due to the orientation of project shoreline, only waves from NW clockwise to ESE would reach the nearshore site and approximately 65% of the sea is calm due to winds blowing towards offshore. As shown in Figure 5-2, long-term waves are evenly distributed with larger waves from NNE and NE directions.

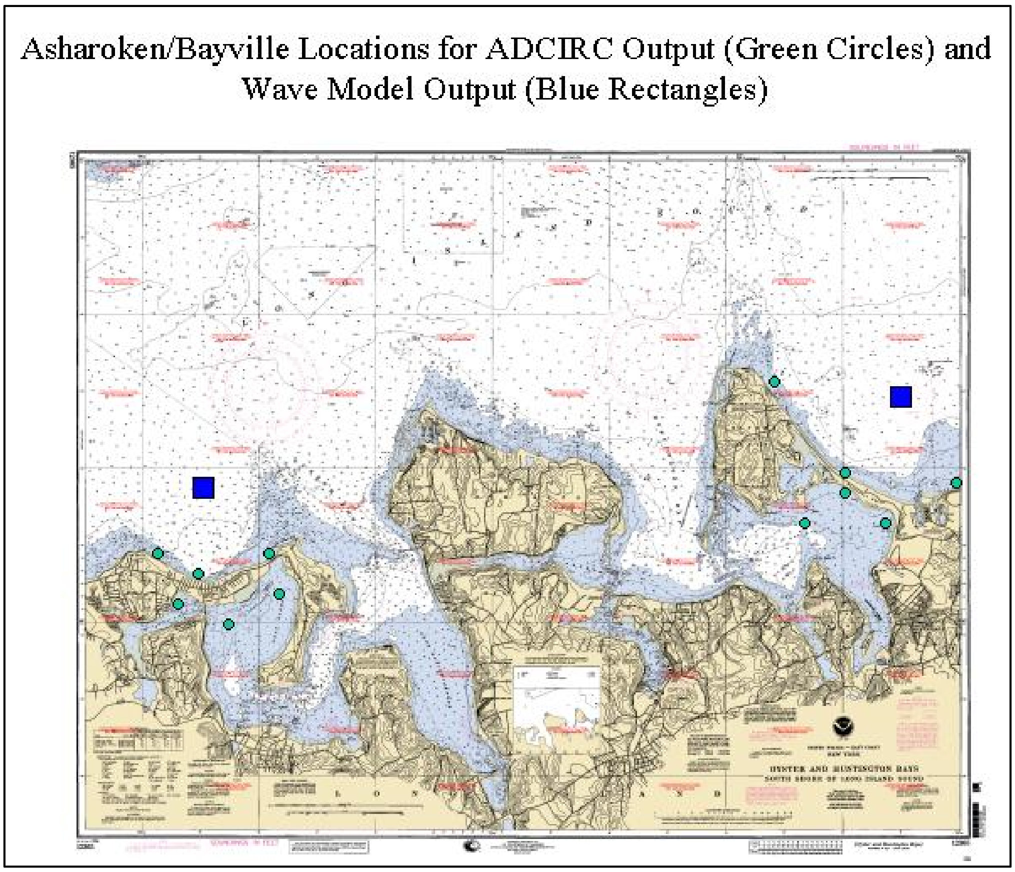


Figure 5-1. Locations of Wave Model Output

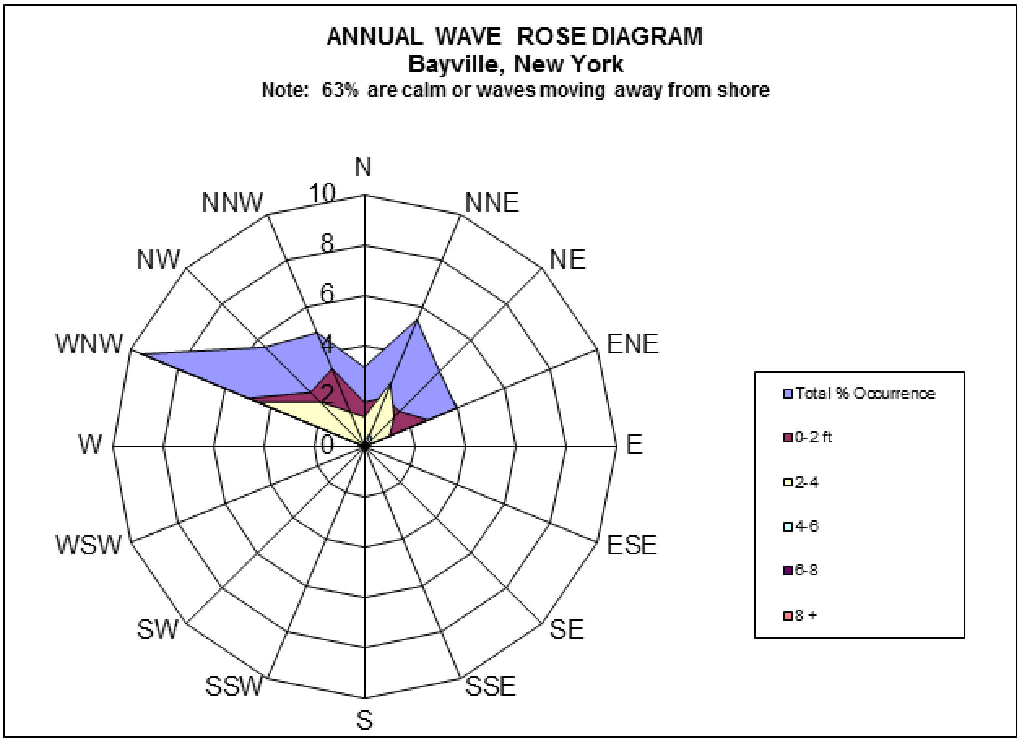


Figure 5-2. Annual Wave Rose Diagram

5.3 Extreme Wave Statistics

Extreme wave statistics at the offshore locations were determined by extracting peak wave heights for each storm event from the hindcasted time history. Those peaks were then analyzed with a best-fit Gumbel distribution using the ACES system to determine extreme wave heights for storm return periods. Results are fetch-limited at the higher return periods based on the shallow water wave prediction curves in the Shore Protection Manual and a fetch to the northeast. The predicted extreme waves are presented in Table 5-1. The wave heights taken from the hindcast modeling effort appeared to be high based on personal experience so a check was performed and discussed in Section 4.4. To further investigate this offshore storm wave height-frequency was hindcasted based on storm wind-frequency (Table 3-1), fetch length, and average water depth along the critical wind direction from Northeast. The significant wave height and corresponding peak wave period were calculated based on Equations 3-39 and 3-40 of SPM 84. The average fetch along the critical wind direction (northeast) is approximately 15.0 miles and the average water depth used for hindcast is 75 ft NAVD88 based on NOAA NOS chart. The hindcast waves are shown in Table 5-1 and shown graphically in Figure 5-2. The hindcast waves based on storm wind and fetch length are used for design calculation since they were lower and were believed to better represent the site condition.

Table 5-1. WISWAVE Hindcast – Deep Water Storm Wave Frequency

Return Period (years)	Based on Storm Events and Gumbel Distribution			Hindcast Based on Storm Wind and Fetch Length (ft)
	Tropical (ft)	Extratropical (ft)	All Storms (ft)	
5	9.5	6.7	5.9	7.8
10	11.9	9.4	9.7	9.3
25	14.7	12.2	13.6	11.2
50	16.8	14.1	16.2	12.6
100	18.8	16.0	18.7	13.8
200	20.8	17.9	21.1	15.2

Note: 1.Wave heights shown are Hmo or Hs

2. Hindcast waves based on storm wind and fetch length were used for design.

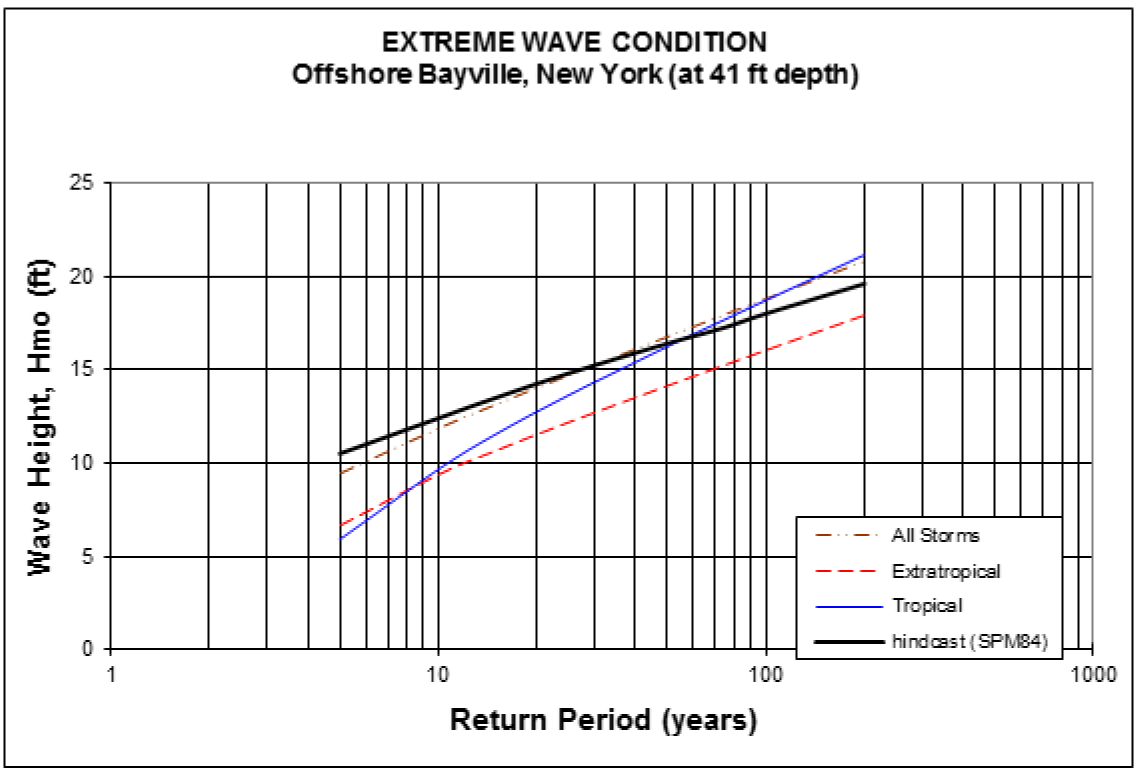


Figure 5-2. Extreme Wave-Frequency Curve

5.4 Nearshore Wave Condition

As deep water storm waves propagate onshore, it start refraction, shoaling, and breaking when feeling the nearshore bottom slope. Deep water waves continue to break as water depths become shallower, which is defined as depth-controlled wave. Nearshore wave heights are calculated based on shoaling of hindcasted deep water waves along an average 1 vertical on 100 horizontal foreshore slope at 0, 10, and

20 ft contour. The calculated wave heights are compared with peak storm wave (of all storms) using SBEACH transformation (see OCTI Report) with reasonable agreement as shown in Table 5-2.

Table 5-2 Bayville Nearshore Wave Condition

Return Period (years)	Based on SPM84 Shoaling Calculation of Hindcast Waves at Various Bottom Contours			Based on SBEACH Transformation of Peak Storm Waves
	0 ft NGVD	-10 ft NGVD	-20 ft NGVD	
5	6.1	13.1	15.6	13.4
10	7.1	14.2	18.7	17.9
25	8.3	15.4	21.7	23.3
50	9.1	16.2	22.6	23.8
100	9.9	17.0	23.4	24.2
200	10.8	17.9	24.4	24.2

Note: Nearshore waves are depth-controlled breaking waves

The nearshore values provided in the New York District draft report appear high and the nearshore wave height calculations/transformations could not be located for verification. This situation carriers over into the next section related to design wave conditions as well. Given that the wave heights are depth limited and will ultimately control the design wave heights, and that they appear to be conservatively high, they were used for this effort. Further wave height calculations will be taken during the next phase of the study.

5.5 Design Wave Condition

The storm wave-frequency based on limited fetch hindcast are used as design wave for project calculations. The design deep water and nearshore waves are summarized in Table 5-3.

Table 5-3. Design Wave Condition

Return Period (years)	Offshore (deep water) waves		Nearshore (shallow water) Waves (ft)		
	Hs (ft)	Tp (sec)	0 ft NGVD	-10 ft NGVD	-20 ft NGVD
5	7.8	5.3	6.1	13.1	15.6
10	9.3	5.6	7.1	14.2	18.7
25	11.2	6.0	8.3	15.4	21.7
50	12.6	6.3	9.1	16.2	22.6
100	13.8	6.5	9.9	17.0	23.4
200	15.2	6.7	10.8	17.9	24.4

Note: Nearshore waves are depth-controlled breaking waves.

6.0 Sea Level Rise Analysis

Given the uncertain future regarding the rate of sea level rise (SLR), USACE regulation requires that projects impacted by coastal/tidal forcing be studied and analyzed with three rates of sea level rise. The range of rates includes the historical rate of sea level rise (approximately 1 foot/century in the Northeast) to the high rate curve which is approximately 5 feet/century in the Northeast. The determination of the increased SLR rate for this study area was taken from USACE's climate change web page (http://www.corpsclimate.us/ccaceslcurves.cfm). The web page has a SLR calculator for numerous stations along the coast that allows the user to alter various input considerations. For the study area the three SLR rate curves required by USACE regulation have been shown in Table 6-1 and Figure 6-1.

Table 6-1. SLR elevation increases (2018 to 2100)

8514560, Port Jefferson, NY
NOAA's Published Rate: 0.00801 feet/yr
All values are expressed in feet relative to NAVD88

Year	USACE Low	USACE Int	USACE High
2018	0.02	0.08	0.27
2020	0.03	0.10	0.33
2025	0.07	0.17	0.48
2030	0.11	0.24	0.65
2035	0.15	0.32	0.84
2040	0.19	0.40	1.05
2045	0.23	0.48	1.28
2050	0.27	0.57	1.52
2055	0.31	0.67	1.79
2060	0.35	0.77	2.07
2065	0.39	0.87	2.37
2070	0.43	0.98	2.69
2075	0.47	1.09	3.03
2080	0.51	1.20	3.39
2085	0.55	1.32	3.76
2090	0.60	1.45	4.16
2095	0.64	1.58	4.57
2100	0.68	1.71	5.00

Print Table

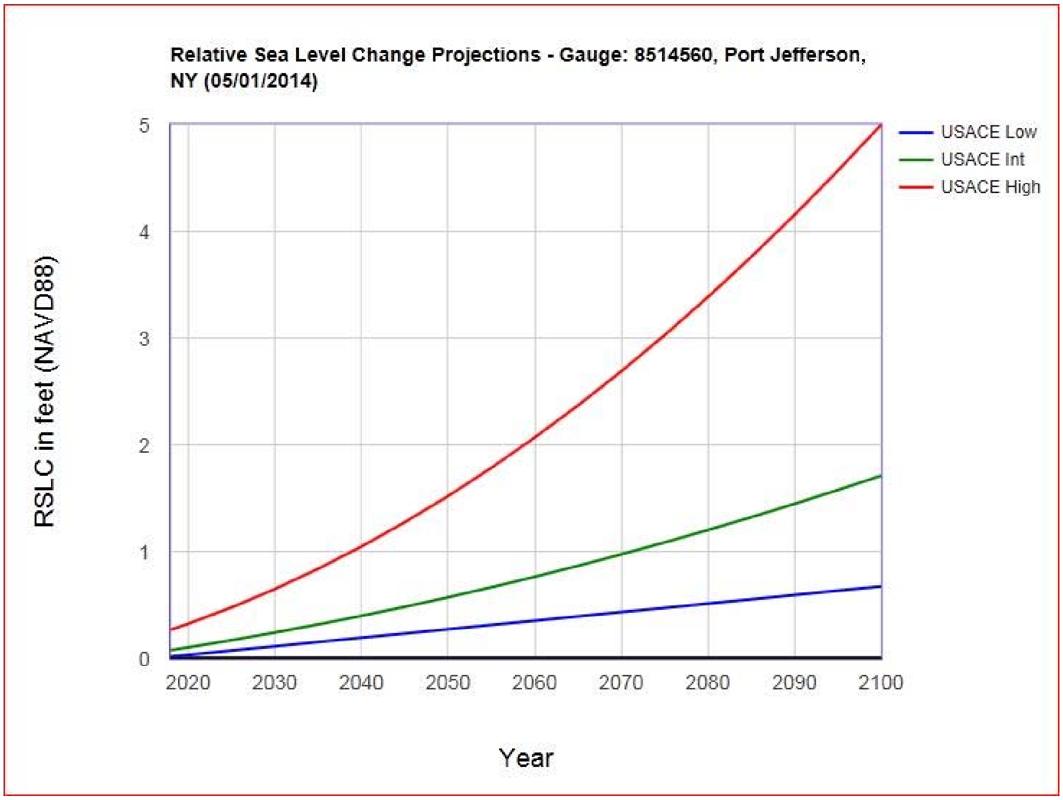


Figure 6-1. SLR rate plot for three curves.

For this study the approach was to design the alternatives for the historical rate of sea level rise and then determine the impacts of the higher rates of sea level rise on those alternatives. Generally along the east coast these types of analyses has shown that potential benefits increase with higher sea level rise rates since property and infrastructure is impacted sooner and more often during the study life. It has also been generally found that the alternatives that were selected under the historical SLR rate are still the ones selected under the higher SLR rate analysis. As some examples if a beach fill is the selected alternative it will still be the alternative under the higher SLR rate scenarios but the volume of renourishment sand will increase. The additional cost of more sand is offset by the aforementioned increase in benefits. If a seawall or sea dike was the selected plan it has been shown that increasing the elevation of those structures at a later date during the project life was the most effective approach of dealing with increased sea level. Essentially increase the wall or sea dike elevation in the future according to the level of increase prescribed by the increase in water level. With those future elevation increases comes considerations that must be addressed at the time of initial design and planning such as wall foundation conditions (ensure wall is still stable with increased elevation), ensuring the top of the wall is designed with future elevation increases considered, increased land access for wider sea dike foot prints, etc.. Once again it has been shown that if those consideration are planned for the selected plans generally do not change.

With the discussion in the above paragraph in mind, each alternative was investigated for impacts of increased SLR rates. As shown in Table 6-1 and Figure 6-1 over the 50 year economic life (around the 2070 mark) the low or historical curve shows about a half foot of increase, the intermediate curve shows about one foot increase and the high curve shows about three feet of increase. For the historical rate and the intermediate rate the increases are easy to deal by increasing the alternative elevations/volumes slightly and in reality fall within the error of analysis and within the optimization effort that will occur in the next phase of the study. This will be demonstrated in Section 7.0 in the comparison to the NACCS model results. For the high rate of SLR the 2.5 to 3 feet of increased sea level is more challenging given the nature of the study area and the low, bowl shaped topography. Increasing wall heights is structurally feasible and can be addressed with future wall lifts as long as the foundation conditions are pre-constructed to accept such lifts. The controlling issues really become more of a planning type societal issue with a significant consideration being residual risk.

Beyond the 50 year economic life the longer project life horizon is similar with the low and intermediate curves showing less than 2 feet of increase (which can be fairly easily addressed) but the high curve is in the range of 5+ feet. That will likely not be addressed with an engineering solution and given the low lying nature of the study area, the normal astronomical tidal conditions would be at or above the land elevation. This would essentially mean the Bayville area would be at or perhaps below sea level setting of conditions similar to New Orleans.

7.0 North Atlantic Comprehensive Coastal Study (NACCS) Comparison

After reviewing the work done it was concluded that the work was of high quality and thorough and absent the NACCS information, was similar to the type of study that would be undertaken for a present day study. The modeling approach and abilities have improved since the 1990's and early 2000 time frame but the general approach would be similar. As discussed in Section 1, to meet project schedules existing information from the New York District study efforts was used for designing the various alternatives and selecting the TSP. It was also agreed upon by the PDT to use the NACCS model information to refine and optimize the TSP in the next study phase. With that approach though it was decided that when the NACCS model data was available that a comparison of that information to the previous study information would be made to make sure there was not a significant difference in water levels and wave heights used that would ultimately change the TSP or negate the project feasibility. That comparison is provided in the following sections. It was shown that the information used from the previous study was adequate for selecting the TSP, for determining project feasibility, and was in fairly good agreement with the NACCS model data.

7.1 Water Level Findings

The first forcing condition reviewed was frequency of water level or return period water levels. The applicability of the NY water level information for using in the alternative design and selection of the TSP was done by a simple comparison to the NACCS data. Return period water level from the NACCS study was accessed through the CHS website. Three data saves points were used from both Long Island Sound and Oyster Bay. The save points were () and the locations can be seen in Figure 7-1. The data from each of the points was within a couple of inches so any of the points from either water body could have been used to represent the return period water levels for the respective water body but since the data was already downloaded the data was averaged for each of the two locations. Water level from the NY District report was taken from Table 4-3 (of this report). The data from that table and from the NACCS report has been provided in Figures 7-2 and 7-3. Figure 7-2 shows the plot of return period water levels for Long Island Sound and Figure 7-3 shows the water level information for Oyster Bay (Bayville Back Bay).

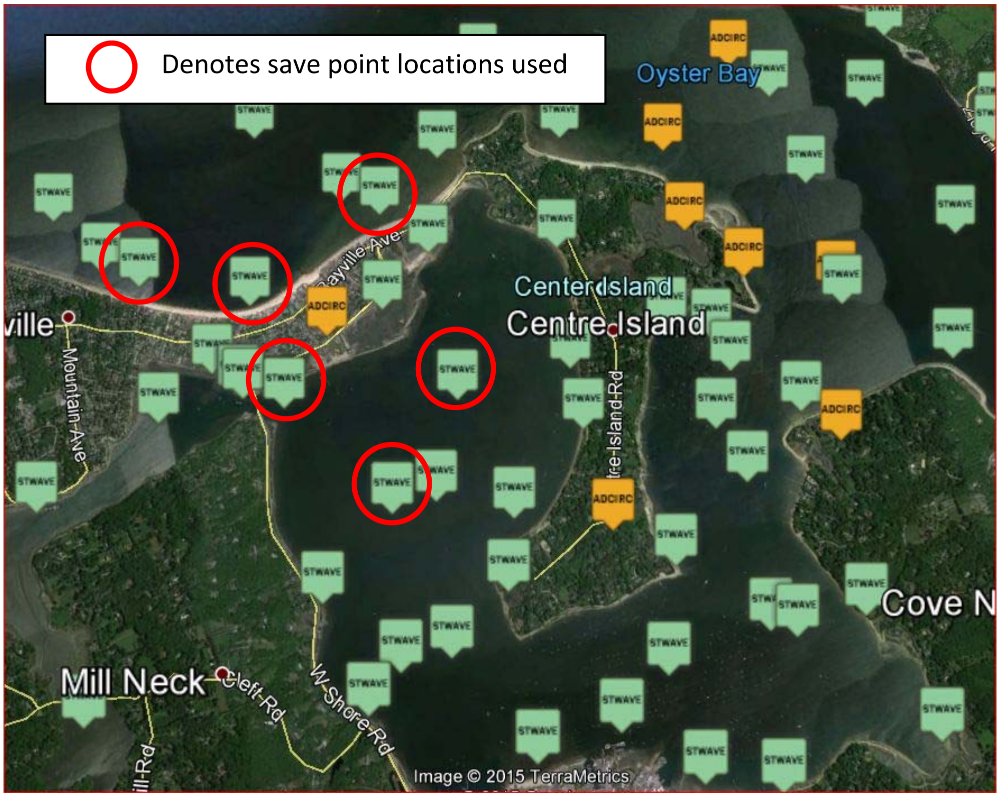


Figure 7-1. CHS save point data locations

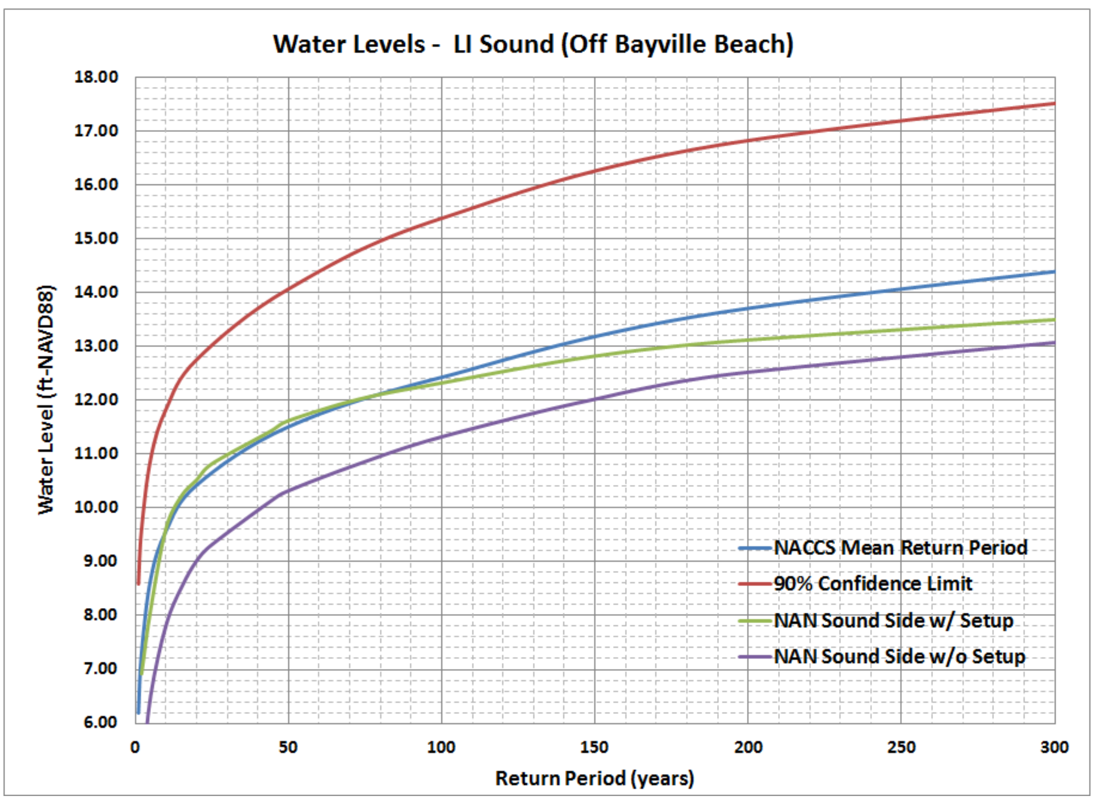


Figure 7-2. Return period water levels LI Sound

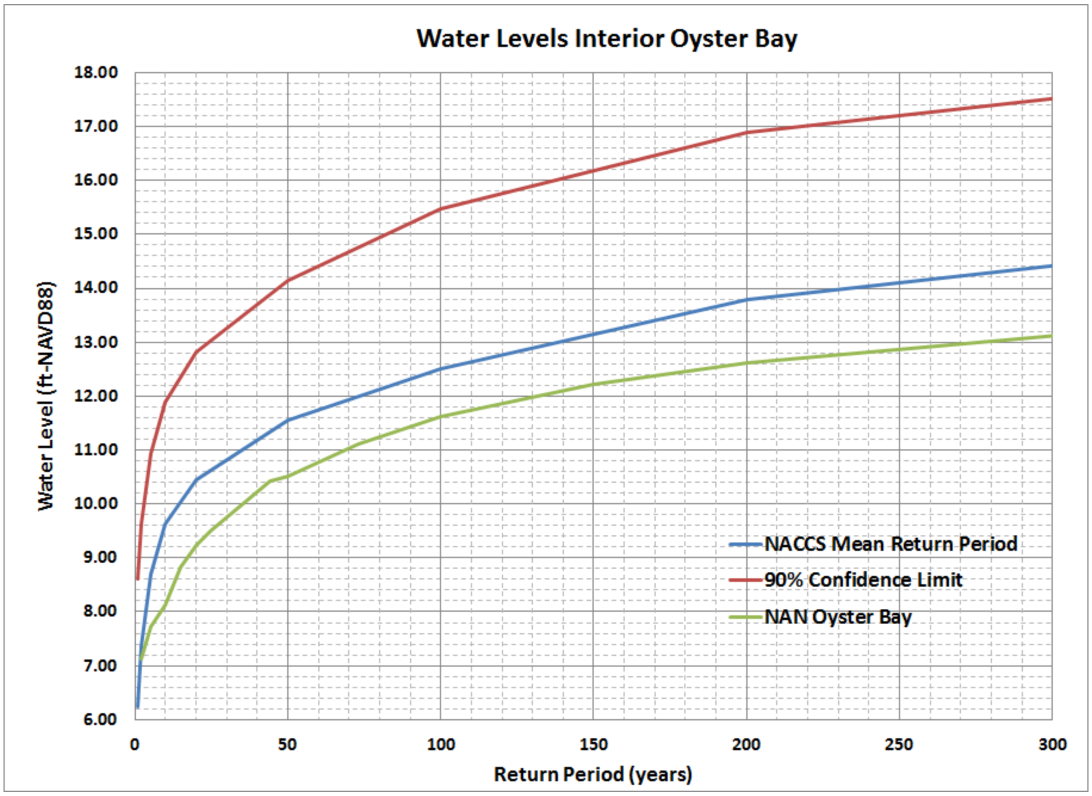


Figure 7-3. Return period water levels Oyster Bay (Bayville backbay)

As can be seen the water levels for both the Long Island Sound side and the Oyster Bay side are in fairly close agreement. Generally for the Long Island Sound side the water levels for the 25, 50, and 100 year water levels are nearly identical with no real discernable differences. There are factors that must be considered though and they will be looked at further in the next study phase. The first is the level of localized wave set up that is included in the NACCS study. The save point data from the CHS (NACCS) was taken from near shore but the wave breaking and resulting setup calculations will be looked at closer and will be refined if necessary to achieve the final design water levels. The changes in water level should not be significant in comparison to the existing level of study and in comparison to the TSP optimization. Further, the adjustments will fall well within the error bands of the NACCS data that have been included in Figures 4 and 5 as the 90% upper bound confidence limit. As demonstrated in the NACCS/CHS data there exists an approximate two foot (at the 90%) confidence band on either side of the reported data mean. Generally the mean of the data is what is first looked at during a study by the error band or confidence bands must be considered during the TSP optimization and when reporting residual risk. This will be done in the next study phase.

Similar to the Long Island Sound water levels, the Oyster Bay water levels were close to the NACCS/CHS data, however there was more of a difference than on the Long Island Sound side. For the 25, 50, and 100 year return periods the NY District report water levels were 1 to 1.5 feet lower than the NACCS/CHS data. As with the Long Island Sound data the discrepancy will be addressed and the NACCS/CHS data will be used for the TSP optimization. It was determined that the 1.5 foot difference on the Oyster Bay side was not significant enough to alter the TSP or the elevations of the TSP significantly. The difference in barrier elevations caused by the difference in model data falls within the differences that will be evaluated during the optimization effort, during the SLR consideration, and the final designs of the project.

7.2 Wave Height Findings

Similar to the water levels it was found that the wave height analysis completed in the draft appendix from the NY District was well done and met the expected coastal engineering rigor. The approach was to use both a numerical wave model that covered Long Island Sound and for more localized waves analytic or formula based calculations. As discussed the wave height values in the NY District effort from the wave model appeared to be high, with the analytical formula based results being more reasonable Presented in Table 7-1 are the wave heights from the NY District work and the NACCS effort for the "offshore" location. The NACCS data save point selected was near the wave model save point shown in Figure 5-1 offshore of Bayville.

As shown in Table 7-1, the wave heights in the previous effort were higher than the NACCS results when comparing the previous work to the mean values from the NACCS effort. However, as with the water level information, confidence or error bands were provided for the NACCS information. For the upper end of the confidence band (90% confidence band) the difference between the NY District work and the NACCS work is small, especially at the lower frequency return periods. Based on this information the offshore wave conditions used from the NY District report were shown to be reasonable for this level of study and the selection of the TSP. Selection of the design wave heights will be looked at in detail in the next phase of study.

Table 7-1. Wave height comparison.

	Wave Height (Hmo ft)		
		NACCS Wave Information	
Return Period	NY Study	Mean	90% Conf. Limit
1		4.2	8.3
2		5.4	9.5
5	7.8	6.6	10.7
10	9.3	7.5	11.7
20		8.0	12.2
25	11.2		
50	12.6	8.5	12.6
100	13.8	8.6	12.7
200	15.2	8.7	12.8

8.0 Shoreline Processes and Sediment Budgets

8.1 Geomorphology

Long Island belongs to the inner part of the Atlantic Coastal Plain. Part of the deposits of the island are true coastal plain deposits, whereas, the greater portion of both the surficial and underlying materials are of Pleistocene age and represent morainal and outwash accumulations associated with the continental glaciers. Cretaceous forms underlying those of Pleistocene age are exposed at several locations within the study area. The extensive unconsolidated sediments underlying the study area are of Cretaceous, Pleistocene and Recent origin, ranging from fine silts and clays to sands and coarse gravel.

The North Shore of Long Island consists of features that include beaches, bluffs, dunes, wetlands, and barrier landforms. Topographic character and sediment composition of the area determines the manner in which these landforms interact with the marine environment and directly impact coastal erosion and flooding.

The Long Island Sound shoreline vicinity is highly irregular, indented by several deep harbors and bays. Peninsulas or necks extending into Long Island Sound separate these bays and harbors. The narrow beaches of the necks are backed mostly by bluffs composed of Manhasset formation, till and outwash deposit. Bluff heights are generally low (approximately 30 feet) along the westernmost portion of the study area. Heights generally increase in an easterly direction, ranging from 75 to 110 feet in the vicinity of Lloyd Point, Eatons Neck, and Nissequogue. Eroded bluff material has formed small pocket beaches in many locations between the projecting points of the necks. Additionally, material eroded from the necks and offshore islands has been deposited as spits, baymouth bar, and tombolos.

Eroding bluffs and headlands to the east and west of the project site contribute sediment to the littoral environment with much of its material transported into the pocket beach or lost offshore during storm events. In general, grain sizes along the project shoreline vary significantly ranging from fine sand to gravel. Median sediment diameter in the area range from 0.14 to 58.0 mm. For Bayville, the median sediment diameter is 2.0 mm.

8.2 Historical Shoreline Change

Historic shoreline changes in the study area have been affected by coastal construction activities, such as bulkheads, groins, and artificial beach nourishment. Artificial nourishment project was less frequent at Bayville, only one completed in 1947 based on available record. New bulkheads fronting houses have been constructed after storm damage. Elevations of bulkheads in the study area range from +12 to +14 ft NGVD. Stone groins were constructed at Oak Neck Point to the west of the project site and at Rocky Point to the east of the project site. The groins may have slowed the erosion of the headlands and reduced the sediment transport into and out of the project area.

Shoreline change analyses were conducted for shore segments at the project site based on historic aerial photos. Shoreline positions taken from aerial photoraphs were digitized and transposed to depict shoreline evolution at the project site based on descriptions in North Shore of Long Island Beach Erosion Control Reconnaissance Study Report, September, 1995. Historical shorelines were obtained for the years of 1976, 1980, 1990, 1995 and 2002. All shorelines correspond to shoreline position in either the months of March or April. The data analyzed are within recent time period and is a better representation of current and future coastal processes. Aerial photos were reproduced with 1”=400’ scale which would yield a consistent accuracy range between +4 ft and –4 ft due to digitization and average error range of +/- 10 ft due to judgement of shoreline position on the aerial photo. The percent error ranges from 3% to 5% depending on the length of transects measured from shoreline position to the baseline.

The Mean High Water (MHW) shoreline positions were used to represent the typical shoreline locations of the available data. An artificial baseline was established landward parallel to the shoreline with station distances on the baseline range from 0 to 12,400 ft. Transect lines were set up at every 400 ft station normal to the baseline. The MHW shoreline positions were measured at each transect and the corresponding shoreline change rates are summarized in Table 8-1.

As shown in Table 8-1, MHW shoreline positions from baseline were measured along 32 transect stations at 400 ft spacing for the years 1976, 1980, 1990, 1995, and 2002. Ten time-period combinations based on the five historic records are shown in the Table. The following is a summary of the findings based on the historic shoreline evolution analysis:

- The project area shoreline is generally stable with minor erosion or accretion located at isolated sector. Overall historical shore evolution at project site is accresive;
- Historical shoreline accretion is primarilly on the western shore while the historical erosion occurred primarilly on the eastern shore;
- The project shoreline was stable with zero net shoreline changes in the two short periods (1976-1980 and 1990-1995);
- Higher shore evolution rate took place in the two periods of 1980-1990 and 1995-2002 with net accretion rate of 1.4 and 1.5 ft/year;
- The long term shore evolution is accresive at the rate of approximately 1 ft/year.

In conclusion, the project shoreline is relatively stable historically. There are isolated sections of erosion while the net shoreline evolution is accresive. The sources of sediment transport in the project shoreline is primarily due to erosion of headlands to the east and west of the project shoreline which is functioning as a pocket beach. Net shoreline change rates varied historically from 0.0 to +1.5 ft/year. The average net accretion rate at the project shoreline is 1.0 ft/year. These processes have been occuring under the historic or low rate of SLR previoulsy dicussed in Section 6.0. If the rate of SLR increases the accretionary trend may slow or reverse.

Table 8-1. Historic shoreline postion change

BAYVILLE HISTORIC SHORELINE CHANGES

Profile	MHW Shoreline Position from Baseline (ft)					Annual Shoreline Position Change (ft/year)									
Station	1976	1980	1990	1995	2002	1976 to 1980	1976 to 1990	1976 to 1995	1976 to 2002	1980 to 1990	1980 to 1995	1980 to 2002	1990 to 2002	1990 to 2002	1995 to 2002
0	-	-	301	314	314	-	-	-	-	-	-	-	1.1	1.1	0.0
400	523	549	578	579	570	1.9	3.9	2.9	1.8	2.9	2.0	1.0	0.1	-0.7	-1.3
800	510	534	570	558	558	1.7	4.3	2.5	1.8	3.6	1.6	1.1	-1.0	-1.0	0.0
1200	490	496	535	530	530	0.4	3.2	2.1	1.5	3.9	2.3	1.5	-0.4	-0.4	0.0
1600	476	472	504	498	500	-0.3	2.0	1.2	0.9	3.2	1.7	1.3	-0.5	-0.3	0.3
2000	457	453	488	481	445	-0.3	2.2	1.3	-0.5	3.5	1.9	-0.4	-0.6	-3.6	-5.1
2400	398	370	441	415	420	-2.0	3.1	0.9	0.8	7.1	3.0	2.3	-2.2	-1.8	0.7
2800	376	365	406	394	395	-0.8	2.1	0.9	0.7	4.1	1.9	1.4	-1.0	-0.9	0.1
3200	351	332	383	373	380	-1.4	2.3	1.2	1.1	5.1	2.7	2.2	-0.8	-0.3	1.0
3600	336	327	358	358	365	-0.6	1.6	1.2	1.1	3.1	2.1	1.7	0.0	0.6	1.0
4000	334	330	357	347	350	-0.3	1.6	0.7	0.6	2.7	1.1	0.9	-0.8	-0.6	0.4
4400	341	346	373	367	365	0.4	2.3	1.4	0.9	2.7	1.4	0.9	-0.5	-0.7	-0.3
4800	370	380	414	403	415	0.7	3.1	1.7	1.7	3.4	1.5	1.6	-0.9	0.1	1.7
5200	326	332	358	363	360	0.4	2.3	1.9	1.3	2.6	2.1	1.3	0.4	0.2	-0.4
5600	257	256	276	264	285	-0.1	1.4	0.4	1.1	2.0	0.5	1.3	-1.0	0.8	3.0
6000	237	250	230	243	250	0.9	-0.5	0.3	0.5	-2.0	-0.5	0.0	1.1	1.7	1.0
6400	288	277	279	306	318	-0.8	-0.6	0.9	1.2	0.2	1.9	1.9	2.3	3.3	1.7
6800	395	369	381	418	432	-1.9	-1.0	1.2	1.4	1.2	3.3	2.9	3.1	4.3	2.0
7200	393	379	396	425	428	-1.0	0.2	1.7	1.3	1.7	3.1	2.2	2.4	2.7	0.4
7600	427	411	421	445	458	-1.1	-0.4	0.9	1.2	1.0	2.3	2.1	2.0	3.1	1.9
8000	430	418	421	448	459	-0.9	-0.6	0.9	1.1	0.3	2.0	1.9	2.3	3.2	1.6
8400	473	472	483	479	490	-0.1	0.7	0.3	0.7	1.1	0.5	0.8	-0.3	0.6	1.6
8800	500	488	499	495	504	-0.9	-0.1	-0.3	0.2	1.1	0.5	0.7	-0.3	0.4	1.3
9200	493	480	478	489	500	-0.9	-1.1	-0.2	0.3	-0.2	0.6	0.9	0.9	1.8	1.6
9600	438	462	436	456	472	1.7	-0.1	0.9	1.3	-2.6	-0.4	0.5	1.7	3.0	2.3
10000	373	399	392	384	405	1.9	1.4	0.6	1.2	-0.7	-1.0	0.3	-0.7	1.1	3.0
10400	307	313	323	313	322	0.4	1.1	0.3	0.6	1.0	0.0	0.4	-0.8	-0.1	1.3
10800	261	263	259	238	262	0.1	-0.1	-1.2	0.0	-0.4	-1.7	0.0	-1.8	0.3	3.4
11200	250	271	251	224	274	1.5	0.1	-1.4	0.9	-2.0	-3.1	0.1	-2.3	1.9	7.1
11600	297	334	295	297	350	2.6	-0.1	0.0	2.0	-3.9	-2.5	0.7	0.2	4.6	7.6
12000	442	432	432	425	458	-0.7	-0.7	-0.9	0.6	0.0	-0.5	1.2	-0.6	2.2	4.7
12400	714	714	714	714	714	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
					Average:	**0.0**	**1.1**	**0.8**	**1.0**	**1.5**	**1.0**	**1.1**	**0.0**	**0.8**	**1.4**

Notes:
1. The 1976, 1980, 1990, and 1995 shoreline widths are based on 1995 North Shore Of LI Recon. Report prepared by M&N.
2. The 2002 shoreline width is based on April 2001 aerial photo, all aerial photos were taken in the months of March or April.
3. "+" indicate shoreline accretion, "-" indicate shoreline erosion.

8.3 Sediment Budget

As discussed above, the historical shore evolution on the Long Island Sound Side at the project area is mildly accretion with an approximately 1.0 ft/yr rate. Assume an active surf zone depth of –24 ft NAVD88, the average MHW volume accretion rate is approximately 1.0 cy/yr. The long term accretion volume for the 1.5 mile study shoreline between east and west headlands is estimated at 8,000 cy/yr. The mildl shoreline accretion could be attributed to several favorable coastal processes:

- The natural crenulate-shaped bay formation of the study shoreline between the east and west headlands provide a static equilibrium shore formation. Once the crenulate shape is formed, offshore incident wave arriving from any direction would adjust itself and arrive simultaneously at all points along the shoreline. This would result in a static equilibrium shoreline with little longshore sediment transport;

- The continued bluff erosion at the east and west headlands (Rocky Point and Oak Neck Point) contribute sediment sources into the project shoreline. The littoral flow into the crenular shaped bay will be trapped between two headlands. A portion of the net in-flow littoral material would move further offshore near the headlands;

- Coarse foreshore littoral material (median size at 2.0 mm to 3.0 mm) at the project shoreline provides a relatively small sediment transport rate and stable beach formation;

- The recent offshore bathymetric survey (in October 2001) map indicates a mild offshore shoal formation at the east and west headlands and deeper bay area at the center of the crenular bay. The results suggests the project shoreline is in dynamic equilibrium formation with net inflow of eroded material trapped near the headlands.

8.4 Existing Shoreline Characteristics and Typical Beach Profiles

8.4.1 LONG ISLAND SOUND FRONT

As shown in the general site map (Figure 1-2), the Long Island Sound front shoreline is on an enclosed cresent-shape waterfront located between two headlands: Oak Neck Point to the west and Rocky Point to the east. The shoreline is populated with residential development with year-round houses built to the edge of water. Typical beach profiles range from residential homes built behind dunes, to buildings sitting on foundations with concrete bulkhead seawall fronting beach. A public recreational beach is located in the middle of the project shoreline .

The existing project shoreline on Long Island Sound is approximately 1.3 miles in length with relatively steep foreshore slope and a range of dune height. Foreshore beach widths range from 50 to 100 ft with low berm formation. The project shoreline is divided into five typical reaches based on beach profile and the waterfront structural characteristics. The profiles are summarized in Table 8-2 and dicussed in more detail below in the individual reach discusions. The five reaches will be analyzed separately for with and without project coastal processes and both existing and future conditions and for economic analysis of the alternatives considered.

Table 8-2. Beach Profile Characteristics

Reach No.	General Location	Approx. Length (ft)	Dune Elevation (ft NAVD88)	Berm Avg. Elevation (ft NAVD88)	Beach Width (ft)	Foreshore Slope (1 v on x h)	Offshore Slope (1 v on x h)
1	West Cliff Drive to Washington Ave	1,450	+14 to +15 (Conc. Wall)	+9	100	8	25
2	Washington Ave to Sound Beach Rd	1,600	+12 to +13	+7	80	8	15
3	Sound Beach Rd to Ships Lane	1,250	+11 (Conc. Wall)	+7	100	8	25
3a	Ships Lane to Greenwich Av	550	+11 (Conc. Wall)	+7	100	8	25
4	Greenwich Av to 7th th St	400	+12.5	+10	200	10	25
5	7th St to West Harbor Dr	1,450	+14 to +15 (Conc. Wall)	+6	100	8	flat

Reach 1. This reach extends approximately 1,450 ft along the shoreline from the western border of the project at West Cliff Drive east to Washington Avenue. The waterfront along this stretch of shoreline is densely developed with residential building. The majority of the buildings in this reach are constructed on the edge of beach berm with timber bulkhead foundation containment at approximately +11 to 13 ft NAVD88 crest elevation during the site inspection conducted in August 2001. A field inspection conducted in March 2004 indicated a significant improvement of the waterfront bulkhead seawall. The entire waterfront had been replaced with vertical concrete bulkhead seawall with crest elevation in the range from +14 to +15 ft NAVD88 and approximately 6 to 8 inches in thickness. The condition of the seawall foundation is unknown since it is buried under beach sand. Based on visual inspection, there is no existing splash blanket or rock toe protection at the concrete seawall. The average ground elevation landward of the bulkhead is approximately 11.5 ft above NAVD88. The waterfront community in this reach consists predominantly of private home accesses. Foreshore slope seaward of the bulkhead is approximately 1 vertical on 8 horizontal to elevation –5 ft NAVD88 and the offshore slope is approximately 1 vertical on 25 horizontal. The existing berm slopes down from +9 ft NAVD88. The average beach width from base of concrete seawall to NAVD shoreline is approximately 100 ft. Typical Beach profile within this reach is described by profile B-1 in Figure 8-11.

Reach 2. This reach extends from Washington Avenue east approximately 1,600 ft along waterfront shoreline ending at Sound Beach Road. This stretch of shoreline is characterized by a relatively steep foreshore slope at 1 vertical on 8 horizontal down to –6 ft NAVD88, and a berm sloping down from elevation +9 ft NAVD88 backed with a relatively high dune line. The offshore slope is approximately 1 vertical on 15 horizontal. Residential buildings are scattered landward behind the dune. Beach widths in this reach are approximately 100 ft from NAVD8 shoreline to the landward base of dune. The seaward dune slope is approximately 1 vertical on 5 horizontal and dune crest elevation range from approximately +12 to +13 ft NAVD88. The landward dune slope is steeper and levels off to approximately +11 ft NAVD88 existing ground elevation. The average space between landward toe of dune and the building front porch is approximately 20 ft. The typical beach profile for this reach is described by Profile B-2 in Figure 8-2.

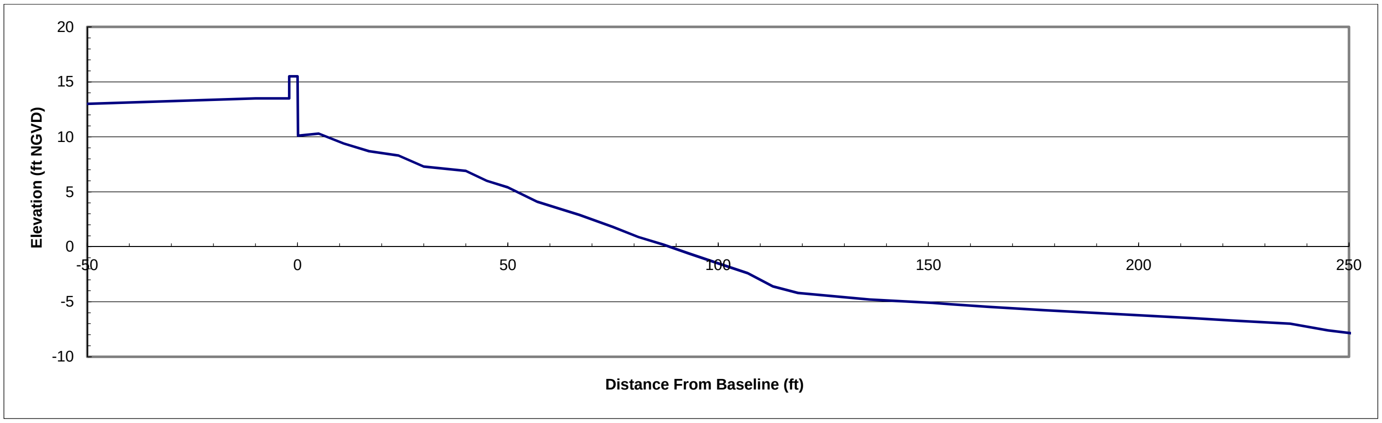
Reach 3. This reach contains approximately 1,250 ft waterfront extending from Sound Beach Road east to Ships Lane. The shoreline characteristics in this reach is similar to Reach 1 with dense residential building landward of timber bulkhead. The building foundations and timber bulkheads are constructed on the edge of berm with crest elevation of bulkhead at approximately +11 ft NAVD88. Based on the March 2004 field inspection, all timber bulkhead were replaced with concrete seawall at the same crest elevation. It should be noted that the concrete wall in this reach is not continuous. The average ground elevation landward of the concrete seawall is approximately +11.5 ft above NAVD88. The waterfront community in this reach consist of private home owners with minimal street access to the beach front. Foreshore beach slope seaward of the concrete wall is approximately 1 vertical on 8 horizontal down to elevation –4 ft NAVD88 NAVD88. The berm elevation seaward of the bulkhead is approximately +7 ft NAVD88. Average beach width from base of seawall to NAVD88 shoreline is approximately 100 ft. Typical Beach profile of this reach is described by profile B-3 in Figure 8-3.

Reach 3a. This 550 ft waterfront extends from Ships Lane east to Greenwich Avenue. The foreshore characteristics in this reach is similar to Reach 3, however, this reach consists of a mix of residential buildings landward of timber bulkhead and a park. The building foundations and timber bulkheads are constructed on the edge of berm with crest elevation of bulkhead at approximately +11 ft NAVD88. Based on a March 2004 inspection, two new restaurants were built within this reach and the timber bulkheads were replaced with concrete wall at the same crest elevation. The average ground elevation landward the bulkhead is approximately 11.5 ft above NAVD88 and the average ground elevation in the park range from +15 to +17 ft NAVD88. Typical Beach profile of this reach is described by profile B-3 in Figure 8-4.

Reach 4. This reach contains approximately 400 ft of public bathing beach and a beach club between Greenwich Avenue and 7th Avenue. The beach profile in this reach consists of a relatively milder 1 vertical on 10 horizontal forshore slope down to elevation –4 ft NAVD88. The berm is approximately 100 ft wide with elevation at approximately +10 ft NAVD88. The dune crest elevation just seaward of parking facility is approximately +12.5 ft NAVD88. The average beach width from base of dune to NAVD88 shoreline is approximately 200 ft. Typical Beach profile of this reach is described by profile B-4 in Figure 8-5.

Reach 5. Reach 5 is similar to Reach 1 in beach profile, berm formation, and seawall construction. Total length of this reach is approximately 1,450 ft extending from 7th street east to West Harbor Drive with predominantly private homes built directly behind the bulkheads. Based on a March 2004 inspection, the original timber bulkhead seawall were replaced with concrete seawall at crest elevations range from +14 to +15 ft NAVD88. Similar to Reach 1, the seawall foundation condition is unknown. The beach profile on this reach is composed of a berm at crest elevation +6 ft NAVD88. The berm is approximately 100 ft wide from seawall to NGVD shoreline. Foreshore slope is approximately 1 vertical on 8 horizontal to –6 ft NAVD88 and the offshore slope is flat. Typical Beach profile of this reach is described by profile B-5 in Figure 8-6.

The shoreline further east of Reach 5 is comprised of high dunes at crest elevation ranging from +16 to +17 ft NAVD88 with a relatively wide beach. Further east along Centre Island Road, the highway embankment is reinforced with riprap slope protection and maintained by the State.



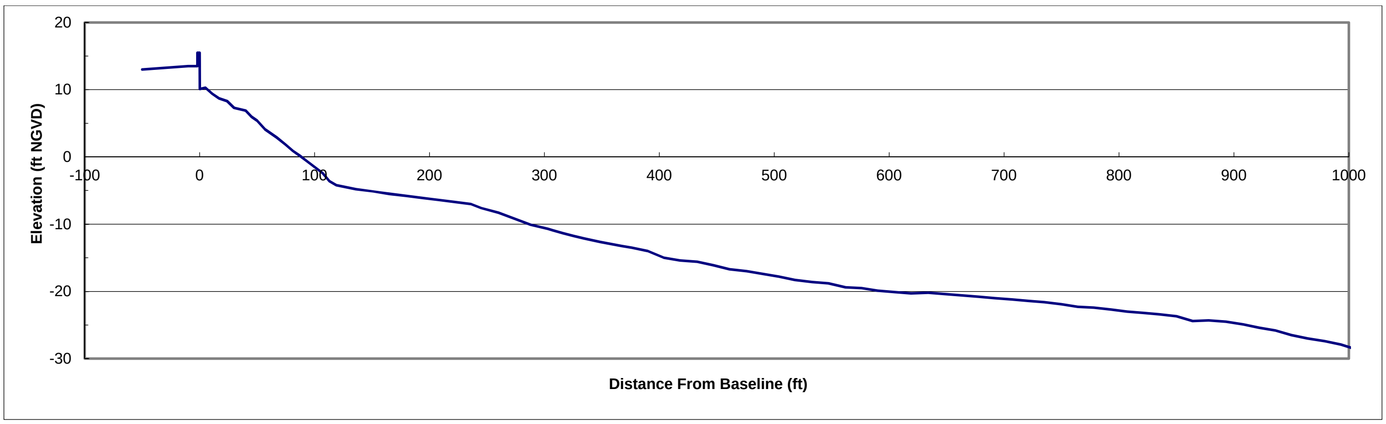
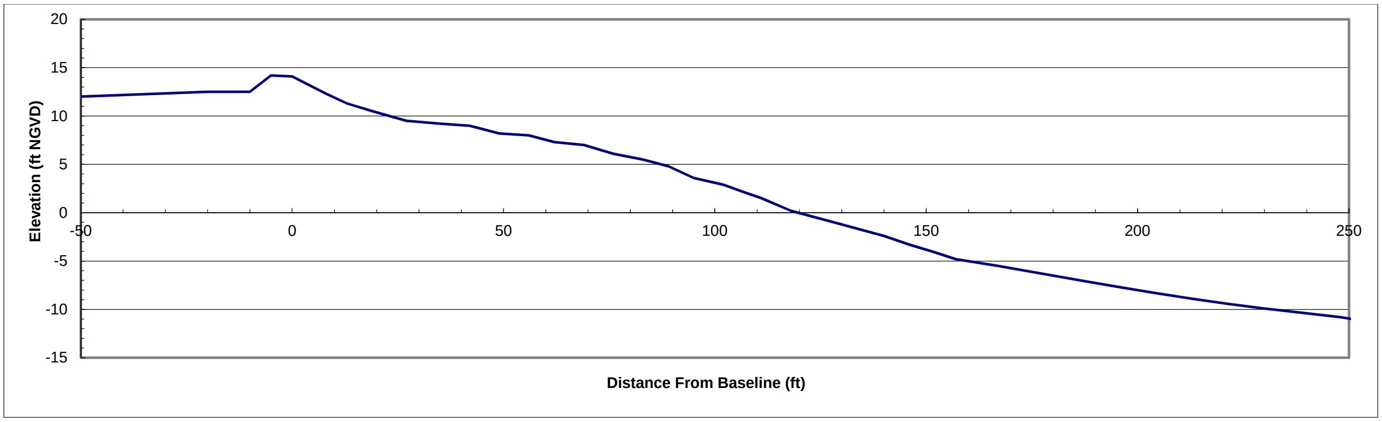


Figure 8-1 Typical Profile B-1 for Reach 1 Shoreline



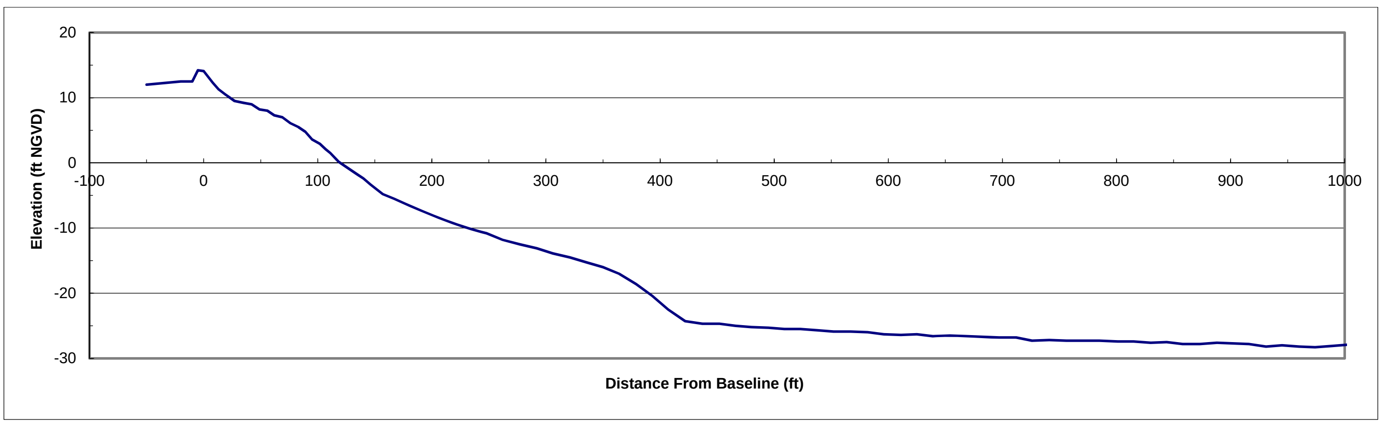
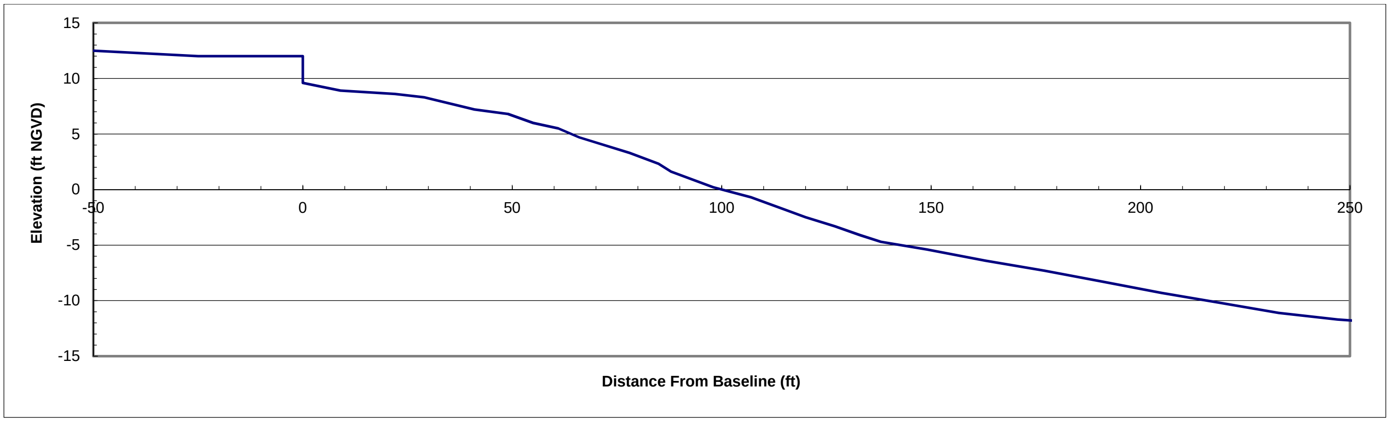


Figure 8-2. Typical Profile B-2 for Reach 2 Shoreline



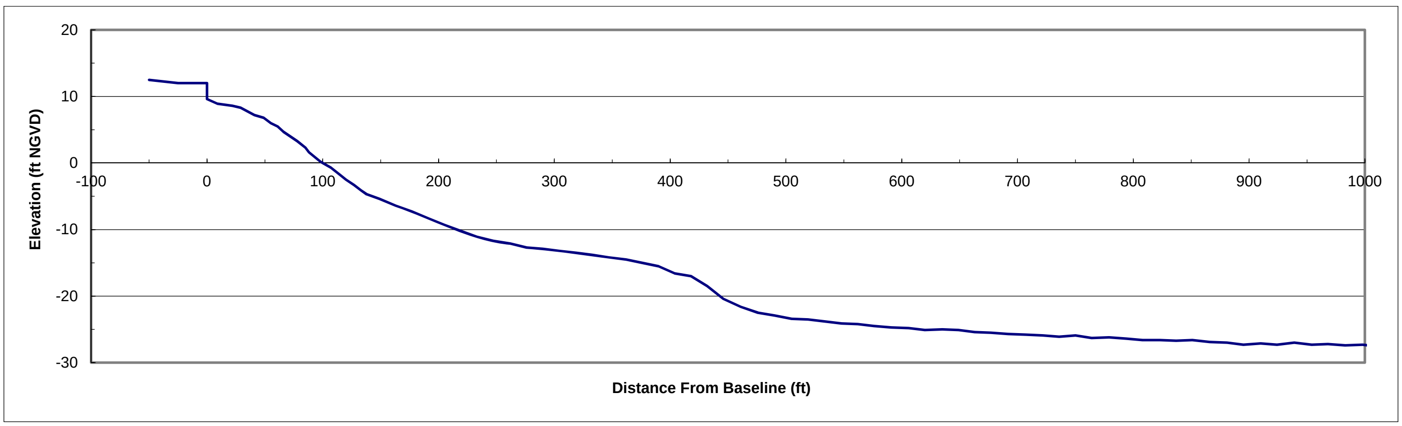
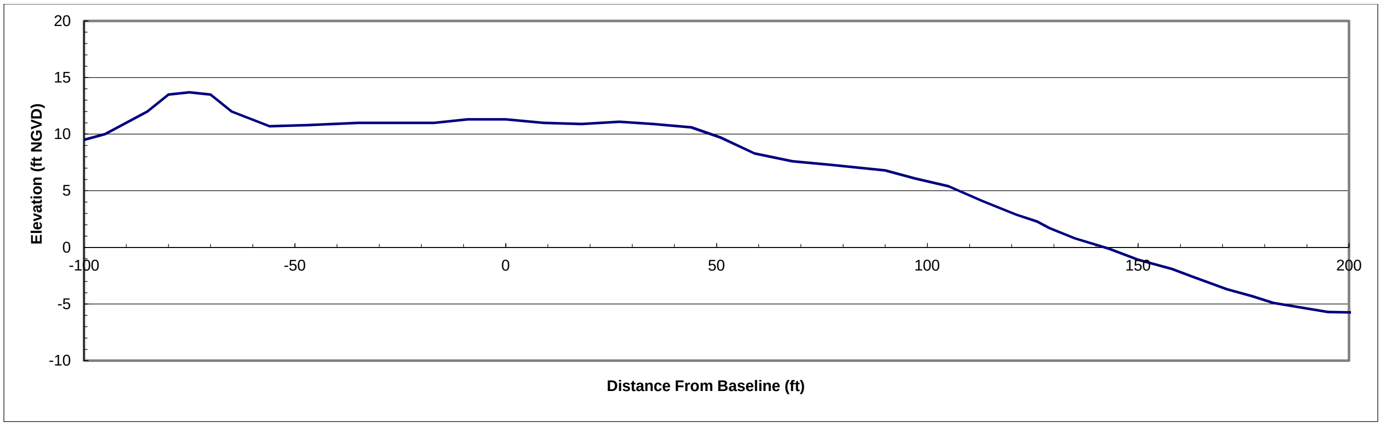


Figure 8-3. Typical Profile B-3 for Reach 3 Shoreline



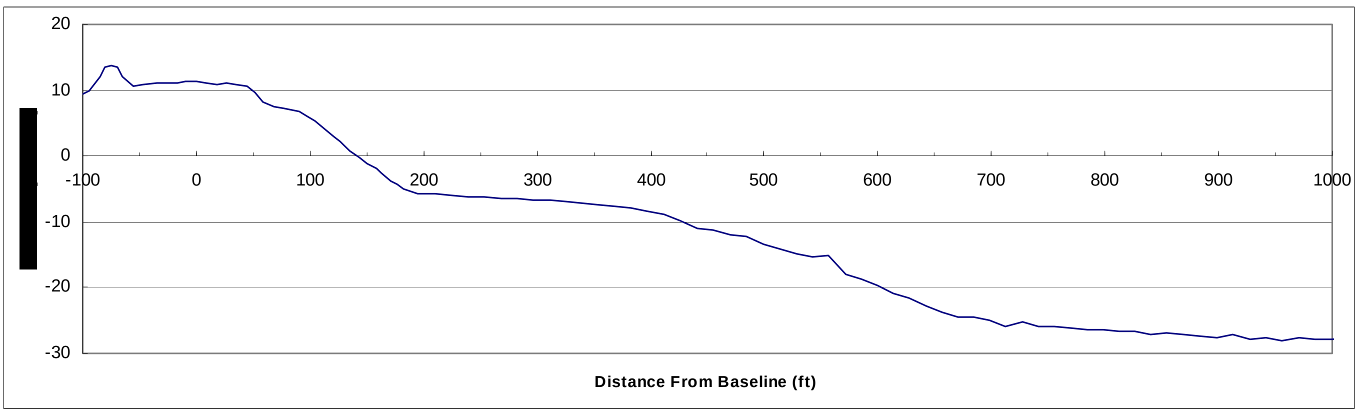
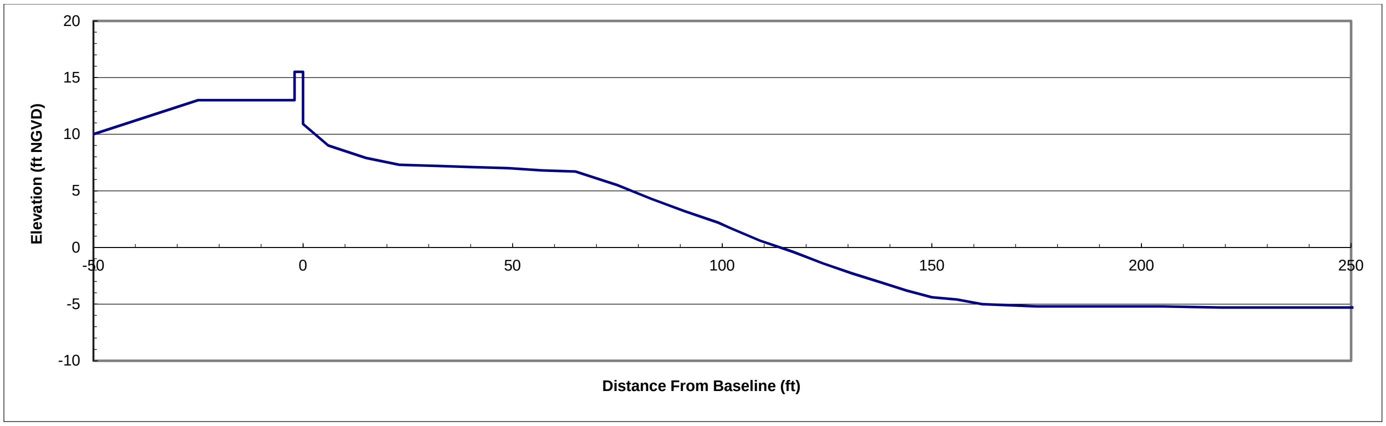


Figure 8-4. Typical Profile B-4 for Reach 4 Shoreline



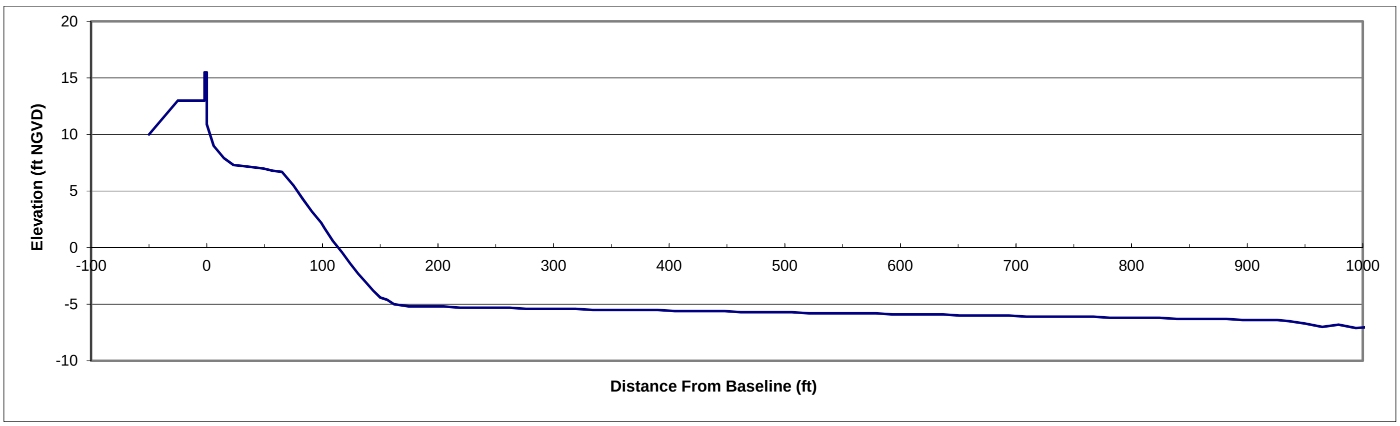


Figure 8-5. Typical Profile B-4 for Reach 5 Shoreline

8.4.2 Back Bay Waterfront

The existing backbay project shoreline can be divided into two reaches based on waterfront features. The western reach is approximately 2,500 ft from the western boundary of Saltair Lane and Shore Road to the eastern boundary of Ludlam Avenue. Except for the eastern 600 ft developed waterfront of yacht club and marina, the rest of bayfront shoreline in the western reach are low marshlands against the timber bulkhead retaining wall along private properties at an average elevation of +9 ft NAVD88. The eastern reach is bordered along approximately 5,500 ft road (West Harbor Drive) from Ludlam Avenue to approximately 200 ft east of the intersection with Centre Island Avenue. Average road elevation is +10.5 ft NAVD88 for most of the western 4,700 ft road pavement and sloping down to approximately +9.5 ft NAVD88 for the eastern 800 ft road pavement near the intersection of the Centre Island Road.

Table 8-3. Backshore Condition

Reach No.	General Location	Approx. Length (ft)	Backshore Condition	Structure	Avg. Elevation At Crest (ft NGVD)	Avg. Elevation Behind Crest (ft NGVD)
1	West Cliff Drive to Washington Ave	1,450	Wall	Concrete Seawall	+14 to +15	+11 to +12

2	Washington Ave to Sound Beach Rd	1,600	Dune	N/A	+12 to +13	+11
3	Sound Beach Rd to Ships Lane	1,250	Wall	Concrete Seawall	+11	+11.5
3a	Ships Lane to Greenwich Av	550	Wall	Concrete Seawall	+11	+11.5
4	Greenwich Av to 7th th St	400	Dune	N/A	+12.5	+11
5	7th St to West Harbor Dr	1,450	Wall	Concrete Seawall	+14 to +15	+9

9.0 Without Project Future Condition

The without project future conditions at the project site are evaluated separately for the Long Island Sound shoreline and the Oyster Bay (back bay) waterfront. Results are used as baseline condition for economic analysis. The expected without project future condition at the project site include:

- Long term shoreline recession
- Storm wave erosion
- Wave runup and overtopping induced dune and coastal structure failure
- Inundation (from both L.I.Sound side and Back Bay side)

9.1 Long Term Erosion Rates

Based on the sediment transport study and sediment budget analysis discussed in Sections 8.2 and 8.3, the long term shoreline erosion rate along L.I.Sound is small and negligible. The back bay side of shoreline is relatively stable.

9.2 Short Term (Storm) Erosion Rates

The dune erosion distances during storms for reaches 2 and 4, and the approximate erosion distances after failure of concrete seawall at the rest of the reaches are estimated based on EDUNE storm erosion model. The model input condition is summarized in Table 9-1. A typical dune and beach profile with dune crest elevation at +12.5 ft NAVD88 and berm elevation at +7 ft NAVD88 was used. The resulting maximum beach erosion distances are summarized are shown graphically in Figure 9-1. The resulting maximum dune erosion distances are summarized in Table 9-2 and shown graphically in Figure 9-2.

Table 9-1. Input Condition for EDUNE

Return Period (years)	Hb at -10 ft NGVD	Hb(rms) at –10 ft contour	Surge Elevation (ft-NAVD88)
2	10.2	7.2	4.92
5	13.1	9.3	6.52
10	14.2	10.0	7.82
25	15.4	10.9	8.52
50	16.2	11.5	9.02

100	17.0	12.0	9.32
200	17.9	12.6	12.52

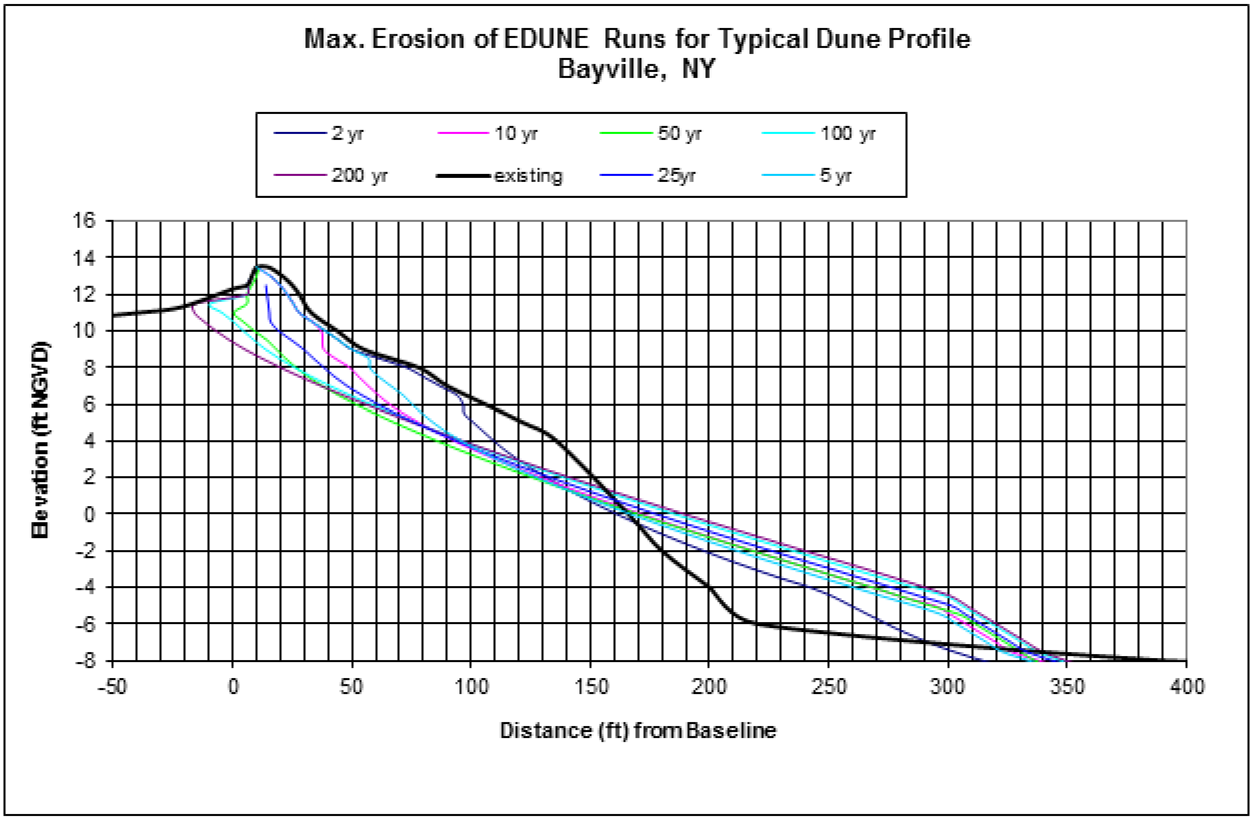


Figure 9-1. Beach berm and dune erosion due to various storm events (note table is provided in NGVD)

Table 9-2. Dune Erosion Distance for Typical Dune Reach

Return Period (year)	Dune Erosion Distance (ft)
2	7
5	22
10	30
25	42
50	50
100	56
200	60

Note: Dune erosion distance is measured from seaward base of dune at +7ft NAVD88

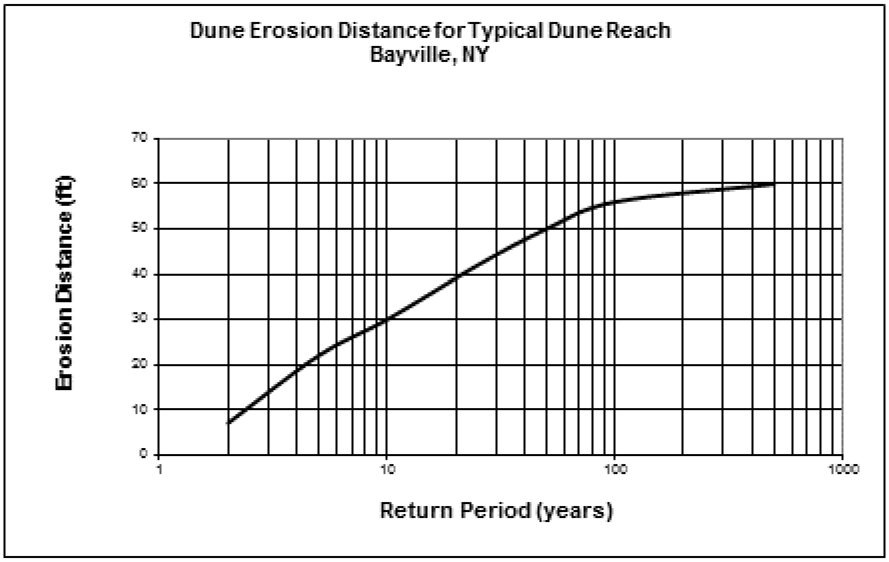


Figure 9-2. Estimated Maximum Dune Erosion Distance

9.3 Wave Runup/Overtopping Analysis

Based on observation of failure mode, it was determined based on engineering judgment that the primary concrete seawall failure mechanism at Bayville is due to wave overtopping, resulting in erosion of backfill material behind the structure and eventually foundation failure. Overtopping rates at seawall and dunes were estimated based on the crest elevations at the typical reaches of Long Island Sound front shoreline. Overtopping estimates were compared with the allowable overtopping rates determined for various failure conditions of interest to determine the level of protection provided by existing shore conditions. Seawall failure due to overtopping was assumed to occur when the backfill material behind the wall being eroded. Dune failure or breaching would occur after over washing waves erode the backslope. It should be noted that the threshold overtopping rate varies widely due to various lab testing results and the actual site conditions. The critical values of average overtopping discharges for this study is based on CEM2001 Table VI-5-6. The threshold overtopping rates used to determine bulkhead and dune failure are summarized as follows:

- Threshhold overtopping rate for protected crest: 0.20 cfs/ft
- Threshhold overtopping rate for unprotected crest: 0.05 cfs/ft

The overtopping rate at the concrete seawall can be estimated based on the formula by van der Meer and Janssen (1995) and van der Meer (1998) as discussed in Coastal Engineering Manual (CEM2001). The proposed formula for estimating the average wave overtopping on coastal structures subject to random waves can be expressed as:

$$\frac{q}{\sqrt{gH_s^3}} = \frac{0.06}{\sqrt{\tan\alpha}}\gamma_b\xi_p \exp\left[-5.2\frac{R_c}{H_s}\frac{1}{\xi_p\gamma_b\gamma_f\gamma_\beta\gamma_v}\right]$$

$$\text{Maximum: } \frac{q}{\sqrt{gH_s^3}} = 0.2\exp\left[-2.3\frac{R_c}{H_s}\frac{1}{\gamma_f\gamma_\beta}\right]$$

Where: q = mean wave overtopping discharge per unit width

ξ_p = breaker parameter

H_s = significant wave height

R_c = revetment crest freeboard (height of structure above still water)

γ_b = reduction factor for a berm

γ_f = reduction factor for slope roughness

γ_β = reduction factor for oblique wave attack

γ_v = reduction factor due to a vertical wall on a slope

For prediction of wave overtopping rates over dunes, Kobayashi et al. (1996) conducted seven small-scale tests to measure wave reflection, overtopping, and over wash of dunes. He compared measured overtopping rates with the empirical formula from van der Meer (which was developed for coastal structures) and through the use of an equivalent uniform slope (m_o) and showed that this formula can predict the order of magnitude of the measured overtopping rates. The equivalent uniform slope (m_o) is assumed to be the overall slope between the dune crest and the point where the water depth equals the significant wave height. Kobayashi's analysis, however, relies on small-scale tests, which generally do not accurately extrapolate to prototype conditions. Therefore, van der Meer's expression was further adapted through the use of results from a limited number of large scale tests performed by Delft Hydraulics Laboratory (1983). A similar approach was used in the analysis of overtopping, wave run-up, and wave impacts analysis for the Westhampton Interim Project (Moffatt and Nichol, 1993) and the Breach Contingency Plan (Moffatt & Nichol Engineers, 1995). Note, however, that those two previous analyses were based on an older overtopping formulation (Pilarczyk, 1990) which does not readily allow for a berm reduction factor and has not been directly applied to other dune overtopping measurements like the new van der Meer formula has.

Overtopping data from the Delft experiments were used to derive new empirical coefficients for van der Meer's formula (i.e., 0.06 and –4.7). The following overtopping formula, which includes the new coefficients (0.013 and –2.33), is considered to be a better estimate of dune overtopping under prototype conditions.

$$\frac{q}{\sqrt{gH_s^{\,3}}} = \frac{0.013}{\sqrt{\tan\alpha}}\gamma_b\xi_p \exp\left[-2.33\frac{R_c}{H_s}\frac{1}{\xi_p\gamma_b\gamma_f\gamma_\beta\gamma_v}\right]$$

The above formula was developed using the equivalent slope assumption proposed by Kobayashi, which in the case of the Delft tests was computed to be approximately 1 on 15. The overtopping rates were estimated for seawall elevations at +11.0, and +13.0 (Reachs 1,3, 3a, and 5); and for dune elevations at +11 and +13 (Reaches 2 and 4). The following is a summary of parameters used:

- Offshore Slope: 1 vertical on 100 horizontal
- Toe Slope: 1 vertical on 3 horizontal

- Berm Height: +7.0 ft NAVD88
- Oblique Wave Angle: 0 degree (no reduction)

Wave Height and Surge Level used are summarized in Table 9-3 which are values taken from the previous stage frequency and wave frequency tables in this report. The estimated overtopping rates are summarized in Table 9-4 and shown graphically in Figures 9-3 and 9-4.

Table 9-3. Wave and Surge Input for Overtopping Estimates

Return Period (years)	Deep Water Wave Height Hs (ft)	Storm Surge Elevation (ft-NAVD88)
2	8.4	4.92
5	10.5	6.52
10	12.4	7.82
25	14.8	8.52
50	16.4	9.02
100	18.0	9.32
200	19.6	12.52

Table 9-4. Estimated Overtopping Rates for Existing Seawall and Dune

Return Period (years)	**Seawall Elevation (ft-NAVD88)**		**Dune Elevation (ft-NAVD88)**	
	+11.0	**+13.0**	**+11.0**	**+13.0**
2	0.0001	0.0000	0.0001	0.0000
5	0.0006	0.0001	0.0058	0.0003
10	0.0272	0.0008	0.1032	0.0072
25	0.7233	0.0339	1.2723	0.1262
50	4.1276	0.2512	4.8898	0.5883
100	18.0392	1.3626	15.3846	2.1658
200	41.1979	7.5961	38.3121	8.2135

Note: Estimated Overtopping Rates in cfs/ft

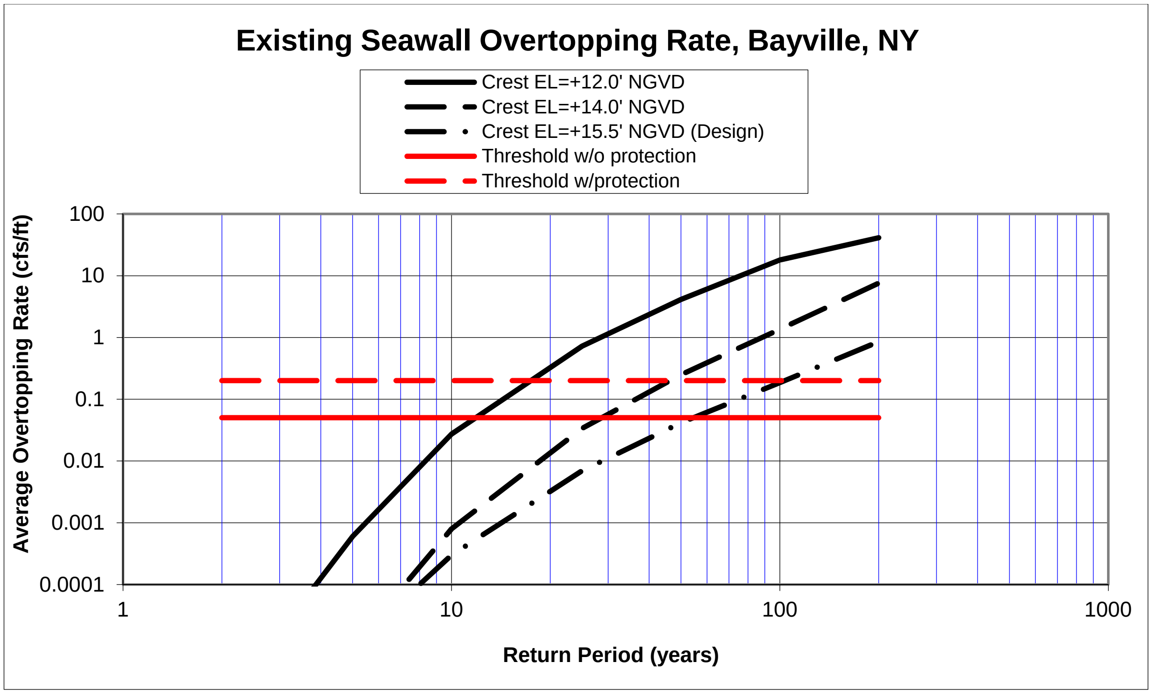


Figure 9-3. Estimated Overtopping Rate at Existing Concrete Seawall (note elevations are in NGVD)

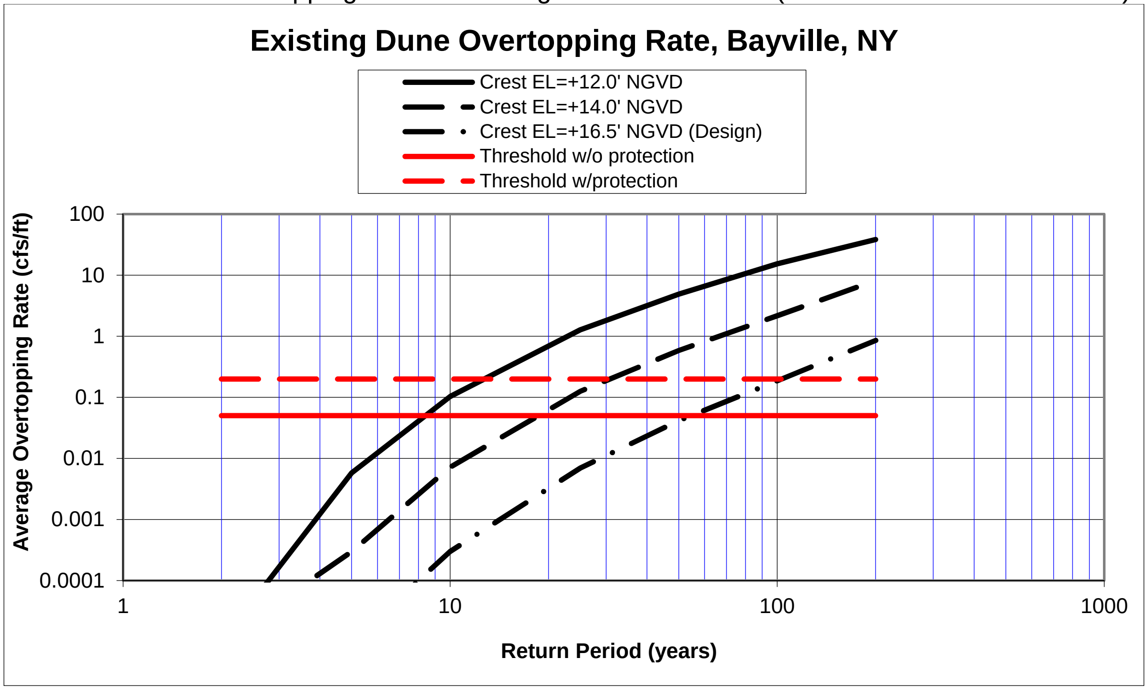


Figure 9-4. Estimated Overtopping Rate at Existing Dune (note elevations are in NGVD)

9.4 Level of Protection

The level of protection afforded to the study area for the without project conditions was determined based on the information provided in previous sections and various field observations.

9.4.1 Long Island Sound Shoreline

The LI Sound project shoreline is delineated into six reaches from west to east based on coastal features and beach profile characteristics. Each reach was discussed previously in Section 8.4.1. The expected without project future conditions at each reach are discussed in the following paragraphs with a summary provided in Table 9-5.

Reach 1: This reach extends approximately 1,450 ft along the shoreline from the western border of the project at West Cliff Drive east to Washington Avenue. The waterfront along this stretch of shoreline is densely developed with residential building. The majority of the buildings in this reach are constructed up to the edge of beach with vertical concrete bulkhead seawall generally at +15 to +15 ft NAVD88 crest elevation. Based on visual inspection, there are no existing splash blanket or rock toe protection at the concrete seawall. The average ground elevation landward of the concrete seawall is approximately 11 to 12 ft above NAVD88, and gradually sloping down to Bayville Avenue at elevation 7 to9 ft NAVD88. Without project, continued erosion during storm and exceptional high water near the foundation of the seawall would encroach structure toe and cause eventual failure. In addition, existing seawall would fail during extreme storm wave overtopping. The cause of storm induced failure is primarily due to wave overtopping and toe erosion. Based on the existing condition, the existing seawall elevation is up to 50 year RP level of protection, however, continued toe scouring without adequate toe protection would reduce the level of protection to 25 year RP. After seawall failure, storm wave with surge would continue to erode the building and properties behind seawall with wave runup, overtopping, overwash and breaching. It is estimated that the first two rows of building will subject to wave action and inundation. Flood water would flow along the low-lying road and inundate low-lying structures further inland.

Reach 2: This reach extends from Washington Avenue east approximately 1,600 ft along waterfront shoreline ending at Sound Beach Road. Residential buildings are scattered landward behind a dune system with crest elevation range from approximately +12 to +13 ft NAVD88. The landward dune slope levels off to approximately +11 ft NAVD88 ground elevation, then gradually slope down to Bayville Avenue at elevation 7 to 8 ft above NAVD88. The average space between landward toe of dune slope and the building front porch is approximately 20 ft. The existing dune would provide up to 25 years level of protection against combined storm surge and wave attack. Continued dune lowering and storm erosion without repair would lower the existing level of protection to approximately 15 years. After dune failure, storm wave with surge would continue to erode the building and pavement, with wave runup, overtopping, overwash and breaching of up to first two rows of building. Storm surge would further inundate low-lying roads and structures further inland.

Reach 3: This reach contains approximately 1,250 ft waterfront extending from Sound Beach Road east to Ships Lane with dense residential building constructed on the edge of beach berm protected with concrete wall at approximately +11 ft NAVD88 crest elevation. It should be noted that the concrete walls in this reach are not continuous. The average ground elevation landward of the concrete seawall is approximately 11 to 12 ft above NAVD88, gradually sloping down to Bayville Avenue at elevation 7 to 8 ft NAVD88. Due to discontinuous seawall in this reach, storm surge with 15 year return period combined with breaking wave would make way through gaps or overtop the seawalls and inundate inland building, roads, and utilities.

Reach 3a: This reach includes 550 ft shoreline with combination of bulkhead, dune and beach with building landward of dune or bulkhead. Waterfront protection is discontinuous and the inland ground elevation range from 9 to 11 ft NAVD88, gradually sloping down to Bayville Avenue at elevation 7 to 8 ft above NAVD88. Due to discontinuous shore protection system in this reach, storm surge with 15 year return period combined with breaking wave would find a way through gaps or overtop the seawalls and damage inland building, road, and utilities.

Reach 4: This reach includes approximately 400 ft of public bathing beach and a beach club between Greenwich Avenue and 7th Avenue. Beach berm is approximately 100 ft wide with +10 ft NAVD88 crest elevation. The existing +12.5 ft NAVD88 dune crest elevation gradually slopes down to 7 to 8 ft NAVD88 at parking facility and Bayville Avenue. The level of protection for the existing dune is approximately 15 years. After dune failure, storm wave and surge flow would overtop and breach through, inundating the low-lying building, utilities, and properties further inland.

Reach 5: is similar to Reach 1 in beach profile, berm formation, and seawall construction. Total length of this reach is approximately 1,450 ft extending from 7th street east to West Harbor Drive with predominantly private homes built directly against the bulkheads. Based on the structural assessment 2004, the original timber bulkhead seawall were replaced with concrete seawall at crest elevations range from +14 to +15 ft NAVD88. The average ground elevation landward of the bulkhead is approximately 9-11 ft above NAVD88, gradually sloping down to Bayville Avenue at elevation 7 to 8 ft NAVD88. Similar to Reach 1, the seawall foundation condition is unknown, and lack of continuous toe protection. The existing structures level of protection and potential storm damage-inundation are similar to Reach 1, up to 50 year return period for existing condition and 25 years RP for future without project condition.

In summary, the existing Long Island Sound project shoreline condition is characterized by continuous, hardened concrete seawall structure located at the eastern and western reaches and with a combination of dune and individual (dis-continuous) concrete seawall in the middle. The general ground elevation range from +9 to +11 ft NAVD88 just landward of the dune and seawall and gradually slope down to +7 to +8 ft NAVD88 along Bayville Avenue, which is in general the low point between the northern and southern waterfront. Without project improvement, the toe of waterfront structures would continue to scour and eventual failure of the existing concrete seawall. Oceanfront dune would erode during storm wave attack, leading to dune failure and storm surge inundation. The existing level of protection range from 15 year at low-lying waterfront gap to 25 year at concrete seawall with armor toe protection. After failure of existing concrete seawall or dune, up to two rows of waterfront building would be subject wave action and storm surge flow. The storm surge would continue travel south along low-lying roads and/or low grounds and eventually meet with flood water from back-bay.

Table 9-5. Estimated Future level Protection Based on Overtopping Rates

Reach	General Location	Approx. Length (ft)	Shore protection Type	Crest Elevation (ft-NAVD88)	Level of Protection (Years)
1	West Cliff Drive to Washington Ave	1,450	Concrete Seawall	+13 to +15	25
2	Washington Ave to Sound Beach Rd	1,600	Dune	+12 to +13	15

3	Sound Beach Rd to Ships Lane	1,250	Concrete Seawall	+11	15
3a	Ships Lane to Greenwich Av	550	Concrete Seawall (dis-continuous)	+11	15
4	Greenwich Av to 7th th St	400	Dune (reinforced)	+12.5	15
5	7th St to 350' east of West Harbor Dr	1,450	Concrete Seawall	+13 to +15	25

9.4.2 Oyster Bay (Back Bay) Waterfront

The existing back bay shoreline is divided into two reaches based on waterfront features. The western reach is approximately 2,500 ft in distance from the western boundary of Saltair Lane and Shore Road to the eastern boundary of Ludlam Avenue. Except for the eastern 600 ft developed waterfront of yacht club and marina, the rest of bayfront shoreline in the western reach are low marshlands against the existing timber bulkhead retaining wall along private properties at an average elevation of +7.5 to +9 ft NAVD88. The eastern reach is bordered along approximately 5,500 ft paved road (West Harbor Drive) from Ludlam Avenue east to approximately 200 ft east of the intersection of West Harbor Drive and Centre Island Avenue. Average road elevation is +10.5 ft NAVD88 for most of the western 4,700 ft road pavement and sloping down to approximately +9.5 ft NAVD88 for the eastern 800 ft road pavement near the intersection of West Harbor Drive and Centre Island Road. During storms, the expected without project future condition at the back bay site include:

- Inundation due to interior flooding and storm surge;
- Blockage of gravity storm drain;
- Rise of groundwater elevation through seepage;

High water at back bay would start to inundate the low-lying area along the western waterfront starting with a 10-year storm event and slowly migrate to the east. The eastern bay front would start flooding with a storm water elevation associated with a 25-year storm event. The slow-rising back bay water level would raise the ground water level, reducing underground drainage capacity. Several storm water drainage outfall would be blocked during high surge stage level. Storm wave damage including wave run up and overtopping on the bay front is negligible.

9.5 Inundation

An inundation analysis was performed in the New York District effort and but that information has not been included to avoid confusion with the updated work conducted within the H&H rainfall analysis and the FDA storm damage modeling effort. Inundation levels were not explicitly needed from the coastal engineering analysis for this effort. The key information required from the coastal engineering analysis was the stage frequency information that was used to feed the FDA model analysis.

10.0 With Project Future Condition

The with project future condition looked at numerous alternatives which included both structural and nonstructural measures. Much of this work can be found in the main report, the Structural Appendix, the Civil Design Appendix, and the Economics Appendix. For the structural alternatives the measures included seawalls on the Long Island Sound side, reinforced dunes on the long Island Sound side and flood walls on the back bay side (Oyster Bay). The alternatives looked at various combinations of these measures as well as various elevations. The alternatives were evaluated using the stage frequency information provided in this appendix, the cost of each alternative, and the benefits provide by each alternative. The evaluation was performed using the HEC FDA model and the description of that effort can be found in the Economics Appendix. A complete description of the development of measures and alternatives for Bayville may be found in the main report in Chapter 3. Table 10-1 provides a list of the measures which were considered by the study team.

Table 10-1. Measure Screening Summary for the Bayville Study Area

Measure	Carried Forward	Eliminated	Reason for Consideration/Elimination
Beach Nourishment		X	Not effective in addressing problems, not applicable to site conditions.
Groins with Beach Fill		X	Not effective in addressing problems, not applicable to site conditions.
Offshore Breakwaters		X	Not effective in addressing problems, not applicable to site conditions.
Rock Reinforced Dune		X	Not Cost Effective, and greater environmental impacts than other measures
Buried Floodwalls	X		Cost Effective, meets Planning Objectives
Bulkhead / Seawalls / Floodwalls	X		Cost Effective , meets Planning Objectives
Levees	X		Cost Effective option for Mill Neck Creek Neighborhood , meets Planning Objectives
Reinforcement of Existing Bulkheads		X	Not Viable, existing walls cannot be incorporated into a Federal project (Design & Real Estate Concerns).
Set-Back Flood walls	X		Cost Effective option for West Harbor Drive
Road Raising	X		Cost Effective option for West Harbor Drive
Interior Drainage Features (pumps)	X		Necessary element for structural measures
Buy-Outs		X	Not Cost effective based upon the value of the Real Estate

Floodplain Management / Zoning		X	Not effective in addressing problems for existing structures
Building Elevation & Floodproofing	X		Cost-effective measure to address buildings that are flooded.
Evacuation Plan	X		Necessary component for all structural alternatives and nonstructural

Through the aforementioned evaluation process the TSP was Alternative 4 from Section 3.11.4 of the Main Report. The plan view and components of the TSP are provided Figures 10-1 to 10-5. The TSP wall elevations were +14 ft-NAVD88 on the Long Island Sound side of the project and elevation +13 ft-NAVD88 on the back bay side. These wall elevations provided flood risk management for the 2% chance annual exceedence storm or in the more familiar terms the 50 year return period storm. The stage frequency for this level of storm, as listed in Table 4-3 has a stage frequency water level of 11.62 ft-NAVD88 on the Long Island Sound side and 10.52 ft-NAVD88 on the back bay side. As can be seen when comparing the flood risk management measure elevations to the stage frequency elevations there is approximately 2.5 feet in difference. This “extra” elevation is to account for SLR increase but more importantly for the potential difference between the NY District stage frequency elevations and the NACCS elevations. The exact elevations will be refined during the next study phase with minor corrections in elevations. During that refinement effort SLR will be considered in more detail, wave overtopping will be analyzed in detail, and the error band/confidence band information provided in the NACCS stage frequency and wave height information will be considered. The alternative elevations will also be further optimized in the economic/cost analysis. All of these considerations may change the alternative measure elevations but not to a level where the TSP would have been different.

Figure 10-1. Alternative 4 - TSP

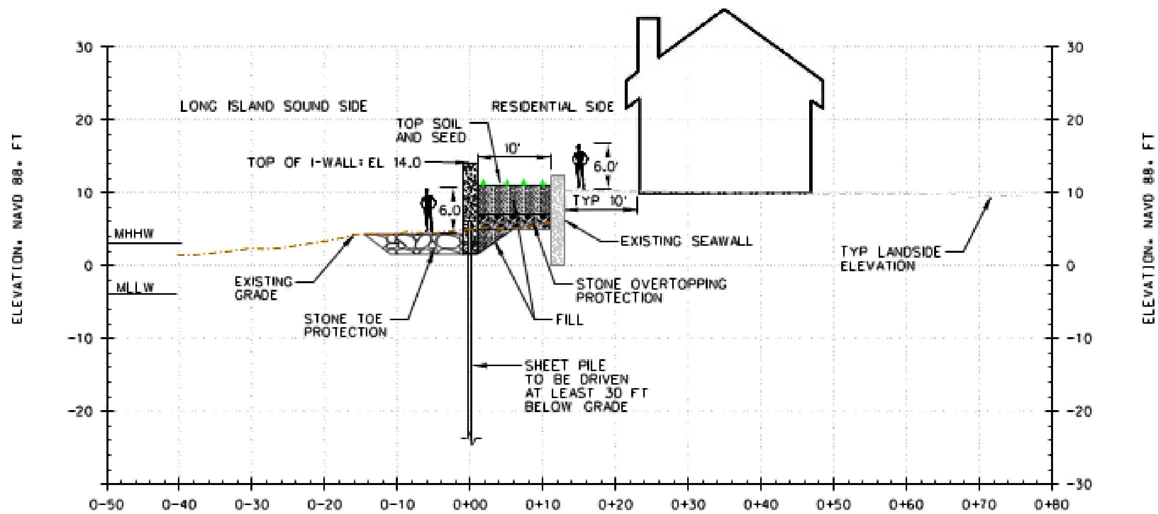


Figure 10-2. Proposed I-Wall along Long Island Sound Reach

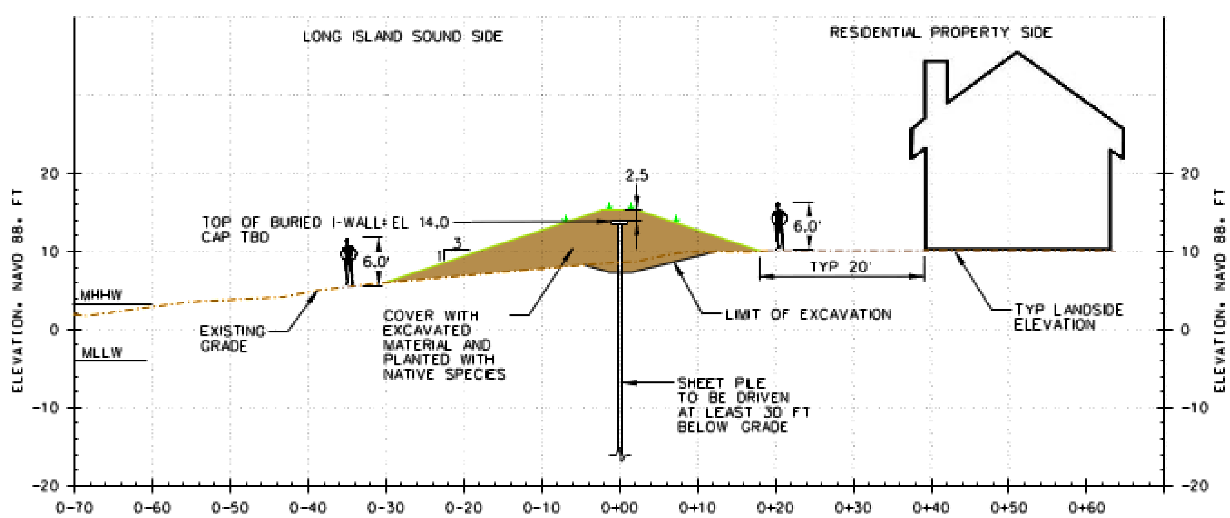


Figure 10-3. Proposed Reinforced Dune Along Long Island Sound Reach.

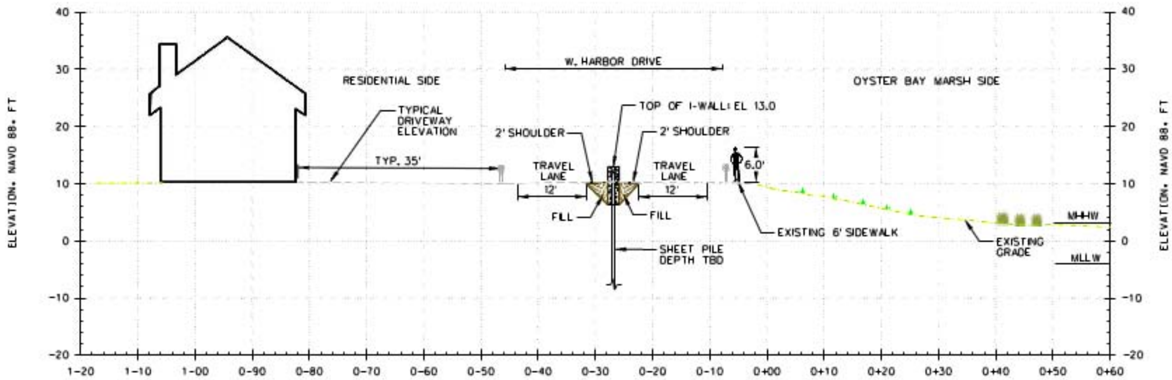


Figure 10-4. Proposed Set-back Floodwall along W. Harbor Drive Reach

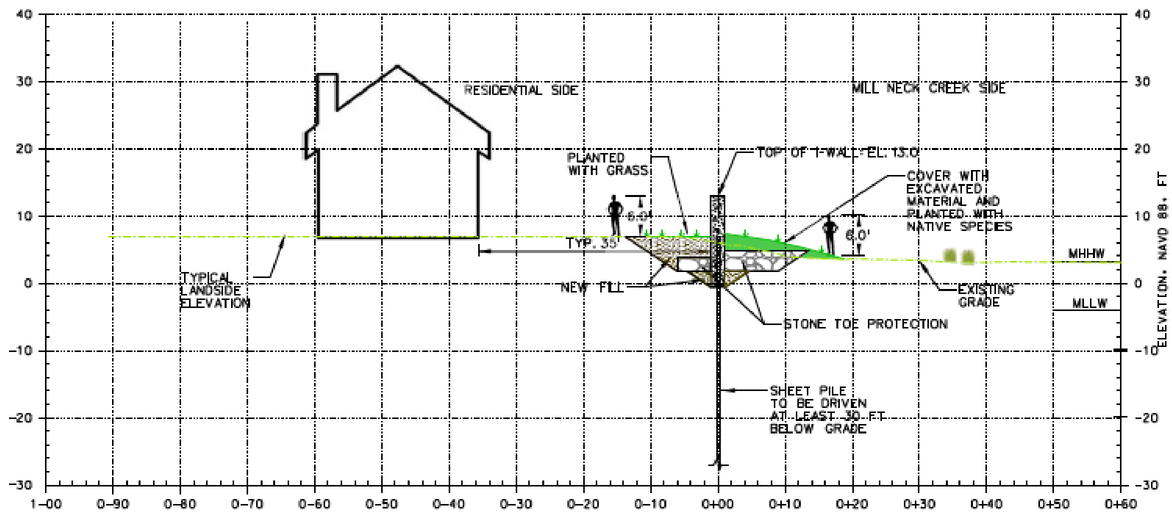


Figure 10-5. Proposed I-Wall along Mill Creek Reach

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North Shore of Long Island, New York, Bayville
Storm Risk Management Feasibility Study
Civil Design Appendix

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Chapter 1: Introduction

1.1 Study Area

The Village of Bayville is a low-lying community located on the north shore of Long Island in Nassau County within the Town of Oyster Bay. The project study area consists of approximately 1/3 of a square mile of densely developed marine, commercial, and residential buildings extending approximately 1.0 mile along Long Island Sound to the north western end and 1.4 miles along Oyster Bay to the south. Washington Ave serves as the western boundary and the intersection of Bayville Ave and W. Harbor Drive serve as the eastern boundary of the project limits. Bayville Avenue is the only major road connecting Centre Island located to the east of the Village of Bayville. The community located between Bayville Avenue and the southern bay front is generally low-lying. The terrain to the west is largely rolling and hilly, due to its position on the Harbor Hill terminal moraine.

The study area has been divided into three major reaches, based on shoreline characteristics and existing structures. The reach to the southwest is Mill Creek Reach. The reach to the south-east is W. Harbor Drive Reach. The reach to the north is the Long Island Sound Reach.

1.2 Shoreline Condition

The northern shoreline along Long Island Sound is a fully developed, mostly private beachfront community. The Long Island Sound beachfront is approximately 1.0 mile facing Long Island Sound with beach widths ranging from 100 to 200 feet. The majority of the beachfront properties are built behind bulkheads and the remainder are located behind dunes. The existing concrete bulkheads range in elevation from around +6 feet NAVD 88 at low points to approximately +10 feet NAVD 88 at the high points. Residential properties, a restaurant and a public beach characterize the development along the shoreline. The existing beach is relatively stable. The existing conditions of the private bulkheads are unknown, because they vary considerable in age and design standards. For analysis purposes, USACE assumed that the existing bulkheads could not be incorporated into the Storm Damage Protection design due to the unknown structural integrity and real estate limitations. Some of the existing bulkheads are incorporated into the foundation of the residential properties, making real estate acquisition and further maintenance difficult.

The southern shoreline along Mill Creek is fully developed with private homes. The majority of homes are generally set back approximately 30 feet from the existing walking path which borders the salt marsh to the south. The walking path is within the tidal zone. Some of the residents have privately constructed small revetments or floodwalls along the southern border of their property abutting the walking path. Generally, the revetments and floodwall crests are around +5 feet NAVD88. The existing conditions of the private bulkheads are unknown, because they vary considerable in age and design standards. For analysis purposes, USACE assumed that the existing bulkheads could not be incorporated into the Storm Damage Protection design due to the unknown structural integrity.

The southern and eastern shoreline along W. Harbor Drive is bordered by a 40-foot-wide asphalt road (which includes two travel lanes, a median and shoulders) and a 6-foot-wide sidewalk. The road elevation is generally constant at +10 feet NAVD 88. The grade slopes down towards the salt marsh from the back of the existing sidewalk. The salt marsh area to the south of W. Harbor Drive is mostly undeveloped; however, there is one designated area with recreation fields, tennis courts, and a well house.

Chapter 2: Survey Data

2.1 Topographic/Bathymetric Data

Topobathy Lidar data is available from 2012 Joint Airborne Lidar Bathymetry Technical Center of Expertise. The data was collected as part of the Post Sandy effort to depict the elevations above and below the water in the New York Coastal Zone. The data was available in an LAS format file containing Lidar point cloud data. The data was received in geographic coordinates of New York Long Island NAD83, U.S. survey feet and vertically referenced to NAVD88, U.S. survey feet.

The topographic features such as structure and utility locations and utility elevations are available from the New York District survey of Bayville during 2002. The elevations of this survey are expressed in U.S. survey feet and refer to vertical datum of NGVD 29. The 2002 New York District Survey was received in multiple sheet file format. For the Feasibility Study effort, the sheet files were referenced into the design documents for information regarding existing utility locations. The locations of utilities shown in the 2002 survey were referenced to determine existing utility interferences with the project alternatives. The more comprehensive and up-to-date survey data from 2012 was used for elevations.

2.2 Vertical Datum

The New England project analyses have been conducted in reference to the NAVD88, U.S. survey feet. Previous coastal analyses by New York District were conducted in NGVD 29 feet. The tidal bench mark nearest to the Bayville project site is located at Bayville Bridge, Oyster Bay, and has a Station ID of 85162299. In order to convert NGVD29 elevations to NAVD88, 1.083 feet will be subtracted from the NGVD29 elevations. For the project area, mean higher high water is 3.82 ft NAVD 88 and mean lower low water is -4.18 ft NAVD 88.

Chapter 3: Project Alternatives

3.1 Preliminary Structural Alternatives Array

During the formulation of preliminary alternatives, the following storm damage reduction structural features were analyzed:

1. Beach Nourishment
2. Groins
3. Offshore Breakwaters
4. Flood/Seawalls
5. Bulkheads/Bulkhead Stabilization
6. Raised and Buried floodwalls
7. Set-back Floodwalls
8. Portable Floodwalls and Gates
9. Road Raising

The preliminary alternatives such as beach nourishment, groins, and offshore breakwater were quickly screened out. These features minimize beach erosion by reducing wave energy. The major storm damage impacts at Bayville are from flooding and inundation, not beach erosion. The features listed above have limited effect on minimizing flooding within the community, and were therefore screened out.

Although the portable floodwalls and gates would be effective in minimizing flooding impacts from the Long Island Sound and Oyster Bay, they were screened out due to the labor-intensive effort in erecting the structures prior to storm events. A significant effort will need to be made to train personnel to erect the temporary structures. This effort would be costly, and in the end, unreliable. The portable floodwalls and gates would also need to be stored and maintained within the village during non-storm days, while readily accessible in advance of an approaching storm.

The remaining structural elements were selected in various combinations. Preliminary alternatives considered for this study include:

- **Alternative 1:** Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Floodwall
- **Alternative 2:** Floodwall/Buried floodwalls along LI Sound, Mill Creek Levee, W. Harbor Raised Road
- **Alternative 3:** Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Raised Road
- **Alternative 4:** Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Set-back Flood Wall

The interior drainage features and pumping stations discussed in Chapter 6 of this appendix and in the Hydraulic Appendix are constant for all four structural alternatives listed above.

3.1.1 Alternative 1: Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Floodwall

Refer to Figure 13 of main report for General Site Plan of Alternative 1

Reach	Feature	Length (LF)	Height (Ft)*
LI Sound	Floodwall	4,500	9
LI Sound	Buried floodwall	3,000	7 to 1
Mill Creek	Floodwall	2,260	6
W. Harbor Drive	Floodwall	5,300	3

*Height is approximate and refers to the average difference in elevation between the existing grade and design elevation.

This alternative includes the construction of Floodwalls and Buried floodwalls along the Long Island Sound Reach. The selection of Floodwall or Buried floodwall along the Long Island Sound Reach was meant to replicate existing features. In areas where there are existing concrete bulkheads, a new Floodwall is proposed. In areas where there is an existing dune, a new buried floodwall is proposed. As discussed in section 1.2 Shoreline Condition, the existing private bulkheads were not incorporated into the alternative due to the unknowns of each structure's condition and design standards, and real estate limitations.

A total of 4,500 LF of new Floodwall along the Long Island Sound Reach is proposed (refer to Figure 1 below). To the maximum extent possible, the proposed Floodwall aligns with stretches of existing bulkheads. The new Floodwall consists of a sheet pile core and concrete cap. The proposed Floodwall alignment was set 10 feet seaward of the existing bulkheads. The offset distance was selected in order to protect the existing bulkheads during construction, and allow for enough room to construct the new Floodwall while minimizing beach encroachment as much as possible. Generally, the top-of-wall elevation of the existing bulkheads are at elevation 10 feet. The 10-ft offset from the existing bulkhead typically aligns with the 5-foot contour. From the perspective of the landward property owner, the new Floodwall design elevation of 14 will seem to be 3-4 ft high. From the perspective of the beach, the Floodwall will be approximately 9 feet high.

The Floodwall design, including sheet pile depth, was determined by the structural engineer (refer to structural engineering appendix). As specified by the geotechnical engineer, stone toe protection and scour protection are required along the length of all Floodwalls. The stone toe protection is carried along the seaward toe of the Floodwall. The toe protection extends 10-ft seaward of the Floodwall, and consists of a 3-foot-thick layer of 24"-diameter stone. The scour protection is carried along the landward toe of the Floodwall, and of a 1.5-foot-thick filter layer, a 2-f00t-thick layer of 10"-diameter stone, and then common fill up to the top of the existing bulkhead elevation. The common fill is not required for scour protection, but the additional fill will provide increased stability of the Floodwall, and provide 10 additional feet for the residents to utilize for passive recreation.

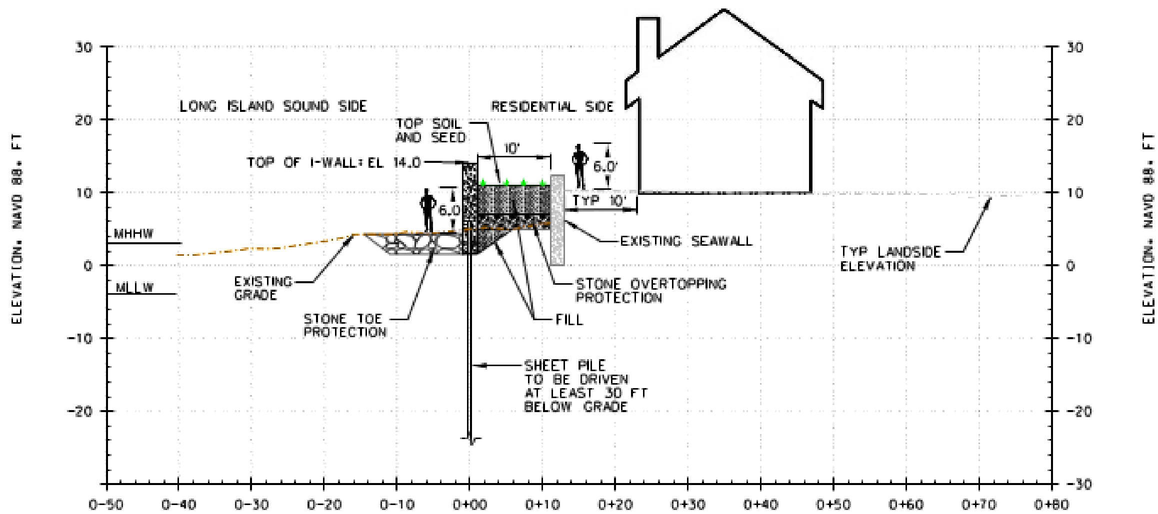


Figure 1: Proposed Floodwall along Long Island Sound Reach

In addition to the new Floodwalls, this alternative consists of 3,000 LF of new buried floodwall along the Long Island Sound Reach (refer to Figure 2 below). . The buried floodwall consists of a sheet pile core and concrete cap that is buried in sand and planted with native dune grass species. The dune is not necessary for structural support of the Floodwall, but is considered as a betterment. To the maximum extent possible, the proposed buried floodwall alignment was set in the same location as the existing dune. The existing dune elevation varies from elevation 13 to elevation 7. The top of the proposed concrete cap is set to the design elevation of 14. The sheet pile and concrete cap is designed by the structural engineer (refer to structural engineering design appendix). As specified by the geotechnical engineer, the elevation of the sand dune crest is set 2.5 feet above the concrete cap. The 2.5 ft of sand fill above the concrete cap is sacrificial and will bring the overall dune elevation to 16.5 feet. The dune crest width is 5 feet and the side slopes are set to the typical angle of repose for sand (1V:3H). During final design, the side slopes will be re-evaluated to select the most stable design. A more gradual side slope, however, will result in a larger footprint.

The buried floodwalls will likely require buried floodwall walkovers to maintain waterfront access. Likewise, construction of a timber stair walkover may need to be constructed at few points along the Floodwall feature to allow for continued access to the beach. These types of access structures can be somewhat unsightly but can be architecturally treated to improve the aesthetic character.

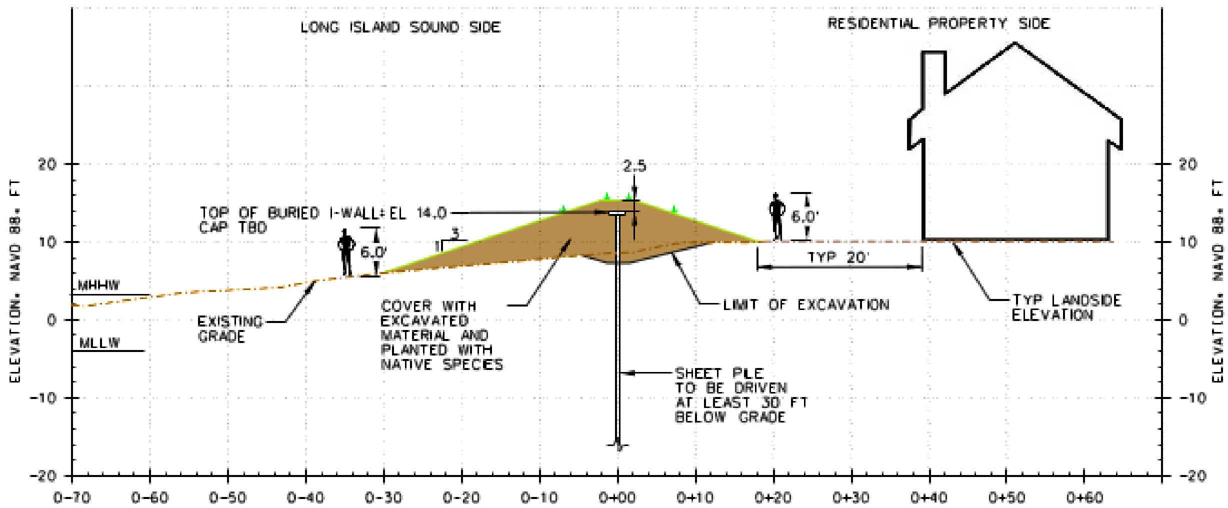


Figure 2: Proposed Buried floodwall Along Long Island Sound Reach.

This alternative includes the construction of Floodwalls along the Mill Creek Reach and the W. Harbor Drive Reach. The Floodwall was selected for these reaches because the Floodwall design has a relatively small footprint while meeting the design elevation. The small footprint is important when attempting to minimize impacts to private property and salt marsh.

The Mill Creek Reach consists of 2,260 LF of new Floodwall (refer to figure 3 below). The alignment is typically set 6 feet off of the southern real estate boundary of the residential properties and runs landward of the Marina at Ludlam Ave. The 6-ft offset is generally consistent with the 5-foot contour. The top of Floodwall is set to the design elevation 13 feet. Since the residential backyard elevations are generally at elevation 7 feet, the Mill Creek Floodwall will be perceived as 6 feet tall. The Floodwall design, including sheet pile depth, was determined by the structural engineer (refer to structural engineering appendix). As specified by the geotechnical engineer, stone toe protection is carried along the seaward toe of the Floodwall. The toe protection extends 8 feet marsh side of the Floodwall, and consists of a 3-foot-thick layer of 18”-diameter stone. The intent of this design is to have the toe protection buried and planted with native salt marsh species in order to prevent encroachment of the salt marsh. As specified by the geotechnical engineer, scour protection is carried along the landward toe of the Floodwall. The scour protection will consist of a 6-inch-thick filter layer, and a 5-foot-wide, 2-foot-thick layer of 10”-diameter stone. Fill will be required above the scour protection in order to match the existing grade of the residential properties.

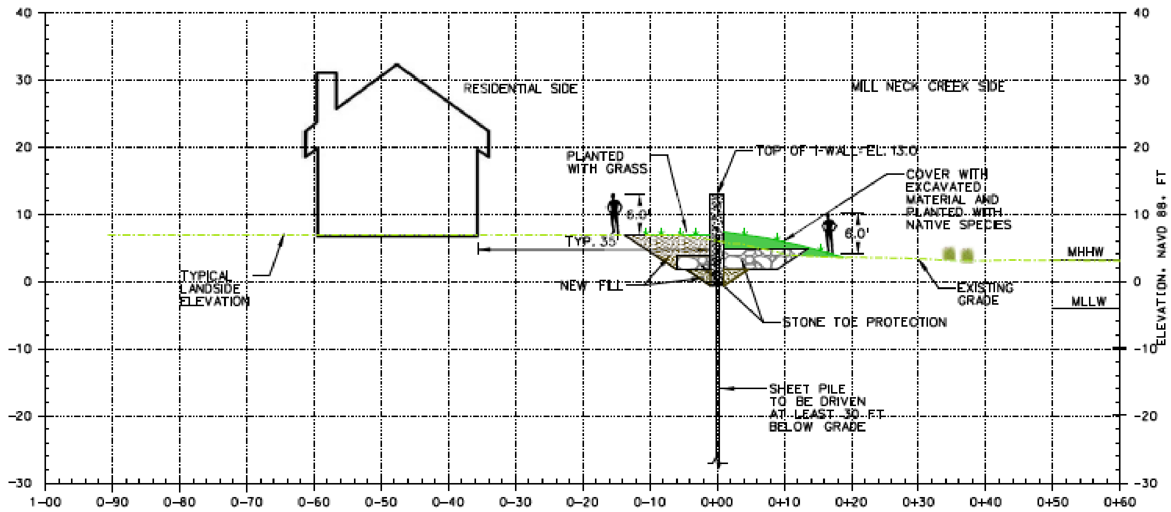


Figure 3: Proposed Floodwall along Mill Creek Reach.

The W. Harbor Drive Reach consists of 5,300 LF of new Floodwall (refer to Figure 4 below). The alignment is set along the southern edge of W. Harbor Drive sidewalk. The existing road and sidewalk elevation is generally elevation 10 feet. The Floodwall design, including sheet pile depth, was determined by the structural engineer. As specified by the geotechnical engineer, stone toe protection is carried along the seaward toe of the Floodwall. The toe protection extends 6 feet marsh side of the Floodwall, and consists of a 3-foot-thick layer of 18”-diameter stone. The intent of this design is to have the toe protection buried and planted with native salt marsh species in order to prevent encroachment of the salt marsh. In order to utilize the existing concrete sidewalk alignment, a new concrete sidewalk will act as the scour protection on the landside of the wall.

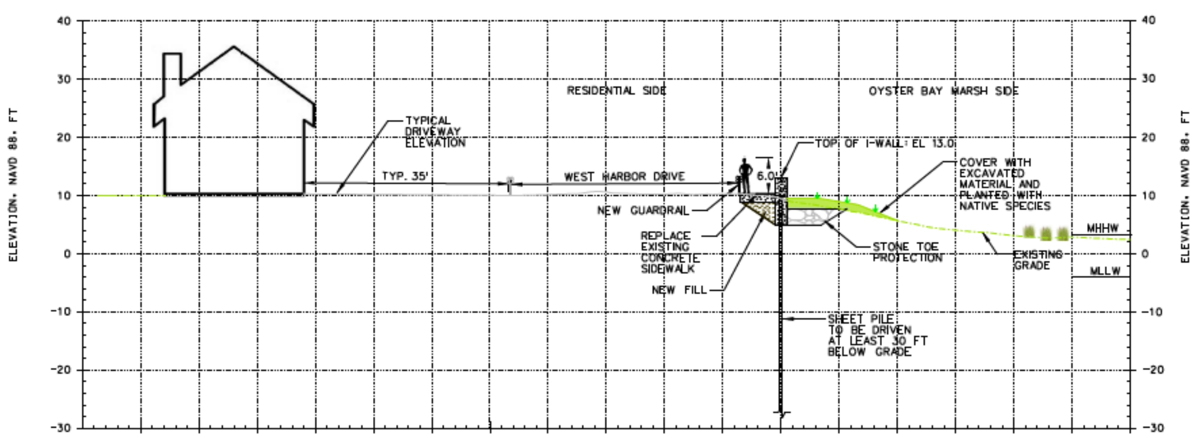


Figure 4: Proposed Floodwall along W. Harbor Drive Reach

In locations where the roads cross the Floodwall alignment, road raises are included to maintain a consistent level of protection around the village. Road raises are utilized in the design instead of portable gates, because the use of portable gates requires storage space and installation prior to storm events. The road raising is permanent and does not require modification prior to storm events. For Alternative 1, road raising occurs at Ludlam Ave, Yacht Club, Arlington Lane, Center Island Road, and two municipal roads along W. Harbor Drive. The road grades are raised approximately 3 feet. The roads are then sloped at a maximum of 6% to match existing road grades. It is assumed that drainage structures such as manholes and catch basins will need to be adjusted with risers, and that valve box castings will need to be reconstructed to match the proposed grade.

This alternative meets the overall project objective of reducing storm flooding frequency in the Village of Bayville from Long Island Sound and Oyster Bay inundation. There are waterfront access impacts and partial water view obstructions for this alternative. The Floodwall along Mill Creek Reach, for example, poses the most drastic change from the existing natural shoreline. This reach proposes a change from the open views of the salt marsh to a concrete Floodwall along the southern border of the residential properties.

3.1.2 Alternative 2: Floodwall/Buried floodwalls along LI Sound, Mill Creek Levee, W. Harbor Raised Road

Refer to Figure 14 of main report for General Site Plan of Alternative 2

Reach	Feature	Length (LF)	Height (Ft)*
LI Sound	Floodwall	4,500	9
LI Sound	Buried floodwall	3,000	7 to 1
Mill Creek	Buried floodwall	2,260	6
W. Harbor Drive	Raised Road	5,700	3

*Height is approximate and refers to the average difference in elevation between the existing grade and design elevation.

This alternative carries the same storm damage reduction features as Alternative 1 for the Long Island Sound Reach (refer to Figure 1). It varies from Alternative 1 in the selection of the storm damage reduction features for the Mill Creek and W. Harbor Drive Reaches. The Mill Creek Reach is protected by a buried floodwall and the W. Harbor Drive reach is protected by a raised road.

The Mill Creek Reach consists of 2,260 LF of new buried floodwall (refer to Figure 5 below). The buried floodwall was selected for the Mill Creek Reach to better represent the existing natural landscape. The proposed Floodwall design of Alternative 1 is a hardened concrete structure, and the buried floodwall proposed in Alternative 2 may be more aesthetically pleasing to the community. The buried floodwall, however, has approximately four times the footprint of the proposed Floodwall in Alternative 1. The larger footprint increases encroachment of private property and/or salt marsh. In the attempt to minimize impact to the residential property and of the vegetated salt marsh, the alignment of the buried floodwall is typically set so that the landward toe of the levee aligns with the southern real estate boundary of the residential properties.

The buried floodwall will be constructed of a silty sand fill with a sheet pile core. Seepage through/under the buried floodwall could be controlled with a sheet pile cut-off wall. The depth of the sheet pile will be optimized after the TSP milestone. The buried floodwall is designed with a 10-foot-wide crest and 1V:2H side slopes. The crest of the buried floodwall will be set to the design elevation of 13.0. Since the residential backyard elevations are generally at elevation 7 feet, the Mill Creek buried floodwall will be perceived as 6 feet tall. In order to protect the levee from erosion, the seaward face of the levee will be armored with 6"- to 8"-diameter stone. The landward slope of the levee will be planted with native grasses to blend the levee into the natural environment.

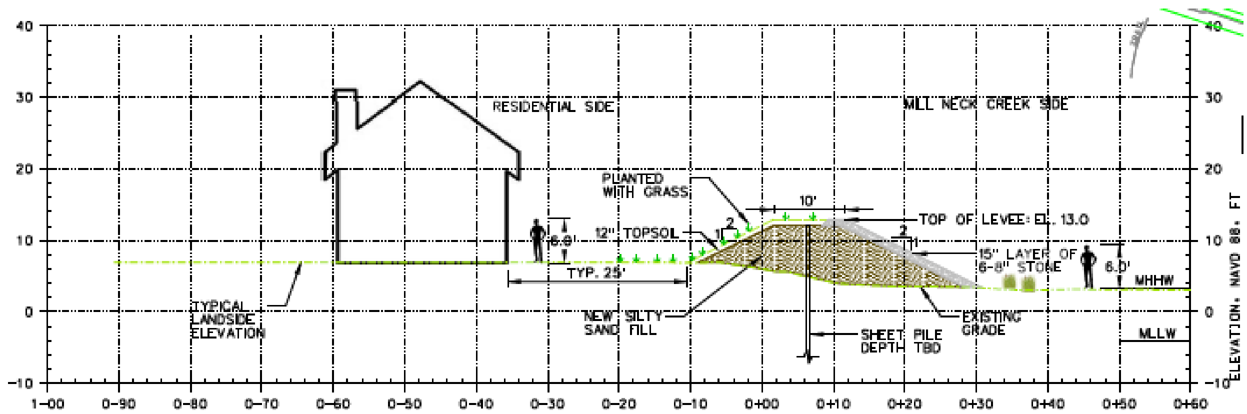


Figure 5: Proposed Buried floodwall along Mill Creek Reach.

The W. Harbor Drive Reach consists of 5,700 LF of new raised road (refer to Figure 6 below). The alignment of the existing W. Harbor Drive will remain unchanged. The road is raised 3 feet from El 10 to the design elevation of 13 feet. The geotechnical engineer determined that seepage is not a concern for this reach, and therefore, a sheet pile cut-off wall was not included in the design. In locations where other roadways intersect with the elevated reach of W. Harbor Drive or the buried floodwall, road raisings are included to maintain a consistent level of protection around the village, and facilitate vehicular access. Alternative 2 maintains the same road raising at intersections as Alternative 1, but also raises 7 driveways and 3 roads that intersecting with the elevation section of W. Harbor Drive. The roads and driveways that intercept with W. Harbor Drive are sloped at a maximum grade of 6% in order to match existing road elevations.

For this alternative, W. Harbor Drive will need to be repaved, and a new concrete sidewalk and guardrails will be installed. The road-raising alternative causes conflict with existing utilities. For the W. Harbor Drive road, it is assumed that the existing water main will remain in place and be further buried by the raised road. The shut off valves and fire hydrants, however, will need to be adjusted to finished grade. Storm water structures will need to be adjusted with new risers and grates. It is assumed that the overhead electrical will maintain appropriate clearance after the road is raised, and will therefore remain in place.

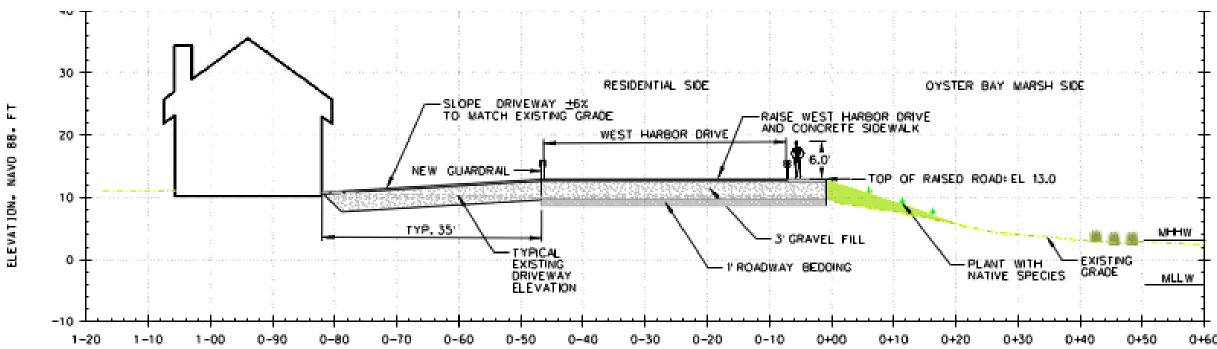


Figure 6: Proposed Raised Road along W. Harbor Drive Reach

This alternative meets the overall project objective of reducing storm flooding in the Village of Bayville from Long Island Sound and Oyster Bay inundation. There are waterfront access impacts and partial water view obstructions for this alternative. The property owners along W. Harbor drive will incur the same water view obstructions as identified in Alternative 1, but those driving or walking on the raised road and sidewalk will not have obstructed views. Even though the same water view obstruction exists for the buried floodwall or Floodwall along the Mill Creek reach, residents may prefer the natural looking obstruction instead of the concrete Floodwall proposed in Alternative 1. The buried floodwall will require more annual maintenance than the Floodwall alternative, because the buried floodwall will necessitate semi-annual mowing.

3.1.3 Alternative 3: Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Raised Road

Refer to Figure 15 of main report for General Site Plan of Alternative 3

Reach	Feature	Length (LF)	Height (Ft)*
LI Sound	Floodwall	4,500	9
LI Sound	Buried floodwall	3,000	7 to 1
Mill Creek	Floodwall	2,260	6
W. Harbor Drive	Raised Road	5,300	3

*Height is approximate and refers to the average difference in elevation between the existing grade and design elevation.

Alternative 3 is a combination of Alternatives 1 and 2. This alternative carries the same storm damage reduction features as Alternative 1 for the Long Island Sound Reach (refer to Figure 1 and 2) and the Mill Creek Reach (refer to Figure 3). However, this alternative carries the same storm damage reduction feature as Alternative 2 for the W. Harbor Drive Reach (refer to Figure 6). This alternative includes the construction of Floodwalls and Buried floodwalls along the Long Island Sound Reach, and a raised road along the W. Harbor Drive Reach.

3.1.4 Alternative 4: Floodwall/Buried floodwalls along LI Sound, Mill Creek Floodwall, W. Harbor Set-back Flood Wall

Refer to Figure 16 of main report for General Site Plan of Alternative 4

Reach	Feature	Length (LF)	Height (Ft)*
LI Sound	Floodwall	4,500	9
LI Sound	Buried floodwall	3,000	7 to 1
Mill Creek	Floodwall	2,260	6
W. Harbor Drive	Set-back Flood Wall	5,300	3

*Height is approximate and refers to the average difference in elevation between the existing grade and design elevation.

Alternative 4 is similar to Alternative 1 by carrying the same storm damage reduction features for the Long Island Sound Reach (refer to Figure 1 and 2), and the Mill Creek Reach (refer to Figure 3). This alternative varies from the others with the selection of a flood wall along the median of W. Harbor Drive (refer to Figure 7 below).

This alternative consists of 5,300 LF of new Floodwall set-back along W. Harbor Drive Reach. The new Floodwall consists of a sheet pile core and concrete cap. The existing W. Harbor Drive is approximately 40 feet wide and generally at elevation 10 feet. The existing width allows for two proposed 12-foot-wide travel lanes, 2-foot-wide shoulders and an 8-foot-wide median. The proposed Floodwall will be installed in the center of the 8-foot median. The top of wall will be set to the design elevation of 13.0 feet. According to the geotechnical engineer, seepage is not a concern along W. Harbor Drive, so the depth of the Floodwall is designed for stability only. The Floodwall design, including sheet pile depth, will be optimized after the TSP milestone.

There are three breaks in the set-back Floodwall for traffic access. The breaks are located at the intersection of Hilary Drive and W. Harbor Drive, the intersection of Alan Drive and W. Harbor Drive, and the intersection of Park Access Road and W. Harbor Drive. In these locations, both sides of W. Harbor Drive and the road intersection is raised to the design elevation of 13.0 feet to allow for traffic at the introduction to travel in either direction on W. Harbor Drive. The raised portion of the road will then sloped at a maximum of grade 6%.

This alternative meets the overall project objective of reducing storm flooding in the Village of Bayville from Long Island Sound and Oyster Bay inundation. There are the same waterfront access impacts and partial water view obstructions for this alternative as the other alternatives. The major difference between this alternative and the other alternatives, is that the entire W. Harbor Drive is not protected to the same level. During storm events, the bayside of the road is not protected or raised, and is therefore at a higher risk for inundation. The village will need to prepare for one-way travel along W. Harbor Drive during storm events. The storm damage reduction feature selected for W. Harbor Drive in Alternatives 1, 2 and 3, however, protects the entire width of the road. One benefit that sets it apart from the other alternatives, is there is zero encroachment on the Oyster Bay Marsh. The other three alternatives will require some excavation and/or fill of the seaward of the existing sidewalk.

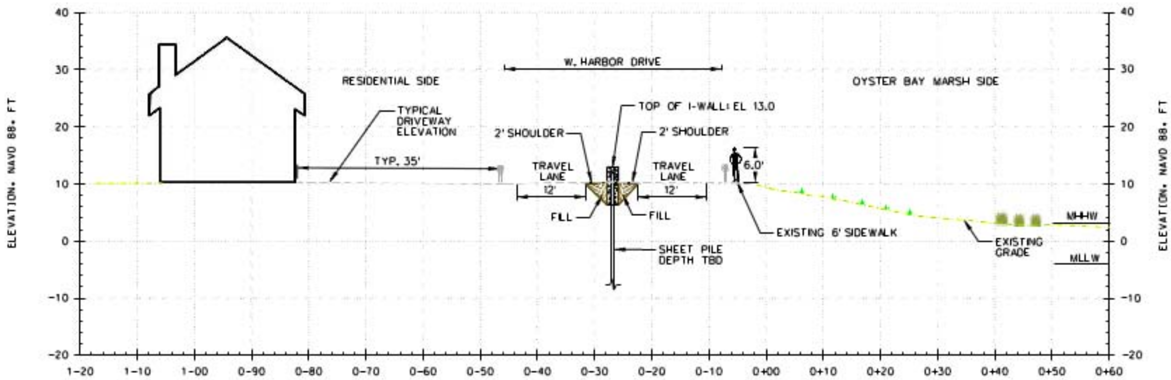


Figure 7: Proposed Set-back Floodwall along W. Harbor Drive Reach

Chapter 4: Project Tie-In Elevations

The natural topography of the northwest portion of the project area rapidly rises in elevation. This higher elevation will be the western tie-in for the Long Island Sound Reach. The north east portion of the project area does not require a tie in because the Long Island Sound Reach transitions to the W. Harbor Drive Reach with a raised road at the intersection of W. Harbor Drive and Bayville Ave. The south west tie-in is the limiting factor when determining the maximum design elevation for the project. The elevation along Shore Road, which is directly west of the project area, drops in elevation from elevation 9 at the intersection with Arlington Lane, to elevation 7 at the intersection with Godfrey Ave. The storm damage reduction feature must have an upland tie-in elevation so that the feature does not get flanked during flood elevations under the design height. The highest elevation in the southwest portion of the project area is elevation 13 located at Saltaire Lane. Elevation 13, therefore, becomes the limiting design elevation. The storm damage reduction feature (i.e. Floodwall or buried floodwall) terminates at the intersection of Shore Road and Saltaire Lane. Then, to complete the tie-in, Saltaire Lane will be raised to elevation 13 and match the existing elevation 13 contour located further up Saltaire Lane. The elevation 13 contour will delineate the extent of storm flooding protection. Increasing the design elevation in the south west portion of the project beyond elevation 13.0 could not be achieved because of the elevation of existing private properties in this area.

Chapter 5: Right-of-Way

The proposed alternatives will require acquisition of a right-of-way corridor wide enough to allow for the footprint of all permanent design features as well as enough room for future flood event monitoring and recurring inspection activities. For the final array of alternatives, a permanent easement of 10 feet landward of any permanent design feature and 20 feet seaward of any permanent design feature is needed.

For Floodwalls, permanent easement offsets are taken from the limit of toe protection on the seaward side. On the landside, offsets are taken from the back of the scour protection. For buried floodwalls, permanent easement offsets are taken from the toe of the seaside and landside slopes. For raised roads, offsets are taken from the edge of road.

Chapter 6: Utilities

Utility costs at this point in the study have been captured with the interior drainage and pumping stations associated with the Minimum Facility Analysis. See the Hydrology and Hydraulics Appendix for additional details on the "Minimum Facility Analysis". The interior drainage features and pumping stations are incorporated into all four structural alternatives. The features will be further optimized following the TSP milestone.

This hydraulic analysis calls for the installation of new storm drainage system along existing streets, and pumping stations and one-way tide valves at the perimeter of the proposed storm damage reduction features. The majority of utility conflicts encountered on this project are due to the proposed storm drains and trench drains along the existing street and roads of Bayville. The intent of this design is to keep the existing storm water system intact to the maximum extent possible, however in locations where the proposed storm drain cannot be co-located with the existing storm drains, the existing structures such as pipes, manholes, and catch basins will be demolished. It is assumed that the existing gas main can be avoided during the installation of the proposed storm and trench drains, however there will be conflicts with existing gas service lines. It is assumed that the homes on one side of the street will have their gas service adjusted. It is also assumed that the proposed storm drains will interfere with the water main at all street intersections. The proposed storm and trench drains will also interfere with water services. It is assumed that half of the homes will have their water service adjusted. The existing water main will need to be adjusted to run below the proposed storm drains.

Chapter 7: Pumping Station Design

7.1 Pumping Station Introduction

This Appendix presents proposed designs for three pumping stations which are required to remove interior surface runoff behind the proposed storm damage reduction structural features described elsewhere in this document.

The pumping stations will be located underground adjacent to the storm damage reduction feature and will pump through the structural feature. A pumping station (Pump Station "A") with total capacity of 72 cubic feet per second will be located at the bay end of Jefferson Avenue to account for flow within Drainage Area A Lower. A second pumping station (Pump Station "B") with total capacity of 45 cubic feet per second will be located adjacent to West Harbor Drive, between 14^{th} Street and June Avenue to account for flow within Drainage Area B. A third pumping station (Pump Station "C") with total capacity of 33 cubic feet per second will be located adjacent to West Harbor Drive close to the intersection with Center Island Road to account for flow within Drainage Area C. All stations will have gravity conduits with duckbill valve closures to allow interior drainage to flow into the bay by gravity under normal conditions. Back-up power will be provided for each station by means of natural gas powered generators located adjacent to each station.

In the attempt to reduce visual impact, it was decided to use submersible pumps for this project. The emergency generators will be housed inside watertight structures with ventilation openings located above maximum predicted flood level.

This report provides design recommendations and a list of major mechanical and electrical equipment required for the pumping stations.

7.2 Pump Selection

Design flow rate for Pump Station "A" is 72 cfs, Pump Station "B" is 45 cfs, and Pump Station "C" is 33 cfs. The intent of the pump selection is to choose one universal pump which can be used in multiples for each station in order to achieve the peak flow condition. Selecting one type of pump allows ease of operation and maintenance because parts will be interchangeable between all pump stations.

The pumps will be configured to alternate between starts and will be controlled with variable frequency drive (VFD) equalization. The VFP regulates the speed of individual pumps and fine tunes the transition stage of hydraulic flow when multiple pumps are running. As multiple pumps are activated, the analog output signal recalculates so that the net flow through the discharge piping is averaged over the activated pumps. The controlling signal adjusts, so that all the pumps are running at the same speed to convey the required flow. Equally sharing the flow across multiple pumps allows the pumps to create less starting load on the emergency generators and allows them to operate within a more efficient range.

In all pump stations, each pump will be capable of approximately 5400 gallons per minute at 29 feet TDH. This represents the worst-case flood scenario and is based on flood water level and the invert elevation. The flood water level is set from the crest of the flood reduction structure. For the bayside, that elevation is 13.0 ft. A 12-inch ductile iron force main will act as an outlet and will have a Tideflex (or equivalent) valve set to elevation 7.0 ft. Actual layouts of pumping stations and associated force mains will vary with final design and actual pump analysis will be refined at that time. Using VFD equalization, each pump will be capable of pumping the same required station capacity with lower or no flood conditions at the outlet of the force mains.

7.3 Pumping Station Design

In order to allow the pumps to be left in place all year, they must be installed low enough so that the normal water level remaining in the pump stations is roughly 4 feet. This will also provide sufficient submergence to meet NPSH requirements.

The pumps will be automatically controlled using a conductive probe with VFD equalization programming. The pump control unit will be programmed with the capability of a start and stop level percentage for each pump, VFD start speed, and VFD maximum speed percentage. The controller will be capable of monitoring pump run time, maximizing pumps to run simultaneously, maximizing pump starts per hour, blocking pump detection, and setting alarm lights for pump malfunction, seal failure, low level in sump and power failure.

The pumping stations will be pre-cast concrete tunnel tanks assembled on-site with approximate size and configuration as shown in Figure 1, Sheet C-501. Submersible pumps and connection fittings will be located at the bottom of the chamber as detailed. The pumps will be provided with guide rails and a quick-disconnect feature. The quick disconnect is a sliding flange on the pump discharge which allows the pump to be lowered into the sump or lifted out without bolting. When the pump is lowered into position, the weight of the pump wedges the pump flange against the mating discharge pipe flange, thus ensuring a tight fit. Lockable aluminum hatches, large enough to pass the pump through the opening, will be provided over the center of each pump.

Three-phase power at 480 volts will be required to run the pumps and backup power will be required in the form of a gas-fired engine generator with automatic transfer switch. The local electric and gas companies will be contacted to determine availability for providing 3-phase power and medium-pressure gas to the pumping station generators.

The generator and pump control panels will be housed in a watertight building with ventilation openings above flood elevation. Each pumping station will use a similar building. Each building will require ventilation to provide the generator and panel with adequate air circulation. The buildings will not be heated.

7.4 Outlet Works

Every effort will be made to maintain gravity discharge to the maximum extent possible. The pumping station will include a bypass weir for low-flow events (Refer to Figure 8 for schematic). The outlet structures discharge toward the marsh and will be fitted with duckbill valves. The outflow will dissipate across a stone level spreader, and then travel along a proposed low flow channel. The level spreader consists of stone bedding and rip rap. The low-flow channel will need to be dredged within the salt marsh to encourage flow during low tide.   The low-flow channel will follow natural low flow channel to the maximum extent practical. The outlet works will result in permanent modifications to the existing salt marsh.

7.5 Future Design Considerations

Elevations shown on C-501 for the pipe inverts, finished grade, and bottom of pumping station are preliminary at this time. When elevations are finalized during final design, the pumping station profiles should be adjusted accordingly to provide adequate NPSH and sufficient vertical pumping range to take advantage of drain line storage. Calculations and details for buoyance forces acting on the pump station have been not been completed at this time.

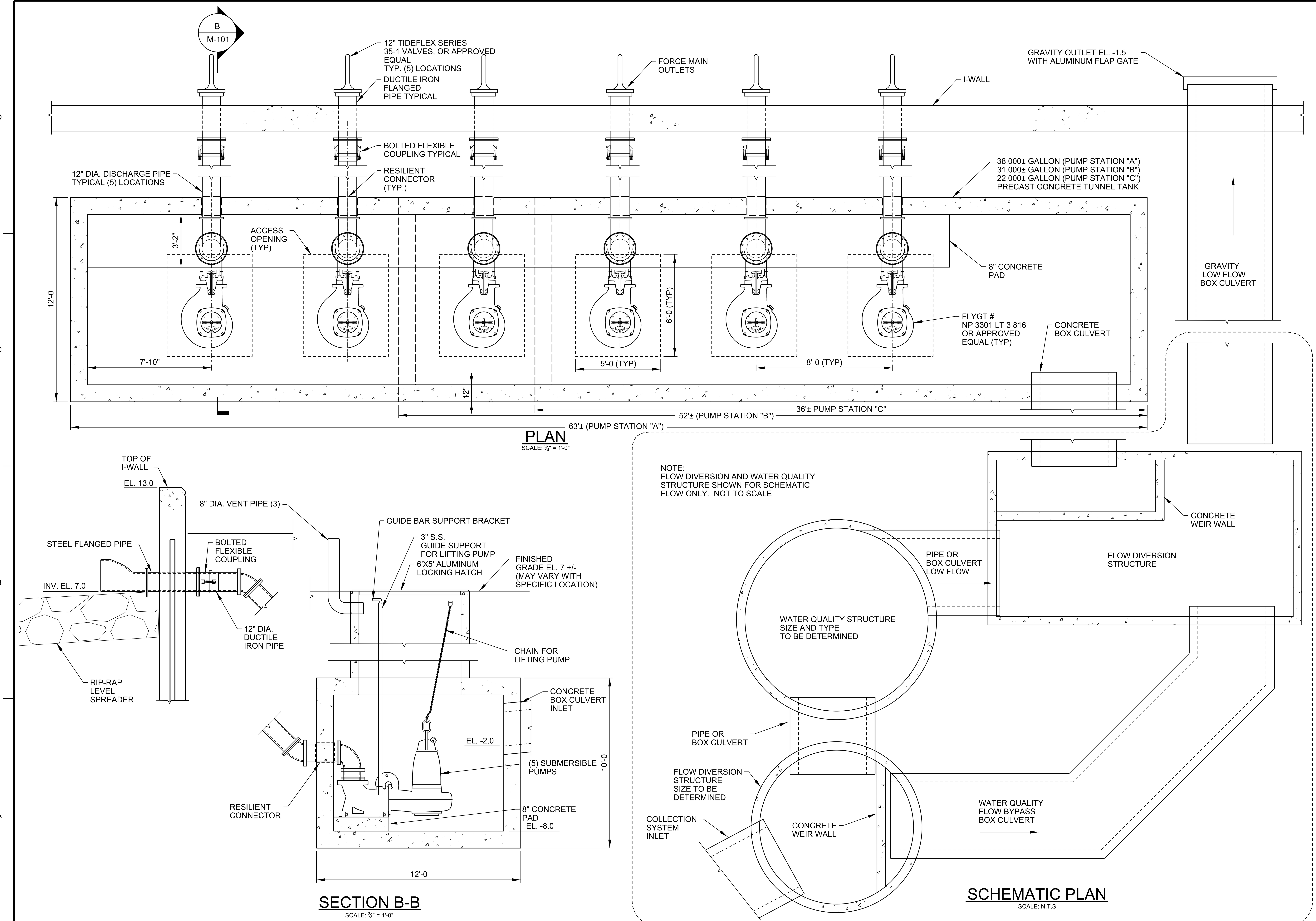
7.6 List of Major Mechanical and Electrical Equipment

- Pumping Station “A”: 6 centrifugal, submersible pumps, 12” DI discharge, flow capacity 5500 gpm at 29 ft TDH. Pumps capable of 5500 gpm at 22 ft TDH using a VFD controller to account for required pump runs with no flood condition acting on force main outlets. Pumps are Flygt model 14" NP3301.185-816 impeller with 60hp 460/3/60 885rpm motors
- Pumping Station “B”: 4 centrifugal, submersible pumps, 12” DI discharge, flow capacity 5250 gpm at 30 ft TDH. Pumps capable of 5100 gpm at 24 ft TDH using a VFD controller to account for required pump runs with no flood condition acting on force main outlets. Pumps are Flygt model 14" NP3301.185-816 impeller with 60hp 460/3/60 885rpm motors
- Pumping Station “C”: 3 centrifugal, submersible pumps, 12” DI discharge, flow capacity 5250 gpm at 30 ft TDH. Pumps capable of 5100 gpm at 24 ft TDH using a VFD controller

to account for required pump runs with no flood condition acting on force main outlets. Pumps are Flygt model 14" NP3301.185-816 impeller with 60hp 460/3/60 885rpm motors

- Pump control system including automatically controlled using a conductive probe with VFD equalization programming. Alarm signals to be provided for pump malfunction, pump blockage, pump shaft seal leakage, low water level in sump, and loss of power.
- Back-up generator, natural gas powered, (Kw to be determined), 480 Volt, 3-phase with automatic transfer switch and standard controls and alarms.
- Medium-pressure gas service from local utility.
- Three-phase, 480 Volt electric service from local utility.
- Wiring and conduit for pumps
- Power and lighting for generator building.
- Grounding and surge protection.

Figure 8: Pump Station Schematic



US Army Corps of Engineers®
New England District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U. S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DISTRICT
CONCORD, MASSACHUSETTS

DESIGNED BY: L. THIBODEAU		DATE: OCTOBER 2015
DWN BY: LT	CKD BY: M. TESSIER	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE: 3/8" = 1'	PLOT DATE:	DRAWING CODE:
SIZE: ANSI D	FILE NAME: BAY_1A_C-501.dgn	

NORTH SHORE OF LONG ISLAND
STORM DAMAGE REDUCTION AND BEACH
EROSION CONTROL - FEASIBILITY STUDY
VILLAGE OF BAYVILLE, NEW YORK

FIGURE 8
PUMP STATIONS
SCHEMATIC FLOW, PLAN, & SECTION

SHEET IDENTIFICATION
C-501
SHEET 1 OF 1

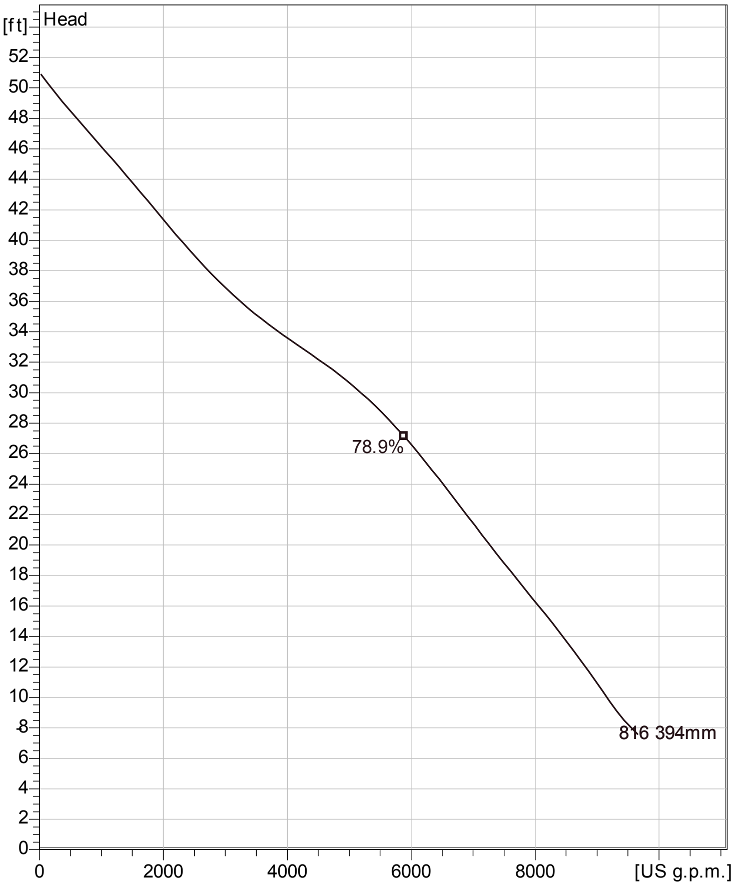
APPENDIX 1

TYPICAL PUMP ANALYSIS BY

MANUFACTURER'S VENDOR

NP 3301 LT 3~ 816

Technical specification



Water, pure *Curve ISO*

Note: Picture might not correspond to the current configuration.

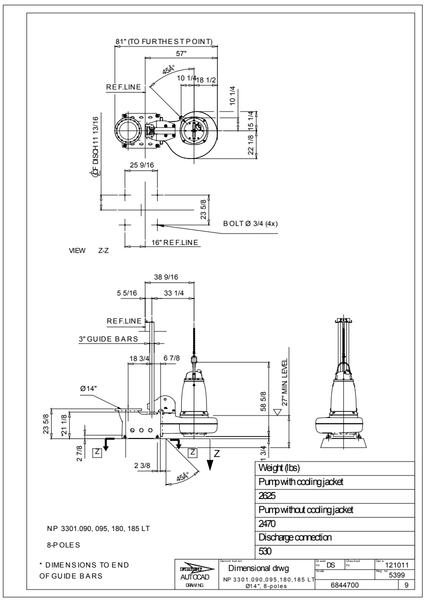
General

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Possible to be upgraded with Guide-pin® for even better clogging resistance. Modular based design with high adaptation grade.

Impeller

Impeller material	Hard-Iron ™
Discharge Flange Diameter	13 3/4 inch
Inlet diameter	350 mm
Impeller diameter	394 mm
Number of blades	2

Installation: P - Semi permanent, Wet



Motor

Motor #	N3301.185 35-29-8AA-W 60hp
Stator variant	1
Frequency	60 Hz
Rated voltage	460 V
Number of poles	8
Phases	3~
Rated power	60 hp
Rated current	80 A
Starting current	440 A
Rated speed	885 rpm
Power factor	
1/1 Load	0.77
3/4 Load	0.72
1/2 Load	0.62
Efficiency	
1/1 Load	90.5 %
3/4 Load	91.0 %
1/2 Load	91.0 %

Configuration

Project	Project ID	Created by	Created on	Last update
Bayville Feasibility Study	**Pump Station A**	**Peter Pastore**	**2015-09-16**	**2015-09-16**

NP 3301 LT 3~ 816

Performance curve

Pump

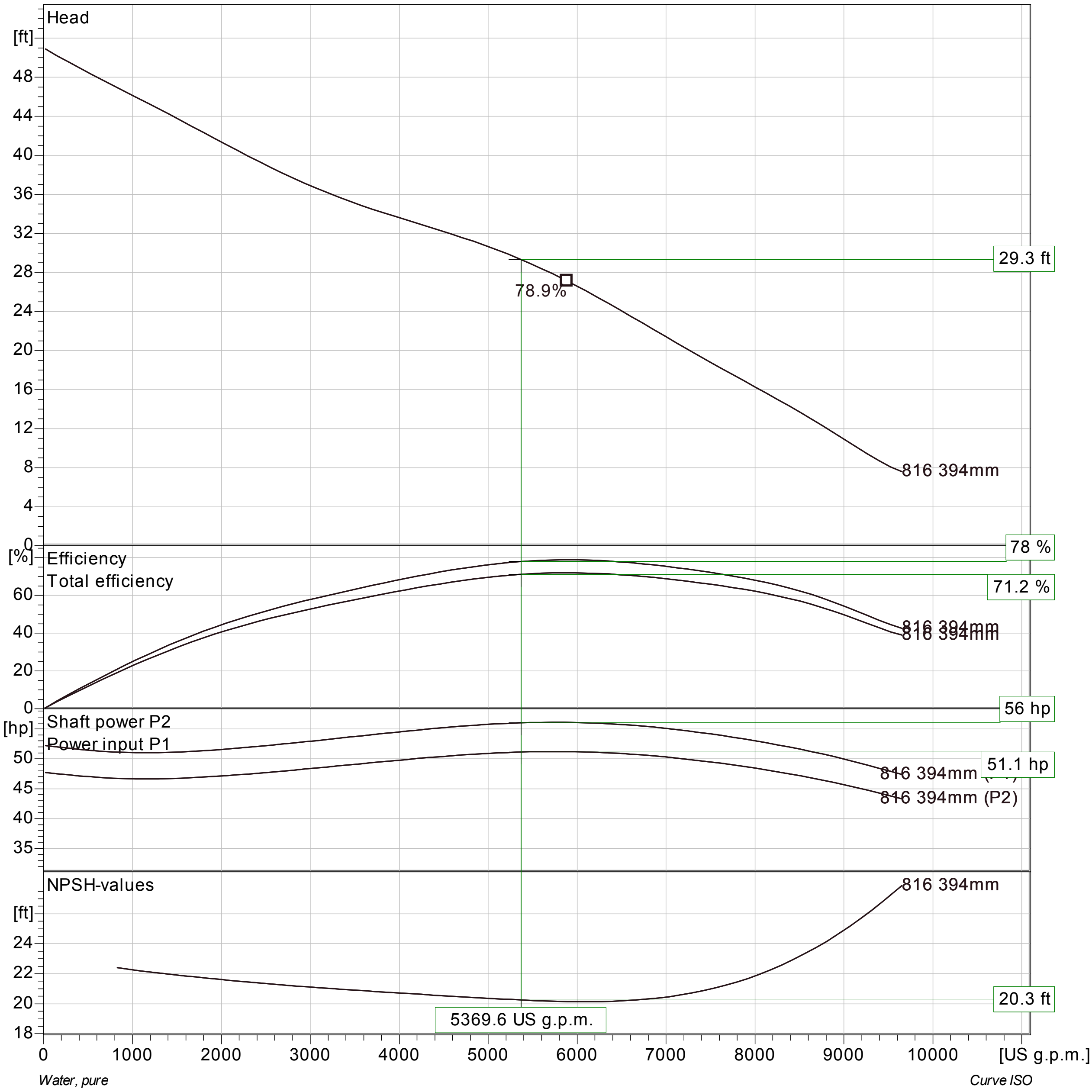
Discharge Flange Diameter	13 3/4 inch
Inlet diameter	350 mm
Impeller diameter	15 1/2"
Number of blades	2

Motor

Motor #	N3301.185 35-29-8AA-W 60hp
Stator variant	1
Frequency	60 Hz
Rated voltage	460 V
Number of poles	8
Phases	3~
Rated power	60 hp
Rated current	80 A
Starting current	440 A
Rated speed	885 rpm

Power factor	
1/1 Load	0.77
3/4 Load	0.72
1/2 Load	0.62

Efficiency	
1/1 Load	90.5 %
3/4 Load	91.0 %
1/2 Load	91.0 %

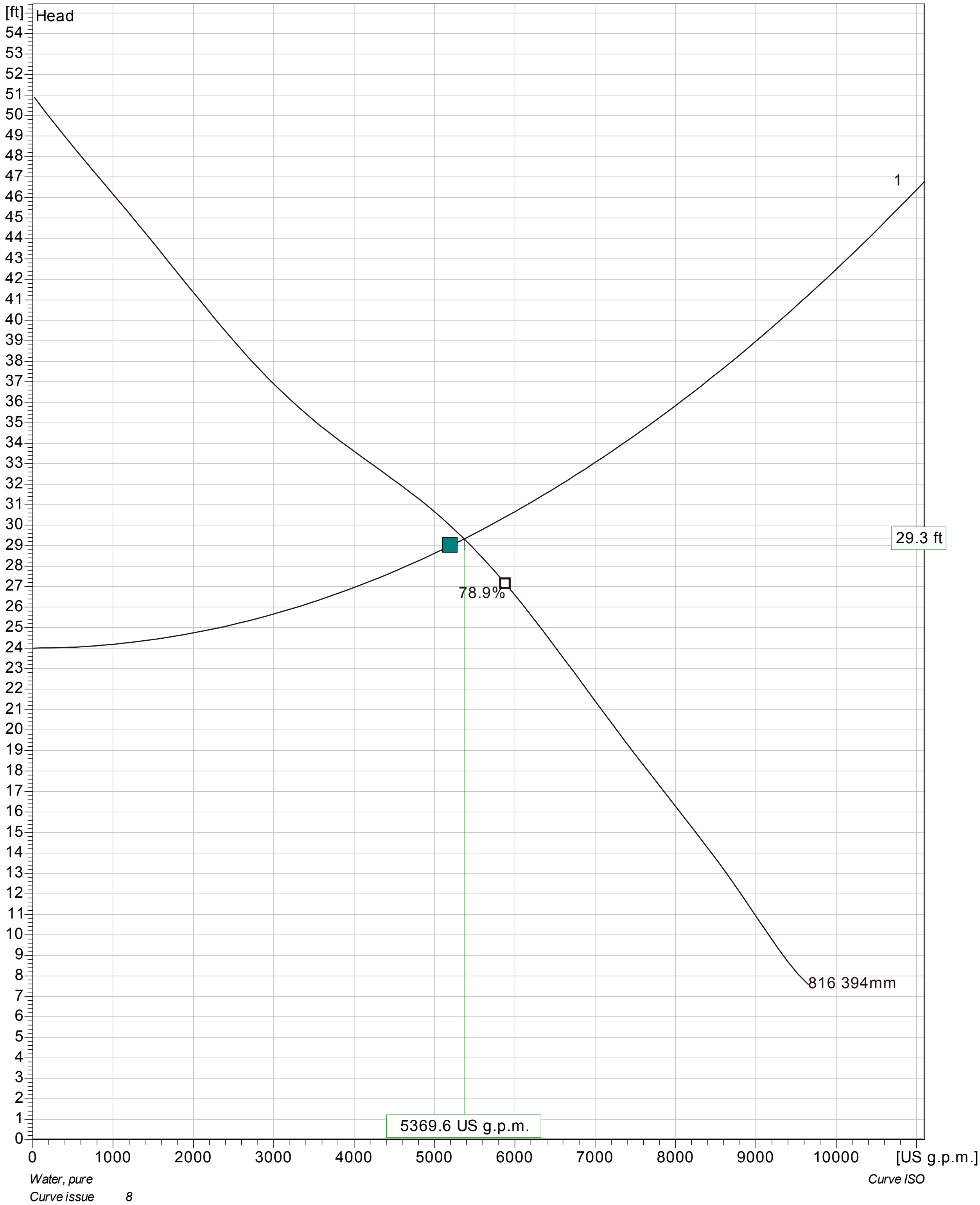


Project	Project ID	Created by	Created on	Last update
Bayville Feasibility Study	**Pump Station A**	**Peter Pastore**	**2015-09-16**	**2015-09-16**

xylem Let's Solve Water

NP 3301 LT 3~ 816

Duty Analysis



Pumps running /System	Individual pump Flow	Head	Shaft power	Total Flow	Head	Shaft power	Pump eff.	Specific energy	NPSHre
1	5370 US g.p.m.	29.3 ft	51.1 hp	5370 US g.p.m.	29.3 ft	51.1 hp	78 %	130 kWh/US MG	20.3 ft

Project	Project ID	Created by	Created on	Last update
Bayville Feasibility Study	**Pump Station A**	**Peter Pastore**	**2015-09-16**	**2015-09-16**

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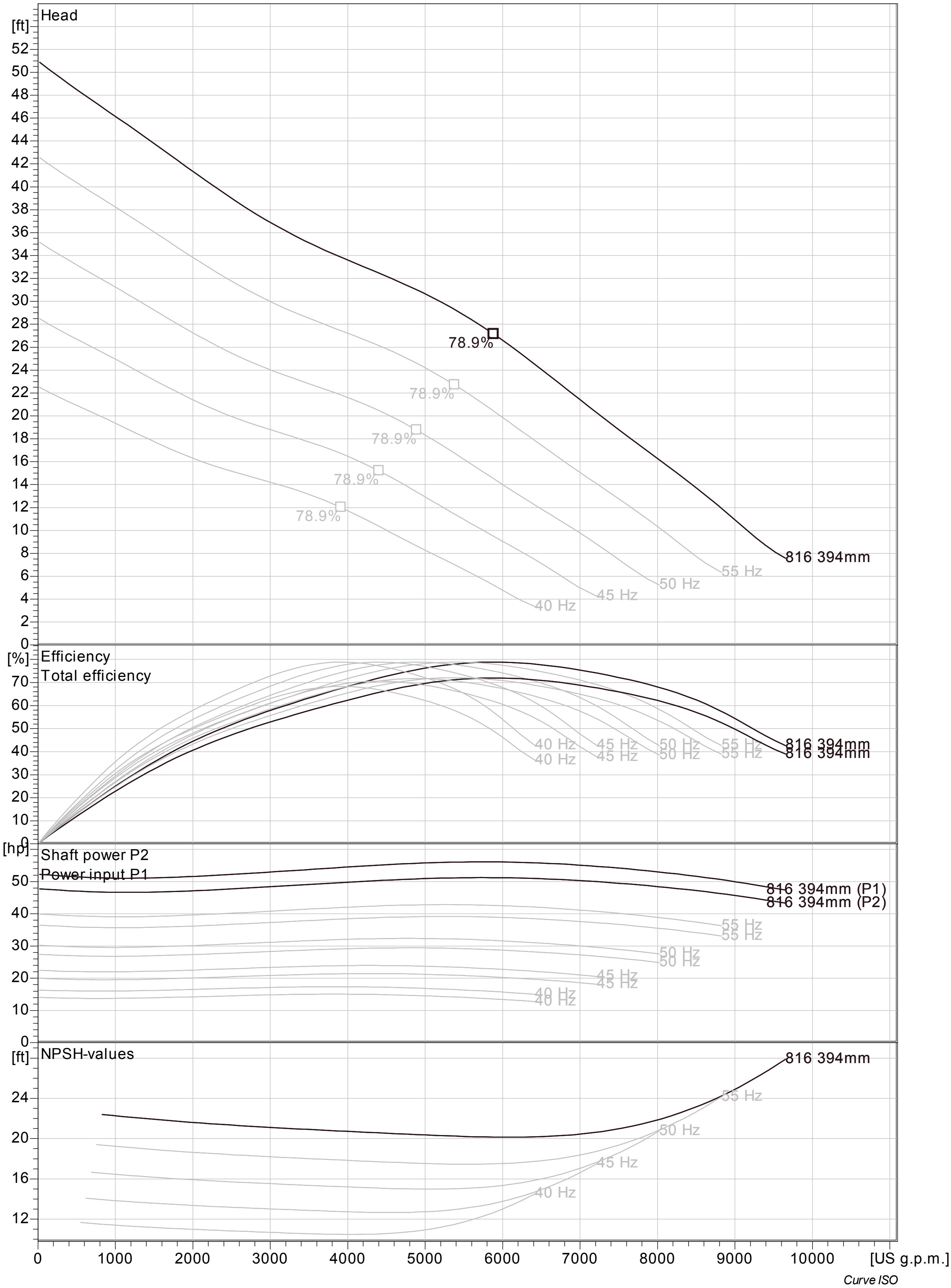
NP 3301 LT 3~ 816

VFD Curve

FLYGT



Project	Project ID	Created by	Created on	Last update
Bayville Feasibility Study	**Pump Station A**	**Peter Pastore**	**2015-09-16**	**2015-09-16**

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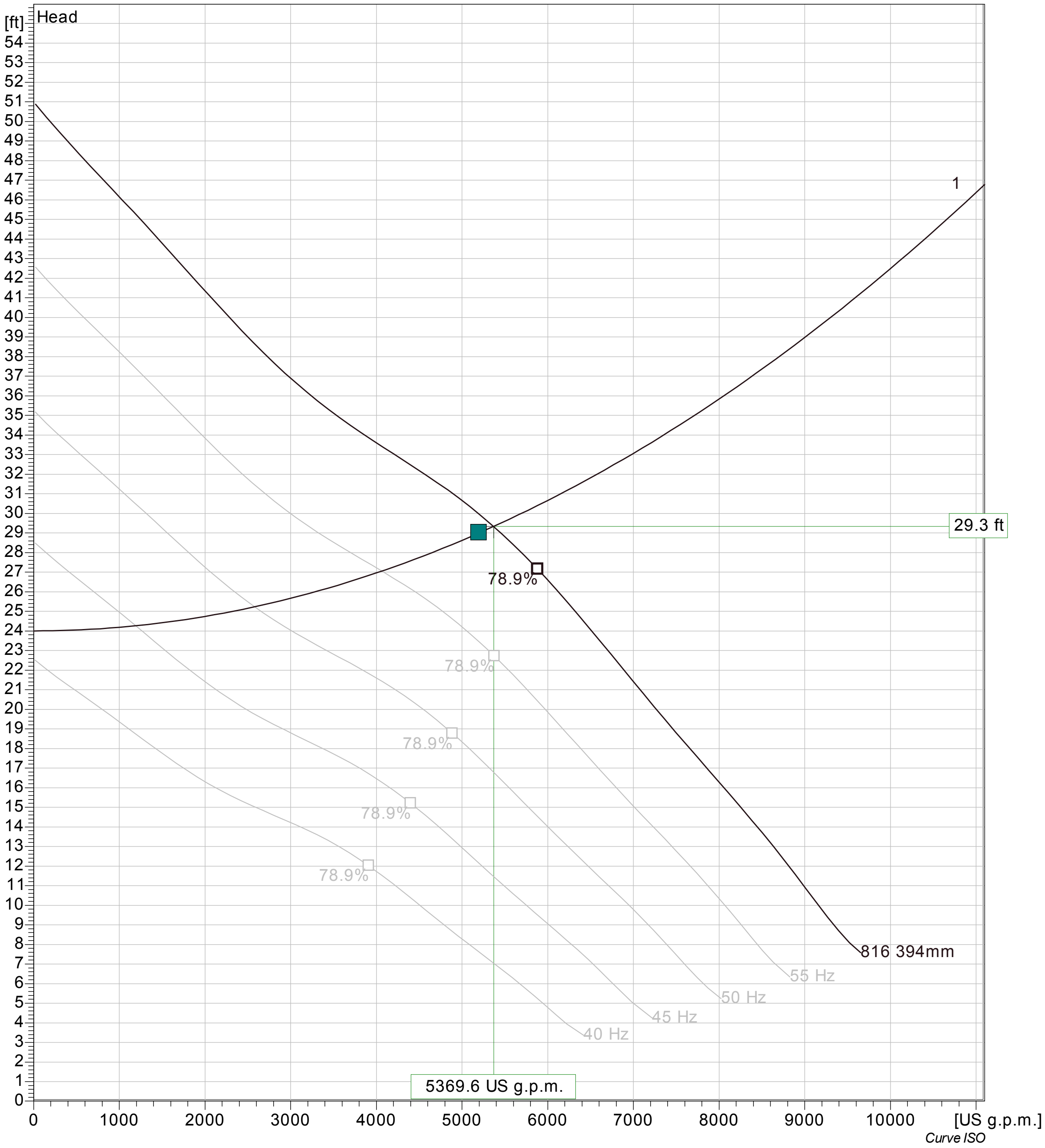
NP 3301 LT 3~ 816

VFD Analysis

FLYGT



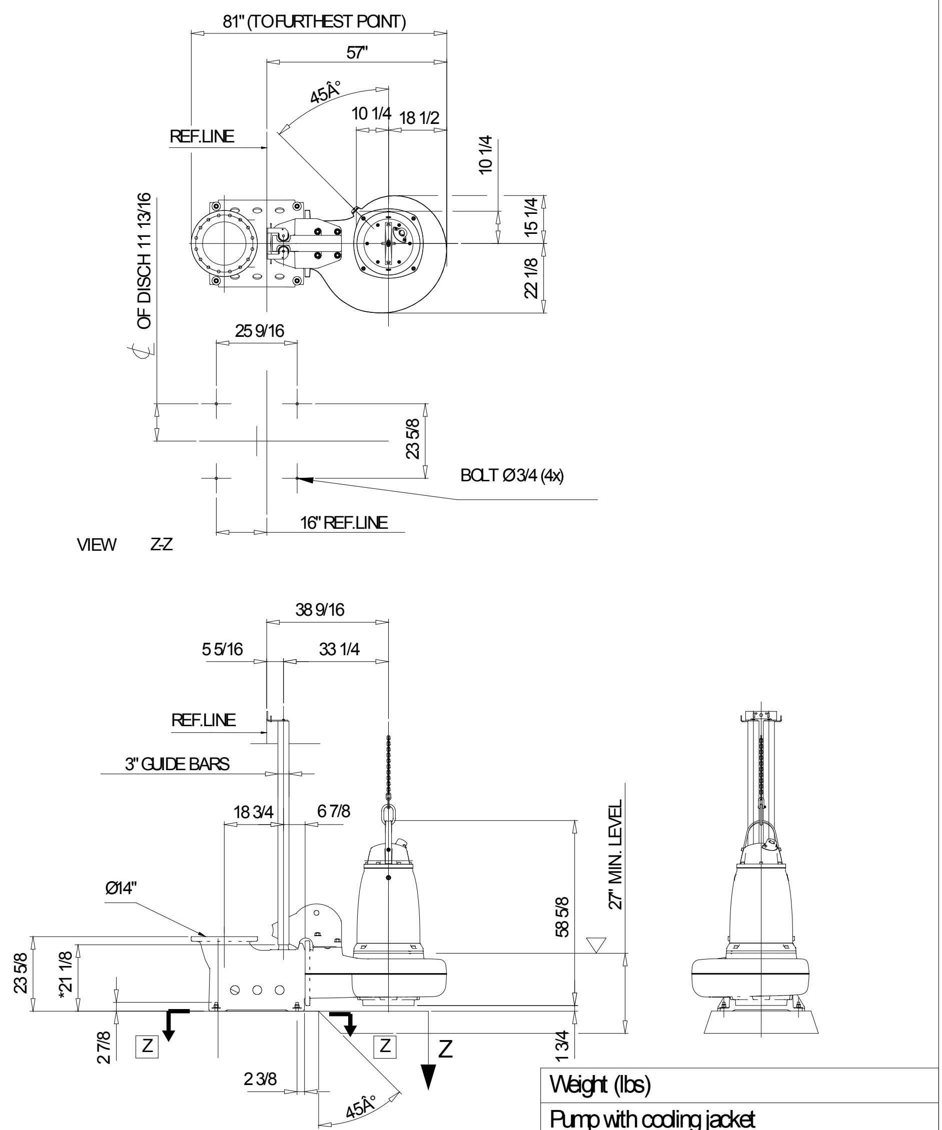
Pumps running /System	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hyd eff.	Specific energy	NPSHre
1	60 Hz	5370 US g.p.m.	29.3 ft	51.1 hp	5370 US g.p.m.	29.3 ft	51.1 hp	78 %	130 kWh/US MG	20.3 ft
1	55 Hz	4060 US g.p.m.	27.1 ft	38.5 hp	4060 US g.p.m.	27.1 ft	38.5 hp	72.3 %	129 kWh/US MG	17.8 ft
1	50 Hz	2580 US g.p.m.	25.2 ft	27.9 hp	2580 US g.p.m.	25.2 ft	27.9 hp	59 %	148 kWh/US MG	15.7 ft
1	45 Hz	1190 US g.p.m.	24.3 ft	19.6 hp	1190 US g.p.m.	24.3 ft	19.6 hp	37.3 %	230 kWh/US MG	13.7 ft
1	40 Hz									

xylem
Let's Solve Water

NP 3301 LT 3~ 816

Dimensional drawing

FLYGT



Weight (lbs)
Pump with cooling jacket
2625
Pump without cooling jacket
2470
Discharge connection
530

NP 3301.090, 095, 180, 185 LT

8-POLES

* DIMENSIONS TO END OF GUIDE BARS

FLYGT AUTOCAD DRAWING	Denomination Dimensional drwg NP 3301.090,095,180,185 LT Ø14", 8-poles	Drawn by DS	Checked by	Date 121011
		Scale		Reg no 5399
		6844700		9

Project	Project ID	Created by	Created on	Last update
Bayville Feasibility Study	Pump Station A	Peter Pastore	2015-09-16	2015-09-16

APPENDIX 2

PUMP DESIGN WORKSHEETS

Pump Design Worksheet

BAYVILLE COASTAL STORM FEASIBILITY STUDY
AREA "A"

Flow Calculations

	Flow	
Flow from H & H to Pump Station Area "A":	72	cfs
Assume 6 equal sized pumps:	12.0	cfs
or	5386	gpm

Pump Chamber Dimensional Data

Height of Inv.In.	6.00	ft (above chamber bottom)
(assumes 24" box culvert)		(assumed)

Static Head Determination

	Elevations	
I-Wall	13.00	(assumes flood to 13.0)
Pump chamber in	-2.00	
Chamber Bottom	-8.00	
Low Low Level Alarm	-7.25	
Low Level Alarm	-4.25	
All Pumps Off	-3.75	
Lead Pump On	-1.25	
12" Tideflex loss	6.5	(assumes force main outlet at 7.0')
Min. Static Head	**20.75**	
Max. Static Head	**23.25**	

Dynamic Head Determination

Total Equivalent Pipe Length

	DI force main	30	
1	DI check valve(s)	27	feet
1	90 deg. bend(s)	32	feet
2	45 deg. bend(s)	25	feet
0	Enlargement	0	feet
	Total	114	feet

Average Inside Diameter vs. Nominal Inside Diameter

DI PIPE - CL 52 (Not lined)

Nom. I.D. (in)	Avg. I.D. (in)
3.00	3.40
4.00	4.22
6.00	6.28
8.00	8.39
10.00	10.40
12.00	12.46
14.00	14.52
16.00	16.60

values based on AWWA/ANSI C150/A21.50-81

Pipe: 12.0"

Average I.D. = 12.46 in.

Area = 0.8468 sf

Piping System Curve Data

Q gpm	v ft/sec	Hf / 100lf ft	Hf ft	TDH (min) ft	TDH (max) ft
0	0.00	0.00	0.00	20.75	23.25
500	1.32	0.05	0.06	20.81	23.31
1000	2.63	0.18	0.21	20.96	23.46
1500	3.95	0.39	0.45	21.20	23.70
2000	5.26	0.67	0.76	21.51	24.01
2500	6.58	1.01	1.15	21.90	24.40
3000	7.89	1.41	1.61	22.36	24.86
3500	9.21	1.88	2.14	22.89	25.39
4000	10.53	2.41	2.74	23.49	25.99
4500	11.84	2.99	3.41	24.16	26.66
5000	13.16	3.64	4.15	24.90	27.40
5500	14.47	4.34	4.95	25.70	28.20
6000	15.79	5.10	5.81	26.56	29.06
6500	17.10	5.91	6.74	27.49	29.99
7000	18.42	6.78	7.73	28.48	30.98
7500	19.74	7.70	8.78	29.53	32.03
8000	21.05	8.68	9.90	30.65	33.15
8500	22.37	9.71	11.07	31.82	34.32
9000	23.68	10.80	12.31	33.06	35.56

values based on average pipe I.D. and the Hazen-Williams formula (C=140)

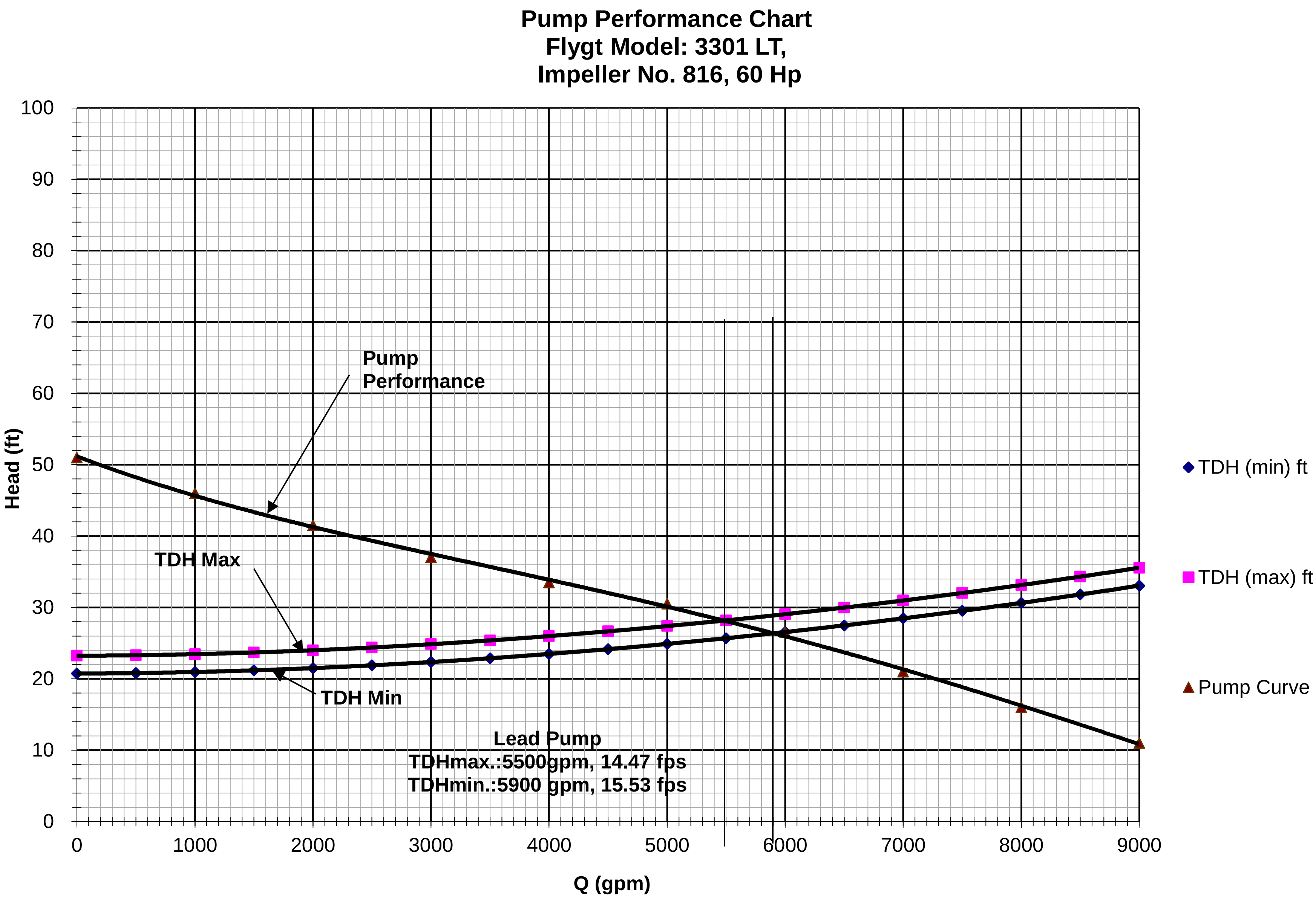
Pump Performance Curve Data

Pump: Flygt 3301.185-816

Q gpm	Head ft
0	51.0
1000	46
2000	41.5
3000	37.0
4000	33.5
5000	30.5
6000	26.5
7000	21.0
8000	16.0
9000	11.0

Velocities

	Flow	Velocity	
at TDH Max.	5500	14.47	ft/sec
at TDH Min.	5900	15.53	ft/sec



Pump Performance Chart
Flygt Model: 3301 LT,
Impeller No. 816, 60 Hp
Head (ft)
Q (gpm)
100
90
80
70
60
50
40
30
20
10
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Pump
Performance
TDH Max
TDH Min
Lead Pump
TDHmax.:5500gpm, 14.47 fps
TDHmin.:5900 gpm, 15.53 fps
TDH (min) ft
TDH (max) ft
Pump Curve

Pump Design Worksheet

BAYVILLE COASTAL STORM FEASIBILITY STUDY
AREA "B"

Flow Calculations

	Flow	
Flow from H & H to Pump Station Area "B":	45	cfs
Assume 4 equal sized pumps:	11.3	cfs
or	5049	gpm

Pump Chamber Dimensional Data

Height of Inv.In.	6.00	ft (above chamber bottom)
(assumes 2-24")		(assumed)

Static Head Determination

	Elevations
I-Wall	13.00
Pump chamber in	-2.00
Chamber Bottom	-8.00
Low Low Level Alarm	-7.25
Low Level Alarm	-4.25
All Pumps Off	-3.75
Lead Pump On	-1.25

12" Tideflex loss 6.5 (assumes force main outlet at 7.0')

Min. Static Head	**20.75**
Max. Static Head	**23.25**

Dynamic Head Determination

Total Equivalent Pipe Length

	DI force main	70	
1	DI check valve(s)	27	feet
1	90 deg. bend(s)	32	feet
2	45 deg. bend(s)	25	feet
0	Enlargement	0	feet
	Total	154	feet

Average Inside Diameter vs. Nominal Inside Diameter

DI PIPE - CL 52 (Not lined)

Nom. I.D. (in)	Avg. I.D. (in)
3.00	3.40
4.00	4.22
6.00	6.28
8.00	8.39
10.00	10.40
12.00	12.46
14.00	14.52
16.00	16.60

values based on AWWA/ANSI C150/A21.50-81

Pipe: 12.0"

Average I.D. = 12.46 in.

Area = 0.8468 sf

Piping System Curve Data

Q gpm	v ft/sec	Hf / 100lf ft	Hf ft	TDH (min) ft	TDH (max) ft
0	0.00	0.00	0.00	20.75	23.25
500	1.32	0.05	0.08	20.83	23.33
1000	2.63	0.18	0.28	21.03	23.53
1500	3.95	0.39	0.60	21.35	23.85
2000	5.26	0.67	1.03	21.78	24.28
2500	6.58	1.01	1.55	22.30	24.80
3000	7.89	1.41	2.17	22.92	25.42
3500	9.21	1.88	2.89	23.64	26.14
4000	10.53	2.41	3.70	24.45	26.95
4500	11.84	2.99	4.61	25.36	27.86
5000	13.16	3.64	5.60	26.35	28.85
5500	14.47	4.34	6.68	27.43	29.93
6000	15.79	5.10	7.85	28.60	31.10
6500	17.10	5.91	9.10	29.85	32.35
7000	18.42	6.78	10.44	31.19	33.69
7500	19.74	7.70	11.87	32.62	35.12
8000	21.05	8.68	13.37	34.12	36.62
8500	22.37	9.71	14.96	35.71	38.21
9000	23.68	10.80	16.63	37.38	39.88

values based on average pipe I.D. and the Hazen-Williams formula (C=140)

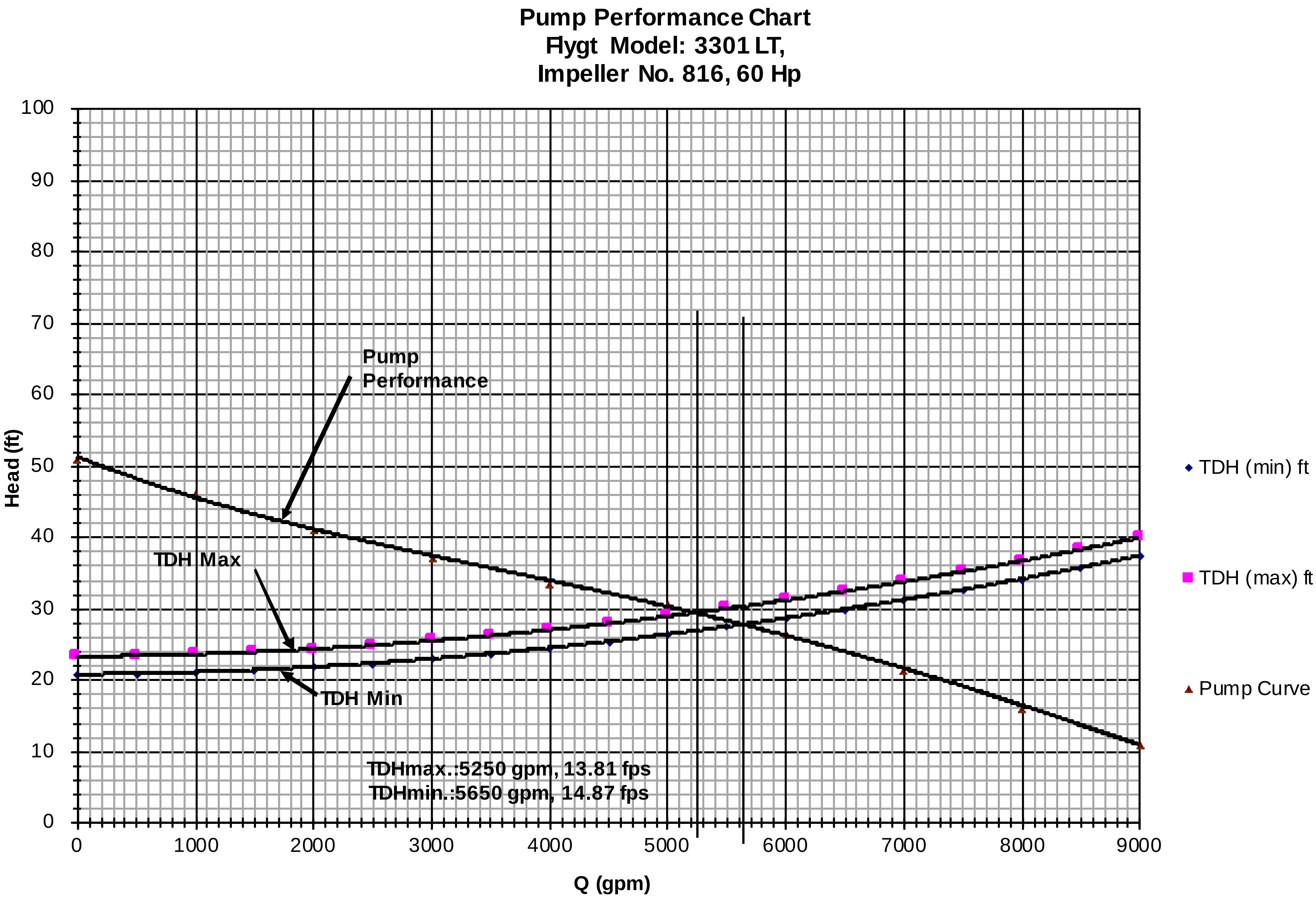
Pump Performance Curve Data

Pump: Flygt 3301.185-816

Q gpm	Head ft
0	51.0
1000	46
2000	41.0
3000	37.0
4000	33.5
5000	30.7
6000	26.5
7000	21.4
8000	16.1
9000	11.0

Velocities

	Flow	Velocity	
at TDH Max.	5250	13.81	ft/sec
at TDH Min.	5650	14.87	ft/sec



Pump Performance Chart
Flygt Model: 3301 LT,
Impeller No. 816, 60 Hp
Head (ft)
Q (gpm)
Pump
Performance
TDH Max
TDH Min
TDHmax.:5250 gpm, 13.81 fps
TDHmin.:5650 gpm, 14.87 fps
TDH (min) ft
TDH (max) ft
Pump Curve

Pump Design Worksheet

BAYVILLE COASTAL STORM FEASIBILITY STUDY
AREA "C"

Flow Calculations

	Flow	
Flow from H & H to Pump Station Area "C":	33	cfs
Assume 3 equal sized pumps:	11.0	cfs
or	4937	gpm

Pump Chamber Dimensional Data

Height of Inv.In.	6.00	ft (above chamber bottom)
(assumes 24" box culvert)		(assumed)

Static Head Determination

	Elevations	
I-Wall	13.00	
Pump chamber in	-2.00	
Chamber Bottom	-8.00	
Low Low Level Alarm	-7.25	
Low Level Alarm	-4.25	
All Pumps Off	-3.75	
Lead Pump On	-1.25	
12" Tideflex loss	6.5	(assumes force main outlet at 7.0')
Min. Static Head	**20.75**	
Max. Static Head	**23.25**	

Dynamic Head Determination

Total Equivalent Pipe Length

	DI force main	70	
1	DI check valve(s)	27	feet
1	90 deg. bend(s)	32	feet
2	45 deg. bend(s)	25	feet
0	Enlargement	0	feet
	Total	154	feet

Average Inside Diameter vs. Nominal Inside Diameter

DI PIPE - CL 52 (Not lined)

Nom. I.D. (in)	Avg. I.D. (in)
3.00	3.40
4.00	4.22
6.00	6.28
8.00	8.39
10.00	10.40
12.00	12.46
14.00	14.52
16.00	16.60

values based on AWWA/ANSI C150/A21.50-81

Pipe: 12.0"

Average I.D. = 12.46 in.

Area = 0.8468 sf

Piping System Curve Data

Q gpm	v ft/sec	Hf / 100lf ft	Hf ft	TDH (min) ft	TDH (max) ft
0	0.00	0.00	0.00	20.75	23.25
500	1.32	0.05	0.08	20.83	23.33
1000	2.63	0.18	0.28	21.03	23.53
1500	3.95	0.39	0.60	21.35	23.85
2000	5.26	0.67	1.03	21.78	24.28
2500	6.58	1.01	1.55	22.30	24.80
3000	7.89	1.41	2.17	22.92	25.42
3500	9.21	1.88	2.89	23.64	26.14
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5500	14.47	4.34	6.68	27.43	29.93
6000	15.79	5.10	7.85	28.60	31.10
6500	17.10	5.91	9.10	29.85	32.35
7000	18.42	6.78	10.44	31.19	33.69
7500	19.74	7.70	11.87	32.62	35.12
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8500	22.37	9.71	14.96	35.71	38.21
9000	23.68	10.80	16.63	37.38	39.88

values based on average pipe I.D. and the Hazen-Williams formula (C=140)

Pump Performance Curve Data

Pump: Flygt 3301.185-816

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Velocities

	Flow	Velocity	
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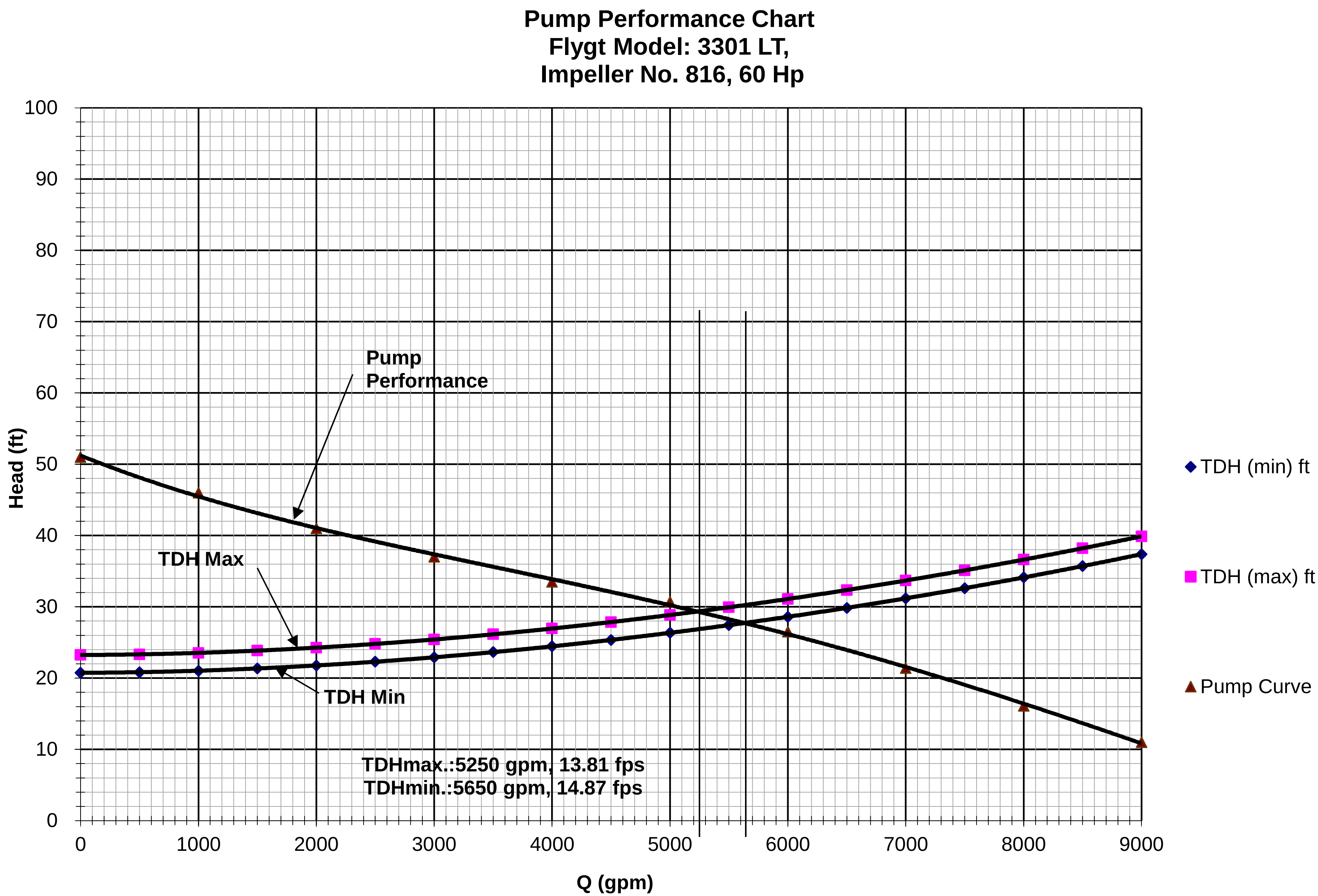

Pump Performance Chart
Flygt Model: 3301 LT,
Impeller No. 816, 60 Hp
Head (ft)
Q (gpm)
0
10
20
30
40
50
60
70
80
90
100
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Pump
Performance
TDH Max
TDH Min
TDHmax.:5250 gpm, 13.81 fps
TDHmin.:5650 gpm, 14.87 fps
TDH (min) ft
TDH (max) ft
Pump Curve