Leonardo, Raritan Bay and

Sandy Hook Bay, New Jersey Coastal Storm Risk Management Feasibility Study

Appendix C Engineering

March 2015

Leonardo, NJ Feasibility Study

Engineering and Design Appendix

March 2015

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C-1. Introduction

C-1.1 Description of the Study Area

Leonardo is located in the northeastern portion of the Township of Middletown in Monmouth County, New Jersey. It approximates 0.5 square miles, and is bounded by Sandy Hook Bay to the north, Wagner Creek to the east, the pier at US Naval Weapons Station Earle to the west, and New Jersey State Route 36 to the south. The Leonardo State Marina divides the shoreline of Leonardo. Figure 1 shows the location map.

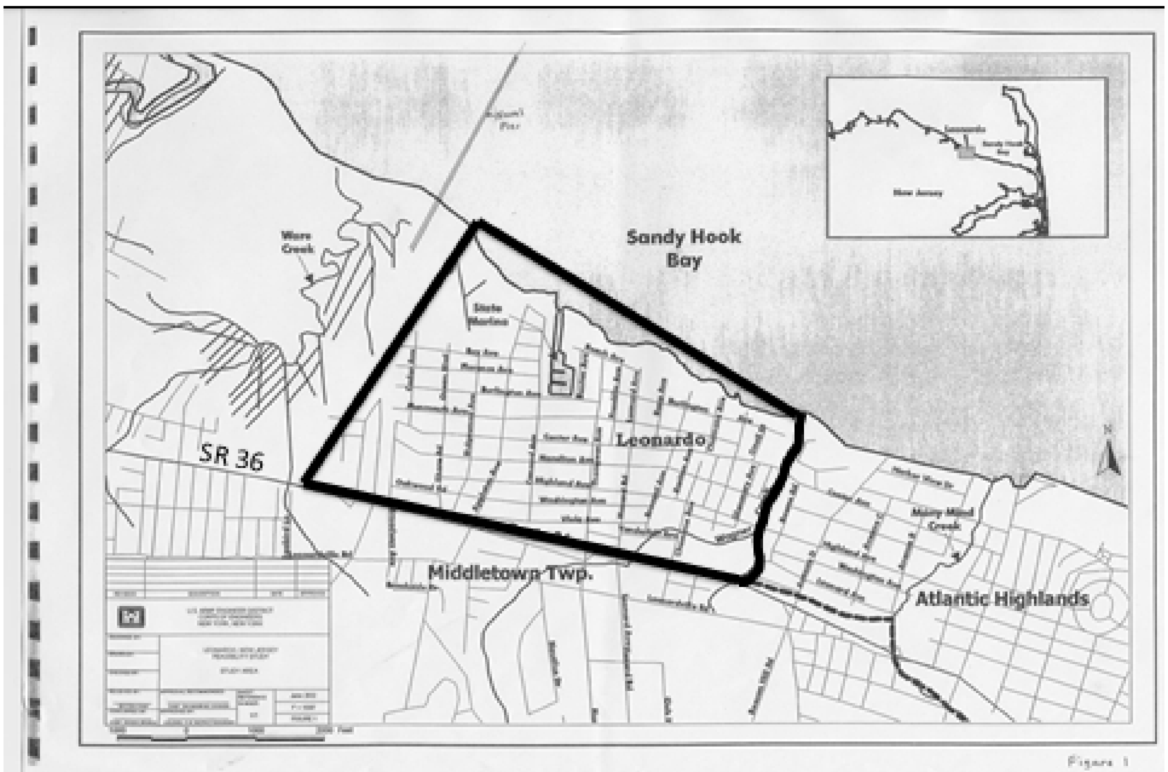


Figure 1: Location Map

C-1.2 Characteristics and Problem Identification of Study Area

Leonardo's topography is dominated by a small knoll with a maximum elevation of about +39 ft North American Vertical Datum of 1988 (NAVD 88). The shoreline areas are of low elevation and subject to storm induced flooding and in some locations, erosion and/or wave damage. Water levels at Leonardo reached +11.7 ft NAVD88 during Hurricane Sandy. In the aftermath of Hurricane Sandy, approximately 250 FEMA claims were filed in Leonardo. Shoreline features, from west to east, are described in further detail below.

The pier at US Naval Weapons Station Earle (traps as a groin)

This formation of a sand bar under the pier may be creating a littoral barrier.

Marsh Reach

A marsh characterizes the western shoreline between the pier at US Naval Weapons Station Earle to the west and the Leonardo State Marina jetties to the east. A low narrow beach with no significant dune fronts a wide low-lying marsh (extending approximately 2,000 ft inland). Residential areas landward of the marsh are subject to tidal and storm-induced flooding. At the mouth of the wetland creek in this marsh, a small ebb shoal is growing.

Leonardo State Marina

The Leonardo State Marina is located at the center of Leonardo, and is stabilized by two timber jetties. The shoreline is offset across the harbor entrance, with the west side having a well-developed fillet. In the last half century, the shoreline adjacent to the harbor jetties has not changed significantly. Bulkhead repair/replacement is performed periodically by the New Jersey Department of Environmental Protection (NJDEP). Low-lying areas surrounding the marina are subject to frequent tidal and storm-induced flooding.

Beach and Dune Reach

The center section of Leonardo (east of the marina) consists of a beach and dune system, extending approximately 1,500 ft. to the east to the intersection of Beach and Leonard Aves., and contains five timber groins. The dune itself is vegetated and broken by several pedestrian-induced low elevation areas. The groins are highly effective, stabilizing the shoreline. The shoreline has maintained its size in the last half century. Each groin has a distinct down drift offset. A comparison of the 1961 and 1974 aerial photographs shows one offset grew substantially. Net littoral drift is to the east based upon fillets to the west side of the groins and offsets to the east. During significant storms in 1992, 2011, and 2012 the intersection of Beach and Leonard Ave. was undermined and washed out by waves.

Bulkhead Reach

The eastern portion of shoreline between Leonard Ave and Wagner Creek is armored with seawalls and timber bulkheads in poor to fair condition. The shoreline in this location is eroded to the structure line. Minimal dry sand fronts most of these structures. Portions of this reach with no bulkheads, poor condition bulkheads, or low elevation bulkheads are subject to storm-induced flooding. A groin and a bulkhead that acts as a groin trap sand and are effective. A distinct offset is apparent at these structures, but the shoreline has not changed in the last half century. During the significant storm event in 1992, Hurricane Irene in 2011 and Hurricane Sandy in 2012 many of the bulkheads themselves experienced wave damage and were repaired or replaced by local owners. However, no damage to buildings was experienced.

Wagner Creek

Leonardo has experienced storm surge induced flooding , along the banks of Wagner Creek. A terminal groin adjacent and west of Wagner Creek effectively traps sand. Ten existing storm water outfalls discharge directly into Wagner Creek. Wagner Creek additionally accepts drainage from upland areas to the south of Leonardo.

C-1.3 Existing Features

Existing features include Federal Navigation and State Marina projects.

Leonardo Channel

This channel is 8 ft deep and 150 ft wide, and extends from the Leonardo State Marina jetties approximately 2,500 ft into Sandy Hook Bay. Initial channel dredging removed 71,592 cubic yards (cy) in 1958, and maintenance operations occurred in 1967, 1991 and 2014; 56,717 cy, 60,412 cy and 35,000 cy, respectively, as shown on Table 1. The average dredging rate from the period between initial dredging and 2014 is approximately 2,700 cy/yr. There are no plans for future dredging.

US Naval Weapons Station Earle Pier Channel

The Navy dredged Piers 2, 3, and 4 in 1997 and 2002 whereby 513,794 cy and 262,518 cy, respectively, was removed and placed at the Mud Dump Site. 56,170 cy of material was dredged in 2003, and was placed in an upland facility. The material contained less than 10% sand. The Navy replaced Pier 3, reducing its length and deepening the adjacent grade from –36.1 ft. NAVD88 to –46.1 ft. NAVD88 in 2008. The 2008 dredging quantity was 318,000 cy which went to the Mud Dump Site. The average dredging rate between 1997 and 2008 is shown in Table 1.

State Marina Dredging

Several State Marina dredging operations occurred during the period between 1982 and the present: 17,800 cy in 1982; 16,000 cy in 1986; 200 cy in 1993, and 2,500 cy in 2003. The resulting maintenance-dredging rate from 1982-2003 is approximately 1,700 cy/yr. These values are shown in Table 1. Dredging was tentatively planned for 2014, but did not occur.

Table 1: Dredging Volumes

Leonardo Federal Channel Dredging Operations			
Year	**Type**	**Volume**	
1957	Initial	71,592	cy
1967	Maintenance	56,717	cy
1991	Maintenance	60,412	cy
2014	Maintenance	35,000	cy
Sum Maintenance		152,129	cy
Avg. Rate 1957-2014		2,700	cy/yr
Earle Naval Pier Dredging Operations			
Year	**Location**	**Volume**	
1997	Peirs 2, 3, and 4	513,794	cy
2002	Peirs 2, 3, and 4	262,518	cy
2003	Unknown (<10% Sand)	56,170	cy
2008	Pier 3 Deepening	318,000	cy
Sum Removed		1,150,482	cy
Avg. Rate 1997-2008		105,000	cy/yr
State Marina Dredging Operations			
Year	**Type**	**Volume**	
1982	Maintenance	17,800	cy
1986	Maintenance	16,000	cy
1993	Shoal Removal	200	cy
2003	Maintenance	2,500	cy
Sum Maintenance		36,500	cy
Avg. Rate 1982-2003		1,700	cy/yr

State Marina Bulkhead

NJDEP constructed sections of bulkhead replacement at Leonardo Marina in 1981, 1984, 1986, 1992, and 2000 using timber sheeting. Crest elevation vary between +4.8 ft NAVD88 to +5.8 ft NAVD88.

C-1.4 Prior Federal Studies

Preliminary Examination of Navy Breakwater (1946)

Concluded that breakwaters and dredging desired by the U.S. Navy are not justified from a commercial navigation standpoint and that the work can best be accomplished with military funds.

Survey Report (1960)

A shore protection project at Leonardo was found to be uneconomic after detailed investigation, although the preliminary analysis indicated the possibility for improvement.

Reconnaissance Report (1996)

The Reconnaissance Report recommended Leonardo for further study noting institutional constraints such as gate impacts to navigation interests, and numerous private shorefront ownerships.

Pre-Feasibility Report (1998)

The following economically justified proposed plan of improvement for Leonardo included construction of a coastal storm risk management dune and beach fill section, bounded by a terminal groin at Wagner Creek to the east and by the Marina jetty to the west. A floodwall and stone revetment was proposed at the eastern end of the fill to tie into higher ground and to prevent storm surge from entering the study area from Wagner Creek. A closure (sector type) gate was proposed at the Marina entrance to provide protection against storm surge entering into the study area through the Marina, along with adding a bulkhead for a 350 ft section fronting existing bulkhead to raise elevation to +13.9 ft NAVD88 on the eastern jetty/bulkhead tip to provide required improvement height. A second floodwall was proposed on the western side of the Marina (extending south from the closure gate along the edge of Concord Ave, approximately 1,200 ft in length) tying into high ground to protect against storm surge from entering the western part of the study area west of the Marina. A third floodwall was proposed along a short segment (approx. 200 lf) of Burlington Ave to prevent storm surge from entering the western study area behind the wetlands area. Elevation or flood proofing was proposed for the residential structures prone to residual flooding damage (outside of the proposed line of coastal storm risk management).

C-2. Existing Conditions

C-2.1 Currents

Tidal currents along the shore of the study area are generally weak except at the entrances to Raritan and Shrewsbury Rivers where the average velocity at strength of the current is 1.8 and 2.6 knots, respectively. A large part of the tidal circulation in the bay occurs in relatively deep-water along an east-west axis approximately 2 miles offshore from the study area.

C-2.2 Water Surface Elevations

Stage-frequency curves for existing conditions were acquired from FEMA for the project location. The stage-frequency curves for the entire region were developed through surge and wave modeling of a suite of synthetic design storms using the ADCIRC (ADvanced CIRCulation)+SWAN (Simulating WAves Nearshore) models. More information on how FEMA develops stage-frequency can be found at http://www.r3coastal.com/home/storm-surge-study. The stage frequency data were taken directly from FEMA without manipulation, although an adjustment was made to get the stage data into the NAVD88 datum. The FEMA stage-frequency curves are referenced to the Mean Sea Level (MSL) datum, so a shift to the NAVD88 datum was necessary for this particular project. The datum conversion from the MSL datum to the NAVD88 datum was calculated to be 0.24 feet. This conversion factor was used since the Sandy Hook gauge is located relatively close to the project site. Table 2 contains the datum information for the Sandy Hook Gauge. The NAVD88 datum is located approximately 1.1 feet above the National Geodetic Vertical Datum of 1929 (NGVD29) datum. Therefore, the conversion for all elevations shown in this report is as follows:
NAVD88 = NGVD29 - 1.1 feet.

The raw ADCIRC+SWAN output, which includes peak surge elevation and associated significant wave heights and mean wave periods, was processed to estimate statistical wave parameters. Figure 2 displays the results of a regression analysis which determines the 20, 10, 6.7, 5, 4, 2, 1.3, 1, 0.4, 0.2, and 0.1% annual chance exceedance wave parameters. The peak surge elevation each of the synthetic storms is plotted against the associated significant wave height and peak wave period. From this trend, we can estimate the wave heights for different surge elevations. Plugging the 20, 10, 6.7, 5, 4, 2, 1.3, 1, 0.4, 0.2, and 0.1% chance surge elevations gives the associated waves for each frequency. The results of this regression analysis give the required wave-frequency information. Table 3 contains the resulting stage and wave frequency curves for Node 395391, located offshore of the project site (N 40.42437, W 74.05972), and also for the average onshore stage frequency with wave effects included.

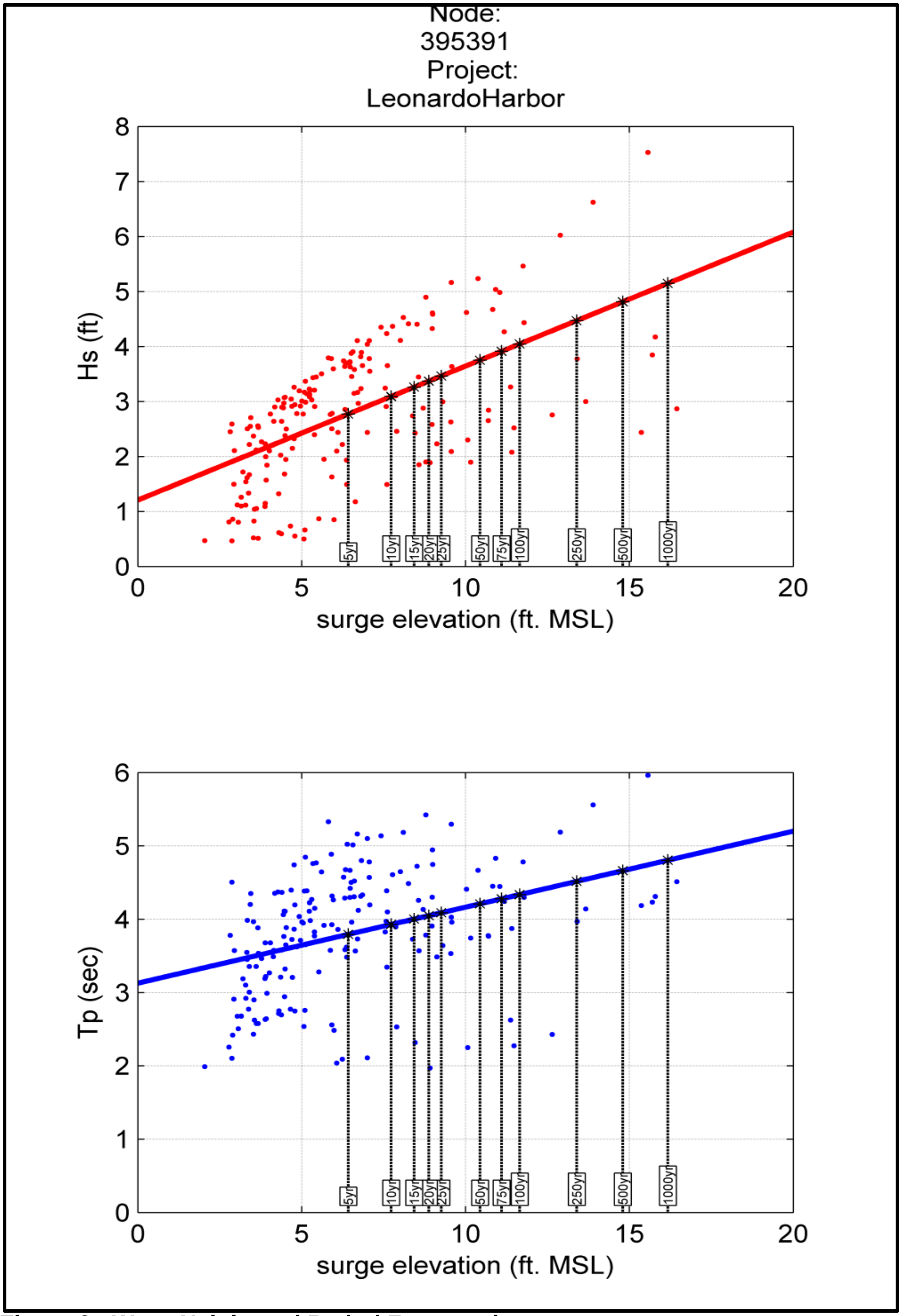


Figure 2: Wave Height and Period Frequencies

Table 2: Tidal Datums

Elevations on Station Datum

Station: 8531680, Sandy Hook, NJ
Status: Accepted (Apr 17 2003)
Units: Feet
T.M.: 75 W
Epoch: 1983-2001
Datum: STND

Datum	Value	Description
MHHW	7.74	Mean Higher-High Water
MHW	7.41	Mean High Water
MTL	5.06	Mean Tide Level
MSL	5.09	Mean Sea Level
DTL	5.13	Mean Diurnal Tide Level
MLW	2.71	Mean Low Water
MLLW	2.51	Mean Lower-Low Water
NAVD88	5.33	North American Vertical Datum of 1988
STND	0.00	Station Datum
GT	5.22	Great Diurnal Range
MN	4.70	Mean Range of Tide
DHQ	0.33	Mean Diurnal High Water Inequality
DLQ	0.19	Mean Diurnal Low Water Inequality
HWI	0.29	Greenwich High Water Interval (in hours)
LWI	6.64	Greenwich Low Water Interval (in hours)
Maximum	12.60	Highest Observed Water Level
Max Date & Time	09/12/1960 13:00	Highest Observed Water Level Date and Time
Minimum	-2.20	Lowest Observed Water Level
Min Date & Time	02/02/1976 16:00	Lowest Observed Water Level Date and Time
HAT	9.11	Highest Astronomical Tide
HAT Date & Time	10/16/1993 12:48	HAT Date and Time
LAT	1.14	Lowest Astronomical Tide
LAT Date & Time	01/21/1996 19:36	LAT Date and Time

Tidal Datum Analysis Periods

01/01/1983 - 12/31/2001

C-2.2.1 FEMA Stage Frequency and Wave-Frequency for Future Conditions (2067) including Sea Level Change (SLC)

Stage and frequency data for future conditions were not available from FEMA. To determine future condition hydraulic boundary conditions, Sea Level Change rates were determined using the methodology outlined in two USACE publications, “Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation” (ETL 1100-2-1), and “Incorporating Sea Level Change in Civil Works Programs” (ER 1110-2-8162).

A website tool (www.corpsclimate.us/ccaceslcurves.cfm) was used to estimate the SLC rates at the Sandy Hook gauge, which is located near the project site. Section C-5 includes a discussion of the effects on the project if sea level change is greater than the historic levels that were used in the design.

Table 3: Stage-Frequency

Annual Chance of Exceedance (%)	FEMA 2014 Offshore Node 395391 Mean Still Water Elevation in ft. NAVD88	2014 Average Onshore Mean Still Water Elevation in ft. NAVD88 including wave effects	Significant Wave Height, Hs, in ft.	Peak Wave Period, Tp, in seconds
20%	6.6	7.9	2.8	3.8
10%	7.9	8.3	3.1	3.9
6.7%	8.6	8.9	3.3	4.0
5%	9.1	9.3	3.4	4.0
4%	9.5	9.7	3.5	4.1
2%	10.6	10.8	3.7	4.2
1.3%	11.3	11.5	3.9	4.3
1%	11.9	12.0	4.0	4.3
0.4%	13.6	13.8	4.6	4.5
0.2%	15.0	15.3	4.8	4.7
0.1%	16.4	16.8	5.1	4.8

The three curves displayed in Figure 3 give rates for the low, intermediate and high estimates of SLC. Table 5 contains the tabular SLC data for the Sandy Hook gauge. Assuming the project begins in 2017 and ends in 2067, the incremental SLC value is +0.7 ft for the low estimate, +1.1 ft for the intermediate estimate, and +2.5 ft for the high estimate. To determine future condition stage-frequency data, the incremental SLC rates are added directly to the base condition curve. For example, if the 20% chance 2017 stage is +6.6 ft NAVD88, the future 2067 low-SLC 1% chance flood stage would become +7.3 ft NAVD88, which is a 0.7 ft increase. Significant wave heights and peak wave periods for future conditions were developed by plugging in the future condition surge values into the same trend lines developed for 2017 conditions. The higher future condition surge elevations produce large waves. Table 4 contains the stage-frequency and wave-frequency data for the offshore node 395391 and for the average onshore frequency for the 2067 condition, for low, intermediate, and high SLC rate.

The methodology described above gives information for Node 395391 (offshore of the project site). The stage-frequency and wave-frequency curves were developed for all structure locations using the same methodology described previously.

Table 4: Future Stage-Frequencies

Year	Sea Level Change Scenario	Annual Chance of Exceedance (%)	FEMA 2014 Offshore Node 395391 Mean Still Water Elevation in ft. NAVD88	2014 Average Onshore 1Mean Still Water Elevation in ft. NAVD88 including wave effects	Significant Wave Height, Hs, in ft.	Peak Wave Period, Tp, in seconds
2067	Low/Historic	20%	7.3	8.5	2.9	3.9
2067	Low/Historic	10%	8.6	9.0	3.2	4.0
2067	Low/Historic	6.7%	9.3	9.6	3.4	4.1
2067	Low/Historic	5%	9.7	10.0	3.5	4.1
2067	Low/Historic	4%	10.1	10.3	3.6	4.2
2067	Low/Historic	2%	11.3	11.5	3.9	4.3
2067	Low/Historic	1.3%	12.0	12.1	4.1	4.3
2067	Low/Historic	1%	12.5	12.7	4.2	4.4
2067	Low/Historic	0.4%	14.2	14.5	4.6	4.6
2067	Low/Historic	0.2%	15.7	16.0	5.0	4.7
2067	Low/Historic	0.1%	17.0	17.4	5.3	4.9
2067	Intermediate	20%	7.7	8.0	3.0	3.9
2067	Intermediate	10%	9.0	9.4	3.4	4.0
2067	Intermediate	6.7%	9.7	10.0	3.5	4.1
2067	Intermediate	5%	10.2	10.4	3.6	4.1
2067	Intermediate	4%	10.6	10.7	3.7	4.2
2067	Intermediate	2%	11.7	11.9	4.0	4.3
2067	Intermediate	1.3%	12.4	12.6	4.2	4.3
2067	Intermediate	1%	13.0	13.1	4.3	4.4
2067	Intermediate	0.4%	14.7	14.9	4.7	4.6
2067	Intermediate	0.2%	16.1	16.4	5.1	4.8
2067	Intermediate	0.1%	17.5	17.8	5.4	4.9
2067	High	20%	9.1	10.5	3.4	4.1
2067	High	10%	10.4	10.8	3.7	4.2
2067	High	6.7%	11.1	11.4	3.9	4.3
2067	High	5%	11.6	11.8	4	4.3
2067	High	4%	12.0	12.1	4.1	4.3
2067	High	2%	13.1	13.3	4.4	4.5
2067	High	1.3%	13.8	14.0	4.5	4.5
2067	High	1%	14.4	14.5	4.7	4.6
2067	High	0.4%	16.1	16.3	5.1	4.8
2067	High	0.2%	17.5	17.8	5.4	4.9
2067	High	0.1%	18.9	19.2	5.8	5.1

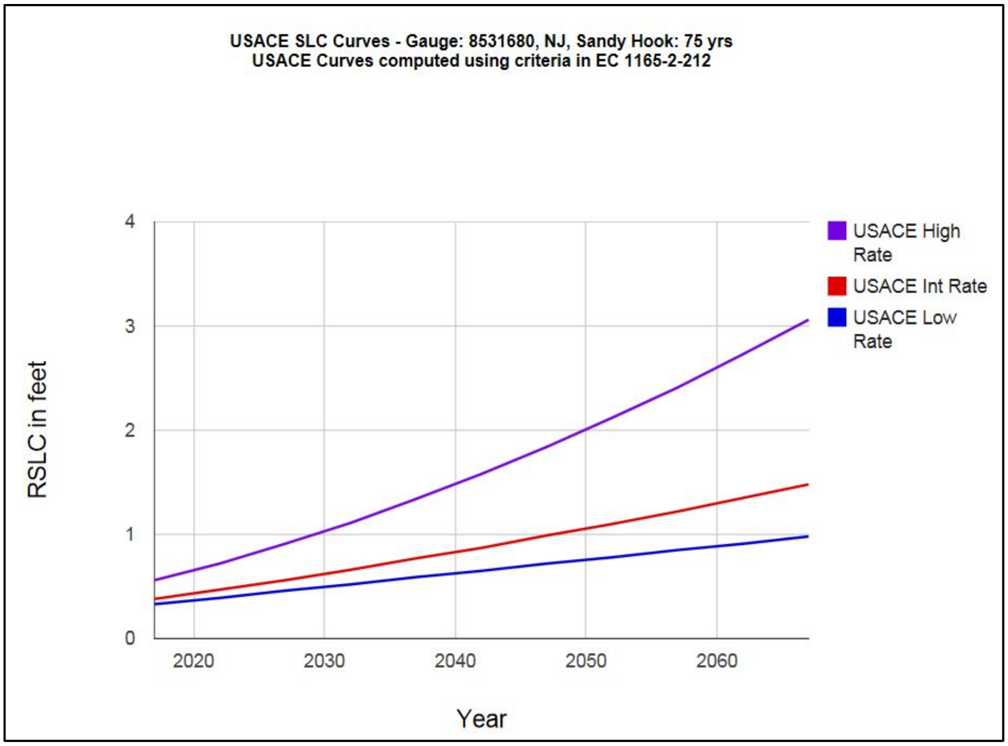


Figure 3: Graph of Three Rates of Sea Level Change

Table 5: Tabular Rates of Sea Level Change

USACE Curves computed using criteria in USACE EC 1165-2-212

Gauge: 8531680, NJ, Sandy Hook: 75 yrs
All values are in feet

Year	USACE Low	USACE Int	USACE High
2017	0.33	0.38	0.56
2022	0.39	0.47	0.72
2027	0.46	0.56	0.91
2032	0.52	0.66	1.11
2037	0.59	0.77	1.34
2042	0.65	0.87	1.58
2047	0.72	0.99	1.84
2052	0.78	1.10	2.12
2057	0.85	1.22	2.41
2062	0.91	1.35	2.73
2067	0.98	1.48	3.06

C-2.3 Storms

Some of the more significant storms affecting the study area are described below.

25 Nov 1950 (Hurricane)

Boats and piers in Leonardo were severely damaged by tide and wave action. Highest water level recorded at Sandy Hook gage was 7.5 ft. above MSL (of1950).

6-7 Nov 1953 (nor'easter)

4,000 ft. of bulkhead located between Leonardo and Atlantic Highlands was destroyed. Approximately 15 residences were inundated between 2 and 4 ft. above the 1st floor, and were structurally damaged by undercutting.

12 Sept 1960 (Hurricane Donna)

The jetties at the marina were damaged, and homes along the shore suffered minor damage due to flooding. Highest water level recorded at Sandy Hook gage was 8.6 ft above MSL (of 1960).

6-8 March 1962 (nor'easter)

Minor damage was reported at Leonardo. Considerable amounts of debris were deposited around residences and marina. Highest water level recorded at Sandy Hook gage was 7.6 ft above MSL (of 1962).

12 March 1984 (nor'easter)

In Leonardo, erosion of the beaches and dune escarpment accompanied street and property flooding near Wagner Creek. Retaining walls were undermined by high water removing sand. Extensive erosion occurred east of the harbor to the harbor light. Highest water level recorded at Sandy Hook gage was +6.0 ft NAVD88.

11-13 Dec 1992 (nor’easter)

During the December 1992 nor’easter, low-lying homes adjacent to the marina suffered significant flood damage. The beach experienced severe erosion damage. The road was undermined and washed out due to erosion damage. Bulkheads and seawalls/revetments were severely damaged or destroyed. Highest water level recorded at Sandy Hook gage was +7.6 ft NAVD88.

12-14 March 1993 (Blizzard)

The Blizzard of March 11-12, 1993 was called the "Storm of the Century". It caused above average flooding for coastal and riverine areas in Monmouth County.

12 March 2010 (nor’easter)

A nor’easter impacted NJ on March 12, 2010. Winds gusted up to 70 mph. The recorded peak stage at Keansburg was +7.01 ft NAVD88. Leonardo experienced flooding, and destruction of the town's 9/11/memorial.

26-28 Aug 2011 (Hurricane Irene)

Hurricane Irene struck Little Egg Harbor in NJ on Aug 28. 200,000 homes and building were damaged, and damages in the state reached $1 billion. Flooding was widespread in Central Jersey. Leonardo experienced fallen trees, flooding, erosion damage to beaches and roads.

29-30 October, 2012 (Hurricane Sandy)

Within Middletown Township (Leonardo is an unincorporated community within Middletown: 322 structures experiences superficial damage (lost tiles, shingles, more severe damage to lighter structures); 98 had minor damage (missing roof segments; destroyed or displaced lighter structures); 8 had major damage (missing roofs, partial collapse of structure walls); and 3 structures were completely destroyed or washed away. The highest water level recorded at the Sandy Hook gage was +11.7 ft NAVD88.

C-2.4 Regional Geology

The study area lies within the Coastal Plain Province, which forms the eastern margin of the State of New Jersey. Its surface has a gentle slope to the southeast, generally not exceeding 5 or 6 ft to the mile. The surface of the plain extends eastward with the same gentle slope beneath the Atlantic Ocean for about 100 miles to the end of the continental shelf, where the depth is approximately 100 fathoms. At this point, the ocean bottom drops abruptly to greater depths. The moderate elevation of the Coastal Plain, which rises to 400 ft in some areas, but is generally lower than 200 ft, has prevented the streams from cutting valleys of any considerable depth. Throughout the greater portion of the plain, the relief is insignificant and the streams flow in open valleys that lie at only slightly lower levels than the broad, flat divides.

The study area, which is contained in Monmouth County, lies in the area that is above the sea level. This sub aerial portion is generally a dissected plain that rises gradually from sea level at the coast to nearly 400 ft in central New Jersey. It then declines to a broad shallow depression less than 100 ft above sea level extending to the Delaware River at Trenton. Some conspicuous features of the sub aerial portion of the plain are the marshes, which border the stream courses and the submerged or drowned valleys, which were formed by erosion when the land was at a higher elevation than at present. During the geologic history, the sea level fluctuated to a large extent. The rise and fall of the water resulted in wide migration of the shoreline across the Coastal Plain. The sub aerial region was especially influenced by these fluctuations during the Cretaceous Period.

The Cretaceous Period resulted in many successive sedimentary formations, each of which was subject to erosion, deposition, submersion, and emergence. Realizing that weathering and its associated agents determined all of New Jersey's geomorphology; this geological period had great influence on the study area. The resulting Cretaceous formations are composed of unconsolidated sand, clay, and greensand marl (glauconitic), which dip 25 ft to 60 ft per mile to the southeast and having a thickness in places of 500 ft to 1,000 ft. The sediments rest on a sloping formation of deep-seated hard rocks. The present surface features were most recently determined during the glacial Pleistocene Period and by subsequent erosion.

The subsurface geology of the Coastal Plain has been determined by study and correlation of well logs and by interpretation of seismic profiles. The Coastal Plain consists of Cretaceous to Recent sediments lapping on the basement material, which is composed of crystalline rock with locally infolded or infaulted Triassic sediments. The basement surface slopes at about 75 ft per mile, reaching a depth of more than 6,000 ft near the coast. The soils overlying the bedrock are of considerable thickness exceeding several hundred ft., and are of the Upper Cretaceous and Tertiary Period. The oldest and therefore the deepest formation, which rests unconformably on the bedrock is the Raritan (Magothy) formation. It consists of dark lignitic sand and clay containing some glauconite at the top overlying light colored sands and clays.

The Mechanville and Woodbury clay formations overlay the Raritan formation discomformably. Both formations are black, glauconitic, micaceous clay, the former being slightly more plastic and firmer than the latter. To the southeast of Waycake Creek (the western boundary of the Keansburg project area), the upper formation, the Englishtown sand, outcrops at the surface along Creek Road, and extends southeastward to Highlands under the recent swamp deposits at Pews Creek (the eastern boundary of the Keansburg project area). It reaches its maximum thickness at the Highlands where some of the beds have been cemented by iron oxide. This material overlays the Woodbury clay formation and it represents a period of emergence. The Englishtown sand consists of a white and yellow quartz sand, slightly micaceous.

With the final uplift of the land and withdrawal of the Cretaceous sea, streams established themselves across the emerging sea bottom. This ushered in the Cenozoic Era. Periods of submergence and emergence were the dominating geological force, but with the exception of a very shallow deposit of sand referred to as the Cape May formation, no other soil material from this era is found in the project area. The Cape May formation is an interglacial formation deposited by streams and overland deposition at the close of the last glacial period. The sea again invaded the area and created valleys, which have been filling with recent swamp material and sediment.

Considering the age of the Cretaceous materials, estimated by geologists to be 120 to 150 million years old and all the intervals of submergence and deposition, and emergence and erosion, one would expect these soils to be very firm on the basis that they have been subjected to relatively high prestresses. However, the clay materials were found to be nominally consolidated and very soft.

C-2.5 Coastal Structure Evaluation

A structure condition survey was conducted in Leonardo in 1957 and is contained in the 1960 Survey Report. A survey of the condition of coastal structures was conducted in 2001, and the results were compared to the 1957 conditions in Table 6.

C-2.6 Existing Beach Parameters

Beach profiles collected September 1999 along Leonardo were assessed for common parameters. One beach profile, collected in the study area between May and July 1957, was also assessed for comparison. Existing beach characteristics are shown in Table 7. Dune elevations in the study area vary between +8.9 ft NAVD88 and +13.9 ft NAVD88, with an average of approximately +10.9 feet NAVD88. The average berm elevation is approximately +3.9 ft NAVD88. The berm width varies from 100 ft (near marina) to 0 ft (near Wagner

Creek). The narrowest berm widths occur in reaches fronted by structures (bulkheads or revetments). Average onshore slope is approximately 1v:10h. A typical offshore slope (determined from 1957 long range profile data) is approximately 1v:228h. The average slope break (between the onshore and offshore slopes) occurs at approximately MLW (-2.7 ft NAVD88). The average structure elevation of the bulkheads and revetments present in the eastern portion of the shoreline is approximately +8.9 ft NAVD88.

Table 6: Existing Coastal Structures

Coastal Structure Evaluation

Structure	Location Easting		Structure	Crest Elevation		Top				Condition	Condition
Type	NAD27	NAD83	Material	Inner End	Outer End	Width	Length	Built	Owner	1957	2001
	(ft.)	(ft.)		(ft. NAVD)	(ft. NAVD)	(ft.)	(ft.)	(year)			
Pier	2,166,650	612,477	Timber/Concrete	N/A	N/A	50-100	11,300	1943-45	U.S. Govt.	Good	N/A
Jetty	2,168,550	614,377	Timber	2	2.5	1	260	pre 1940	State	Good	Good (rebuilt in 1990's)
Jetty	2,168,700	614,527	Timber	4	2.4	1	50	pre 1940	State	Good	Good (rebuilt in 1990's)
Groin Field (4 groins)											
from	2,169,100	614,927	Timber	1.3	0.8	1	55	1942-43	State	Good	Fair
to	2,169,900	615,727	Timber	2.4	2.2	1	145		and Local		
Groin	2,171,200	617,026	Timber	3.6	1.7	1	110	N/A	Private	Good	Poor (Non-functional)
Bulkhead											
from	2,171,400	617,225	Timber	9	N/A	1	400	N/A	Private	Good	N/A
to	2,171,800	617,625		12.1	N/A						
Bulkhead											
from	2,172,050	617,868	Timber	N/A	N/A	1	225	N/A	N/A	N/A	N/A
to	2,172,100	617,918									

C-2.7 Historic Shoreline Change Analysis

The following data was utilized in determining average erosion rates:

1836 MHW (Mean High Water) survey made by U.S. Coast and Geodetic Survey, and digitized from the 1960 Raritan Bay and Sandy Hook Bay Survey Report Plate 7.

1957 survey denoting MHW by use of location of MHW on profile lines, supplemented by use of aerial photography collected during the same year, digitized from the 1960 Raritan Bay and Sandy Hook Bay Survey Report Plate 7.

1988 MHW taken from Township of Middletown Tax Maps, digitized from the Raritan Bay and Sandy Hook Bay Reconnaissance Report Figure 11a.

1999 MHW (elevation +1.9 feet NAVD) taken from the 1999 aerial 1”=100’ photogrammetric mapping of Leonardo, NJ, digital maps. This data was adjusted with profile data.

Table 7: Existing Beach Parameters

PL	date	type	*dune crest el*	*dune crest width*	*dune base el*	*dune foreshore slope*	*berm crest el*		*berm crest width*	*berm foreshore slope*	*berm toe el*	*average offshore slope*
			ft. NAVD	*ft*	*ft. NAVD*	*1V:xH*	*ft. NAVD*		*ft*	*1V:xH*	*ft. NAVD*	*1V:xH*
280	1999	Marsh	n/a	n/a	n/a	n/a	7.2		20	9.3	-3.5	
270	1999	Marsh	n/a	n/a	n/a	n/a	5.9		20	8.7	-3.2	
		avg	n/a	n/a	n/a	n/a	6.55		20	9	-3.35	500
260	1999	Dune	9.2	3	5.4	8.7	5.7		89	13.9	-2.4	
250	1999	Dune	11.9	3	4.5	6.9	4.5		39	9.4	-3.2	
240	1999	Dune	13.1	3	5.4	3.6	6.9		20	9.9	-3.7	
		avg	11.4	3.0	5.1	6.4	5.7		49.3	11.1	-3.1	450
24	**1957**	**dune**	9.8	0	n/a	11.5	n/a		n/a	n/a	-2.1	550
230	1999	Bulkhead Flank	6.9	35	3.9	7	4.2	*	30	12.2	-2.7	
220	1999	Bulkhead	n/a	n/a	n/a	n/a	4	**	0	11	-3.2	
210	1999	Bulkhead	n/a	n/a	n/a	n/a	5.1	**	0	15.8	-3.2	
200	1999	Bulkhead	n/a	n/a	n/a	n/a	5.2	**	0	13.2	-3.4	
190	1999	Bulkhead Flank	6.2	10	4.5	8.2	4.5	*	26	10.8	-4.1	
		avg					4.8		0.0	13.3	-3.3	450

Notes: n/a - not applicable
* - berm fronting dune
** - berm fronting bulkhead

C-2.8 Selection of Long-Term Erosion Rates

The period between 1957 and 1999 was selected as the typical period to represent long-term shoreline erosion rates in ft/yr, and the similar period of 1954-1999 was selected to represent the long-term volumetric changes in cy/yr. These periods have the most “typical” shoreline and volumetric change trends, and the minimum uncertainty of all the data sets. The marsh reach has 0.5 ft/yr of accretion, the dune reach has 3.5 ft/yr of erosion, and the bulkhead reach has 2.7 ft/yr of erosion. The bulkhead erosion rate is assumed to increase approximately 15% (to 3.1 ft/yr of erosion) in the future due to sediment starvation increasing the wave induced scour at the toe of the bulkheads.

C-3. Without-Project Existing and Future Conditions

Damages fall into two categories for this study: inundation damages which occur in all the reaches, and wave-related damages which are more localized and site-specific. Inundation damages are discussed in the Economic Appendix. Wave-related damages are discussed herein. The Marsh Reach is sufficiently set back such that it is not subject to direct wave action and wave-related damages. The Dune/Beach Reach is backed by Beach Ave, utility poles and buried utility lines. The structures themselves, located behind Beach Ave are sufficiently set back from the road such that they are not subject to wave-related damages. However, the road itself and utilities are subject to storm- induced erosion and undermining. The Bulkhead Reach is subject to structure failure from direct wave breaking on the structures, storm wave-induced scour of the grade fronting the bulkhead, and wave-induced scour of the grade on the landward side of the bulkhead. However, the structures are sufficiently setback from the bulkhead line and are at an elevation high enough to not be impacted by wave damage themselves. Only the local repair cost of the bulkhead repair was assumed to be significant. Further details on the wave-related damage calculations for the Dune/Beach and Bulkhead Reaches follow. Structural alternatives were evaluated using these values.

C-3.1 Without-Project Existing and Future Conditions for the Dune/Beach Reach

The EDUNE Dune Erosion Model developed by Dr. David Kriebel in 1989 was used to predict the post-storm profile condition of the Dune/Beach Reach typical profile for the 50, 10, 4, 2, 1, and 0.2% chance exceedance storm events. Other inputs included stage, root mean square breaking wave height, average sand grain size, and average storm duration. Results includes maximum dune erosion distance measured at the MHW elevation (which includes a variability factor of 1.5 to account for variations in model results and profile conditions) in feet, maximum dune elevation reduction in feet, post-storm dune elevation, and maximum erosion volume in cubic ft per linear foot of beach.

The MHW location on the 1999 topographic mapping (1”=200’) was adjusted to account for long term erosion between 1999 and 2007 using the without-project erosion rate of 3.5 ft/yr (at 8 years for a total of 28 ft landward adjustment). The maximum dune erosion distance was superimposed on this 2007 MHW location to determine where the erosion intercepted the roadway. The road was assumed to be undermined and needing replacement when the erosion limit exceeded 5 ft landward of the landward edge of pavement.

Without project impacts to roadways due to a combination of storm-induced and long-term erosion was evaluated for the existing conditions (year T=0), and at T=10, 20, 30, 40, and 50 years into the future (from the base year of 2007, assuming long-term erosion rate of 3.5 ft/yr acting upon the 1999 shoreline), utilizing the following assumptions.

A. It was assumed that local authorities would replace roadway surfaces when erosion undermined road 5 ft. or more. At the time of road replacement, the local authorities were assumed to place sand fill 10 ft wide, fronting the roadway, overlain by riprap in order to provide an approximate 6.7 to 10% chance exceedance level of coastal storm risk management to the new road surface. The sand fill would extend from elevation 9.4 ft NAVD88 to the toe at –3.3 ft

NAVD88 (4.63 cy/ft). The riprap is 1 ft thick, with a 0.5-foot thick bedding layer underneath and extends from the edge of pavement at elevation +9.4 ft NAVD88 to elevation +2.9 ft NAVD88 (12.4 sy/ft).

B. It was assumed that the initial road repair/sandfill/riprap would be constructed for storm intrusion (causing greater than 5 feet of undermining of road surface) occurring in the existing (year T=0) condition.

C. Road repair (Apr 02 PL) was assumed to cost $35/sy. Sand fill (Apr 02 PL) was assumed to be obtained from an upland source at a cost $13/cy. Riprap placement (Apr 02 PL) was assumed to cost $40/sy. Mobilization and demobilization costs (Apr 02 PL) of $20,000 per operation were assumed.

D. In future conditions, storm occurrence on the previously remedially protected areas would require repair of the protection at the following percentages: 33% for a 4% chance exceedance event, 45% for a 2% chance event, 65% for a 1% chance event, and 95% for a 0.2% chance event for previously placed riprap; and 65% for a 1% chance event, and 95% for a 0.2% chance event for previously placed road repair and sand fill.

E. Any storm intrusion (5-foot undermining of road) in future years into areas NOT previously protected (i.e., areas adjacent to repaired areas) would require a full placement section (replace roadways surface fronting with 10-foot wide sand fill overlain by riprap).

F. Three utility poles are located seaward of the road, in the present dunes. These poles were assumed to fail when 3 feet or greater of vertical erosion occurs. Utilizing the post- eroded profiles from EDUNE, it was found that 3 feet of vertical erosion occurred at the location of the poles for a 2, 1, and 0.2% chance exceedance events in years T=0 and T=10, for 10, 4, 2, 1, and 0.2% chance events for years T=20 and T=30, and for 50, 10, 4, 2, 1, and 0.2% chance events for years T=40 and T=50. The poles were assumed to be relocated at the time of repair to a more protected location. Repair costs of $5,000/pole were assumed.

G. Gas and sewer lines are located under the roadway, approximately along the centerline (10 feet from seaward edge of pavement). When storm-induced and long-term erosion undermine the roadway by a minimum of 10 feet, the gas and sewer lines were assumed to need repair. A combined repair cost (Apr 02 PL) of $100/ft of lines was assumed.

H. Storm-sewer lines are also located in the bulkhead reach to the east. Several outfalls carry water to bay. During the 1992 nor’easter, these outfalls and the adjacent street terminuses supporting them were severely damaged. Local authorities authorized repair to a slightly higher level of coastal storm risk management. The cost of the repairs performed in 1993 was approximately

$213,000. For this study, it was assumed that these repairs have a life expectancy of approximately 20 years. Therefore, it was assumed that at the end of every 20 years, approx. $100,000 (Apr 02 PL) would be spent to repair damage from a 2% chance event, approx. $125,000 (Apr 02 PL) would be spent to repair damage from a 1 percent flood, and approx. $150,000 (Apr 02 PL) would be spent to repair damage from a 0.2 percent flood. These repairs were assumed to occur in years T=10 (which is approx. 20 years after local repair in 1993), year T=30, and year T=50.

Pertaining to uncertainty, the range of storm-induced erosion for a given return period storm is minimal as it pertains to variations in annualized cost. Therefore, the mean values utilized are representative of erosion damages to the road system behind the dune.

The end result of this analysis was an estimation of average annual damages of $181,000 (escalated to an Oct 2008 Price Level), for purpose of calculating and comparing the benefit-to-cost ratios (BCR) for each of the proposed alternatives.

C-3.2 Without-Project Existing and Future Conditions for the Bulkhead Reach

Bulkhead failure was predicted from scour. The average horizontal erosion rate of 3 ft/yr corresponds to average vertical erosion at the toe of the walls of approx. 0.5 ft/yr. Assuming 10 ft embedment depth of the walls, 10 years of long-term erosion coupled with a 6.7% chance event (having a corresponding wave height at the toe of the wall of 5.5 ft. causing an equivalent scour depth) would cause failure of the wall. The estimated cost of wall replacement (Apr 02 PL) is approximately $600/linear foot, for 1,800 lf of wall, or $1,080,000 (Apr 02 PL) to be spent every 10 years. This corresponds to an average annual wall replacement cost of $78,000 (50 yr period, 7-1/8% interest Apr 02 PL). When escalated to Oct 08 PL, with 4-3/8% interest, the average annual bulkhead repair cost is $120,000. Wave overtopping was estimated using equations from Smith et al 1994. The results show that a 4% annual chance exceedance event creates sufficient overtopping to scour out the material providing support behind the structure. This was assumed to correspond to $43,000 of damages (Apr 02 PL) annually (or $1,080,000 x 0.04). The total annual damages (Apr 02 PL) for the bulkhead reach are, therefore, $121,000. The total annual damages escalated to Oct 08 PL for the bulkhead reach are $178,000.

C-3.3 Price Level for Without-Project Existing and Future Conditions

Through a sensitivity analysis, it was determined that the update to without-project existing and future conditions would not affect the results of the plan formulation. Consequently, costs and benefits are presented in 2008 Price Level to reflect when these numbers were derived.

C-4. Development of Alternatives

A total of eleven plans were considered. Of these, six were structural alternatives and five were nonstructural alternatives. Descriptions and layouts of the alternatives can be found in the main report. The initial development of the eleven alternatives utilized USACE stage-frequency data from 1998 (which consisted of the 1% annual chance of exceedance still water elevation of +10.7 ft. NAVD added to 50% of the 1.4 ft of wave setup, or +11.4 ft NAVD) . In 2014, FEMA provided draft stage-frequency curves, which have been adopted as the stage-frequency data for Leonardo as of 2014. The initial comparison between the eleven alternatives was completed prior to 2014. The assumption is made: because all eleven alternatives were developed and compared using the same data (1998 stage data), the results of the comparison would come out the same if the 2014 data was used for all eleven.

C-4.1 Structural alternatives

Alternative S1 – Seawall with gate across marina

Alternative S2 – Beach Fill with gate across marina

Alternative S3 – Combination Beach Fill and Seawall with gate across marina

Alternative S4 – Combination Beach Fill and Seawall with gate across marina, protection provided only west of Brevent Avenue

Alternative S5 – Limited structural plan with no gate across marina

Alternative S6 – Road Raising

The alternatives were compared to the planning objectives to determine which features should be considered for more detailed analysis. Table 8 shows the major advantages and disadvantages of each of the structural alternatives. Table 10 shows the resulting total costs for all the alternatives. Locals have expressed lack of support for all the presented structural alternatives. Therefore, structural alternatives were dropped from further consideration.

Table 8: Advantages and Disadvantages of Structural Plans

Major Advantages and Disadvantages of the Structural Alternatives			
Alt. No.	Alternative Description	Major Advantages	Major Disadvantages
S1	Seawall	Access routes remain open during flood events	Increased maintenance of gate across marina
		Does not require beach renourishment in future	Potential for impact on bay views
S2	Beach fill	Access routes remain open during flood events	Increased maintenance of gate across marina
		Provides new beach with additional recreation opportunities	Potential for impact on bay views
S3	Beach fill & Seawall	Access routes remain open during flood events	Increased maintenance of gate across marina
		Provides new beach with additional recreation opportunities	Potential for impact on bay views
S4	Beach fill & Seawall with protection west of Brevent Ave	Access routes remain open during flood events	Leaves areas east of Brevent Ave. exposed
		Provides new beach with additional recreation opportunities	Increased maintenance of gate across marina
			Potential for impact on bay views
			Provides only limited flood protection
S5	Limited structural plan with no gate	Increased beach area may provide recreation opportunities	Leaves areas west of marina and east of Brevent Ave. exposed
			Potential for impact on bay views
S6	Limited Road Raising Plan	No significant environmental impacts identified	Limited potential for storm damage protection
		No view impacts	

C-4.2 Nonstructural alternatives

Alternative N1 –Nonstructural (structures in the 1998 20 percent floodplain (+6.9 ft NAVD88) to the level of a 1 percent flood (+10.7 ft NAVD) plus 0.7 ft of sea level rise plus 0.7 ft of wave setup plus the FEMA recommended freeboard amounts (total +13.1 ft NAVD))

Alternative N2 – Nonstructural (structures in the 1998 4 percent floodplain (+9.4 ft NAVD88) to the level of a 1 percent flood (+10.7 ft NAVD) plus 0.7 ft of sea level rise plus 0.7 ft of wave setup plus the FEMA recommended freeboard amounts (total +13.1 ft NAVD))

Alternative N3 – Nonstructural (structures in the 1998 1 percent floodplain (+11.4 ft NAVD88) to the level of a 1 percent flood (+10.7 ft NAVD) plus 0.7 ft of sea level rise plus 0.7 ft of wave setup plus the FEMA recommended freeboard amounts (total +13.1 ft NAVD))

Alternative N4 – Nonstructural (structures in the 1998 20 percent floodplain (+6.9 ft NAVD88) to the level of a 10 percent flood (+7.7 ft NAVD) plus 0.7 ft of sea level rise plus 0.5 ft of wave setup plus the FEMA recommended freeboard amounts (total +9.9 ft NAVD))

Alternative N5 – Nonstructural (structures with a main floor at or below +9.4 ft

NAVD88 and with a ground elevation below +7.9 ft NAVD88 to the level of a 1 percent flood (+10.7 ft NAVD) plus 0.7 ft of sea level rise plus 0.7 ft of wave setup plus the FEMA recommended freeboard amounts (total +13.1 ft NAVD)). The main floor criteria attempts to target structures with the most frequent and severe damages with water surfaces exceeding the main floors, and not simply basement or crawlspace flooding.)

Table 9: Advantages and Disadvantages of Nonstructural Plans

Alt. No.	Alternative Description	Major Advantages	Major Disadvantages
N1	Non-structural, 20% Floodplain	No hard structure to impede view of bay	Access routes would not remain open during flood event
		No additional maintenance requirements	Temporary inconvenience to structure owners during construction phase
			No coastal storm risk management outside of 20% floodplain
N2	Non-structural, 4% Floodplain	No hard structure to impede view of bay	Access routes would not remain open during flood event
		No additional maintenance requirements	Temporary inconvenience to structure owners during construction phase
			No coastal storm risk management outside of 4% floodplain
N3	Non-structural, 1% Floodplain	No hard structure to impede view of bay	Access routes would not remain open during flood event
		No additional maintenance requirements	Temporary inconvenience to structure owners during construction phase
N4	Non-structural, 20% floodplain to a 10% flood level	No hard structure to impede view of bay	Access routes would not remain open during flood event
		No additional maintenance requirements	Temporary inconvenience to structure owners during construction phase
			Provides only limited coastal storm risk management and no coastal storm risk management outside of 20% floodplain
			May violate floodplain management rules
N5	Non-structural Main Floor ≤ +9.4 ft. NAVD88	No hard structure to impede view of bay	Access routes would not remain open during flood event
		No additional maintenance requirements	Temporary inconvenience to structure owners during construction phase
			Provides no coastal storm risk management for houses on grades > +7.9 ft. NAVD88 or houses with mainfloor elevations > +9.4 ft. NAVD88

The alternatives were compared to the planning objectives to determine which features should be considered for more detailed analysis. Table 9 shows the major advantages and disadvantages of each of the nonstructural alternatives. Table 10 shows the resulting total costs for the alternatives. The results of the preliminary economic analysis indicate a marginal economic justification for Alternatives N1 and N5. However, economic justification may be improved once costs are refined during subsequent analysis. Furthermore, refinement of the scope of the nonstructural plan in terms of the number of structures included in a plan may also result in a favorable BCR.

Limited nonstructural alternatives, such as N1 and N5, are recommended for more detailed development. Figure 4 and Figure 5 show the structures in the project area for Alternatives N1 and N5, respectively.

Table 10: Summary of Alternatives Costs

Alternative	Description	Total Cost (Oct. 08 PL)	Annual Cost (Oct. 08 PL)
S1	Seawall Plan	$30,097,000	$1,695,000
S2	Beach Fill Plan	$31,508,000	$2,009,000
S3	Combination Beach Fill & Seawall plan	$31,191,000	$1,795,000
S4	Combination Plan West of Brevent Avenue	$23,554,000	$1,389,000
S5	Limited Structural Plan	$14,334,000	$817,000
S6	Limited Road Raising Plan	$499,000	$28,000
N1	20% Floodplain Non-structural (NS)1*	$2,379,000	$118,000
N2	4% Floodplain NS1*	$11,026,000	$547,000
N3	1% Floodplain NS1*	$16,202,000	$803,000
N4	20% Floodplain NS2*	$1,570,000	$78,000
N5	Main Floor ≤ +9.4 ft NAVD88 NS1*	$2,772,000	$137,000
*(1) Coastal storm risk management to the level of a 1 percent flood			
*(2) Coastal storm risk management to the level of a 10 percent flood			

C-5. Tentatively Selected Plan

Design and costs of nonstructural alternatives N1 and N5 were further refined during the feasibility phase of design, based on the 2014 FEMA stage-frequency data. Alternative N1 selected all structures having a ground elevation lower than +6.9 ft NAVD88 and brings their main floors to the 1 percent flood water surface elevation plus 0.7 ft of historic sea level change plus estimated wave crest effects plus one foot of freeboard for structures in the AE zone or 3 feet of freeboard for structures in the VE zone. Alternative N5 selected all structures with main floors at or lower than +9.4 ft NAVD88 and ground elevations lower than +7.9 ft NAVD88 and brings their main floors to the 1 percent flood water surface elevation plus 0.7 ft of historic sea level change plus estimated wave crest effects plus one foot of freeboard for structures in the AE zone or 3 feet of freeboard for structures in the VE zone. The physical impacts on the elevated structures, should greater than historic sea level change occur, would be more frequent main floor flooding and miscellaneous flooding damages.

The preliminary windshield structure inventory resulted in 61 structures being identified as potential candidates for nonstructural measures. A more detailed structure inventory of these 61 structures collected the following information:

- ground elevation
- main floor elevation
- area of building footprint
- building type
- building style
- number of stories
- foundation type
- foundation condition
- siding material
- condition

C-5.1 Evaluation of Structures

Structures in the project area were analyzed for eligibility for a flood protection measure. The nonstructural alternatives, N1 and N5, were evaluated with the use of an algorithm developed for another nonstructural coastal storm risk management project in the New York District. The evaluation process is documented here in detail, including the recommendations for nonstructural treatment for the eligible structures.

C-5.1.1 Inventory of Structures

The inventory included the following information; the structure ID number, Residential or Commercial, Usage Code, Number of Stories, Wood or Masonry Exterior, Foundation Type, Basement, First Floor Area, Total Size, Ground Elevation, and Main Floor Elevation.

The structures were inspected to confirm the information obtained through previous structure surveys. The GIS locations were used for identification of the structures in reference to the coastal floodplain limits. This information was utilized to determine reference points for determining the various hurricane flood frequencies that would be used to evaluate each structure.

C-5.1.2 Sorting of Structures

The first step for preparing the structure list for the evaluation was to sort the structures in the following order: Residential or Commercial, Foundation Type, and Usage Code. For commercial structures, another sorting level was the type of exterior, wood or masonry. The Usage Codes are shown in Table 11.

C-5.1.3 Criteria for inclusion to plan N1 and N5

The descriptions of Alternatives N1 and N5 provide the criteria for including structures in each alternative, which are as follows:

N1: All structures within the 20% floodplain, as determined by the ground elevation.

N5: All structures with a main floor elevation at or below +9.4 ft NAVD88 and with a ground elevation below +7.9 ft NAVD88.

Accordingly, the inventory of structures was copied into separate spreadsheets for the computations and evaluation of each alternative. The structures that did not qualify for each alternative were simple removed from the tables. Thirty-seven (37) structures remained to be considered for Alternative N1, and 25 structures remained for Alternative N5.

Table 11: Structure Inventory Codes

STRUCTURE INVENTORY CODES					
RESIDENTIAL	**COMMERCIAL**	**INDUSTRIAL**	**UTILITY**	**MUNICIPAL**	
TYPE = RES	**TYPE = COM**	**TYPE = IND**	**TYPE = UTL**	**TYPE = MUN**	
USAGE CODES	**USAGE CODES**	**USAGE CODES**	**USAGE CODES**	**USAGE CODES**	
1. Colonial	21. Art Gallery	71. Food and	101. Sewage Treatment	201. Fire House	
2. Cape Cod	22. Auto Sales	Associated Product	102. Pump Station	202. Storage Garage	
3. Ranch	23. Auto Service	72. Extraction	103. Gas Substation	203. Municipal Building	
4. Split Level	24. Bank	73. Textiles and	104. Water Treatment	204. Municipal Complex	
5. BiLevel	25. Bar	Apparel	105. Wells	205. Police Station	
6. Raised Ranch	26. Bath House	74. Lumber & Wood	106. Electric Substation	206. Schools	
7. Bungalow	27. Church	75. Furniture and	107. Miscellaneous	207. Rescue Squad	
8. Custom	28. Clothing Store	Fixtures		208. Library	
9. Mobile Home	29. Department Store	76. Paper Products		209. Post Office	
10. 2-Family	30. Diner	77. Printing and		210. General Storage	
11. Duplex	31. Drug Store	Publishing			
12. Multi-Family	32. Dry Cleaning	78. Chemicals			
13. Garden Apt.	33. Food Store	79. Fuel Storage			
14. High-Rise	34. Funeral Home	80. Glass, Clay	**CONSTRUCTION**		**BUILD QUALITY**
15. Town House	35. Hair Salon	and Concrete	M Masonry		L Low
	36. Hardware	81. Metal Working	W Wood (non-masonry)		M Medium
	37. Home Furnishings	82. Electrical			H High
	38. Hospital	83. Transportation			
	39. Indoor Sports	Equipment	**BASEMENTS**		**CONDITION**
	40. Jewellers	84. Warehouse	0 No Basement/Slab On-Grade		(at time of survey)
	41. Liquors	85. Building	1 Full Subgrade Basement		N New
	42. Marina	Contractor	2 Partial Subgrade Basement		E Excellent
	43. Medical Office	86. Light Industry	3 Crawl Space/Raised Foundation/Piers		G Good
	44. Office	87. Medium Industry	4 Piles		A Average
	45. Office Warehouse	88. Heavy Industry			F Fair
	46. Outdoor Sports				P Poor
	47. Restaurant				D Delapidated
	48. Rooming House				
	49. Small Retail				
	50. Theaters				
	51. Vacant*	* Can be used for all types except residential			
	52. Farm				

C-5.1.4 Excel Formulas created from Flow Charts

The next step in the structure evaluation was to create formulas in Microsoft Excel to perform the algorithm that was outlined in flow charts used for similar nonstructural projects. Figure 6 And Figure 7 contain the flow charts for residential structures and Figure 8 and Figure 9 contain the flow charts for non-residential structures. The formulas were based on the decision points in the flow charts and physical characteristics of each structure. The end result from the Excel formulas provides the recommended nonstructural treatment for each structure for each alternative. Cost estimates were then developed from the required treatments.

C-5.1.5 Revisions to Structure List

In some cases, structures were removed from consideration for either or both alternative due to previous flood protection treatment, or if the structure has been removed or is too poor of a condition to receive a floodproofing treatment. The structures that were removed for each alternative are listed below, with the reason for removal.

C-5.1.5.1 Structures Removed from Alternative N1

163, 184, 243, 256, 344: *Condition too poor to raise structure, or no structure exists.*

164, 187, 312A: *Algorithm result not practical or cost effective.*

181, 186, 193, 194, 241, 258, 312: *Structure already raised, or planned to be raised.*

C-5.1.5.2 Structures Removed from Alternative N5

143, 163, 243, 256, 344: *Condition too poor to raise structure, or no structure exists.*

164, 312A: *Algorithm result not practical or cost effective.*

181, 193: *Structure already raised, or planned to be raised.*

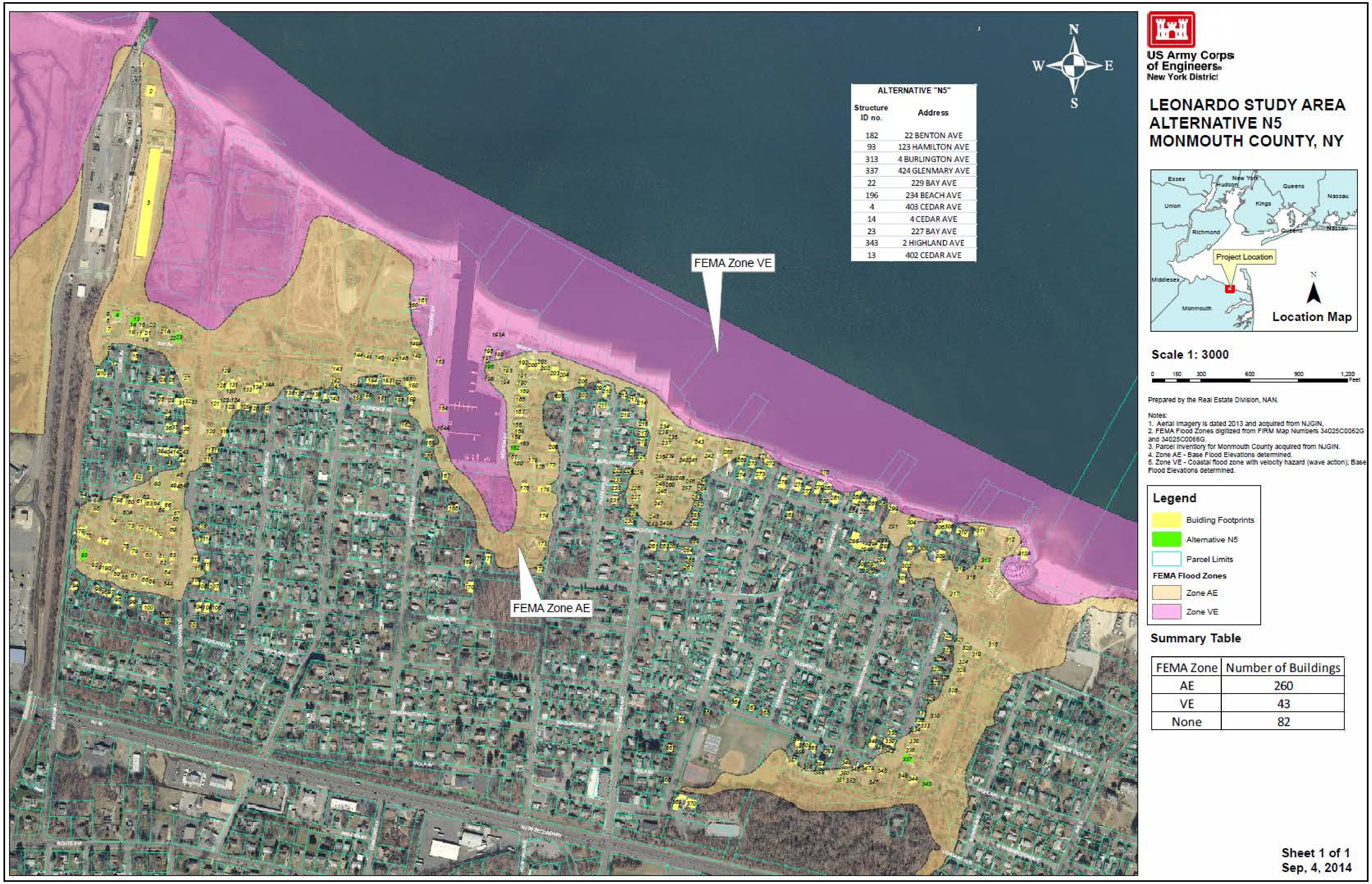
The final results for this nonstructural evaluation are presented in Table 12, which is a list of structures that are included in the project, along with the height of the raise required to meet the necessary protection level. A total of twenty-five (25) structures are eligible for either Alternative N1 or N5 (or both).

Table 12: Structure List and Results for TSP

Structure ID no.	Foundation Type	# of Stories	First Floor Area (sq ft)	FEMA Zone	FEMA Base Flood Elevation (+ft NAVD88)	Ground Elevation (+ft NAVD88)	First Floor Elevation (+ft NAVD88)	Increase to Structure Height (ft)	1% Flood Still Water Elevation with Historic Sea Level Change (+ft NAVD88)	Wave Component (ft)	Freeboard (ft)	First Floor Elevation of Elevated Structure** (+ft NAVD88)	Included in Pre-Hurricane Sandy Plan
4	Crawl Space	1	1,500	AE	13	6.9	8.9	6.9	12.7	2.2	1	15.8	Yes
13	Crawl Space	2.5	1,200*	AE	13	6.8	6.8	9.0	12.7	2.2	1	15.8	No
14	Crawl Space	1	600	AE	12	8.9	8.9	6.9	12.7	2.2	1	15.8	No
22	Crawl Space	2	600	AE	13	5.4	8.3	7.5	12.7	2.2	1	15.8	Yes
23	Crawl Space	1	600	AE	13	4.9	8.3	8.1	12.7	2.8	1	16.4	Yes
27	Slab on Grade	2	1,125	AE	13	5.9	11.3	4.5	12.7	2.2	1	15.8	No
38	Crawl Space	1	1,200	AE	12	6.9	12.7	3.1	12.7	2.1	1	15.8	No
93	Slab on Grade	1	1,500	AE	12	8.4	8.9	6.3	12.7	1.6	1	15.2	No
149A	Crawl Space	2	1,800	AE	13	6.9	9.9	6.5	12.7	2.8	1	16.4	No
161***	Subgrade Basement	1.5	1,600	VE	14	6.9	10.3	8.1	12.7	2.8	3	18.4	No
179	Slab on Grade	2	900	AE	12	6.4	10.4	4.8	12.7	1.6	1	15.2	No
182	Slab on Grade	2.5	1,600	VE	13	4.9	6.6	10.6	12.7	1.6	3	17.2	Yes
185	Slab on Grade	2	900	AE	13	5.4	10.1	5.1	12.7	1.6	1	15.2	Yes
188	Subgrade Basement	1	1,000	AE	13	4.9	10.6	4.7	12.7	1.6	1	15.2	No
189	Subgrade Basement	1	2,025	AE	13	4.9	10.4	4.9	12.7	1.6	1	15.2	No
190	Subgrade Basement	1.5	800	AE	13	4.9	9.6	5.6	12.7	1.6	1	15.2	No
191	Subgrade Basement	1.5	1,000	AE	13	5.9	9.8	6.0	12.7	2.2	1	15.8	Yes
192	Subgrade Basement	1	1,800	VE	13	5.9	12.3	5.5	12.7	2.2	3	17.8	No
196	Crawl Space	2	1,200	VE	13	4.9	9.2	8.6	12.7	2.1	3	17.8	No
268	Crawl Space	1	2,500	VE	12	6.9	10.6	7.2	12.7	2.1	3	17.8	No
313	Subgrade Basement	1.5	750	AE	13	6.9	8.9	7.3	12.5	2.7	1	16.2	No
319	Subgrade Basement	1	1,250	AE	12	6.9	14.0	1.8	12.7	2.1	1	15.8	No
337	Subgrade Basement	1	2,400	AE	11	8.9	8.9	6.3	12.7	1.6	1	15.3	No
343	Crawl Space	1	1,200	AE	11	4.9	6.9	8.8	12.7	2.1	1	15.8	Yes
345	Subgrade Basement	2	900	AE	11	6.9	13.6	2.2	12.7	2.1	1	15.8	No

Notes:
**** - First Floor Area estimated (not recorded by survey)***
***** - This is equivalent to the one percent flood design elevation, which includes the one percent flood still water elevation with historic sea level change, plus wave component and freeboard.***
****** - All structures in the table are residential except #161, which is commercial***

Figure 4: Alternative N1



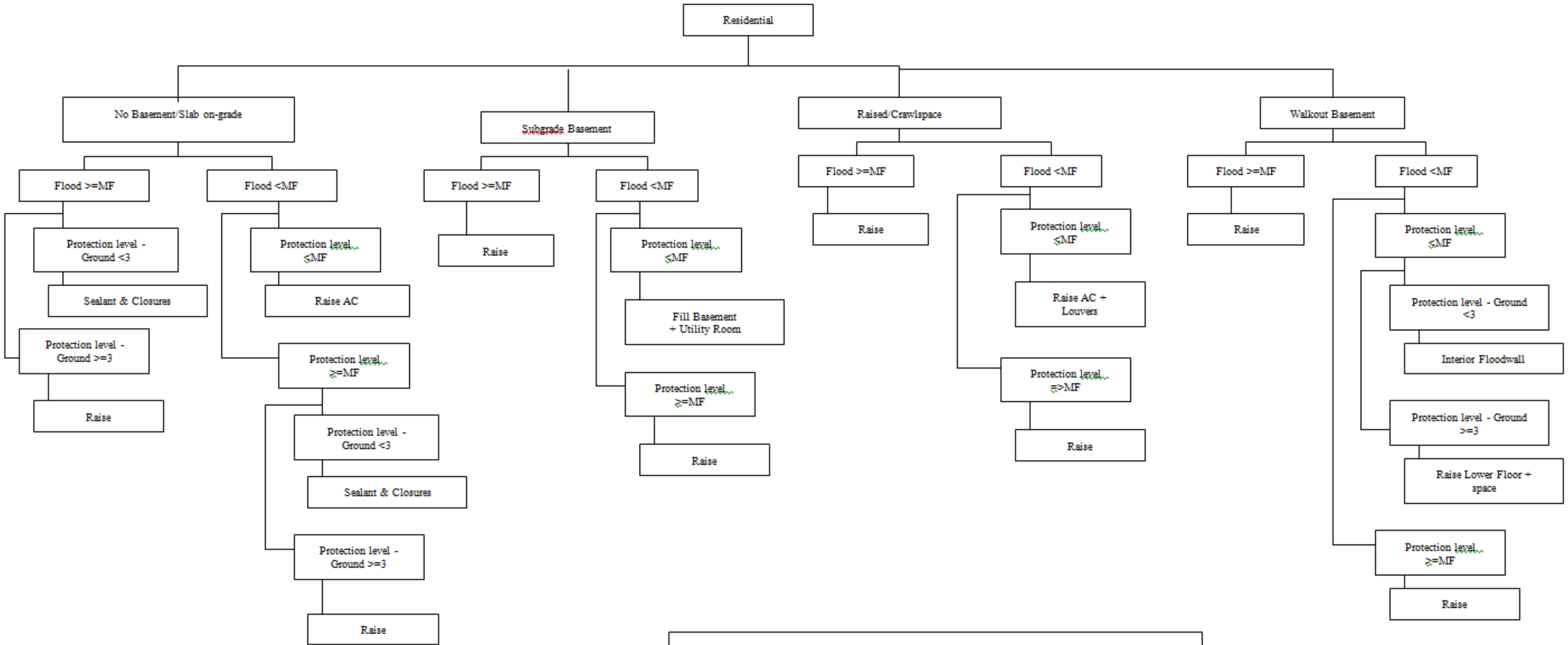
ALTERNATIVE "N5"

Structure ID no.	Address
182	22 BENTON AVE
93	123 HAMILTON AVE
313	4 BURLINGTON AVE
337	424 GLENMARY AVE
22	229 BAY AVE
196	234 BEACH AVE
4	403 CEDAR AVE
14	4 CEDAR AVE
23	227 BAY AVE
343	2 HIGHLAND AVE
13	402 CEDAR AVE

Summary Table

FEMA Zone	Number of Buildings
AE	260
VE	43
None	82

Figure 5: Alternative N5



Typical Structures
(Structures are defined by foundation type)

1. RAISED (crawl space)
2. SLAB-ON-GRADE
3. BASEMENT (Subgrade (Traditional) and Walkout)
4. BI-LEVEL/RAISED Ranch
5. SPLIT-LEVEL

Assumptions for TYPICAL STRUCTURES

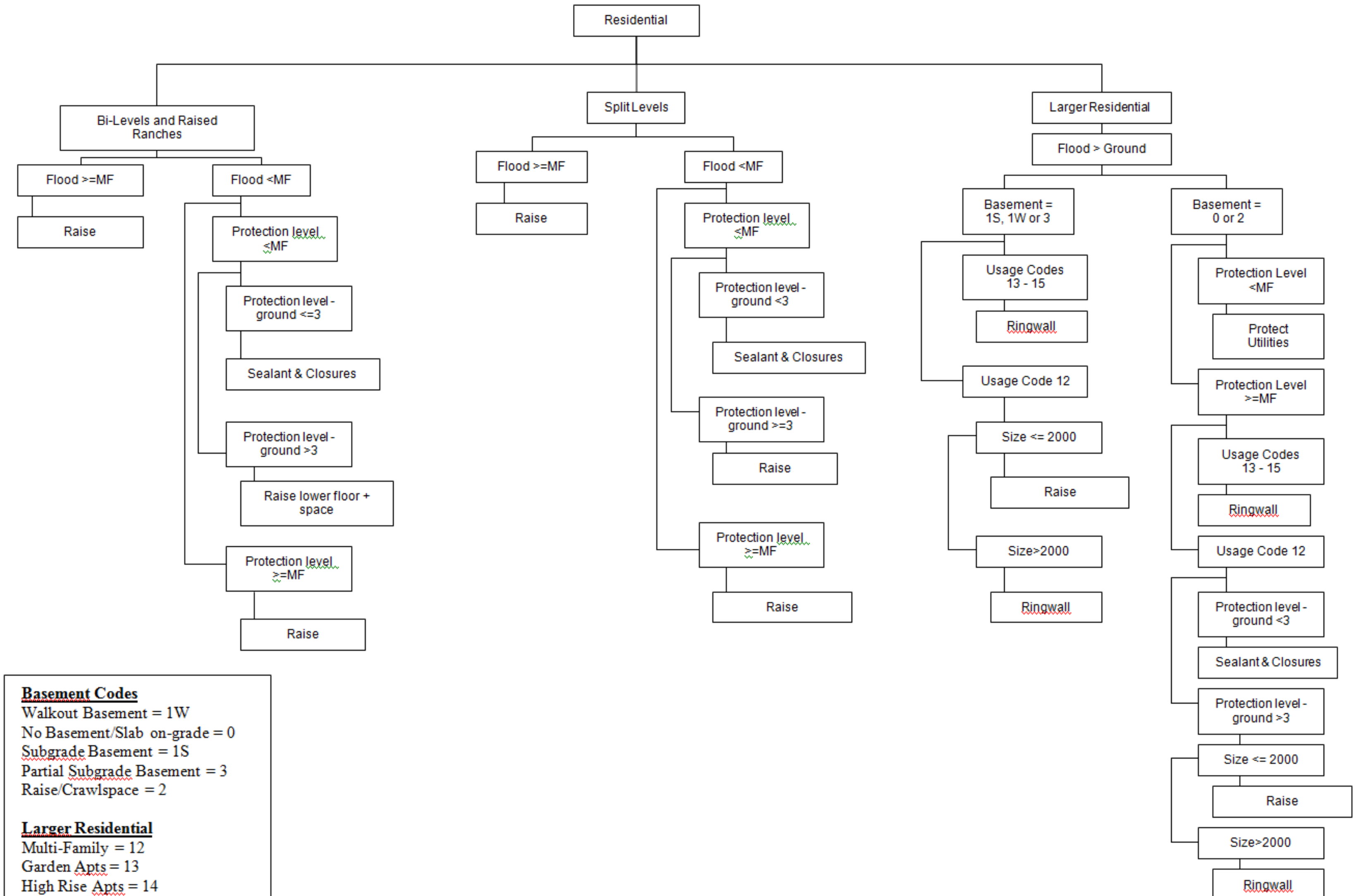
1. (a) Raised(Crawl Space): No utilities are located in the crawl space.
1. (b) Raised(Crawl Space): Wet Flood Proofing includes exterior utilities only.
2. (a) Slab: Wet Flood Proofing is possible when flood depth is below the MF (shallow flooding). Typically exterior utilities only (e.g. AC).
3. Basement: All basements are unfinished.
4. (a) Bi-level/Raised Ranch: The lower floor is a minimum of 4-ft of masonry wall
4. (b) Bi-level/Raised Ranch The lower level is slab on grade; walkout
5. (a) Split-Level: The lower level (LL) is slab on grade; the main floor is raised over a crawl space
5. (b) The Main Floor and the upper level can be separated from the lower level in order to raise the structure.

Criteria for flood proofing TYPICAL STRUCTURES

1. Loss of living space due to flood proofing measures will be minimized or replaced.
2. No dry flood proofing for depths>2 feet

Larger Residential
Multi-Family = 12
Garden Apts = 13
High Rise Apts = 14
Townhouses = 15

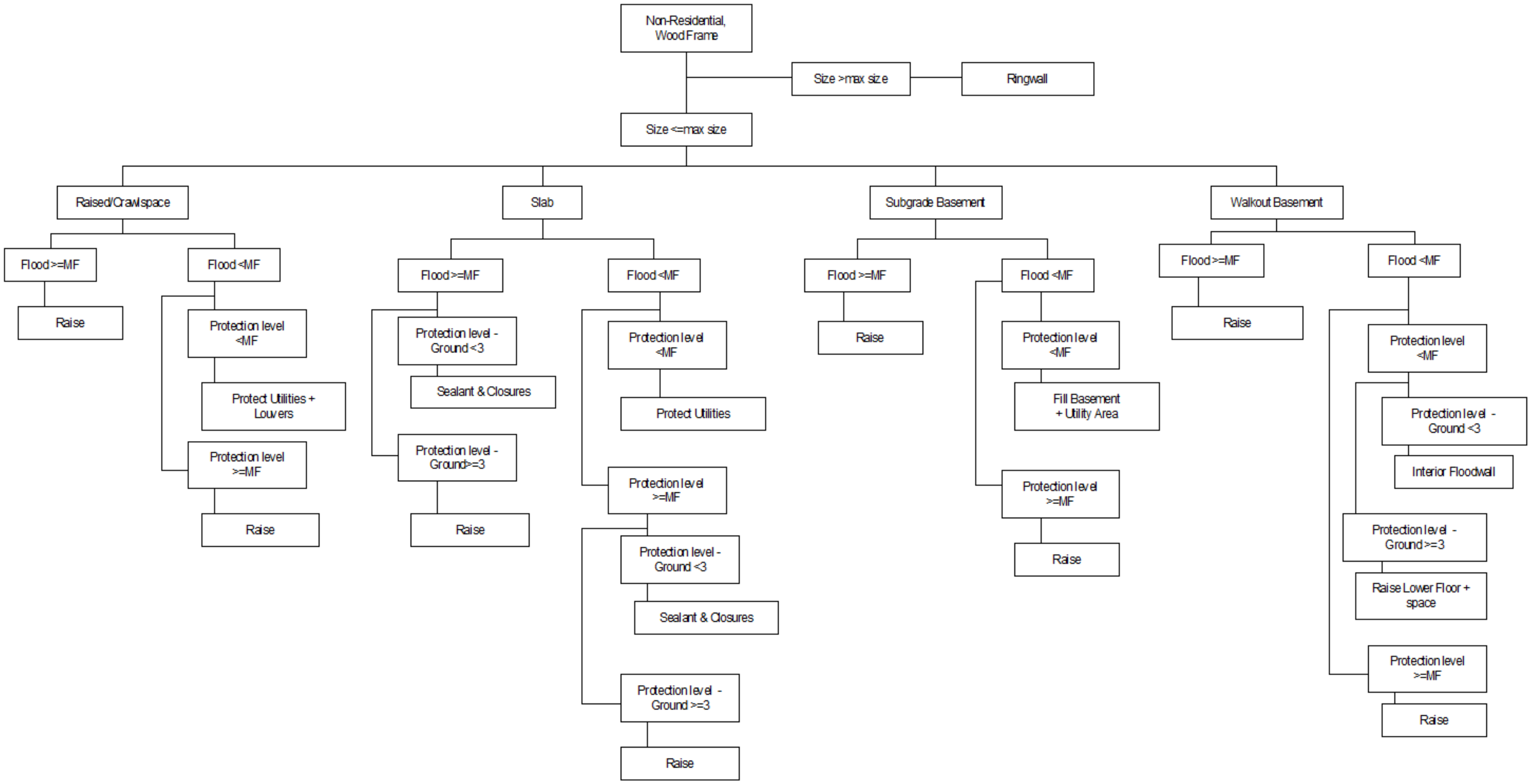
Figure 6: Residential Flowchart



Basement Codes
Walkout Basement = 1W
No Basement/Slab on-grade = 0
Subgrade Basement = 1S
Partial Subgrade Basement = 3
Raise/Crawlspace = 2

Larger Residential
Multi-Family = 12
Garden Apts = 13
High Rise Apts = 14
Townhouses = 15

Figure 7: Residential Flowchart (continued)



Basement Codes
Walkout Basement = 1W
No Basement/Slab on-grade = 0
Subgrade Basement = 1S
Partial Subgrade Basement = 3
Raise/Crawlspace = 2

Figure 8: Non-Residential Flowchart

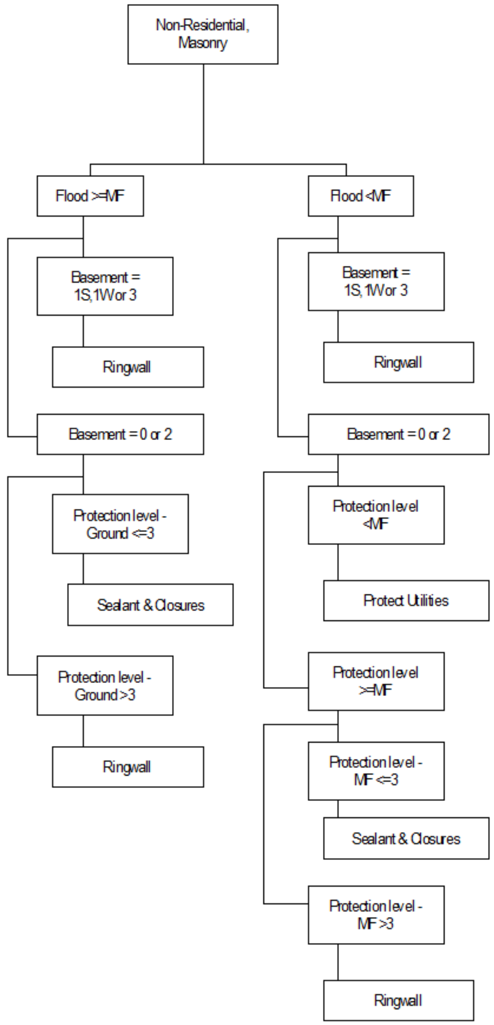


Figure 9: Non-Residential Flowchart (continued)

C-6. Proposed Structural Treatments

The following sketches indicate generic solutions and elevations, which are intended for conceptual purposes only. Actual designs will be based on specific conditions at each site.

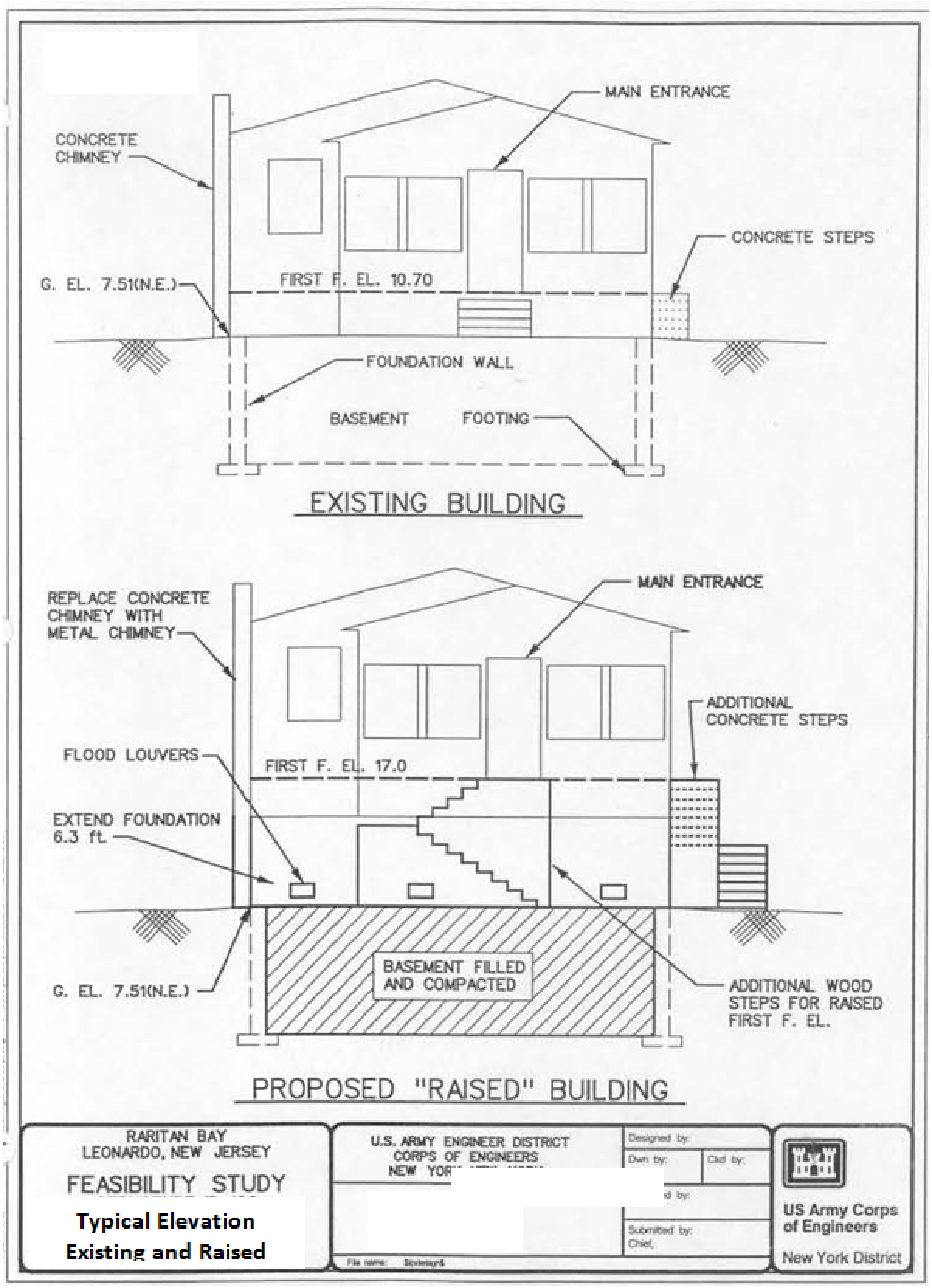


Figure 10: Type A Proposed Structural Treatment

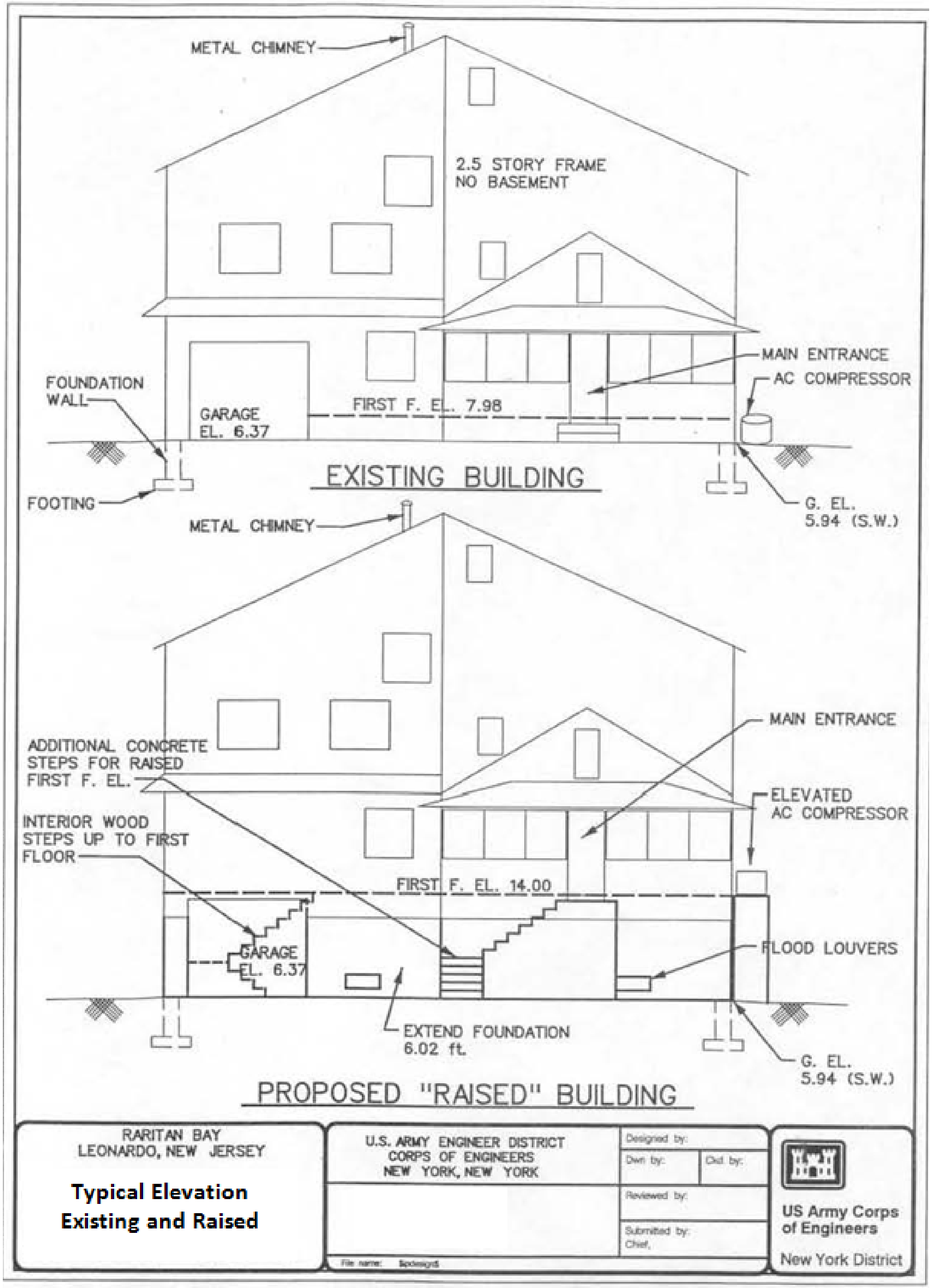


Figure 11: Type B Proposed Structural Treatment

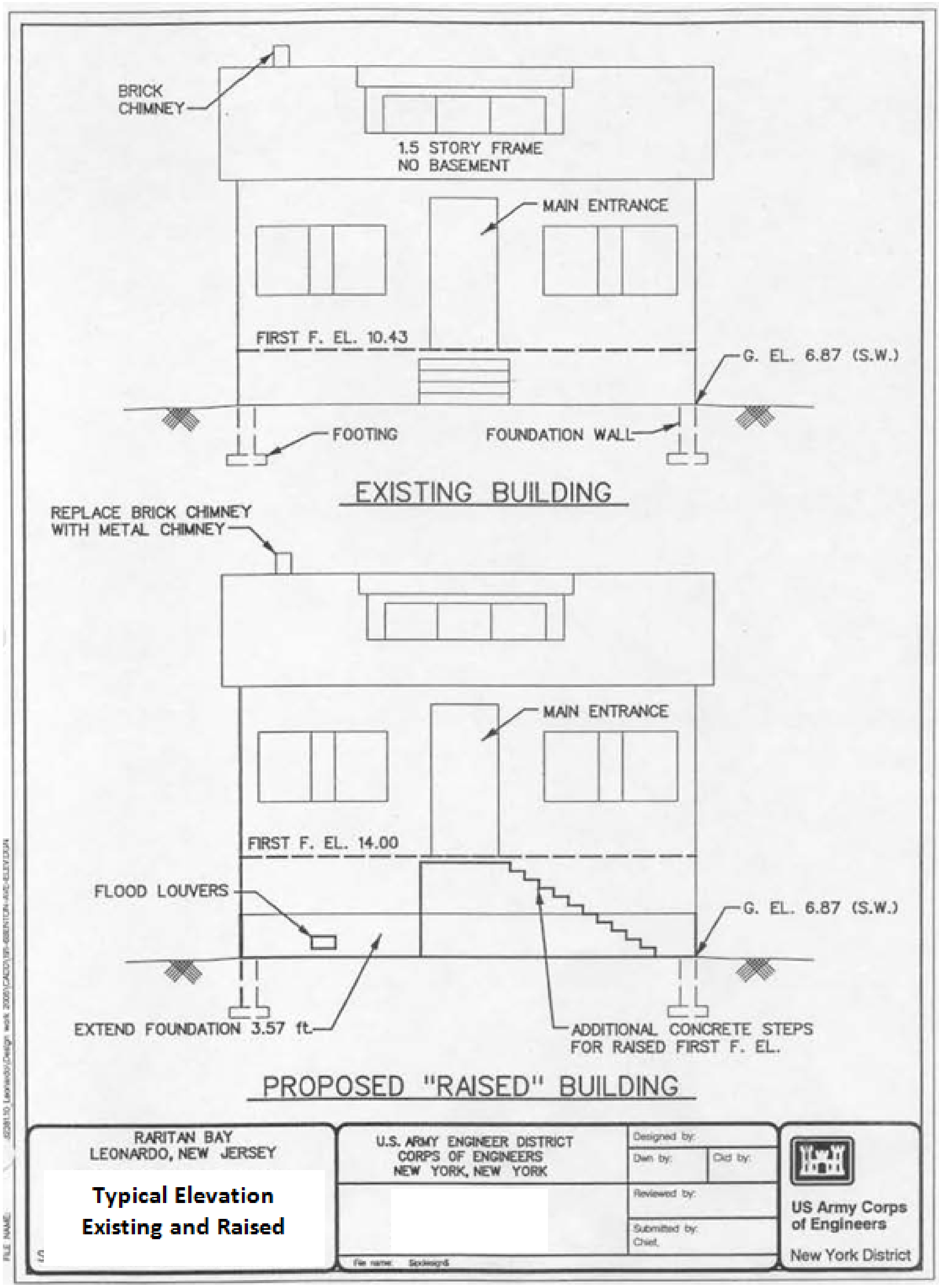


Figure 12: Type C Proposed Structural Treatment

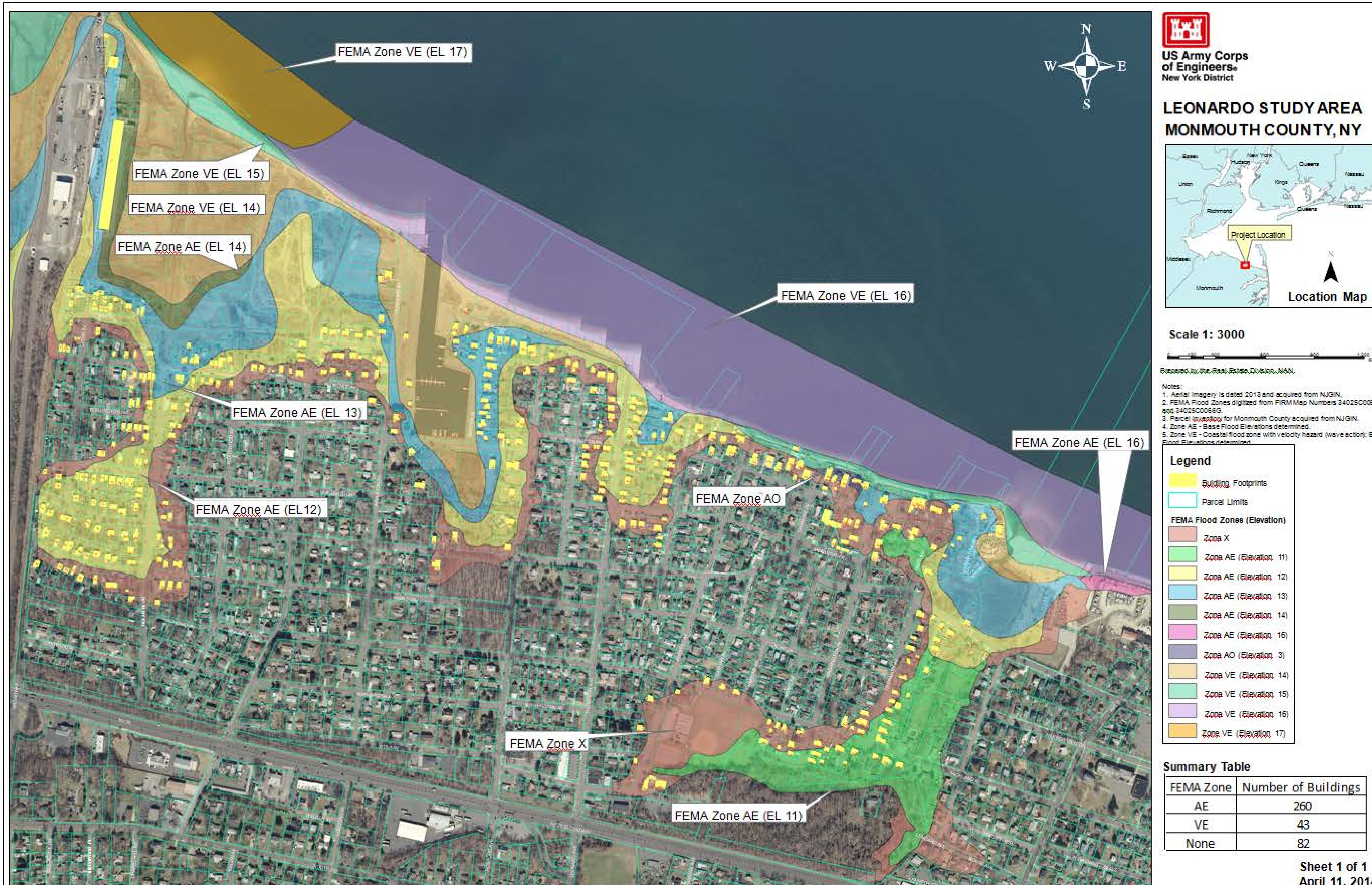


Figure 13: FEMA Sub Zones Map