SOUTH SHORE OF STATEN ISLAND, NY
COASTAL STORM RISK MANAGEMENT

INTERIM FEASIBILITY STUDY
FOR
FORT WADSWORTH TO OAKWOOD BEACH

Draft Interior Drainage Appendix

US Army Corps of Engineers
New York District

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1 INTRODUCTION

1.1 Scope

1. This appendix documents existing interior drainage facilities including major storm sewer outfalls, gates and natural/excavated ponds. In addition, this appendix documents the analysis and design of proposed interior drainage facilities including natural storage, excavated ponds, channels, pipe outlets, pump stations and tide gates to control the interior precipitation runoff. The analysis herein represents the results of the interior drainage facility formulation.

2. The appendix has been organized to provide the reader with a summary of the hydrologic/hydraulic models with their results, design and economic criteria, followed by an overview of the formulation process leading to the selected and optimized plans. The formulation effort incorporates an analysis of varying types and sizes of interior drainage facilities to determine the plan which maximizes net benefits while meeting the Minimum Facility design criteria.

1.2 Existing Interior Facilities

3. Existing interior drainage facilities lie landward or upland of the beach dunes, levee and elevated road beds that run along Staten Island's south shore. The crests of the existing structures and landforms range in elevation between 9 and 10 feet NGVD 1929.

4. One portion of the existing drainage facility is a tide gate structure and levee system that crosses along the east branch of Oakwood Creek near the Oakwood Beach Waste Water Treatment Plant. The crest elevation of the levee is approximately 10 ft. NGVG29 (approximately a 15-year level of protection). The length of the levee is approximately 730 feet (including the tide gate structure). The tide gate length is approximately 21 feet with three sluice gates (each gate opening is approximately 5' X 5'). The typical operation plan for the tide gate would include it being open under normal conditions and closed under storm conditions (i.e., where the ocean water level is higher than the mean high water). .

1.3 Future Interior Facilities Conditions

5. The existing facilities are located on land that is scheduled to be part of the Mid-Island Bluebelt Drainage Plan. The Bluebelt program is managed by New York City Department of Environmental Protection (NYCDEP), Bureau of Water and Sewer Operations. The program is scheduled to be constructed over a 30-year period to develop stormwater management systems utilizing constructed ponds, existing wetlands, improved capacity of hydraulic structures and other Best Management Practices (BMPs) to mitigate impacts of urban runoff on local flooding and to reduce pollutant loads. Typical Bluebelt drainage plans include the construction of extended detention basins, pocket wetlands, sand filters, meandering streams and stilling basins, along with the restoration of streams, retrofit of existing ponds and culvert reconstruction.

6. The Mid-Island Bluebelt Drainage Plan received environmental approval in 2013. Some real estate has already been acquired by the City under the Plan and some small construction contracts are in progress. Because the overall Bluebelt program is not included in the 2013 capital budget, the stormwater management systems have not been considered as part of the existing or future conditions. Details on the program are available at the New York City Department of Environmental Protection (NYCDEP) website.

1.4 Climate Change

7. In accordance with Corps of Engineers ECB 2014-10, “Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects”, documentation of a qualitative response to the question “Is climate change relevant to the project goals or design?” is required. The primary feature of the South Shore of Staten Island project is a line of protection consisting of levees and floodwalls which provides protection against coastal storm events. Potential precipitation change might only be relevant to the interior drainage hydrology and would not be a significant risk to the primary features of this project.

8. Because the selected interior drainage plan consists of existing storm sewer outlets (i.e. substantially sized box culverts) and natural/excavated ponding, the possible impact of climate change on these interior facilities is fairly limited. The selected line of protection, with selected interior drainage plan, will reduce the interior water surface elevations within the project area by approximately 2.8 to 6.4 feet for the proposed interior design event (1% (100yr)). If large events, like the 1% event, do indeed become larger, the interior water levels will be still be significantly lower than the exterior water level that could impact the project area without the line of protection. Existing gravity outlets, with excavated ponds, are unlike features such as interior levees, or pump stations, which can result in sudden and catastrophic increases in flood depths once their design capacity is exceeded, or fails. Ponds within the selected interior drainage plan will provide some flood risk reduction, even if the peak discharge for each return period increases over time.

9. Also, the actual amount of available storage within the project area is increasing based upon the following factors:

1) NYCDEP has released a report that outlines a plan called the “BluebBelt Plan”, in which the main goal is to develop storm water management systems that utilize constructed ponds, existing wetlands, improved capacity of hydraulic structures, and other Best Management Practices (BMP’s) to mitigate impacts of urban runoff on local flooding. This plan includes preserving and/or acquiring lands for natural/excavated storage. The document is located at the following website if more information is desired:

http://www.nyc.gov/html/dep/html/environmental_reviews/midisland_bluebelt_drainage_plan.shtml.

2) Following Tropical Storm Sandy, NYSDEC has proceeded with an acquisition plan for structures within the project area. Most of these structures lie within the Oakwood Beach area. Once these structures are acquired and demolished, this will leave additional natural storage within the project area. Additional information can be found on the following website:

http://www.dec.ny.gov/enb/20130821_not2.html

10. In effect, there will be an increase in natural storage, a decrease of structures within the project area, and a limited amount of area for potential development within the project area low-lying locations.

11. The interior drainage facilities, like most Flood Risk Management Projects, are designed based upon a limited estimate of a large and infrequent event and, as such, are designed with the appropriate allowances for risk and uncertainty. Projected climate change impacts appear to be well within the normal range of hydrologic variability for Flood Risk Management Projects.

1.5 Study Location

12. The overall study area lies within the borough of Staten Island, County of Richmond, within the limits of the City of New York. The study area consists of approximately six miles of coastline extending along the Lower New York Bay and Raritan Bay (See Figure 1). The approximate west and east limits (i.e. along the south shoreline) of the study area are Oakwood Beach and the easternmost point of land within Fort Wadsworth at the Narrows. Across from Staten Island's western shore is the New Jersey shoreline at the southern shore of Raritan Bay, which extends from the community of South Amboy to the Sandy Hook peninsula. East of Staten Island is Brooklyn on the Narrows, Coney Island on the Lower New York Bay, and Rockaway Point on the Atlantic Ocean—all within New York City. The approach to Lower New York Bay from deep water in the ocean is through a 6-mile wide opening between Sandy Hook, New Jersey and Rockaway Point, New York.

13. The principal communities along the south shore of Staten Island (from east to west) are South Beach, Midland Beach, New Dorp, Oakwood Beach, Great Kills, Annandale Beach, Huguenot Beach, Price's Bay and Tottenville Beach. The reach evaluated in the Interim Feasibility Study is Fort Wadsworth to Oakwood Beach.

1.6 Physical Characteristics

14. The interior drainage area that conveys precipitation run-off to the project reach is approximately 8 square miles. The topography from Fort Wadsworth to Oakwood Beach varies from moderately steep near the drainage divide to somewhat flat near the Atlantic Ocean and Raritan Bay. Elevations vary from about 400 ft. NGVD 1929 at Todt Hill, the highest area, to 1 ft. NGVD 1929 at the lowest areas of Oakwood, Midland and South Beaches. The developed, or urbanized, areas are predominantly residential in nature, with commercial establishments centered on Hylan Boulevard, a main auto route extending approximately through the center of the Fort Wadsworth to Oakwood Beach drainage area. A portion of this area is occupied by

Miller Field, formerly a U.S. Army Base, and now under the jurisdiction of the National Park Service.

15. Richmond Road, another major auto route, forms the base of Todt Hill and separates the steepest part of the Fort Wadsworth to Oakwood Beach drainage area from the remainder to the southeast. The floodplain of New Creek, a flat tidal creek, is partially filled in by residential development. Oakwood Creek drains Midland Beach, an area located near the center of the Fort Wadsworth to Oakwood Beach drainage area bounded by Hylan Blvd., the Atlantic Ocean, Miller Field, and the South Beach Psychiatric Center.

16. The major streets, which are roughly perpendicular to Hylan Blvd. and Richmond Road, run downhill in a northwest to southeast (seaward) direction and act as channels to bring surface runoff to the low-lying areas of the project area. The major storm sewers (see Figure 2) that lie under some of these streets (Sand Lane; Quintard St. and Raritan Avenue; Seaview Avenue, Naughton Avenue, Midland Avenue, Greeley Avenue; and New Dorp Lane, Ebbitts St., Tysens Lane) serve to bring some upland interior runoff through the existing coastal barrier (i.e. a mix of levees, dunes, and elevated topography for roadways) and out into the Upper Raritan Bay. Aside from discharging interior runoff, these storm sewer outfalls form jetties on the beach, which create barriers to costal sediment transport, trapping sand on the updrift side.

1.7 Source of Flooding

17. Flooding in this area can result from either high storm surges from the Bay or interior precipitation runoff that cannot be conveyed to the Bay through the existing interior drainage system. The study area is mostly protected from storm surge until floodwaters rise above Father Capodanno Boulevard or other local topographic features, such as the dunes or levees. These existing landforms provide relief from surge levels during high frequency storm events (e.g. a 2-year coastal storm event), but for higher surge levels, large low-lying portions of the inland area become inundated causing extensive property damages and risks to life-safety.

18. The frequency of inland inundation will continue and increase as sea level is projected to rise. Relative sea level in the project area has been rising at an average of 0.014 feet per year. It is also anticipated that continued development and fill placement will occur within the floodplain. As new construction is elevated above the base interior flood elevation, the fill will reduce storage for interior runoff and may exacerbate interior flooding conditions during high intensity rainfall events.

2 ANALYSIS OF HYDROLOGIC AND HYDRAULIC CONDITIONS

19. The analysis and design of the Interior Drainage Plan is intended to supplement the Engineering and Design Plan and manage the residual risks from flooding. The Tentatively Selected Engineering and Design Plan includes an improved coastal barrier system made up of levees, tide gates, floodwalls, and a buried seawall/armored levee for the project reach. With the introduction of these new flood management measures, the hydraulic characteristics between the with and without project conditions may change during tidal and interior runoff flooding events.

20. The main objective of implementing a new Coastal Storm Risk Management System is to reduce the risk associated with flooding and while the Tentatively Selected Engineering and Design Plan intends to achieve this objective for high surge levels, interior measures are needed in order to meet this objective during high precipitation rainfall events. At a minimum, the Interior Drainage Plan must demonstrate that the Minimum Facility is met or that the local storm drainage system functions essentially as it would without the Engineering and Design Plan in place (EM 1110-2-1413). Alternative interior drainage measures may be introduced to further improve the interior flooding conditions under the condition that the additional cost of incorporating the additional design features does not outweigh the additional benefit resulting from a reduction in flood related damages.

2.1 Basis of Interior Drainage Design

21. The analysis presented herein is based on the concepts and guidelines contained in EM 1110-2-1413 "Hydrologic Analysis of Interior Areas", dated 15 Jan 1987, ER 1105-2-100 “Planning Guidance Notebook”, dated 22 April 2000, ER 1105-2-101 “Risk Analysis for Flood Damage Reduction Studies”, dated 03 Jan 2006, and EM 1110-2-1417 “Flood Runoff Analysis”, dated 10 Jul 2013.

2.1.1 Rainfall and Storm Surge Correlation Analysis

23. For the with and without-project conditions, the exterior stage (stillwater elevation within Raritan Bay) is an important factor in the drainage of the interior precipitation runoff. The exterior stage is controlled by the tide cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Bay through the existing high ground (i.e. Fr. Capodanno Boulevard and other local high ground) via stormwater outfalls. In the without project condition, these outfalls cease to operate when the exterior stage (tide/storm surge level) rises above the outfall opening because they rely on gravity to facilitate the transport of interior surface runoff. Similarly if a new coastal storm risk management structure is introduced (with project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under high exterior (tailwater) stage conditions, would not be able to leave through gravity flow. Therefore it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both the with and without project conditions.

24. To understand the relationship between the interior and exterior stage conditions, if any, a correlation analysis needs to be performed. In accordance with EM 1110-2-1413, the correlation analysis should include a data analysis of the correlation, dependence, and coincidence of the interior and exterior stage relationship. In the vicinity of the South Shore of Staten Island study area, recent Corps correlation analyses have been conducted as part of the South River, NJ and Port Monmouth Feasibility Studies as depicted on the Figure 2A. From these two study areas, we can expect that the storm surge in the Raritan Bay does not correlate to the precipitation events, is lightly dependent upon precipitation events, and that its peak stage is unpredictable but could coincide with peak interior discharges. Both previous Feasibility Studies are at this time are authorized projects and have a correlation analysis that was accepted through the HQ review process. A summary of the previous analyses and their applicability to the South Shore of Staten Island Interim Feasibility Study is provided in the in this section and its subsections.

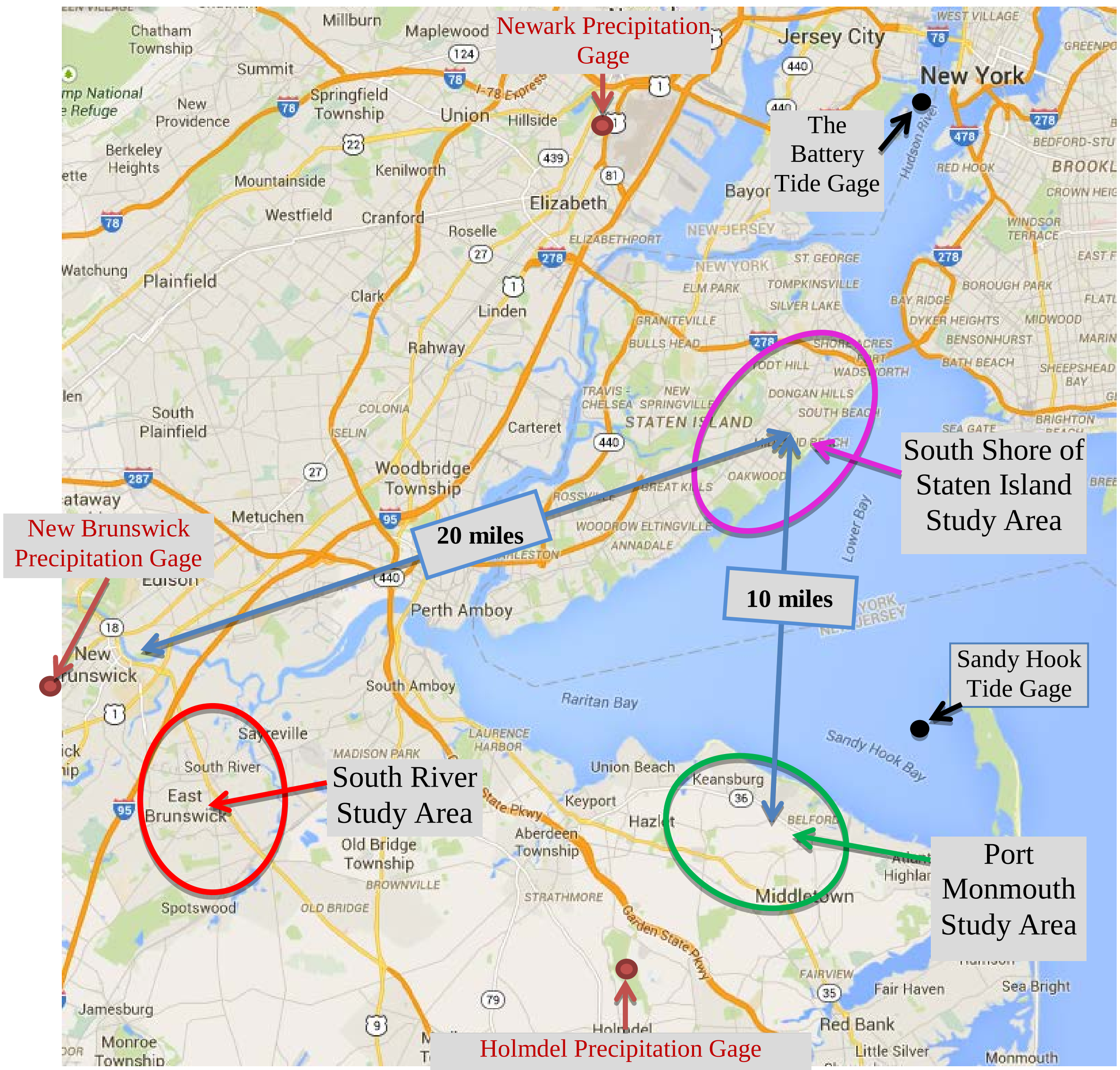

Figure 2A Gage and Correlation Study Area Plot

25. The South Shore of Staten Island Interim Feasibility Study, the South River, and Port Monmouth Feasibility Studies are within the Raritan Bay Inlet and have reasonably similar tidal conditions. Storm surge conditions during extreme events may vary slightly between the three study areas. A less than 0.5 feet peak stage difference was recorded between The Battery, NYC (see Figure 2A) and Sandy Hook, NJ during Hurricane Sandy, NHC-NOAA.

26. All three study areas are within 20 miles from each other and have similar geomorphological conditions. They've experienced relatively similar rainfall conditions during past severe storm events. Figure 2A shows the locations of three local rainfall gages used to measure the variance in rainfall among the study areas. Table 1 presents the total rainfalls during the last two severe weather events at these gages. The observed variance in rainfall totals between study areas would not be significant enough impact the correlation analysis results between sites.

TABLE 1 RAINFALL TOTALS NEAR STUDY AREA DURING IRENE AND SANDY

	Rainfall Total (inches)	
Precipitation Gage Location	**Hurricane Irene**	**Hurricane Sandy**
Holmdel	7.75	1.84
New Brunswick	8.08	1.77
Newark International Airport	8.92	1.06

27. In accordance with EM 1110-2-1413, the correlation analyses performed for the South River and Port Monmouth studies considered the correlation, dependence, and coincidence of the exterior flood levels and interior flood levels.

2.1.1.1 Correlation

28. For the South River correlation analysis, hourly water surface elevations were obtained from the gage at Sandy Hook for the time period from Jan 1933 to Feb 2000. They were then reduced to obtain daily high tide records for that time period (It should be noted that since these were hourly readings and not peak values, the actual peak values may have been slightly higher.). Daily rainfall data for the same time period were also obtained from the New Brunswick precipitation gage (location shown on Figure 2A). After cleaning the datasets for unpaired data points and other suspect data, the aforementioned 67 years of systematic data (as adapted from the South River Study) along with the peak information from local storm events of record from the last 14 years (Hurricane Irene and Sandy) were combined and plotted on Figure 2B. The upper right axis of Figure 2B also includes stage frequency information for Sandy Hook based upon preliminary numerical model results, and the rainfall frequency information for New Brunswick performed as part of the South River Study.

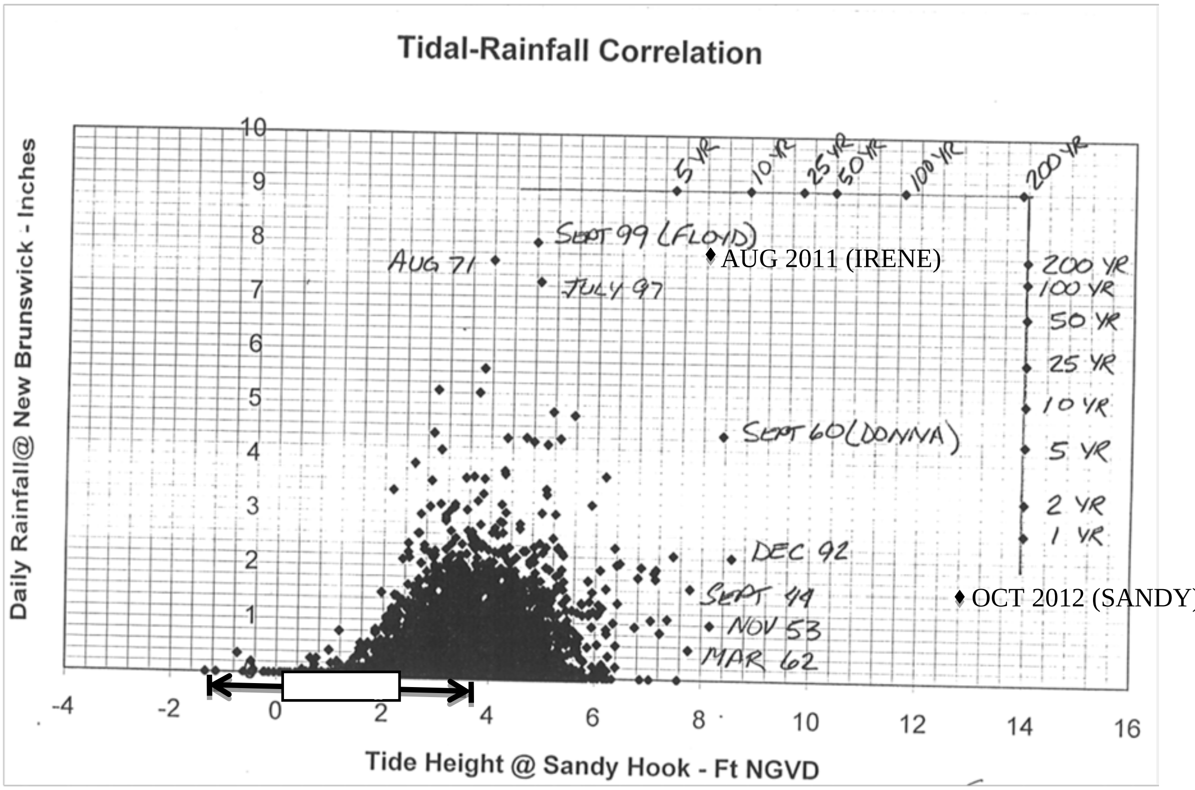

Figure 2B Tide-Rainfall Correlation Plot

29. As demonstrated in Figure 2B, most of the higher tide events occurred with little rainfall, and most high rainfall events occurred with normal tides (normal tide range is shown on x-axis). This along with the general wide scatter of precipitation amounts with a constant storm surge and vice versa indicates that there is no correlation between the surge events and precipitation. Therefore, it's not reasonable to say that we could predict one condition from the other based on these historic records.

2.1.1.2 Dependence

30. It is understood that the storms that typically produce tidal surges, *i.e.*, hurricanes and northeasters, can also produce somewhat significant rainfall. Likewise many of the high rainfall events are accompanied by some degree of storm surge. If this were not true, then the high surge events would not likely have any rainfall and vice versa and the paired data in the Figure above would fall much closer to each axis. As expected the Figure reveals a minor dependence between the interior and exterior conditions. The fact that the main cluster of points that include some rainfall (1-2 inches) also include a tide height greater than the mean tide level (0.9 ft. NGVD 1929) is evidence of this.

2.1.1.3 Coincidence

31. The coincidence between the interior and exterior conditions involves the timing of the peak discharge from the interior drainage analysis and the timing of the peak exterior stage from

the exterior storm surge analysis. In the exterior condition, the timing of the peak exterior stage is unpredictable because of the impacts of tidal fluctuation to the overall storm surge elevation. Therefore, predicting the coincidence of the peak exterior event and the peak interior flows is uncertain. Assuming that the interior and exterior events occur at the same time would be considered the worst case scenario and a conservative approach for modeling coincidence. Given that this coincidence was observed during Hurricane Donna in 1960, it has been incorporated into the model assumptions.

2.1.1.4 Modeling in the Study

32. Since there is not a correlation between rainfall/runoff events (interior condition) and tidal flooding events (exterior condition) but because there is a minor dependence between the two, it is considered most likely that only limited surface runoff will coincide with severe storm surge and significant storm surge will coincide with only moderate rainfall. Historic data indicates that the majority of interior runoff events will coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

33. The interior stage analysis was conducted for events with five recurrence intervals: the 2-year, 10-year, 50-year, 100-year, and 500-year frequency events. In order to develop a stage-frequency relationship, the interior events were routed against exterior tidal marigrams. For the most likely or expected flooding scenarios, the five interior storm events were routed against a 2-year exterior tide, and a 2-year interior storm event was routed against the five exterior events. Table 2 presents the different interior and exterior runs analyzed and the risk condition associated with each.

TABLE 2 RECOMMENDED ANALYSIS APPROACH – COMBINATION OF INTERIOR AND EXTERIOR CONDITIONS

Varied Interior Condition		Varied Exterior Condition		Risk Condition
Interior Flow	Exterior Stage	Interior Flow	Exterior Stage	
2yr	Normal	N/A		Lower Bound
10yr	Normal	N/A		Lower Bound
50yr	Normal	N/A		Lower Bound
100yr	Normal	N/A		Lower Bound
500yr	Normal	N/A		Lower Bound
2yr	2yr	2yr	2yr	Expected
10yr	2yr	2yr	10yr	Expected
50yr	2yr	2yr	50yr	Expected
100yr	2yr	2yr	100yr	Expected
500yr	2yr	2yr	500yr	Expected
2yr	2yr	2yr	2yr	Upper Bound
10yr	10yr	10yr	10yr	Upper Bound
50yr	10yr	10yr	50yr	Upper Bound
100yr	10yr	10yr	100yr	Upper Bound
500yr	10yr	10yr	500yr	Upper Bound

34. As demonstrated in the Risk Condition column of Table 1, uncertainty was incorporated into the analysis by establishing lower and upper coincidental frequency bounds. For the lower bound, the interior storm events were routed against a normal exterior tidal condition and for the upper bound the interior events were routed against a 10-year external tide. The maximum water surface elevation (WSEL) of corresponding coincidental frequencies (e.g., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency. In the with-project analysis, only the 500-year exterior event was found to be more damaging than its corresponding reversed condition frequency because it would overtop the proposed seawall/armored levee. The analysis was performed for both the current and expected future conditions to include the impacts of sea level rise.

35. The Plan Formulation Section of this Appendix only presents the selected interior stage utilized in the economic comparison. Water surface elevation calculations under all conditions demonstrated in the above table are presented as a sub-appendix to this Interior Drainage Appendix.

2.1.2 Hydrologic Analysis

36. The HEC-HMS model, version 3.5, developed for the interior drainage areas of South Shore of Staten Island is described in the subsequent sections of this appendix. Basic input parameters developed for the hydrologic models include: surface area, rainfall generated for a

series of hypothetical storm events (2 to 500-year return periods), runoff curve number, and time of concentration (Tc).

2.1.3 Hydraulic Analysis

38. Proposed outlet structures, such as culverts and pipes, running through the proposed levee and buried seawall/armored levee were analyzed within HEC-HMS using inlet and outlet control analyses as described in Federal Highway Administration's Hydraulic Design Series No. 5 "Hydraulic Design of Highway Culverts" (HDS-5).

2.2 Hypothetical Storm Surge Data

39. For storm events (tropical events such as hurricanes and extratropical events such as nor'easters), a storm hydrograph was developed to simulate surge levels during storm conditions. Two main assumptions were made to develop the storm hydrograph: (1) the peak elevation of the storm will occur at high tide and (2) the duration of the storm is approximately two days. Storm hydrographs were developed for return periods from 2-year to 500-year and the peak elevation for each return period was developed as described in the Engineering Appendix. Hypothetical tide marigrams (hydrographs) used in this interim study for the exterior stages are plotted in Figure 3. The storm surge data utilizes the stage frequency curves from FEMA's forthcoming New York City coastal Flood Insurance Study (FIS).

40. The relationship between rainfall/runoff (including river flow) and storm surge is highly uncertain and may have a significant impact on interior stages. Uncertainty was incorporated into the analysis by routing the interior storm events against a normal exterior tidal condition to establish a lower bound of interior flood levels, and routing the interior events against a 10-year external storm surge conditions to establish a reasonable upper bound of interior flood levels. This methodology was then applied with a 2-year external surge level to create the expected interior flood levels. The three conditions: expected (design), lower bound, and upper bound were then incorporated into the economic analysis using a triangular probability distribution. The interior stages in Table 1 represent the water surface elevations based upon the precipitation runoff and tidal routing whichever is more restrictive.

TABLE 3 HYPOTHETICAL INTERIOR STAGES WITH UNCERTAINTY EXISTING CONDITIONS - 100 YEAR FREQUENCY EVENT

Drainage Area	Expected Interior WSEL ft. NGVD 1929	Lower Bound Interior WSEL ft. NGVD 1929	Upper Bound Interior WSEL ft. NGVD 1929
Area A	7.10	5.82	8.22
Area B	6.21	4.75	6.98
Area C	6.36	5.52	7.93
Area D	9.78	7.97	10.35
Area E	8.40	7.99	9.36

2.3 Storm Surge Duration

41. While storms with longer surge duration are possible, multiple peak conditions have a significantly lower probability of occurring. From the South River Hydrology and Hydraulic Appendix, dated September 2002 (a study area closely located to the South Shore of Staten Island study area), a preliminary sensitivity analysis was conducted to determine if further study was needed to evaluate the effects of longer surge duration events. The results of the preliminary analysis indicated that multiple exterior peak high tides did not significantly impact interior water surface elevations. The interior drainage facilities were modeled with the storm surge causing one peak high tide.

2.4 Development of HEC-HMS Models of Interior Inflow

42. HEC-HMS was used in order to simulate the interior runoff inflow landward of the tentatively selected plan alignment. The model consisted of multiple sub-basin runoff computations, hydrograph combinations and routing, and also hydrograph diversions to capture the behavior of the major storm sewers in the study drainage area.

43. Figure 5 through Figure 11 depict the progression and behavior of the interior drainage system per sub-basin. The schematics match the set-up of the interior flow HEC-HMS models.

2.5 Drainage Area Delineation

44. Interior drainage basins and sub-basins were delineated on two U.S. Geological Survey (USGS) quadrangles, *The Narrows, NY* and *Arthur Kill, NY-NJ*, at a scale of 1 inch = 2000 ft. with 10-foot contour intervals and supplemented by City of New York 500 scale (1 inch = 500 feet) topographic mapping, using the 2 ft. contour intervals in flatter areas where ten foot contours were inadequate. Drainage plans of Sanitary and Storm Sewers, dated March 1969, at a scale of 1 inch = 150 feet, from the NYCDEP Division of Sewer Design, showing existing and proposed sub-surface sanitary and storm drainage, were used to further refine the drainage divides, where storm sewer and surface runoff divides did not coincide and where a distinct surface runoff divide was unapparent. One area where this was especially evident was in the extremely flat and irregular Midland Beach New Creek floodplain area, southeast of Hylan Blvd. and between Seaview and Greeley Avenues. Drainage divides were also field checked in mid-July 2002 for the flat Midland Beach area and the beach boardwalk along the Bay.

45. Information presented on the NYCDEP Division of Sewer Design storm sewer mapping was also used to define a relationship between storm sewer peak or design discharges, rational runoff coefficients, and time of concentration, drainage area, and storm frequency, to ensure that the interior drainage hydrology developed as part of the interim feasibility report would conform to existing NYCDEP criteria for storm sewer design, as NYC storm sewers may be incorporated into the Corps of Engineers flood control design for the South Shore of Staten Island.

2.5.1 Delineation Methods

46. An Interior Drainage Area is defined, for the purpose of engineering analysis, to be a distinct land area which drains to one primary outlet location landward of the tentatively selected plan alignment. The identification and distinction of such areas is complicated by the presence of man-made features such as storm sewers, which may divert flow into or out of a Drainage Area. In some cases, otherwise distinct and discrete interior areas have low-lying lands that may combine during low frequency storms because the high pooling elevations that overtop the divide between Drainage Areas.

47. For low frequency events where the rainfall exceeds the capacity of the storm sewer system, which are typically designed for the five or ten year storm, any additional storm runoff that occurs will remain on the surface and will flow towards the Bay along the path of lowest energy, which is, in many cases, the streets that run downhill from northwest to southeast or the remaining natural channels of New Creek in the Midland Beach area.

2.6 Delineated Interior Drainage Areas

48. The locations and naming of major interior drainage areas follow a pattern from west to east starting with A, continuing on to B, C, D, and ending with E. The interior drainage areas are depicted on Figure 4, and the interior drainage sub-basins used for the HEC-HMS analysis are depicted in Figure 4A.

2.6.1 Drainage Area A

49. Drainage Area A is located along a tributary of Oakwood Creek a few hundred feet to the northwest of the Oakwood Beach Waste Water Treatment Plant (see Figure 4). There is no existing storm sewer outfall in this area. The interior drainage area consists of approximately 0.46 square miles (approximately 295 acres) of developed urban land, with freshwater/saltwater wetlands (approximately 20 acres). During a storm event, Riga Street starts to flood at about elevation 7 feet NGVD 1929. The ground elevation adjacent to the lowest buildings is approximately 9 feet NGVD 1929, but many of the homes along the low-lying portion of the study area were destroyed by Hurricane Sandy (October 29-30, 2012). Since Hurricane Sandy, several agencies have been participating in efforts to acquire low-lying properties and convert them into open space to decrease losses form future coastal flooding events. Within the Areas A and B, New York State Department of Environmental Conservation (NYDEC) has slated numerous structures for acquisition and demolition. Programs affecting other sub-basins have not progressed to a point that specific properties for acquisition can be specified. Overall, the economic inventory of buildings has been updated to reflect recent property acquisitions.

2.6.2 Drainage Area B

50. Area B is located to the East of Interior Drainage Area A (see Figure 4). A small segment of levee was constructed in the year 1999 under Section 103 of the Continuing Authorities Program along with a gate structure housing sluice gates across Oakwood Creek. The tentatively selected plan alignment will be located significantly landward of the levees and

landforms that make up the existing coastal barrier; therefore the post-project sub-basin storage volume will be less than the existing conditions. The Drainage Area consists of approximately 1.75 square miles (1120 acres) of developed urban land, with freshwater/saltwater wetlands in the lower, seaward end (approximately 90 acres). The existing storm sewer outfalls on Tysens Lane, Ebbits Street and New Dorp Lane drain stormwater out to the Bay. During a flood event, Kissam Avenue begins to flood at approximately 4 feet NGVD 1929. The ground elevations adjacent to the lowest buildings are approximately 5 feet NGVD 1929.

2.6.3 Drainage Area C

51. Area C (Midland Beach) is the largest of the Interior Drainage Areas and is located in the center of the Fort Wadsworth to Oakwood Beach reach, and includes Miller Field. It's approximately diamond-shaped, with its four corners at the principal points of a compass (see Figure 4). The northern, highest point is near Todt Hill at an elevation of about 400 ft. NGVD 1929. The southernmost point is where the buried seawall/armored levee plan alignment crosses New Dorp Lane. The easternmost point is where it crosses Seaview Avenue. The westernmost point is on a hilltop between Moravian Cemetery and the High Rock Girl Scout Camp. The northeastern most portion of Area C is drained by the Liberty and Seaview Avenue storm sewers and accounts for approximately 0.45 square miles (288 acres) of drainage area. The southwestern most area, including Miller Field, is drained by the Bryant and Greeley Avenue storm sewer and is approximately 1.40 square miles (896 acres). The central area, which contains the major ponds and wetlands (approximately 175 acres), is drained by the Naughton, Seaview, Hunter and Midland Avenue storm sewers and is approximately 1.41 square miles (902 acres). New Creek drains into the Naughton Avenue storm sewer. Some of the New Creek tributaries are enclosed in culverts at street crossings. The total interior drainage area is approximately 3.26 square miles (2,086 acres). During a storm event, Quincy Avenue starts to flood at about elevation 3 feet NGVD 1929. The ground elevations adjacent to the lowest buildings are approximately 4 feet NGVD 1929.

2.6.4 Drainage Area D

52. Area D (Midland/South Beach) is an oblong, irregular area, with roughly a convex hexagon shape (see Figure 4). Its high point, at Todt Hill, is at the same elevation as that of Area C, 400 ft. NGVD 1929. The combined Quintard/Vulcan St. - Raritan Avenue storm sewer outfall drains Area D out toward the Bay. The lowermost part of Area D is roughly a rectangular shaped piece of land on the grounds of Richmond College, the South Beach Psychiatric Center, and Staten Island Hospital. The interior drainage area consists of approximately 1.12 square miles (716.8 acres) of developed urban land, with minimal freshwater wetlands (approximately 2 acres). The ground elevations of the lowest buildings are approximately 8 feet NGVD 1929. During a storm event, Quintard Street starts to flood at about elevation 7.5 feet NGVD 1929.

2.6.5 Drainage Area E

53. Part of the runoff from 0.40 square miles (0.256 acres) of Area D, along the southwest border of Area E, is intercepted by the Quintard/Vulcan St. storm sewer and diverted into Area

E. Area E is generally located in the northeastern most section of the Fort Wadsworth to Oakwood Beach study reach (see Figure 4). Its shape is roughly trapezoidal with Quintard Street the southwest and Ocean Avenue the northeast edges. Sand Lane, which lies midway between Quintard Street and Ocean Avenue, drains Area E to the Bay. Area E consists of approximately 0.87 square miles (556.8 acres) of developed urban land, with some freshwater wetlands (approximately 43 acres). The ground elevations adjacent to the lowest buildings are approximately 6 feet NGVD 1929. During a storm event, McLaughlin Street starts to flood at about elevation 5.5 feet NGVD 1929.

2.7 Interior Drainage Areas Inter-Relationship

54. During some low frequency storm events, high ponding elevations behind the tentatively selected plan alignment may cause the accumulated flood water of two adjacent Interior Areas to combine or overflow from one to the other. This phenomenon was modeled by combining the inflow hydrographs or was, in some cases, eliminated from the model by implementing road raising designs into the Minimum Facility Plan. The inflow hydrographs of the lower-eastern east sub-basin of Area D with Area E, and the lower sub-basin of Area A with Area B, at the flat pools, or ponds, common to both of them were combined in the HEC-HMS model. For Drainage Area C, part of the proposed Minimum Facility Plan includes elevating Seaview Avenue to eliminate the predicted overflow of floodwaters from/to Area D during the studied range of storm-events.

2.8 Future Storm Drainage System

55. Future drainage plans by the City Of New York as part of the Staten Island Bluebelt program will consist of the following:

- Area C – One box culvert: 6 ft. by 7.5 ft. and 912 ft. in length. The inlet elevation will be -2 ft. NGVD 1929 with an exterior elevation of -2.38 NGVD 1929

- Area E – One circular culvert (siphon shape): 24 to 30 in. diameter and 1255 ft. in length. The inlet elevation will be 0.69 ft. NGVD 1929 with an exterior elevation of -1.08 NGVD 1929

56. The Kissam Avenue Outfall, which is part of the Oakwood Beach Bluebelt, has been removed from the NYCDEP Bluebelt Plans.

2.9 Development of Interior Inflow Runoff Hydrographs

57. HEC-HMS was used to model the interior runoff for a range of hypothetical rainfall frequencies and durations. NRCS runoff curve numbers, NRCS unit hydrograph lag times, routing reach travel times, and hydrograph combinations and diversions, were used to define the interior basin response to the specific frequency hypothetical rainfall. Each input parameter is described in more detail in the subsequent sections.

58. Generally, within the hydrograph identifications (variable Hydrologic Element) within the HEC-HMS models, the following capital letters have the following meanings:

S = sub-basin runoff computation. R = hydrograph routing. C = hydrograph combination.
D = hydrograph diversion.

59. The streams within the Fort Wadsworth to Oakwood Beach drainage area do not have gage stations; therefore calibrating the rainfall-runoff model to previous precipitation events was not performed. Without gages for calibration, the Drainage Areas were modeled to reflect peak flows per unit area on the upper bounds of the reasonably possible value spectrum for a conservative interior flood control design and to conform with NYCDEP storm sewer design criteria.

2.9.1 Rainfall Data

60. Specific frequency hypothetical point rainfall depths for durations of 1, 2, 3, 6, 12 and 24 hours, and return periods of 1, 2, 5, 10, 25, 50 and 100 years were taken from *Technical Paper No. 40, Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 Hours and Return Periods from 1 to 100 Years (1961).* Point rainfall data for durations of 5 and 15 minutes were taken from *Technical Memorandum NWS Hydro 35, Five to 60-minutes Precipitation Frequency for Eastern and Central United States (1977).* 48 hour rainfall data was taken from *Technical Paper No. 49, Two-to-Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States (1964)*. Data were plotted on log probability paper and extrapolated to project a value for the 500 year storm. Hypothetical point rainfall depths for the 1 through 500 year storms are shown in Table 2.

TABLE 4 SPECIFIC FREQUENCY HYPOTHETICAL POINT RAINFALL DEPTHS IN INCHES

Duration	Return Period							
	1 Year Rainfall [in]	2 Year Rainfall [in]	5 Year Rainfall [in]	10 Year Rainfall [in]	25 Year Rainfall [in]	50 Year Rainfall [in]	100 Year Rainfall [in]	500 Year Rainfall [in]
5 min.	0.36	0.42	0.50	0.55	0.64	0.70	0.77	0.92
15 min.	0.68	0.80	0.99	1.12	1.31	1.46	1.61	2.01
1 hour	1.22	1.44	1.85	2.15	2.48	2.77	3.10	3.90
2 hours	1.49	1.80	2.28	2.64	3.14	3.47	3.82	4.65
3 hours	1.64	2.00	2.60	3.00	3.50	3.83	4.28	5.30
6 hours	2.00	2.38	3.15	3.65	4.28	4.70	5.15	6.40
12 hours	2.38	2.86	3.69	4.30	5.10	5.70	6.30	7.70
24 hours	2.70	3.34	4.34	5.06	5.85	6.55	7.38	9.10
48 hours	3.13	3.87	5.23	6.00	7.05	8.00	9.00	11.10

61. A standard, small correction from point to finite area rainfall was made within the HEC-HMS models. The specific frequency hypothetical point rainfall depths were also inverted into an intensity-duration-frequency (IDF) diagram to enable comparison with the IDF diagram used in the NYCDEP storm sewer design. The two diagrams were found to be essentially the same. The IDF comparison was also used to ensure that the peak discharges computed by the HEC-HMS model would comply with NYCDEP criteria for storm sewer design.

62. A 48-hour hypothetical storm was used to allow for HEC-HMS interior inflow routing against the exterior time-varying marigrams (astronomic tide plus storm surge) through four tide cycles.

2.9.2 NRCS Runoff Curve Numbers

63. The NRCS, formerly known as the Soil Conservation Service (SCS) runoff curve number procedure as outlined in *NRCS Technical Release No. 55 (TR-55), Urban Hydrology For Small Watersheds* was used to define the rainfall-loss-excess (or runoff) behavior of the interior drainage sub-basins in the HEC-HMS model. The runoff curve numbers (CN) relate total accumulated excess to total accumulated precipitation and are based on factors such as hydrologic soil group, land use, ground cover, quality of vegetative cover, and antecedent moisture conditions.

64. NRCS soils maps were not available for the NYC borough of Staten Island/New York State County of Richmond. Therefore the hydrologic soil groups of the soils within the study area were estimated to be 50 % Group C and 50 % Group B based on a field survey of the project area in 1995.

65. The land use was determined by the 1990 land use and zoning maps, developed for the City of New York, City Planning Commission – Department of City Planning for Staten Island. Zoning categories were condensed into four land use categories (See Table 3).

66. The June 1995 recon study used planimetry to determine a fraction for each land use category. The resulting land use fractions and soil group fractions were used to compute an area-average Curve Number (CN) value for each interior sub-basin. The 1995 field survey was determined to be an acceptable means for inferring land use conditions as there have been only minor changes in development within the study area since the 1995 field survey.

67. The interim feasibility study took the 1995 interior sub-basin values and attributed those CN values to the corresponding sub-basins. Because the Interim Feasibility Study had smaller sub-basin areas, CN values were assumed to be equivalent to the associated, larger 1995 recon study sub-basin areas. The values of CN used for the interim feasibility study appear in Table 3.

TABLE 5 NRCS RUNOFF CURVE NUMBERS

Land Use	Approximate Lot or Unit Size	NRCS Runoff Curve No. CN Based on 50% soil group B, 50% C
R1 Single family residential	1/4 acre	79
R2/R3 multi-family residential	1/6 acre	83
Commercial and business	N/A	93
Open space	N/A	74

2.9.3 Time of Concentration

68. The longest hydraulic path for each sub-basin was identified using the 500 scale mapping. The travel times of surface runoff along the longest hydraulic paths were then computed incrementally between the 2 foot contour lines. This was done by first computing the slope between each 2 foot contour (if a path appeared to have a constant slope between multiple contour lines, it was computed as one whole segment), then the velocity was identified by using charts with velocity versus slope plots for one of two types of local drainage path scenarios: primary residential street or parabolic natural channel. After computing the velocity, the travel times were computed by dividing each incremental length by each incremental velocity, which were then summed together over the entire path to find a total travel time. The total travel time was then taken to be the time of concentration (Tc) for the associated interior drainage sub-basin.

2.9.4 NRCS Dimensionless Unit Hydrograph

69. The NRCS dimensionless unit hydrograph is based on a dimensionless table of discharge per unit area versus time, normalized to the peak discharge and time of concentration respectively. The actual sub-basin unit hydrograph is created within HEC-HMS when supplemented with a specific drainage area and a lag time. The lag time is the time from the center of mass of excess rainfall to the time of the peak discharge of the unit hydrograph.

70. For the interim feasibility study, the lag time was taken to be 0.6 times the time of concentration (Tc) as recommended by the NRCS (see values in Table 4). The specified duration of unit rainfall excess was taken to be five minutes, based on the sub-basin lag times and the recommendation to have at least three or four ordinates on the rising limbs of as many of the sub-basin unit hydrographs as reasonably possible. Alternatively sub-basin lag times were estimated from an NRCS empirical equation utilizing the longest hydraulic path length, its slope, and the local CN. These secondary values were found to be larger than those computed from the primary Tc computation. The smaller sub-basin lag times computed from Tc were selected to be used in the HEC-HMS model because they produced a greater and more defined peak discharge per unit drainage area. Additionally, the lag times computed by the Tc result in design peak flows that more closely match NYCDEP storm sewer design criteria.

TABLE 6 HEC-HMS MODEL SUB-BASIN DATA

Drainage Area	Sub-Basin (HEC-HMS Hydrologic Element)	Drainage Area Square Miles	Runoff Curve No. CN	Longest Length, Feet	Average Slope of Longest Length	NRCS Unit Hydrograph Lag, Minutes
E	SE	0.87	83	6230	0.021	21.72
D	SDLE	0.40	80	7530	0.015	26.52
	SDUW	0.25	80	5870	0.061	10.02
	SDLWD	0.19	80	3580	0.019	19.90
	SDUE	0.08	80	2230	0.075	5.88
	SDLW	0.61	80	5900	0.017	28.38
Total D	DTOTAL	1.53	80	-	-	-
C	SCUE3	0.09	82	3180	0.086	5.52
	SCUE2	0.10	82	5020	0.052	10.84
	SCLBSV	0.26	82	3380	0.021	12.78
Sub-total	CCLBSV	0.45	82	-	-	-
C	SCUE1	0.12	82	1320	0.102	2.04
	SCLE	0.42	82	5850	0.000684	88.27
	SCLW	0.87	82	7150	0.000839	91.02
Sub-total	CPOND	1.41	82	-	-	-
C	CMRVBK	0.71	75	10,520	0.033	23.40
	CUW	0.09	75	3580	0.055	7.56
	SCBRGY	0.60	82	3750	0.0092	21.60
Sub-total	CCBRGY	1.40	78	-	-	-
Total C	-	3.26	80	-	-	-
B	SBUW	0.19	83	3860	0.017	22.70
	SBUE	0.62	83	6650	0.02	24.24
	SBL	0.54	84	4780	0.0029	38.29
	SAEAST	0.39	71	2910	0.0062	17.26
Total B	BTOTAL	1.74	81	-	-	-
A	DRAINAGE AREA A	0.46	71	8440	0.0083	36.12
Total A	ATOTAL	0.46	71	-	-	-
Total A through E	-	7.86	-	-	-	-

Notes:

1. CN shown for Hydrologic Element's that are a combination of two or more individual sub-basins are area-averaged values.
2. HEC-HMS input variable Hydrologic Element is a hydrograph label or identification.

2.9.5 Routing Reach Travel Time

71. For some sub-basin reaches, it was appropriate to calculate routing reach travel times with the aforementioned velocity vs. slope chart plots. "Dummy" Modified Puls storage-outflow routing data equivalent to these travel times was then input to HEC-HMS for the

routing reach. By definition, the reach storage divided by its corresponding outflow is the travel time through the reach, at that outflow. Storage in acre-ft. divided by outflow in cfs, multiplied by 12.1, gives the reach travel time in hours, accounting for unit conversion. Modified Puls routing was used to allow reach storage to have the maximum effect of hydrograph peak inflow attenuation that would result from interior flood runoff spreading out over the sidewalks, lawns and lots of residential streets, and over the floodplains of natural channels such as New Creek and Oakwood Creek. For other reaches, it was appropriate to enter the reach length, slope, estimated channel and overbank Manning "n" values, and scaled-off typical eight-point cross section into HEC-HMS via the normal depth routing option. HEC-HMS computes a table of storage-outflow-elevation, which is used to perform hydrograph routing. Values from the HEC-HMS routing reach data are summarized in Table 5. For the flattest, most spread-out, and most irregularly defined routing reaches, like those found along lower portion of New Creek in Area C (Midland Beach), the models did not include the rising flat pool storage that would be encountered by an incoming hydrograph accumulating behind the tentatively selected plan alignment.

2.10 Diversion, Retrieval, and Translation of Hydrographs to Simulate Flow Through Major Storm Sewers

72. Diversion, retrieval, and translation of hydrographs to simulate interior drainage runoff inflow into local major storm sewers and its continued flow through the storm sewers was modeled in HEC-HMS with diversion functions and translation routing. The routing through storm sewers was estimated to be pure translation because of their minimal storage capacity. Pure translation routing indicates no change in hydrograph shape, only a single time delay for the reach travel time along all its ordinates.

73. Most direct runoff hydrographs computed at the outlets of the interior sub-basins have the potential to split in two (potentially more) before they reach a storage area, pond or pump system. Part of the runoff may move towards the Bay through the existing storm sewer system through intakes or catch basins. Once the storm sewer system is charged to capacity, the remainder of the runoff will flow down the streets as open channel flow or overland flow.

TABLE 7 HEC-HMS MODEL ROUTING REACH DATA

Basin	Routing Reach Hydrologic Element	Routing And Path Type	From	To	Length in feet	Slope	Travel time in minutes	Channel "n"	Overbank "n"
D	RTSDUW	Normal depth, street	SDUW outlet	Buried seawall/armored levee	5120	0.014	17.5	0.015	0.134
D	RTSDUE	Normal depth, street	SDUE outlet	Buried seawall/armored levee	4744	0.013	15.0	0.015	0.146
D	RTDVUE	Translation, storm sewer	SDUE outlet	Buried seawall/armored levee	4744	-	25.0	0.016	N/A
D	RTDLW	Normal depth, natural channel	Basin D comb. pt. 3	Buried seawall/armored levee	3420	6.4 10000	35.0	0.035	0.08
C	RCUE31	Normal depth, street	SCUE3 outlet	Lower East C Pt. 1	3300	.0182	6.3	0.015	0.14
C	RCUE21	Normal depth, street	SCUE2 outlet	Mason & Seaview Aves.	2370	.0236	4.7	0.015	0.14
C	RTLBSV	Translation, Storm sewer	Richmond Rd. & Seaview Ave.	Buried seawall/armored levee	7000	-	20.0	0.016	N/A
C	RTCD1	Translation, Storm sewer	Mason Ave.	Buried seawall/armored levee	3500	-	10.0	0.016	N/A
C	RTCUE1	Normal depth,street	SCUE1 outlet	Hylan Blvd. & Seaver Ave.	4020	.0134	25.0	0.015	0.14
C	RTCULE	Normal depth, New Creek	Hylan Blvd. & Seaver Ave.	Buried seawall/armored levee	5850	.0007	75.0	0.035	0.08
C	RTMVBK	Modified Puls, street	Moravian Brook watershed outlet	North Railroad Ave. & Otis St.	1000	7.53 1000	8.0	N/A	N/A
C	RTCUW	Modified Puls, street	North RR Ave & Otis St.	North RR Ave, Bryant	300	1.74 1000	5.0	N/A	N/A
C	RTCUW2	Normal depth,street	North RR Ave, Bryant	Hylan Blvd. & Greeley Ave.	2705	0.011	None apparent	0.015	0.14
C	RTGRLY	Translation, storm sewer	Hylan Blvd.& Greeley Ave.	Buried seawall/armored levee	4250	-	25.0	0.016	-

TABLE 7 HEC-HMS MODEL ROUTING REACH DATA (CONTINUED)

Basin	Routing Reach Hydrologic Element	Routing And Path Type	From	To	Length in feet	Slope	Travel time in minutes	Channel "n"	Overbank "n"
B	RTSBUW	Modified Puls, street	SBUW Outlet	Ebbitts St. and Hylan Blvd.	1500	.0434	5.0	-	-
B	RTCB1	Normal depth, street	Ebbitts St. and Hylan Blvd.	Buried seawall/armored levee	4120	.0039	20.0	0.015	0.14
B	RTYSNS	Translation, storm sewer	Hylan Blvd. and Tysens Lane	Tysens Lane at Buried seawall/armored levee	4125	-	15.0	0.016	-
B	RTEBTS	Translation, storm sewer	Hylan Blvd. and Tysens Lane	Ebbitts St. at Buried seawall/armored levee	4650	-	10.0	0.016	-
B	RTROND	Translation, storm sewer	Hylan Blvd. and Tysens Lane	New Dorp Lane and Buried seawall/armored levee	5450	-	20.0	0.016	-

Notes:

1. For storm sewer translation routing, the slope is not given, because capacity in cfs was identified on the NYC storm sewer maps, and then divided by a cross-sectional area computed from dimensions to find the average velocity. Overbank "n" values are not given for these routings because storm sewers are self-contained and have no overbanks.
2. Muskingum routing reach RTCRLW is the same as Modified Puls routing because the weighting factor X was set to zero. This means that outflow is a single-valued function of reach storage alone and the slope of the storage-outflow relation is equal to the reach travel time of 13.6 minutes.

74. Translation routing reach travel times for the major storm sewers were found by computing their capacities using the Manning formula with roughness "n" = 0.016, as recommended by NYCDEP standard practice for rough concrete, and with the cross-section and slope, as measured from the NYC storm sewer maps, assuming six inch clearance from inside free water surface to crown (also as recommended by NYCDEP). Average velocity was found by dividing capacity by the cross-sectional area of flow read from the storm sewer maps. Appropriate dimensional conversions were made prior to calculating the average velocity. Reach travel times were computed as length divided by velocity and rounded to the nearest five minute mark in order to input them as an integer number of translation routing steps into the HEC-HMS models.

75. The diversion functions in the HEC-HMS model input file are also based on the storm sewer capacities computed as described above. Diversion functions assign labels to the diverted and residual hydrographs and pair inflow with diverted flow. Zero inflow is paired with zero

diverted flow. Diverted flow equals inflow up to, and including, storm sewer capacity. Diverted flow then remains constant at this maximum value of storm sewer capacity. No matter how high inflow becomes (10,000 cfs was evaluated in HEC-HMS to cover all size floods up to and including the 500 year flood) the diverted flow remains the same and the remaining residual flow is then routed downstream as open channel flow.

76. The general trend of the diversions was toward the southwest, from basin E to basin D, D to C, C to B, and B to A. The schematics for the inter-basin diversions are shown on Figure 5 through Figure 11. Major storm sewer diversion data is given in Table 6. Uniquely for the last three diversions of Area B, the capacity, or diverted flow, was input as a ratio of the total capacity because the diversions had to account for the chambers and manholes with two storm sewer exits on the downstream ends.

2.11 Interception and Early Exit of Interior Inflow via Major Storm Sewers in Interior Area C (Midland Beach)

77. Preliminary studies of the runoff for Interior Area C had its entire contributing drainage area (3.26 square miles) as flowing into the downstream ponding area and wetlands. As a result significant pond excavation volume, a prohibitively expensive pump, or a combination of both theoretically would be necessary to achieve the Minimum Facility concept for Area C. It evolved that an outlier such as this may be the result of an incorrect assumption for one of the initial hydraulic parameters. To test for sensitivity, the largest 500 year peak flows within interior Area C were compared with the capacities of the storm sewer outfalls at the edges of the lower part of Area C, bounded by Seaview Avenue at the northeast edge and Greeley Avenue at the southwest edge. The 500 year peak flows and capacities were found to be comparable. It followed that the most rapidly occurring runoff from the steep upper parts of Drainage Area C may escape through the major storm sewer outfalls under Seaview and Greeley/Midland Avenues, into the bay, without ever overflowing down the streets and into the local pond and wetlands. To determine the potential for this phenomenon, the HEC-HMS model of interior inflow was modified to compute inflow hydrographs to the Seaview and Greeley Avenue/Midland Avenue storm sewer outfalls separately from the direct inflow to the Area C pond. These inflow hydrographs were input to a computation spreadsheet to analyze a flow calculation based on headwater, tailwater (accounted for time-varying marigrams), the resulting driving head difference, and the Manning roughness “n”, length, slope and dimensions of the storm sewer outfalls. The output was a separation of each inflow hydrograph into the part that escaped into the Atlantic Ocean through the storm sewer outfall, and the remainder that overflowed into the Area C pond. The recreated overflow hydrographs were combined with the HEC-HMS computed hydrographs of direct inflow to the Area C pond to form the total inflow to the Area C pond for its interior drainage flood routing.

TABLE 8 HEC-HMS MODEL DIVERSION DATA

Hydrologic Element	Capacity, cfs (diverted flow)	Major Basin	From	To	Via
DVSDLE	682	D	Sub-basin SDLE outlet	Buried seawall/armored levee at D	Junction of Quintard + Raritan storm sewers
DVSDUW	203.4	D	Sub-basin SDUW outlet	Basin C	Richmond Road storm sewer at Delaware Ave.
DVCD1	376.4	D	Sub-basin DLWD outlet	Basin C	Mason Ave. storm sewer at Alter Ave.
DVSDUE	222.4	D	Sub-basin SDUE outlet	Buried seawall/armored levee at D	Bergher and Raritan Ave. storm sewer
DVCUE3	200.9	C	Sub-basin SCUE3 outlet	Liberty & Seaview Aves. storm sewer	Richmond Road storm sewer
DVCUE2	281.4	C	Sub-basin SCUE2 outlet	Liberty & Seaview Aves. storm sewer	Richmond Road storm sewer
DVMVBK	381.4	C	Sub-basin CMRVBK outlet	Greeley Avenue storm sewer	Bryant Avenue storm sewer
DVCUW	174.3	C	Sub-basin CUW outlet	Greeley Avenue storm sewer	Bryant Avenue storm sewer
DV1BUW	290.4	B	Sub-basin SBUW outlet	Various storm sewers	Hylan Blvd. & Tysens Lane storm sewer Jct. chamber
DVSBUE	337.4	B	Sub-basin SBUE outlet	Tysens Lane storm sewer	Hylan Blvd. storm sewer
DV2BUE	487.5 / 1143.0	B	Sub-basin SBUE outlet	Ebbitts St. & Rose Ave. storm sewers	Hylan Blvd. storm sewer
DV2BUW	104.8 / 290.4	B	Hylan Blvd. & Ebbitts St.	Basin A upper	Hylan Blvd. storm sewer

Hydrologic Element	Capacity, cfs (diverted flow)	Major Basin	From	To	Via
DV3BUE	285.5 / 487.5	B	Hylan Blvd. & Ebbitts St.	Rose Ave. & New Dorp Lane	Hylan Blvd. storm sewer

Notes:

1. For the first ten diversions in the table, the figure in the "capacity" column is the diverted flow. All inflow up to and including this value of "capacity" is diverted. For inflows above this value, the diverted flow remains constant at "capacity".
2. For the last three diversions in the table, the bottom figure is the inflow and the top figure is the corresponding diverted flow. All diverted flows are ratioed from inflows using these values.

2.12 Inflow Hydrographs

78. Inflow hydrographs for drainage areas A, B, D, and E are shown on Figure 12 through Figure 16 for studied range of frequencies. For drainage area C, Figures 17 to 20 show the amount of surface runoff that exceeds the capacity of the storm sewer lines and that is conveyed by natural terrain to the interior pond in Drainage Area C. Figure 21, represents the inflow hydrographs for the case where the interior pond elevation in Drainage Area D rises above the dividing elevation that connects Drainage Area D and E. This only occurs when the exterior tide condition is above the 2-year exterior condition. The peak inflow of the discrete sub-basins is summarized in Table 7; its corresponding discharge vs. frequency curves shown in Figures 22 to 28.

TABLE 9 INTERIOR DRAINAGE AREAS PEAK INFLOWS IN CFS

Frequency in years	Interior Drainage Areas Peak Inflows (cfs)						
	ETOTAL: Area E (1.27 sq. mi.)	DTOTAL: Area D (1.12 sq. mi.)	CCLBSV: Area C Seaview Ave. storm sewer inflow (0.45 sq. mi.)	CPOND: Area C: Pond Naughton Avenue storm sewer (1.41 sq. mi.)	CCBRGY: Area C Bryant and Greeley Aves. storm sewer (1.40 sq. mi.)	APLUSB: Area B (1.75 sq. mi.)	SAWTTL: Area A (0.46 sq. mi.)
1Year	540	280	570	340	490	750	130
2 Year	720	420	750	480	710	1030	200
5 Year	1050	660	1080	710	1280	1450	340
10 Year	1260	820	1250	870	1620	1770	440
25 Year	1540	1050	1390	1070	2010	2160	560
50 Year	1770	1240	1520	1220	2350	2480	660
100 Year	2120	1430	1670	1380	2690	2850	770
500 Year	2960	1770	2100	1760	3520	3800	1000

3 DESIGN PROCEDURE

79. As described in EM 1110-2-1413, procedures for formulating and evaluating flood loss reduction measures for interior drainage areas are similar to planning procedures used in other types of investigations. The complexity of the process is dependent upon the nature of the study area, flood hazard, damage potential, and environmental and social factors. A comprehensive array of alternatives is formulated and evaluated through an iterative process until a final array of plans is developed. Data necessary to conduct the investigation includes basin hydrology, stage-frequency curves, hydraulic parameters of plan components, the annualized cost of construction and maintenance, and estimated residual damages. Using this data, with and without project benefits can be determined in order to identify the plan which maximizes NED benefits.

3.1 Interior Flood Control Simulation Models

80. Two mathematical models were used to simulate the hydrologic response of the interior drainage areas and the operation of the interior drainage facilities. The first model, developed by the Corps' Hydrologic Engineering Center (HEC), is the Hydrologic Modeling System (HEC-HMS).

81. The program HEC-HMS has some limitations in the modeling of existing storm sewer systems and natural flood storage area. It may therefore underestimate outflow through the Tentatively Selected Coastal Storm Risk Management System and thereby overestimate interior water surface elevations by failing to account for runoff that may never enter and accumulate in the natural flood storage area. This would be the case when runoff passes directly into the Bay when the head difference between tailwater elevation and the ground elevation behind the plan alignment is greater than zero feet. In these cases, a separate time-series pressure flow spreadsheet utility program, Excel@Model, was used to correct this discharge relationship.

3.1.1 HEC-HMS Model

82. HEC-HMS version 3.5 is a computer program designed to both compute runoff and to route floods through interior drainage facilities to adjacent rivers, estuaries or oceans accounting for variable tailwater conditions. This program was utilized to simulate the surface runoff response of the interior basins to precipitation while taking into account both the hydrologic and hydraulic components of these basins.

3.1.2 Excel@Model

83. An Excel spreadsheet was programmed to evaluate the maximum amount of flow that can exit from the interior drainage system. This spreadsheet was only used for Midland Beach (Drainage Area C.)

Overview

84. In the Excel spreadsheet, outflow is mainly from gravity outlets. The downstream storm surge elevation versus time relationship is selected by the user. After defining the upstream and downstream conditions, outflow over time is computed by defined formulas for gravity and pressure flows. These principles are from culvert analysis "Type 4" condition for pressure flow and the typical Manning's equation for gravity flow. For gravity and pressure outflow, only positive flow is allowed.

Calculations

85. *Gravity Outflow.* The model incorporated a Manning's equation to model flow through the proposed gravity outlets:

$$Q = [1.486/n]*A*R_h^{2/3}*S^{1/2}$$

86. where Q equals the discharge through the outlet under gravity flow, n is the resistance coefficient for Manning's n, A is the cross-section area of the flow in the box or circular culvert, R_h is the hydraulic radius, and S is the slope of the box or circular culvert at the upstream and downstream ends. The assumption with the parameter R_h is that the entire box or culvert covert is covered by water.

87. *Pressure Outflows.* The model incorporated a culvert "Type 4" equation to model flow through the proposed gravity outlets:

$$Q = C_D*A*[\{2*g*(h_1-h_4)\}/\{1+[(29*n^2*L)/(R_h^{4/3})]\}]^{1/2}$$

88. where Q equals the discharge through the outlet under pressure flow, n is the resistance coefficient for Manning's n, A is the cross-section area of the flow in the box or circular culvert, C_D is Coefficient of Discharge, g is the gravity constant (32.2 feet per second squared), R_h is the hydraulic radius, h_1 is upstream elevation of the water surface (which is to be estimated to be the grate/ground elevation of the upstream elevation of the culvert), and h_4 is the downstream elevation of the water surface (which is the tailwater/tide elevation). The assumption with the parameter R_h is that the entire box or culvert covert is covered by water.

89. In both outflow conditions, flow occurs where the head difference is greater than zero because the outlets will be installed with a backflow valve, preventing negative flow conditions in the culvert.

4 INTERIOR DRAINAGE HYDRAULICS

90. In addition to the development of hydrologic data, the analysis of interior drainage facilities required additional input to describe the physical and operational characteristics of the Minimum Facility and other alternatives. Input requirements consisted of potential storage volumes, and diversion and pumping rates. HEC-HMS was utilized to evaluate the effects of existing or proposed hydraulic structures by routing interior fluvial flood events through the line-of-protection. The assumptions and criteria used to inform the models are described below.

4.1 Minimum Head

91. The minimum head to open the flap valves for gravity outlet operation through the levees and floodwalls were estimated to be 0.25 feet. The minimum head to open the flap valves for the box culverts along South Shore of Staten Island was estimated to be 0.25 feet.

4.2 Elevation/Storage Relationships

92. In order to evaluate the storage capacity at the line-of-protection, elevation-storage relationships were developed. Using project mapping and commencing with the lowest elevation at the natural ponding site behind the line-of-protection, the planimetric area enveloped by a particular elevation was computed. For consecutive elevations, the average end-area method was used to compute the volume. The volumes between elevations were summed to generate an overall elevation-volume relationship for a particular ponding site. Figure 29 presents the storage area relationships in graphical form.

4.3 Potential Hydraulic Measures for Interior Drainage Facilities

93. Potential hydraulic measures for interior drainage flood protection are described briefly below. No single hydraulic measure is effective in all situations and typically no single hydraulic measure is effective by itself. The most cost effective approach to reducing interior flooding stages is likely to be a combination of hydraulic measures.

4.3.1 Gravity Outlets

94. The driving head of runoff outflow from the protected areas is the elevation difference between two water surfaces; the elevation of runoff that is accumulated landward of the plan alignment (headwater) and the elevation of the surge seaward of the plan alignment (tailwater).

95. There is no modeled backflow from the bay into natural flood storage areas because tide gates, which permit flow in only one direction, are assumed to be in place for the Minimum Facility as well as all interior drainage alternatives. The program HEC-HMS would assume zero flow when tide level is higher than the interior headwater level.

96. Gravity outlets, typically the least expensive drainage measure, function best during the high rainfall coupled with low tide events, when there is sufficient head for gravity discharge. Gravity outlets also work well when the existing grade landward of the plan alignment is higher, again providing additional head. Conversely, gravity outlets are ineffective during high tide events when the tailwater elevations are higher than the interior elevations. During these events, outlets are effectively blocked and thus the gravity discharge is zero. Gravity outlets do not function well with large, low-lying natural flood storage areas such as freshwater wetlands, where even a moderate tide can prevent gravity discharge.

4.3.2 Ponding

97. Ponding can be an effective means for flood risk management. Runoff is stored in low-lying, non-damaging areas until the tailwater (tidal surge) drops sufficiently to permit gravity discharge. Ponding is most effective when runoff is first discharged through gravity outlets during low tailwater conditions, and then diverted into the pond as the gravity outlets become blocked. Directing all runoff into a pond will increase the size of the pond required. Excavating ponds to increase the runoff storage volume can be expensive, so natural flood storage areas should be used wherever possible, especially where development has already occurred or is expected to occur in floodplains.

4.3.3 Pressure Outlets

98. If a significant portion of the drainage area is higher than the crest of the coastal storm risk management plan structure, it may be possible to divert the runoff from that higher area directly into the bay through pressure conduits. Typically, there must be sufficient head between the higher ground and the maximum tailwater to divert this runoff. Diversion effectively reduces the volume of runoff reaching the structure that would otherwise need to be handled by other means such as ponding or pumping. Pressurizing an existing gravity line by removing or sealing all of the lower catch basins is usually the least costly method but in some cases construction of a new pressure line is justified.

4.3.4 Pumping

99. Pumping is usually the most costly option in initial construction as well as operation and maintenance, and therefore is typically considered the “last resort”. Today’s submersible pumps, however, are much less costly (including operation and maintenance costs) than the old style pump stations that were part of the 1970’s plan of protection for the South Shore of Staten Island. Similar to pond excavation and pressure outlets, pumping is most effective during higher exterior stages when gravity outlets are blocked and there is insufficient natural flood storage area landward of the plan alignment. Pumping can be used to reduce the volume of a ponding area, or it can be used to handle the peak runoff. In general, the costs of pumping are additional to the Minimum Facility or alternative costs. The construction of a pump station creates additional capital costs and also increases annual maintenance and operation costs. Capital expenditures affected by the addition of pump stations include mechanical equipment, associated housing and any new outfalls. Increases in the cost of project operation and maintenance include power consumption, equipment operation, inspection and testing, maintenance and replacement.

100. Pumps typically have a minimum cycle time of about six starts per hour. To achieve this cycle time an adequate volume of surface runoff from the interior drainage area must be stored and available whenever the pumping operation is initiated. The storage volume in cubic feet required between the lead pump-on and pump-off elevations is based on the following equation:

$$V = [T \text{ x } Q_{pump}]/4$$

101. Where V is the volume in cubic feet, and Q_{pump} is the pump discharge rate in cubic feet per second, and T is the cycle time in seconds.

4.3.5 Interior Levees

102. In large low-lying ponding areas (natural flood storage areas) where further lowering of the interior water levels is not cost justified, interior levees may be used to provide additional flood risk management. Interior levees separate the vulnerable developed areas from the stored runoff in the natural flood storage area. These levee heights are typically low, because the maximum water surface elevation in the natural flood storage area is much lower than the exterior tide levels. However, these interior levees may not be feasible where there is a large drainage area landward of the levee and the potential for interior flood damages still exist.

4.4 Seepage Analyses

Seepage analyses were performed to estimate seepage quantity through and/or underneath the proposed structure, exit hydraulic gradients on the land upside and pore pressures within the embankments were conducted in accordance with EM 1110-2-1413. Both transient and steady seepage analyses were performed for the buried seawall/armored levee and steady seepage analyses performed for the levee and floodwall.

The analysis was performed using the commercially available finite element method (FEM) software program SEEP/WÓ and shows the total seepage through the Line of Protection to be approximately 135gpm (See Geotechnical Appendix for analysis results). Since this volume is relatively small it's not included in the interior drainage analysis.

5 ECONOMIC ASSESSMENT

5.1 Conditions

103. Analysis of benefits and costs for formulation of interior drainage plans is conducted using an interest rate of 3.375% applied over a 50 year period-of-analysis. Baseline conditions consider the current sea level and future conditions consider a 0.7 foot rise in sea level and storm surge elevations.

5.2 Costs

104. Interior drainage consists of Minimum Facility features required to maintain existing drainage and avoid induced flood-damage, and various interior drainage improvements that must be economically justified based on a comparison of benefits (reduction of Minimum Facility damages) and costs (annual cost above Minimum Facility costs). These costs consist of first construction costs, real estate costs, and annual operation and maintenance expenses. Interior drainage facility costs are based on incremental improvements and are additional to Minimum Facility features, which are considered part of the Tentatively Selected LOP Plan.

5.2.1 First Construction Costs

105. First construction costs for interior drainage facilities may include primary and secondary outlets, intake structures and outlet gates, pond excavation, pump stations and new outfalls.

5.2.2 Real Estate Costs

106. Real estate acquisitions associated with interior drainage facilities are based on the purchase of restrictive easements where natural storage must be maintained and permanent or intermittent flowage easements where interior features (drainage ditch, ponds, etc.) are planned and would increase the depth, duration or frequency of flooding from interior runoff. Specific areas requiring flowage easements are associated with excavation of flood storage ponds. Table 10 provides a summary of how flood depths in these areas will change during rainfall events that are coincident with a small tidal surge (2 year).

TABLE 10 POND FLOOD DEPTH COMPARISON

Most Likely Conditions (2 -year Exterior Storm) Change in Flood Depths at Excavated Ponds											
			2 yr Storm Flood Depth		10 yr Storm Flood Depth		50 yr Storm Flood Depth		100 yr Storm Flood Depth		Flood Depth
Interior Area	Pond Name	Pond Area (ac)	Existing	With Project	Existing	With Project	Existing	With Project	Existing	With Project	Maximum Increase
B	East	45.85	0.0	1.9	0.5	2.8	1.0	3.3	1.2	3.5	2.3
C	Pond 1	15.69	0.2	0.5	1.4	1.3	2.3	2.2	2.4	2.5	0.3
	Pond 2	12.01	0.2	0.5	1.4	1.3	2.3	2.2	2.4	2.5	0.3
	Pond 3	16.39	0.2	0.5	1.4	1.3	2.3	2.2	2.4	2.5	0.3
	Pond 4	20.46	0.2	0.5	1.4	1.3	2.3	2.2	2.4	2.5	0.3
	Pond 7	12.08	0.0	0.5	0.4	1.3	1.3	2.2	1.4	2.5	1.2
	Last Chance	18.14	0.0	0.5	0.0	1.3	0.3	2.2	0.4	2.5	2.2
	Midland Pond	5.74	0.0	0.5	0.0	1.3	0.0	2.2	0.0	2.5	2.5
E	Pond 1	15.64	0	1.1	0.6	2.5	1.16	3.4	1.4	3.8	2.4
	Pond 2	18.7	0.99	1.1	1.6	2.5	2.16	3.4	2.4	3.8	1.4

5.2.3 Operation and Maintenance

107. Annual costs attributed to the operation and maintenance of interior drainage facilities consist of, but are not limited to, labor charges for the inspection, care and cleaning of pond areas, outlets and pump stations, as well as anticipated energy charges and annualized replacement costs.

5.3 Benefits

108. Flood damage reduction benefits for interior drainage facilities are calculated as the difference between the Minimum Facility damages and the residual damages associated with the project and the interior drainage alternative being evaluated.

5.3.1 Interior Flood Damage

109. As described in the Interim Benefits Appendix, the expected damage to each structure was calculated for the required range of flooding depths. These damages were then aggregated to determine composite stage vs. damage relationships for each interior area.

5.3.2 Annual Damage

110. Annual damage was calculated using a risk based simulation technique, and the stage frequency and discharge frequency relationships calculated in HEC-HMS were input into HEC-FDA. The HEC-FDA model calculates the Average Annual Damages (AAD) for both the base and future conditions (with sea level change). Equivalent Annual Damages (EAD) for the 50 year Period of Analysis was also calculated.

5.3.3 Minimum Facility Damages

111. As noted above, the Minimum Facility becomes the starting point for evaluating interior drainage alternatives. The magnitude of these damages helps to guide decisions on the type and scale of interior flood risk management measures to consider. Table 11 provides a summary of the Minimum Facility AAD and EAD for each of the interior areas. The majority of the interior damages occur in Area C and Area E.

TABLE 11 MINIMUM FACILITY DAMAGES

Interior Drainage Area	Expected Annual Damage		Equivalent Annual Damage*
	Base Year	Future Year	
A	$77,800	$97,900	$85,000
B	$96,600	$136,000	$110,700
C	$5,178,700	$6,421,100	$5,623,100
D	$116,300	$175,500	$137,500
E	$2,107,200	$2,377,600	$2,204,000

*3.375% Discount Rate

6 FUTURE WITHOUT PROJECT DRAINAGE CONDITION

112. The extent of interior flooding along the project area may be impacted by: increasing sea levels, which reduce the ability of gravity outfalls to drain low-lying areas; fill and development of low lying areas, which store floodwaters until tides recede; construction of additional outfalls; and upland development that could increase runoff rates or volume. Sea level change has been evaluated by changing the exterior flood elevations. Prior to the economic downturn in 2008, there was rapid development of any vacant land between Hylan Boulevard and the coast. Much of this development occurred in low-lying areas, reducing available flood storage volume. A resumption of such development is anticipated as described in subsequent sections. Hydraulic sensitivity analysis indicates that the construction of new outfall identified in the New York City drainage plans will have minimal impact on peak interior water levels. Because these structures are not part of capital budgets they are excluded from the without project analysis. A review of development trends indicated that increased upland development would have little impact on interior flood conditions.

113. The following sections provide a summary of the future development and flood storage conditions used for the interior drainage analysis.

6.1 Drainage Area A

114. The available natural flood storage for this drainage area mostly encompasses the existing freshwater wetlands. Freshwater wetlands can provide opportunities for ecosystem restoration; however, it is assumed that some of the freshwater wetlands and/or adjacent areas will be developed in the near future and would not provide effective flood storage.

6.1.1 Development of Vacant Property

115. Given that some of the current natural flood storage areas (freshwater wetlands) for Area A are zoned for residential development, and that construction of homes in other nearby freshwater wetlands areas and adjacent areas was recently observed along the South Shore of Staten Island between Fort Wadsworth and Oakwood Beach, it is evident that the project area is under development pressure. Based on historic trends, it can be anticipated that development will spread outwards along the fringes of existing development. As such, it is assumed that 24% (9.3 acres) of the currently available natural flood storage area, primarily along the fringes, will be developed as part of the near future condition. It is also assumed that the 41% (16.1 acres) of current natural flood storage area owned by New York City Department of Parks and Recreation (DPR, or Parks) will be preserved for present and future conditions. The remaining 35% (13.8 acres) of privately owned natural flood storage area is expected to see full development. The drainage analysis was conducted assuming both the undeveloped fringe area and the remaining developable land were raised to legal grade (estimated to be at +7 feet NGVD 1929).

116. Following Sandy (October 29-30, 2012), the State of New York is proceeding with the acquisition and demolition of 349 properties located in drainage areas A and B.

6.2 Drainage Area B

117. The existing natural flood storage for Drainage Area B mostly encompasses the existing freshwater wetlands. It is assumed that some of the freshwater wetlands and/or adjacent areas will be developed in the near future and would not provide effective flood storage.

6.2.1 Development of Vacant Property

118. Portions of the currently natural flood storage areas (freshwater wetlands) for Area B are zoned for residential development and construction of homes in some nearby freshwater wetlands areas suggests that future development of some properties is likely. Based on historic trends, it can be expected that development will occur first along the fringes of existing development. It is assumed that 6% (7.7 acres) of the currently available natural flood storage area for Area B, primarily along the fringes, will be developed as part of the near future condition. It is also assumed that the 67% (92 acres) of currently available natural flood storage area owned by NYC Parks will be preserved for present and future conditions. The remaining 27% (36 acres) of privately owned natural flood storage area, also known as "Traube" property, has a high probably of being developed, if the coastal storm risk management project is in place." At this time, the owners of this parcel have been approached on numerous occasions by NYCDEP to negotiate a sale as part of the Bluebelt program and have consistently rejected any offers to sell.

6.3 Drainage Area C

119. NYCDEP has proposed to acquire approximately 109 acres (approximately 85%) of the currently available natural flood storage area, mostly encompassing freshwater wetlands along New Creek as well as the area known as Last Chance Pond to preclude future building of homes and provide natural storage.

6.3.1 Development of Vacant Property

120. Based on historic trends, it can be anticipated that development will spread outwards along the fringes of existing development. As such, it's assumed that approximately 15% (19 acres) of natural flood storage area along New Creek will be developed as part of the near future condition.

6.4 Drainage Area D

121. Apart from the Staten Island Hospital and South Beach Psychiatric Center the drainage area is relatively undeveloped.

6.4.1 Development of Vacant Property

122. Because the remaining vacant land is owned by the DPR, it does not need to be acquired, but approximately 30 acres will need to be preserved by DPR for present and future conditions.

6.5 Drainage Area E

123. NYC DEP has proposed to acquire approximately 34 acres (approximately 63%) of the currently available natural flood storage area to preclude future building of homes and provide natural storage. Freshwater wetlands can provide opportunities for ecosystem restoration; however, it is assumed that some of the freshwater wetlands and/or adjacent areas will be developed in the near future and would not provide effective flood storage.

6.5.1 Development of Vacant Property

124. Based on current zoning, nearby construction, and historical trends, it can be expected that development will occur along the fringes of existing development for the remaining 20 acres (approximately 37%) of natural flood storage areas in Area E.

7 PLAN FORMULATION

7.1 Minimum Facility Concept

125. As stated in U.S. Army Corps of Engineers EM 1110-2-1413, "Hydrologic Analysis of Interior Areas", the design Minimum Facility should provide interior flood relief such that during low exterior stages (at gravity conditions for normal astronomic tide) the local storm drainage system (typical 10-year design storm) functions essentially as it would without the Coastal Storm Risk Management System in place.

126. The Minimum Facility Plans are impacted by two physical changes to the existing hydraulic landscape. First, the available natural storage has been reduced in some Drainage Areas, especially Area B, because of the tentatively selected plan alignment, which is landward of the existing coastal barrier. Figure 29 shows a stage-storage relationship for each Drainage Area with the proposed plan alignment in place. Second, with the variety in existing topographical features along the project reach, the Interior Drainage Areas have been categorized as one of two cases as depicted in Figure 30 and Figure 31.

127. <u>Case #1</u> (Figure 30) has an existing dune, berm or road acting as a barrier (e.g. Father Capodanno Blvd.), preventing runoff from reaching the bay. The runoff exceeding the capacity of the local storm drainage system becomes excess runoff and flows overland along streets to low-lying areas. In some cases, the existing drainage system discharges directly into these low-lying areas. The existing barriers prevent excess runoff from flowing overland to Lower New York Bay causing the excess runoff to accumulate landward of the dune or road. Most of the excess runoff will accumulate in an existing freshwater wetland area and will accumulate until the existing barrier is overtopped, at which time it will flow overland towards the bay. Interior

flooding from an accumulation of 10-year runoff during a normal tide condition would be caused by the existing barrier and the Tentatively Selected Design Alignment.

128. Case #2 applies to those near shore areas without an existing barrier in place (e.g. Drainage Area A). In the existing conditions the interior flooding elevation would be equal to the storm surge stillwater elevation, whereas in the proposed condition the surge and the excess interior runoff would be blocked by the plan alignment; therefore a Minimum Facility is provided to ensure that the interior water levels for improved conditions do not exceed the existing water levels.

129. The Minimum Facility is intended to ensure that the existing drainage system performs the same with and without the project put in place as to avoid induced flood damages. This is the starting point from which all additional interior drainage alternatives can be evaluated. Additional interior drainage facilities may be designed to further reduce interior water levels beyond the minimum facilities. These additional interior facilities must be incrementally justified.

7.2 National Economic Development (NED) for Interior Drainage Facilities

130. The benefits accrued from interior drainage alternatives are attributable to the reduction in the residual flood damages that may have remained under the Minimum Facility condition. Finally, an optimum drainage alternative is selected based on meeting NED objectives.

131. The interior drainage facilities must be formulated to maximize NED benefits while meeting NED objectives to provide a complete, effective, efficient, and acceptable plan of protection.

- Completeness is defined in Engineering Regulation (ER) 1105 2 100 as,
  a. The extent to which the alternative plans provide and account for all necessary investments or other actions to ensure the realization of the planning objectives, including actions by other Federal and non-Federal entities.
- Effectiveness is defined as,
  b. The extent to which the alternative plans contribute to achieve the planning objectives.
- Efficiency is defined as,
  c. The extent to which an alternative plan is the most cost effective means of achieving the objectives.
- Acceptability is defined as,
  d. The extent to which the alternative plans are acceptable in terms of applicable laws, regulations, and public policies.

7.3 Analysis of Alternative Plans

132. The Minimum Facility plan was the starting point from which alternative plans (herein called alternatives) were measured. The benefits accrued from alternatives are attributable to the reduction in the residual flood damages that would have remained under the Minimum Facility condition. For an alternative to be justified, it must be implementable and reasonably maximize benefits versus the additional cost required for its construction, operation and maintenance. Alternatives examined include the use of gravity outlets, pump stations/submersible pumps, and excavating ponding. No reasonable options for diversion of upland runoff were identified. The following is a general description of several alternatives that were considered during the development of interior drainage facilities.

133. From the Minimum Facility analysis, it was concluded that drainage areas A and D provided adequate drainage at least equal to that of the existing infrastructure. For Area A the alternative analysis was limited to an assessment of a plan developed by the local sponsor to provide additional excavated storage with environmental restoration.

134. For the hydraulic measures (structures) described in "Hydraulic Structures for Interior Drainage Facilities", no single hydraulic measure could significantly lower the Water Surface Elevation (WSEL) landward of the plan alignment. However, combinations of these hydraulic structures can accomplish this goal. Alternatives consisting of combinations of hydraulic measures are listed under Section 7.5 Formulation Results. Table 12 gives a list of Alternatives that were considered.

TABLE 12 INTERIOR DRAINAGE ALTERNATIVES

Drainage Basins	List of Alternatives
A	Minimum Facility
	DEC Conceptual Plan*
B	DEC Conceptual Plan*^
	DEC Conceptual Plan + Two Ponds
	Interior Levees/Non-structural
C	Minimum Facility
	1500 cfs Pump Station
	900 cfs Pump Station with Two Excavated Ponds
	Non-Structural
	DEP Bluebelt Plan (Midland Beach)*
	Seven Excavated Ponds
	Four Excavated Ponds
	Two Excavated Ponds
D	Minimum Facility
	Non-Structural
E	Minimum Facility
	DEP Bluebelt Plan (Midland Beach)*
	1800 cfs Pump Station
	Two Excavated Ponds
	600 cfs Pump Station with Two Excavated Ponds
	Non-Structural

* - Also known as "Sponsor Identified Plan"
^ - Is also defined as the "Minimum Facility"

7.4 Optimum Plan

135. The optimum plan is defined as the plan that maximizes the net benefits over cost. As outlined within the description of Minimum Facility, the planning and development of interior drainage measures is performed independently from the Tentatively Selected Design Alignment. Each interior drainage area is analyzed to determine the optimum alternative.

7.5 Interior Plan Formulation

136. The formulation of interior plans was an iterative process that considered a full range of measures for each drainage area. Only measures that are reasonably likely to meet the Minimum Facility or NED criteria discussed above were considered at any location. For

example, in areas with relatively low damage the construction of expensive pump stations or large excavated ponds were not considered. A number of plans were developed and dismissed prior to Hurricane Sandy (October 29-30, 2012) and were not updated in this interim document to reflect post-Sandy conditions. The viable options, however, were updated to reflect USACE post-Sandy guidance, post-Sandy buyouts/acquisitions, and the updated stage frequency curves from FEMA's forthcoming New York City coastal Flood Insurance Study (FIS).

7.5.1 Drainage Area A

7.5.1.1 Introduction

137. Drainage Area A as described in Section 7.1 Minimum Facility Concept falls under the category identified as Case #2 which applies to near shore areas without an existing barrier in place. Under existing conditions, the surface runoff is able to be conveyed along the West Branch of Oakwood Creek to Raritan Bay without restrictions. Once the proposed line of protection is constructed, restriction of flow will occur, because the tide gate structure opening (3 box culverts each 5'x5') will be the only means for surface (rainfall) runoff to flow through the line of protection which will lead to an increase in interior water levels. In addition, the tide gate structure opening will be closed under storm conditions, which will add to interior flood stages caused by the line of protection. The minimum facility that is needed to address the increase in water level and duration of flooding is provided in the following paragraphs.

7.5.1.2 Minimum Facility

138. The Minimum Facility for drainage Area A includes 17.19 acres of natural flood storage on property that is owned by DPR (see Figure 32). A restrictive easement will be required for this area. The Minimum Facility also includes a tide gate structure with three 5' X 5' sluice gates that allow Oakwood Creek to flow through the levee design. Tide Gates are designed to permit backflow at low (non-damaging) tidal elevations from Lower New York Bay, which allows intermixing of fresh and salt water to the area wetlands. This would allow for freshwater and saltwater habitats to co-exist in a dynamic system. In addition to the tide gate, two sluice gate structures will help drain the interior flooding for the Minimum Facility Plan. Details of the gates are included in the Engineering and Design Appendix. Ditches will be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the creek and tide gate structure. A total of two intermediate pipe outlets with flap gates will be incorporated to ensure that the proposed ditches will drain properly.

139. The proposed tide gate structure at Oakwood Creek is a stand-alone structure supported on piles that spans the width of the creek. The total length of the structure is approximately 22.75 feet, and top width approximately 16 feet and top elevation 18 feet NGVD 1929. Concrete head and wing walls connect the structure to the earthen levee on either side. The three 5' X 5' stainless steel sluice gates will be housed within the structure. These gates will be equipped with both electrical and backup manual operation to control flow. On both the seaward side and landward side of the tide gate, bar screens cover the flow openings to prevent passage of large debris from Oakwood Creek into the chamber. The gates will remain open during normal tidal elevations to allow passage of saline tidewater into marsh areas and

drainage of rainfall runoff. When exterior water level is forecasted to be higher than normal high tide, the gates will be closed at low tide before the storm event and reopened on the falling tide when the exterior water level drops more than 0.5 ft. below the interior level, allowing the interior to drain naturally, thereby maximizing available storage behind the tentatively selected plan alignment. The gates will again close when the rising tide exceeds interior levels.

140. The Minimum Facility for Area A results in residual flooding above grade for up to twenty structures for the studied range of flood frequencies, but the flooding is contained below the main floor of the structures. Table 13 provides a summary of the most likely flood levels (based on a 2-year exterior storm) and the number of structures impacted. Table 14 provides a similar summary with a higher tailwater event (a 10-year exterior storm) as part of the upper bound analysis. The Equivalent Annual Damages (EAD) for Area A with Minimum Facilities measures in place is estimated to be approximately $85,000 and the Total Annual Cost of the Minimum Facility measures is estimated to be approximately $349,000.

TABLE 13 AREA A: MINIMUM FACILITY IMPACTS – MOST LIKELY CONDITION

Area A Minimum Facility Most Likely Condition (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	5.84	5	0
10 Year	6.41	8	0
50 Year	6.93	11	0
100 Year	7.1	15	0

TABLE 14 AREA A: MINIMUM FACILITY IMPACTS – HIGH TAILWATER CONDITION

Area A Minimum Facility High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	5.84	5	0
10 Year	7.51	19	0
50 Year	8.04	20	0
100 Year	8.22	20	0

7.5.1.3 Sponsor Identified Plan

141. The non-Federal sponsors have identified a plan for Area A that will provide additional flood storage and opportunities to develop or enhance a range of wetland habitats. As seen in Figure 32a, the sponsor plan will provide an additional 30 acre-feet of storage volume. The increased storage provides limited effectiveness for damage reduction.

142. The Sponsor Plan will reduce flood levels from Minimum Facility conditions and result in low level flooding (below main floor) for up to 15 structures under the required range of flood conditions. Table 15 provides a summary of the most likely flood depths (based on a 2-year exterior storm) and structure impacts. Table 16 provides a similar summary when runoff coincides with a higher tailwater event (based on a 10-year storm). The EAD for Area A with the Sponsor Plan is estimated to be $45,500.

TABLE 15 AREA A: SPONSOR PLAN IMPACTS – MOST LIKELY CONDITION

Area A Sponsor Plan Most Likely Condition (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.45	0	0
10 Year	5.95	5	0
50 Year	6.44	8	0
100 Year	6.67	11	0

TABLE 16 AREA A: SPONSOR PLAN IMPACTS – HIGH TAILWATER CONDITION

Area A Sponsor Plan High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.45	0	0
10 Year	6.39	8	0
50 Year	7.28	19	0
100 Year	7.65	19	0

143. The Sponsor Plan will require the excavation and disposal of 12,000 cubic yards (CY) of excavation at a cost in excess of $800,000. The additional cost for the Sponsor Plan in Area A is not justified by the reduction in damages.

7.5.1.4 Optimum Plan

144. Minimum facility as shown on Figure 32 is the selected plan for Drainage Area A. The alternative considered is not justified based on a reduction in storm damages. Consideration of the Sponsor plan as part of the project mitigation plan may be warranted. Figure 32 provides a visual depiction of the Optimum Plan configuration.

7.5.2 Drainage Area B

7.5.2.1 Introduction

145. Drainage Area B has an existing dune with an elevation of 7.5 feet NGVD29 at its lowest which acts as a barrier which places it under Case #1. However with the proposed line of protection (DEC Conceptual Plan) relocated landward of the existing Tide Gate

Structure/Levee System at Oakwood Beach, it resulted in a loss of natural storage and directly causes an increase in interior flood stages which moves Drainage Area B into Case #2. Under existing conditions, surface runoff is able to convey along the East Branch of Oakwood Creek through the Oakwood Beach Tide Gate Structure to Raritan Bay. The surface runoff that does build up behind this existing surface would mostly occur in low-lying areas which are mostly classified by wetlands. The new alignment decreases the natural flood storage volume and the minimum facility that is needed to address the increase in water level and duration, of flooding is provided in the following paragraphs.

7.5.2.2 Minimum Facility (DEC Conceptual Plan)

146. In order to meet the Minimum Facility requirement of not inducing flooding, it was determined that one excavated pond along with three 5'x5' box culverts are needed to drain the lowest segments of Area B which are located at the proposed Line of Protection both east and west of Kissam Avenue. The three 5'x5' box culverts will be utilized to drain the western area underneath Kissam Avenue to the eastern. The pond provides a total of 94,200 CY of additional storage which is required to eliminate the induced flooding. The design for the Minimum Facility pond is a variation on the non-Federal Sponsor's Bluebelt plan and will have an invert elevation equal to 2.75 feet NGVD 1929. The original Bluebelt Plan proposed excavation below 2.5 ft. NGVD 1929 for drainage improvements, and water quality improvements but it would not provide significant additional flood storage. In this way the Minimum Facility Plan for Area B varies from the Bluebelt Plan.

147. The proposed East Pond is consistent with one of the ponds proposed for the Bluebelt Program The minimum facility for Drainage Area B (see Figure 2-3) includes a tide gate on the East Pond to control inflow and outflow from the drainage area. It would be constructed to elevation 20.5 NGVD29 with the same features as the tide gate in Area A, but with slight variations in dimension. New gate chambers would also be added at the existing Ebitts Street, New Dorp Lane, and Tysens Lane outfalls. The minimum facility would also include a road raising along Mill Road to an elevation of approximately 7.1 feet NGVD29 and Kissam Avenue to an elevation of approximately 7.1 feet NGVD29. The Mill Road raising will disallow the spillover of floodwater from Drainage Area A to Drainage Area B, while the Kissam Avenue road raising would provide vehicle access to the buried seawall/armored levee during storm events (USACE 2014a).

148. Figure 33a provides a visual of the stage storage relationships for the with and without project conditions. The proposed excavation offsets the storage lost by relocating the tentatively selected buried seawall/armored levee landward as part of the post-Sandy considerations.

149. The Minimum Facility Plan does not require extension of the major storm sewers because the plan alignment is landward of the outlets. The Minimum Facility Plan includes placing new gate chambers on three existing outlets (Tysens Lane, Ebbitts Street and New Dorp Lane) to prevent backflow through the coastal storm risk management system. Details of the gate chambers are included in the Engineering and Design Appendix. In addition a new tide gate structure will be implemented at the outlet of the excavated pond and will have dimensions and

specifications similar to the Area A Oakwood Creek tide gate. Kissam Avenue will be raised as part of the Minimum Facility Plan and additional drainage culverts to convey flow towards the Area B tide gate. Mill Road will be raised to an elevation of approximately 7.1 feet NGVD 1929 to prevent the spillover of interior floodwaters from Area A to Area B for the entire range of studied frequency events. The Minimum Facility will also require restrictive easements for 35.38 acres and a flowage easement for 45.85 acres.. Figure 33 provides an overview of the Minimum Facilities.

150. Tables 17 and 18 depict the water surface levels and structure impacts for the Minimum Facility assume that the Traube property is not being acquired to preserve natural storage. The Equivalent Annual Damages (EAD) for Area B with Minimum Facilities measures in place is estimated to be approximately $115,890 and the Total Annual Cost of the Minimum Facility measures is estimated to be approximately $1,432,000.

TABLE 17 AREA B: IMPACTS OF MINIMUM FACILITY IMPACTS – MOST LIKELY CONDITION

Area B Minimum Facility (Excavated Pond) Most Likely Condition (2-Year Exterior Storm) Water Surface Elevations and Structures Affected by Minimum Facility			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.65	0	0
10 Year	5.53	11	1
50 Year	6.02	12	1
100 Year	6.19	33	5

TABLE 18 AREA B: IMPACTS OF MINIMUM FACILITY IMPACTS – HIGH TAILWATER CONDITION

Area B Minimum Facility (Excavated Pond) High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected by Minimum Facility			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.65	0	0
10 Year	5.84	11	1
50 Year	6.55	33	1
100 Year	6.86	33	5

7.5.2.3 Interior Levee/ Non Structural Alternative

151. Prior to Hurricane Sandy (October 29-30, 2012), a plan to provide interior levees was developed. The interior levee would run along Fox Lane with a maximum top elevation of +7 feet NGVD 1929. This levee would have protected homes along Fox Lane, which are now schedule for acquisition and demolition. Non-structural protection, such as elevation or flood proofing, on Cedar Grove Avenue and along Kissam Avenue was also included. There were approximately 15 to 20 homes at Kissam Avenue, and approximately 4 homes at Cedar Grove Avenue that would have required non-structural protection under this alternative.

152. Evaluations conducted prior to Hurricane Sandy (October 29-30, 2012), revealed that this plan would not be cost effective. Because most of the structures that would have been protected by this plan are part of the New York State buyout program the alternative was eliminated from consideration and not updated for post-Sandy conditions.

7.5.2.4 Sponsor Identified Plan

153. The non-Federal Sponsors have identified a plan which would provide additional excavation to create permanent ponds and wetlands within the properties identified for acquisition The additional excavation and drainage features allow additional flow from the existing outfall to be directed to these ponding and wetland areas. The additional excavation is located at elevation below 3 ft. NGVD 1929 and will not provide significant effective flood storage.

7.5.2.5 Optimum Plan

154. Minimum facility has been selected as the optimum plan for Drainage Area B. The alternatives considered are not justified based on a reduction in storm damages. Figure 33 provides a visual depiction of the Optimum Plan configuration. In addition, this area will require spraying for 5 years to control the future growth of <u>Phragmites</u> (or common reedgrass). Spraying is needed to avoid potential hydraulic issues stemming from <u>phragmites</u> rhizons clogging openings in screens/trash racks and other hydraulic features and raising the interior ground surface elevation which would reduce the interior drainage storage capacity. Suitable wetland vegetation will be planted to replace the phragmites. This is consistent with the objectives of both the Fish and Wildlife Service and the Bluebelt Program.

7.5.3 Drainage Area C

7.5.3.1 Introduction

155. Drainage Area C has an existing road that acts as a barrier and therefore falls under the minimum facility category identified as Case #1, where the excess runoff is blocked by Father Capodanno Blvd. This means when the proposed line of protection is constructed, there will be no direct impact to the interior water levels due the line of protection. However, the interior water levels will be influenced by exterior conditions (tide levels), which controls how much surface runoff is transported through the existing outfalls. The surface runoff that builds up

behind the existing outfalls, due to restriction of flow, would mostly occur in low-lying areas consisting of wetlands, undeveloped sites including property located adjacent to New Creek and its tributaries and significant amount of developed property with existing structures. During coastal storm events, the amount of surface runoff that would be transported through the existing outfalls would be significantly reduced which would lead to an increase in interior flood stages. Even though preservation of natural storage may not meet the traditional definition of minimum facility within EM 1110-2-1413 (Section 3-2 (b), (c), and (d)), the guidance offer flexibility on the selection of minimum facility and not maintaining natural storage could lead to more development, significantly higher water levels, an increase in structure damage and also impact the capability of the interior flood loss reduction system to function over the project life, which is also mentioned with EM 1110-2-1413 (Section 6-5). At this time, the minimum facility includes restrictive easements to preserve natural storage.

7.5.3.2 Minimum Facility

156. Drainage Area C falls under the Case #1 category, where the excess runoff is blocked by Father Capodanno Blvd. An extension of the storm sewer is not needed because the plan alignment will be built landward of the existing outlets. The Minimum Facility for Drainage Area C includes placing new gate chambers at the existing Greeley Avenue, Midland Avenue, Naughton Avenue and Seaview Avenue oufalls to prevent backflow through the coastal storm risk management structure. Details of the gate chambers are included in the Engineering and Design Appendix. The Plan will also include the acquisition or preservation of 120.44 acres of natural storage as shown in Figure 34. The proposed property acquisitions are consistent with the properties identified as part of the Bluebelt plans. Under the Minimum Facility Plan Restrictive easements are required for these areas. Ditches or drains will be constructed along the landward side of the plan alignment to direct runoff toward all outlets.

157. Table 19 provides a summary of the flood stages with the Minimum Facility in place during a 2-year exterior event and Table 20 provides a summary of conditions during a 10 year exterior event. There is very extensive flooding under these conditions with potential impacts to over 800 structures. Under these conditions the Equivalent Annual Damages (EAD) for Area C with Minimum Facilities measures in place is estimated to be approximately $5,623,100 and the Total Annual Cost of the Minimum Facility measures is estimated to be approximately $1,095,400. To reduce the high annual cost with minimum facility measures in place, alternatives were formulated, to evaluate whether alternatives that further reduce interior flooding have Federal interest. The development of alternatives is presented in Section 7.5.3.3.

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TABLE 19 AREA C: MINIMUM FACILITY IMPACTS – MOST LIKELY CONDITION

Area C Minimum Facility Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.17	334	27
10 Year	5.35	708	87
50 Year	6.26	870	209
100 Year	6.36	870	209

TABLE 20 AREA C: MINIMUM FACILITY IMPACTS – HIGH TAILWATER CONDITION

Area C Minimum Facility High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.17	334	26
10 Year	6.89	870	209
50 Year	7.60	1162	405
100 Year	7.93	1162	405

7.5.3.3 Development of Alternatives

158. Eight Alternatives were developed and analyzed with different combinations of pumps and ponds. Each alternative includes the acquisition of the same properties as the Minimum Facility plan. Some of the plans were eliminated from consideration based on evaluations conducted prior to Hurricane Sandy (October 29-30, 2012). For instance, the evaluation of pump stations initially considered pump station sizes ranging from 600 cfs to 1500 cfs. That analysis identified that the optimum pump station size would be 1500 cfs. Only the 1500 cfs pump station has been updated to reflect post-Sandy conditions because it was known to be the optimum pump size relative to the other pump sizes. For each alternative any areas of excavated ponding are anticipated to require flowage easements due to the increase in flood depths and/or duration.

7.5.3.4 Alternative 1 – Pump Stations

159. Alternative #1 originally considered five possible pump stations 1500 cfs (Alternative #1a), 1200 cfs (Alternative #1b), 900 cfs (Alternative #1c), 750 cfs (Alternative #1d), and 600 cfs (Alternative #1e) in the vicinity of Naughton Avenue. The 1500 cfs station was determined to be the optimum pump station size and was updated to reflect post-Hurricane Sandy (October 29-30, 2012) conditions.

160. The post-Sandy updates also incorporate more details for the pump station layout to reflect some limitations in the capacity of the existing outfall. The design and costs were modified to locate the pump station at the buried seawall/armored levee design alignment. With the new alignment, the pump station Alternative would require the construction of a concrete drainage channel, a diversion weir, pipe culverts, and a new outfall to allow the pump station to operate at the same time as the existing gravity outfalls. Figures 35 through 37 provide plan and section views of these additional features. The initial construction cost for the pump station related features (excluding all of the Minimum Facility features) is estimated to be $36.2 million over the Minimum Facility costs. With pump station O&M costs added to the initial construction costs, the final annual incremental (above Minimum Facilities) cost is $2,115,400.

161. As depicted in Tables 21 and 22, the pump station provides a large reduction in interior flood levels and consequentially the number of structures impacted by the flood hazard is diminished. Alternative 1 is calculated to have an EAD of $1,147,600, which is a $4,475,500 reduction in annual storm damages compared to the Minimum Facility condition.

TABLE 21 AREA C: ALTERNATIVE 1 IMPACTS – MOST LIKELY CONDITION

Area C Alternative 1 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	3.10	6	0
10 Year	3.22	95	6
50 Year	3.51	95	6
100 Year	3.57	95	6

TABLE 22 AREA C: ALTERNATIVE 1 IMPACTS – HIGH TAILWATER CONDITION

Area C Alternative 1 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	3.10	6	0
10 Year	4.18	334	26
50 Year	5.55	710	88
100 Year	6.18	870	209

7.5.3.5 Alternative 2 – Ponding with Pump Station

162. In addition to the Minimum Facility features Alternative 2 includes one 900 cfs pump station in the vicinity of Naughton Avenue and with four excavated ponds (245,350 cubic yards total) along Slater Avenue. A smaller pump in conjunction with excavated ponds provided similar performance to the 1500 cfs pump station but at a higher cost. Alternative 2 was therefore eliminated. This alternative was not reevaluated for post-Hurricane Sandy (October 29-30, 2012) conditions.

7.5.3.6 Alternative 3 – Non-Structural Retrofits

163. Alternative #3 consisted of raising approximately 770 structures in the vicinity of New Creek. An economic analysis of this alternative concluded that it would not be cost effective. Thus, Alternative 3 was eliminated from further consideration.

7.5.3.7 Alternative 4 – 377,200 cy Ponding (7 ponds)

164. In addition to the Minimum Facility features, Alternative 4 included adding 377,200 cy. of additional storage in the form of seven excavated ponds located along Seaview Avenue, Father Capodanno Boulevard, Midland Avenue and Hylan Boulevard. The invert of these ponds would be equal to 3 feet NGVD 1929. Based on initial analyses Alternative 4 was considered a potentially viable Plan and was updated to reflect post-Hurricane Sandy (October 29-30, 2012) conditions.

165. The initial construction cost for Alternative 4 excluding the Minimum Facility features is estimated to be approximately $27 million. With the additional pond O&M costs, this Alternative results in incremental (above Minimum Facilities) costs of $1,296,300 annually.

166. As depicted in Tables 23 and 24, Alternative 4 provides a fairly large reduction in interior flood levels and in the number of structures impacted. Alternative 4 is calculated to have EAD of $1,255,600, which is a $4,367,500 reduction in annual storm damages compared to the Minimum Facility condition.

TABLE 23 AREA C: ALTERNATIVE 4 IMPACTS – MOST LIKELY CONDITION

Area C Alternative 4 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	2.45	2	0
10 Year	3.28	97	5
50 Year	4.17	332	27
100 Year	4.53	335	27

TABLE 24 AREA C: ALTERNATIVE 4 IMPACTS – HIGH TAILWATER CONDITION

Area C Alternative 4 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	2.45	6	0
10 Year	4.89	337	26
50 Year	6.25	870	209
100 Year	6.75	870	209

7.5.3.8 Alternative 5 – 463,100 cy Ponding (9 ponds)

167. In addition to the Minimum Facility features, Alternative 5 included an additional 463,100 cy, of storage in the form of nine excavated ponds located along Seaview Avenue, Father Capodanno Boulevard, Midland Avenue and Highland Boulevard. The invert of these ponds would be equal to 3 feet NGVD 1929. Initial analyses indicated that this alternative did not provide a substantial increase in benefits above Alternative 4, but had a 20% increase in excavation volume and cost. Alternative 5, therefore, was eliminated from further analysis. This Alternative was not updated for post-Hurricane Sandy (October 29-30, 2012) conditions.

7.5.3.9 Alternative 6 – 245,350 cy. Ponding (4 ponds)

168. In addition to the Minimum Facility features, Alternative 6 includes an additional 245,350 cy. of storage in the form of four ponds. Based on initial analyses, Alternative 6 was considered a potentially viable Plan and was updated to reflect post-Hurricane Sandy (October 29-30, 2012) conditions.

169. As depicted in Tables 25 and 26, Alternative 6 provides a moderate reduction in interior flood levels and in the number of structures impacted by flood hazards. Alternative 6 was calculated to have EAD of $2,602,200, which is a $3,071,200 reduction in annual storm damages compared to the Minimum Facility condition.

TABLE 25 AREA C: ALTERNATIVE 6 IMPACTS – MOST LIKELY CONDITION

Area C Alternative 6 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	3.01	6	0
10 Year	4.14	334	26
50 Year	5.02	343	26
100 Year	5.45	708	87

TABLE 26 AREA C: ALTERNATIVE 6 IMPACTS – HIGH TAILWATER CONDITION

Area C Alternative 6 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	3.01	6	0
10 Year	6.01	721	89
50 Year	7.05	876	209
100 Year	7.37	1162	405

7.5.3.10 Alternative 7 – 176,700 cy Ponding (2 ponds)

170. In addition to the Minimum Facility features, Alternative 7 included an additional 176,700 cubic yards of storage in the form of two excavated ponds. The invert of these ponds would be equal to 3 feet NGVD 1929. This alternative provided a relatively small reduction in flood levels and was eliminated from further consideration. Alternative 7 was not updated for post- Hurricane Sandy (October 29-30, 2012) conditions.

7.5.3.11 Alternative 8 – Modified Bluebelt Plan

171. Alternative 8 is based on the Midland Beach Bluebelt Plan developed by NYCDEP. Figure 39 provides an overview of the Bluebelt ponding areas. Details of the Bluebelt plan are available on NYCDEP Website at:

http://www.nyc.gov/html/dep/html/environmental_reviews/midisland_bluebelt_drainage_plan.shtml.

172. In order to allow an accurate comparison of costs and benefits among the Bluebelt plan and the other alternatives, the Bluebelt Plan was modified to exclude items that do not directly

contribute to the project planning objective of providing NED damage reduction. Overall, the Midland Beach Bluebelt Plan includes approximately 850,000 cy, of excavation, but based on a comparison of stage storage data, approximately 270,000 cy, of that excavation would be located below the ground water elevation of 2 ft. NGVD 1929. The volume below the water table would not provide effective flood risk management. Alternative 8 is a variation of the Bluebelt plan that eliminates the quantities and costs of the below groundwater storage. Alternative 8 is estimated to provide 580,000 cy, of storage in the form of ten excavated ponds. The invert of these ponds would be equal to 3 feet NGVD 1929.

173. Alternative 8 also does not include a new 6 ft. x 7.5 ft. outfall that was proposed as part of the Bluebelt Plan. The interior drainage models indicated that this outfall would have only had a minor impact on flood depths, and would have only reduced the areas EAD by approximately $25,000.

174. The cost of Bluebelt features associated with wetland restoration or recreation features were also excluded from the benefit cost comparison for this Alternative. These features may be recommended as part of any required project mitigation, but are not directly a part of the interior flood risk management features.

175. The initial construction cost for Alternative 8 excluding all of the Minimum Facility features is estimated to be $39.7 million. With the additional pond O&M costs, the resulting incremental (above Minimum Facilities) cost would be approximately $1,780,800 annually.

176. As depicted in Tables 27 and 28, Alternative 8 would provide a large reduction in interior flood levels and in the number of structures impacted by flood hazards. Alternative 8 is calculated to have EAD of $774,700, which is a $4,848,400 reduction in storm damages compared to the Minimum Facility condition.

TABLE 27 AREA C: ALTERNATIVE 8 IMPACTS – MOST LIKELY CONDITION

Area C Alternative 8 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	1.19	0	0
10 Year	2.64	6	0
50 Year	3.70	35	6
100 Year	4.22	334	26

TABLE 28 AREA C: ALTERNATIVE 8 IMPACTS – HIGH TAILWATER CONDITION

Area C Alternative 8 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	1.19	0	0
10 Year	4.44	334	26
50 Year	6.15	870	209
100 Year	6.67	870	209

7.5.3.12 Optimum Plan

177. Because Area C has such high annual damages with the Minimum Facilities, a relatively large number of alternatives were considered. Four Alternatives to the Minimum Facility were considered potentially viable and updated for post-Hurricane Sandy (October 29-30, 2012) conditions. The updated analyses indicate that the cost of the Alternatives range from about $17 million to $39.7 million. Each of the four Alternatives is cost effective with Benefit to Cost Ratios (BCRs) between 2.2 for Alternative 1 (1500 cfs Pump Station Plan) and 3.9 for Alternative 6 (245,350cy Pond Plan). A summary of the benefit/cost information is available in Table 29. The highest net benefits in excess of costs occur with Alternative 4 (377,200 cy Pond Plan). At this time Alternative 4 is identified as the optimum plan for Area C, pending the ongoing update of benefits and review of the alternative cost estimates. Figure 40 provides the optimum plan for Area C including both Minimum Facility and ponding features. Figure 40 provides a visual depiction of the Optimum Plan configuration.

TABLE 29 INCREMENTAL COSTS AND BENEFITS (RELATIVE TO MINIMUM FACILITY) AREA C ALTERNATIVES

Alternative	**Incremental First Cost	Incremental Annual Costs	Equivalent Annual Damage	Damage Reduction Benefits	Benefit to Cost Ratio	Net Benefits
Minimum Facility	$23,100,000	$0	$5,623,100	$0		N/A
Alternative 1 – 1500 cfs Pump Station	$36,240,000	$2,144,800	$1,147,600	$4,475,500	2.1	$2,360,100
Alternative 4 - Ponds w/ 377,200cy Excavation	**$27,000,000**	**$1,276,300**	**$1,255,600**	**$4,367,500**	**3.4**	**$3,071,200**
Alternative 6 - Ponds w/ 245,350cy Excavation	$17,000,000	$798,600	$2,602,200	$3,020,200	3.7	$ 2,210,400
Alternative 8 – Modified Bluebelt Plan w/580,000cy of Pond Excavation	$39,700,000	$1,857,800	$774,700	$4,848,400	2.6	$2,966,400

*50 year period-of-analysis, 3.375% Federal Discout Rate, (July, 2014 Price Level)
Highlighted Costs and benefits pending Real Estate Cost

7.5.4 Drainage Area D

7.5.4.1 Introduction

178. Drainage Area D has an existing road that acts as a barrier and therefore falls under the minimum facility category identified as Case #1, where the excess runoff is blocked by Father Capodanno Blvd. This means when the proposed line of protection is placed, there will be no direct impact to the interior water levels due the line of protection. However, the interior water levels would be influenced by the exterior conditions (tide levels), which controls how much surface runoff is transported through the existing outfalls. The surface runoff that could potentially build up behind the existing outfalls, due to restriction of flow, would mostly occur in non-damaging areas. During coastal storm events, the amount of surface runoff that would be transported through these existing outfalls would be significantly reduced. In turn, this would lead to increase in interior flood stages in project area. Even though preservation of natural storage may not meet the traditional definition of minimum facility within EM 1110-2-1413 (Section 3-2 (b), (c), and (d)), the guidance offer flexibility on the selection of minimum facility and not acquiring restrictive easements for natural storage preservation could lead to

more development, higher water levels and impact the capability of the interior flood loss reduction system to function over the project life as mentioned in EM 1110-2. At this time, the minimum facility includes restrictive easements to preserve open land for natural storage. The minimum facility plan proposed for Area D is described in the following paragraphs.

7.5.4.2 Minimum Facility

180. An extension of the storm sewer is not needed because the plan alignment is landward of the existing outfall. The Minimum Facility for drainage Area D includes placing a new and a new gate chamber at the existing Quintard Street/Raritan Avenue outfall to prevent backflow through coastal storm risk management structure. The gate chamber details are included in the Engineering and Design Appendix. The Minimum Facility will also include the preservation of 30.76 acres of natural flood storage area on land owned by NYC Parks. A restrictive easement will be obtained for the needed parcels. As part of the Minimum Facility Plan, Seaview Avenue and Father Capodanno Boulevard will be raised provide high ground to hydraulically separate Drainage Areas C and D. Figure 41 provides a plan view of the Minimum Facility Plan. Ditches will be constructed along the landward side of the tentatively selected buried seawall/armored levee to collect local runoff and overtopping flows. Table 30 and 31 quantify the flooding elevations with the Minimum Facility Plan in place for Area D. The Equivalent Annual Damages (EAD) for Area D with Minimum Facilities measures in place is estimated to be approximately $137,500 and the Total Annual Cost of the Minimum Facility measures is estimated to be approximately $716,000.

TABLE 30 DRAINAGE AREA D MINIMUM FACILITY IMPACTS – MOST LIKELY CONDITION

Area D Minimum Facility Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	6.76	2	0
10 Year	8.62	11	3
50 Year	9.62	33	6
100 Year	9.78	33	6

TABLE 31 DRAINAGE AREA D MINIMUM FACILITY IMPACTS – HIGH TAILWATER CONDITION

Area D Minimum Facility High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	6.76	2	0
10 Year	9.52	33	6
50 Year	10.35	59	6
100 Year	10.35	59	6

7.5.4.3 Optimum Plan

181. Minimum Facility is the selected plan for Drainage Area D. No other alternatives were considered. Figure 41 provides a depiction of the Optimum Plan configuration.

7.5.5 Drainage Area E

7.5.5.1 Introduction

182. Drainage Area E has an existing road that acts as a barrier and therefore falls under the minimum facility category identified as Case #1, where the excess runoff is blocked by Father Capodanno Blvd. This means when the proposed line of protection is placed, there will be no direct impact to the interior water levels due the line of protection. However, the interior water levels would be influenced by the exterior conditions (tide levels), which controls how much surface runoff is transported through the existing outfalls. The surface runoff that could

potentially build up behind the existing outfalls, due to restriction of flow, would mostly occur in non-damaging areas. During coastal storm events, the amount of surface runoff that would be transported through these existing outfalls would be significantly reduced. In turn, this would lead to increase in interior flood stages in project area. Even though preservation of natural storage may not meet the traditional definition of minimum facility within EM 1110-2-1413 (Section 3-2 (b), (c), and (d)), the guidance offer flexibility on the selection of minimum facility and not maintianing land for natural storage could lead to more development, higher water levels and impact the capability of the interior flood loss reduction system to function over the project life as mentioned in EM 1110-2-1413. At this time, the minimum facility includes the acquisition of restrictive easements for preservation of natural storage. The minimum facility plan proposed for Area is described in the following paragraphs.

7.5.5.2 Minimum Facility

184. An extension of the storm sewer is not needed because the plan alignment will be built landward of the existing outlets. The Minimum Facility for drainage area E (see Figure 42) includes gate chambers at Sand Lane to prevent backflow through the tentatively selected buried seawall/armored levee. Details of the gate chamber are included in the Engineering and Design Appendix. The Plan will also require 46.7 acres of natural storage located on properties owned by NYC or approved for acquisition as a part of longer term acquisition plan under the NYC DEP South Beach Bluebelt Plan. The land required for the natural storage will be protected from development using restrictive easements. A piped outfall and junction chamber at Quincy Avenue is part of the Minimum Facility Plan in order to convey flow to the Sands Lane Outfall. Other ditches or drains will be constructed along the landward side of the buried seawall/armored levee to collect local runoff or overtopping flow. As demonstrated in Tables 32 and 33, there are high counts of structures expecting flood related damages throughout the range of studied events. The Equivalent Annual Damages (EAD) for Area E with Minimum Facilities measures in place is estimated to be approximately $2,204,000 and the Total Annual Cost of the Minimum Facility measures is estimated to be approximately $387,000. In light of the high damages under the Minimum Facility conditions, several Alternatives were considered to maximize the Net Benefits. To reduce the high annual cost with minimum facility measures in place, alternatives were formulated, to evaluate whether alternatives that further reduce interior flooding have Federal interest. The development of alternatives is presented in Section 7.5.5.3.

TABLE 32 AREA E: MINIMUM FACILITY IMPACTS – MOST LIKELY CONDITION

Area E Minimum Facility Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	6.99	43	16
10 Year	7.60	125	29
50 Year	8.16	171	44
100 Year	8.40	171	44

TABLE 33 AREA E: MINIMUM FACILITY IMPACTS – HIGH TAILWATER CONDITION

Area E Minimum Facility High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	6.99	43	16
10 Year	8.36	171	44
50 Year	9.16	261	77
100 Year	9.36	261	77

7.5.5.3 Development of Alternatives

185. Given the high level of damage with Minimum Facilities a number of alternatives were developed and analyzed for Area E. These alternatives considered with different combinations of pumps, ponds and non-structural measures. Each Alternative assumes acquisition of the same properties as the Minimum Facility plan. Some of the plans were eliminated from consideration based on evaluations conducted prior to Hurricane Sandy (October 29-30, 2012). For instance, the evaluation of the 1800 pump stations was identified as having annual costs that exceed the annual damages with Minimum Facility and was eliminated from consideration and hydraulic models and damage estimates for that alternative were not updated for post-Sandy conditions. Areas with excavated ponds resulting in increased depth of flooding were identified as potentially needing flowage easements.

7.5.5.4 Alternative 1 – 1800 cfs Pump Station

186. In addition to the Minimum Facilities Alternative 1 included an 1800 cfs pump station at McLaughlin Street. This Alternative has been identified as not cost effective and has been eliminated from consideration. Alternative 1 was not updated for post-Sandy conditions.

7.5.5.5 Alternative 2 – 222,720 cy Ponding (Two Ponds)

187. In addition to the Minimum Facilities Alternative 2 includes construction of two excavated ponds totaling 222,720 cy along McLaughlin Street. The invert of these ponds would be equal to 3 feet NGVD 1929. As seen in Tables 34 and 35, Alternative 2 is effective in reducing flood depths and the number of structures impacted. The EAD with Plan 2 is calculated to be $288,800, which is a reduction of $1,915,100 compared to the Minimum Facility conditions.

188. The initial construction cost for the pond related features of Alternative 2 (excluding all of the Minimum Facility features) is estimated to be $14,279,100 over the Minimum Facility costs. With the pond O&M costs included, the resulting incremental (above Minimum Facilities) cost is $686,400 annually.

TABLE 34 AREA E: ALTERNATIVE 2 IMPACTS – MOST LIKELY CONDITION

Area E Alternative 2 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.07	0	0
10 Year	5.54	34	5
50 Year	6.42	43	15
100 Year	6.84	43	15

TABLE 35 AREA E: ALTERNATIVE 2 IMPACTS – HIGH TAILWATER CONDITION

Area E Alternative 2 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	4.07	0	0
10 Year	6.05	34	5
50 Year	7.39	120	28
100 Year	8.04	123	28

7.5.5.6 Alternative 3 – 222,720 cy Ponding Plus 600 cfs Pump Station

189. In addition to the Minimum Facilities, Alternative 3 includes construction of one 600cfs pump station at Naughton Avenue in conjunction with two excavated ponds (222,720 cubic

yards) from Alternative 2. This plan was found to not be cost effective and was eliminated from further consideration. Alternative 3 was not updated for post-Sandy conditions.

7.5.5.7 Alternative 4 – Non Structural

190. In addition to Minimum Facilities Alternative 4 considered raising approximately 140 structures in the vicinity of Father Capodanno Blvd. and Sand Lane. Economic analysis of this alternative concludes that it would not be cost effective. Alternative 4 was eliminated from further consideration and was not reconsidered during the post-Sandy update.

7.5.5.8 Alternative 5 – Modified Bluebelt Plan

191. Alternative 5 is based on the South Beach Bluebelt Plan developed by NYC DEP. Details of the Bluebelt plan are available from the DEP at

http://www.nyc.gov/html/dep/html/environmental_reviews/midisland_bluebelt_drainage_plan.shtml.

192. In order to allow an accurate comparison of costs and benefits among the Bluebelt plan and the other alternatives, the Bluebelt Plan was modified to exclude items that do not directly contribute to the project planning objective of providing NED damage reduction. Overall the South Beach Bluebelt Plan includes approximately 399,000 cy of excavation; however, approximately 81,000 cy of that excavation is located below the anticipated ground water elevation of 2 ft. NGVD 1929. Excavation of the he volume below the water table does not provide effective flood risk management.

193. Alternative 5 is a variation of the original Bluebelt plan that eliminates the quantities and costs associated with excavating below groundwater level. Overall Alternative 5 is estimated to provide 318,000 cy of effective excavated storage within a single pond. The inverts of the pond would be equal to 3 feet NGVD 1929.

194. The modified Plan also eliminates an outfall proposed as part of the Bluebelt Plan because the interior drainage models indicate that this outfall will have only a minor impact on flood depths and EAD.

195. The cost of Bluebelt features associated with wetland restoration or recreation are also excluded from the benefit cost comparison of the interior drainage. These features may be recommended as part of any required project mitigation, but are not directly a part of the interior flood risk management features.

196. For Alternative 5, the initial construction costs for the pond and related features (excluding all of the Minimum Facility features) are estimated to be $19,350,000 over the Minimum Facility costs. With the pond O&M costs included, the Alternative in incremental (above Minimum Facilities) cost of $915,700 annually.

197. As seen in Tables 36 and 37, Alternative 5 provides a large reduction in interior flood levels and in the number of structures impacted. This level is comparable to the effectiveness of the Alternative 2 Ponds. Alternative 5 is calculated to have EAD of $216,900, which is a $1,987,000 reduction in storm damages compared to the Minimum Facility condition.

TABLE 36 AREA E: ALTERNATIVE 5 IMPACTS – MOST LIKELY CONDITION

Area E Alternative 5 Most Likely Condition - (2-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Structures Flooded Above Ground**	**Structures Flooded Above Main Floor**
2 Year	2.99	0	0
10 Year	5.31	32	5
50 Year	6.60	43	15
100 Year	7.10	52	15

TABLE 37 AREA E: ALTERNATIVE 5 IMPACTS – HIGH TAILWATER CONDITION

Area E Alternative 5 High Tailwater Condition (10-Year Exterior Storm) Water Surface Elevations and Structures Affected			
Event	**Interior WSEL ft. NGVD 1929**	**Flooded Above Ground**	**Flooded Above Main Floor**
2 Year	3.00	0	0
10 Year	5.70	34	5
50 Year	7.40	124	28
100 Year	8.10	129	28

7.5.5.9 Optimum Plan

198. Alternative 2 and Alternative 5 both provide cost effective options for reducing damage in interior Area E. Other alternatives were eliminated as not cost effective. Table 38 shows a comparison of incremental costs, benefits and Benefit to Cost Ratios (BCRs) for the Alternatives. At this time Alternative 2 is identified as the optimum plan for Area E, pending the ongoing update of benefits and review of the alternative cost estimates. The difference in net benefits between Alternative 2 and Alternative 5 is very small and factors such as environmental impacts or community acceptability may outweigh the small difference in NED benefits. Figure 43 provides a visual depiction of the Optimum Plan configuration.

TABLE 38 INCREMENTAL COSTS AND BENEFITS (RELATIVE TO MINIMUM FACILITY) AREA E ALTERNATIVES

Alternative	*Incremental First Cost	Incremental Annual Costs*	Equivalent Annual Damage*	Damage Reduction Benefits	Benefit to Cost	Net Benefits
Minimum Facility	**$8,342,000**	**$0**	**$2,203,900**	**$0**		**N/A**
Alternative 2 - Ponds w/ 222,700cy Excavation	**$14,279,100**	**$671,400**	**$288,800**	**$1,915,100**	**2.9**	**$1,243,700**
Alternative 5 – Modified Bluebelt Plan	$19,350,000	$915,700	$216,900	$1,987,000	2.2	$1,071,300

*50 year period-of-analysis, 3.375% Federal Discout Rate, (July, 2014 Price Level)
Highlighted Costs and benefits pending Real Estate Cost

7.6 Tentatively Selected Interior Drainage Plan

199. Within each interior drainage area, the economics for a series of alternate drainage measures were evaluated and compared to determine which alternative contributes the highest level of net benefits to the project. The optimum interior drainage alternative for each area is presented in Table 39.

TABLE 39 TENTATIVELY SELECTED INTERIOR DRAINAGE PLAN

Drainage Area	Optimum Plans	First Cost	O&M Cost***	Total Annual Cost**	Annual Benefit	Net Benefits
Area A	Minimum Facility	$6,956,400	$39,000	$349,000	$0	$0
Area B	Minimum Facility	$30,048,900	$93,000	$1,432,000	$0	$0
Area C	Alt 4: 7 Ponds (377,200 cy of excavation)	$50,090,300	$159,000	$2,390,800	$4,368,000	$3,071,200
Area D	Minimum Facility	$15,686,500	$17,000	$716,000	$0	$0
Area E	Alt 2: 2 Ponds (222.720 cy of excavation)	$22,621,100	$49,000	$1,056,900	$1,915,000	$1,243,700
Total	**-**	**$125,403,200***	**$357,000**	**$5,944,700**	**$6,283,000**	**$4,314,900**

50 year period-of-analysis, 3.375% Federal Discout Rate, (July, 2014 Price Level)
*Includes $84,120,000 of Minimum Facility Costs
** Includes IDC and O&M Costs
***Includes Annualized Replacement Costs (See Cost Appendix)

8 RESIDUAL FLOOD ANALYSIS

200. The National Economic Development (NED) Plan for the South Shore of Staten Island Interim Study is designed to reduce the risk from exterior coastal surge and either maintain or reduce the risk from interior precipitation-runoff flooding. Residual flooding, by definition, is the flooding that still occurs with the NED Plan in place. For the studied 500 year peak coastal surge level, the peak flooding stage exceeds the design level of the Line of Protection measure in the NED Plan, which is designed to a 15.6 ft. NGVD 1929 stillwater stage. The overtopping in this case will create flood levels throughout the study area equivalent to the without-project condition. While the peak interior and exterior flood stages in the study area will be coincident during a hypothetical 500-yr storm event, they will vary during the other studied frequency intervals.

201. The predicted exterior flood stages from FEMA's forthcoming coastal Flood Insurance report are presented in Table 40 and the residual peak flood stages from the Interior Drainage Analysis are presented in Table 41. The residual peak interior flood stages are the expected flood conditions from the Interior Drainage Analysis. From the analysis it was found that the risk condition can increase or decrease according to the relationship between the interior and exterior stages. This phenomenon is characterized by three separate likelihoods or combinations of interior/exterior events: the lower bound, expected, or upper bound condition. For this study, the expected condition is used as the condition for recording with project damage reduction, but there is still a chance that a worse flooding condition could occur (upper bound condition).

202. To communicate the increased risk associated with the upper bound condition, the with project inundation extents presented in Figure 44 depict both the expected (blue hatch) and upper bound (green) conditions for the 100-yr event. Figure 44 also depicts the without project condition (gray). In addition, residual flood maps, depicting the flood risk for the 10-yr, 50-yr, 100-yr, and 500-yr) expected condition for each Drainage Area, are presented Figures 45-49.

TABLE 40 PEAK EXTERIOR STILLWATER ELEVATIONS FOR PROJECT AREA (FEMA)

Frequency of Occurrence in years	Stillwater Stage (ft. NGVD 1929)
10	8.5
50	11.3
100	12.6
500	15.9

TABLE 41 PEAK RESIDUAL INTERIOR FLOOD STAGES

Drainage Area (TSP Plan)	Peak Residual Flood Stages (ft. NGVD 1929)			
	10-yr Event	50-yr Event	100-yr Event	500-yr Event*
Area A (Minimum Facility)	6.41	6.93	7.10	15.9
AreaB (Minimum Facility)	5.48	6.00	6.21	15.9
Area C (Alternative 4)	3.28	4.17	4.53	15.9
Area D (Minimum Facility)	8.62	9.62	9.78	15.9
Area E (Alternative 2)	5.54	6.42	6.84	15.9

*Exterior Stillwater Elevation exceeds Project Design and overtops into all Drainage Areas

8.1 Line of Protection - Project Performance and Risk Analysis

203. The Line of Protection will be the first line of defense against surge and wave action experienced during coastal events. However, extremely rare frequency coastal events, such as a 500-yr Hurricane, that has a storm surge which exceeds the NED Plan Line of Protection stillwater design height would overtop the LOP and cause extensive damages to structures in the study area, and life-safety risks. Comparably, the surges from Hurricane Sandy overtopped the existing coastal barrier (Father Capaddano Boulevard and other high ground) and resulted in extensive damages to property and the loss of life for 23 residents in Staten Island.

204. ER 1105-2-101, "Risk Analysis for Flood Damage Reduction Studies (USACE, January 3, 2006) stipulates that the risk analysis for a flood protection project should quantify the performance of the plan and evaluate the residual risk, including the consequences of exceedence of the project's capacity. The guidance specifically stipulates, along with the basic economic performance of a project, the engineering performance of the project is to be reported in terms of:

- The annual exceedance probability
- The long-term risk of exceedance
- The conditional non-exceedance probability

205. The overall economic performance of the selected line of protection plan has been computed by HEC-FDA and the results are presented in Table 42.

TABLE 42 EXPECTED AND PROBALISTIC VALUES OF STRUCTURES/CONTENTS DAMAGE REDUCED BY PROJECT

Alternative	Equivalent Annual Damage (Line of Protection Only)			Probability that Damage Reduced Exceeds the Indicated Values		
	Without Project	With Project	Damage Reduced	75%	50%	25%
15.6 NGVD 1929 Stillwater Design	$26,168,000	$5,058,000	$21,110,000	$11295,000	$18,490,000	$28,473,000

206. The annual exceedance probability of a project is the likelihood that a target stage is exceeded by flood waters in any year and can be considered as an indication of the level of risk management provided by the NED Plan. The target stage is the point at which significant damage is incurred in the with-project condition, the significant damage elevation was defined as the water surface elevation which results in damages equal to 5% of damages incurred by the 1% annual chance exceedance event ("100-year" event) in the without-project condition.

207. The target stage for each reach was used in HEC-FDA to calculate the base year median and expected annual exceedance probability for the NED Plan. The median value reflects the basic as-designed performance of the plan without the application of uncertainty to the basic discharge-frequency and stage-discharge functions, while the expected value is computed from the results of the Monte Carlo simulations which take into account uncertainty in hydrologic/hydraulic functions and project features such as diversion structures. Hence the difference between the two is an indication of the uncertainty associated with the project performance.

208. The long-term risk of exceedance is the probability that the design stage will be exceeded at least once in the specified durations of 10, 30, and 50 years, and the conditional non-exceedance probability measures the likelihood that the project will not be exceeded by a specified hydrologic event. For this analysis the base year conditional non-exceedance probability has been computed for each alternative for the 10%, 4%, 2%, 1%, 0.4% and 0.2% annual chance exceedance events (10-, 25-, 50-, 100-, 250- and 500-year floods). These indicators of project performance and residual risk for the NED Plan are presented in Table 43.

TABLE 43 PROJECT PERFORMANCE ANALYSIS – TENTATIVELY SELECTED LINE OF PROTECTION

Project Performance Analysis		
Annual Exceedance Probability of Target Stage	Median	0.2%
	Expected	0.3%
Long Term Exceedance Probability	10 Years	3%
	30 Years	9%
	50 Years	14%
Conditional Non-Exceedance Probability	10%	100%
	4%	100%
	2%	100%
	1%	98%
	0.4%	77%
	0.2%	43%

8.2 Interior Drainage Residual Risk Analysis

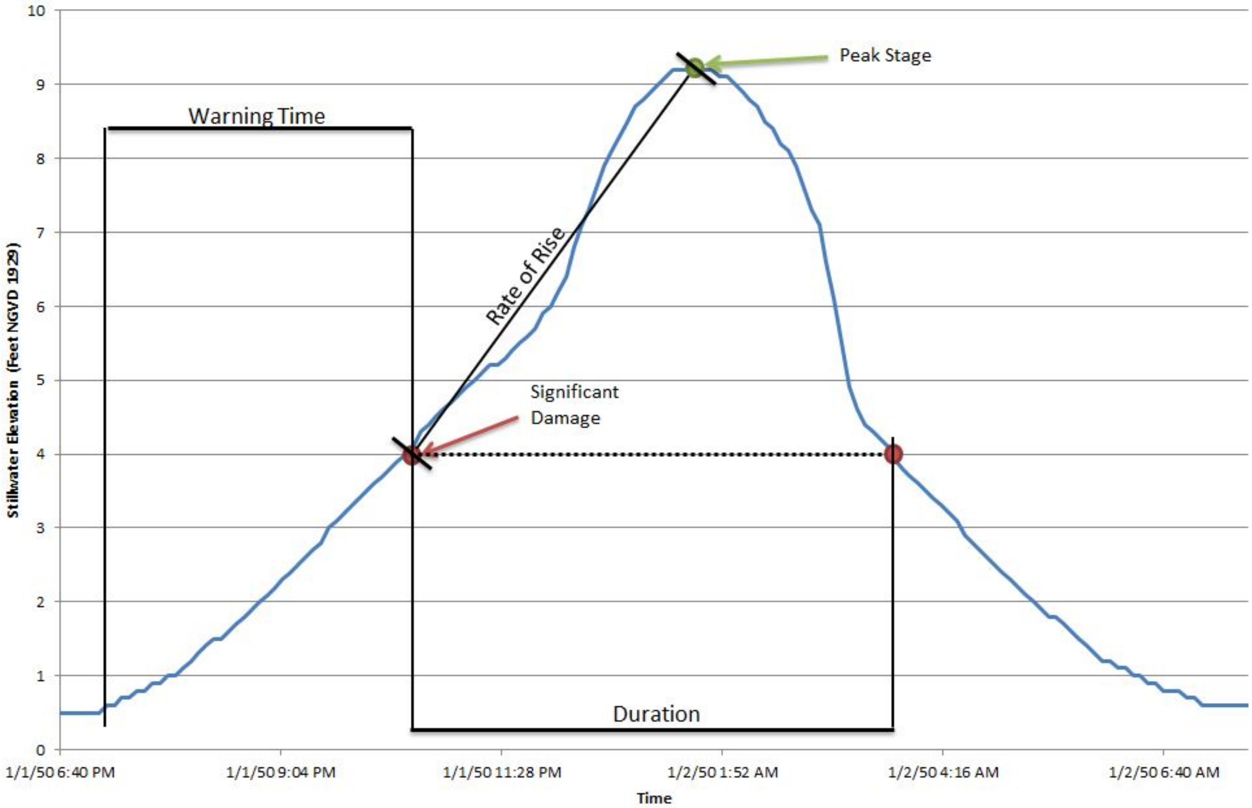
209. For storm events where the Line of Protection stillwater design level is not exceeded, there are still residual flood risks from precipitation-runoff from the Interior Drainage Areas landward of the Line of Protection. As part of the NED Plan, Interior Drainage Measures are to be implemented as to ensure that the project does not induce flooding as mandated by the criteria of the Minimum Facility, but also to be studied as to discover where additional measures may be implemented to increase the Net Benefits of the Plan.

210. Local flooding of roadways and some structural damages will occur around the 10-yr storm event even with the NED Plan in place. A significant damage elevation was defined by the stage in which non-nominal damages begin to occur within each Interior Drainage Area. The significant damage elevations for the study area are:

- Significant Damage Elevation in Drainage Area A = 4.50 ft. NGVD 1929
- Significant Damage Elevation in Drainage Area B = 5.11 ft. NGVD 1929
- Significant Damage Elevation in Drainage Area C = 3.12 ft. NGVD 1929
- Significant Damage Elevation in Drainage Area D = 8.11 ft. NGVD 1929
- Significant Damage Elevation in Drainage Area E = 5.12 ft. NGVD 1929

211. By setting significant damage elevations, it is possible to quantify different important flooding characteristics other than just the peak flood stage such as the warning time, the rate of rise of floodwaters, and the duration of inundation. Other important considerations are the number of structures that will experience flood related damage in the with-project conditions and the remaining possibility for loss of life. The below sample stage-time plot with a

significant damage elevation set to 4 ft. NGVD 1929 presents visual interpretation of warning time, rate of rise, and duration.



Sample Interior Stage-Time Plot

8.2.1 Warning Time of Impending Inundation

212. The start point for the warning times listed below in Table 44 begins at the inflection point on the stage storage curve where the instantaneous change in stage begin to accelerate. In effect, this point in time is when the increase in exterior tide level begins blocking outflow through the stormwater outfalls and the stormwater conveyance system reaches full capacity. Prior to this point in time, there is only a steady and slight change in interior flood stages during an extended period of initial rainfall. The end value for the warning time function is the time when the interior stage equals the established significant damage elevation. Typically the more severe the event, the shorter the warning time.

TABLE 44 WARNING TIME

Drainage Area	Warning time (hours-minutes)		
	10-yr Event	50-yr Event	100-yr Event
Area A	4hr 55min	4hr 20min	4hr 20min
Area B	6hr 05min	5hr 35min	5hr 30min
Area C	7hr 10min	5 hr 30min	5hr 10min
Area D	5hr 05min	5hr 05min	5hr 05min
Area E	5hr 55min	5hr 15min	5hr 10min

8.2.2 Rate of Rise and Duration of Flooding

213. Information on the rate of rise for the 10-year, 50-year, and 100-year storm events, which measures the rate of change in flood levels per minute, is presented in Table 45. The rate is an average speed value from the time where the flood stage first reaches the significant damage elevation until it reaches the peak flood stage.

TABLE 45 RESIDUAL FLOODING RATE OF RISE

Drainage Area	Rate of Rise (in/min)		
	10-yr Event	50-yr Event	100-yr Event
Area A	0.48	0.34	0.31
Area B	0.08	0.24	0.29
Area C	0.07	0.12	0.16
Area D	0.24	0.17	0.13
Area E	0.10	0.31	0.41

214. The amount of time where the flood stage is above the significant damage elevation, or duration of flooding, is presented in Table 46. Here the duration of flooding is controlled by the tide, which blocks the outfalls when the exterior stage is increased above the elevation of the outfall.

TABLE 46 RESIDUAL FLOOD DURATION

Drainage Area	Duration (min)		
	10-yr Event	50-yr Event	100-yr Event
Area A	190	230	245
Area B	120	175	190
Area C	115	380	480
Area D	95	215	265
Area E	155	265	300

8.2.3 Access and Egress Problems & Impacts to Public Services

215. For more frequent storm events (e.g. 2-yr or 5-yr event), local property owners may still experience some local road closures and access issues. For events that produce higher rainfall

and or coastal surge, Hylan Boulevard and other main thoroughfares can be expected to experience some level of inundation. The coastal surge from the 500-yr event will cause extensive road closures and inundation of public facilities throughout the study area, starting from the shoreline and reaching all the way past Hylan Boulevard for a majority of the study area. An overlay of the residual flooding extents on aerial imagery is presented in Figures 44-49.

216. The Oakwood Beach Waste Water Treatment Plant is not shown to be susceptible to residual flooding from Interior Drainage; however, if a rare storm event was to occur such as the 500-yr event, there is a chance that the Wastewater Treatment Plant would become inundated by coastal surge and would cease to serve its function.

8.2.4 Potential Loss of Life

217. The implementation of the NED Plan will not eliminate the potential for loss of life. The NED Plan will reduce the frequency of flooding from Bay surge reaching the structures in the study area and therefore individuals. Instead of high velocity overtopping flows from the coast, the Interior Drainage Areas will experience pools of water in low-lying areas from surface run-off. Interior Drainage flooding is predicted to have waters that rise over two feet per hour in some areas, which may generate life safety risks in addition to those created by the depth of flooding alone.

218. A coastal storm event that produces surges that exceed the capacity of the Line of Protection stillwater design, will create a situation similar to Hurricane Sandy (October 29-30, 2012). Fourteen residents from the study area lost their lives during Sandy after record surge levels overtopped the existing coastal barrier.

8.2.5 Residual Flood Related Damages

219. There are a number of structures within the study area that are still at risk of being inundated during the with project condition. The with and without-project count of structures inundated by frequency and Drainage Area are presented in Table 47. The with project equivalent annual damages, in dollar values, are presented in Table 48.

TABLE 47 STRUCTURES SUBJECT TO RESIDUAL FLOODING

Drainage Area	**Number of Structures Flooded**					
	10-yr Event		**50-yr Event**		**100-yr Event**	
	Without Project	**With Project**	***Without Project***	**With Project**	***Without Project***	**With Project**
Area A	*20*	8	*198*	11	*287*	15
Area B	*335*	11	*962*	11	*1,144*	33
Area C	*1,325*	95	*2,402*	334	*2,579*	337
Area D	*11*	11	*149*	33	*212*	33
Area E	*171*	34	*408*	43	*460*	43
Totals	***1,862***	**159**	***4,119***	**432**	***4,682***	**461**

TABLE 48 RESIDUAL FLOOD DAMAGE

Drainage Area	**Equivalent Annual Damage**
Drainage Area A – Minimum Facility	$85,000
Drainage Area B – Minimum Facility	$115,890
Drainage Area C – Alternative 4: 377,200 cy, 6 Ponds	$1,255,600
Drainage Area D – Minimum Facility	$135,500
Drainage Area E – Alternative 2: 222,720 cy, 4 Ponds	$288,800
Total With Project Damage	**$1,875,600**

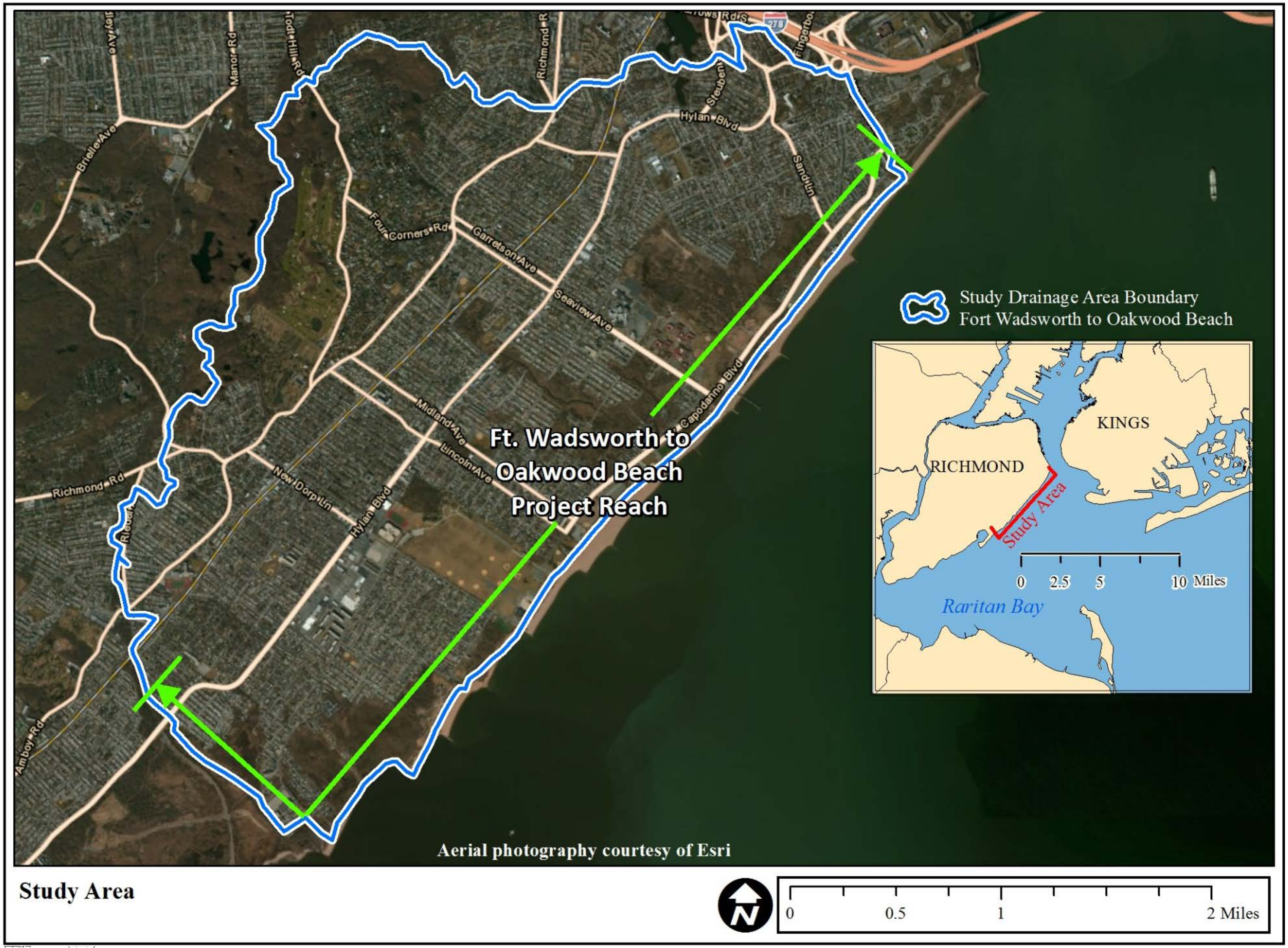
9 CONCLUSION/SUMMARY

220. The Line of Protection Alternative recommended in National Economic Development (NED) Plan for the South Shore of Staten Island Interim Feasibility Report will be the first line of defense against significant coastal surge and wave action. However, if the design were implemented in absence of any interior drainage measures, the plan would not meet Minimum Facility Criteria; the project area would still experience extensive damages to properties, and would have experienced increased Water Surface Elevations in some Interior Drainage Areas. Areas A, and D have implemented interior drainage measures so as to ensure that the overall project would not induce flooding. For Areas C and E, the local flooding damage experienced in the range of studied storm frequency events was severe enough to justify the cost of the construction of excavated ponds to store interior run-off, effectively lowering Water Surface Elevations. Area B has implemented interior drainage measures so as to ensure that this area will not induce flooding below a 10-year event. At a 100-year event flooding is in increased by 0.18 feet. However the interior water level is still lower than without project conditions.

221. The Tentatively Selected Interior Drainage Plans herein will aid in the discharge or controlled storage of interior floodwaters during low frequency precipitation events. Together with the Tentatively Selected Line of Protection Plan, this complimentary system will provide coastal storm risk management in the study area for the two most common forms of severe storm events, Hurricanes and Nor'easters. Figures 44-49, are a visual interpretation of how the inundation extents are expected to change with the introduction of the management measures as part of the NED Plan design. Seaward of the Line of Protection the exterior coastal stage will remain unchanged. Landward of the Line of Protection alignment, the Residual Flood Maps depict a significant retreat in the 100-yr flood extents when compared to the without project inundation conditions.

222. The NED Plan, however, will not eliminate all coastal flooding or Interior Drainage flooding within the study limits along the South Shore of Staten Island. As visible on the Residual Flood Map Figures, the 100-yr event will still result in some localized flooding behind the plan alignment. The Residual Flood Maps along with the Residual Flood analysis will help local officials and property owners better understand the change in risk and may add value to local flood management plans or ordinances after the project is completed.

Draft Interior Drainage Appendix
Attachment 1: Figures



Ft. Wadsworth to
Oakwood Beach
Project Reach
Study Drainage Area Boundary
Fort Wadsworth to Oakwood Beach
KINGS
RICHMOND
Study Area
0 2.5 5 10 Miles
Raritan Bay
Aerial photography courtesy of Esri
Manor Rd
Todt Hill Rd
Richmond Rd
Hylan Blvd
Four Corners Rd
Garretson Ave
Seaview Ave
Midland Ave
Lincoln Ave
New Dorp Ln
Capodanno Blvd
Richmond Rd
Amboy Rd
Brielle Ave
Sand Ln
0 0.5 1 2 Miles

Study Area

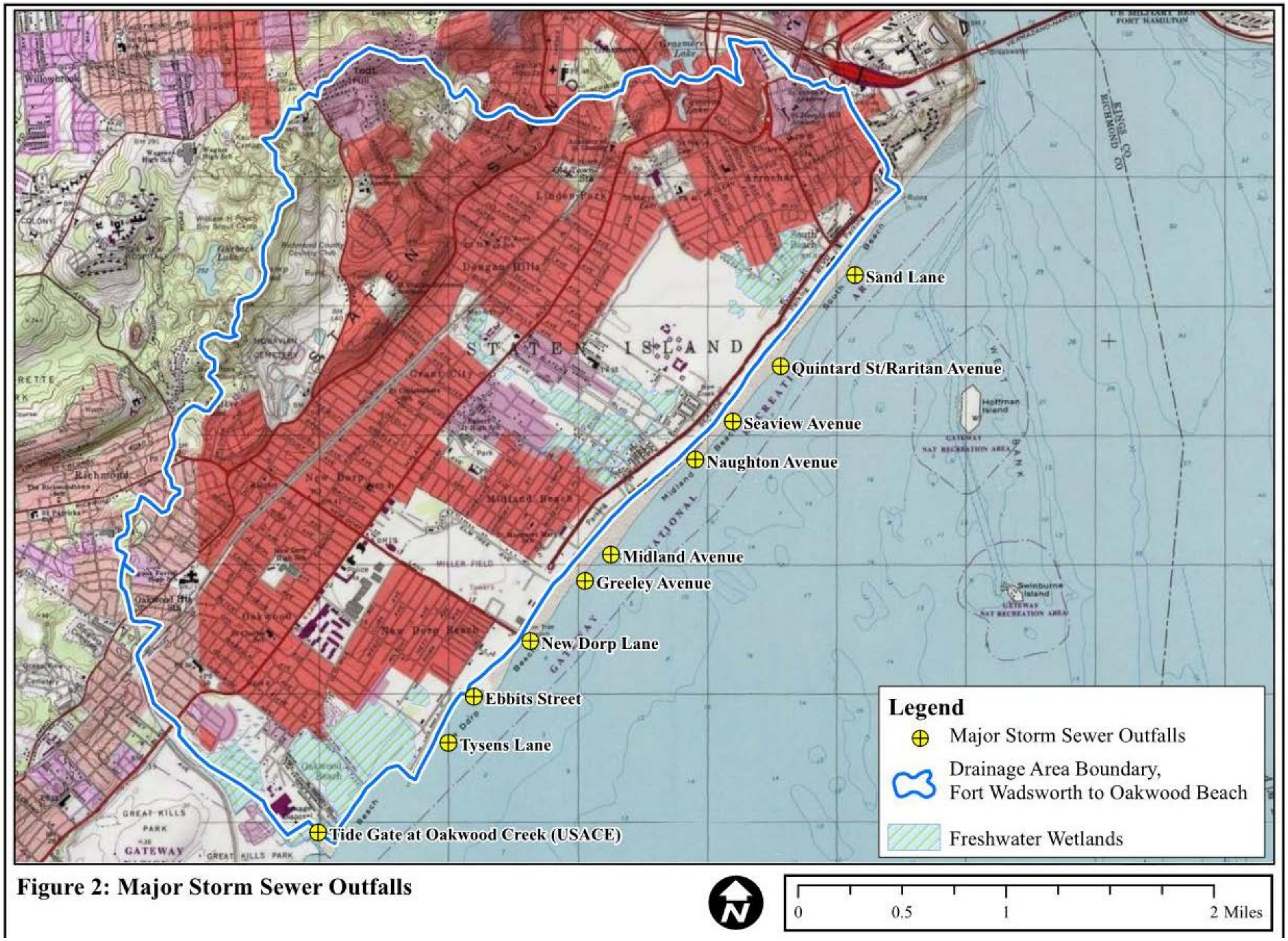
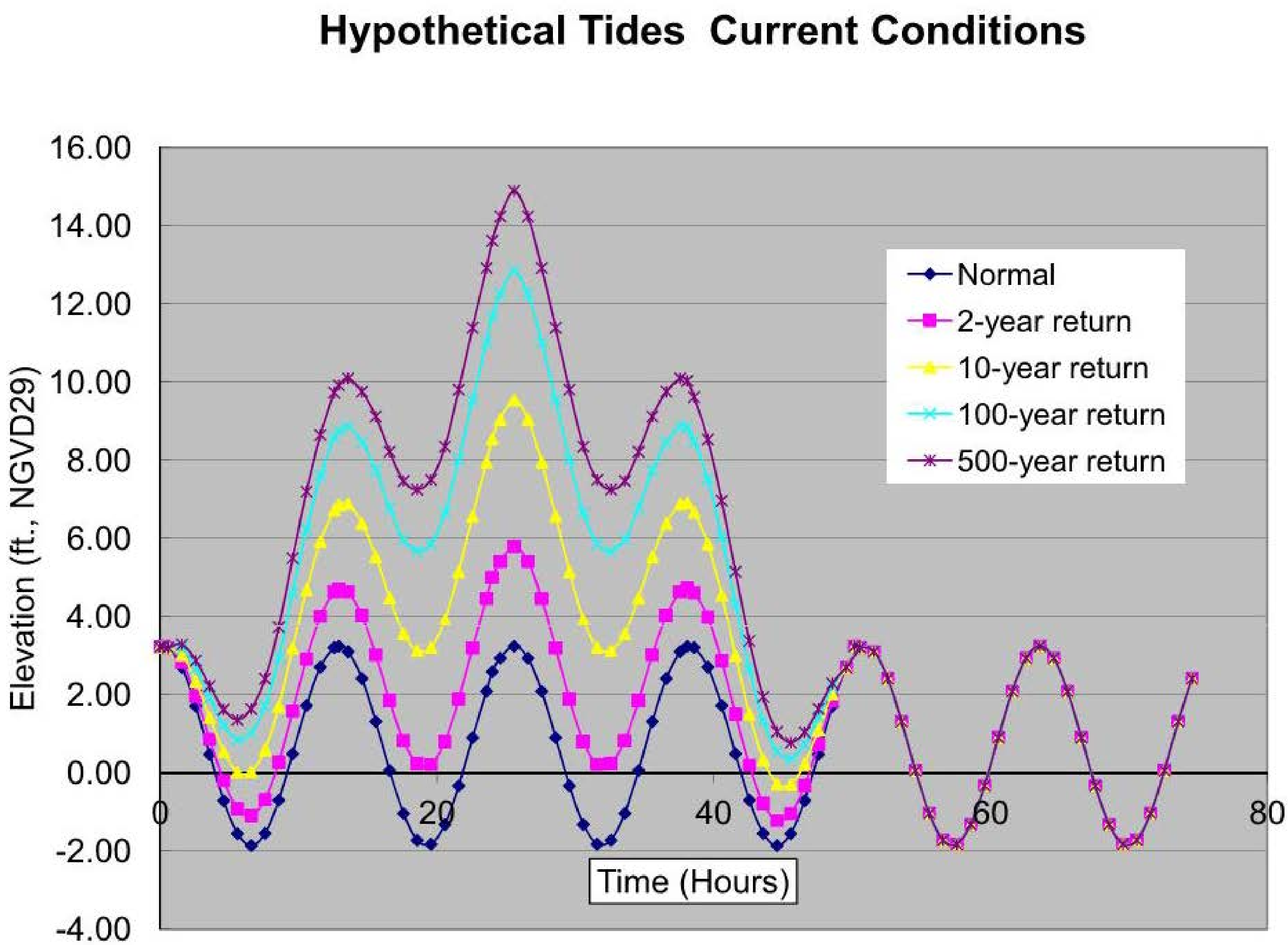


Figure 2: Major Storm Sewer Outfalls



Hypothetical Tide Current Condition

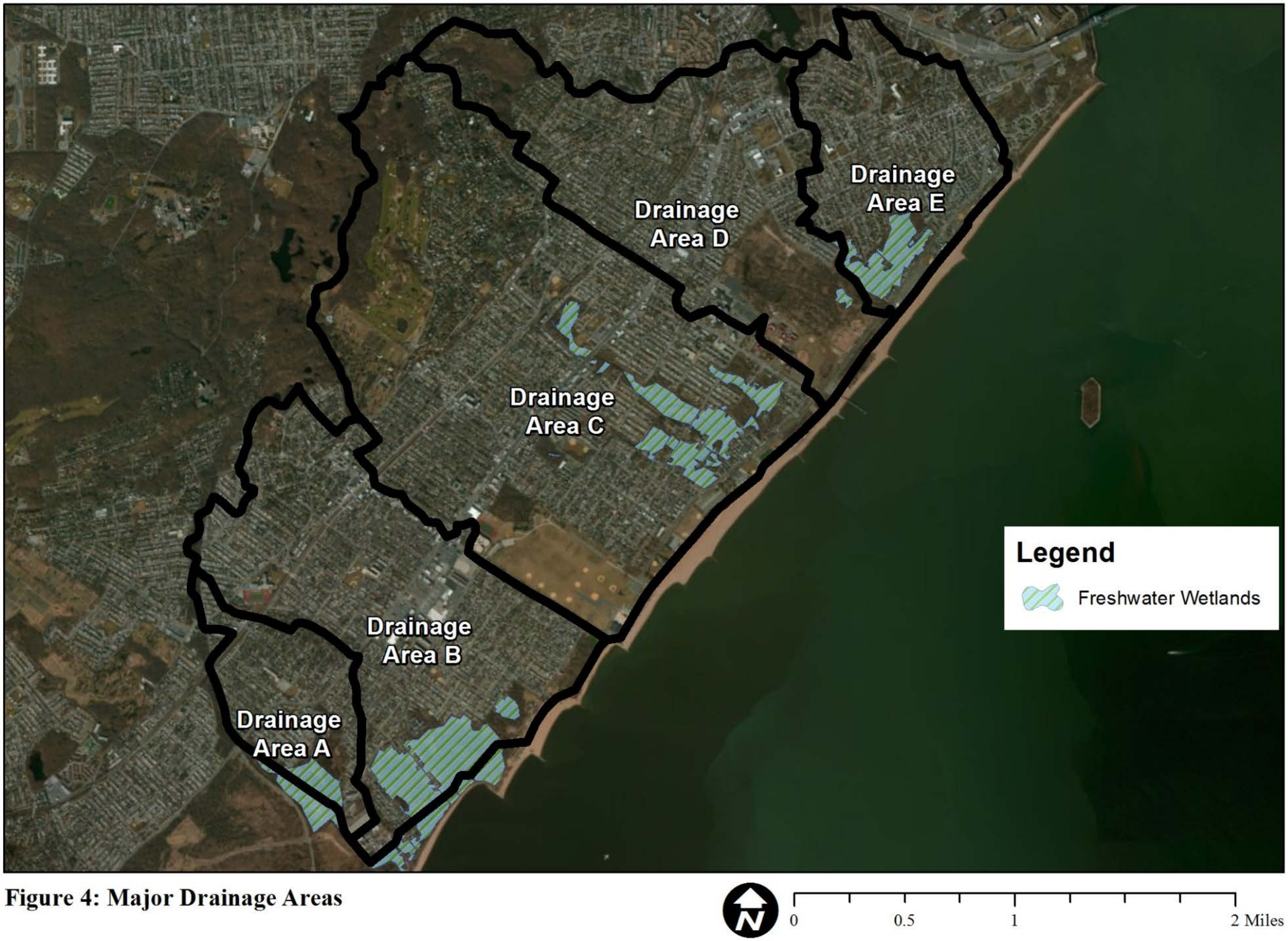


Figure 4: Major Drainage Areas

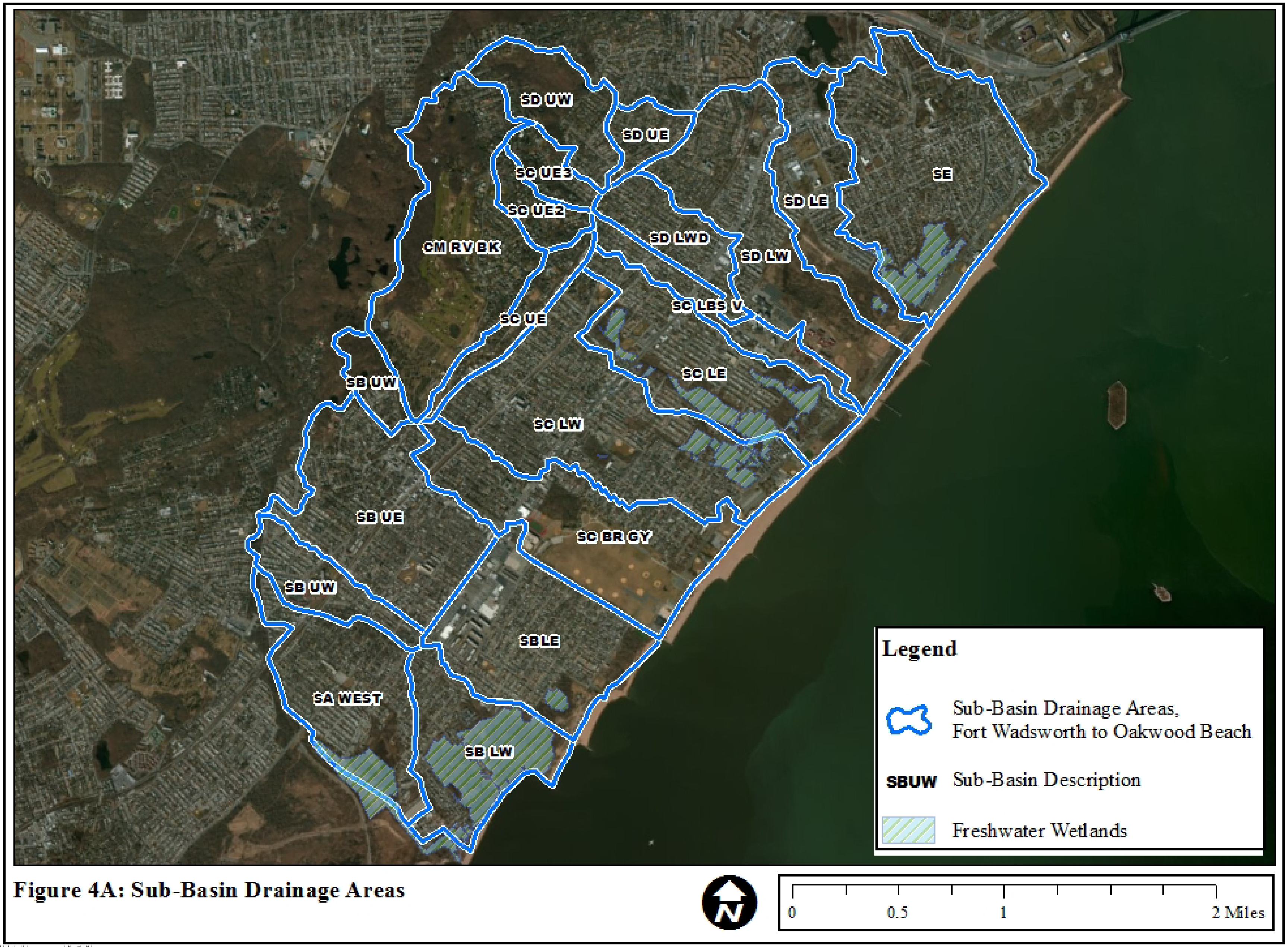
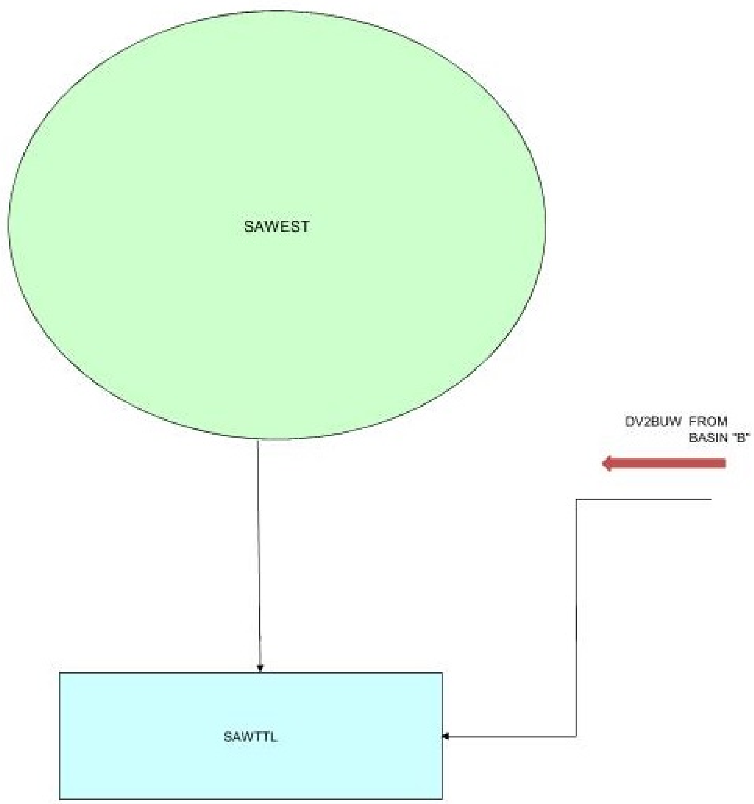
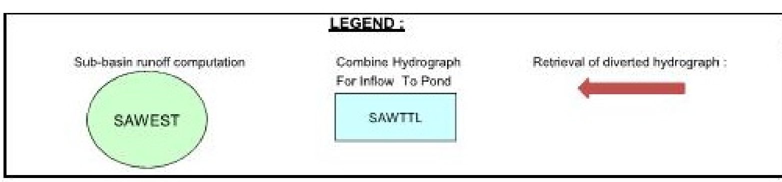


Figure 4A: Sub-Basin Drainage Areas





South Shore Staten Island N.Y.
RICHMOND COUNTY, NEW YORK
STORM DAMAGE REDUCTION STUDY
HYDROLOGY: EXISTING CONDITIONS

HEC-HMS MODEL SCHEMATIC : BASIN A

FIGURE 5

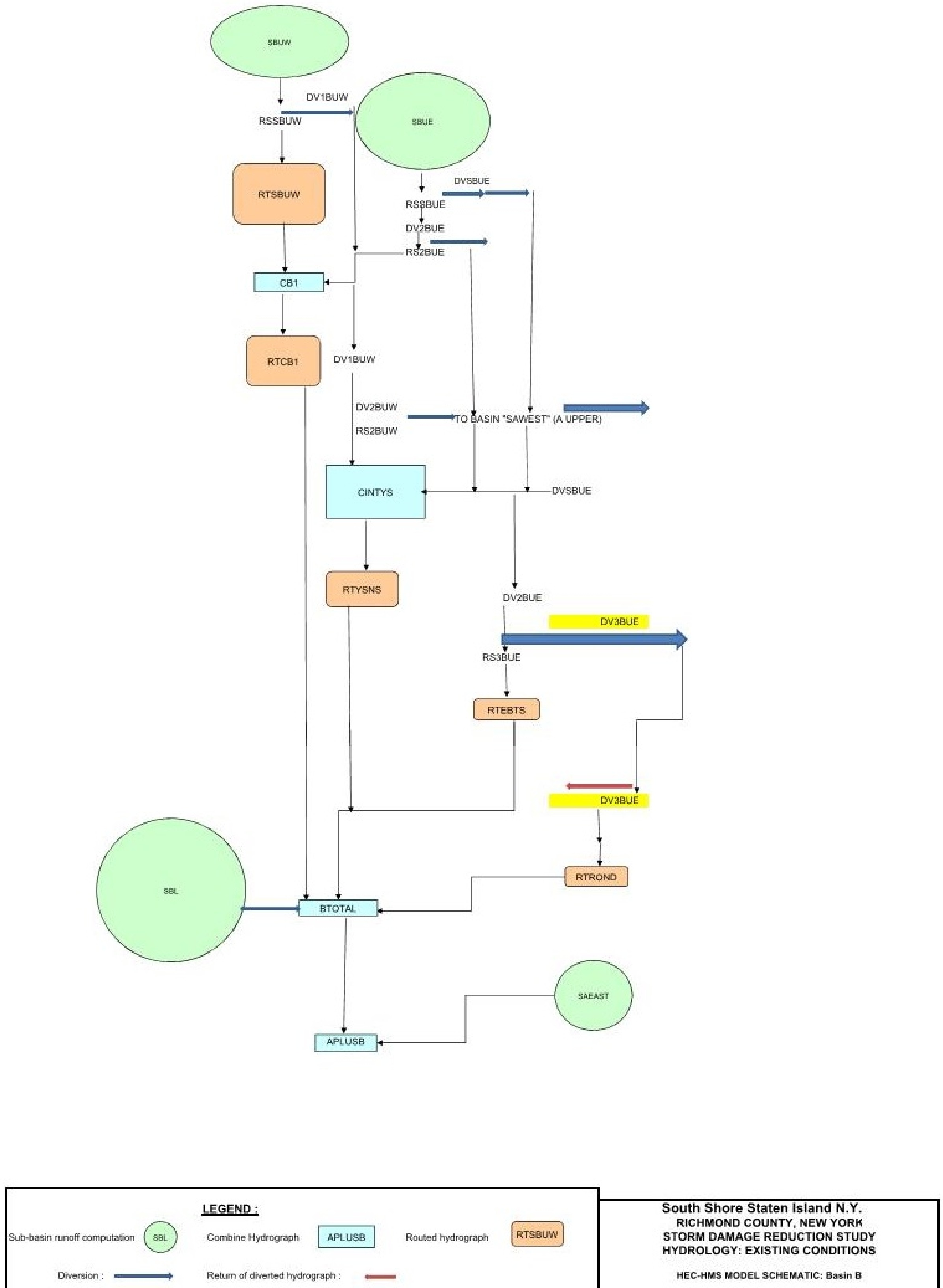
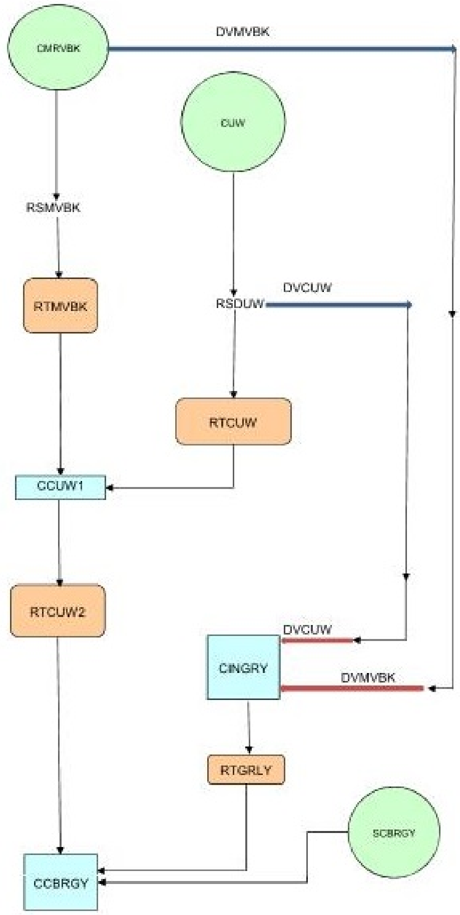
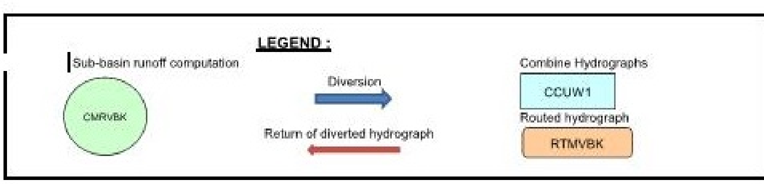

SBUW
DV1BUW
RSSBUW
SBUE
RTSBUW
DVSBUE
RSSBUE
DV2BUE
RS2BUE
CB1
RTCB1
DV1BUW
DV2BUW
RS2BUW
TO BASIN "SAWEST" (A UPPER)
CINTYS
DVSBUE
RTYSNS
DV2BUE
DV3BUE
RS3BUE
RTEBTS
DV3BUE
RTROND
SBL
BTOTAL
SAEAST
APLUSB
LEGEND :
Sub-basin runoff computation
SBL
Combine Hydrograph
APLUSB
Routed hydrograph
RTSBUW
Diversion :
Return of diverted hydrograph :
South Shore Staten Island N.Y.
RICHMOND COUNTY, NEW YORK
STORM DAMAGE REDUCTION STUDY
HYDROLOGY: EXISTING CONDITIONS
HEC-HMS MODEL SCHEMATIC: Basin B

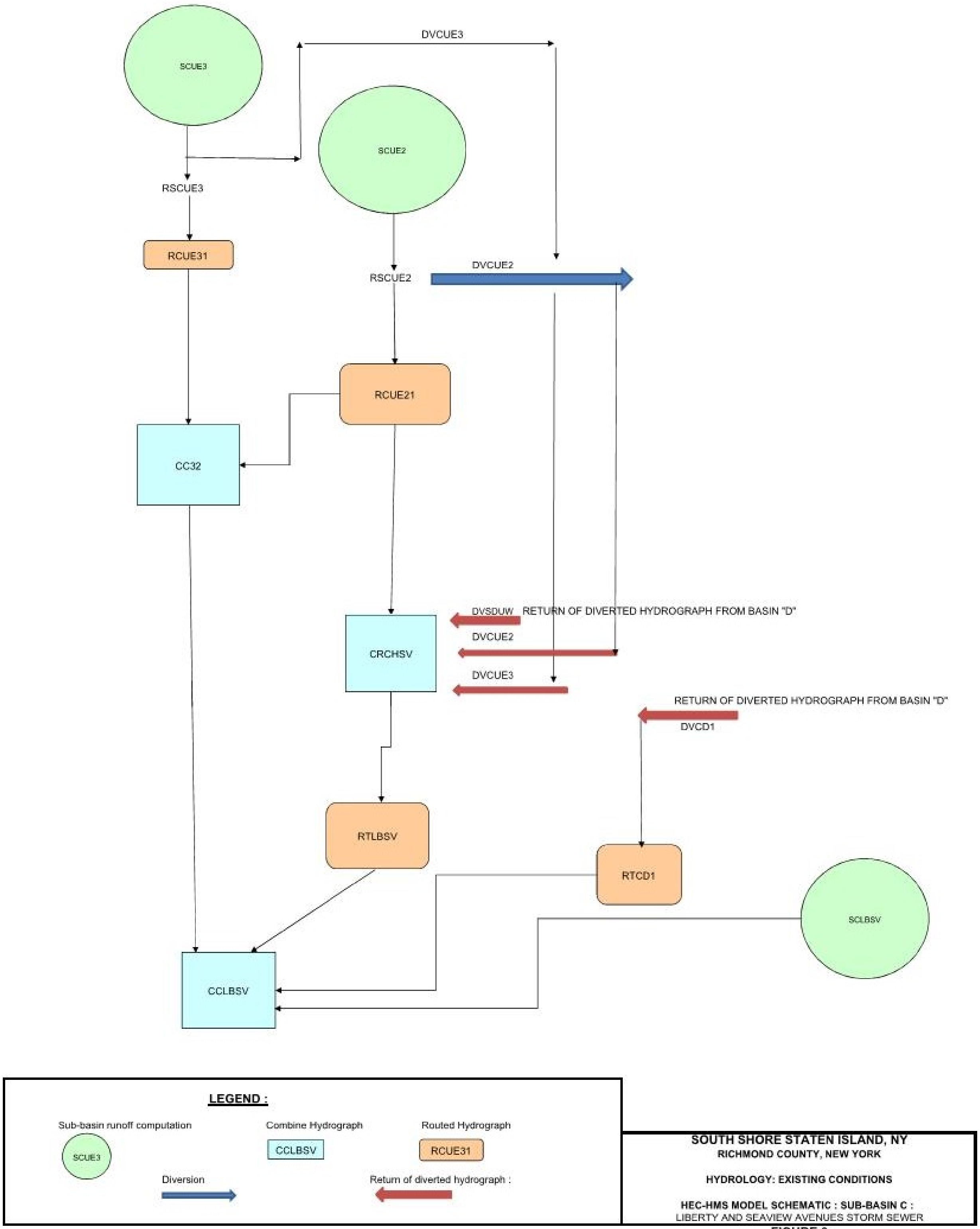
FIGURE 6





South Shore Staten Island N.Y.
RICHMOND COUNTY, NEW YORK
STORM DAMAGE REDUCTION STUDY
HYDROLOGY: EXISTING CONDITIONS

HEC-HMS MODEL SCHEMATIC :
AREA C : INFLOW TO BRYANT AND GREELEY AVENUES STORM SEWER
FIGURE 7

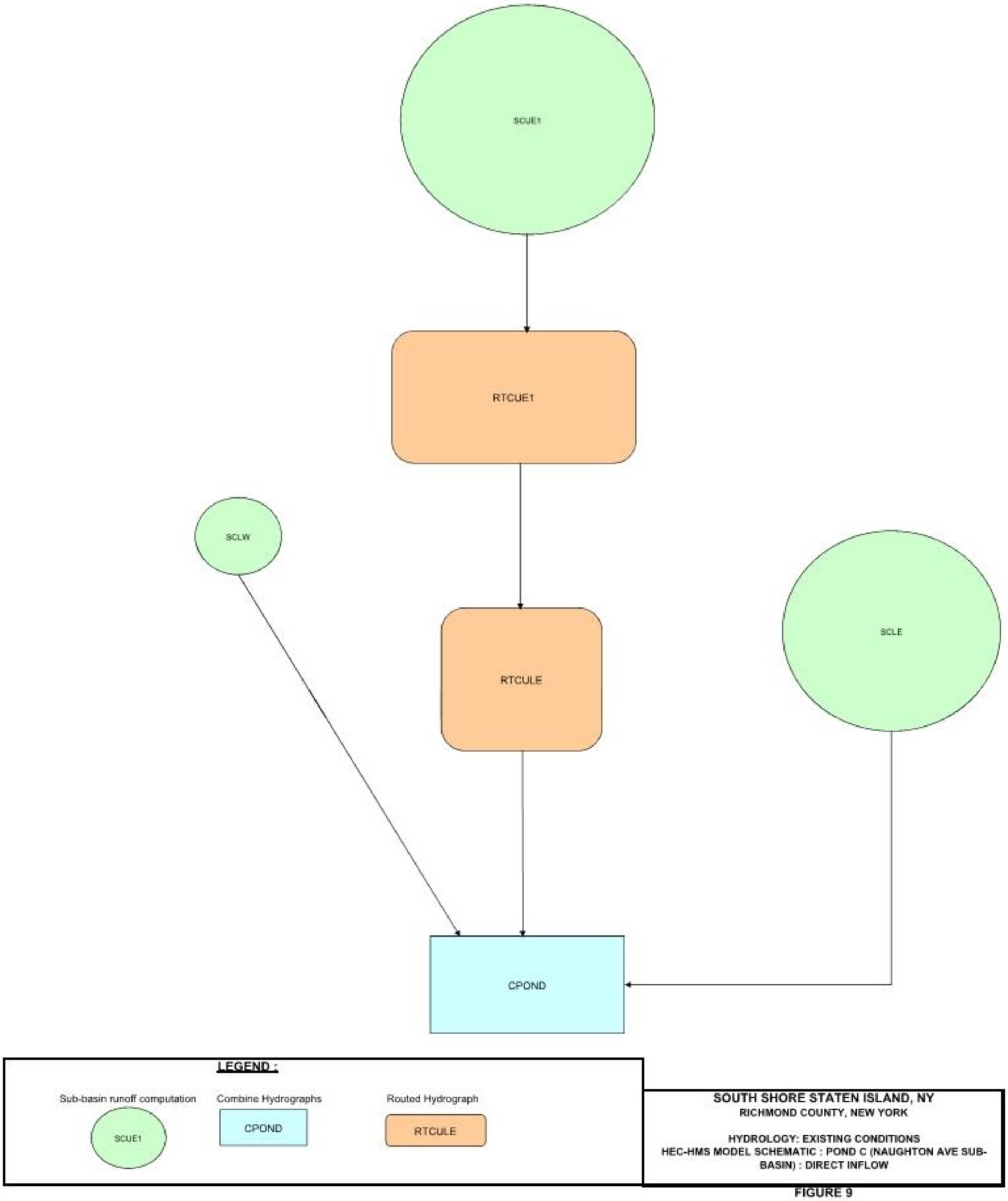


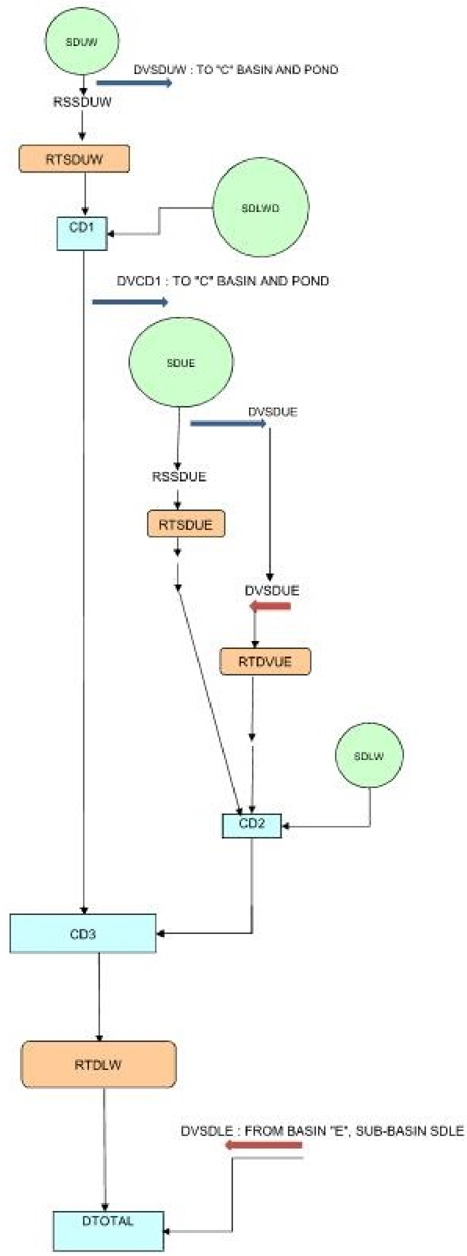
SOUTH SHORE STATEN ISLAND, NY
RICHMOND COUNTY, NEW YORK

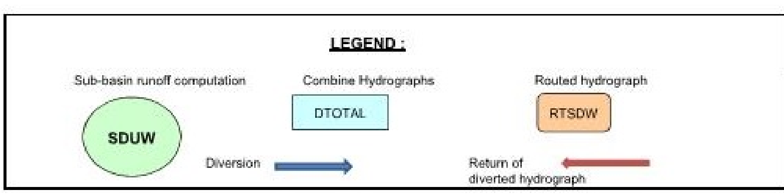
HYDROLOGY: EXISTING CONDITIONS

HEC-HMS MODEL SCHEMATIC : SUB-BASIN C :
LIBERTY AND SEAVIEW AVENUES STORM SEWER

FIGURE 8


SCUE1
RTCUE1
SCLW
RTCULE
SCLE
CPOND
LEGEND :
Sub-basin runoff computation
SCUE1
Combine Hydrographs
CPOND
Routed Hydrograph
RTCULE
SOUTH SHORE STATEN ISLAND, NY
RICHMOND COUNTY, NEW YORK
HYDROLOGY: EXISTING CONDITIONS
HEC-HMS MODEL SCHEMATIC : POND C (NAUGHTON AVE SUB-BASIN) : DIRECT INFLOW
FIGURE 9

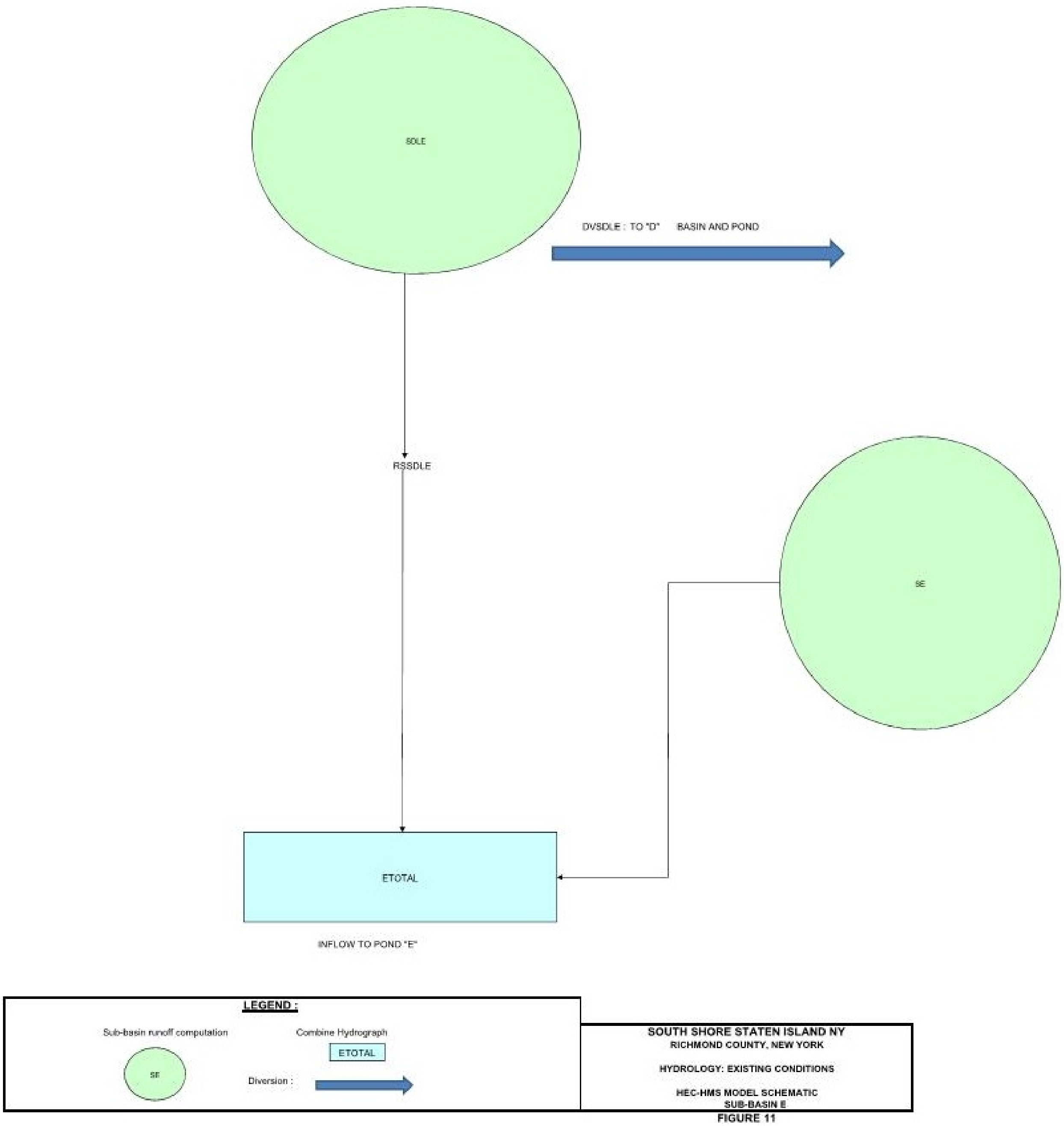


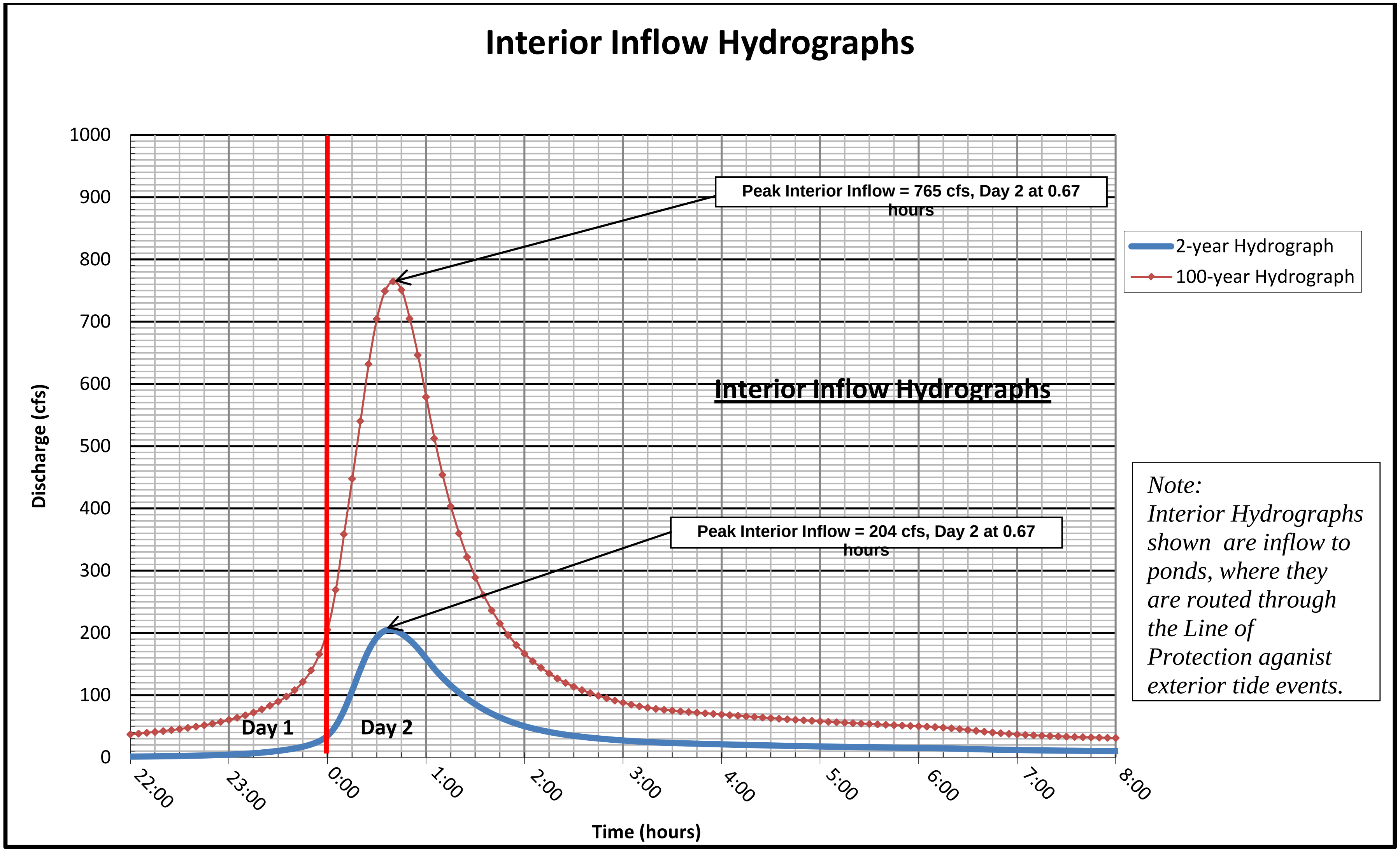


SOUTH SHORE STATEN ISLAND NY
RICHMOND COUNTY, NEW YORK

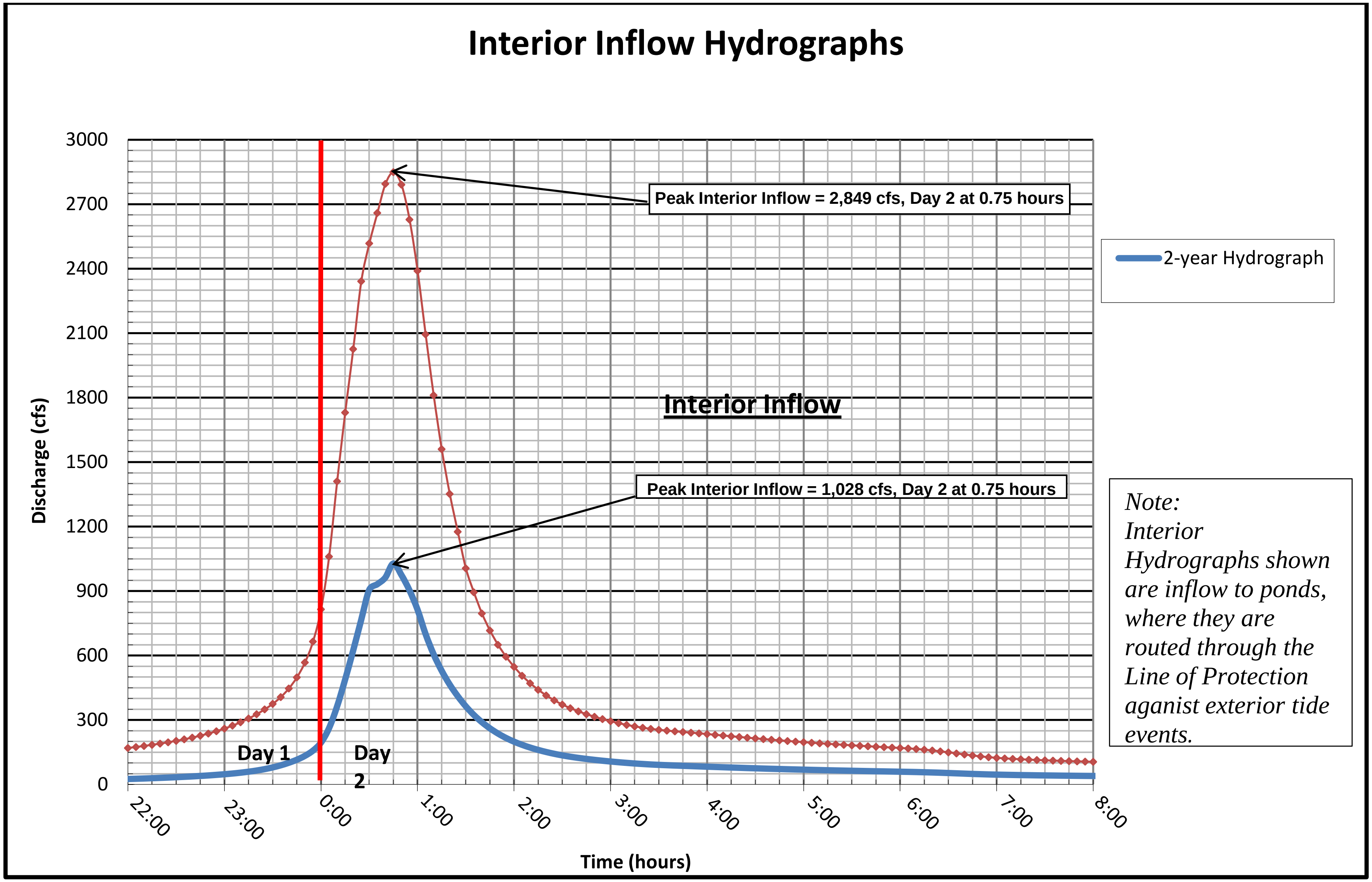
HYDROLOGY: EXISTING CONDITIONS

HEC-HMS MODEL SCHEMATIC
SUB-BASIN D
FIGURE 10

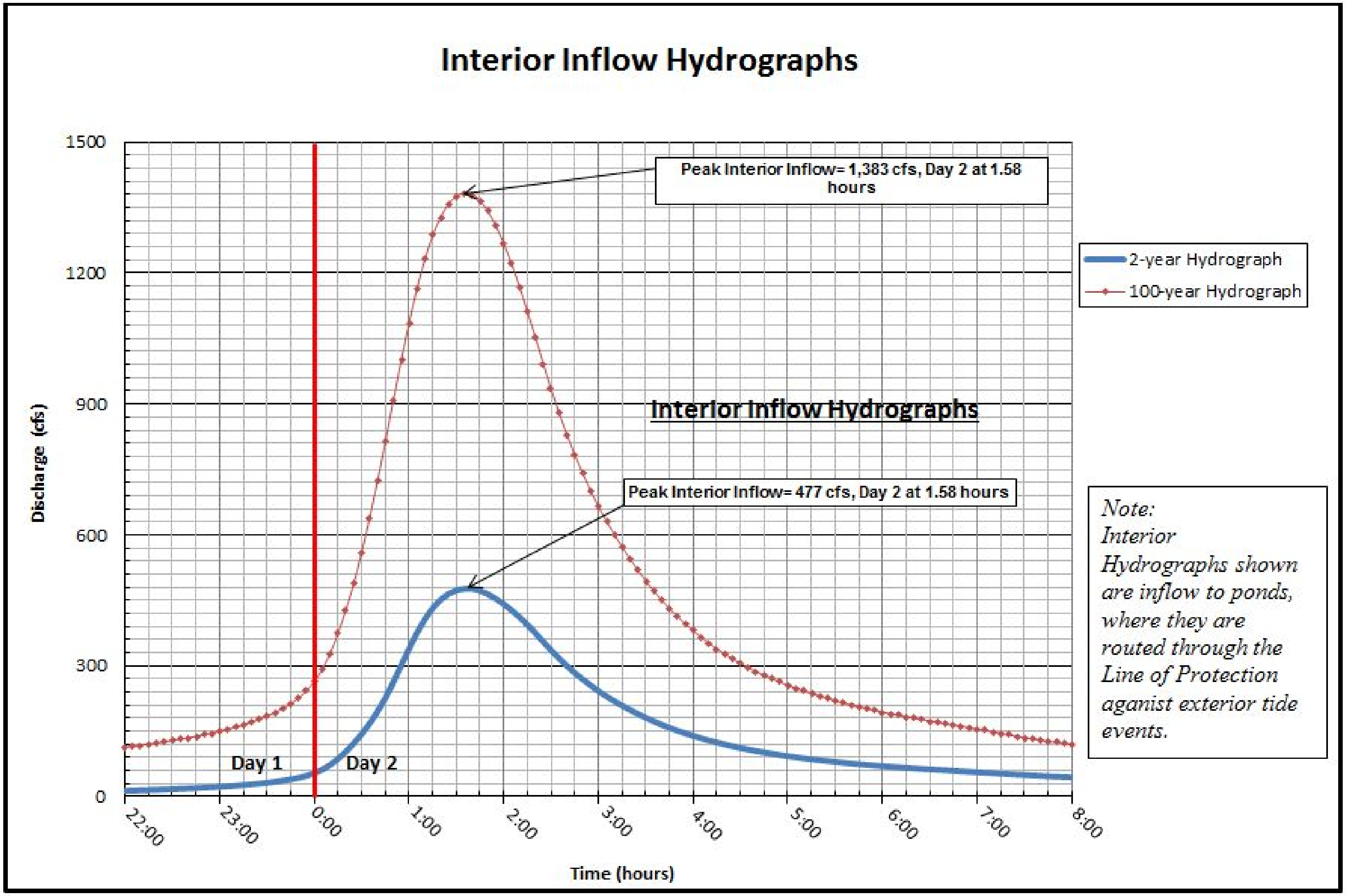

SDLE
DVSDLE : TO "D" BASIN AND POND
RSSDLE
SE
ETOTAL
INFLOW TO POND "E"
LEGEND :
Sub-basin runoff computation
Combine Hydrograph
ETOTAL
SE
Diversion :
SOUTH SHORE STATEN ISLAND NY
RICHMOND COUNTY, NEW YORK
HYDROLOGY: EXISTING CONDITIONS
HEC-HMS MODEL SCHEMATIC
SUB-BASIN E
FIGURE 11



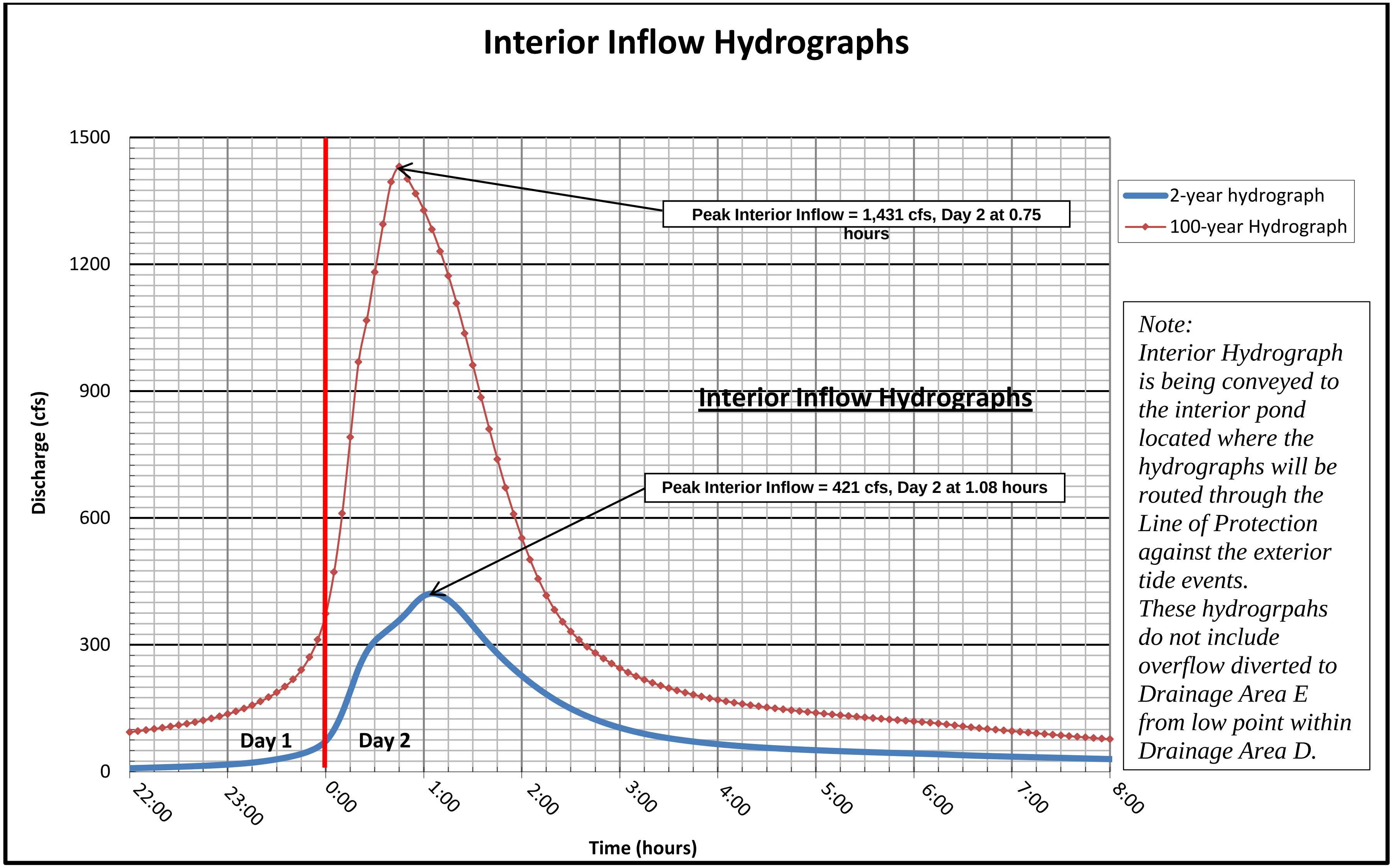
Interior Inflow Hydrographs for Drainage Area A



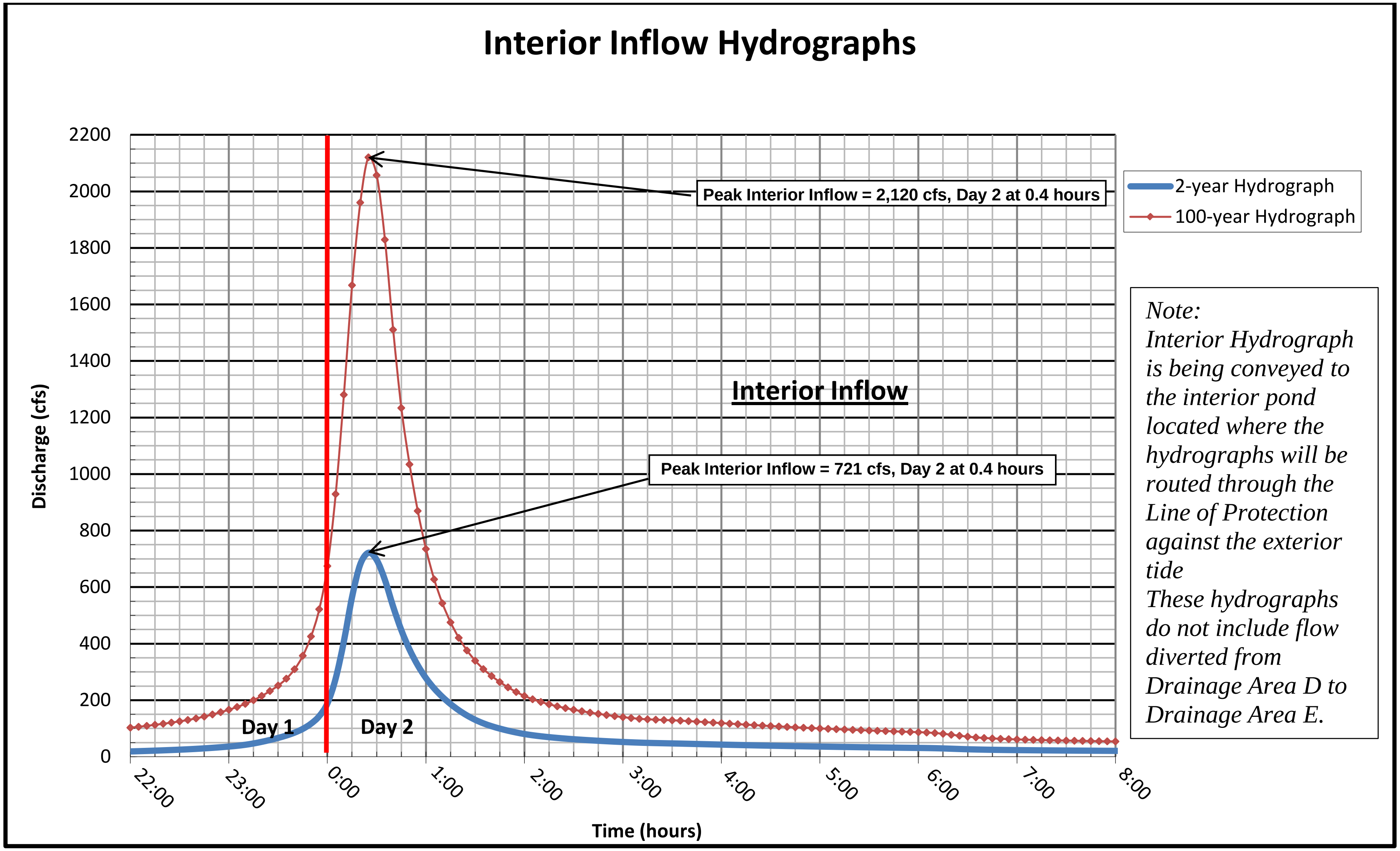
Interior Inflow Hydrographs for Drainage Area B



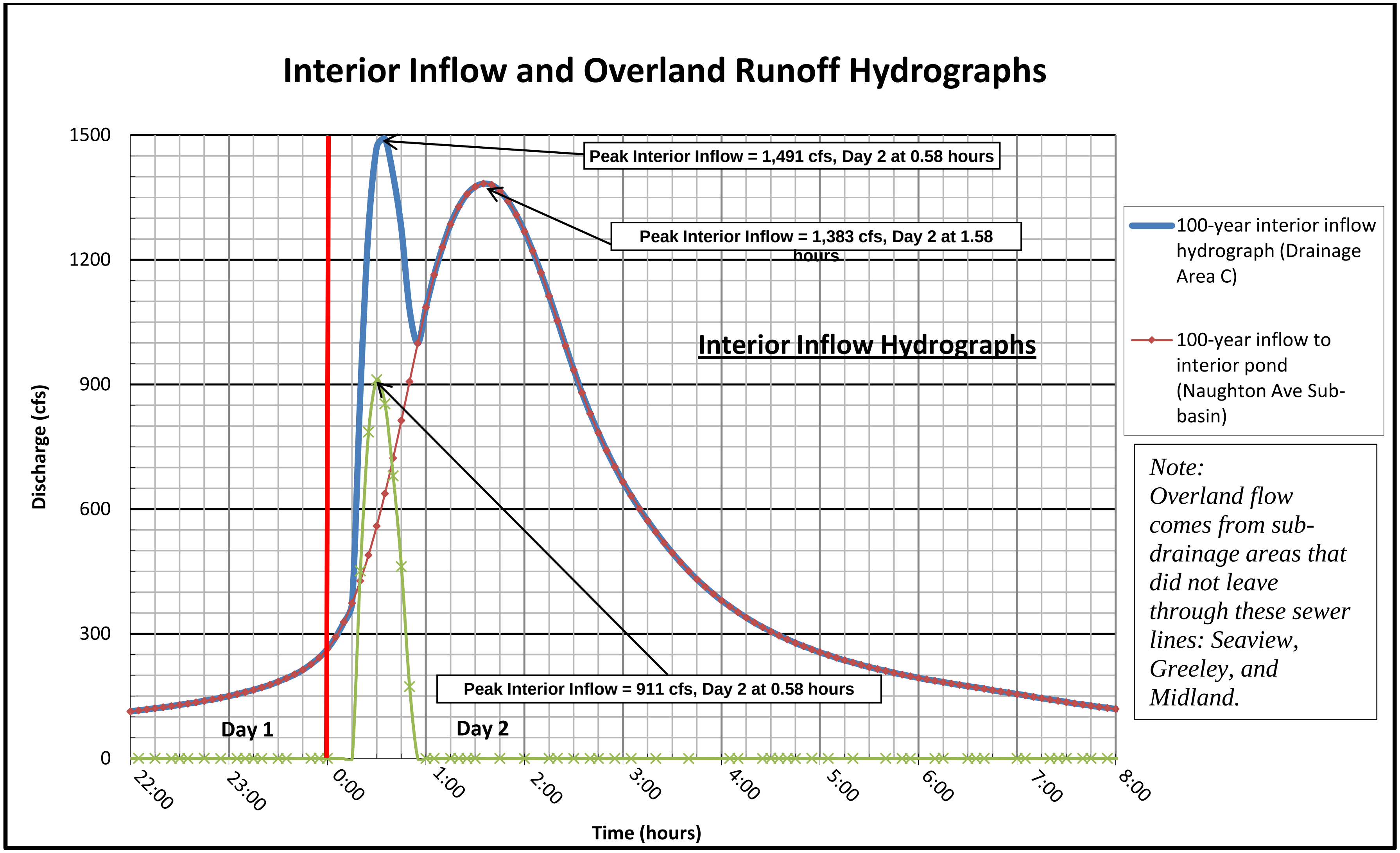
Interior Inflow Hydrographs for Drainage Area C (Naughton Avenue Sub-basin only)



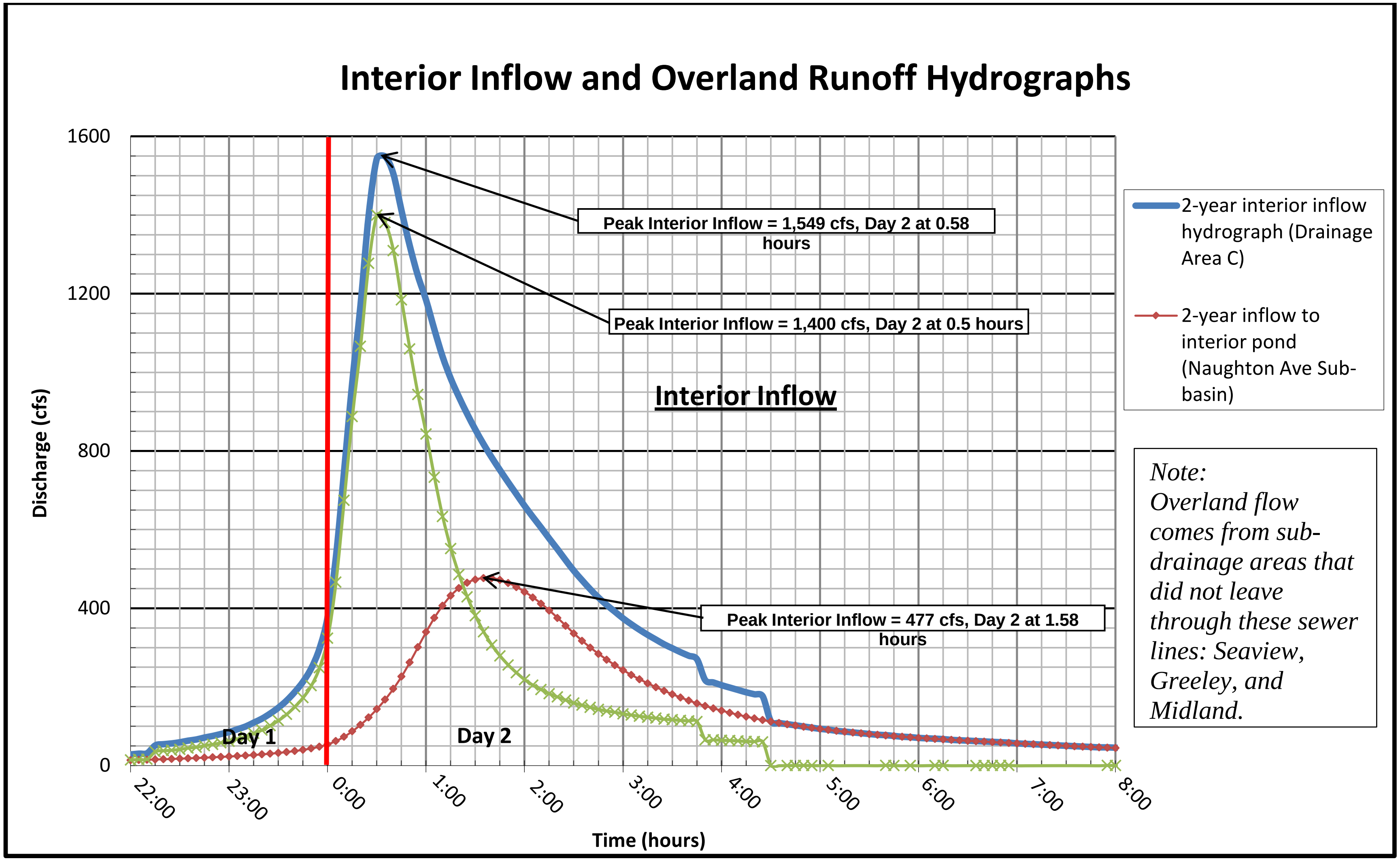
Interior Inflow Hydrographs for Drainage Area D



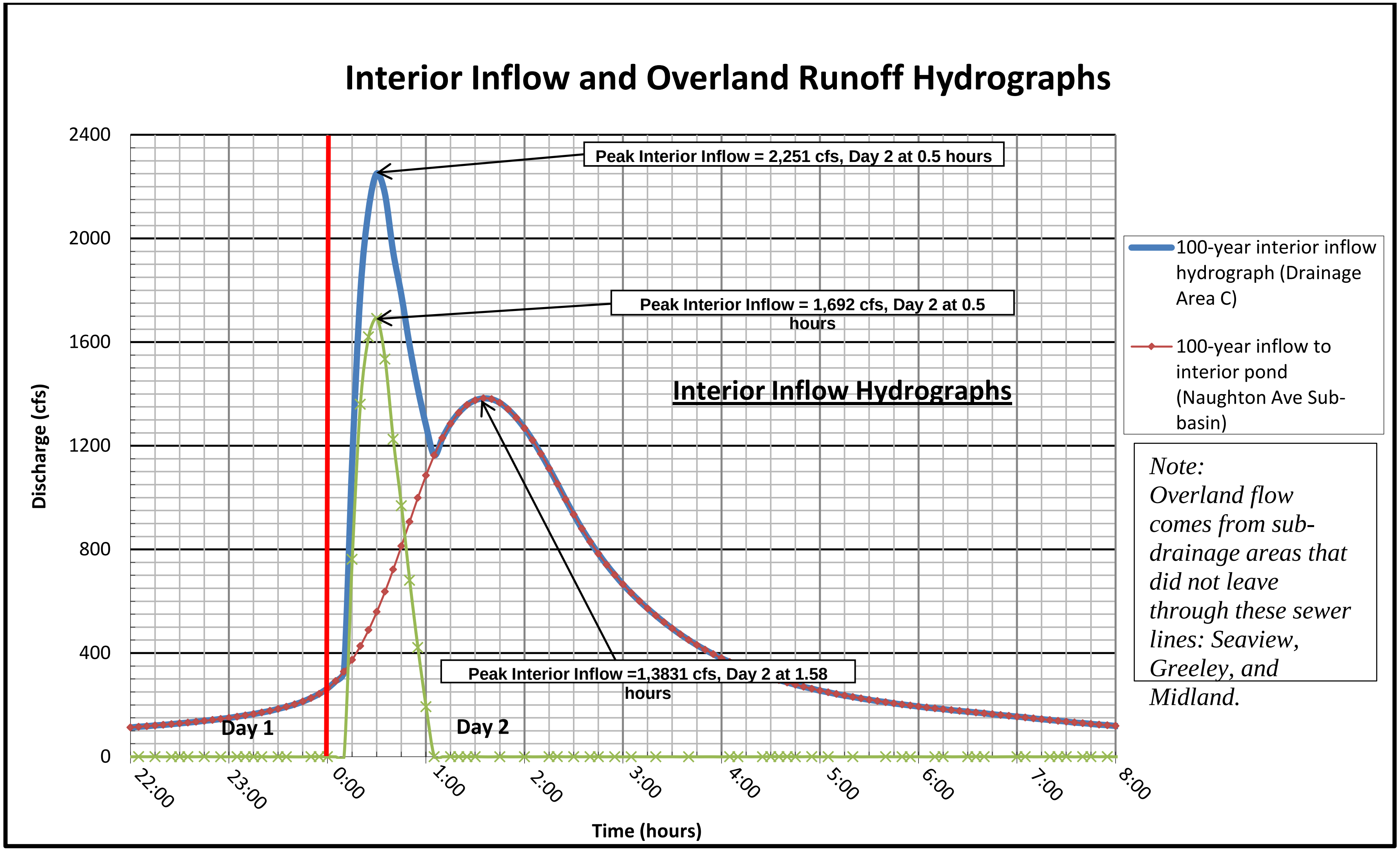
Interior Inflow Hydrographs for Drainage Area E



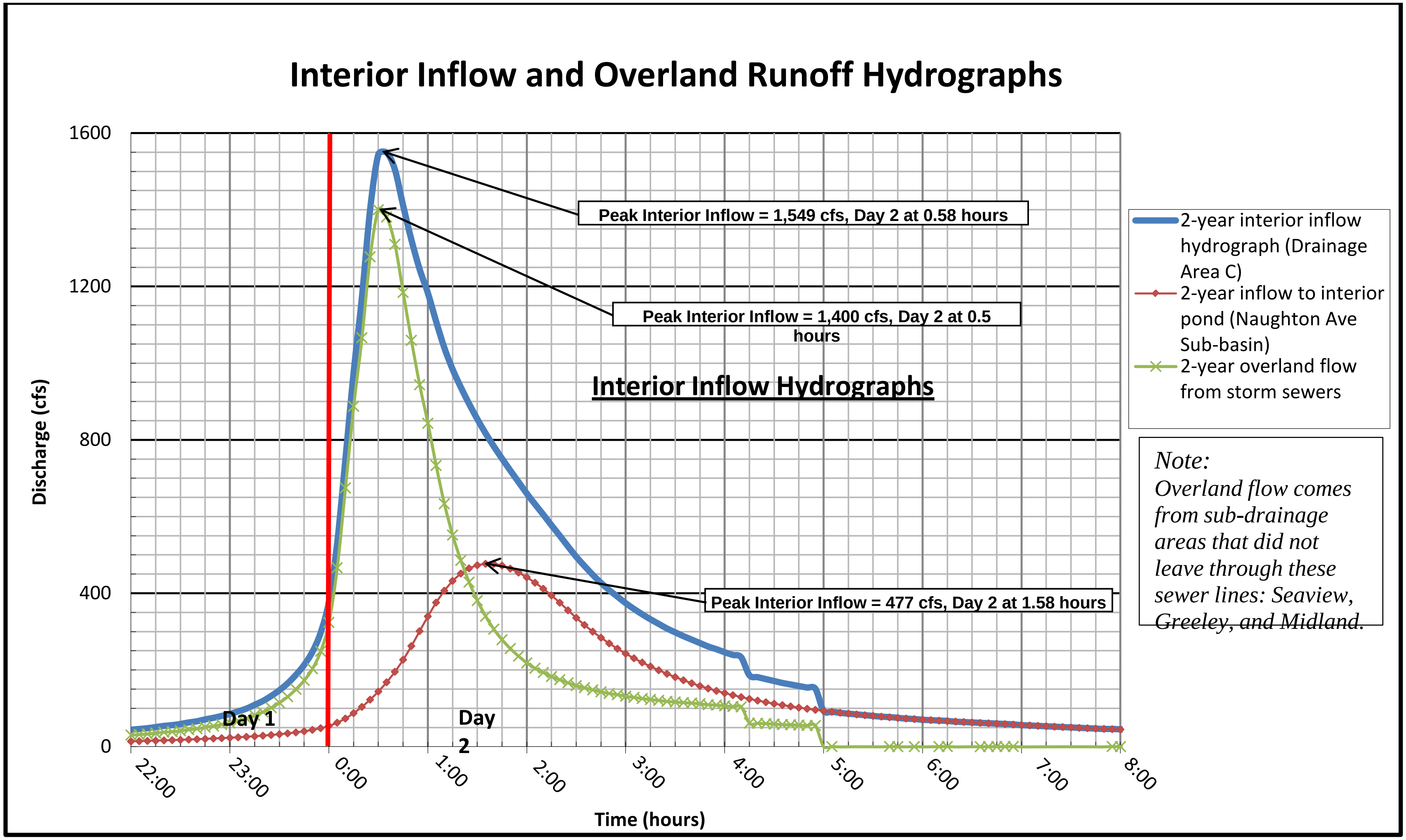
Interior Inflow Hydrographs and Storm Sewer Overland Flows for Drainage Area C (100-year interior hydrograph routed against 2-year stage (Current Tide) hydrograph)



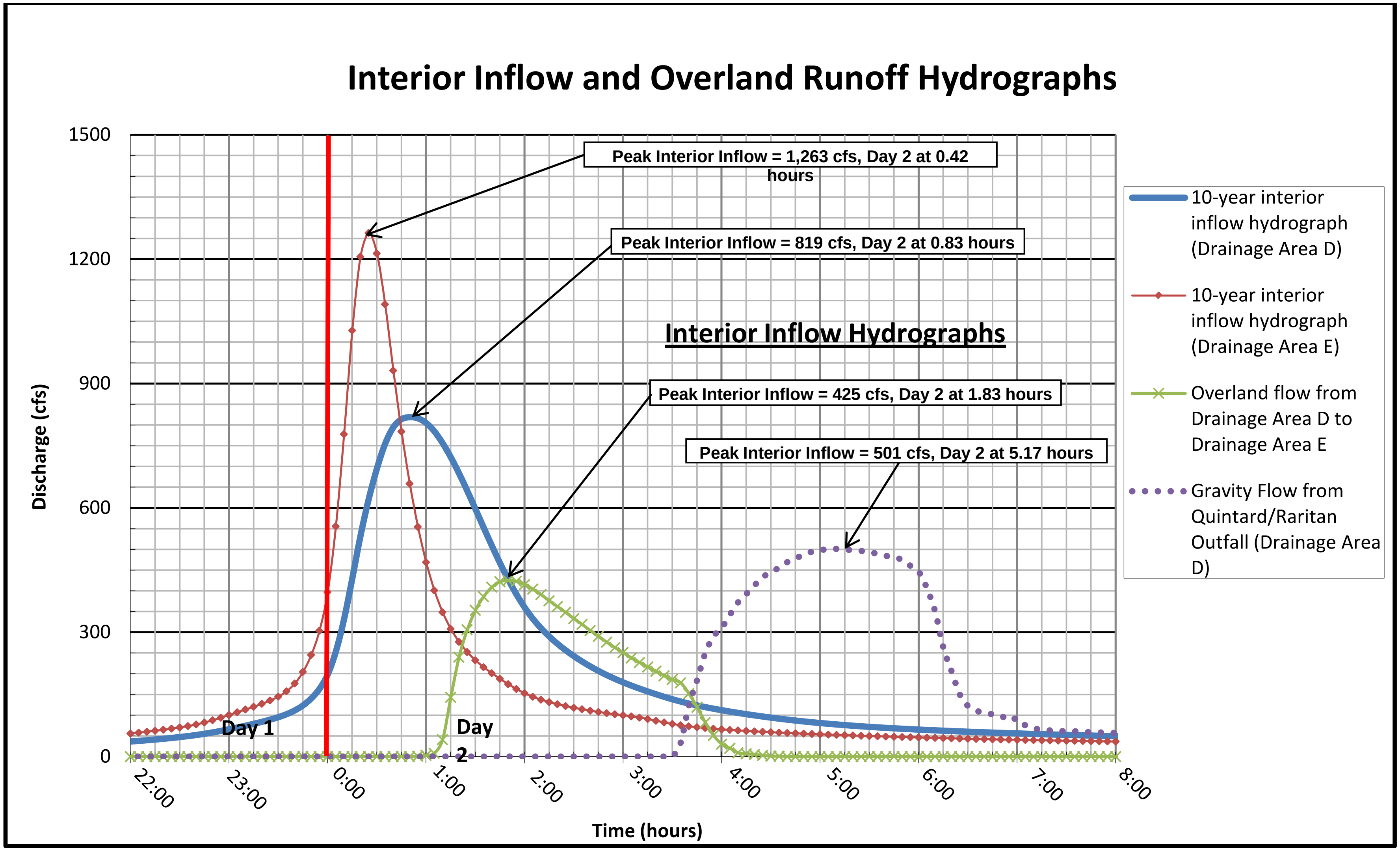
Interior Inflow Hydrographs and Storm Sewer Overland Flows for Drainage Area C (2-year interior hydrograph routed against 100-year stage (Current Tide) hydrograph)



Interior Inflow Hydrographs and Storm Sewer Overland Flows for Drainage Area C (100-year interior hydrograph routed against 2-year stage (Future Tide) hydrograph)



Interior Inflow Hydrographs and Storm Sewer Overland Flows for Drainage Area C (100-year interior hydrograph routed against 2-year stage (Future Tide) hydrograph)



Interior Inflow Hydrograph from Drainage Areas D and E & Overland Flows for Drainage Area D (10-year interior hydrograph routed against 100-year stage (Current Tide) hydrograph)

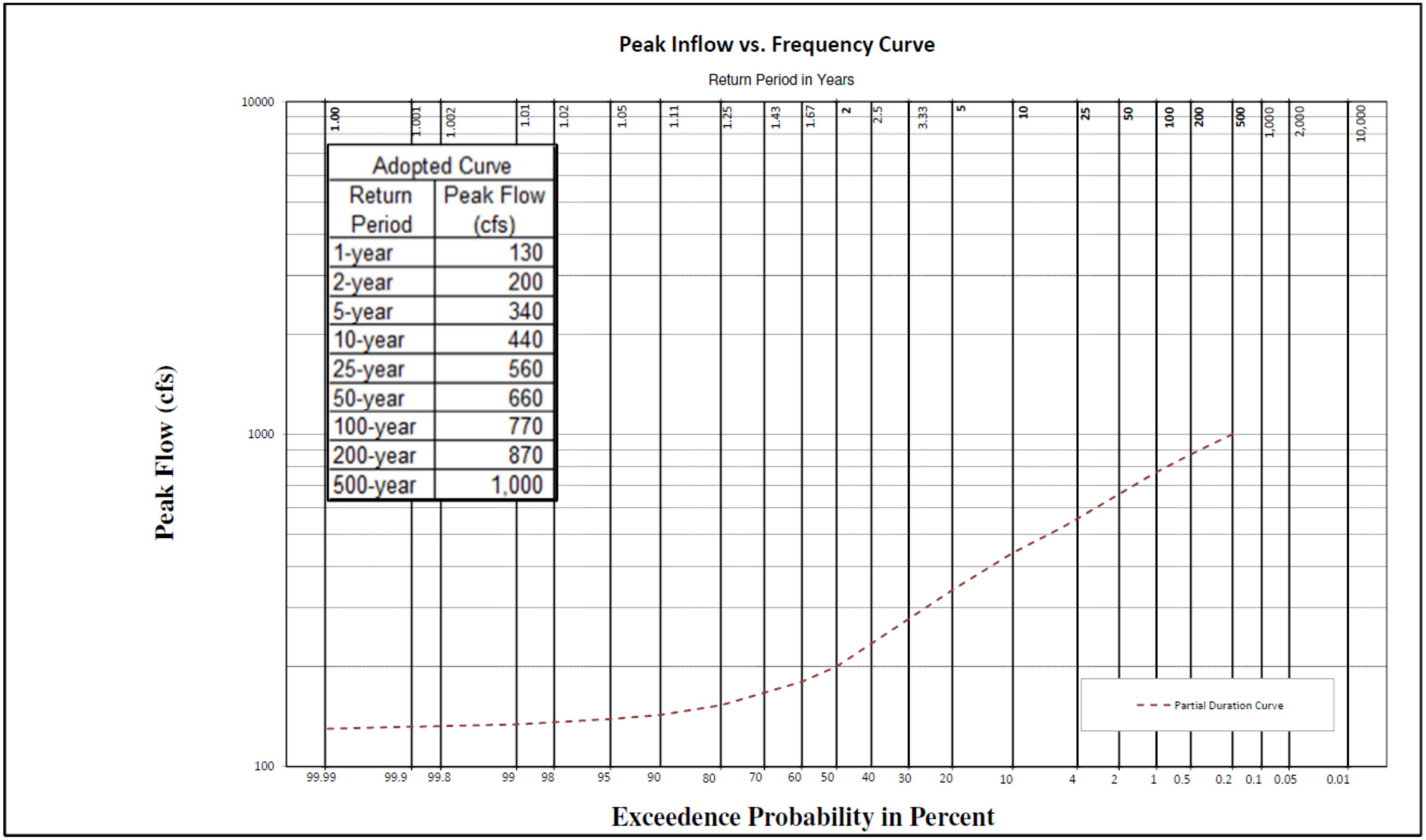


Figure 22: Peak Discharge vs Frequency curve for Drainage Area A (SAWTTL: HEC-HMS NODAL NAME; Drainage Area 0.46 mi^2)

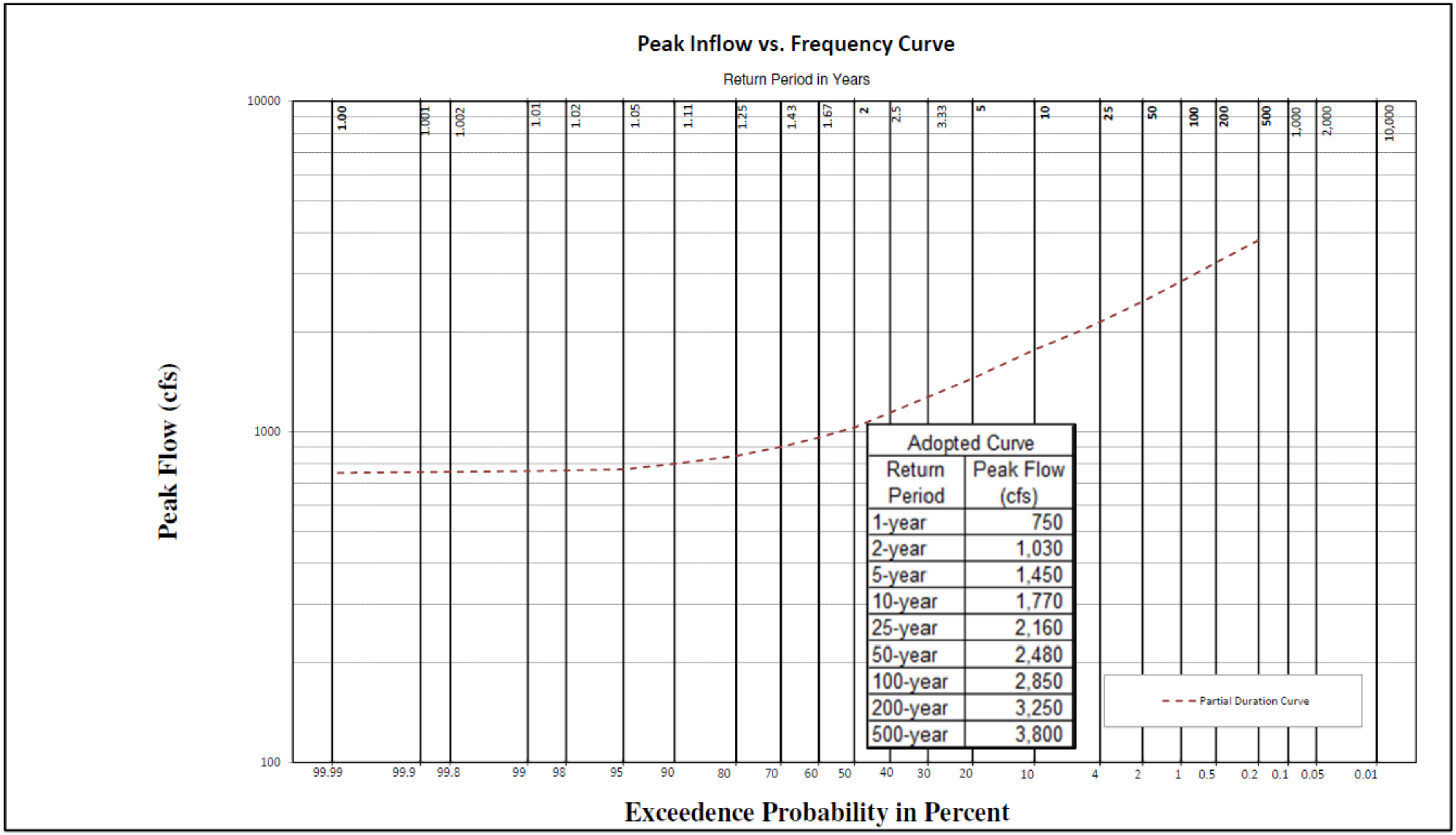


Figure 23: Peak Discharge vs Frequency curve for Drainage Area B (APLUSB: HEC-HMS NODAL NAME; Drainage Area 1.75 mi^2)

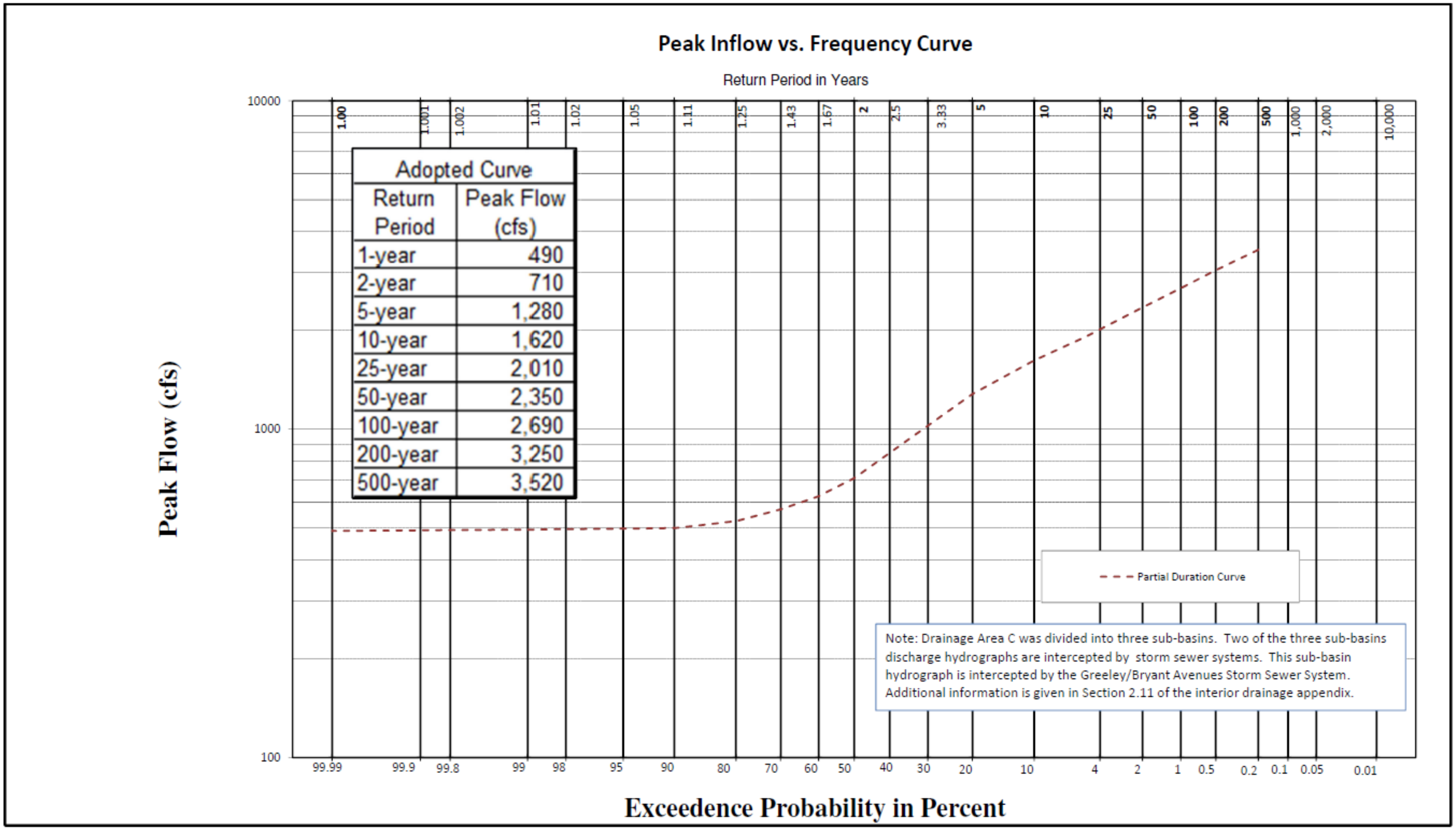


Figure 24: Peak Discharge vs Frequency curve for Drainage C (CCBRGY: HEC-HMS NODAL NAME; Drainage Area1.40 mi^2)

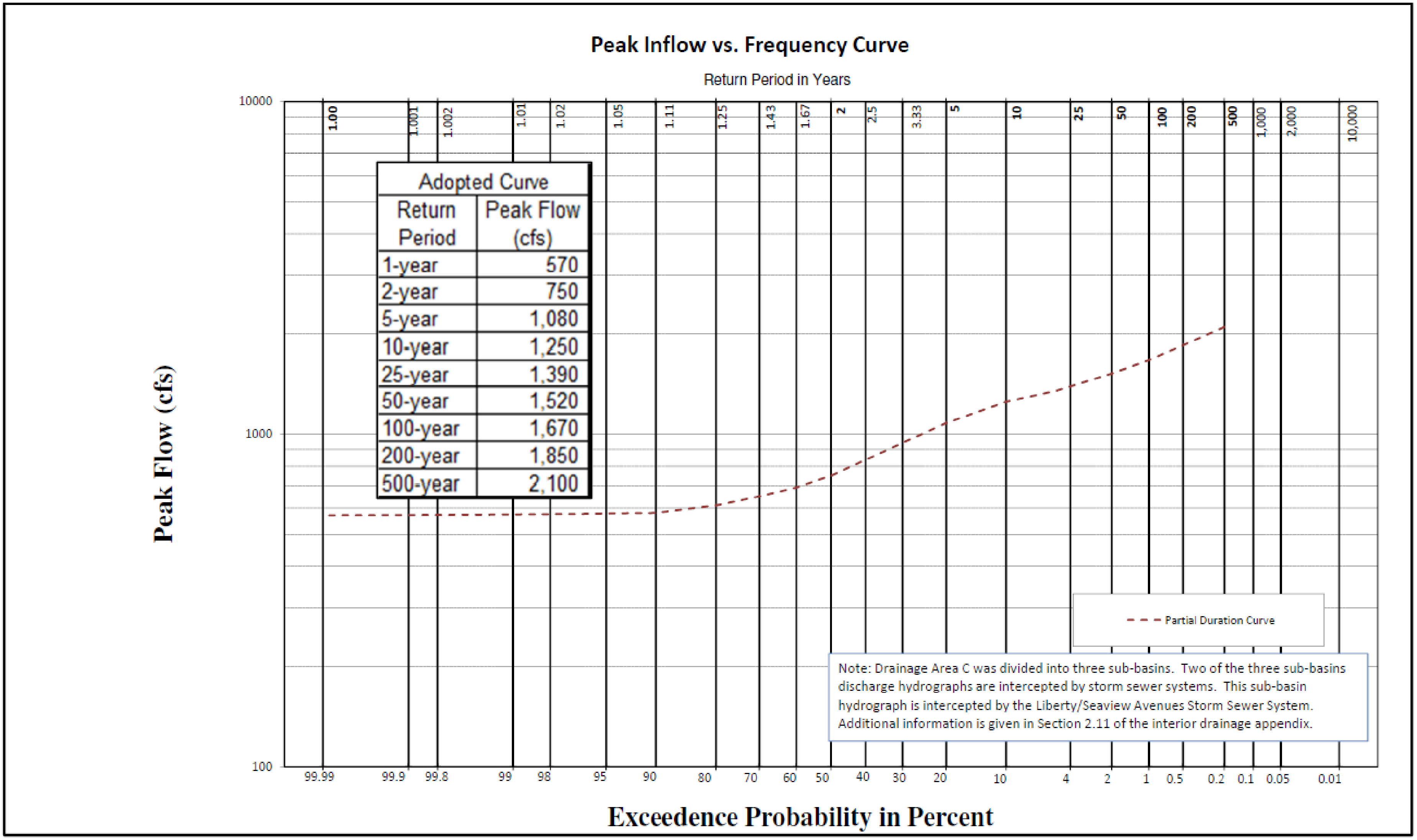
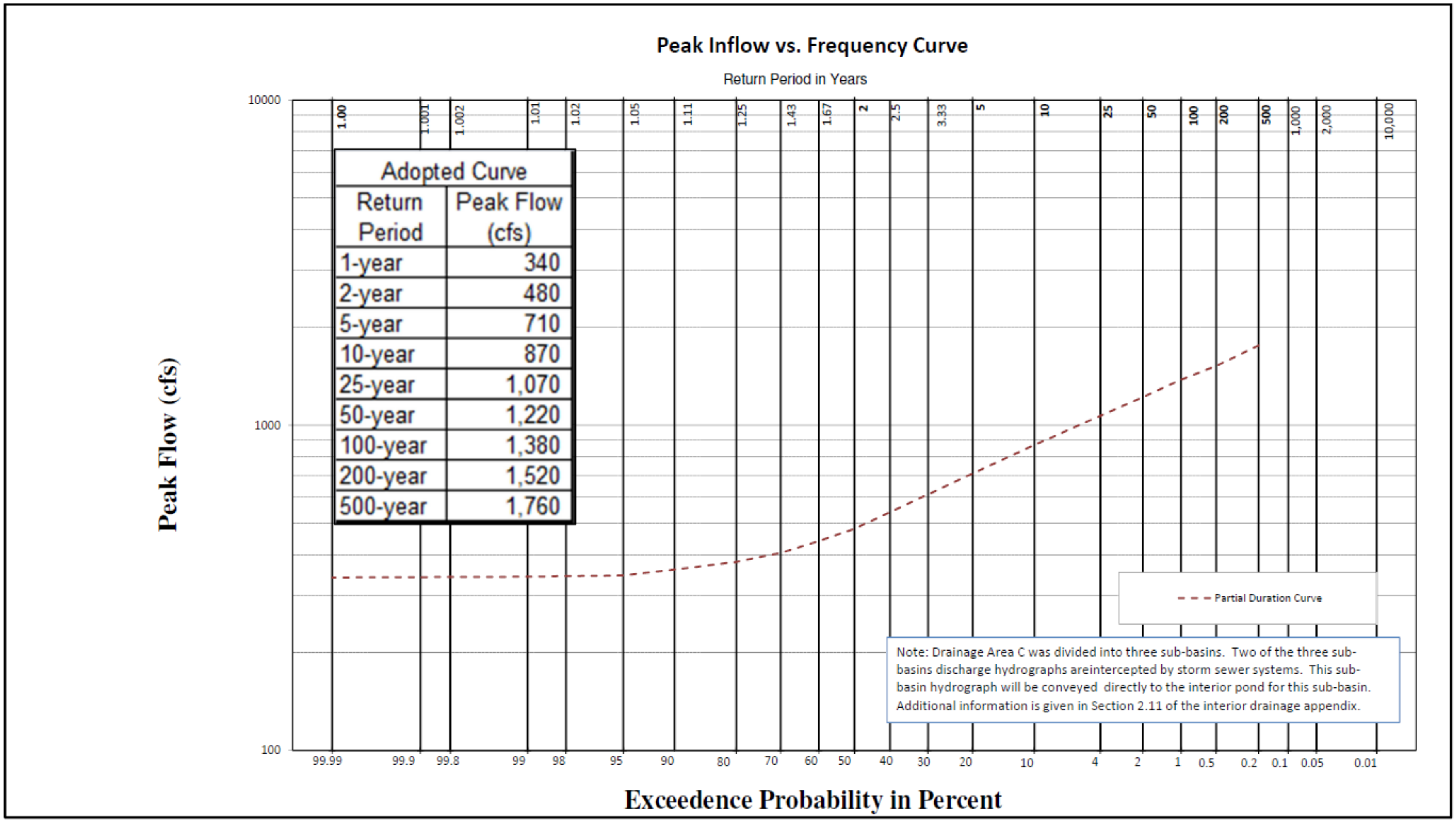


Figure 25: Peak Discharge vs Frequency curve for Drainage C (CCLBSV: HEC-HMS NODAL NAME; Drainage Area 0.45 mi^2)



Adopted Curve	
Return Period	Peak Flow (cfs)
1-year	340
2-year	480
5-year	710
10-year	870
25-year	1,070
50-year	1,220
100-year	1,380
200-year	1,520
500-year	1,760

Figure 26: Peak Discharge vs Frequency curve for Drainage C (CPOND: HEC-HMS NODAL NAME; Drainage Area 1.41 mi^2)

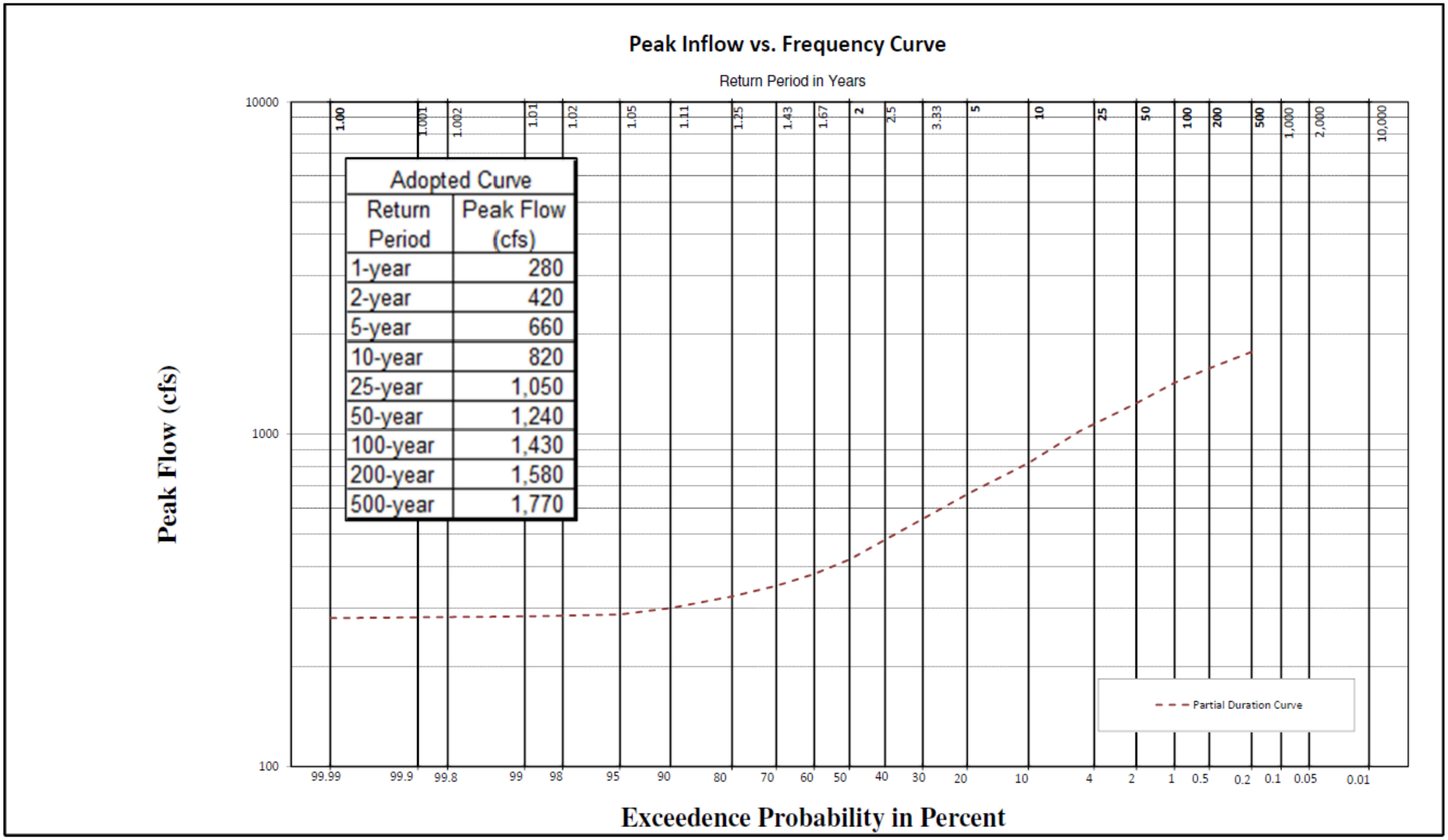
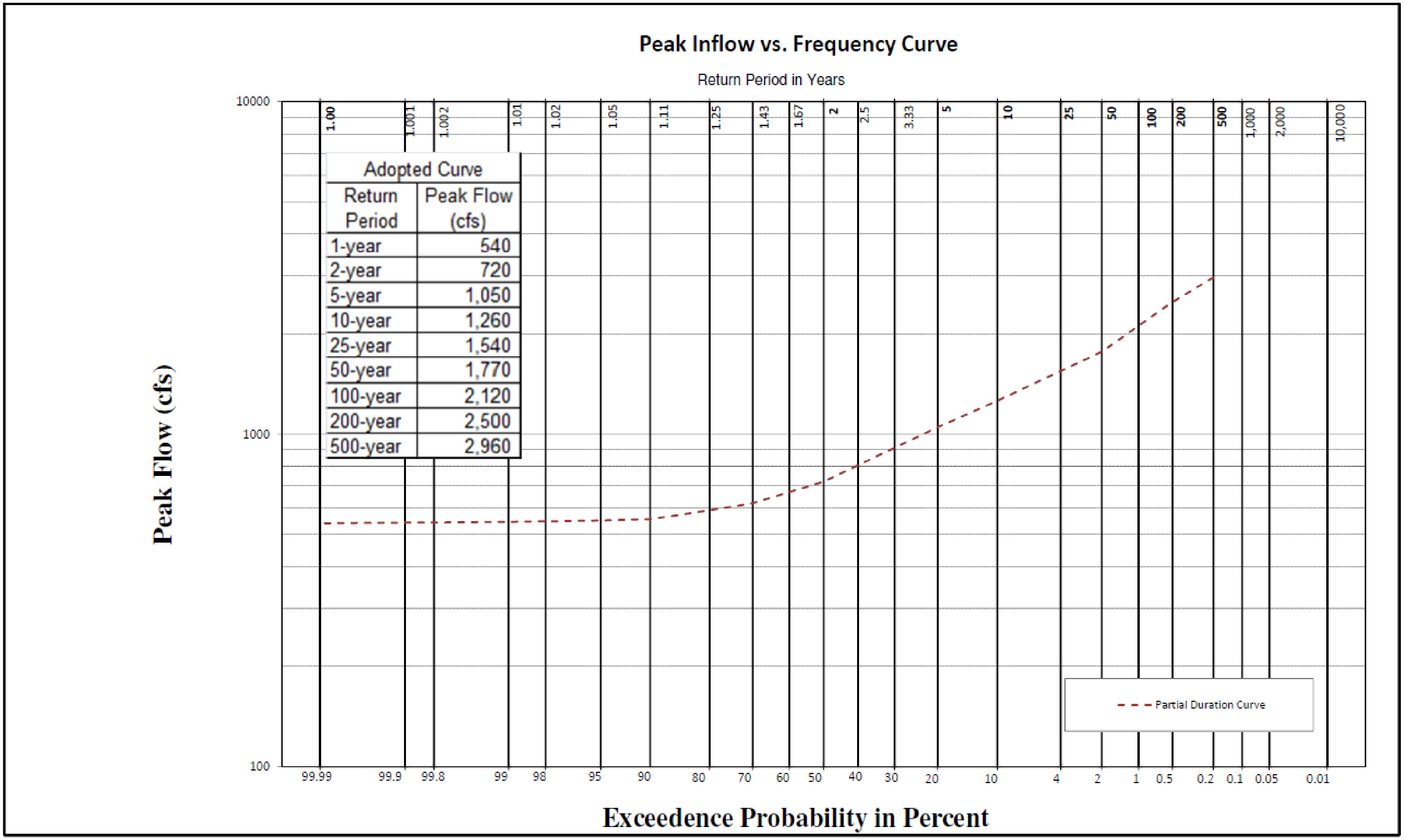
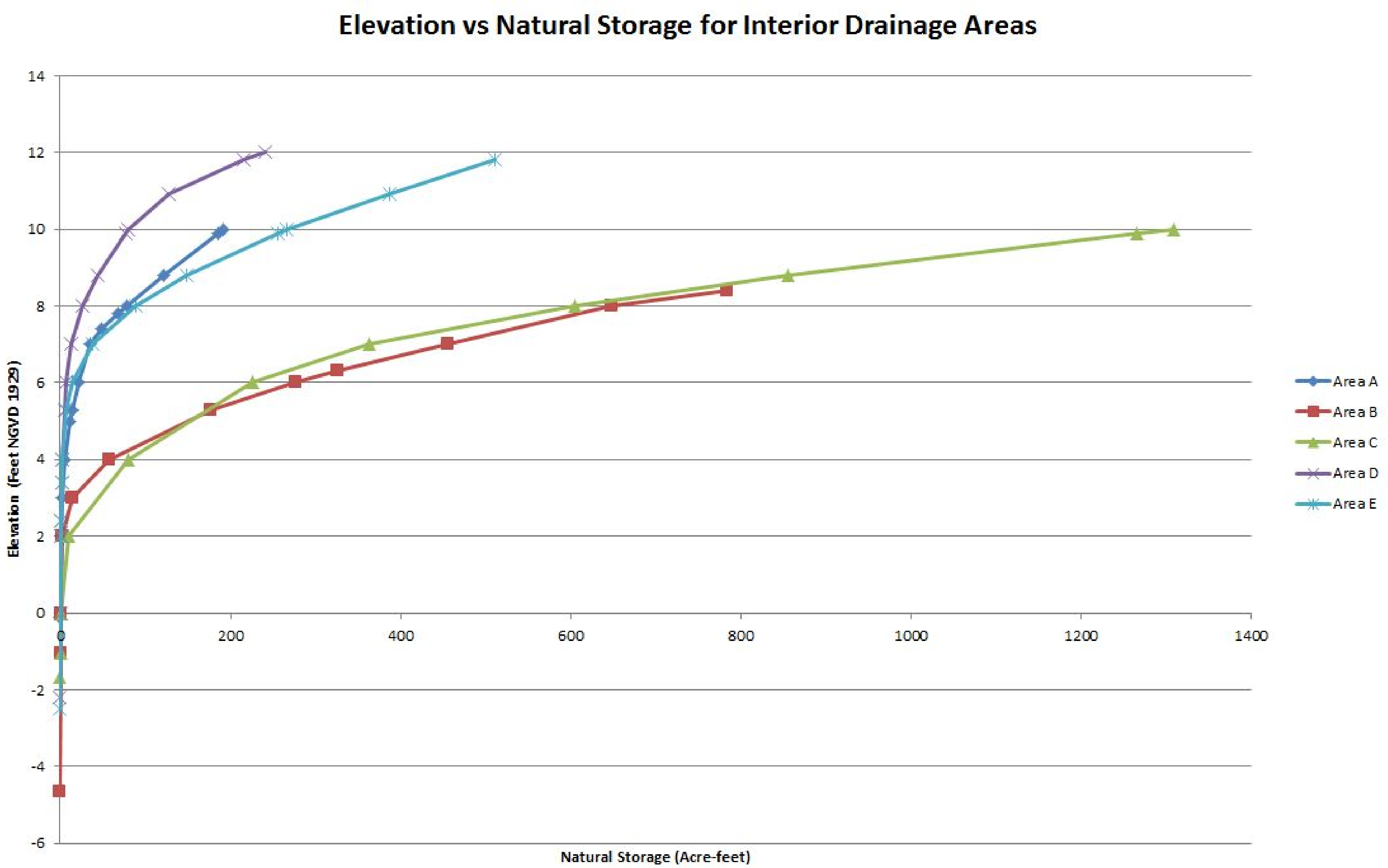


Figure 27: Peak Discharge vs Frequency curve for Drainage D (DTOTAL: HEC-HMS NODAL NAME; Drainage Area 1.12 mi^2)



Adopted Curve	
Return Period	Peak Flow (cfs)
1-year	540
2-year	720
5-year	1,050
10-year	1,260
25-year	1,540
50-year	1,770
100-year	2,120
200-year	2,500
500-year	2,960

Figure 28: Peak Discharge vs Frequency curve for Drainage E (ETOTAL: HEC-HMS NODAL NAME; Drainage Area 1.27 mi^2)



Elevation vs Natural Storage for Interior Drainage Areas from Fort Wadsworth to Oakwood Beach

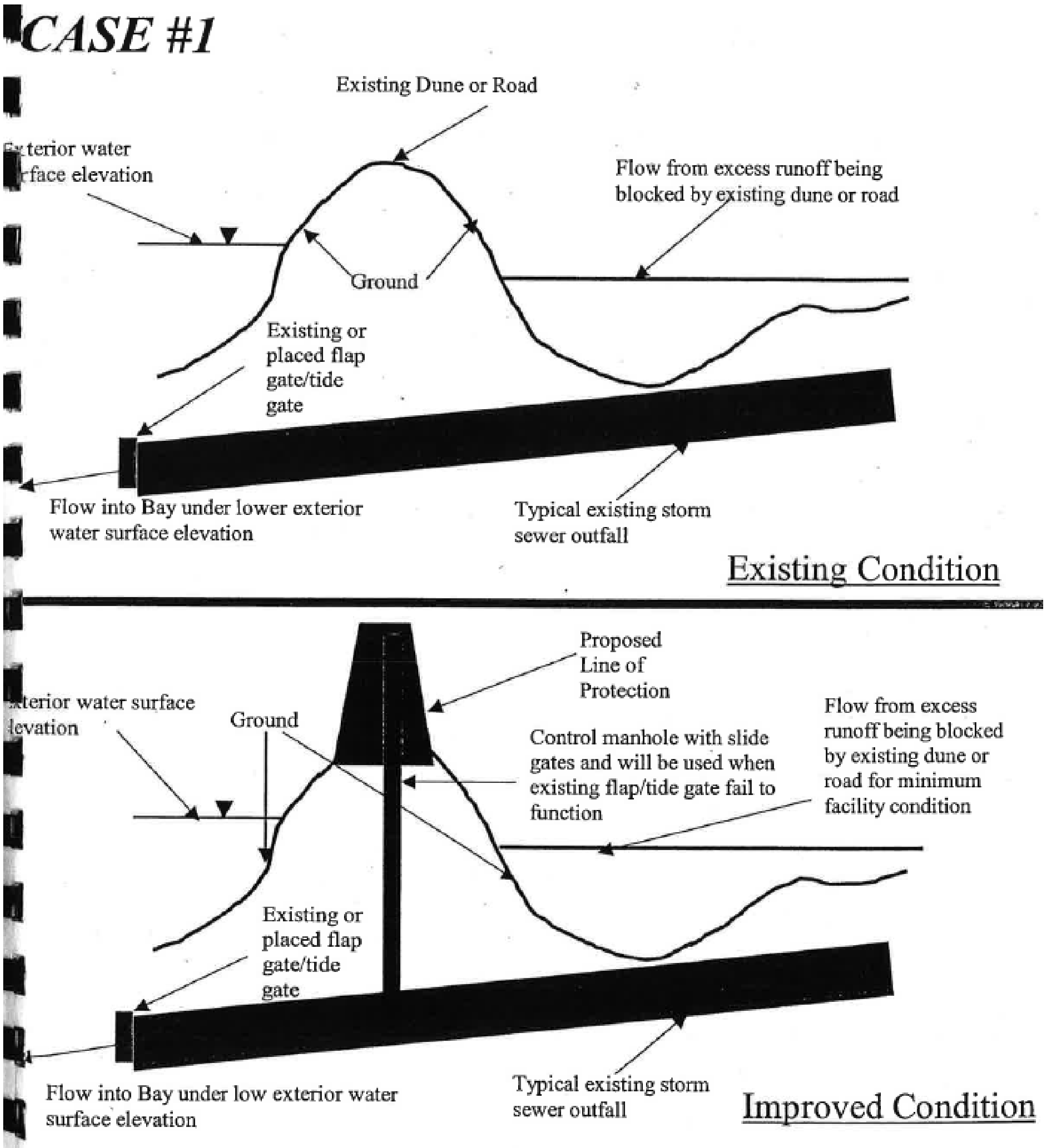


Figure 1: Schematic drawings of the natural sand dune or road with interior runoff storage for existing conditions and improved conditions (placement of the line of protection).

Minimum Facility Case #1 – Existing Barrier

CASE #2

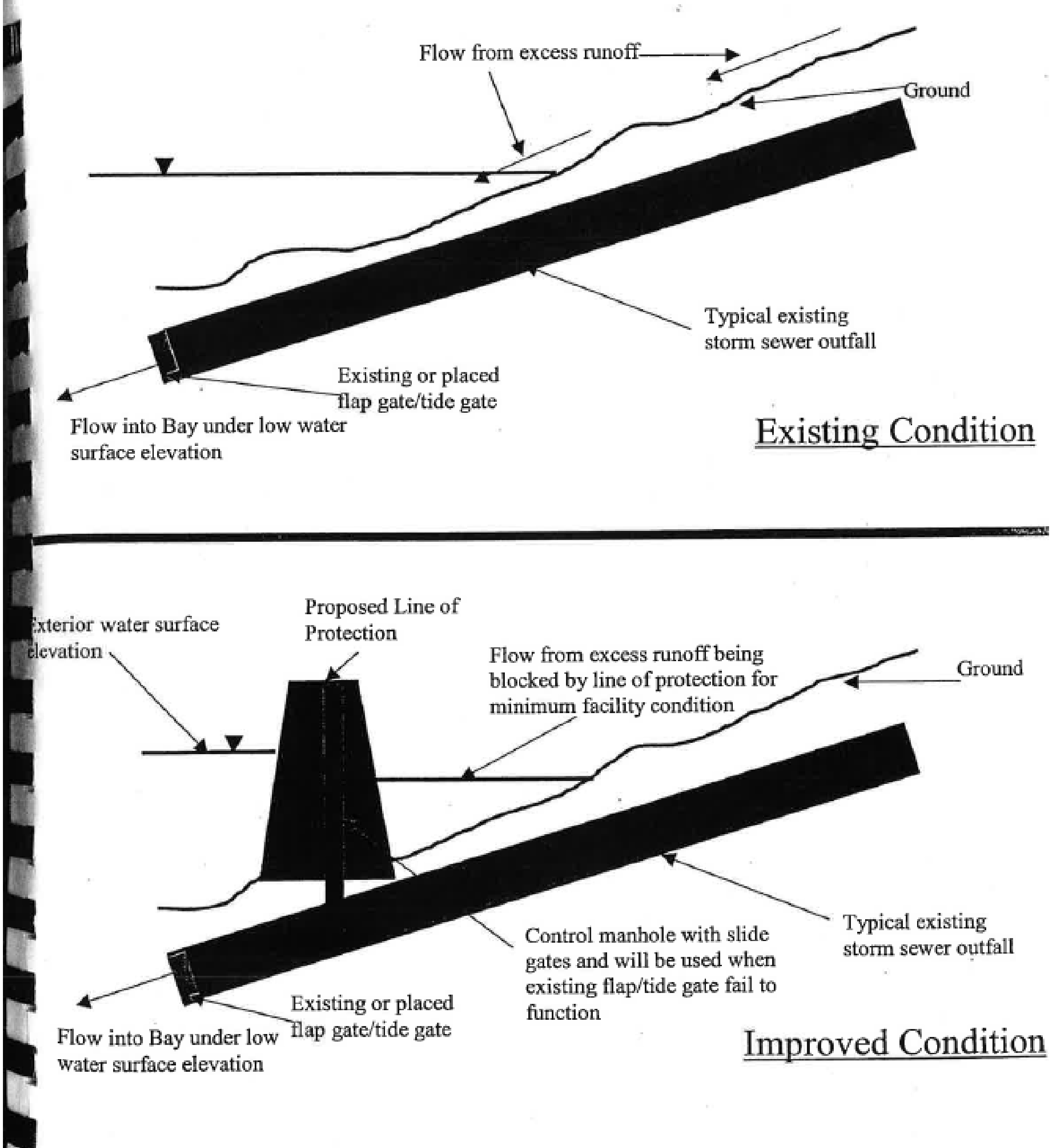
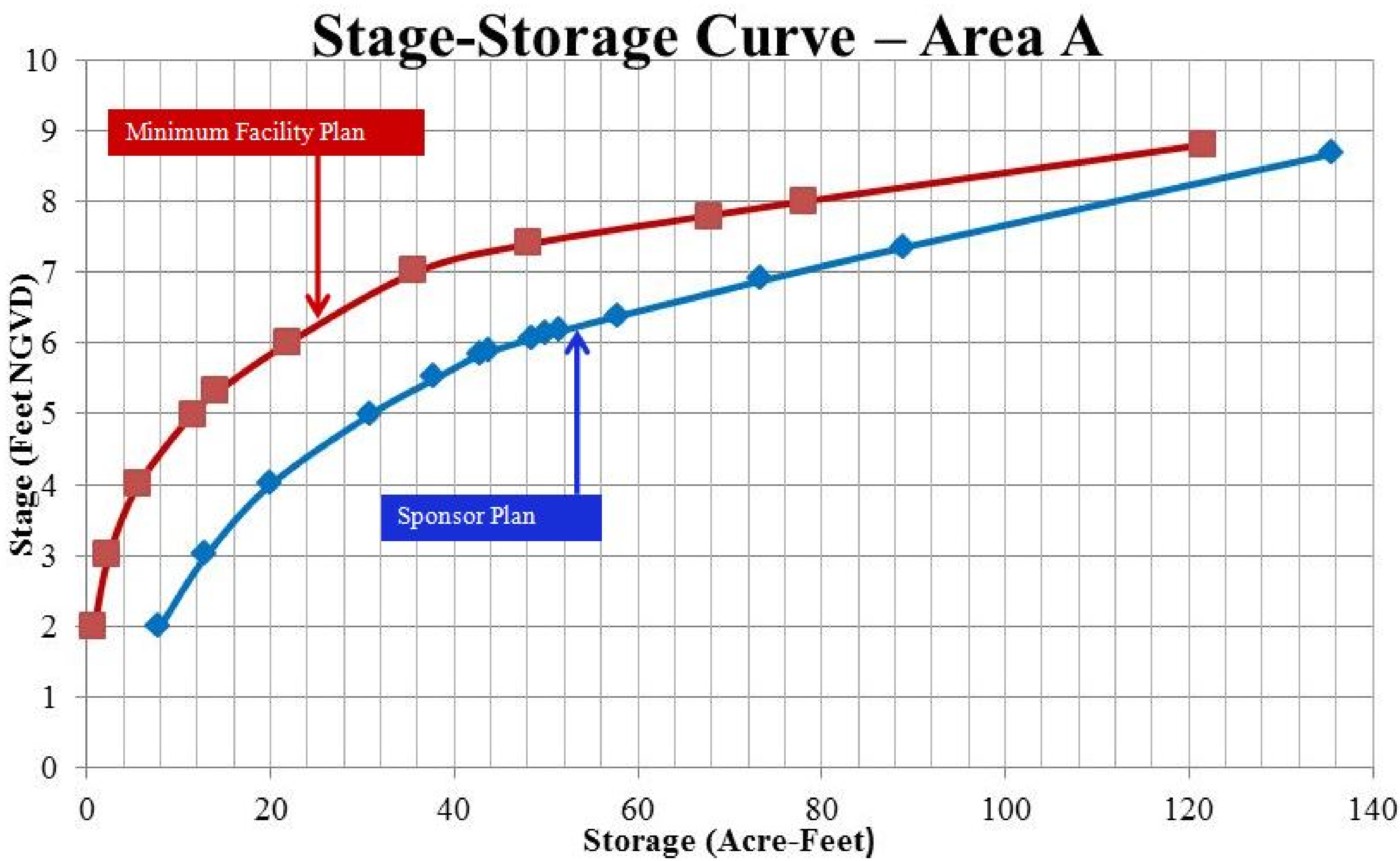


Figure 2: Schematic drawing of the interior runoff storage for existing conditions and improved conditions (placement of line of protection).

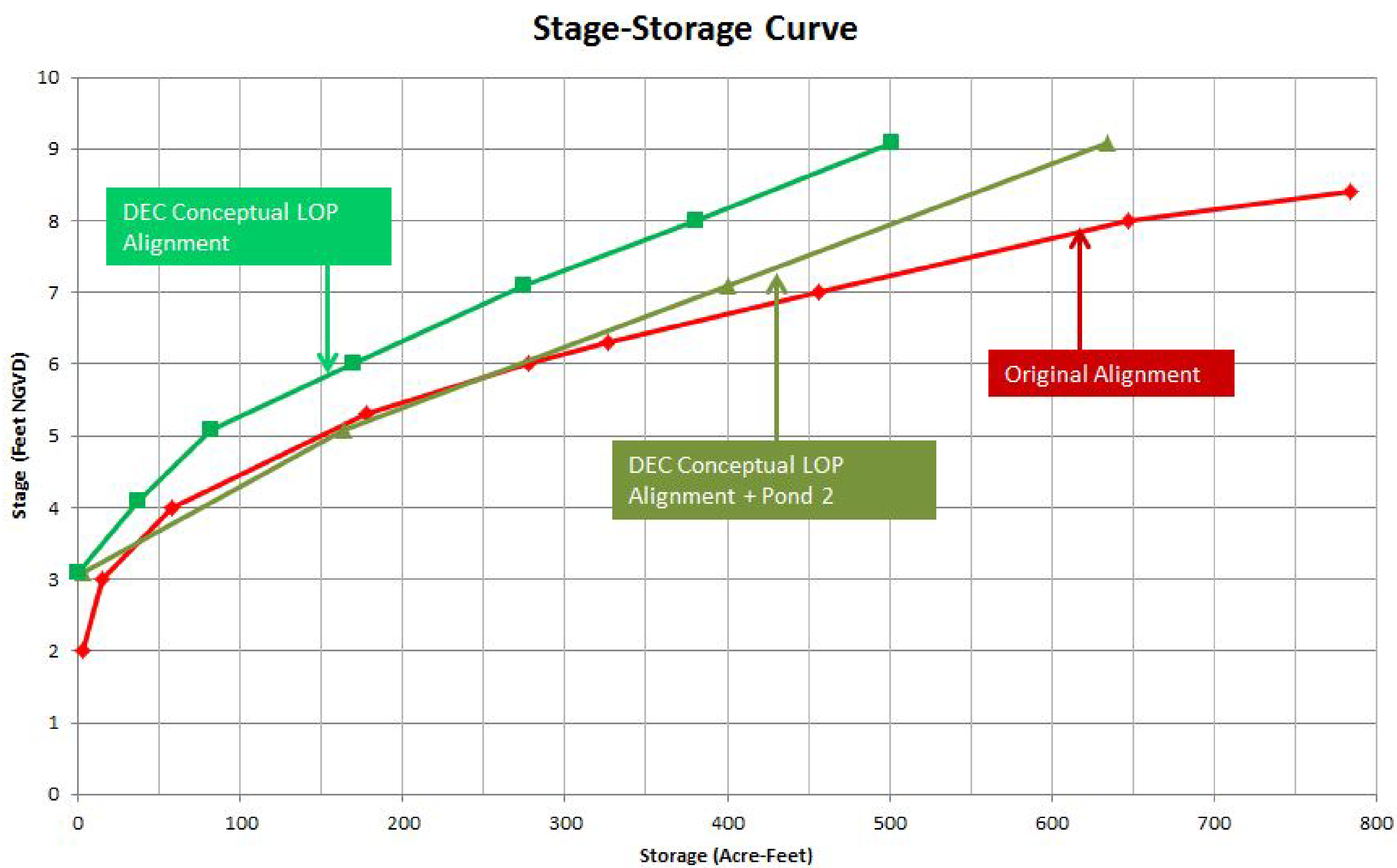
Minimum Facility Case #2 – Without Existing Barrier

Sluice Gate Structure
(2' x 2')
Sluice Gate Structure
(2' x 2')
Sanitary Sluice Gate Structure
Road Raising
Elevation 7.1' NGVD29
Properties Scheduled for
Acquisition by Others
Area A
17.19ac.
Oakwood Creek Tide Gate
(with 3 - 5'X5' Sluice Gates)
Restrictive Easement
Toe of Levee
Excavated Ponding Area
Property Scheduled
for Acquisition by Others
Drainage Structures
Road Raising
Sewer Outfalls
Gate Chamber At Outfall
Sanitary and Drainage Structures
0 125 250 500 Feet
N
Drainage Area A - Minimum Facility (Tentatively Selected Plan)



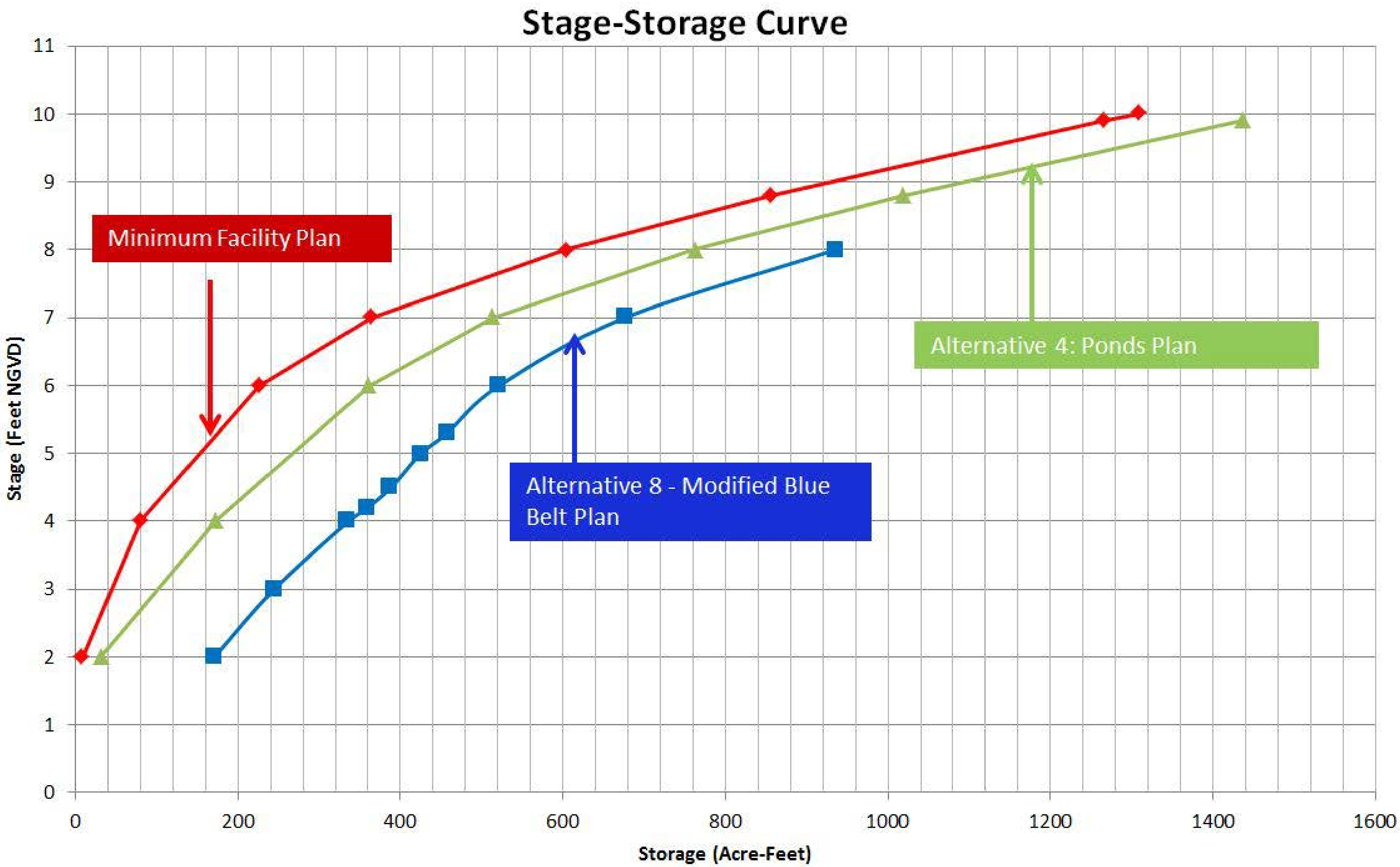
Stage-Storage Curve, Drainage Area A


Road Raising
Elevation 7.1' NGVD29
Excavated Pond 45.85 ac.
Pond Invert 2.7ft NGVD 1929
(94,200 CY)
Road Raising
Elevation 7.1' NGVD29
New Dorp Lane
Major Sewer Outfall
Restrictive Easement
Area = 40.41 Acres
Ebbits Street
Major Sewer Outfall
Properties Scheduled for
Acquisition by Others
Tysens Lane
Major Sewer Outfall
Area B Tide Gate
(with 3 - 5'X5' Sluice Gates)
Kissam Ave Culverts
Oakwood Creek Tide Gate
(with 3 - 5'X5' Sluice Gates)
Restrictive Easement
Toe of Levee
Flowage Easement
Properties to be
Acquired by Others
Drainage Structures
Road Raising
Gate Chamber At Outfall
Sanitary or Drainage Structures
0 250 500 1,000 Feet
Drainage Area B - Minimum Facility (Tentatively Selected Plan)

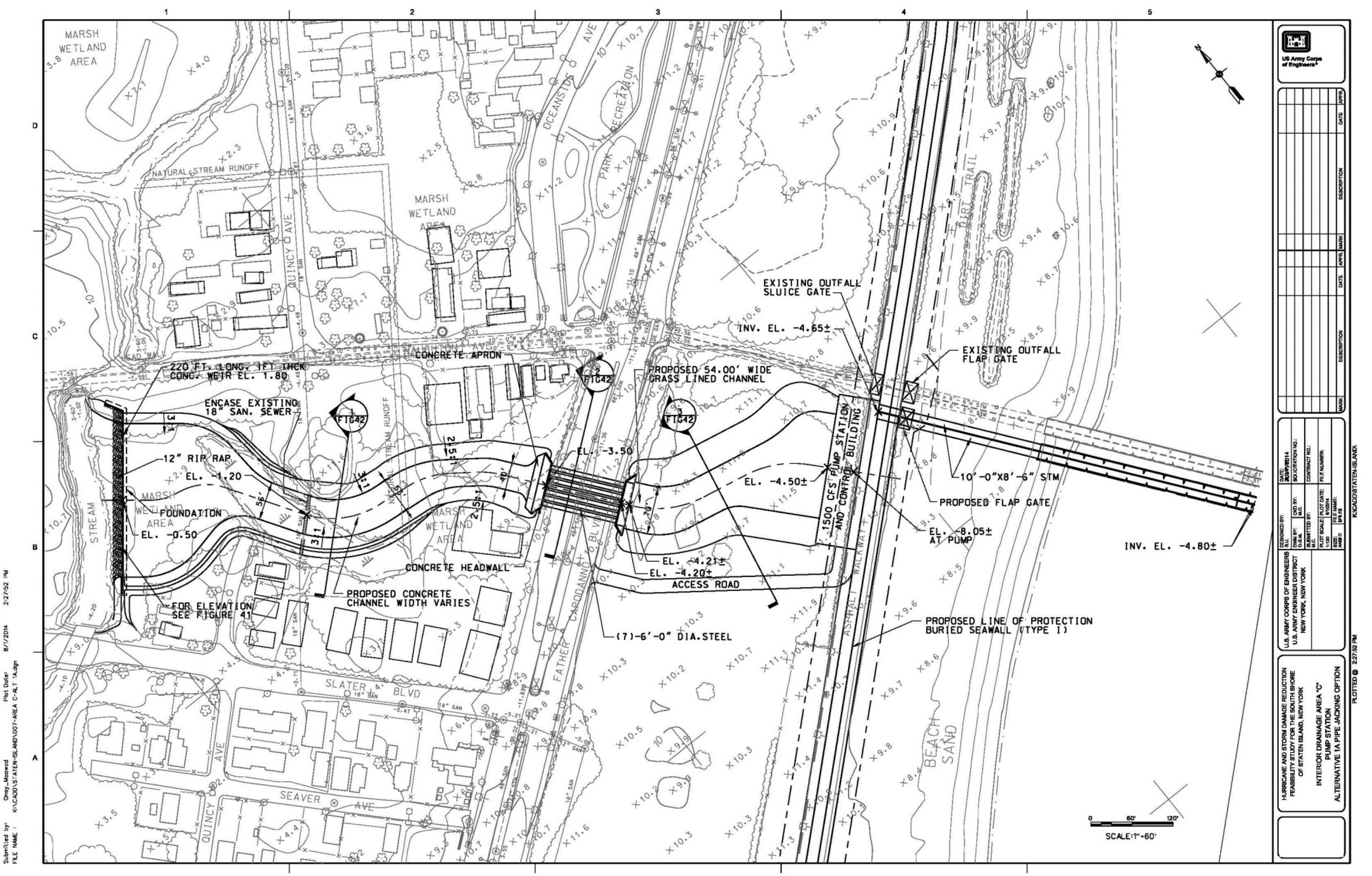


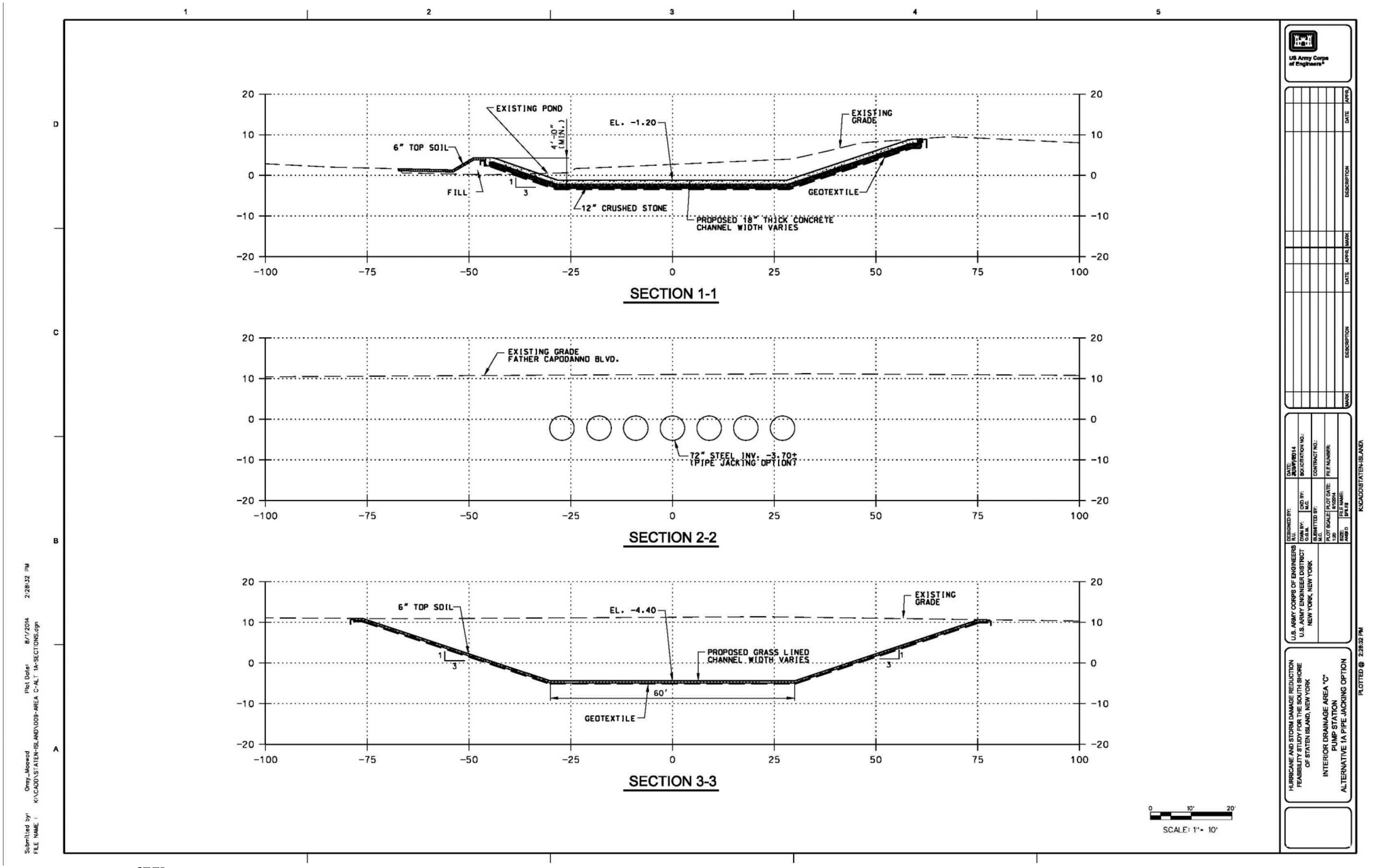
Stage-Storage Curve, Area B

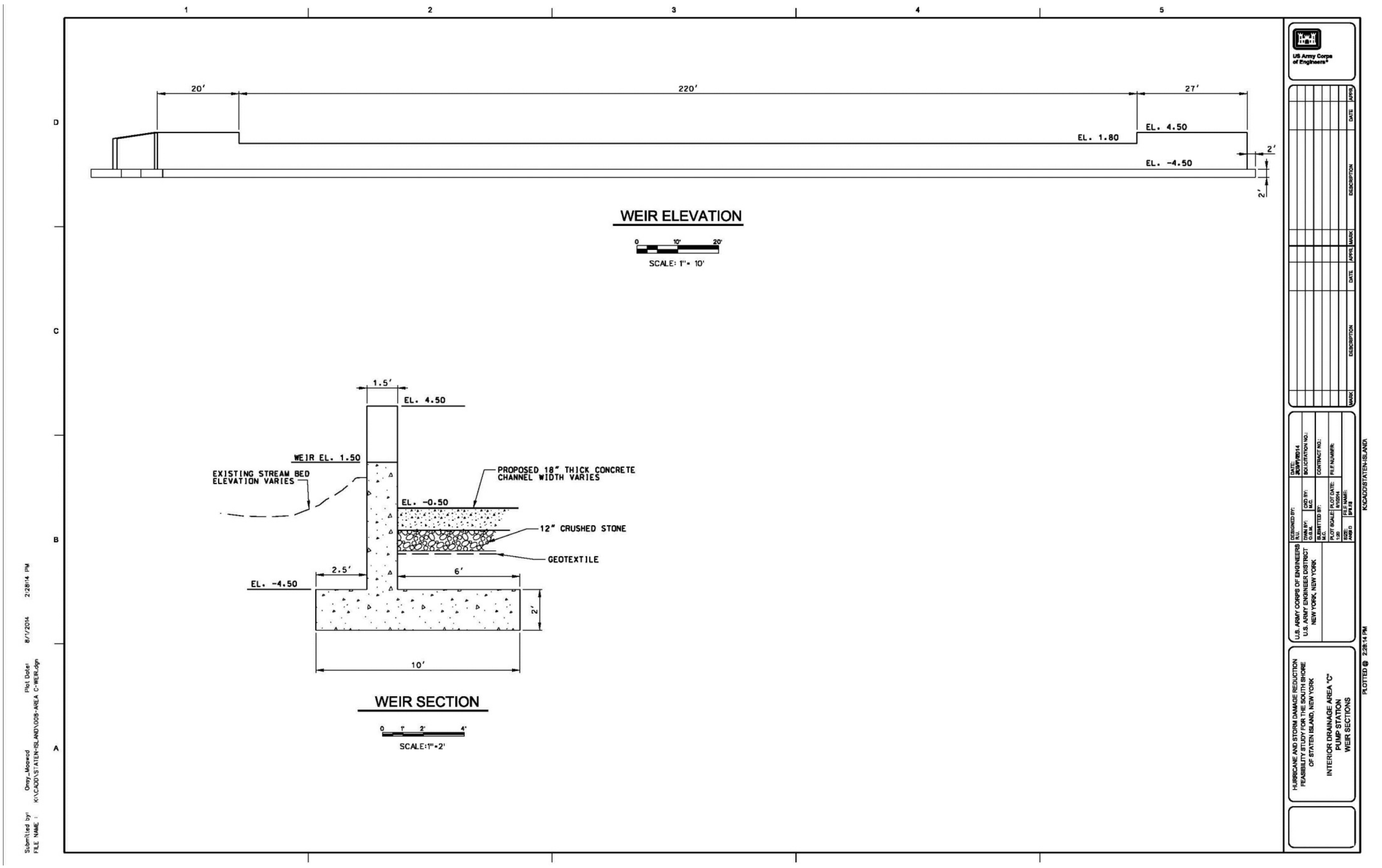
Area C
18.18ac.
Area C
49.07ac.
Area C
18.66ac.
Area C
6.26ac.
Area C
1.74ac.
Area C
26.53ac.
Seaview Avenue
Major Sewer Outfall
Naughton Avenue
Major Sewer Outfall
Midland Avenue
Major Sewer Outfall
Greeley Avenue
Major Sewer Outfall
Restrictive Easment
Toe of Levee
Sewer Outfalls
Gate Chamber At Outfall
0 395 790 1,580 Feet
Drainage Area C Minimum Facility



Stage-Storage Curve, Area C


MARSH WETLAND AREA
NATURAL STREAM RUNOFF
QUINCY AVE
MARSH WETLAND AREA
OCEANSIDE AVE
PARK RECREATION
EXISTING OUTFALL SLUICE GATE
INV. EL. -4.65±
EXISTING OUTFALL FLAP GATE
DIRT TRAIL
220 FT. LONG, 1FT THCK CONC. WEIR EL. 1.80
CONCRETE APRON
PROPOSED 54.00' WIDE GRASS LINED CHANNEL
ENCASE EXISTING 18" SAN. SEWER
FIG42
12" RIP RAP
EL. -1.20
EL. -3.50
EL. -4.50±
1500 CFS PUMP STATION AND CONTROL BUILDING
10'-0"X8'-6" STM
PROPOSED FLAP GATE
MARSH WETLAND AREA
FOUNDATION
STREAM
EL. -0.50
EL. -8.05± AT PUMP
INV. EL. -4.80±
CONCRETE HEADWALL
EL. -4.21±
EL. -4.20±
ACCESS ROAD
FOR ELEVATION SEE FIGURE 41
PROPOSED CONCRETE CHANNEL WIDTH VARIES
(7)-6'-0" DIA.STEEL
PROPOSED LINE OF PROTECTION BURIED SEAWALL (TYPE I)
FATHER CAPODANNO BLVD
ASPHALT WALKWAY
SLATER BLVD
SEAVER AVE
BEACH SAND
SCALE:1"=60'
US Army Corps of Engineers
U.S. ARMY CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT NEW YORK, NEW YORK
HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY FOR THE SOUTH SHORE OF STATEN ISLAND, NEW YORK
INTERIOR DRAINAGE AREA "C" PUMP STATION ALTERNATIVE 1A PIPE JACKING OPTION


EXISTING POND
EL. -1.20
4'-0" (MIN.)
EXISTING GRADE
6" TOP SOIL
FILL
12" CRUSHED STONE
GEOTEXTILE
PROPOSED 18" THICK CONCRETE CHANNEL WIDTH VARIES
SECTION 1-1
EXISTING GRADE FATHER CAPODANNO BLVD.
72" STEEL INV. -3.70± (PIPE JACKING OPTION)
SECTION 2-2
6" TOP SOIL
EL. -4.40
EXISTING GRADE
PROPOSED GRASS LINED CHANNEL WIDTH VARIES
60'
GEOTEXTILE
SECTION 3-3
SCALE: 1" = 10'
US Army Corps of Engineers
U.S. ARMY CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT
NEW YORK, NEW YORK
HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY FOR THE SOUTH SHORE OF STATEN ISLAND, NEW YORK
INTERIOR DRAINAGE AREA "C" PUMP STATION ALTERNATIVE 1A PIPE JACKING OPTION

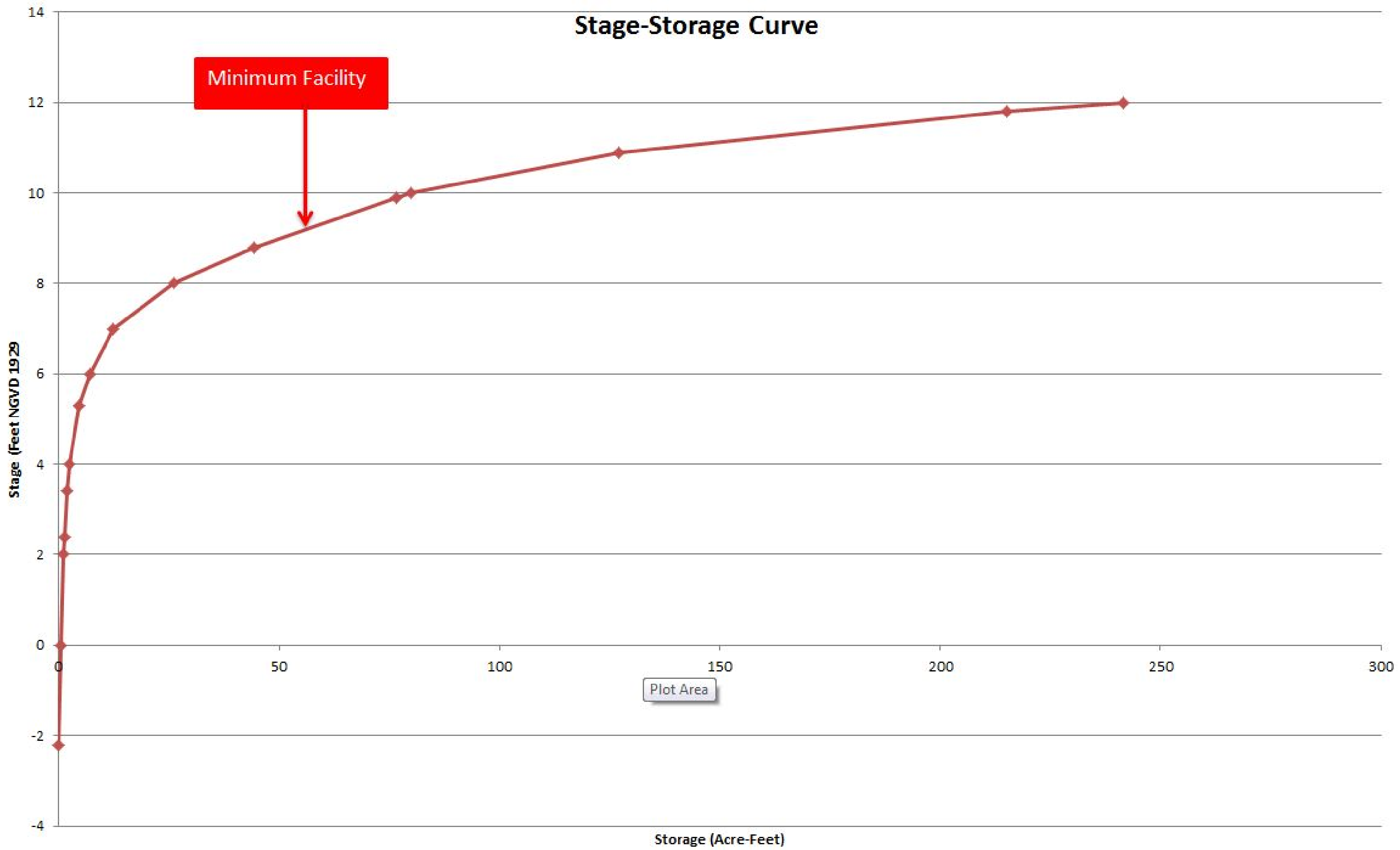

20'
220'
27'
EL. 4.50
EL. 1.80
EL. -4.50
2'
WEIR ELEVATION
SCALE: 1" = 10'
1.5'
EL. 4.50
WEIR EL. 1.50
EXISTING STREAM BED ELEVATION VARIES
PROPOSED 18" THICK CONCRETE CHANNEL WIDTH VARIES
EL. -0.50
12" CRUSHED STONE
GEOTEXTILE
2.5'
6'
EL. -4.50
10'
WEIR SECTION
SCALE: 1" = 2'
US Army Corps of Engineers
U.S. ARMY CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT
NEW YORK, NEW YORK
HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY FOR THE SOUTH SHORE OF STATEN ISLAND, NEW YORK
INTERIOR DRAINAGE AREA "C"
PUMP STATION
WEIR SECTIONS

Last Chance Pond:
18.14 Ac.
117,100 Cy
Pond #7:
12.08 Ac.
27,152 Cy
Pond #4
20.46 Ac.
62,716 Cy
Pond #2
12.01 Ac.
34,250 Cy
Road Raising
Elevation 10' NGVD29
Quintard Street/Raritan Avenue
Major Sewer Outfall
Seaview Avenue
Major Sewer Outfall
Naughton Avenue
Major Sewer Outfall
Midland Pond:
5.74 Ac.
59,600 Cy
Pond #3
16.39 Ac.
41,960 Cy
Pond #1
15.69 Ac.
34,400 Cy
Creek Easement
Midland Avenue
Major Sewer Outfall
Greeley Avenue
Major Sewer Outfall
Legend
Ponding Area
Line of Protection
Limit of Excavation
0 250500 1,000 Feet
Drainage Area C - Alternative 4: 377,200 CY, Seven Ponds
(Tentatively Selected Plan)

Ponding Area:
18.14ac.
Ponding Area:
12.01ac.
Ponding Area:
15.69ac.
Ponding Area:
5.74ac.
Swale
1.12 ac.
Naughton Avenue
Major Sewer Outfall
Legend
Ponding Area
Line of Protection
0 250 500 1,000 Feet
N
Drainage Area C - Alternative 6: 245,350 CY. (Four Ponds)

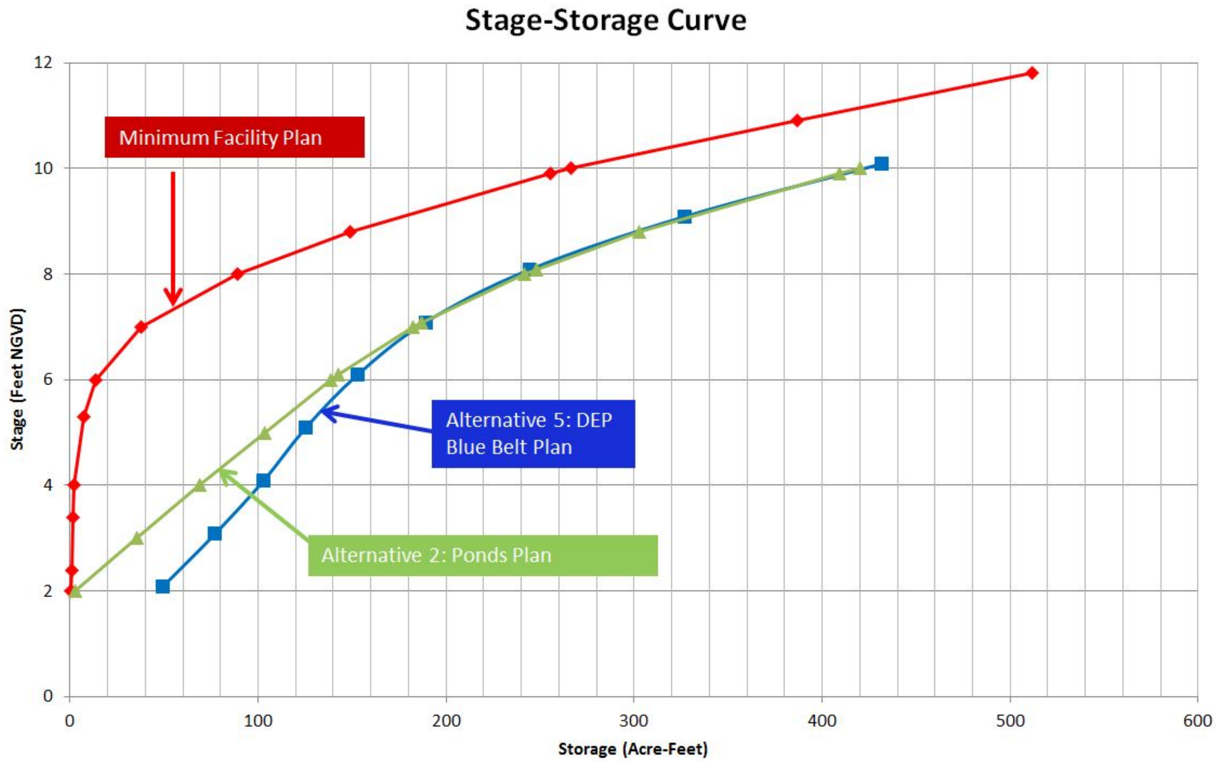
Drainage Area C - Alternative 8: Modified Blue Belt Plan

Area D
30.76ac.
Quintard St. / Raritan Avenue
Major Sewer Outfall
Road Raising
Elevation 10' NGVD29
Seaview Avenue
Major Sewer Outfall
Restrictive Easement
Toe of Levee
Road Raising
Sewer Outfalls
Gate Chamber At Outfall
0
250
500
1,000 Feet
N
Drainage Area D - Minimum Facility (Tentatively Selected Plan)



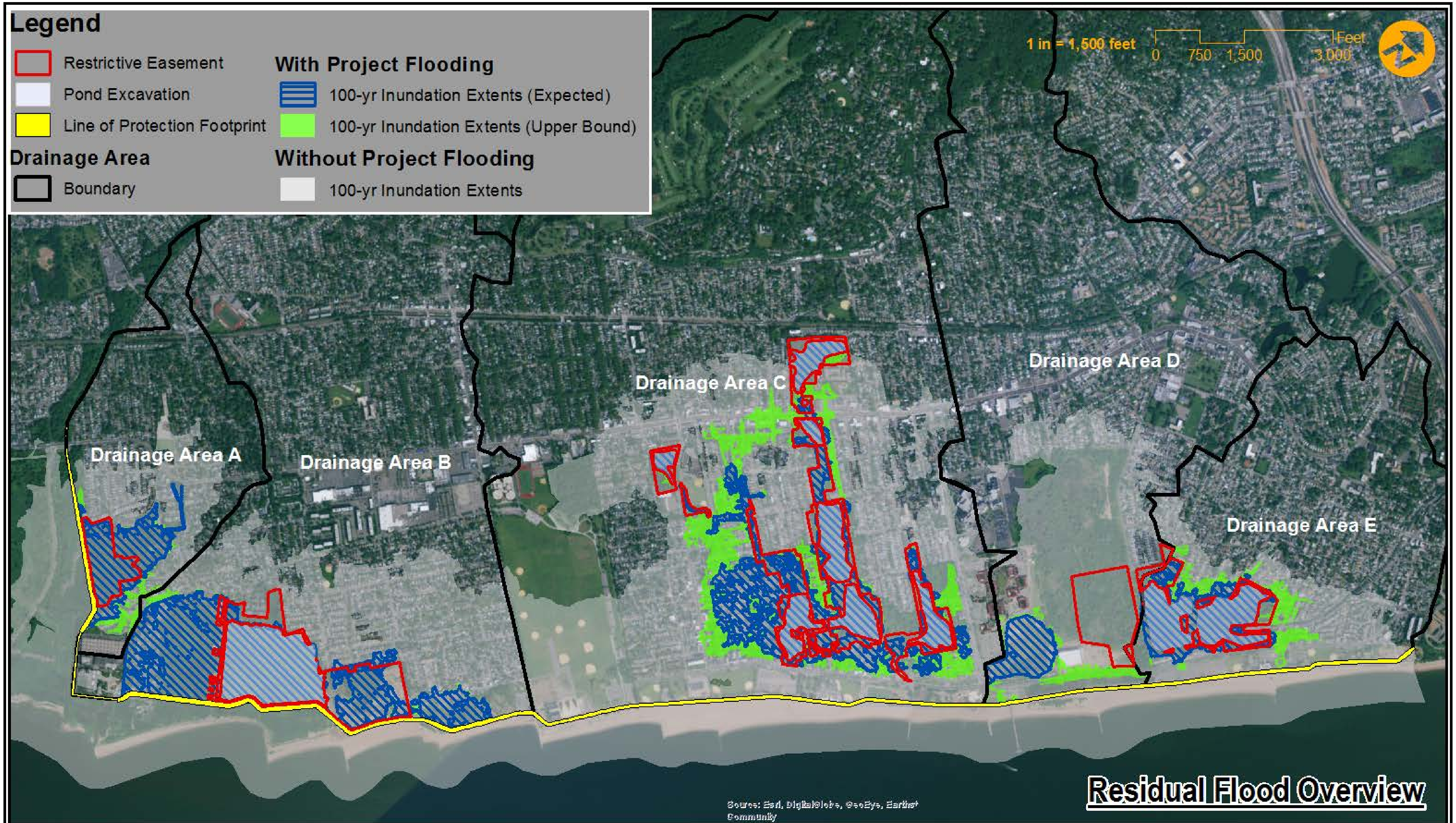
Stage-Storage Curve, Area D

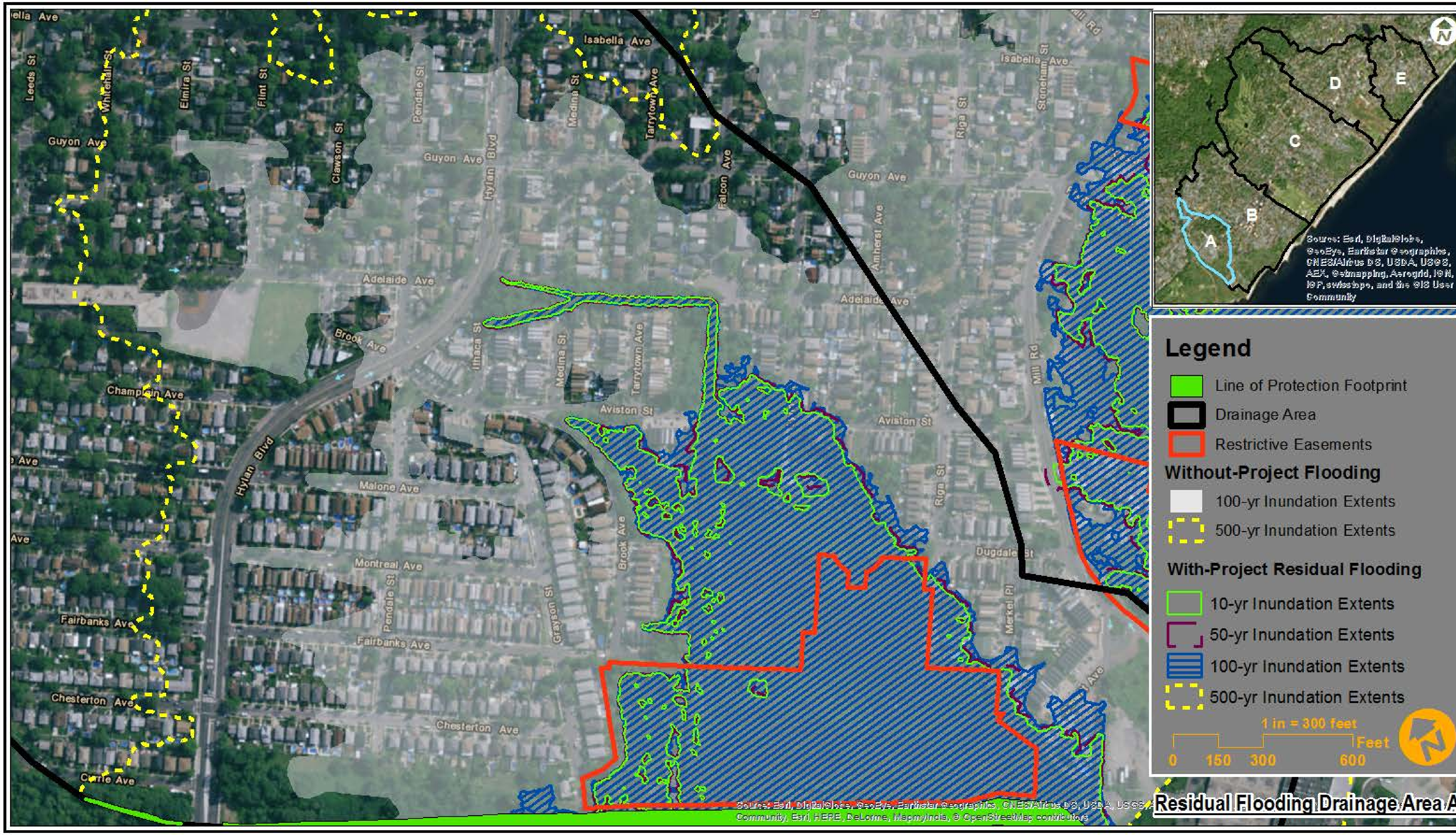

Quincy Ave.
Gate Chamber
Sands Lane
Major Sewer Outfall
Area E
26.5ac.
Area E
20.19ac.
Outlet Pipe
Gate Chamber At Outfall
Restrictive Easement
Toe of Levee
0 125 250 500 Feet
Drainage Area E Minimum Facility



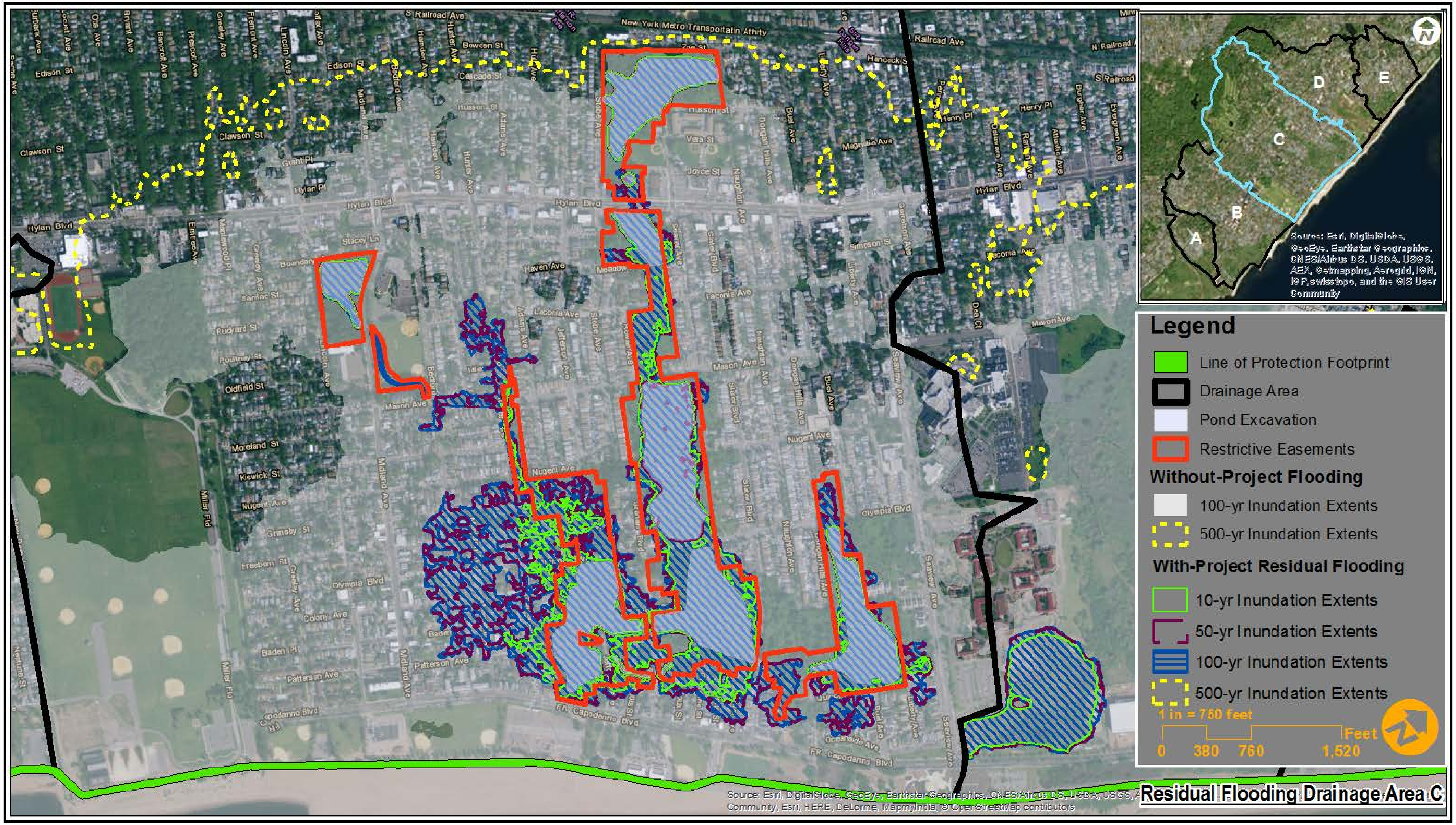
Stage-Storage Curve, Area E

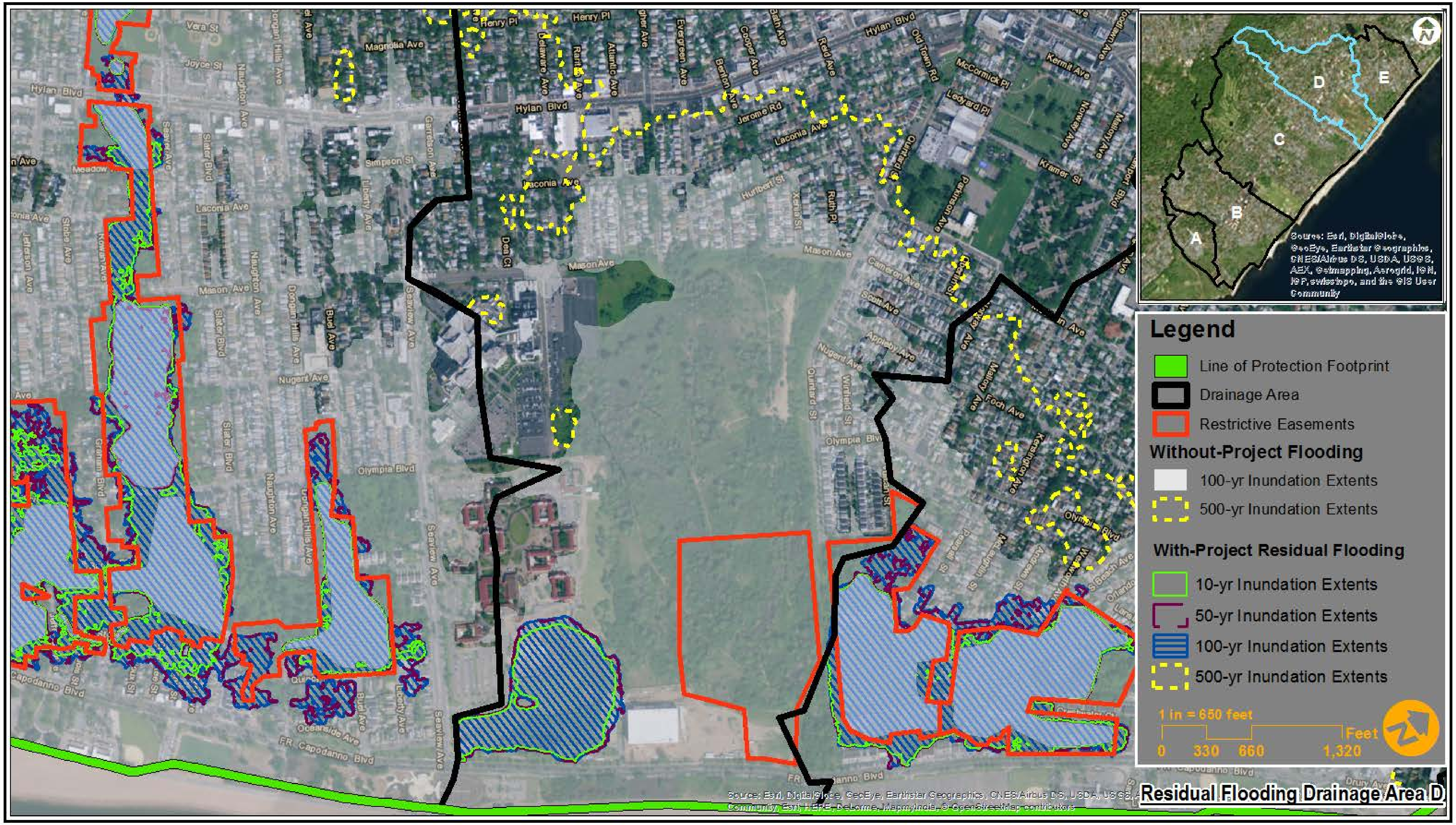

Quincy Ave.
Gate Chamber
Pond #2
18.7 Ac.
104,700 Cy
Pond #1
15.64 Ac.
118,020 Cy
Sands Lane
Major Sewer Outfall
Outlet Pipe
Gate Chamber At Outfall
Excavation Area
Restrictive Easement
Toe of Levee
0 125 250 500 Feet
Drainage Area E - Alternative 2: 222,720 CY, Two Ponds
(Tentatively Selected Plan)

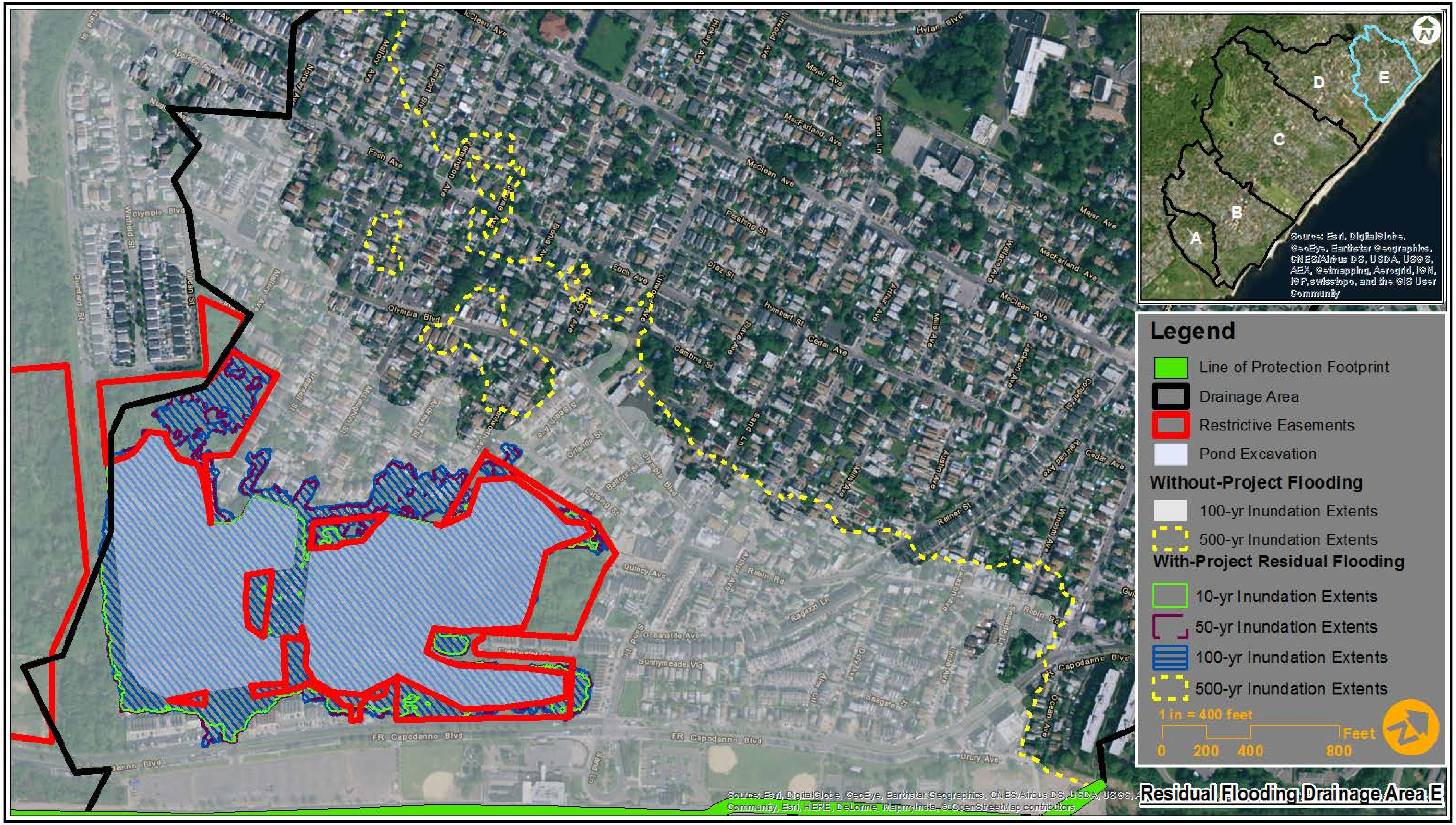

Legend
Restrictive Easement
Pond Excavation
Line of Protection Footprint
Drainage Area
Boundary
With Project Flooding
100-yr Inundation Extents (Expected)
100-yr Inundation Extents (Upper Bound)
Without Project Flooding
100-yr Inundation Extents
1 in = 1,500 feet
0 750 1,500 3,000 Feet
Drainage Area A
Drainage Area B
Drainage Area C
Drainage Area D
Drainage Area E
Residual Flood Overview


Legend
Line of Protection Footprint
Drainage Area
Restrictive Easements
Without-Project Flooding
100-yr Inundation Extents
500-yr Inundation Extents
With-Project Residual Flooding
10-yr Inundation Extents
50-yr Inundation Extents
100-yr Inundation Extents
500-yr Inundation Extents
1 in = 300 feet
0 150 300 600 Feet
Residual Flooding Drainage Area A
A
B
C
D
E
Isabella Ave
Guyon Ave
Adelaide Ave
Aviston St
Malone Ave
Montreal Ave
Fairbanks Ave
Chesterton Ave
Champlain Ave
Carlie Ave
Hylan Blvd
Brook Ave
Dugdale St
Mill Rd
Riga St
Amherst Ave
Tarrytown Ave
Medina St
Pendale St
Clawson St
Elmira St
Flint St
Leeds St
Whitehall St
Ithaca St
Grayson St
Merkel Pl
Falcon Ave


Legend
Line of Protection Footprint
Drainage Area
Restrictive Easements
Pond Excavation
500-yr Inundation Extents
Without-Project Flooding
100-yr Inundation Extents
With-Project Residual Flooding
10-yr Inundation Extents
50-yr Inundation Extents
100-yr Inundation Extents
500-yr Inundation Extents
1 in = 600 feet
0 305 610 1,220 Feet
Residual Flooding Drainage Area B
A
B
C
D
E
Hylan Blvd
Mill Rd
Titus Ave
Cuba Ave
Weed Ave
Winham Ave
Milton Ave
Finley Ave
Roma Ave
Cedar Grove Ave
Manila Pl
Stoneham St
Amherst Ave
Lynn St
Tarrytown Ave
Medina St
Pendale St
Clawson St
Elmira St
Flint St
Thomas Pl
Isora Pl
Primrose Pl
Dartmouth Loop
Ebbitts St
Princeton Ave
Allison Ave
Penn Ave
Peter Ave
Adelaide Ave
Isabella Ave
Tysens Ln
Pelican Cir
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community


Legend
Line of Protection Footprint
Drainage Area
Pond Excavation
Restrictive Easements
Without-Project Flooding
100-yr Inundation Extents
500-yr Inundation Extents
With-Project Residual Flooding
10-yr Inundation Extents
50-yr Inundation Extents
100-yr Inundation Extents
500-yr Inundation Extents
1 in = 750 feet
0 380 760 1,520 Feet
Residual Flooding Drainage Area C
A
B
C
D
E


Legend
Line of Protection Footprint
Drainage Area
Restrictive Easements
Without-Project Flooding
100-yr Inundation Extents
500-yr Inundation Extents
With-Project Residual Flooding
10-yr Inundation Extents
50-yr Inundation Extents
100-yr Inundation Extents
500-yr Inundation Extents
1 in = 650 feet
0 330 660 1,320 Feet
Residual Flooding Drainage Area D


Legend
Line of Protection Footprint
Drainage Area
Restrictive Easements
Pond Excavation
Without-Project Flooding
100-yr Inundation Extents
500-yr Inundation Extents
With-Project Residual Flooding
10-yr Inundation Extents
50-yr Inundation Extents
100-yr Inundation Extents
500-yr Inundation Extents
1 in = 400 feet
0 200 400 800
Feet
Residual Flooding Drainage Area E
A
B
C
D
E