

PRELIMINARY ENGINEERING FEASIBILITY REPORT THE HAGUE WATERSHED

City of Norfolk City-wide Coastal Flooding Contract Work Order No. 1

Prepared for:
CITY OF NORFOLK
DEPARTMENT OF PUBLIC WORKS

March 2012
Fugro Project No. 04.8111023





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March 16, 2012
Project No. 04.81110023

City of Norfolk
Department of Public Works
City Hall Building, Suite 700
Norfolk, Virginia 23510

Attention: Mr. John M. White, Director, Storm Water Division

Subject: Preliminary Engineering Feasibility Report - The Hague Watershed, City of Norfolk, City-wide Coastal Flooding Project, Work Order No. 1

Dear Mr. White:

Enclosed is Fugro Atlantic's report documenting our preliminary engineering feasibility report for the Hague Watershed. This report was authorized by Work Order #1, dated October 28, 2011 of the City-wide Coastal Flooding contract (City of Norfolk Contract 13062). This report provides our technical assessment of flood mitigation options in The Hague and a preliminary engineering feasibility analysis of the preferred design alternative. Our report considers various options for mitigation approach, screens those options relative to their technical merit, flexibility, and projected costs. The report also includes consideration of several different criteria for flood mitigation in terms of severity of storm and potential future sea level rise.

The work, as documented herein, builds on the tide gauge measurements of water levels within the City and the development of a GIS-based mapping capability to translate those measurements to flood depth predictions for various tide levels, as measured at Sewells Point. The results of those measurements and their implications were provided in Fugro's July 2010 *Preliminary Coastal Flooding Evaluation and Implications for Flood Defense Design* report (Fugro, 2010), which provides the starting point for the current evaluation and study. In addition to the technical considerations of flood mitigation alternatives, as discussed herein, the information from this study (and the broader City-wide Coastal Flooding study) also is directly relevant for various planning studies and emergency response preparations within the the Hague area of the City.

On behalf of the project team, we thank you for the opportunity to be of service to the citizens of Norfolk.



Sincerely,

A handwritten signature in blue ink, appearing to read "Kevin R. Smith".

Kevin R. Smith
Senior Engineering Geologist/Project Manager

A handwritten signature in blue ink, appearing to read "Thomas W. McNeilan".

Thomas W. McNeilan, P.E.
Vice President, Fugro Atlantic

Copies Submitted: ()

Draft

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1.0 EXECUTIVE SUMMARY

1.1 BACKGROUND

The City of Norfolk (City) is surrounded by several different bodies of water and their many tributaries. Because the City is low-lying, nearly all portions of the City are below elevation +15 feet and drainage gradients are limited. Thus, a significant percentage of the City is susceptible to flooding from high tides, nor'easters, hurricanes, and other storm events. The flooding ranges from nuisance flooding to severe, albeit less frequent, flooding from hurricanes and major nor'easters, such as occurred in November 2009. The frequency, extent and duration of flooding has been documented to be increasing due to both natural factors and man-induced conditions

In recognition of those considerations, the City initiated a City-Wide Coastal Flooding Evaluation via Contract 11254. The information from the City-wide Coastal Flooding study is considered relevant for not only developing design criteria and designs of public works improvements but also provides important information for various planning studies and emergency response plans within the City.

This Contract was issued to begin a series of tasks intended to help the City programmatically: anticipate flooding scenarios, prioritize problem areas, define design criteria, and develop objectives for various remediation flood defense improvements. The program of activities envisioned by the Contract recognized that: 1) the ability to predict flooding and water depths is only as accurate as the data used to develop those predictions and 2) the availability of tidal records within and surrounding the City has historically been limited to the data provided by three (3) long-term tide gauges at Sewells Point, Money Point, and the Chesapeake Bay Bridge Tunnel. Thus, the initial work orders for the Contract included the deployment of tide gauges to measure water levels and provide a basis for predicting tides throughout the City relative to those at Sewells Point and the development of a GIS-based mapping capability to translate those measurements to predict flood depths for various tide levels, as measured at Sewells Point.

Evaluations of coastal flooding susceptibility within the City and implications for the design of future flood defense improvements were described in the report ***Preliminary Coastal Flooding Evaluation And Implications For Flood Defense Design***, dated July 2010. That report: 1) provided a historical and regional perspective of tidal flooding, 2) summarized and evaluated the initial measurements and implications obtained from the tide gauge deployment, 3) presented relationships between tidal water levels and storm return period, 4) discussed implications of future sea level rise, and 5) provided maps of predicted water depths within the city for various combinations of storm return period and future sea level rise. The report also described the implication of those findings relative to: 1) establishing flood design criteria, 2) developing flood mitigation strategies, 3) potential flood defense options, 4) public policy opportunities and 5) criteria for prioritizing flood mitigation areas and projects.

A second phase of the City-Wide Coastal Flooding Contract began the evaluations of mitigation options for specific watersheds and locations within the City. The Hague watershed was defined to be one of those first priority areas for evaluation. The objectives and priorities for flood improvements will depend on technical considerations, as described herein, that define flood risk (frequency, severity, and extent of flooding) and flood hazards. These technical



factors together with the many societal factors that define the consequences (and their acceptability, or not) of flooding, and the costs of flood mitigation measures all must be considered and evaluated when defining and prioritizing flood mitigation approach and priorities.

There are many ways to reduce the risk, severity, and consequences of flooding. Those approaches can be broadly divided into several categories, such as: 1) drainage and water conveyance system improvements, 2) elevation of the ground surface and structures, 3) construction of barriers to prevent flooding, 4) impoundment and storage of flood waters, 5) adaptive land use to accommodate flooding, and 6) public policy actions.

The present report documents the specific nature of coastal flooding and associated damage estimates, conceptual level evaluation of flood damage mitigation alternatives, selection of a preferred conceptual alternative for further study, and subsequent preliminary (10% level) engineering design of that preferred flood damage mitigation alternative.

1.2 EVALUATION OF CONCEPTUAL FLOOD MITIGATION OPTIONS FOR THE HAGUE

The Hague watershed includes the Ghent residential/commercial community, portions of the Freemason area, and northwestern portions of the downtown Norfolk business district. Much of the area is located in a former tidal estuary historically known as Smith Creek. As the City was developed, much of the former tidal estuary has been filled, improved, and developed upon. The confluence of Smith Creek's branches, where it discharges into the Elizabeth River, is known as The Hague. The watershed (catchment area) from which storm water runoff discharges into The Hague is hereinafter referred to as "The Hague Area."

Flooding in The Hague Area is frequent; and varies from nuisance flooding to events causing significant damage. Flooding is caused by the combined effects of "high tides" and heavy precipitation. The effects of these "high tides" (coastal flooding) are expected to worsen over time as mean sea level rises. In addition, the effects of sea level rise will be compounded by regional and local ground subsidence, themselves resulting from events in geologic time, and ongoing settlement of localized, man-made fill.

This study has shown the inadequacy of the aging storm water collection system in The Hague Area. Improvements to the storm sewerage system could significantly reduce nuisance flooding, and would reduce the worst effects of extra-tidal events in the upper reaches of The Hague area. Because of the inherent limitations in the old storm water system, it cannot effectively deliver the rainfall runoff from large storms to The Hague. Thus, the coastal flooding infrastructure improvements can not eliminate all flooding due to storms with significant precipitation. To mitigate that component of flooding will require future, long-term improvements to the existing storm water drainage system. The construction of the coastal flooding infrastructure does, however, significantly lessen the effects due to the inadequate capacity of the storm drain system. Improvements to the storm water collection system **in combination** with the coastal flood protection improvements will provide the most technically effective means of reducing the risk of flood damage.

The wide spread flooding and density and types of development in The Hague watershed are not conducive to property buyout, elevation of structures or other types of mitigation options. Thus options to mitigate coastal flooding will require capital infrastructure improvements.

This study demonstrates that infrastructure improvements consisting of a flood wall – with a gate to be closed during coastal surge events – can mitigate coastal flooding including much of the worst effects of extreme extra-tidal events from hurricanes and nor'easters. Since The Hague cove is small in comparison with the size of the overall watershed, its capacity to store storm water runoff is limited. Thus, pumps will be required to pass the excess storm water inflow over the flood barrier. These improvements are technically feasible, and can be expected to have a favorable "benefit to cost" (B/C) ratio.

1.3 PRELIMINARY (10% LEVEL) DESIGN AND FEASIBILITY OF THE PREFERRED COASTAL FLOOD MITIGATION ALTERNATIVE

Additional feasibility study was conducted for the preferred coastal flooding mitigation alternative from the conceptual evaluation phase. This Alternative 1a has been developed to an approximate 10% level of preliminary design, and this preliminary design is documented in 11"x17" drawings attached as Appendix E to this report. The later narrative sections of this report (Section 12.0 and following) describe additional design details and considerations relevant to the preferred alternative. Updated opinions of probable cost for the project are presented, and recommendations for next steps are provided.

The opinion of probable capital cost for the preferred alternative, as presently formulated, is approximately \$58.6 million. A detailed breakdown of line items, quantities, and unit costs is provided in Table 13-1.

At the time of the report, the USACE has approved a study to evaluate whether there is Federal interest in the Pretty Lake project. The study is planned to occur during the summer of 2012 and expected to take six months to complete. If the USACE deems there is Federal interest in the project, then the project may be eligible to pursue Federal funding through a partnership with the Federal government.

The proposed project will be required to go through the NEPA process. This study has initiated some of the steps necessary to evaluate potential environmental impacts. This study conducted preliminary hydrodynamic analysis to evaluate the impact to tidal flushing of a wall and gate structure near Shore Drive bridge. These screening-level simulations do not indicate that the proposed tidal barrier would increase flushing times in Pretty Lake (at the barrier or at any point further within the lake). The proposed structure will impact subaqueous bottomlands and potentially limited wetland areas along the shoreline area. This study has not identified potential environmental impacts that would preclude the implementation of the preferred option described in this study.

To manage capital expenditures, it would be logical to sequence the improvements in The Hague by first constructing the coastal flooding barriers and mitigations so as to eliminate the tidal surge from entering The Hague. That can be followed by storm water drainage system improvements.

2.0 INTRODUCTION AND BACKGROUND

2.1 PROJECT BACKGROUND

The City of Norfolk (City) is surrounded by many different bodies of water including the Chesapeake Bay, the Hampton Roads harbor, the Elizabeth and Lafayette Rivers and their many tributaries as well as several small lakes. Because the City is located in a low-lying physiographic region, drainage gradients are limited and nearly all portions of the City are below elevation +15 feet. Thus, a significant percentage of the City is susceptible to flooding from high tides, nor'easters, hurricanes, and other storm events. The intensity of flooding ranges from nuisance flooding, typically associated with high tides, to severe, albeit less frequent, flooding from hurricanes and major nor'easters, such as occurred in November 2009.

In recent years, the City has recognized an increased need to address coastal flooding problems. In 1992 the City created the Environmental Storm Water Fund as a dedicated source of funding for water quality and quantity improvements. Historically, the City has addressed flood mitigation through stand-alone, small to intermediate-sized capital improvement projects. Today, remaining flood mitigation projects are numerous, complex, and may require considerably larger capital improvement budgets. Like all municipalities in the region, the ability to fund flood mitigation and flood defense improvements constrains implementation of such projects.

In addition, relative sea level in the local area is rising (at a current projected rate of 1.45 feet per 100 years (NOAA, 2010a). Assuming that this trend continues (or increases), both nuisance flooding and flooding from storm events will increase. This will further increase the need to address the issue of coastal flooding on both project-specific and a holistic, watershed-scale basis.

The November 2009 Nor'easter has both: 1) reinforced the City's decision to proactively evaluate coastal flooding and 2) elevated the City's needs and priorities for flood defense mitigation. In addition, the short but intense local storm over the Broad Creek area in August 2009 caused local flooding and damage. While the flooding and damage during that storm were significant, they were much less than would have occurred if that storm had coincided with peak high tide rather than low tide conditions.

2.2 CITY-WIDE COASTAL FLOODING PROGRAM

2.2.1 Previous Phases

In 2008, the City began to develop a City-wide evaluation to: anticipate flooding scenarios, help prioritize problem areas, develop design criteria and define objectives for various remediation flood defense improvements. The city-wide flood evaluation was recognized to require a phased and iterative approach to be conducted over several years. The initial efforts of the City-wide coastal flooding contract included the procurement, installation, and monitoring of tide gauges at five locations within the City to define local variations of the tide levels relative to those at Sewells Point, which has the longest history of tidal measurements in the Hampton Roads region. The Sewells Point tide measurements are also the reference that has been and is used to communicate predicted tide levels to the City, the media, and to the population in general.

The City of Norfolk's (City) City-wide Coastal Flooding (Contract 11254) with Fugro Atlantic (and its sub-consultant Moffatt & Nichol) was initiated in 2008 in recognition of the City's increasing susceptibility to flooding. Of concern were the impacts due to both: 1) the recurring combinations of various tidal and meteorological conditions and 2) potential large nor'easter and tropical events.

The program of activities envisioned by the Contract recognized that: 1) the ability to predict flooding and water depths is only as accurate as the data used to develop those predictions and 2) the availability of tidal records within and surrounding the City has historically been limited to the data provided by three (3) long-term tidal gauges at Sewells Point, Money Point, and the Chesapeake Bay Bridge Tunnel. Thus, three (inter-related) work orders issued by the City included: Work Order No. 1- development of a program for installing and monitoring tide gauges, Work Order No. 4 - the installation of those tide gauges, and Work Order No. 3 - the development of a GIS-based model to be subsequently used to apply the knowledge gained from the tidal measurements for anticipating and predicting flooding, prioritizing flood projects, and developing flood remediation measures.

The results of these studies and activities were documented in Fugro's July 2010 ***Preliminary Coastal Flooding Evaluation and Implications for Flood Defense Design*** report (Fugro, 2010).

2.2.2 Current Phase

With the culmination of those initial evaluation's work orders, the focus of the city-wide coastal flooding contract has evolved to focus on: 1) flood mitigation alternative evaluations/concept development for different areas of the City and 2) prioritizing projects for different areas and approaches within and throughout the City. This current report provides the alternatives evaluation and preliminary design of a coastal flooding mitigation option for the Hague watershed in the City. The location of this drainage basin within the City is shown on Figure 2-1. Figure 2-2 shows the extent of the drainage basin and Figure 2-3 shows the area at the outlet of the basin.

2.3 AUTHORIZATION

Work Order No. 1 for the City-Wide Coastal Flooding Study was issued by the City on October 28, 2011. The intent of this current work order is to provide an Preliminary Engineering Feasibility Report that can be used by the City for evaluation, budgeting and project development scheduling. The Fugro team's work scope included the following activities:

- Task A - Site characterization tasks,
- Task B - Hydrological analyses,
- Task C - Initial evaluations and flood design criteria development,
- Task D - Flood mitigation options alternative analyses, and
- Task E - Alternatives analyses report.

As per the City's request, our alternatives evaluations will consider two levels of flood protection, specified as follows:

- A 100-year design, as required for a FEMA certified floodwall, and
- A 10-year design event.

2.4 INCORPORATED DOCUMENTS

The following external documents are incorporated into this report by reference:

The report ***Preliminary Coastal Flooding Evaluation And Implications For Flood Defense Design***, dated July 2010, described preliminary evaluations of coastal flooding susceptibility within the City and its implications for the design of future flood defense improvements. Design water levels for the Hague area and other project areas are based on measurements and analysis presented in this report, hereinafter referred to as the **Preliminary Flooding Evaluation**.

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3.0 ORGANIZATION OF REPORT AND LIST OF ACRONYMS

3.1 REPORT ORGANIZATION

This section intentionally left blank pending discussion of preferred report organization.

3.2 LIST OF ACRONYMS

This section will be updated prior to final report submittal.

- FEMA = Federal Emergency Management Agency
- FIRM = Flood Insurance Rate Map
- FIS = Flood Insurance Study
- SWL = Still Water Level, as determined in effective FEMA FIS
- BFE = Base Flood Elevation, as determined in effective FEMA FIRM and FIS
- FB = freeboard
- SLR = Sea Level Rise
- SP = Sewells Point
- LF = linear feet, e.g. to describe the running length of a floodwall
- % a.c. = percent annual chance of exceedance; terminology used by FEMA to describe exceedance frequency, e.g. 100-year "return period" has 1% annual chance
- 100-year Return Period (RP) = 1% annual chance of occurrence
- 50-year Return Period (RP) = 2% annual chance of occurrence
- 25-year Return Period (RP) = 4% annual chance of occurrence
- 10-year Return Period (RP) = 10% annual chance of occurrence
- 5-year Return Period (RP) = 20% annual chance of occurrence
- 2-year Return Period (RP) = 50% annual chance of occurrence
- 1-year Return Period (RP) = 100% annual chance of occurrence

4.0 THE HAGUE WATERSHED LOCATION AND DESCRIPTION

4.1 WATERSHED DESCRIPTION AND RECEIVING WATER BODY

The Hague watershed is in the southwest portion of the City of Norfolk (Figure 2-1). The watershed includes 2,373 parcels within the 894 acres of land in the watershed. Approximately 8,850 residents of the City live within the drainage basin (as defined by the City's Planning Department).

The Hague, formally known as Smith Creek, is the receiving body of water from the Hague watershed which subsequently feeds into the Elizabeth River. Both bodies of water are tidally influenced and subject to storm surges.

4.2 TOPOGRAPHY AND BATHYMETRY

4.2.1 Topography

The topography of the Hague watershed is generally flat and below elevation (El.) 12 feet NAVD88. Figure 4-1a presents the topography from a 2009 LiDAR-based survey conducted by Pictometry, Inc under contract to the City of Norfolk. Elevation ranges are color coded by 1-foot intervals on Figure 4-1a. A statistical summary of the ground surface elevation is provided on Figure 4-2 and Table 4-1. Approximately 30 percent of the study area lies below El. 8 feet NAVD88. The regional ground surface slopes gently to the southwest. For reference, the 100-year return period (1% annual chance) still water elevation is given as +7.6 ft NAVD88, in the September 2, 2009 effective FEMA Flood Insurance Study for the project area.

The watershed is bifurcated by two primary surface drainage systems that trend northeast-southwest and coincide with reclaimed land overlying former streams/low-lying areas. The two primary drainage systems extend up gradient from the two ends of the Hague's "U"-shaped water body. The axis of the western drainage system is roughly aligned with Stockley Gardens and the eastern drainage system is roughly aligned with Virginia Beach Boulevard and Monticello Avenue. The eastern branch has two secondary reaches that are roughly aligned with Olney Road and Llewellyn Avenue.

The regional slope of the ground surface is toward the southwest. In general, the ground surface slope is less than 0.5 percent but may be locally steeper.

Table 4-1: Summary of Watershed Topography

Elevation (ft, NAVD88)	Number of Acres	Cumulative Number of Acres	Percent of Watershed	Cumulative Percent
Lower than 3	9	9	1.0%	1.0%
3 to 4	17	26	1.9%	2.9%
4 to 5	26	52	2.9%	5.8%
5 to 6	45	98	5.1%	10.9%
6 to 7	72	170	8.1%	19.0%
7 to 8	103	272	11.5%	30.5%
8 to 9	128	400	14.3%	44.8%

Elevation (ft, NAVD88)	Number of Acres	Cumulative Number of Acres	Percent of Watershed	Cumulative Percent
9 to 10	153	553	17.1%	61.9%
10 to 11	146	699	16.3%	78.2%
11 to 12	108	807	12.1%	90.3%
12 to 13	66	874	7.4%	97.7%
13 to 14	12	886	1.3%	99.1%
14 to 15	3	889	0.3%	99.4%
15 to 25	6	894	0.6%	100.0%

4.2.2 Bathymetry

The bathymetry of the Hague water body is markedly different between the areas north and south of Brambleton Bridge. Figure 4-1b shows the bathymetry of the study area based on a hydrographic survey performed by Fugro in December 2010 using a R2Sonic 2024 multibeam echosounder. The survey area extends from the entrance of the Hague water body to approximately 500 feet north of Brambleton Bridge and into the base of the “U”-shaped Hague Cove. Bathymetric elevation in the surveyed area range from El. -7.7 feet to -28 feet NAVD88 (Figure 4-1b).

The bathymetry north of Brambleton Bridge is generally flat and featureless with a the lowest elevation of approximately El. -14 feet NAVD88. Water depth within the Cove increases toward the center of the channels and toward the Brambleton Bridge. The lowest bathymetry elevation is near Brambleton Bridge where is about -28 feet NAVD88. The bathymetry deepening south of the Brambleton Bridge is primarily due to dredging for the purpose of vessel berthing at the NOAA facility, located at the eastern flank of the Hague entrance. Due to dredging operations, the bathymetry south of the bridge is also more irregular than to the north with increasing irregularity toward the shore.

The Hague water body is influenced by natural occurrences as well as by man. Tidal influences may cause loosening and lateral migration of sediment through ebb and flood flow, directions of which are depicted in Figure 4-1b. Storm events are also likely to increase sediment mobility in the area.

4.3 LAND USE

The number of acres and percent of the watershed with the following land use classification (as defined by the City's Planning Department) is summarized in Table 4-2. Figure 4-3 presents a map of the land use in the Hague watershed. As can be seen from the table below, the watershed is nearly fully built out and residential, commercial, and institutional land uses are fairly equal.

Table 4-2: Hague Watershed Land Use Classifications

Usage	Number of Acres	Percent of Watershed
Low Density Residential	83	11.5

Usage	Number of Acres	Percent of Watershed
Medium Density Residential	61	8.5
High Density Residential	124	13.7
Commercial	135	18.8
Institutional	163	22.6
Open Space/Recreational	125	17.4
Transportation/Utility	1	0.1
Industrial	30	4.1
Mixed Use	6	0.9
Vacant	17	2.4

Note: The land usage statistics represent only the area of land within the watershed and do not include the Hague body of water.

4.4 BASIN RIM DESCRIPTION

The perimeter of the watershed is about 33,600 feet (6.4 miles). The perimeter is delineated by the Hospital Complex and Colley Avenue on the west and the railroad paralleling 23rd Street to the north. The eastern perimeter is meanders through several neighborhoods the outskirts of Downtown Norfolk.

Depending on the level of flood protection (i.e., the water level elevation at the basin outlet), there will be a number of areas along the basin rim that will be lower than the elevation of the flood protection at the basin outlet. The low areas around the basin rim are shown on Figure 4-4. The number of locations along the basin rim and the length of the segments below different threshold elevations are summarized as in Table 4-3.

Table 4-3: Low Ground Surface Conditions along Watershed Perimeter

Elevation (ft, NAVD88)	Number of Low Segments	Length of Low Segments (ft)
2.2	3	33
4.2	17	401
4.8	31	691
6.2	47	1,138
7.0	67	2,026
7.6	84	2,775
8.2	107	3,571

Note: The elevation thresholds coincide with the design criteria elevations covered in section 5.0.

As can be seen from the above table, the lengths of elevations below a given elevation do increase as elevations increase. Depending on the elevation selected, additional floodwalls, berming, or road raising will be needed, and the required lengths can range from hundreds to

thousands of feet. Based on review of the available data however, it would appear that protection can be afforded up to and beyond the 100-yr surge event.

4.5 SITE CONDITIONS AT BASIN OUTLET

The basin outlet represents the location of the confluence between The Hague (Smith Creek and the Elizabeth River. The shoreline along the outlet has been modified by land reclamation and construction activities since the late 1800s. Figure 4-5 compares conditions at the basin outlet depicted in an 1894 map and a 2009 aerial photograph. Several structures including piers and bridges have been modified, demolished, or buried over time. Remnants of the former structures may be present in the subsurface and present obstructions for future subsurface structures (e.g. piles, sheetpile walls, etc.).

Currently, Brambleton Avenue Bridge and the recently constructed Light Rail bridge cross the outlet. On the upstream side of the bridges, the outlet is approximately 500 feet wide from shore-to-shore. On the downstream side, the opening is narrower and is approximately 375 feet wide from shore-to-shore. Earthen embankments are present on both sides of the outlet and represent the shore landings of the Brambleton and light rail bridges.

4.6 NAVIGATION REQUIREMENTS

Although The Hague is not a navigable channel, there is some incidental usage of The Hague for small craft. Thus, the City has specified that the entrance to The Hague at the Elizabeth River should provide a minimum draft of 2 to 4 feet, relative to MLLW datum. That elevation corresponds to El.-4 to -6 feet re: NAVD88.

4.7 SUBSURFACE GEOTECHNICAL CONDITIONS (C-4)

Fugro compiled and reviewed available information relative to the subsurface conditions. Primary sources of information were 1961 boring logs from existing Brambleton Bridge design documents and logs from borings conducted in 2006 as part of Light Rail bridge project. After review of the existing geotechnical data, three cone penetration tests were conducted by Fugro in March 2012 to provide additional information used to provide additional geotechnical data for characterization and used to validate the older geotechnical information. Figure 4-6 shows a photograph of a cone penetration test (CPT) being conducted north of Brambleton Avenue on the eastern shore of the Hague Cove. Software applications developed by Fugro were used to characterize the engineering and stratigraphic information in the database. Figures 4-7 shows a key to subsurface cross sections and 4-8 through 4-10 present cross sections depicting interpreted subsurface conditions at the basin outlet.

4.7.1 Geology and Subsurface Stratigraphy

Based on the information reviewed, the subsurface stratigraphic conditions are generally comprised of three stratigraphic units at the basin outlet. In descending sequence, the units are artificial fill, Quaternary age alluvium, Pliocene age Yorktown Formation. The artificial fill represents the Brambleton Avenue embankment and fill materials placed along the shoreline. Exploration logs suggest the material is primarily sand soils with various amounts debris (e.g. brick, gravel, etc.). The artificial fill ranges from about 8 to 20 feet thick. Artificial fill does not appear to be of appreciable thickness in The Hague channel.

Quaternary age alluvium generally underlies the artificial fill. The alluvium is primarily comprised of soft, fine grained silt and clay. Locally, sandy layers up to 10 feet thick may be present (e.g. beneath the southeastern Brambleton Avenue embankment). The thickness of the soft fine-grained sediments encountered by the explorations, range from 5 to 55 feet. The base of this unit likely represents an erosional surface and ranges in elevation from El. -14 to -62 feet. Due the low strength and high variability in thickness, understanding the engineering properties and thickness of this unit may be critical to future foundation designs in this area.

Pliocene age Yorktown formation sediments underlie the fine-grained alluvium. The Yorktown formation is generally comprised of marine silty sands. Regionally, this unit is commonly the end-bearing strata for many piled foundations. As discussed in the previous paragraph, the elevation of the interface between this unit and the overlying soft alluvium can vary significantly in the basin outlet area and will likely play an important role in foundation designs.

4.7.2 Design Subsurface Profiles for Concept Evaluation

In order to conceptually evaluate possible flood mitigation systems at The Hague, it was necessary to idealize the subsurface conditions, and determine soil properties that will govern the flood mitigation system selection and design. Based on the available data and published correlations between different soil parameters, the following were interpreted:

- Two idealized soil profiles representing an upper and lower bound of expected stratigraphy;
- Idealized moisture content profiles;
- Idealized undrained shear strength profiles for the Norfolk Clay layer;
- Friction angle profiles for the artificial fill and Yorktown Sand layers;
- Ultimate bearing capacity values for the upper and lower boundary profiles based on a continuous strip footing with a unit width;
- Active and passive earth pressure coefficients. A drained condition was assumed for the Norfolk Clay material;
- Compressibility values for the Norfolk Clay layer.

Appendix A provides the idealized profiles and description of the data and methods used to develop them.

4.7.3 Foundation Considerations

The soft compressible soils of the Norfolk Clay layer is limited in its ability to support loads. Typically, shallow foundations can only support light loads without experiencing excessive settlement. Therefore, it is common for foundations supporting moderate or greater loads to be founded on piles that are embedded in the Yorktown formation. Appendix A includes a preliminary analysis for foundations of the preferred project alternative and discusses potential implications and recommendations for design.

5.0 COASTAL FLOODING: TIDE- / SURGE-DRIVEN TAILWATER ELEVATIONS (C-2)

5.1 PREVIOUS INTERPRETIVE REPORT AND STUDY IMPLICATIONS

Fugro's July 2010 Preliminary Coastal Flooding Evaluation and Implications for Flood Defense Design report (Fugro, 2010) provided our preliminary evaluations of coastal flooding susceptibility within the City and its implications for the design of future flood defense improvements. The information from the City-wide Coastal Flooding study is considered relevant for not only developing design criteria and designs of public works improvements but also provides important information for various planning studies and emergency response plans within the City.

5.2 TIDES AND SURGE-DRIVEN WATER LEVEL ELEVATIONS

The Hague watershed drains to the Elizabeth River, which is directly connected to the Chesapeake Bay. Inundation by rising waters in the Bay and the Elizabeth River in high tide and coastal storm surge events is a primary source of flooding in the Hague. Long-term measured water levels, supplemented with shorter periods of record from gauges at points around the City, were used in developing extreme water levels to apply in the flooding evaluations, analysis of alternative flood mitigation approaches, and preliminary design of structural and hydraulic elements of the preferred alternative.

5.2.1 Long-term Measured Water Levels at Sewells Point

The most relevant long-term tide gauge to this project site is NOAA #8638610 at Sewells Point. This data set was analyzed using extreme-value statistical methods to estimate water level return periods. Daily maximum measured water levels are available for this location since the original gauge deployment in 1928. The historical data were adjusted to account for historical sea level rise and peak storm water levels were extracted for the statistical analysis. The results of those analyses, which show the relationship of water level (adjusted to the current elevation of sea level) versus return period, are shown on Figure 5-1 and the water levels for various return periods are listed in the following table.

Table 5-1: Tide Elevations at Sewells Point for Various Return Periods

Return Period (years)	Water Level at Sewells Point (ft, NAVD88)
MHHW	1.2
1	3.2
2	3.8
5	4.6
10	5.2
25	6.0
50	6.6
100	7.2

Previous work orders under this contract (see Incorporated Documents) included the installation of five tide gauges within various watersheds. These gauges have provided quantitative data to measure and predict tides throughout the City relative to those at Sewells

Point which – having the longest history of tidal measurements in the area – is the reference location used to communicate predicted tide levels. The approximately 1.5 years of measured tide data include both the normal day-in variations of tidal and meteorological conditions as well as several unusual extreme conditions. The tide gauges captured the November 2009 nor'easter that produced the fourth highest recorded water level at the Sewells Point tide gauge, since it was established in 1928.

5.2.2 Short-term Water Level Measurements in Other Parts of the City

The 2009 - 2010 tide gauge data provide a unique picture of the propagation of flood waters from the Chesapeake Bay and the main stems of the Elizabeth River into the various water bodies within the City. Measured water levels at the five gauge locations vary from less than **0.1 foot below** the water level at Sewells Point to localized water levels nearly **1.5 feet above** Sewells Point in the small Haven's Creek cove. Elsewhere, water levels at the other gages are interpreted to generally range from **0.3 to 0.6 feet above** that at Sewells Point. The elevated water level (as compared to Sewells Point) throughout most of the City has important implications for flood defense design criteria and flood defense performance.

While no tide gauge was located in The Hague, the tide gauge measurements at the Downtown Pump Station gauge provide an appropriate basis for estimating the difference between the water level in the Hague compared to that at Sewells Point. Two days of measurements from a temporary USGS tide gauge in the Hague during the November 2009 nor'easter were within 0.1 foot of the comparable measurements at the Downtown Pump Station tide gauge. The statistical analyses of the measurements at this gauge relative to those at Sewells Point indicated that the high and low tide water levels at this location are on average **0.5 foot above** those at Sewells Point.

The differences of the tide level offset between the local tide gauge and Sewells Point can be due to many local factors, such as wind driven setup (which varies with wind direction and location), localized storm water discharge effects, and local geometric amplifications the effects of wind direction and local geometric amplification (e.g., cove effects). For design applications it is appropriate to consider those temporally variations between the local tide and those at Sewells Point. ***A 0.5 foot increase in tailwater elevations, over the base Sewells Point value, is recommended for the Hague watershed to account for temporal, local effects.***

5.3 CONSIDERATION OF FUTURE SEA LEVEL RISE

Prediction of the rate of potential future sea level rise (and/or future regional subsidence or more local ground settlement) is not part of the current analyses. However, it is appropriate to recognize that if sea level rise continues or accelerates it will increase the frequency and severity of flooding events. Thus, it is appropriate to acknowledge how the potential for future sea level rise may increase flooding within the City.

Published data and evaluations (NOAA, 2010) interpret that the recent rate of **relative sea level rise** at Sewells Point is 1.46 feet/century. To evaluate how a continuation of that rate of sea level rise will affect flooding in the City, the return periods associated with various tide elevations at Sewells Point have been computed assuming a 0.5 foot and a 1.0 foot rise in

future sea level. At the NOAA estimated rate of 1.46 feet/century, these rises correspond approximately to years 2045 and 2080, respectively.

The return periods associated with different tide elevations at Sewells Point – and their modification based on discreet values of future sea level rise – are summarized in Table 5-2.

Table 5-2: Predicted Storm Surge Levels and Return Periods, Current Sea Level Elevation and after 0.5- and 1.0-Foot Increases in Relative Sea Level

Sewells Point Tide Elevation, (ft, NAVD88)	Approximate Return Period (years)		
	based on Current Sea Level	after 0.5-foot rise in Sea Level	after 1.0-foot rise in Sea Level
+5	8	5	2.5
+6	25	15	8
+7	80	50	25

Table 5-2 implies that continuation of the current rate of sea level rise will double the probability of exceeding a particular coastal flood elevation in any given year by about 2045. Put another way, the implication is that in a future with sea level rise, a less severe storm will be able to produce a specific total flood water level. Figure 5-2 illustrates the implications future sea level rise has on the flood water levels for various storm return periods. In addition to increasing the frequency of a specific flood event, future sea level rise also will increase the area of flooding for a specific size storm event.

5.4 COASTAL TAILWATER ELEVATIONS FOR THE HAGUE WATERSHED

Historically, the tailwater elevation for drainage improvement in the City have been based on various water elevations (e.g., mean high water, mean low water, etc.) at Sewells Point. The measurement of water levels using tide gauges throughout the City (Fugro, 2010) has shown that water levels in the various drainage basins within the City are typically elevated over the measurements at Sewells Point. In addition, consideration of sea level rise here-to-before has not been considered in the design of storm water drainage and flood mitigation improvements. The following table documents how those effects have been accounted for in the current storm water and flood mitigation alternatives evaluation.

The following approach was taken to evaluate tailwater elevations for further study and design at the Hague watershed. Starting with extreme total water level values determined from Sewells Point gauge data, a basin offset was added based on the findings of the April 2010 report as discussed above. Second, an additional offset was added to account for wind setup and/or cove setup effects. Finally, a 1.0 foot allowance for future sea level rise was considered. The 1.0 ft allowance for sea level rise is based on a continuation of the rate of sea level rise as documented over the last decade and a structure designed to last 50 to 60 years (NOAA, 2010a). The incremental and cumulative offsets for the Hague watershed are indicated in Table 5-3.

Table 5-3: Tailwater Correction from Sewells Point and Allowance for Sea Level Rise at the Hague Watershed

Consideration	Offset Relative to Sewells Point (ft)	
	Incremental	Cumulative
Basin Offset	0.5	0.5
Wind Direction and/or Cove Offset	0.5	1.0
Allowance for Future Sea Level Rise	1.0	2.0

The 1-ft allowance for sea level rise is based on a continuation of the rate of sea level rise as documented over the last decade and a structure designed to last 50 to 60 years (NOAA, 2010a).

The storm water system's ability to discharge precipitation runoff through the existing outfalls is hindered during high tides and surge events by the elevated tailwater. Figure 5-3 illustrates the tailwater phenomena and the implications it has on storm water drainage systems. Table 5-4 below details the recurrence interval tailwater elevations at Sewells Point and the resulting design tailwater elevations for the Hague watershed (Fugro, 2010), based on Sewells Point plus the basin offset and wind direction / cove offset from Table 5-3.

Table 5-4: Tailwater Elevations at Sewells Point and the Hague Watershed

Return Period (years)	Sewells Point Water Level (ft, NAVD88)	Hague Watershed Design Tailwater Elevation (ft, NAVD88)
MHHW	1.2	2.2
1	3.2	4.2
2	3.8	4.8
5	4.6	5.6
10	5.2	6.2
25	6.0	7.0
50	6.6	7.6
100	7.2	8.2

It was decided to conduct the hydrologic and hydraulic modeling studies and conceptual alternatives analysis without inclusion of future sea level rise, so that focus could be placed on determining the overall costs to meet the desired level of protection for **present flooding levels**. However, a sea level rise component between 0.5 ft and 1.0 ft was included in the subsequent preliminary design phase described later in this report.

6.0 COASTAL FLOODING: PRECIPITATION HYDROLOGY AND HYDRAULICS (C-2)

Coastal flooding events with high tailwater elevations in the Elizabeth River and Smith Creek are highly likely to be associated with intense and/or prolonged rainfall over the entire Hague watershed. The storm water system's ability to discharge precipitation runoff through the existing outfalls is hindered during high tides and surge events by the elevated tailwater. Any engineered solution for mitigating coastal flooding in the Hague watershed must account for this interaction between the storm water system and the elevated water surface in the receiving waters.

An extensive set of hydrologic and hydraulic analyses and model simulations have been conducted, characterizing flooding due to joint precipitation and elevated tailwater events. These analyses are summarized below for the watershed's existing condition and for the various alternative flood mitigation solutions evaluated.

6.1 RAINFALL AND PRECIPITATION

The synthetic 24-hour Soil Conservation Service (SCS) Type II rainfall distribution was used to generate rainfall-runoff hydrographs for the evaluation of design alternatives. The Type II distribution represents the most intense short duration rainfall (NRCS, 1986). The design rainfall duration-frequency depths were derived from precipitation frequency estimates published by the National Oceanic and Atmospheric Administration (NOAA) for the Norfolk International Airport (NOAA, 2004 - nearest station). These 24-hour rainfall amounts are listed in Table 6-1 below.

Table 6-1: NOAA Return Frequency Rainfall Depths for Norfolk KORF Airport

Average Recurrence Interval (ARI) (years)	24-hr Precipitation Frequency Estimate (inches)
1	2.93
2	3.57
5	4.62
10	5.51
25	6.82
50	7.96
100	9.21

6.2 ELEVATION OF PROTECTION

The coastal flood mitigation alternatives evaluation includes the consideration of two different levels of flood mitigation/defense:

- a 100-year return period design, as required for a FEMA certified floodwall, and
- a 10-year return period design event.

6.2.1 10-Year and 100-Year Return Periods

As noted, the water level elevations at Sewells Point that are associated with the 100-year and 10-year return periods are elevation +7.2 and +5.2 feet NAVD88, respectively. Those water levels at Sewells Point correspond to design water elevations in the Hague watershed equal to elevation +8.2 and +6.2 feet NAVD88.

While an additional +1.0 ft may ultimately be added to these elevations for use in final design to account for future sea level rise, the April 2011 concept level designs were completed with present-day water levels given the uncertainty associated with the rate of future sea level rise. Adjustments to required barrier heights and extents have been made during the preliminary design of the preferred engineered solution, but these adjustments are unlikely to significantly influence the hydrologic and hydraulic analyses underpinning the evaluation of conceptual alternatives.

The protection associated with an elevation +8.2-ft NAVD88 is approximately equivalent to a 100-year return period design based on current sea level. After a future 1-foot sea level rise, the +8.2-ft crest elevation corresponds to approximately a 31-year return period event.

The protection associated with an elevation +6.2-ft NAVD88 is approximately equivalent to a 10-year return period design based on current sea level. After a future 1-foot sea level rise, the +6.2-ft crest elevation corresponds to approximately a 3-year return period event.

Given the watershed topography for the Hague, ultimately the floodwall could be designed for an additional one to two feet to accommodate sea level rise. For the purposes of the conceptual alternatives evaluation, it was determined that the designs of the primary, over water floodwalls would be designed with a 2 ft freeboard above the 100-year return period elevation of +8.2-ft NAVD88. This elevation would allow for 1 foot of freeboard in a future scenario with 1 foot of sea level rise. This factor should be confirmed with the City and other stakeholders at each future, subsequent design phase.

6.3 DESIGN COMBINATIONS OF COASTAL WATER ELEVATION AND PRECIPITATION

Based on the expected number of alternatives to be considered for mitigation of coastal flooding, the project team determined that a fixed matrix of tailwater vs. precipitation would be utilized in the study. The simulation matrix includes individual simulations of six different rainfall conditions with (1) tailwater of mean higher high water (MHHW) tide and separately with (2) coincident return period tailwater and rainfall events (e.g., 1-year return period rainfall with 1-year return period coastal tailwater). These scenarios would serve to bracket the expected range of conditions that the proposed alternatives would likely be subjected to during the design life. The combinations of tailwater elevation and precipitation shown in Table 6-2 have been considered in the design alternatives analyses.

Table 6-2: Design Combinations of Tailwater and Precipitation

Design Case	24-hr Design Storm Precipitation (in)	Tailwater Elevation (ft, NAVD88)
1yr RP rainfall, MHHW tide	2.93	+2.2

Design Case	24-hr Design Storm Precipitation (in)	Tailwater Elevation (ft, NAVD88)
2yr RP rainfall, MHHW tide	3.57	+2.2
10yr RP rainfall, MHHW tide	5.51	+2.2
25yr RP rainfall, MHHW tide	6.82	+2.2
50yr RP rainfall, MHHW tide	7.96	+2.2
100yr RP rainfall, MHHW tide	9.21	+2.2
1yr RP rainfall, 1yr RP coastal surge	2.93	+4.2
2yr RP rainfall, 2yr RP coastal surge	3.57	+4.8
10yr RP rainfall, 10yr RP coastal surge	5.51	+6.2
25yr RP rainfall, 25yr RP coastal surge	6.82	+7.0
50yr RP rainfall, 50yr RP coastal surge	7.96	+7.6
100yr RP rainfall, 100yr RP coastal surge	9.21	+8.2

6.4 EXISTING SYSTEM HYDROLOGIC/HYDRAULIC EVALUATION

6.4.1 Selection of Model: XPSWMM

The XP-SWMM software package utilizes the EPA Stormwater Management Model version 5 (SWMM 5) one-dimensional (1-D) analytical engine for running rainfall-runoff simulations for single event or long-term simulations of runoff quantity and quality. XP-SWMM simulates runoff from subcatchment areas and routes it through systems of pipes, channels, pumps, and storage devices.

XP-SWMM also incorporates a two-dimensional (2-D) analytical module for the routing of surface flood flows, based on the TUFLOW program developed by WBM Oceanics Australia and The University of Queensland. TUFLOW is specifically orientated towards establishing the flow patterns in coastal waters, estuaries, rivers, floodplains and urban areas where the flow patterns are essentially 2-D in nature and would be difficult to appropriately represent using a 1-D model. A powerful feature of TUFLOW is its ability to dynamically link to the 1-D network of the XP-SWMM engine. In XP-SWMM, the user sets up a model as a combination of 1-D storm-drain network domains linked to 2-D domains, i.e. the 2-D and 1-D domains are linked to form one model.

6.4.2 Development of Model Inputs

The pipe network for the storm water collection system was modeled using the unsteady state 1-D XP-SWMM's link node modeling module. The 2-D surface model grid, representing street flooding, is linked to the nodes of the 1-D model (representing inlets). Runoff from the hydrologic portion of the simulation enters the 1-D hydraulic model within the pipe system. Storm water that surcharges from the pipe system then becomes surface flow in the 2-D model. The rate at which 2-D surface flow is recaptured by the pipe system is restricted by a maximum inlet capacity, based on the equation:

$$Q \text{ (cfs)} = \text{coefficient} \times \text{grid cell depth (ft)} ^ \text{exponent}$$

The default parameters in XP-SWMM were applied, with the coefficient = 13.385, and the exponent = 0.5. Between the depths of 0ft - 2ft, this approximates an inlet area of roughly 3 sq.ft.

The primary inputs to the XP-SWMM model for this study include:

- Rainfall: time series of rainfall,
- Subcatchment Data: area, overland flow, % slope, % impervious, curve number,
- Junction Data: inverts, depth, ponded area,
- Conduit Data: shape, size, length, roughness, inverts, loss coefficients,
- Outfall-inverts, tide gate, tidal boundary condition,
- Building footprints within the Hague watershed, and
- Topographic Data as a Digital Elevation Model (DEM).

The sources of data used for each of these categories of input are described below.

6.4.3 Rainfall Data

The precipitation frequency depths for the project were based on the published NOAA Atlas 14 values for the Norfolk KORF Airport (NOAA, 2004), applied over the NRCS (formerly SCS) Type-II 24-hour rainfall distribution (USDA, 1986).

6.4.4 Subcatchments

The Hague drainage area was divided into 360 smaller subcatchments based the Light Detection And Ranging (LiDAR) topographic data collected by the City of Norfolk in 2009. Figure 6-2 shows the division of the drainage area into 18 larger catchment areas. Each subcatchment was analyzed to determine input parameters for XP-SWMM. Percent imperviousness and curve number were estimated from USGS data sets representing land use and imperviousness provided by the City. Percent slope was estimated from topography. Other model inputs were simply left as the default values.

6.4.5 Junctions

Junctions represent the point where runoff enters the storm water pipe network in each subcatchment. Junction locations, invert elevations, and rim elevations were derived from the stormdrain database provided by the City. The topography and stormwater junction rim elevations were examined to eliminate erroneous data points.

6.4.6 Conduits

The storm water infrastructure network present in each subcatchment was simplified in XP-SWMM by using one or two stormwater pipes per subcatchment. Conduit sizes and geometries were derived from the stormdrain database provided by the City.

6.4.7 Outfalls

The hydraulic connection between the Elizabeth River and the Hauge watershed / outfall basin was included in the model as part of the 2-D TUFLOW hydrodynamic grid. The storm water outfalls that drain water from the Hague into the Hague cove (Smith Creek) were set up as 1-D nodes with their inverts linked to the 2-D grid. The inverts of the outfalls were determined from the stormdrain database provided by the City. The boundary conditions for the

model simulations were set as a fixed water surface elevation on the edge of the 2-D model grid at the Brambleton Avenue Bridge, where the Hague cove flows into the Elizabeth River. The boundary condition water surface elevation was based on recurrence interval tailwater elevations in Table 6-1.

6.4.8 Buildings

The building footprints were entered into the XP-SWMM model to act as ineffective flow area in the 2-D surface flow calculations. The buildings were derived from the database of GIS information provided by the City.

6.4.9 Topographic Data

In 2009 Pictometry, Inc., under contract to the City of Norfolk, performed a LiDAR survey which provided topographic data at a 3-ft by 3-ft horizontal resolution. Those survey data provide the basis for the 10-ft x 10-ft grid size DEM that was used in the XP-SWMM model for the Hague.

6.4.10 Model Calibration

Detailed calibration data were not available for the Hague watershed. However, the XP-SWMM model results reasonably matched the patterns and depths of flooding in the area as noted by City stormwater staff and were determined to be acceptable.

6.4.11 Existing System Flooding During Various Storm Events

Storm events of various return intervals were run in the XP-SWMM model to evaluate the behavior of the Hague watershed under existing conditions. Design storms were developed for 1-, 2-, 10-, 25-, 50-, and 100-year return period, 24-hr duration rainfall events based on Norfolk International Airport precipitation frequency estimates, which were downloaded from NOAA. This report includes only results for the 10-year and 100-year return period design storms will be presented. Full results from the other design storms are presented in Appendix B.

MHHW Tailwater

The five design rainfall events were simulated in the existing condition XP-SWMM model using a boundary condition water level where the Hague cove outlets to the Elizabeth River equal to MHHW. MHHW for the Hague was determined to be +2.2-ft NAVD88 (Moffatt and Nichol, 2010). Model results for the 10-year and 100-year return period design rainfall events with a MHHW tailwater condition are presented in Figure 6-3 and Figure 6-4, respectively. Model result statistics for each simulation are presented in Table 6-3.

Storm Surge Tailwater

The five design rainfall events were also simulated in the existing condition XP-SWMM model using the corresponding return period coastal surge-driven tailwater elevation as the outlet boundary condition. The recurrence interval storm surge levels used in the modeling are presented in Table 5-4. Model results for the joint 10-year return period rainfall and storm surge and the joint 100-year return period rainfall and storm surge are presented in Figure 6-5 and Figure 6-6, respectively. Model results for each design storm scenario are presented in Table 6-3.

For perspective, the extent of flooding for the 10-year and 100-year coastal storm surges ***without any coincident rainfall*** are presented in Figure 6-7 and Figure 6-8, respectively. As can be seen from the figures, the elevated tailwater associated with tidal surge has a significant impact on the extent and depth of interior flooding. The duration of flooding also is increased with higher tailwater – as the tailwater elevation increases, the gradient decreases, and it takes longer for the storm water system to move the ponded rainfall runoff. This effect is greatest for the higher return periods (larger storms). Nonetheless, it is also apparent from the existing conditions modeling that the interior drainage system also is a serious constraint; the existing storm water conveyance system appears to be able, at best, to carry a ~2-year return period, 24-hr design rainfall with the tailwater at MHHW before flooding moves well beyond the extents of the streets themselves.

Table 6-3: Existing Condition XP-SWMM Model Results

Hague Scenario	Total Storm Runoff Volume (ac-ft)	Max Flood Volume (ac-ft)	Max Flooded Area (ac)	Average of Max Flood Depth (ft)	Average Duration of Flooding (hrs)
1yr RP rainfall, MHHW tide	129.7	66.8	94.0	0.71	1.2
2yr RP rainfall, MHHW tide	172.4	86.1	116.7	0.74	1.4
10yr RP rainfall, MHHW tide	304.7	148.6	175.5	0.85	2.4
25yr RP rainfall, MHHW tide	396.5	192.2	210.8	0.91	3.0
50yr RP rainfall, MHHW tide	479.8	230.1	236.6	0.97	3.5
100yr RP rainfall, MHHW tide	569.5	271.1	262.1	1.03	4.0
1yr RP rainfall, 1yr RP coastal surge	129.9	92.1	112.7	0.82	2.4
2yr RP rainfall, 2yr RP coastal surge	172.2	127.1	141.0	0.90	3.4
10yr RP rainfall, 10yr RP coastal surge	304.7	263.8	224.2	1.18	6.9
25yr RP rainfall, 25yr RP coastal surge	396.6	388.7	281.6	1.38	9.5
50yr RP rainfall, 50yr RP coastal surge	477.4	507.1	329.7	1.54	11.0
100yr RP rainfall, 100yr RP coastal surge	566.4	656.9	380.2	1.73	13.1

7.0 EXISTING CONDITION ESTIMATES OF DAMAGE COSTS

7.1 METHODOLOGY

Flood damage estimates, in terms of monetary costs, were assessed for a range of flooding scenarios under existing conditions and for many of the flood mitigation alternatives, to aid in selection of a preferred design alternative. The initial analysis focused on direct damage to structures and contents of private and public buildings. The purpose of this analysis is to estimate the economic damages associated with future flood events in the Hague watershed, under existing infrastructure conditions, as a basis for performing a benefit-cost comparison of flood damage mitigation alternatives. It is noted that future damage estimates can be further refined by incorporating additional factors such as vehicle damage, displacement costs, emergency response and management costs, and damage reductions resulting from responses to flood warnings.

Structure and contents flood damage assessments were based on predicted flood water depth above the first floor in a structure and the value of the structure. Damage estimates were calculated based on a percentage of the building value where the percentage is a function of the flood water depth. This Depth-Damage Function (DDF) generally increases as the flood water depth increases. DDFs have been developed for various types of buildings by the United States Army Corps of Engineers (USACE), and are published in the "Catalog of Residential Depth-Damage Functions" (USACE 1992), USACE's EGM 01-03 (USACE, 2000) and EGM 04-01 (USACE, 2003). This study used a building inventory file developed by the project team with assistance from the City, output flooding extent and depth results from the hydrologic/hydraulic modeling analyses, high-resolution LiDAR topography data, and flood water DDF curves. A GIS-based routine was developed to calculate and compile the damage estimates for the various flooding scenarios and mitigation alternatives.

Damage assessments were conducted for all 12 of the existing condition scenarios evaluated in XP-SWMM. This section of the report describes the procedure and inputs utilized and presents the results of the damage assessment estimates for existing conditions. Detailed outputs are included in Appendix D.

7.1.1 Building Inventory Methodology

A GIS file of the building footprints was developed for this study and was used to define the spatial locations of buildings in the Hague watershed. The project team coordinated with the City to update building footprints based on 2009 aerial photography. Approximately 2,000 buildings were identified within the Hague watershed for use in the damage assessments.

The buildings were then classified by type using the updated building footprints. The building type was used to determine which DDF would be used for damage estimates. The building type was based primarily on information provided by the City's assessor's office. The information was further refined using high-resolution aerial photographs and site reconnaissance conducted during the study. Building classifications are summarized in Table 7-1.

Table 7-1: Typical Building Classifications

Primary Type	Sub-type	Sub-type	Comment
Residential			Dwelling
	1-Story		
	2-Story		Includes 2 or more stories
	Split-Level		
		Basement	
		No Basement	
Accessory			Detached garage, shed, etc.
Auto Supply			
Clothing			
Department Store			
Grocery Store			
Lodging			Hotel, motel, etc.
Single Story Office			
Multiple Story Office			
Restaurant			
School			
Service Station			

7.1.2 Building Values

Building values were assigned to the buildings based on information provided by the City's assessor's office. Where available, the City's 2010 assessed values were used. In some cases, assessment values were not available and had to be estimated based on similar structures and usage type.

7.1.3 First Floor Elevations

In order to estimate the flood depth at a building, first floor elevations (FFE) were developed. FFE derived from surveyed results were not available for most buildings. Therefore, FFE were developed for using the following procedure. For buildings outside of the 100-year flood zone or were constructed during in 1979 or earlier, we used the 2009 LiDAR data to estimate the FFE. If a building did not have a crawl space (as defined in the assessor's database), we assumed the FFE is 0.5 feet above the ground surface. This assumes an offset for a 6-inch ground slab. If the building has a crawl space, then the offset for the ground surface was assumed based on reconnaissance work conducted during the study. During the study, reconnaissance through the watershed was conducted to estimate and assign the FFE where crawl space height data was incomplete in the database.

If buildings were inside the 100-year flood zone and constructed after 1979, FFE were assigned based on 100-year flood elevation + 1 foot (e.g. 7.3 ft [NAVD88] + 1 ft = 8.3 feet). In August of 1979 the City of Norfolk entered the National Flood Insurance Program (NFIP).

Therefore, per the NFIP, buildings constructed within 100-yr flood zones are required to be 1 foot above the 100-year flood elevation.

7.1.4 Depth Damage Functions - Structures and Contents

A depth-damage function is a mathematical relationship between the depth of flood water above or below the first floor of a building and the amount of damage that can be attributed to that water. The depth damage functions used in this study for residential and non-residential buildings estimate the damage based on a function of the flood water depth at the building and a percentage of the building value. Depth damage functions have been developed for various building types based on statistical studies. Figure 7-1 illustrates the DDF concept and how it relates to FFE. The depth damage curves published by the USACE (1992, 2000, 2003), as described above, were used in this study. The guidance documents provide a "mean" percentage and a "standard deviation" percentage to use when estimating damage from various flood water depths.

7.1.5 Damage Assessment Estimates

The GIS-based damage assessment tool, developed for this study, reads the flood water body outputs from the modeling simulations and estimates the flood water depth for each building based on the building's FFE and flood model output. Structure and content damages were estimated using the flood water depth and respective DDFs. The predicted damage for structure and contents assessments for existing conditions are provided in Table 7-2. The distribution of estimated damages for the 10-year rainfall with MHHW tailwater and the 100-year rainfall with MHHW tailwater are presented in Figures 7-2 and 7-3 respectively. The distribution of estimated damages for 10-year rainfall with 10-year coastal surge and the 100-year rainfall with 100-year coastal surge are presented in Figures 7-4 and 7-5.

Table 7-2: Existing Condition Structure and Contents Flood Damage Estimates

Hague Scenario	Number of Buildings Impacted	Structural Damage ^a (\$, millions)	Contents Damage ^a (\$, millions)	Total Damage ^a (\$, millions)
1yr RP rainfall, MHHW tide	148	10.3 (3.7)	6.3 (2.9)	16.6 (6.7)
2yr RP rainfall, MHHW tide	184	12.9 (4.6)	7.9 (3.7)	20.8 (8.3)
10yr RP rainfall, MHHW tide	334	22.3 (7.5)	13.8 (6.0)	36.2 (13.5)
25yr RP rainfall, MHHW tide	490	24.4 (9.0)	15.7 (7.0)	40.1 (16.1)
50yr RP rainfall, MHHW tide	623	28.9 (10.0)	18.9 (7.8)	47.9 (17.8)
100yr RP rainfall, MHHW tide	757	33.5 (11.0)	22.2 (8.5)	55.7 (19.6)
1yr RP rainfall, 1yr RP coastal surge	150	10.6 (3.9)	6.5 (3.1)	17.2 (7.0)
2yr RP rainfall, 2yr RP coastal surge	185	13.5 (4.9)	8.3 (3.9)	21.9 (8.9)
10yr RP rainfall, 10yr RP coastal surge	336	25.6 (8.7)	15.8 (6.8)	41.5 (15.5)
25yr RP rainfall, 25yr RP coastal surge	493	32.4 (11.4)	20.5 (8.7)	53.0 (20.2)
50yr RP rainfall, 50yr RP coastal surge	625	43.5 (13.3)	27.6 (10.1)	71.1 (23.5)
100yr RP rainfall, 100yr RP coastal surge	760	58.0 (15.3)	36.9 (11.6)	94.9 (26.9)

^a Number in parentheses represents one standard deviation based on recommended depth damage function (DDF) percentage.

8.0 DEVELOPMENT OF FLOOD DAMAGE MITIGATION ALTERNATIVE CONCEPTS

8.1 INTRODUCTION

There are many ways to mitigate the risk, severity, and consequences of flooding. Those approaches can be broadly divided into several categories, such as: 1) drainage and water conveyance system improvements, 2) elevation of the ground surface and structures, 3) construction of barriers to prevent flooding, 4) impoundment and storage of flood waters, 5) adaptive land use to accommodate flooding, 6) relocation and/or abandonment and 7) public policy actions.

The objectives and priorities for flood improvements will depend on technical considerations, as described herein, that define flood risk (frequency, severity, and extent of flooding) and flood hazards. These technical factors together with the many societal factors that define the consequences (and their acceptability, or not) of flooding, and the costs of flood mitigation measures all must be considered and evaluated when defining and prioritizing flood mitigation approach and priorities.

It is important to recognize that the Hampton Roads region has always been subject to flooding. As the region has been developed over the last four centuries, man's activities have altered the landscape. Both human activities (e.g., land filling and changes to runoff patterns) and natural processes (e.g., sea level rise and ground subsidence) have altered the severity and extent of flooding that occurs during any particular event. As the region has been developed, the changes in the land surface have altered the patterns, extent, and severity of flooding - these changes have been ongoing for four centuries.

8.2 FLOOD MITIGATION/DEFENSE STRATEGIES AND OPTIONS

The development of a flood mitigation/defense project requires a sequence of steps; namely: 1) the identification of the flooding hazards, 2) an assessment of the flooding risks, 3) the evaluation of the consequences of flooding, 4) the degree to which those consequences can be accepted or tolerated, 5) an evaluation of mitigation alternatives, and 6) the development and implementation of mitigation and risk management plans.

The nature and risk of flood hazards are defined by technical considerations, such as the predicted:

- Depth of the flooding,
- Size and location of the flooded region, and
- Recurrence intervals or frequency of flooding.

The consequences of flooding are dependent on the potential for loss of life or injury, population and population density, economic losses, disruption of City services, access, and other societal factors. Together the risks and consequences provide the formative information for defining flood mitigation objectives and priorities.

Flood mitigation involves either preventing the flood waters from entering an area, moving the flood waters from the area at a sufficient rate to mitigate consequences, and/or adapting the area to accommodate the flood. These strategies can include both structural and non-structural measures. Different types of flood mitigation strategies can be grouped by the following categories of objectives:

- Drainage or conveyance system improvement,
- Elevation of ground surface or structures above flood elevation,
- Barriers to prevent flooding,
- Impoundment and storage of flood waters,
- Relocation and/or abandonment,
- Adaptive land use to accommodate flooding, and
- Public policy.

Mitigation approaches often include more than one of the above strategies, through combinations of flood mitigation elements such as the following:

- Drainage and conveyance improvements
 - Channelization or improved flood conveyance (stream channel improvements)
 - Storm drainage system improvements
- Elevation of the ground surface and/or structures
- Barriers to flooding
 - Earthen berms and levees
 - Floodwalls
 - Tidegates and barriersDams
- Impoundment and storage
 - Permanent detention and storage ponds or reservoirs
 - Temporary use of land
- Adaptive land use
 - Wetlands, dunes, beach nourishment, and floodplain protected areas
 - Setbacks and buffer areas
 - Land acquisition/relocation and set aside/abandonment
- Public policy
 - Local building and construction code modifications
 - Zoning and land use restrictions
 - Education
 - Flood warning systems, modeling, and forecasting

Although some flood mitigation strategies listed are more commonly thought of as approaches to control flooding from precipitation and rainfall runoff, they can also be components of coastal flooding defense. This is because extreme tides are associated with meteorological events that often produce large amounts of rainfall. For this reason, the design of any barriers to coastal flooding must also be designed to accommodate impounded rainfall and storm water runoff from the area behind the flood barrier. Thus, conventional upland storm water improvements and storage options can and should be components of flood mitigation strategies for mitigating coastal flooding.

A further overview of the different approaches and their applicability is provided in Fugro (2010).

8.3 FLOOD DAMAGE MITIGATION OPTIONS ELIMINATED

Prior to defining the alternate flood mitigation/defense options for evaluation, it was possible to eliminate some approaches due to obvious lack of technical feasibility or other intrinsic factors associated with the approach. Table 8-1 illustrates how the initial screening process was used to eliminate the approaches described below.

Table 8-1: Flood Mitigation Alternatives Feasibility Assessment

Flood Mitigation Alternative Options	Options Deemed Technically/Economically Unfeasible	Potentially Feasible Options	Feasibility Explanation
Drainage & Conveyance Improvements			
Channelization			Lack of land availability
Storm Drainage Improvements			Based on Benefit/Cost Analysis
Elevation of Ground Surface			
Building Elevation			Historical Buildings/Expensive
Grade Raise			Based on Benefit/Cost Analysis
Flood Barriers			
Earthen Berms & Levees			Based on Benefit/Cost Analysis
Floodwalls			Based on Benefit/Cost Analysis
Dams			Based on Benefit/Cost Analysis
Temporary Dams			Based on Benefit/Cost Analysis
Tidegates			Based on Benefit/Cost Analysis
Pump Stations			Based on Benefit/Cost Analysis
Impoundment & Storage			
Permanent Retention Ponds			Lack of land availability
Temporary Use of Land			Lack of land availability
Adaptive Land Use			
Wetlands			Lack of land availability
Beach Nourishment			Lack of land availability
Protected Floodplain Areas			Lack of land availability
Setbacks & Buffers			Lack of land availability
Land Acquisition & Set Aside			Potentially very expensive
Public Policy			
Building Codes			Protect newly built structures
Zoning & Land Use			Limit structures in flood-prone areas
Education			Enhance understanding of flood risks
Warning Systems			Attempt to limit potential damage

The potential flood mitigation approaches that are deemed to be technical unfeasible and the reason for that determination are as follows:

- Storm Water Channelization - There are no open storm water channels in the Hague, and the density of development precludes the use of such storm conveyance device without substantial modification of the land use pattern within the drainage basin.
- Elevation of Structures - The area subject to potential flooding is far too large to consider elevation of structures as a cost-effective mitigation/defense approach.
- Impoundment and Storage - The area is too densely developed and there is insufficient open area for consideration of either permanent or temporary retention ponds.
- Wetlands and Protected Floodplain Areas - There are no wetlands or floodplain areas within the high density developed area of the drainage.
- Beach Nourishment - The area is not located along a beach.
- Setbacks and Buffers - The area is too densely developed and there is negligible open area for consideration of either setbacks or buffers.

8.4 ALTERNATIVES SELECTED FOR FURTHER EVALUTION

Based on the preliminary evaluation, it was determined that four of the flood mitigation elements could be used collectively to aid in mitigating coastal flooding within the Hague watershed. These elements include:

- Ground Surface Improvements
- Storm Drainage System Improvements, and
- Implementation of Flooding Barriers
- Adaptive Land Use

Within these collective elements, several different types of alternatives for flood barriers and drainage improvements were considered to reduce flooding. A total of **11 alternatives** were conceptualized as presented below and were evaluated under the various design storm events. These alternatives are grouped into five categories as presented in Table 8-2. The differentiation between alternatives subscripted Xa, Xb, and Xc has to do with the nature of the gate in the fixed tidal barrier:

- The gate in Alternatives subscripted Xa is a sliding steel tide gate
- The gate in Alternatives subscripted Xb is an Obermeyer gate, and
- The gate in Alternatives subscripted Xc is an inflatable dam.

Table 8-2: Flood Damage Mitigation Alternatives Evaluated

Alternative	Category
1a, 1b, 1c	Tidal Barrier with Tide Gate, Two 60" Dia. Pumps , and Closure Walls and Berms
2a, 2b, 2c	Tidal Barrier with Tide Gate, Four 60" Dia. Pumps, and Closure Walls and Berms
3a, 3b, 3c	Tidal Barrier with Tide Gate, Four 96" Dia. Pumps, and Closure Walls and Berms
4	Bulkhead Wall and Earthen Berm
5	Property Buyout

Each alternative was evaluated for joint recurrence frequency rainfall and coastal surge at the 2-, 10-, 25-, 50-, and 100-year return periods. The conceptual design elevations for the structures were calculated by adding 1.5 feet of freeboard to the coastal surge water levels from Table 5-4. This would provide some protection from wave overtopping and provide 1 foot of additional freeboard (based on FEMA, 2009a). Table 8-3 provides the flood barrier crest elevations for the evaluation of alternatives at the conceptual level.

Table 8-3: Elevation of Structures Based on Storm Events

Storm Event	Analyzed Storm Elevation (ft, NAVD88)	Barrier Elevation with Freeboard* (ft, NAVD88)
2yr RP rainfall, 2yr RP coastal surge	+4.8	+6.3
10yr RP rainfall, 10yr RP coastal surge	+6.2	+7.7
25yr RP rainfall, 25yr RP coastal surge	+7.0	+8.5
50yr RP rainfall, 50yr RP coastal surge	+7.6	+9.1
100yr RP rainfall, 100yr RP coastal surge	+8.2	+9.7

*Heights for the fixed portions of the tidal barriers in Alternatives 1, 2, and 3 are 2.3' higher than elevations shown in order to accommodate the sliding gate into the adjacent wall sections.

A description of each alternative is provided below. Opinions of probable cost for each alternative, over the range of storm recurrence intervals evaluated, are provided in the "Opinion of Probable Cost" section of the report (Section 10.0), along with operation and maintenance considerations over the typical expected service life of each concept. A schematic of the three tide gate type options that were evaluated is shown in Figure 8-1. Figures 8-2 through 8-11 present detailed drawings of all the concepts that were evaluated.

8.4.1 Alternatives 1 through 3: Tidal Barrier Across Smith Creek with Gate, Pumps, and Overland Closure Walls

Alternatives 1 through 3 utilize three main components to protect against coastal (tidal surge) and rainfall runoff. These components include:

- Fixed tidal barrier structures with a movable tide gate to protect against inundation from tidal surge
- Pumps to remove rainfall runoff when the tide gate is closed, and

- Closure walls and/or berms across low lying segments of the area's perimeter to prevent flanking

Tidal Barrier Structures with Tide Gate

The tidal barrier and tide gate would be constructed on the upstream side of the Brambleton Street Bridge in the Hague. The overall length of the conceptual barrier would be approximately 750 linear feet (LF), and it would tie into the existing adjacent ground elevations. The initial design concept for the tidal barrier consisted of two steel sheetpile bulkheads filled with aggregate base and capped with concrete, with a concrete fascia above tidal water levels to preserve aesthetics in this historic area of the City.

The gate assembly would be located in-line with the existing navigational channel and fender system of the bridge. The opening width of the gate would vary with gate type, from approximately 50 LF for the sliding steel gate and Obermeyer gate to 110 LF for the inflatable dam, due to abutment angle requirements. At the gate location, a navigation clearance to at least elevation -4 ft NAVD 88 would be necessary to allow small boat traffic to access Smith Creek through the navigation span of the existing bridge. The three conceptual options for the tide gate are described below and in Figure 8-1:

Steel Gate. The steel gate will utilize steel framing and roll on a guide which will be attached to the foundation by anchor bolts. This gate is similar in nature to the gates utilized within the City of Norfolk's Downtown Floodwall. During the open position, the gate will be stored in a pocket located on one of the opening. Because the steel gates are required to be stored in a pocket this option requires the bulkhead to be an additional 2.3 feet higher than Table 8-3 indicates.

Obermeyer Gate. The Obermeyer gate system utilizes steel gate panels and reinforced air bladders to open and close the gate. The steel gates are attached to the bulkhead by anchor bolts and secured with epoxy grout. The air bladders are clamped to the steel gate anchor bolts and air supply hoses are connected to the bladders. The air supply hoses are used with the operating system and provide a controlled source of compressed air for inflating and deflating the bladders during storm events. The operating systems main components (compressor, motor, etc.) will be stored in the substation with all electrical components for this system and the pumps.

Inflatable Dam. The inflatable dam utilizes a composite material bladder comprised of multiple layers of nylon fabric coated with synthetic rubber with a pneumatic air system to inflate and deflate the dam. The inflatable dam assembly is attached to the bulkhead with a clamp plate and anchor bolt system and connected to the air supply pipes. The air supply pipes are used with the operating system of the dam and will provide a controlled source of compressed air for inflating and deflating the dam during storm events. The operating systems main components (compressor, motor, etc.) will be stored in the substation with all electrical components for this system and the pumps.

Pumps

The pumps which will be used to discharge accumulated storm water on the upstream side of the tidal barrier will vary in size and quantity depending on the alternative:

- Alternative 1 scenarios will utilize three 60-inch diameter pumps (2 operational and 1 backup)
- Alternative 2 scenarios will utilize five 60-inch diameter pumps (4 operational and 1 backup)
- Alternative 3 scenarios will utilize five 96-inch pumps (4 operational and 1 backup).

For all three conceptual alternatives, the intake lines of the pumps would be located upstream of the tide gate and the discharge lines would penetrate through the tidal barrier, discharging immediately downstream of the barrier. Flap gates would be necessary on the discharge side of the pumps to prevent water infiltration back-into the pump system. The pumps would primarily be powered via a connection to underground electric service (via Dominion's existing utility lines in the project vicinity). Emergency backup generators would be located on-site to allow operation during power outages. Given the aesthetics of the Hague community, all electrical components including the generators would be housed in an aesthetically appropriate structure.

Closure Walls and Berms

Closure walls and berms will be constructed on the downstream side of the Brambleton Bridge (see Figure 8-2) and will be used to prevent water infiltration at low lying areas around the basin perimeter. On the west side of the Brambleton Bridge, a closure wall will be constructed parallel to the City of Norfolk's Light Rail and terminate near the Red Cross. On the east side of the bridge, two options have been identified. The primary option would construct a closure wall parallel with north side of Brambleton Avenue starting from the tidal barrier structure and ending just west of Duke Street. In addition to this option, an additional option was analyzed which constructed the wall along the waterfront and connected to the existing City of Norfolk Downtown Floodwall. For the purpose of this study, the most conservative option (closure wall connected to the existing City of Norfolk Floodwall) was used to determine the Opinion of Probable Cost and Benefit Cost ratios. The closure wall will be constructed of steel sheet piling with a decorative cap/face on the landward and channelward side of the bulkhead. Utility relocation and modifications are envisioned for this section of floodwall due to the heavy residential area.

8.4.2 Alternative 4: Bulkhead Wall and Earthen Berm

Alternative 4 includes installing a bulkhead wall on the landward side of the existing granite retaining wall located around Smith Creek. In addition to the bulkhead wall an earthen berm will be constructed on the north side of West Brambleton Avenue west of the Brambleton Bridge (Figures 8-10 and 8-11).

The bulkhead wall consists of 5,900 linear feet of wall constructed of concrete encased H-piles spaced on 10 foot centers. Between the H-piles a precast concrete panel similar to color and style of the existing granite wall will be installed. Since the new wall is not tied into the existing granite retaining wall a slurry trench will be installed. This trench will aid in preventing water infiltration under the precast panels and will be installed the entire length of the wall and extend three feet below the mudline. Landward of the bulkhead wall, fill will be placed to raise the existing grade elevation. A 60-inch wide sidewalk will also be installed landward of the wall.

To prevent tidal infiltration into the existing storm water infrastructure, Tide-Flex valves or flap gates will be installed on all 32 outfalls draining into Smith Creek located around the Hague.

The earthen berm is estimated to be 1,200 linear feet in length and be constructed of earthen fill with a 3:1 side slope. The berm will tie into a fix elevation adjacent to the Brambleton Bridge on the east end and tie into the proposed bulkhead wall on the west end.

This alternative also included installing closure walls at the low points similar to Alternatives 1 through 3.

8.4.3 Alternative 5: Property Buyout

Alternative 5 includes purchasing the property with structures that are identified as high damage risks. Since FEMA does not have an established buy-out criteria for this mitigation option, review of the depth damage function was completed to determine the most feasible correlation. Based on this function, it was determined that a depth damage function of 20% would provide the City an optimal characterization of the required property buyout within the Hague. In addition to buying the property, several other factors were included in the buyout cost. Those factors included:

- Legal & processing cost
- Demolition cost of the existing infrastructure on the property
- Restoration of the purchased property to a park or other low-impact use
- Loss of City Property Tax

9.0 EVALUATION OF CONCEPTUAL ALTERNATIVES

9.1 HYDROLOGIC / HYDRAULIC MODELING EVALUATIONS

Five alternatives were considered in order to reduce flooding of the Hague watershed during storm events. For the first three alternatives, an artificial barrier was placed in the model at the outlet of the Hague cove into the Elizabeth River. Then either two 60-inch pumps, four 60-inch pumps, or four 96-inch pumps were used to drain flood waters out of the cove. These pump sizes were selected based on the magnitude of the pipe flows discharging into Smith Creek and the expected pump flow rates that would be needed to provide some flooding relief. The pump-curves used for the 60-inch and 96-inch pumps are presented in Figure 9-1. Within the XPSWMM model, the pumps started when the water level at the intake exceeded -2 ft NAVD88 and stopped when the water level fell below -6 ft NAVD88. For reference, MLLW at the Sewells Point tide gage is roughly -1.6-ft NAVD88, with a lowest observed water level of -2.7-ft NAVD88.

The fourth alternative simulated the construction of a bulkhead wall around the Hague cove, which prevented storm surges from flooding onto the lower-lying areas adjacent to the cove. In this scenario, the cove was removed from the 2-D model grid and Hague watershed boundary acted as the 2-D grid boundary. The outfalls which drain from the Hague were given tide-gates preventing backflow, and each was assigned a fixed 1-D water-surface boundary condition associated with the model-scenario.

In the analysis, the 1, 2, 10, 25, 50, and 100 yr 24-hr design storms were run in XPSWMM for each alternative for both the MHHW and coincident surge events. The corresponding design event storm surge was used as the tailwater elevation at the pump-outlet or at the outfalls. For the purpose of this report, only results for the 10 year and 100 year design storms will be presented in Figures 9-2 through 9-9. Results from the other design storms are presented in Appendix B. It is important to note that the XPSWMM models show that the upland piping system is adequate for approximately a 2-yr rainfall event and that no appreciable gains in flooding reduction from upland precipitation flooding could be realized no matter the number and size of pumps. The reason for this behavior is that the inlets and upland pipes are so undersized that the floodwaters cannot reach the outfall and Smith Creek fast enough for additional pumps to be effective. In order to provide additional capacity for these systems, significant additional investments would also have to be made and it was determined that the project's main goal should be to reduce the coastal flooding (tailwater) influence on the system to the extent practicable. This would also allow the City to move in a proactive approach to work toward providing coastal flooding relief throughout the City first and get everyone on "a more level playing field" and then start to tackle the upland piping system which would be very expensive due to the limited working space and utility conflicts in highly urbanized areas.

9.1.1 Induced Flooding With Mitigation Alternatives

Construction of the flood walls and gates alone would mitigate inundation by rising coastal tailwater, but could also serve to impound rainfall-runoff that may occur simultaneously with the coastal surge event. The project has been designed to avoid this type of induced flooding by including the described pump systems, which have been sized to remove water impounded during the design rainfall event at a sufficient rate to avoid induced flooding.

9.1.2 Residual Flooding With Mitigation Alternatives

The results for the three pump-alternative scenarios during the 10yr design storm with 10yr storm surge event and the 100yr design storm with 100yr storm surge event are presented in Table 9-1 below. The table includes a comparison of these pump-alternative results versus the existing condition XP-SWMM results. The difference between three pump-alternatives is negligible, because the inlets and upland pipes are so undersized that the floodwaters cannot reach the outfall fast enough for larger pumps to be effective. The on/off trigger elevations for the pumps were the same for the three cases; the minor difference between the three results stems from the oversized pumps draining the pump-well more quickly and rapidly switching on and off. Consequently, for the 4x 96-inch pump alternative, the pumps were active for less time than the other two 60-inch pump alternatives. Figures 9-2 through 9-4 present the results of the three pump alternatives for the 10yr design storm with 10yr storm surge; and Figures 9-5 through 9-7 present the results of the three pump alternatives the 100yr design storms with 100yr storm surge.

Table 9-1: Summary of XP-SWMM Results for Pump Alternatives Modeling

Hague Proposed Pump Scenario	Total Storm Runoff Volume (ac-ft)	Max Flood Volume (ac-ft)	Max Flooded Area (ac)	Average of Max Flood Depth (ft)	Average Duration of Flooding (hrs)
10yr, 10yr 4x96"	304.7	151.2	175.1	0.86	2.01
10yr, 10yr 4x60"	304.7	151.5	175.3	0.86	1.93
10yr, 10yr 2x60"	304.7	151.5	175.4	0.86	1.93
100yr, 100yr 4x96"	566.4	270.0	259.3	1.04	3.54
100yr, 100yr 4x60"	566.4	270.0	259.3	1.04	3.55
100yr, 100yr 2x60"	566.4	270.2	259.3	1.04	3.63
Change vs. Existing Conditions					
10yr, 10yr 4x96"	-	-42.7%	-21.9%	-26.6%	-70.9%
10yr, 10yr 4x60"	-	-42.6%	-21.8%	-26.5%	-72.1%
10yr, 10yr 2x60"	-	-42.5%	-21.8%	-26.6%	-72.1%
100yr, 100yr 4x96"	-	-58.9%	-31.8%	-39.7%	-72.9%
100yr, 100yr 4x60"	-	-58.9%	-31.8%	-39.7%	-72.8%
100yr, 100yr 2x60"	-	-58.9%	-31.8%	-39.7%	-72.2%

The results for the bulkhead wall alternative during the 10yr design storm with 10yr storm surge event and the 100yr design storm with 100yr storm surge event are presented in Table 9-2 below, including a comparison of these results versus the existing condition XP-SWMM results. The bulkhead wall alternative prevented storm surges from flooding inland, but also resulted in storm water accumulating behind the wall. Figures 9-8 and 9-9 present the

results of the bulkhead wall alternative for the 10yr design storm with 10yr storm surge and the 100yr design storms with 100yr storm surge.

Table 9-2: Summary of XP-SWMM Results for Bulkhead Wall Alternatives Modeling

Hague Proposed Bulkhead Wall Scenario	Total Storm Runoff Volume (ac-ft)	Max Flood Volume (ac-ft)	Max Flooded Area (ac)	Average Max Flood Depth (ft)	Average Duration of Flooding (hrs)
10yr, 10yr	304.7	241.0	213.3	1.13	5.08
100yr, 100yr	566.4	550.5	347.0	1.59	8.91
Change vs. Existing Conditions					
10yr, 10yr	-	-8.6%	-4.9%	-4.0%	-26.5%
100yr, 100yr	-	-16.2%	-8.7%	-8.2%	-31.8%

Table 9-3 below summarizes the comparison of proposed condition XP-SWMM results versus the existing condition results. What the table shows is how the pump and barrier alternatives perform better than the bulkhead wall alternative at reducing the volume and areal extent of flooding for all the events, as well as the average duration of flooding for the rainfall and storm surge coincident events. The bulkhead wall alternative only prevented storm surges from flooding inland. The pump alternatives blocked storm surges at the Brambleton Avenue Bridge with a tidal barrier, but also affected the tailwater condition at the outfalls of the storm drain system by allowing the Hague cove to be pumped down to elevations below normal tidal range. During the pump-alternative XP-SWMM simulations, the water surface in the Hague cove was maintained at an elevation 2 to 3 feet below MLLW (-3 to -4 ft NAVD88). This reduction in tailwater elevation improved the hydraulic efficiency of the storm drain system, allowing inland flooding to be drained more quickly.

Table 9-3: Comparison of XP-SWMM Results for Pump vs. Bulkhead Wall Alternatives

Hague Scenario	Change in Max Flood Volume		Change in Max Flooded Area		Change in Average Max Flood Depth		Change in Average Duration of Flooding	
	(vs. Existing)		(vs. Existing)		(vs. Existing)		(vs. Existing)	
	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall
10yr, 10yr	-43%	-9%	-22%	-5%	-27%	-4%	-72%	-26%
100yr, 100yr	-59%	-16%	-32%	-9%	-40%	-8%	-73%	-32%

9.2 FLOOD DAMAGE REDUCTION ESTIMATES

Flood damage estimates were assessed for the flood mitigation alternatives previously described. The procedures followed to estimate the flood damages were exactly the same as used to determine the existing condition damages. The estimated damage results for coincident events are summarized in Table 9-4.

Table 9-4: Estimated Flood Damage Reductions

Alternative	Estimated Structure Damages (\$ Millions)			
	10yr, 10yr ^a	100yr, 100yr ^a	Change vs. Existing Conditions	
			10yr, 10yr	100yr, 100yr
1a, 1b, 1c (2 x 60" Pumps)	18.7 (6.3)	25.8 (8.7)	-27%	-68%
2a, 2b, 2c (4 x 60" Pumps)	18.7 (6.3)	26.3 (9.0)	-27%	-55%
3a, 3b, 3c (4 x 96" Pumps)	18.7 (6.3)	26.3 (9.0)	-27%	-55%
4	24.4 (8.4)	50.1 (14.)	-5%	-14%
5	N/A	N/A	N/A	N/A
Estimated Contents Damages, millions				
1a, 1b, 1c (2 x 60" Pumps)	11.4 (5.0)	16.4 (6.7)	-28%	-56%
2a, 2b, 2c (4 x 60" Pumps)	11.4 (5.0)	16.7 (6.9)	-28%	-55%
3a, 3b, 3c (4 x 96" Pumps)	11.4 (5.0)	16.7 (6.9)	-28%	-55%
4	15.0 (6.6)	31.8 (10.)	-6%	-14%
5	N/A	N/A	N/A	N/A
Estimated Structure and Contents Damages, millions				
1a, 1b, 1c (2 x 60" Pumps)	30.2 (11.3)	42.2 (15.4)	-27%	-55%
2a, 2b, 2c (4 x 60" Pumps)	30.2 (11.3)	43.0 (16.0)	-27%	-55%
3a, 3b, 3c (4 x 96" Pumps)	30.2 (11.3)	43.0 (16.0)	-27%	-55%
4	39.5 (15.0)	81.9 (24.8)	-5%	-14%
5	N/A	N/A	N/A	N/A

^a Number in parentheses represents one standard deviation based on recommended depth damage function (DDF) percentage

10.0 CONCEPTUAL LEVEL OPINION OF PROBABLE COSTS FOR FLOOD DAMAGE MITIGATION ALTERNATIVES (C-19)

10.1.1 Capital Costs

A conceptual opinion of probable costs was developed for each of the modeled alternatives. Unit costs were based on available data from local contractors, RS Means, vendors, VDOT and other sources as needed. The opinions of probable cost include:

- Construction costs for civil, structural, electrical, mechanical, and environmental components of the project,
- Overhead & Profit for construction,
- Engineering/Construction Observation, and
- Contingency

Table 10-1 presents a summary of the probable cost in 2010 dollars for each alternative. Details of the preliminary opinions of probable costs are presented in Appendix C. Each alternative includes a price breakdown relative to the storm event analyzed. These elevations include storm events for the 2, 10, 25, 50 and 100 year storm events for both MHHW and coincident events.

Table 10-1: Opinion of Probable Capital Costs for Flood Mitigation Alternatives

Alternative	Opinion of Probable Costs (\$ Millions)	
	10-year Storm	100-year Storm
1a	\$44.6	\$47.4
1b	\$47.2	\$50.8
1c	\$52.2	\$56.7
2a	\$56.1	\$59.5
2b	\$58.7	\$62.3
2c	\$63.8	\$68.7
3a	\$90.1	\$94.0
3b	\$92.7	\$97.4
3c	\$97.9	\$102.2
4	\$22.4	\$26.4
5	\$76.9	\$462.1

Based on the conceptual opinion of probable cost breakdowns, the tidal barrier options relative to the type of tide gate had a variance of approximately \$9 million with the Steel Gate being the most cost-effective option and the Inflatable Dam being the most expensive.

10.1.2 Operational & Maintenance (O&M) Costs with Respect to Design Life (C-15)

The standard serviceable design life for Alternatives 1 through 3 are estimated to be 50-years. This design life means that if it is properly maintained, the structure will be able to maintain a functional level of serviceability for at least 50 years before requiring replacement due to either deterioration or operational changes. The operational and maintenance costs associated with these alternatives will vary given the different components such as pumps (sizes and quantities) and gate structures (rubber, rubber & steel, and steel). Maintenance costs and operational costs take into account a wide range of variables which include but are not limited to:

- Inspection costs,
- Minor repairs,
- Major repairs,
- Replacement costs,
- Equipment upgrades,
- Machine maintenance,
- Pumps and power costs, and
- Labor costs during "closure" events.

Operational and Maintenance Costs for each alternative are provided in Table 10-2 and breakdowns for each alternative are provided in Appendix C. Assumptions for the operational and maintenance costs included:

- Routine inspections on bulkheads, gates, floodwalls (Typically on a 5-year cycle)
- Minor repairs (Years 15,35, and 45)
- Major repairs (Years 25 and 40)
- Replacement of pumps (Year 30)
- Operational costs for storm events per year (8 events per year)

Table 10-2: Alternative Operational & Maintenance Costs

Alternatives	Annual Operational Costs (\$)	50-yr Operational Costs (\$) Present Worth
Alt 1a: Tidal Barrier with Steel Gate, 2 - 60" Dia. Pumps, Closure Walls and Berm	\$231K	\$3.2M
Alt 1b: Tidal Barrier with Obermeyer Gate, 2 - 60" Dia. Pumps, Closure Walls and Berm	\$251K	\$3.5M
Alt 1c: Tidal Barrier with Inflatable Dam, 2 - 60" Dia. Pumps, Closure Walls and Berm	\$275K	\$3.8M
Alt 2a: Tidal Barrier with Steel Gate, 4 - 60" Dia. Pumps, Closure Walls and Berm	\$360K	\$5.0M
Alt 2b: Tidal Barrier with Obermeyer Gate, 4 - 60" Dia. Pumps, Closure Walls and Berm	\$380K	\$5.2M
Alt 2c: Tidal Barrier with Inflatable Dam, 4 - 60" Dia. Pumps, Closure Walls and Berm	\$404K	\$5.6M
Alt 3a: Tidal Barrier with Steel Gate, 4 - 96" Dia. Pumps, Closure Walls and Berm	\$465K	\$6.4M
Alt 3b: Tidal Barrier with Obermeyer Gate, 4 - 96" Dia. Pumps, Closure Walls and Berm	\$485K	\$6.7M
Alt 3c: Tidal Barrier with Inflatable Dam, 4 - 96" Dia. Pumps, Closure Walls and Berm	\$509K	\$7.0M
Alt 4: Bulkhead Wall and Earthen Berm	\$127k	\$1.8M

These maintenance and operational costs will be used in conjunction with the Opinion of Probable Cost and damage assessments to determine the Benefit - Cost for all alternatives.

Alternative 5 - Buyout Option does require some maintenance or operational costs due to the fact that the passive use ultimately envisioned (park, etc.) The estimates included demolition, legal processing, site clean-up, reconstruction and a contingency to account for this. Loss of City revenue from property tax was also considered under this evaluation. This loss was calculated by taking the property value purchased and multiplying it by the current property tax rate of \$1.10 per \$100 dollars of property value. City revenue loss over the life of 50 years for each storm event scenario is provided below in Table 10-3.

Table 10-3: Property Buyout Revenue Loss

Buyout -	Revenue Loss (\$ Millions)
20% Damage Buyout - 2 Year Storm Event	\$4.75
20% Damage Buyout - 10 Year Storm Event	\$11.67
20% Damage Buyout - 25 Year Storm Event	\$30.26
20% Damage Buyout - 50 Year Storm Event	\$44.47
20% Damage Buyout - 100 Year Storm Event	\$70.15

The Revenue Loss will be used in Opinion of Probable Cost and damage assessments to determine the Benefit - Cost for all alternatives.

The 11 alternatives varied in cost from \$26.4M (Bulkhead Wall and Berm) to \$47.4M (Steel Gate and 2-60" Pumps) to \$462.1M (Property Buyout) for the 100-Year storm events. However, from the flood damage results determine in Section 9, Alternative 4 - Bulkhead Wall and Earthen Berm may not be the most cost effective option. Section 9 indicated that the pump and barrier alternatives perform better than the bulkhead wall alternative at reducing the volume and extent of flooding for all the events resulting in lower damage results. In order to select a preferred alternative entirely based on performance, a benefit-cost ratio analysis was completed for the studied alternatives. The benefit-cost ratio analysis can be found in Section 11.0.

11.0 SELECTION OF PREFERRED CONCEPTUAL ALTERNATIVE

11.1 BENEFIT - COST (B/C) ANALYSIS RATIO (C-19)

For this portion of the assessment, the FEMA Benefit-Cost Analysis (BCA) analysis procedure was used because it is an established process and will be required in the event that there becomes an opportunity to solicit FEMA funding. This analysis calculated the benefit-cost for all flood mitigation options described above and took into account several factors including:

- Probability of storm events and their re-occurrence related to damages and benefits on an annual basis,
- Design life of the mitigation option,
- Capital costs with O&M cost at present value,
- Estimated flood damages avoided with implementation of mitigation options.

FEMA traditionally calculates these flood damage options by taking into several factors; however, as described in the previous Section 7.0 Flood Damage Estimates only direct damages to the structure and its contents were calculated for this particular assessment. If the City indicates interest in soliciting FEMA funding then the damage values incorporated will need to be refined by incorporating additional factors such as vehicle damage, displacement costs, emergency response, management costs, lost business income, lost rental income, and damage reductions resulting from responses to flood warnings (FEMA, 2009b).

11.1.1 Probability of Storm Events and Their Re-Occurrence Related to Damages

This factor was used to estimate the total damages that may occur within the design life of a mitigation option on an annual basis for each storm event. For example, a 2-yr event has a factor of 0.5 given that it has an annual probability of occurrence of $1/R = 1/2 = 0.5$. Likewise, a 100-yr event has a probability of $1/100 = 0.01$ of happening in a given year. These probabilities could then be multiplied for the pre- and post-project damages for the individual storms and summed to determine an overall annualized damage for pre- and post-project conditions. The difference between the two would be the project benefit.

11.1.2 Design life of the Mitigation Option

Based on FEMA B/C requirements, the required design life for structures is estimated to be 50 years (FEMA, 2009b).

11.1.3 Present Value of Project

Based on FEMA and OMB direction a 7% interest rate was utilized for the present value analysis. The initial costs as well as the ongoing O&M costs were brought to present value as well as the benefits which are defined as the reduction in damage with the project in place (see Appendix D for calculations)(FEMA, 2009b).

11.1.4 B/C Ratio

Once the project benefits and costs are brought to present value, the B/C ratio can be computed which is simply the benefits divided by the costs. A B/C ratio over 1.0 would denote that the project benefits outweigh the project costs and the higher the B/C ratio the more cost effective and advantageous the project. Table 11-1 summarizes the B/C ratios for the various

alternatives. The B/C ratio of the alternatives analyzed indicates that Alternative 1a - Tidal Barrier with Steel Gate, 2 - 60" Pumps, and Closure Walls and Berms is the most cost effective alternative with a Benefit Ratio of 1.34 for a 100-year storm event. Figures 11-1 illustrates the relationship of the various alternatives for 10-year versus 100-year design events.

Table 11-1: Benefit-Cost Ratio (relative to damage to structure and contents)

Alternative	Estimated Benefit to Cost Ratio	
	10yr, 10yr	100yr, 100yr
1a	0.97	1.34
1b	0.91	1.25
1c	0.83	1.12
2a	0.76	1.05
2b	0.72	1.00
2c	0.67	0.91
3a	0.48	0.67
3b	0.47	0.65
3c	0.44	0.62
4	0.45	0.57
5	0.99	0.42

A review of the previously developed cost information shows that the inflatable dam and Obermeyer gate options are more expensive than the steel gate option (mainly due to the additional width and materials needed to provide navigation access). Furthermore, steel gates are likely to be more reliable than the Obermeyer gate and inflatable dam options. Therefore, our recommendation is that a steel gate be utilized. Figure 11-2 shows the relative cost vs. return period for a coastal event. It is observed estimated costs for the different flood mitigation options only slightly increases for the range of design storm return periods. This is because these structures are so deep (due to geotechnical considerations) that adding another one to two feet is within 5-15% of the total project cost. Therefore, the 100-yr return period event should be selected in design flood mitigation structures.

In addition to the project costs, the various B/C ratios were plotted to determine the optimal solution. As shown in Figure 11-2, the B/C ratio analysis also points to the fact that the Alternative 1a steel gate, 2 -60" pump option should be selected and designed for the 100-yr event.

Additional benefits provided by Alternative 1a include: 1) improved access to Hospital during flood emergency, 2) increased protection for the Freemason District, and 3) increased protection for Light Rail.

This option will provide flood damage mitigation today for a 100-yr return period coastal surge level and approximately a 2-yr rainfall event. Additional upland drainage improvements can be phased in over time to improve the upland flooding situation. Depending on future

improvements to the drainage system delivering rainfall-runoff to the Hague cover, additional pumps through the main floodwall may be warranted to keep pumping capacity in-line with the conveyance ability of the upland system.

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12.0 PRELIMINARY CIVIL AND STRUCTURAL ENGINEERING DESIGN OF PREFERRED ALTERNATIVE 1A (ITEMS C-6,C-7,C-8)

The purpose of this report is to document further design and feasibility study work on the the preferred design alternative for mitigating coastal flood penetration into the Hague area. This preferred alternative is referred to in the conceptual flood mitigation alternatives evaluation (Section 8.4) as Alternative 1a, and it was chosen primarily because it indicated the highest benefit/cost ratio of the several design alternatives considered.

The purpose of this additional design work on the single preferred alternative is to further evaluate the technical feasibility of Alternative 1a and to refine the opinion of probable capital cost for the project. This preliminary design is also intended to serve as a basis for discussion o of the project with various stakeholders and potential project partners. To that end, Alternative 1a has been developed to an approximate 10% level of preliminary design, and this preliminary design is documented in 11"x17" drawings attached as Appendix E.

No additional field geotechnical data, field environmental data (wetland delineations, habitat assessments, water quality / flow measurements, etc.), topographic survey field data, or field utility data have been collected, beyond the data described above and used in the conceptual alternatives analysis.

12.1 FUNCTIONAL DESIGN REQUIREMENTS AND DESIGN CRITERIA

12.1.1 Functional design requirements

Design coastal tailwater elevations and conceptual design elevations are discussed in various sections above in this report. Design water levels and crest elevations utilized in the present preliminary design of the Alternative 1a style project are provided in Table 12-1.

**Table 12-1: Design Water Surface Elevations for Components of Alternative 1a
Preliminary Design**

Storm Event	Barrier Elevation (ft, NAVD88)	Allowance for Wave Setup, Sea Level Rise	Remaining Freeboard
Primary tidal barrier and tide gate across Smith Creek entrance	+10.7	0.5 ft + 1 ft	1.5 ft
West overland flood barrier	+9.7	0 ft + 0.5 ft	1 ft
East overland flood barrier – connection to existing Downtown Flood Wall	+9.9	0.2 ft + 0.5 ft	1 ft
Start-up of pumps at primary tidal barrier (min.)	+2.5	-	-
Shut-off of pumps at primary tidal barrier (max.)	--2.0	-	-
100-year return period tailwater level (Error! Reference source not found.)	+8.2	-	-

At the +9.7 ft NAVD88 design water level, the western overland barrier alignment is approximately 840 feet long, and street grade elevations are necessary around two intersections to the west of the overland barrier. To accommodate a meaningfully higher design water level in the area west of Smith Creek – for example a +10.7 ft NAVD88 elevation to

provide additional freeboard with future sea level rise – the western overland barrier would have to be extended to a length of approximately 2,560 feet over relatively low-lying areas along Front Street requiring a greater exposed barrier height. Details and drawings of the western overland barrier alignment are discussed in the relevant section below.

The eastern overland flood barrier is intended to connect to the existing Downtown Flood Wall, which has a crest elevation – at the tie in point – of approximately +9.9 ft NAVD88. The design crest of this barrier could be higher to allow for additional sea level rise or wave effects. The linear extent of an eastern overland barrier tying into the existing Downtown Flood Wall would remain the same, regardless of design crest elevation.

The Hague cove is utilized by small water craft, and the City has specified that the entrance to the cove – through the Brambleton Ave. Bridge and the tidal barrier gate – should provide a minimum draft of 2 to 4 feet relative to MLLW datum (equivalent to elevation -4 to -6 ft NAVD88 datum). This requirement has been incorporated into the preliminary design by setting the gate sill at an elevation of -6 ft NAVD88.

The XPSWMM modeling of the Hague watershed shows that the pumping system at the Smith Creek tidal barrier need to be able to convey approximately 620 cubic feet per second (cfs) – or 278,275 gallons per minute (gpm) – during the peak of the 100-year return period 24-hr rainfall event, against a 100-year return period coastal tailwater elevation of +8.2 ft NAVD88.

Additionally, review of the available information on the City's existing storm drain system indicates that a smaller pumping system may be needed in the Freemason District to discharge interior drainage that would be impounded by the proposed east overland flood wall through that area. The rainfall-runoff volumes for the design storm event have not been modeled in XPSWMM for this small drainage area, which is isolated from the larger storm drain network presently flowing to the Downtown Pump Station.

12.1.2 Applicable codes and standards

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12.1.3 Design loads and load combinations

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12.2 OVERVIEW OF THE PREFERRED PRELIMINARY DESIGN ALTERNATIVE

A total of 11 alternatives were evaluated in the conceptual stage of project development, covering a range of coastal surge and rainfall event magnitudes. From that evaluation, the design alternative with the highest Benefit-Cost (B/C) ratio was selected, and further engineering effort has been put into developing that Alternative 1a to a 10% preliminary level of design.

Alternative 1a consists of a fixed tidal barrier across Smith Creek (at the Brambleton Ave. bridge), with a movable (sliding) steel gate at the navigation section of the bridge. The gate would be closed in advance of a predicted extreme high tide or coastal surge event.

Additional fixed barriers, in the form of short to moderately high steel and concrete walls, are required to enclose low-lying adjacent areas and prevent flood penetration by flanking of the Smith Creek tidal barrier. The present development of Alternative 1a consists of approximately

840 ft of overland flood wall to the west of Smith Creek. A second, longer overland flood wall (approximately 2,625 ft) is included to the east of Smith Creek to connect with the existing Downtown Flood Wall.

Most of the rainfall-runoff captured within the Hague watershed drains to Smith Creek and from there to the Elizabeth River. With the Smith Creek tidal barrier gate closed, stormwater discharges to Smith Creek would be impounded landward of the Brambleton Ave. bridge. The completed Alternative 1a project therefore requires the inclusion of a pump station at the Smith Creek barrier, with associated equipment including back-up power generators.

A smaller pump station along the eastern overland flood wall would also likely be necessary to remove impounded storm water that currently discharges through storm water outfalls near Botetourt St.; the existing storm water outfalls would be closed in the design flood event in order to prevent inundation behind the east overland flood barrier due to storm drain backup.

Preliminary (10% level) design plan and section drawings have been prepared, based on the conceptual Alternative 1a described in a previous section of this report. The drawings, included as Appendix E to this report, are intended to be sufficient for utility coordination, interagency discussion, and refined cost estimation relative to the preferred flood damage mitigation design alternative.

12.2.1 Overview of Appendix E Drawings

The cover sheet identifies the project owner as the City of Norfolk and shows the location of the project area within the City. The cover sheet also provides an index to the contents of all of the sheets in the drawing set. The scale values identified on all of the drawing sheets are applicable at full size, in this case a 22" x 34" printed sheet. The drawings have been included in Appendix E at half-size (11" x 17"), and dimensions scaled off of the drawings must be adjusted from the printed scale values.

Sheet G-101 gives an overview of the locations and alignments of the various project elements. Starting in the east, the east overland barrier is shown as a steel sheet pile wall winding through the downtown area and historic Freemason District, and connecting to an existing but limited length of higher ground at Brambleton Ave. The main tidal barrier across Smith Creek begins on the north side of Brambleton Ave. and extends on the landward side of the bridge to existing high ground at the northwest end of the bridge. The project continues to the west of the bridge in the form of additional steel sheet piles walls and – potentially – elevated crosswalks at two intersections as shown. It is currently proposed to locate a building on City-owned property at the northwest end of the tidal barrier, to house back-up power generators and electrical equipment for the pump and gate motors. More detailed descriptions of each of the project elements are given in the sections below.

Sheet V-101 shows existing topographic and bathymetric contours, while known available data on utility types and alignments are shown on Sheet V-102. The utility data held at present is not complete and likely contains inaccuracies that will need to be resolved in any subsequent stages of project design, as will be discussed below.

Sheets GR-101 illustrates the locations of various historical geotechnical borings noted in Section 4.7 above.

The remaining Appendix E sheets consist of Structural (S), Civil (C), and/or Electrical (E) design plans and sections. These will be discussed by project element in the sections below.

12.3 DESIGN OF PRIMARY SMITH CREEK TIDAL BARRIER AND GATE

The fixed tidal barrier across the Smith Creek entrance, along with its gate and the associated pumps, are shown in plan view on Sheet S-101. This sheet also includes approximate existing grade contours (re: NAVD88 datum) and the known available information on existing utility alignments. Structure elevation and section views for these project elements are shown in Sheets S-201, S-401, and S-501.

12.3.1 Fixed Wall

The tidal barrier and tide gate would be constructed on the upstream side of the Brambleton Street Bridge in the Hague. It must be constructed far enough landward of the bridge to allow sufficient room for construction access and operations, primarily considering requirements for batter pile driving angles. The overall length of the conceptual barrier would be approximately 780 linear feet (LF).

The initial design concept for this tidal barrier – during alternatives evaluation – consisted of two steel sheetpile walls filled with aggregate base and capped with concrete. It was understood that a modified section would be needed where the gate would be housed inside the wall. A visually attractive concrete fascia would be applied above tidal water levels to preserve aesthetics in this historic area of the City.

During preliminary (10% level) design development, structural engineering calculations were conducted. These calculations, based in part upon the geotechnical information discussed in Section 4.7, indicated a change in the wall section type (Sheet S-201) would be necessary to prevent unacceptable deflection under the 100-year return period design water level condition described above. It is particularly important to limit deflection in sections where the sliding steel gate will interact with the fixed wall.

In the center reaches of the barrier, where water depths are greater and geotechnical conditions generally poorer, the preliminary design section consists of a combination of 60-inch diameter by 0.75-inch thick steel pipe piles (PP 60" ϕ x 3/4"), spaced 9.8 ft on center, connected by AZ 26-700 steel sheet piles over the 4.8 ft gap between pipe piles. The pipe piles provide significant additional structural resistance to overturning and deflection than would be feasible with sheet piles alone. Required pipe pile lengths range from approximately 80 to 100 ft; at this 10% level of design, the sheet piles are considered to extend to the same depth as the pipe piles.

The combination wall is not necessary at the southeast and northwest land tie-in segments of the wall. Along these segments, the wall is proposed to be a single row of AZ 26-700 steel sheet pile. Required sheet pile lengths range from 50 to 100 ft.

Pipe pile and sheet pile sizes and embedment depths depend heavily upon the available geotechnical data and derived engineering properties of the existing soil layers. No additional field geotechnical data has been collected and analyzed to date along the proposed alignments of any of the project elements. As such, this preliminary design of required pile penetration is subject to substantial revision at later stages of design development, pending required

additional field data collection. These revisions may include value-engineering to optimize the lengths of steel sheet piles between the steel pipe piles. As noted above, sheet pile lengths are presently considered to be the same as adjacent pipe pile lengths.

The crest of the wall along most of its length is set at +10.7 ft NAVD88 as shown in Table 12-1 above. The crest is elevated by an additional 2.25 ft in the gate abutment sections, to accommodate the sliding gate.

The steel pipe piles and sheet piles would be covered with a concrete cap above elevation -2 NAVD88 (approximately 0.5 ft below Mean Low Water), to address aesthetic requirements in this historic district. The concrete cap may also provide personnel access for pump inspection and maintenance, if appropriate safety features are included.

12.3.2 Steel Gate

The preliminary design of the sliding steel gate is shown on Sheet S-201, S-401, and S-501. The overall conceptual design of the gate is substantially as assumed during the conceptual design evaluation, and the details of member types and connects have been significantly developed during the preliminary design phase.

The gate utilizes welded steel framing and roll on a guide which will be attached to a concrete sill supported by the steel combination wall. Additional supports against deflection, in the form of 24-inch square concrete batter piles, are provided at the abutments of the gate opening on the main flood wall.

The gate assembly would be located in-line with the existing navigational channel and fender system of the bridge. The opening width of the gate is 65 ft, coinciding with the width and position of the Brambleton Ave. Bridge navigation span. At the gate location, the gate sill elevation has been set to -6 ft (NAVD 88) to allow small boat traffic to access Smith Creek.

12.4 DESIGN OF PUMPING FACILITIES

12.4.1 General description

The preferred Alternative 1a requires two operational pumps capable of jointly conveying approximately 620 cfs against approximately 10 ft of total dynamic head, in order to achieve the flood damage reductions utilized in the conceptual design alternatives evaluation. A third pump is included in the design as a back-up, in case one of two primary pumps fails to operate during a flooding event. The present preliminary design of the pump systems is based on using the Moving Water Industries (MWI) model SEA360 or substantially similar model of pump. submersible electric motor drive. The SEA360 is a submersible electric motor driven pump with a 60-inch diameter impeller and a 60-inch diameter discharge pipe flange. Information supplied by MWI indicates that the SEA360 is capable of conveying the necessary flow rates against the range of total dynamic head expected during the design coastal flooding storm events.

Each of the three pumps would be mounted behind and immediately against the combination wall. The pumps must be mounted at an elevation that allows at least 4 ft of water depth over the intake pipe elevation at pump startup. The intake bell entrance must be at least 3 ft above the bed or sump elevation. For both criteria, greater depths are allowed and preferred. Setting the pumps at a relatively lower elevation than absolutely necessary will allow the pumps to be started at lower water levels, for example during maintenance testing or earlier during the

rising leg of a storm event. Sheet S-201 shows the pump intake elevation as -4.0 ft NAVD88, which would allow a minimum water surface elevation of 0 ft NAVD88 for pump startup.

At present, it is considered that the most appropriate location for the pumps is to the east of the gate opening, where greater existing and historical water depth is available under the pump intake bells. The pumps are shown supported on a typical strainer bar stand (e.g. by MWI) and a large roller. The strainer stand provides vertical support and prevents large trash or debris objects from impinging on the pump impellers. The roller also provides vertical support and will allow for some movement of the assembly perpendicular to the wall, if the wall deflects slightly during a flood event. The stand and roller are supported by a concrete slab on a concrete pile foundation; the slab will help to avoid bed scour during operation of these high-capacity pumps.

Pump dimensions and elevations shown in the preliminary plans and sections are based on the designer's interpretation of information provided by MWI. MWI did not supply, nor were they requested to supply, site-specific or pump model-specific drawing or CAD files at this early preliminary stage of design development. The relatively large size of the proposed individual pumps in this system is likely to require considerable custom mechanical, electrical, and structural mounting design work during subsequent detailed design phases.

The discharge lines would penetrate through the wall, discharging immediately downstream outside the barrier. During flood events, the discharge would be submerged beneath the elevated coastal surge water levels. Flap gates would be necessary on the discharge side of the pumps to prevent water infiltration back-into the pump system.

12.4.2 Electrical and mechanical requirements

The pumps would primarily be powered via a substation with electric. Emergency backup generators would be located on-site to allow operation during power outages. Given the aesthetics of the Hague community, all electrical components including the generators would be housed in an aesthetically appropriate structure.

Sheet E-001 shows the proposed location of the generator and equipment building in relation to Brambleton Ave. and the proposed tidal barrier across Smith Creek. Preliminary building dimensions and major equipment arrangement are shown on Sheet E-101 and Sheet E-201.

Sheet E-601 is a single-line diagram of the envisioned electrical system supplying the pumps. The arrangement assumes that primary power would be supplied from a metered connection to the electric power grid (via Dominion's existing lines). Secondary (emergency back-up) power would be supplied by generators housed in the proposed building. The generators are envisioned to be fueled by natural gas (via VNG existing lines), and they would be connected to the pump motors in parallel with the external electric power. Utility alignments (Sheet V-102) obtained from Dominion and VNG indicate that both types of lines are present near the proposed generator building location. Each pump would be operated by a variable frequency drive (VFD).

12.5 DESIGN OF ADDITIONAL PERIMETER WALLS

12.5.1 General description

In order to make the main tidal barrier and gate across Smith Creek effective, by preventing inundation through various low lying areas around the overland perimeter of the Hague area, additional fixed closure walls and/or berms would be required to the west and east of the Brambleton Ave. bridge as shown on Sheet S-101.

The preliminary design of these walls is similar to the design of the existing Norfolk Downtown Floodwall, and it consists of a concrete wall supported by AZ 12-770 steel sheet piles of varying embedment depths. The concrete cap and wall face would be constructed with a decorative fascia to preserve aesthetics in this historic district.

The alignments of the wall segments described below are indicative of the extent of wall required and issues that would be faced with wall implementation. However, there are potentially many alterations that may be made to the wall alignments – and barrier type in some locations – that may be preferable to the City, local stakeholders, and federal or Commonwealth partners. It is expected that the barrier alignments and details will change during subsequent stages of design development as feedback from the various stakeholders is taken into account and as the project is value-engineered.

12.5.2 West Option 1

The Option 1 tidal barrier west of the Brambleton Ave. bridge (Sheet S-102) is a steel sheet pile wall extending from the northwest bridge abutment, along Brambleton Ave. and along parcel boundaries, to an existing Red Cross building that represents a tie-in with higher ground.

At its crossing of Second St., the wall would conflict with several utility lines, and a decision would need to be made as to whether Second St. could be permanently closed off – and traffic re-routed – or whether an operable gate would be needed. It would also be possible to terminate the wall immediately east of Second St. and then raise the elevation of the existing street and adjacent grade. The opinion of probable cost (Section 13.2) presently assumes a fixed wall with no gate crossing Second St., with associated utility conflict resolution work, thus permanently closing off Second St. at Woodis Ave.

Additional low spots in existing grade exist to the west of the Option 1 wall extent, near the intersections of Colley Ave. / Southampton Ave. and Woodis Ave. / Third St. These low spots can be addressed using elevated crosswalks (alternately thought of as wide speed humps) as shown on Sheet G-101.

12.5.3 West Option 2

The Option 2 tidal barrier west of the bridge – which would completely replace the Option 1 wall if selected – is shown on Sheet S-106 and S-107. This option consists of a steel pile wall extending from the northwest bridge abutment across to the south side of Front St., then along Front St. to the Norfolk District USACE property, then turning north and terminating a short distance from Front St. at existing higher ground. The Option 2 west overland flood barrier would be designed to a crest elevation of +10.7 ft NAVD88 to accommodate a higher allowance for potential future sea level rise, and the extended alignment of the Option 2 wall is a direct result of the need to connect with existing ground at or near the +10.7 ft NAVD88 elevation.

A greater number of utility relocation and modifications would likely be necessary for the Option 2 west wall alignment, compared with Option 1. An operable gate would be needed across the eastern end of Front St. Decisions would need to be made regarding whether to permanently close off Front St. at its western end – with traffic re-routing – or if an operable gate would be installed instead. Existing street grade elevations appear to be too low in that area to practically raise grades to the design elevation of +10.7 ft NAVD88.

Either of the west wall Options would provide protection from flooding, at the stated design level, for Brambleton Ave. west of the bridge.

12.5.4 East

Following the conceptual design evaluations described in Section 9.0, it was decided that the preferred alternative would include the extended wall alignment east of the Brambleton Ave. bridge. This alignment would construct a wall along the waterfront of and weaving through the interior of the Freemason District, finally connecting to the existing Norfolk Downtown Floodwall.

As shown in Sheet S-103 and S-104, the present design of the east overland flood wall alignment would cross several streets and parking area entrances, and it would have numerous conflicts with existing underground utilities. Opportunities may exist to connect the wall to two sides of existing flood-proofed buildings (such as may be possible at College Cross / Harbour Square), negating the need to close off or install gates across existing traffic. It may also be considered more feasible to permanently close off a few streets and re-route traffic, instead of installing operable gates at several locations. These decisions will need to be made in consultation between the City and local stakeholders.

The proposed east overland flood wall alignment would cross at least two existing storm drains that outfall to the river near Harbour Square and Botetourt St. at College Cross. In order to prevent flood penetration through these existing outfalls, which may surface behind the proposed flood wall, gates would be needed on the existing outfalls. In addition, a new, relatively small pump station may need to be installed nearby to convey storm water runoff from these two drains through the wall and into the river during the design storm event. However, the need for and size of this potential pump system has not yet been confirmed, and hydrologic/hydraulic model simulations are required to accurately evaluate the system requirements.

An option with shorter wall alignment may also be considered, as discussed in Section 8.4, which would extend along Brambleton Ave. east of the bridge and tie into a point several blocks north of the existing Downtown Floodwall terminus. This option would have lower capital construction and O&M costs due to (1) shorter alignment length, (2) fewer utility conflicts to resolve, and (3) likely fewer operable vehicle and/or pedestrian gates required. However, this option would provide no additional protection beyond existing conditions to the Freemason District and immediately adjacent areas. The flood damage reduction benefits and B/C ratio calculations presented in Section 9.0 and Section 11.0 included protection of these districts. For this reason, the lower-cost option has not been explored further in the preliminary design effort, but it may be revisited in future phases of project planning.

12.6 CORROSION MITIGATION

This section intentionally left blank.

12.7 REAL ESTATE REQUIREMENTS

The fixed tidal barrier across Smith Creek, plus the associated tide gate, pumps, and generator / equipment building, are located primarily on City-owned land parcels.

The proposed west (Option 1) and east overland flood barriers cover a total of approximately 3,470 linear feet. It is assumed at present that the City would need to obtain easements of width approximately 15 ft in order to permanently install the walls on private property. The opinion of probable cost presented in Section 13.2 of this report assumes a value of \$8.00 per square foot for easement acquisition, based on an analysis of average property values along the east and west (Option 1) wall alignments.

12.8 FACILITY / UTILITY RELOCATION REQUIREMENTS

Appendix E, Sheet V-102 shows a large-scale overview of **known** existing utility types and alignments, and the individual plan sheets for each of the project elements show the same information at a closer scale and in relation to flood barrier alignments.

Utility conflicts have been estimated based on available line work and descriptions provided to the project team regarding the following utilities:

- City of Norfolk storm water, sanitary sewer, and water supply – information in the form of GIS shapefiles and/or geodatabase
- Hampton Roads Sewer District – information in the form of GIS shapefiles and/or geodatabase
- Virginia Natural Gas – information supplied by VNG in the form of map images, subsequently georeferenced and digitized by the project team
- Dominion Electric – information supplied by Dominion in the form of map images, subsequently georeferenced and digitized by the project team

No independent field investigations have been performed to validate or add to the information. Telecommunications and/or other potential utilities not listed explicitly above have not been considered at this stage, due to lack of readily available georeferenced information.

Table 12-2 indicates the presently estimated type and number of utility conflicts for the preliminary design alignment of the project elements, based on the presently available utility location and type information provided to the project team.

Table 12-2: Known Utility Conflicts with Preferred Preliminary Alternative

Project Element	Location of Utility Conflict	Description of Utility Conflict
Tidal barrier across Smith Creek, and associated systems	none	none
West overland barrier (Option 1)	2nd St. & Woodis Ave.	12" Ductile Iron Water Main
	2nd St. & Woodis Ave.	4" Ductile Iron Water Main
	2nd St. & Woodis Ave.	12" Clay or Ductile Iron Sewer Line
	2nd St. & Woodis Ave.	Gas Line
	2nd St. & Woodis Ave.	Underground Electric Ductbank
West overland barrier (Option 2)	Front St. & Colley Ave.	4" Ductile Iron Water Main
	Front St. & 3rd St.	6" Ductile Iron Water Main
	Front St. & 3rd St.	6" Ductile Iron Water Main
	Front St. & Colley Ave.	10" Ductile Iron Water Main
	Front St. & Colley Ave.	12" Ductile Iron Water Main
	Front St. & Rader St.	12" Ductile Iron Water Main
	Front St. & Rader St.	8" Clay or Ductile Iron Sewer Line
	Front St. & Colley Ave.	8" Clay or Ductile Iron Sewer Line
	Front St. & Colley Ave.	12" Clay Stormwater Line
	Front St. & Rader St.	18" Clay or Concrete Stormwater Line
	Front St. & 2nd St.	18" Clay or Concrete Stormwater Line
	Front St. & Rader St.	Gas Line
	Front St. & Colley Ave.	Gas Line
	Front St. & 2nd St.	Underground Electric Ductbank
	Front St. & Colley Ave.	Underground Electric Ductbank
East overland barrier	Harbour St. & W. Tazewell St.	6" PVC Water Main
	W. Tazewell St. & Yarmouth St.	8" PVC Water Main
	College Pl. & Harbor Sq.	8" PVC Water Main
	Duke St. & W. Tazewell St.	8" PVC Water Main
	W. Brambleton Ave.	12" Ductile Iron Water Main
	W. York St.	6" PVC Sewer Line
	Duke St. & W. Tazewell St.	8" PVC Sewer Line
	Harbour St. & W. Tazewell St.	8" PVC Sewer Line
	College Pl. & Harbor Sq.	8" PVC Sewer Line
	W. Freemason St. & Botetourt St.	8" PVC Sewer Line
	W. Bute St. & Botetourt St.	8" PVC Sewer Line

Project Element	Location of Utility Conflict	Description of Utility Conflict
	College Pl. & Harbor Sq.	18" Clay or Concrete Stormwater Line
	W. Tazewell St. & Yarmouth St.	24" Concrete Stormwater Line
	College Cross & College Pl.	15"X30" Arch Concrete Stormwater Line
	College Pl. & Harbor Sq.	48" Concrete Stormwater Line
	W. Freemason St. & Botetourt St.	Gas Line
	W. Bute St. & Botetourt St.	Gas Line
	W. York St.	Gas Line
	W. York St.	Gas Line
	W. Tazewell St. & Yarmouth St.	Underground Electric Ductbank
	W. York St.	Underground Electric Ductbank
	W. Brambleton Ave.	Underground Electric Ductbank
	W. Brambleton Ave.	Underground Electric Ductbank

Utility conflicts estimated based on available linework and utility descriptions provided to the project team. No independent field investigations have been performed to validate or add to the information. Telecommunications utilities have not been considered due to lack of readily available georeferenced information.

Rigorous verification of existing and planned utility alignments, depths, and line characteristics should be completed prior to detailed engineering design of the proposed flood walls and other significantly ground-penetrating project elements.

13.0 PRELIMINARY DESIGN LEVEL OPINION OF PROBABLE COSTS AND BENEFIT-COST ANALYSIS FOR PREFERRED DESIGN ALTERNATIVE

13.1 PURPOSE OF THIS SECTION

The purpose of this section of the report is to present an updated opinion of probable cost for the Hague area coastal flood mitigation project based on the preliminary (10% level) design described in the previous section and shown in Appendix E. The opinion of probable cost presented in this section supersedes the conceptual level estimates presented in Section 10.0 above, for the preferred design based on Alternative 1a.

The present 10% preliminary designs and associated cost information are developed solely for the joint occurrence of a 100-year return period coastal surge and a 100-year return period, 24-hour design rainfall event, as documented previously in this report.

13.2 CAPITAL COSTS

Unit costs were based on a combination of available data from local contractors, equipment vendors, records of prior construction project costs, RS Means construction cost publications, VDOT publications, and other sources as needed.

The opinion of probable cost is developed in 2012 dollars and does not include an allowance for inflation between the present and a future construction date. Items considered in the opinion of probable cost include:

- Materials and construction / installation costs for civil, structural, electrical, and mechanical components of the project;
- Required easements, acquisitions, and environmental mitigation;
- Contractor mobilization / demobilization, overhead & profit, and erosion / sediment / traffic control for construction;
- Federal feasibility and environmental studies, based on experience of projects of similar size and scope (not quoted from agencies for this specific project);
- Engineering/Construction Observation; and
- Contingency to allow for unknown conditions discovered or arising between the present and actual project construction.

The opinion of probable capital cost for the preferred alternative, as presently formulated, is approximately \$58.6 million. A detailed breakdown of line items, quantities, and unit costs is provided in Table 13-1.



Table 13-1: Opinion of Probable Capital Costs for Preferred Preliminary Design Alternative

		Opinion of Probable Cost		DATE PREPARED 16-Mar-12	
				DRAFT Submittal	
ACTIVITY AND LOCATION		CONSTRUCTION CONTRACT NO.		IDENTIFICATION NUMBER	
City of Norfolk					
Norfolk, Virginia		ESTIMATED BY		CATEGORY CODE NUMBER	
PROJECT TITLE		Moffatt & Nichol			
Hague Option 1 - Tidal Barrier with Steel Gate, 3-60" Dia. Pumps, and Closure Walls		STATUS OF DESIGN		JOB ORDER NUMBER	
For 100 Year Event @ ELEV +7.7 (Design Water Surface ELEV +9.7)		Preliminary (10%)		7607-01	
ITEM DESCRIPTION		QUANTITY		ENGINEERING ESTIMATE	
		NUMBER	UNIT	UNIT COST	TOTAL
SMITH CREEK Flood Wall with Steel Gate					
Flood Wall					
AZ26 (50 VLF)	82	EA	\$4,300.00	\$352,600	
AZ26 (80 VLF)	26	EA	\$6,880.00	\$178,880	
AZ26 (100 VLF)	94	EA	\$8,600.00	\$808,400	
PP60x0.75 (80 VLF, Coated w/ Connectors)	16	EA	\$46,549.00	\$744,784	
PP60x0.75 (100 VLF, Coated w/ Connectors)	48	EA	\$55,221.00	\$2,650,608	
Connectors (assume 20% of total steel)	1	LS	\$947,054.40	\$947,054	
Concrete cap / fascia	2,786	CY	\$1,000.00	\$2,786,375	
Handrail	1,560	LF	\$178.00	\$277,680	
Steel Gate and Equipment					
24" SQ Conc Batter Piles (20 piles)	20	EA	\$10,587.33	\$211,747	
W33x152	38	TON	\$10,125.00	\$382,442	
3/4" Thick Outside Plate	19	TON	\$10,125.00	\$193,737	
1/2" Thick Plate @3'-0"	11	TON	\$10,125.00	\$116,434	
3/4" Thick Bottom Plate	4	TON	\$10,125.00	\$36,175	
Roller Guide	136	LF	\$298.00	\$40,528	
Flanged Steel Wheel Assembly	8	EA	\$3,120.00	\$24,960	
Gate Rail	136	LF	\$298.00	\$53,640	
Capstan & Cabling	1	LS	\$193,500.00	\$193,500	
UHMW Rollers	16	EA	\$2,587.00	\$41,392	
Pocket Seal	35	LF	\$4,650.00	\$162,750	
SMITH CREEK Flood Wall Pump Station					
Pumps and Mounting					
60" pumps	3	EA	\$1,380,000.00	\$4,140,000	
Support Structure - piles,header,rods, etc.	3	EA	\$70,000.00	\$210,000	
Misc 60" Pipe Sections	3	EA	\$8,000.00	\$24,000	
60" Flapgates	3	EA	\$19,200.00	\$57,600	
Generator Building					
Concrete Block Building for Generator Equipment	3,878	SF	\$200.00	\$775,500	
Aesthetic Features of Pump Station	1	LS	\$100,000.00	\$100,000	
Clearing, Grubbing & Grading	667	SY	\$10.00	\$6,667	



Electrical				
Dominion Power Installation Costs	1	LS	\$200,000.00	\$200,000
Installation Equipment	1	LS	\$20,228.20	\$20,228
Installation Site Work	1	LS	\$9,943.24	\$9,943
Switchboard	1	LS	\$74,231.00	\$74,231
Conduits & Fittings	1	LS	\$191,405.00	\$191,405
VFD Drive	3	EA	\$150,000.00	\$450,000
2500 KW Standby Generator	2	EA	\$1,245,875.00	\$2,491,750
Paralleling Switchgear	1	LS	\$429,800.00	\$429,800
# 500 kcmil XHHW	7,500	LF	\$14.10	\$105,750
Other Electrical Equipment	1	LS	\$181,582.84	\$181,583
Insurance & Taxes for Electrical	1	LS	\$159,404.00	\$159,404
Sales Tax for Electrical	1	LS	\$173,859.00	\$173,859
WEST Overland Flood Wall				
Wall Section Type A				
Demo Concrete / Asphalt	86	SY	\$15.00	\$1,286
AZ 12-700 (12 VLF)	113	EA	\$1,032.00	\$116,616
Concrete Wall	87	CY	\$1,000.00	\$87,010
Wall Section Type B				
Demo Concrete / Asphalt	38	SY	\$15.00	\$573
AZ 12-700 (18 VLF)	51	EA	\$1,548.00	\$78,948
Concrete Wall	60	CY	\$1,000.00	\$59,563
Concrete Footing	19	CY	\$1,000.00	\$19,403
Wall Section Type C				
Demo Concrete / Asphalt	129	SY	\$15.00	\$1,936
AZ 12-700 (27 VLF)	170	EA	\$2,322.00	\$394,740
Concrete Wall	233	CY	\$1,000.00	\$232,960
Concrete Footing	66	CY	\$1,000.00	\$65,542
WEST Overland Flood Wall Utility Conflicts				
12" Ductile Iron Water Main Relocation	1	EA	\$32,400.00	\$32,400
4" Ductile Iron Water Main Relocation	1	EA	\$21,600.00	\$21,600
12" Clay or Ductile Iron Sewer Line Relocation	1	EA	\$21,600.00	\$21,600
Gas Line Relocation	1	EA	\$18,000.00	\$18,000
Underground Electric Ductbank Relocation	1	EA	\$18,000.00	\$18,000
Allowance for unknown utility conflicts	1	LS	\$55,800.00	\$55,800
WEST Overland Area - Street Grade Raising				
Elevated Crosswalk on Colley Ave.	1	EA	\$10,000.00	\$10,000
Elevated Crosswalk on Southampton Ave.	1	EA	\$10,000.00	\$10,000
Elevated Crosswalk on Woodis Ave.	1	EA	\$10,000.00	\$10,000
Elevated Crosswalk on Third St.	1	EA	\$10,000.00	\$10,000



EAST Overland Flood Wall - connect to Downtown Flood Wall				
Wall Section Type A				
Demo Concrete / Asphalt	181	SY	\$15.00	\$2,708
AZ 12-700 (12 VLF)	239	EA	\$1,032.00	\$246,648
Concrete Wall	180	CY	\$1,000.00	\$180,272
Wall Section Type B				
Demo Concrete / Asphalt	445	SY	\$15.00	\$6,681
AZ 12-700 (18 VLF)	587	EA	\$1,548.00	\$908,676
Concrete Wall	656	CY	\$1,000.00	\$656,291
Concrete Footing	226	CY	\$1,000.00	\$226,111
Wall Section Type C				
Demo Concrete / Asphalt	156	SY	\$15.00	\$2,347
AZ 12-700 (27 VLF)	207	EA	\$2,322.00	\$480,654
Concrete Wall	282	CY	\$1,000.00	\$282,376
Concrete Footing	79	CY	\$1,000.00	\$79,444
Gates at Street Crossings and Parking Access				
Street Crossings	4	EA	\$285,656.07	\$1,142,624
Parking Area Crossings	1	EA	\$173,082.05	\$173,082
Historical area considerations		%	20%	\$877,583
EAST Overland Flood Wall Utility Conflicts				
6" PVC Water Main Relocation	1	EA	\$19,200.00	\$19,200
8" PVC Water Main Relocation	3	EA	\$20,400.00	\$61,200
12" Ductile Iron Water Main Relocation	1	EA	\$32,400.00	\$32,400
6" PVC Sewer Line Relocation	1	EA	\$16,800.00	\$16,800
8" PVC Sewer Line Relocation	5	EA	\$16,800.00	\$84,000
18" Clay or Concrete Stormwater Line Relocation	1	EA	\$18,000.00	\$18,000
24" Concrete Stormwater Line Relocation	1	EA	\$27,600.00	\$27,600
15"X30" Arch Concrete Stormwater Line Relocation	1	EA	\$28,800.00	\$28,800
48" Concrete Stormwater Line Relocation	1	EA	\$43,200.00	\$43,200
Gas Line Relocation	4	EA	\$18,000.00	\$72,000
Underground Electric Ductbank Relocation	3	EA	\$18,000.00	\$54,000
Allowance for unknown utility conflicts	1	LS	\$228,600.00	\$228,600
EAST Overland Flood Wall Pump Station				
Concrete Block Building for Generator Equipment	1,600	SF	\$200.00	\$320,000
Aesthetic Features of Pump Station	1	LS	\$50,000.00	\$50,000
Clearing, Grubbing & Grading	431	SY	\$10.00	\$4,308
48" Pump	1	EA	\$455,000.00	\$455,000
Piping and Flap Gates	1	LS	\$50,000.00	\$50,000



Easements and Mitigation				
City of Norfolk Easement	51,615	SF	\$8.00	\$412,920
Temporary Easement (8% of CoN Easement)	1	LS	\$33,033.60	\$33,034
Wetland Mitigation	0.9	AC	\$500,000.00	\$450,000
Monticello Culvert - Pump Station				
18" HDPE Line	2,400	LF	\$90.00	\$216,000
Installation of HDPE Line	2,400	LF	\$35.00	\$84,000
Dewatering	1	LS	\$40,000.00	\$40,000
Stormwater Vaults	2	EA	\$35,000.00	\$70,000
Stormwater Pumps	2	EA	\$150,000.00	\$300,000
Flapgates	1	EA	\$2,500.00	\$2,500
Culvert Realignment	1	EA	\$100,000.00	\$100,000
SUBTOTAL - CONSTRUCTION ELEMENTS				\$29,810,442
Contractor Markups and Contingency				
Contractor - Overhead & Profit		%	15%	\$4,471,566
Contractor - Mobilization/Demobilization		%	12%	\$3,577,253
Contractor - Difficult Waterside Conditions	est. lump sum	LS	\$1,000,000.00	\$1,000,000
Contractor - Erosion/Sediment Control		%	5%	\$1,490,522
Contractor - Traffic Control		%	2%	\$596,209
Construction Subtotal Contingency		%	30%	\$8,943,133
SUBTOTAL - CONSTRUCTION, WITH MARKUPS				\$49,889,126
Planning, Design, Permitting, and Construction Observation				
USACE Recon study		LS	\$200,000.00	\$200,000
USACE Feasibility study		LS	\$2,000,000.00	\$2,000,000
USACE EIS and NEPA coordination		LS	\$500,000.00	\$500,000
Engineering, P&S, Const. Obs.		%	12%	\$5,986,695
SUBTOTAL - PLANNING AND ENGINEERING				\$8,686,695
TOTAL				\$58,575,821
			SAY	\$58,600,000

13.3 OPERATIONAL & MAINTENANCE COSTS WITH RESPECT TO DESIGN LIFE (C-15)

The standard serviceable design life for the preferred preliminary design alternative is estimated to be 50 years. It is assumed that all components of the system will be properly maintained such that the system of barriers, gate, and pumps will be able to maintain a functional level of serviceability over this design life. Operational and Maintenance (O&M) costs for the proposed system generally include (but may not be limited to):

- Inspection costs,
- Minor repairs,
- Major repairs,
- Replacement costs,
- Equipment upgrades,
- Machine maintenance,
- Pumps and power costs, and
- Labor costs during "closure" events.

Assumptions for the operational and maintenance costs included:

- Routine inspections on bulkheads, gates, floodwalls (typically on a 5-year cycle)
- Minor repairs (Years 15, 35, and 45)
- Major repairs (Years 25 and 40)
- Replacement of pumps (Year 30)
- Operational costs for storm events per year (8 events per year)

Operational and maintenance costs for the preliminary design alternative are provided in Table 13-2. The estimated O&M cost for Alternative 1a is unchanged from the conceptual level analysis, and it consists of an expected annual cost of \$231,000, or a total of approximately \$3.2 million in present value over a 50 year design life. Based on FEMA and OMB direction, a 7% interest rate was utilized for the present value analysis.

Table 13-2: Operational & Maintenance Costs for Preferred Preliminary Design Alternative

Alternatives	Annual Operational Costs (\$)	50-yr Operational Costs (\$) Present Worth
Alt 1a: Tidal Barrier with Steel Gate, 2 - 60" Dia. Pumps, Closure Walls and Berm	\$231K	\$3.2M

These maintenance and operational costs will be used in conjunction with the Opinion of Probable Cost and damage assessments to determine the Benefit - Cost for all alternatives.

13.4 BENEFIT-COST RATIO ANALYSIS

The benefit cost ratios were updated based on the opinion of probably cost developed for the preliminary (10%) design of the preferred design alternative. **Error! Reference source not found.** summarizes the B/C ratios for the various alternatives. The B/C ratio of the alternatives analyzed indicates that Alternative 1a - Tidal Barrier with Steel Gate, 2 - 60" Pumps, and Closure Walls and Berms is the most cost effective alternative with a Benefit Ratio of 1.34 for a 100-year storm event.

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14.0 ENVIRONMENTAL OBJECTIVES AND REQUIREMENTS (ITEM C-13)

This section of the report is intended to provide an overview of the additional studies required, to present the status of coordination with stakeholders, and to describe the existing environmental resources that may be impacted by the project. It is not intended to document detailed consideration of all of the environmental requirements associated with a full project feasibility study.

14.1 OVERVIEW OF NEPA AND REGULATORY REQUIREMENTS

The National Environmental Policy Act (NEPA) of 1969 is a procedural law that establishes the requirement that all federal agencies' actions including funding or permitting decisions be made with full consideration of the impact to the natural and human environment through a systematic interdisciplinary approach. In the case of the Hague Floodgate Project, the project is being proposed and designed by the City of Norfolk. The federal action that will require compliance with NEPA for this project is the USACE Federal Permitting Decision under Section 404 of the Clean Water Act.

There are three levels of analysis that a federal agency may undertake to comply with NEPA. These three levels include: preparation of a Categorical Exclusion (CE), preparation of an Environmental Assessment (EA) and Finding of No Significant Impact (FONSI); or preparation of an Environmental Impact Statement (EIS) and Record of Decision (ROD). The level of documentation and review depends on the nature of the project and the likelihood the project could have significant impacts. The environmental review process must include an evaluation of reasonable alternatives, an assessment of potential environmental impacts of the proposed action and alternative actions, and disclosure of potential impacts to interested parties and the general public.

In addition to NEPA, major Federal civil works projects must be in compliance with other applicable environmental statutes including the Endangered Species Act, the Clean Air Act, the Clean Water Act, the Fish and Wildlife Coordination Act, Resource Conservation and Recovery Act, the Historic Preservation Act, and many more. Review of these protected resources is covered through the NEPA assessment and documentation of potential impacts to the natural and human environment.

The NEPA process requires the participation of multiple agencies at the federal, state, and local levels. At the federal level, the USACE will be the lead agency to coordinate federal activity for this project. (As mentioned above, the NEPA trigger for this project is the USACE 404 permitting decision). The USACE will either direct the City of Norfolk to complete NEPA documentation in support of the Federal permitting decision process or will complete the NEPA documentation itself (either internally or through the use of an independent, third-party contractor). Other Federal, State, and Local regulatory agencies will be consulted during the NEPA process dependent on applicable laws as discussed below. Those agencies include:

- U.S. Army Corps of Engineers – Norfolk District (USACE)
- U.S. Fish and Wildlife Service (FWS)
- U.S. Environmental Protection Agency, Region III (EPA)
- U.S. Coast Guard, Fifth District (OAN)

- U.S. National Oceanic and Atmospheric Administration, Habitat Conservation Division (NOAA)
- U.S. Department of the Interior, Office of Environmental Policy and Compliance (DOI)
- Virginia Marine Resources Commission (VMRC)
- Virginia Department of Conservation and Recreation (DCR)
- Virginia Department of Environmental Quality (VDEQ)
- Virginia Department of Game and Inland Fisheries (DGIF)
- Virginia Department of Mines, Minerals, and Energy (DMME)
- Virginia Department of Historic Resources (DHR)
- Hampton Roads Planning District Commission (HRPDC)
- Hampton Roads Transit (HRT)

14.2 COORDINATION WITH FEDERAL, STATE, AND LOCAL STAKEHOLDERS

Early agency coordination is an important component to identifying project stakeholders and regulatory permitting agencies. This early engagement helps identify and shape project design elements, roles of key stakeholders, and potential study and design needs.

Coordination activities between the City of Norfolk and the USACE Norfolk District have been initiated for this project through informal and formal meetings. Initial engagements between the City and the Norfolk District representatives occurred during November 2011 to discuss the scope of the City's project and avenues of Federal participation. During a meeting between Norfolk District USACE and the City on November 28, 2011 several possible avenues for Federal participation and their pros and cons were discussed.

The general avenues for Federal participation include:

- Partnership between Federal Government and City of Norfolk: Eligible projects with Federal-City partnerships may receive partial Federal funding through Section 205 or Section 14 of the 1948 Flood Control Act. Federal contributions are capped depending on the program that the project enters.
- Interagency Support: The USACE can coordinate interagency (e.g. USACE, FEMA, NRCS, etc.) reviews during the project permitting process. Recent regional project example includes the Wallops Island Design and Construction project funded by NASA.
- Permit Support and Review: USACE conducts review of permits and provides support to the City during the permit application and review process.

During December of 2011, the City submitted a Fact Sheet to the USACE for the Hague project that described the preferred project alternative being considered and cost estimate. On December 8, 2011 USACE representatives from the Atlantic Division headquarters and Norfolk District office met and conducted a site visit of the Hague area. During the meeting, the City presented the project being considered and interactively discussed with the USACE a range of options for progressing the study. The USACE indicated that they would first have to evaluate

whether there is Federal interest in a project before committing Federal resources toward the planning, engineering, and construction of a project. The evaluation of the potential Federal interest is generally conducted through a Reconnaissance study under new authority or existing authority.

In April 2012, the USACE indicated that they would evaluate the Federal interest in this project under the Continuing Authority Program (CAP). The study is planned to be conducted during the early summer of 2012 and anticipated to take approximately six months to complete. Based on the study, if the Federal government deems there is adequate Federal interest in the project, then the project may be eligible to enter the Section 205 program and receive Federal funding through a partnership between the sponsor (City of Norfolk) and Federal government.

In addition to coordination at the Federal level, the City has also initiated discussions with various stakeholders at the State level, including: congressional representatives; Virginia Dept. of Environmental Quality (DEQ); Virginia Marine Resources Commission (VMRC); and others involved with emergency management services.

At the City level, various levels of the City's Public Works department have been engaged since the early planning stages. Representatives from various City departments and the City's elected officials have been briefed regularly on the study since 2010.

The City has also engaged the public during the process. The City has met with local leaders, provided public presentations about the project and solicited input and feedback regarding potential issues or opportunities related to the project. Public meetings were held on November 29, 2011 and February 29, 2012. City staff and their consultants presented an overview of the City-wide study and the preferred alternative discussed in this report. The meetings were attended by civic league presidents, citizens, Hampton Roads Planning District Commission, and representatives from Old Dominion University.

14.3 NOTABLE AREA FEATURES AND LIKELIHOOD OF POTENTIAL IMPACTS

14.3.1 Smith Creek

Smith Creek (the Hague cove) is the primary open water body landward of the proposed tidal barrier. The cove exchanges water directly with the tidal Elizabeth River through the Brambleton Avenue bridge opening. Smith Creek is populated by numerous aquatic species. Specific species inhabiting Smith Creek are not presently documented, but it is assumed that species may include benthic organisms, insects, fish of all life stages, aquatic birds, and water-dependent mammals. Documentation of species actually inhabiting Smith Creek will need to be developed, and potential specific impacts will need to be considered, during the environmental assessment phase of project development.

The primary concern with regard to water quality is that the proposed tidal barrier may act to limit tidal exchanges between Smith Creek and the Elizabeth River. Limited flushing may result in undesirable dissolved oxygen, nutrient, and /or temperature levels in the Hague cove basin.

No data are presently available on the current state of water quality within the cove. As a first step in evaluating the potential water quality impacts of the tidal barrier, a limited analysis

of hydraulic flushing, under typical tidal conditions, was conducted using a two-dimensional depth-integrated hydrodynamic model.

The hydrodynamic model was developed in the DELFT3D modeling software, and a typical tidal water level time series (based on Sewells Point tide records) was input as the boundary condition outside the cover. Model simulations used a conservative (non-decaying) “tracer” constituent deployed uniformly throughout the Hague cover – behind the tidal barrier – to evaluate flushing times for both existing conditions and proposed Alternative 1a conditions. The simulations indicated that the proposed tidal barrier may increase by approximately 1 day the time required to exchange 90% of the water in the western arm of the basin (which appears to have the slowest existing flushing behavior).

The model simulations evaluate the movement of a conservative tracer as a means of estimating the time to exchange water in the basin with water in the Elizabeth River. The processes controlling water temperature and dissolved oxygen are more complex, and a full analysis of the potential water quality impacts of the proposed project would require significant additional model simulations.

Neither the model’s hydrodynamic parameters nor the advection / dispersion parameters for existing conditions could be calibrated, due to the lack of available measured data or prior model studies of the local water body. If additional analysis of water quality in the cove are required for environmental compliance, it will be necessary to collect synchronous tidal water levels, current velocity data, and water quality measurements / samples, in order to calibrate the numerical model.

14.3.2 Shallow Bottom Habitat

A shallow water area (shoal) and non-protected shoreline exists on the western side of the Brambleton Avenue bridge at Smith Creek. This area would be on the Elizabeth River side (i.e. outside) of the proposed tidal barrier, but it would be close enough to the barrier that its morphology may be affected by the presence of the vertical wall. Species inhabiting the shoal and shoreline area are not presently documented.

Vertical walls are reflective of incident wave energy, and they may also alter the flow patterns and velocities of tidal currents. The proposed tidal barrier would block any sediment movement between the Hague cove and the shoal / shoreline area just outside of the wall. However, given the relatively low existing current speeds, it is unlikely that significant sediment exchange is occurring under present existing conditions.

The main pumps to be located near the eastern bank of the cove, discharging through the tidal barrier directly into the water columns, may impact submerged habitat on either or both sides of the wall. On the intake (cove) side of the wall, running the pumps at the design flow rate would have the potential to scour the existing muddy cove bed. However, the pump intakes are proposed to be supported by strainer stands bearing on concrete slabs, with the slabs supported by concrete piles. Having the pumps pull water laterally through the strainer stand, rather than pulling water vertically from directly under the intake bells, would mitigate the potential for scour hole formation.

On the discharge (Elizabeth River) side of the wall, the pump discharge may generate vertical and horizontal eddies in the immediate vicinity of the wall. These may impact on the

adjacent shoreline and/or on the muddy river bed. However, except during maintenance inspection when the pumps may be run at a lower flow rate, the pumps will run at the design flow rates only during significant coastal flooding events. The natural events themselves would also have flow- and wave-induced impacts on the shorelines and river bed, and it is not expected that the pump discharges will have significant additional impacts in addition to the natural impacts of the storm event.

14.3.3 Freemason Historic District

An overland flood wall connecting the proposed Smith Creek tidal barrier with the existing Downtown Flood Wall would go through the historic Freemason District and would run close to existing historic buildings. It may be necessary for the wall to abut certain buildings, with appropriate allowances made to avoid making the walls structurally dependent upon the buildings. It may also be most technically and economically advantageous to permanently close off certain streets within the district, to allow a continuous flood barrier without the need to provide an excessive number of pedestrian and/or vehicle traffic gates.

It may also be necessary to locate a small pump station in the Freemason District, for example at the City-owned park at College Cross and Harbour Square. Above-ground components of the pump station would need to be designed to blend with the architectural characteristics of the adjacent neighborhood. The pump station may generate measurable noise during operation; however, it is not expected that the pumps would operate except during coastal flood events or during pump maintenance inspections.

Even with an aesthetically and architecturally appropriate design, the construction of a wall – which in some places will be more than 5 ft high above existing ground – may impact the viewshed of the district. The most obvious example is that the wall may block views of the water from various perspectives; other visual impacts may be noted during consultation with stakeholders.

14.3.4 Local Community

The floodwall would extend into a residential district. The socio-economic impacts of altering or impeding access on local roads and to recreational resources and businesses will have to be considered during NEPA compliance. The socio-economic make-up of this community will also need to be documented, in order to show that the project will not have socio-economically disproportionate adverse impacts.

Along some segments, the presence of the wall may create areas that are not visible from a distance, or it may block light from existing sources, thus creating pedestrian safety issues. For example, dark areas or areas that cannot be easily observed may encourage criminal activity or hinder police response to criminal activity. This could be mitigated with additional lighting, but there are also environmental impacts of over-lighting a waterfront historic area.

It has been noted that the area west of Brambleton Avenue bridge, south of Brambleton Avenue and along Front Street may be considered by the City as an attractive area for redevelopment in the medium-term. The design of a wall, or combination wall / berm barrier, needs to consider the potential for redevelopment of this area. For example, if a mixed-use development is proposed, a wall without enough pedestrian or vehicle passageways may

adversely impact on the primary concept of a mixed-use area. This could be mitigated by ensuring enough attractive, useful passageways through or over the barrier, but these would add additional cost to the project.

14.3.5 Wetlands

Several shallow water areas may exist along the alignment of the proposed east overland flood wall where it runs along the Freemason District waterfront. The nature of these areas, and whether they would be considered jurisdictional wetland areas, is not documented.

14.3.6 Recreation

The Hague cove is also used by small recreational water craft. It is not expected that the proposed project will substantially impact recreational boating activity or access in the project vicinity except during major storm and flooding events.

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15.0 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN (ITEM C-10)

This section intentionally left blank at present, pending further discussions with USACE to gain additional understanding of these requirements from a Corps perspective.

15.1 MAIN FLOODWALL AND GATE

15.2 PUMPING FACILITIES

15.3 ADDITIONAL PERIMETER WALLS

15.4 ACCESS AND STAGING AREAS

Draft

16.0 IMPLICATION OF POTENTIAL FUTURE SEA LEVEL RISE

The analyses results as presented hereto are based primarily on the present sea level. As discussed previously, sea level rise (absolute or relative [the latter which includes the absolute sea level rise plus ground subsidence]) has been widely documented. The magnitude of the historical relative sea level rise in the Hampton Roads area (specifically as measured at Sewells Point) is among the highest of such data in the mid-Atlantic.

To evaluate how potential sea level rise may affect the capital costs and damages for the various design scenarios, the following process should be used. Rather than repeating the various analyses for different sea level rise scenarios, it is logical to shift the return period as a function of different magnitudes of sea level rise. This can be accomplished by raising the assumed tailwater elevation associated with different magnitudes of relative sea level rise.

For example if the objective is to evaluate how a 1-foot rise in relative sea level will affect the evaluation of Alternate __, the following process can be conducted.

1. Plot the cost and damage curves versus return period for the design to be evaluated. For example, Figure __a shows such a plot for the Alternate __.
2. Convert the costs versus return period to costs versus tailwater elevation, using the tailwater versus return period plot shown on Figure __b to create the costs and damage curve shown on Figure __c.
3. Convert today's tailwater versus return period for a 1-foot rise in sea level as shown on Figure __d,
4. Add the "after 1-foot" of sea level rise tailwater versus design period to plot compare the relationship between those two variable for the current conditions, as shown on Figure __e, and
5. Shift the cost and damage curves versus return period so as to account for the change in tailwater that will be created by a 1-foot rise in sea level. Figure __f shows the resulting change in cost and damage versus return period after a 1-foot rise in sea level.

As stated in earlier sections of the report, sea level rise was not implicitly accounted for in the analyses. The height of the structures however does have an allowance of 2 ft to account for some sea level rise, wave overtopping, and still provide 1 ft of freeboard as FEMA requires. Nonetheless, raising the structures should be further investigated during the next design phase and a final design elevation selected. In many ways, it would be prudent to include an allowance for sea level rise since adding elevation will be more difficult after the fact, than the added (delta) cost associated with raising the top of the structure by another foot. The estimated delta cost to raise the crest of the floodwall by an additional 1- is ~5-15% of the initial cost. Where this relationship would breakdown is when the flood levels approach elevations where significant portions of the watershed rim would have to be raised - the costs would then likely underweigh the benefits.

17.0 CONCLUSIONS AND RECOMMENDATIONS

The Hague watershed includes the Ghent residential/commercial community, portions of the Freemason area, and northwestern portions of the downtown Norfolk business district. Much of the area is located in a former tidal estuary historically known as Smith Creek. As the City was developed much of the former tidal estuary has been filled and improved. The confluence of Smith Creek's branches, where it discharges into the Elizabeth River, is known as The Hague. The watershed (catchment area) from which storm water runoff discharges into The Hague is hereinafter referred to as "The Hague Area."

Flooding in The Hague Area is frequent; and varies from nuisance flooding to events causing significant damage. Flooding is caused by the combined effects of "high tides" and heavy precipitation. The effects of these "high tides" (coastal flooding) are expected to worsen over time as mean sea level rises. In addition, the effects of sea level rise will be compounded by regional and local ground subsidence, themselves resulting from events in geologic time, and ongoing settlement of localized, man-made fill.

The primary conclusions and recommendations from the current study include:

- The existing upland storm water piping system is adequate for approximately the 2-yr rainfall event before the inlet and pipe systems become overwhelmed and floodwaters cannot reach Smith Creek in a hydraulically efficient manner.
- The wide spread flooding and density and types of development in The Hague watershed are not conducive to property buyout, elevation of structures or other types of mitigation options. Thus options to mitigate coastal flooding will require capital infrastructure improvements.
- The pumping capacity of two MWI SEA360 (60" discharge diameter) pumps is adequate to address the flow rates which can be delivered by the existing storm water piping system. Additional pumping capacity will not be needed until improvements are made to the upland system.
- The preferred alternative is the construction of a floodwall, tide gate, a pump station (with two 60" pumps operating simultaneously and one 60" pump as a backup) and closure walls with a total capital cost of \$58.6M. The preferred alternative has a B/C ratio of 1.15, and is therefore economically justified.
- The floodwall south of Brambleton Avenue, to the west and east of Smith Creek will provide additional benefits including improved access to the hospital during flood events and increased protection for the Freemason area and for the Light Rail system.
- This option will provide coastal flood mitigation today for a 100-yr surge level and approximately a 2-yr rainfall event. This option will provide adequate protection for coastal flooding and upland drainage improvements can be phased in over time in the future to improve the upland flooding situation with additional pumps thru the floodwall so that the pumping capacity stays in-line with the ability of the upland system to deliver floodwaters to Smith Creek.

- The proposed project will be required to go through the NEPA process. This study has initiated some of the steps necessary to evaluate potential environmental impacts. This study conducted preliminary hydrodynamic analysis to evaluate the impact to tidal flushing of a wall and gate structure near Brambleton Avenue bridge. These screening-level simulations do not indicate that the proposed tidal barrier would increase flushing times in Pretty Lake (at the barrier or at any point further within the lake). The proposed structure will impact subaqueous bottomlands and potentially limited wetland areas along the shoreline area.
- The delta costs for building the floodwall higher for sea level rise concerns would be on the order of 5-15% per foot. A final decision concerning what height should control should be made during the next design phase.

In summary, this study demonstrates that infrastructure improvements consisting of a flood wall with gate can mitigate coastal flooding including much of the worst effects of extreme extra-tidal events from hurricanes and nor'easters. Because The Hague cove is small in comparison with the size of the watershed, its capacity to store storm water runoff is limited. Thus, pumps will be required to pass the excess storm water inflow over the flood barrier. These improvements are technically feasible, and can be expected to have a favorable "benefit to cost" ratio.

Because of the inherent limitations in the old storm water system, it cannot effectively deliver the rainfall runoff from large storms to The Hague. Thus, the coastal flooding infrastructure improvements can not eliminate all flooding due to storms with significant precipitation. To mitigate that component of flooding, will require future, long-term improvements to the existing storm water drainage system. The construction of the coastal flooding infrastructure does, however, significantly lessen the effects due to the inadequate capacity of the storm drain system.

To manage capital expenditures, it is logical to sequence the improvements in The Hague by first constructing the coastal flooding barriers and mitigations so as to eliminate the tidal surge from entering The Hague. That can be followed by storm water drainage system improvements.

At the time of the report, the USACE has approved a study to evaluate whether there is Federal interest in the Pretty Lake project. The study is planned to occur during the summer of 2012 and expected to take six months to complete. If the USACE deems there is Federal interest in the project, then the project may be eligible to pursue Federal funding through a partnership with the Federal government.

18.0 LIMITATIONS

All documents have been prepared for the exclusive use of the City of Norfolk for the preliminary evaluation of flood mitigation options for the project location. The data, findings, and conclusions presented herein were prepared in accordance with generally accepted civil engineering practices of the project region.

In performing our professional services we have used generally accepted civil engineering principles and have applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable civil engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in these documents.

Draft

19.0 REFERENCES

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FIGURES

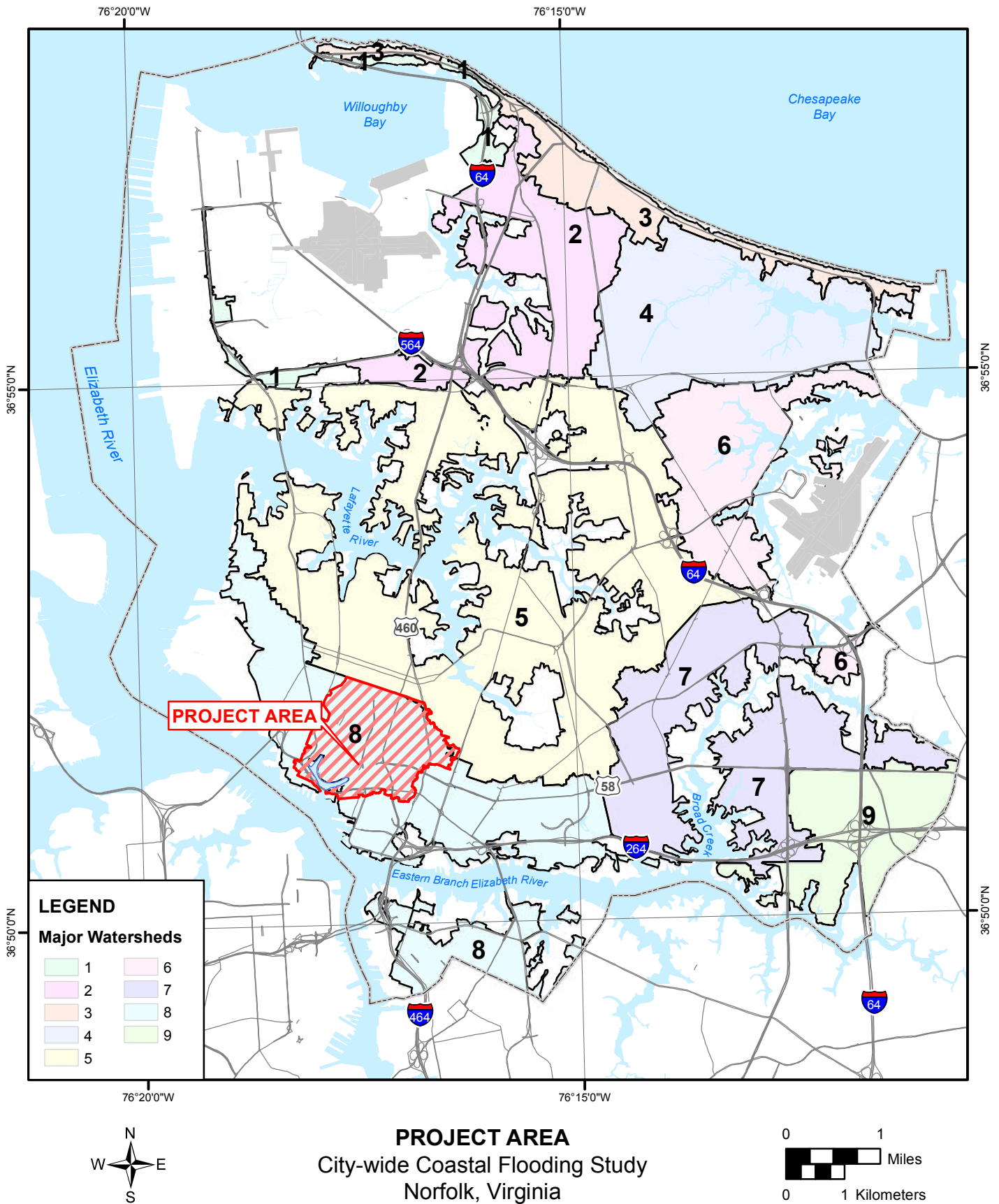


FIGURE 2-1



N:\Projects\3627_City_Norfolk\04_8111_0023_Hague\Outputs\Draft_PER\mxd\Fig-2-2_Hague_Watershed.mxd, 3/20/2012, kstevens



0 100 200
Feet

BASIN OUTLET
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 2-3



LEGEND

Topographic Elevation (ft. NAVD88)
Based on 2009 LiDAR Survey

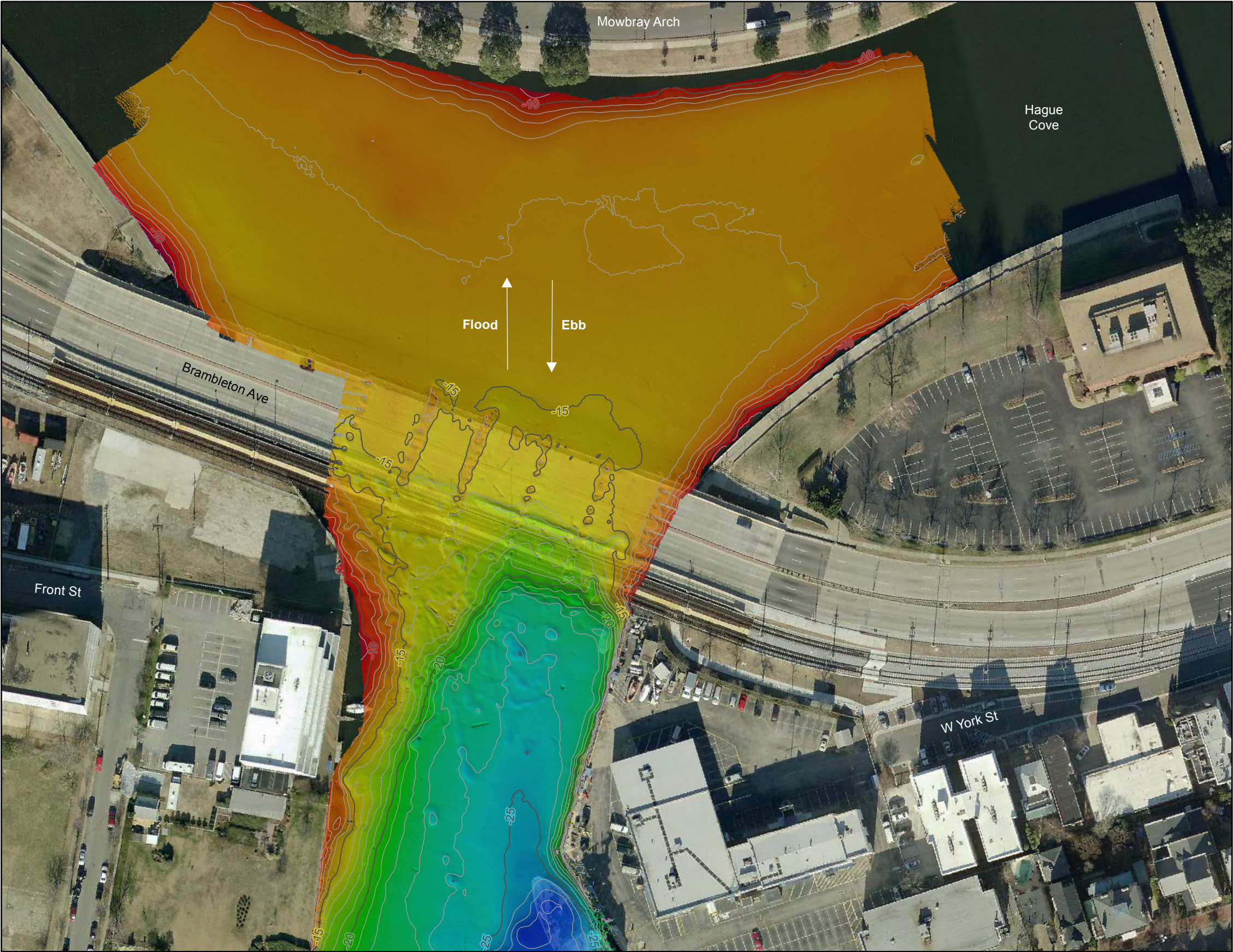
- < 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 9
- 9 to 10
- 10 to 11
- 11 to 12
- >12

TOPOGRAPHY
City-wide Coastal Flooding Study
Norfolk, Virginia



0 500 1,000
Feet

FIGURE 4-1a

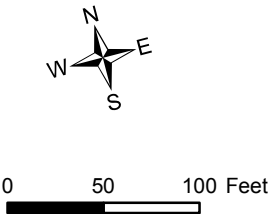


LEGEND

- Bathymetric Elevation (re: NAVD88)**
- Contour interval is 5 feet
 - Contour interval is 1 foot

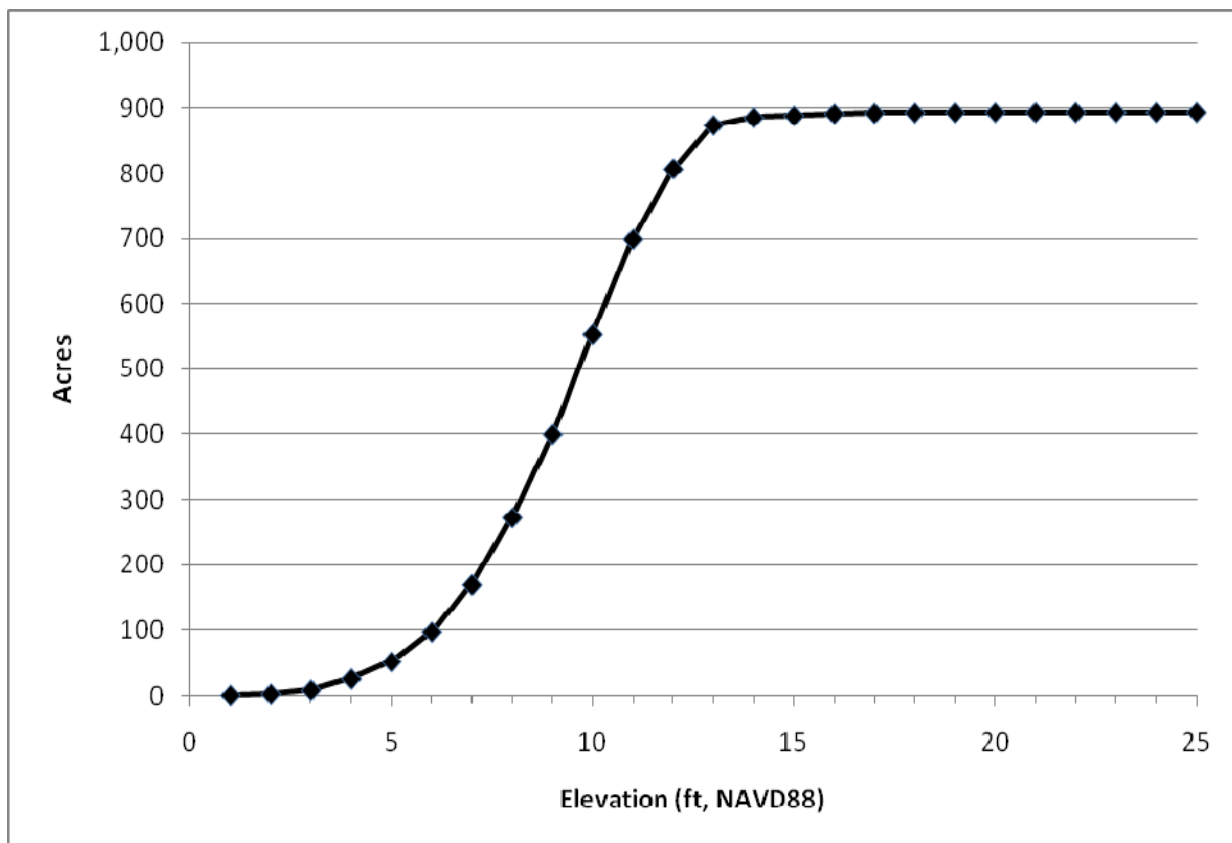
- Bathymetric Elevation (re: NAVD88)**
-  -7.7 feet
-  -28.4 feet

Note:
Bathymetry data collected by Fugro in December 2010 using a R2Sonic 2024 multibeam echosounder.



BATHYMETRY
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 4-1b

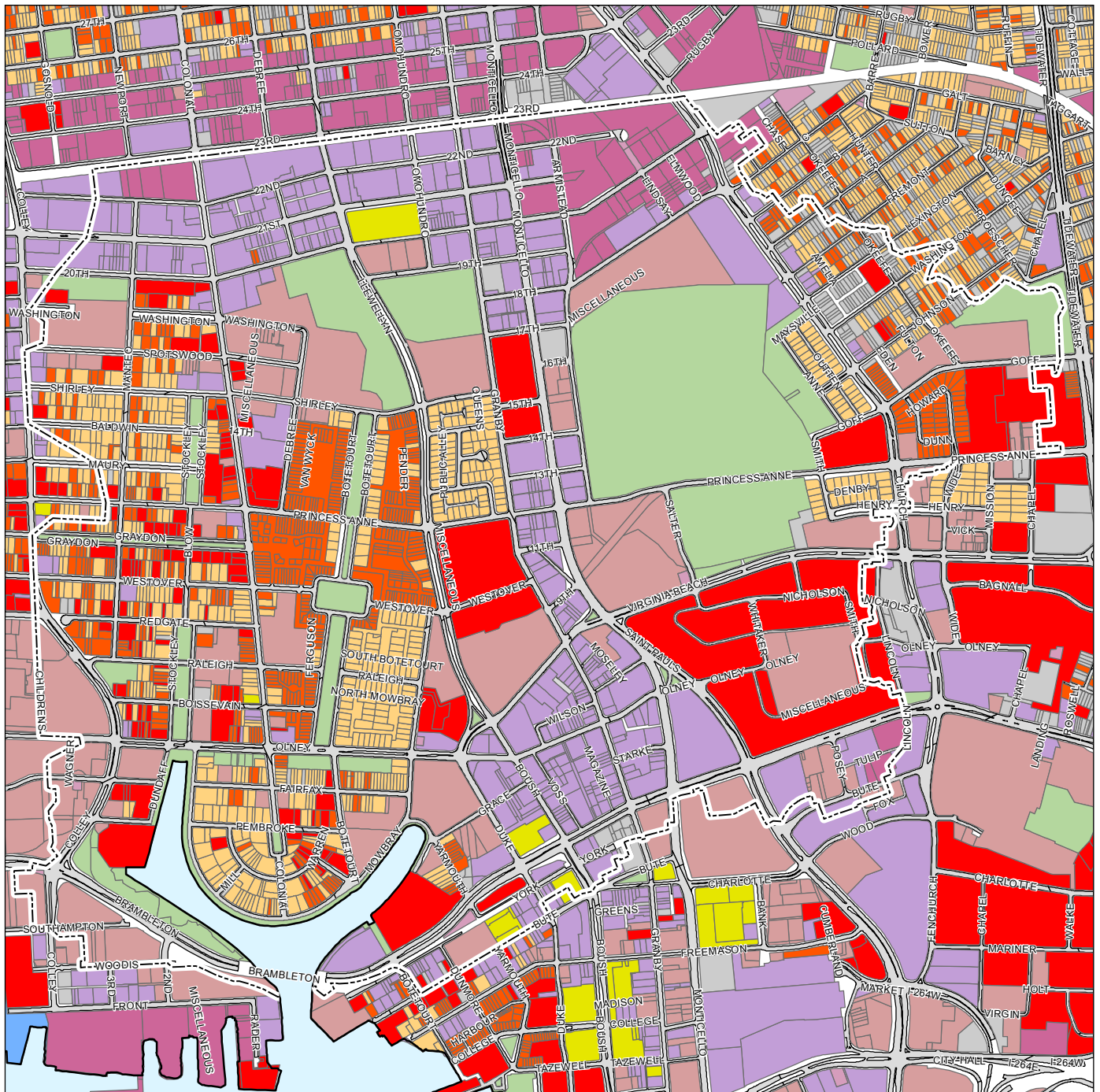


This graph represents a statistical characterization of the ground surface elevation within the Hague watershed. This cumulative frequency graph is based on the 2009 LiDAR survey data that has a 3-ft by 3-ft bin size (horizontal footprint is 3-ft by 3-ft). The watershed encompasses approximately 895 acres. Acreage estimates in this graph do not include the Hague water body.

Examples of how this graph may be interpreted:

- 1) 50 acres of the study area is equal to or below elevation 5 feet (NAVD88).
- 2) 400 acres of the study area is equal to or below elevation 9 feet (NAVD88).

STATISTICAL DISTRIBUTION OF TOPOGRAPHY
Cumulative Frequency
City-wide Coastal Flooding Study
Norfolk, Virginia



LEGEND

----- Hague Watershed Boundary

Existing Land Use

- High Density Residential
- Medium Density Residential
- Low Density Residential
- Mixed Use
- Commercial
- Industrial
- Institutional
- Military
- Transportation/Utility
- Open Space/Recreation
- Vacant

EXISTING LAND USE AREAS
City-wide Coastal Flooding Study
Norfolk, Virginia



0 500 1,000
Feet

FIGURE 4-3

N:\Projects\3627_City_Norfolk\04_8111_0023_Hague\Outputs\Draft_PER\mxd\Fig-4-4_Rim_Elevations.mxd, 3/20/2012, kstevens



LEGEND

Rim Elevation Segments (feet, NAVD88)

- Less than 2.2
- Less than 4.2
- Less than 4.8
- Less than 6.2
- Less than 7.0
- Less than 7.6
- Less than 8.2
- Greater than 8.2

BASIN RIM ELEVATION

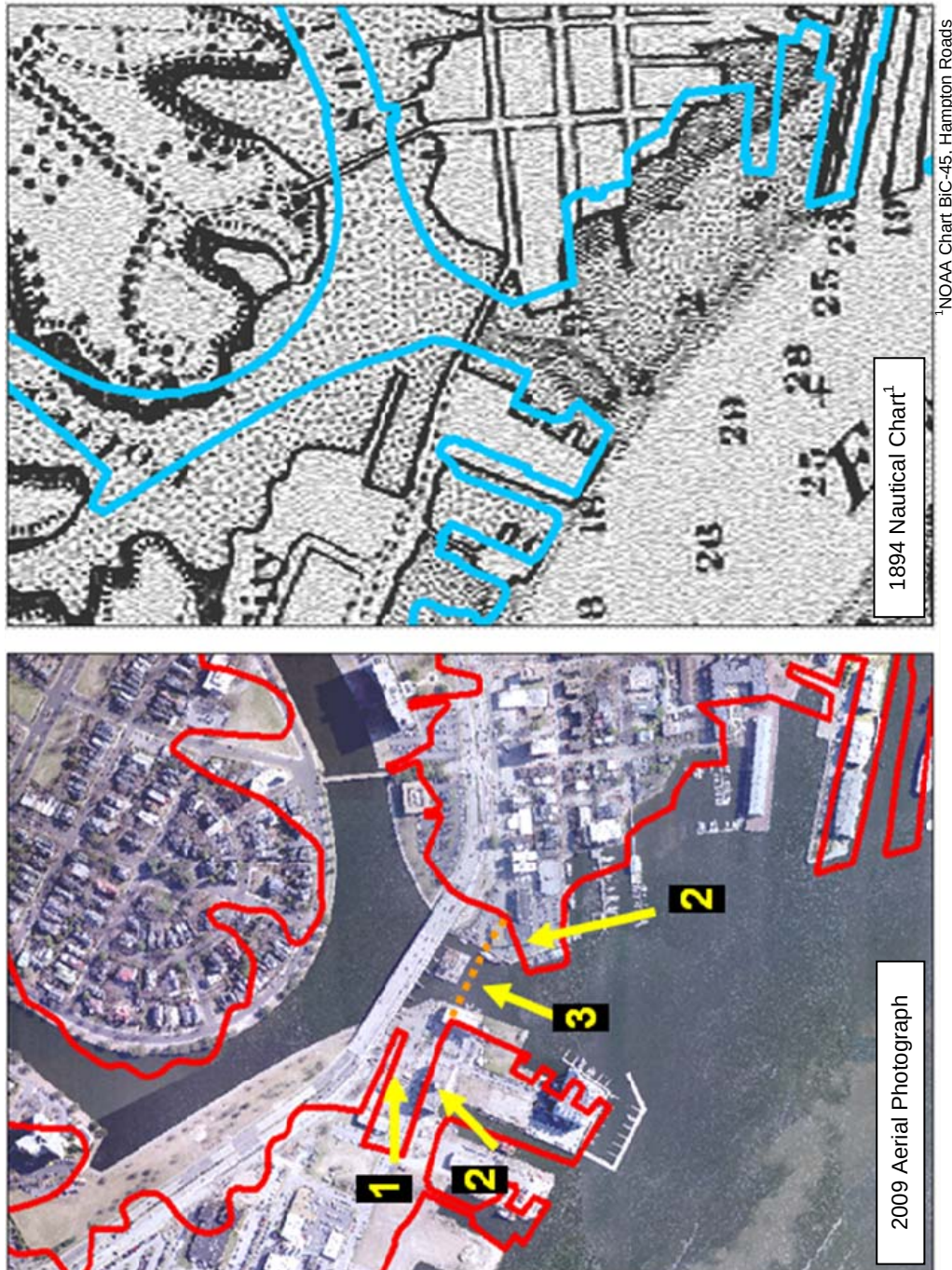
City-wide Coastal Flooding Study

Norfolk, Virginia



0 500 1,000
Feet

FIGURE 4-4



Numerous former structures may be present and challenging to tie new structures into shore

- 1 - Former Pier
- 2 - Former Hard Shoreline; possible concrete or piling structures
- 3 - Former Bridge

FORMER SHORELINE STRUCTURES

Basin Outlet

City-wide Coastal Flooding Study
Norfolk, Virginia



Photograph is of a cone penetration test (CPT) being conducted during the City-wide flooding study. Three CPTs were conducted at the Hague project area to collect geotechnical data. Photograph was taken at the CPT C-3 location north of Brambleton Avenue on the eastern shore of the Hague Cove.

PHOTOGRAPH OF CONE PENETRATION TEST
City-wide Coastal Flooding Study
Norfolk, Virginia

N:\Projects\3627_City_Norfolk\04_8111_0023_Hague\Outputs\Draft_PER\mxd\Fig-4-7_Key_to_Xsect.mxd, 3/26/2012, kspencer

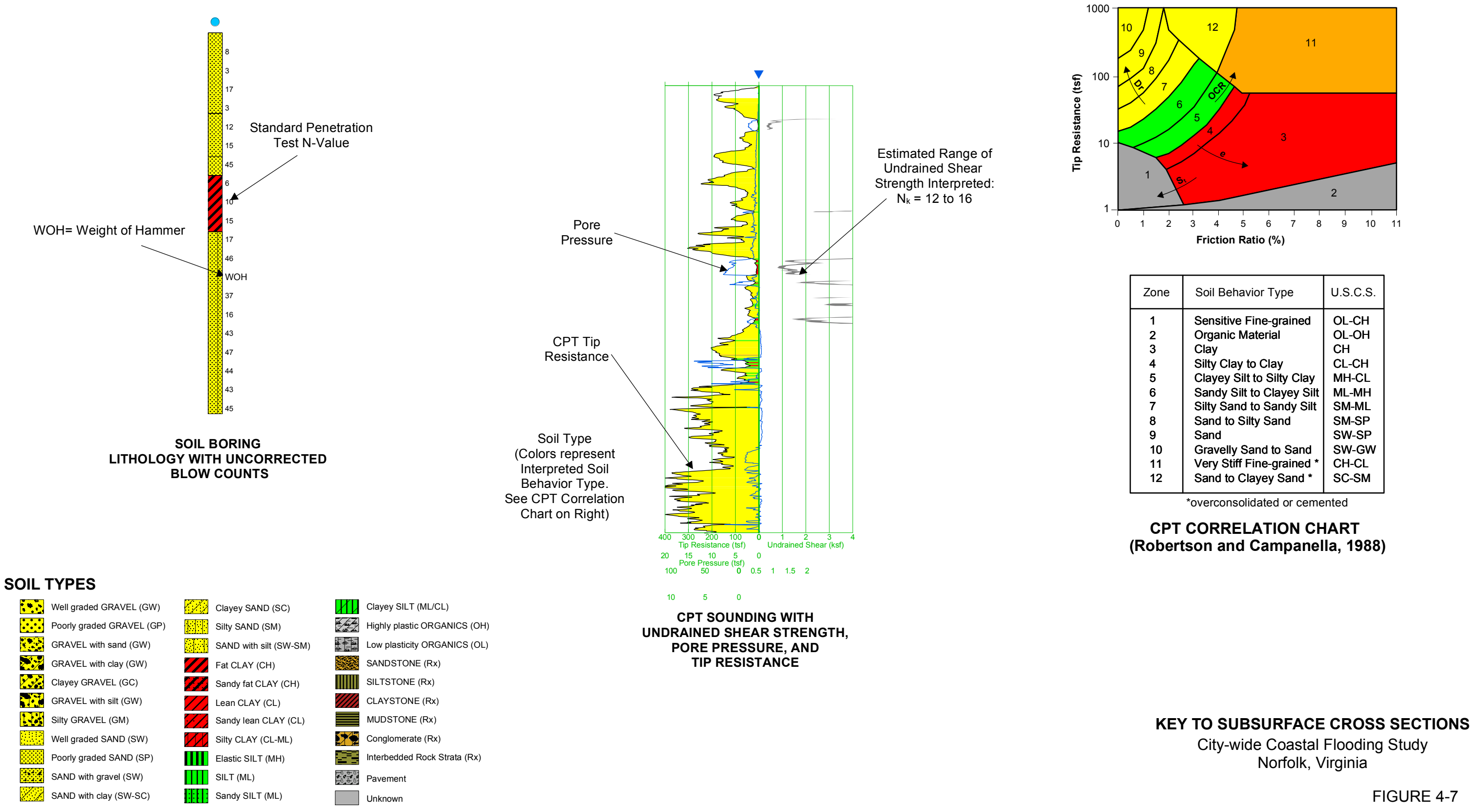
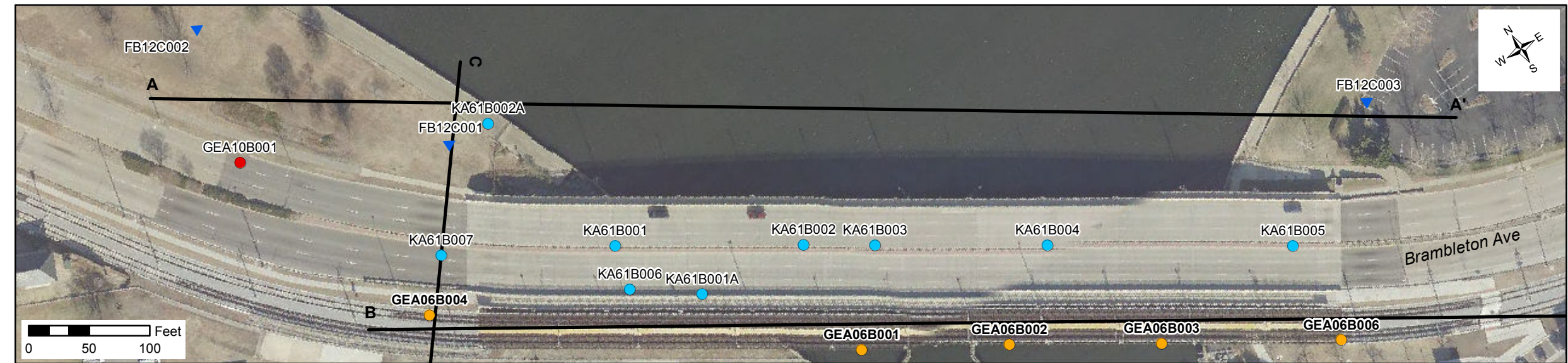


FIGURE 4-7



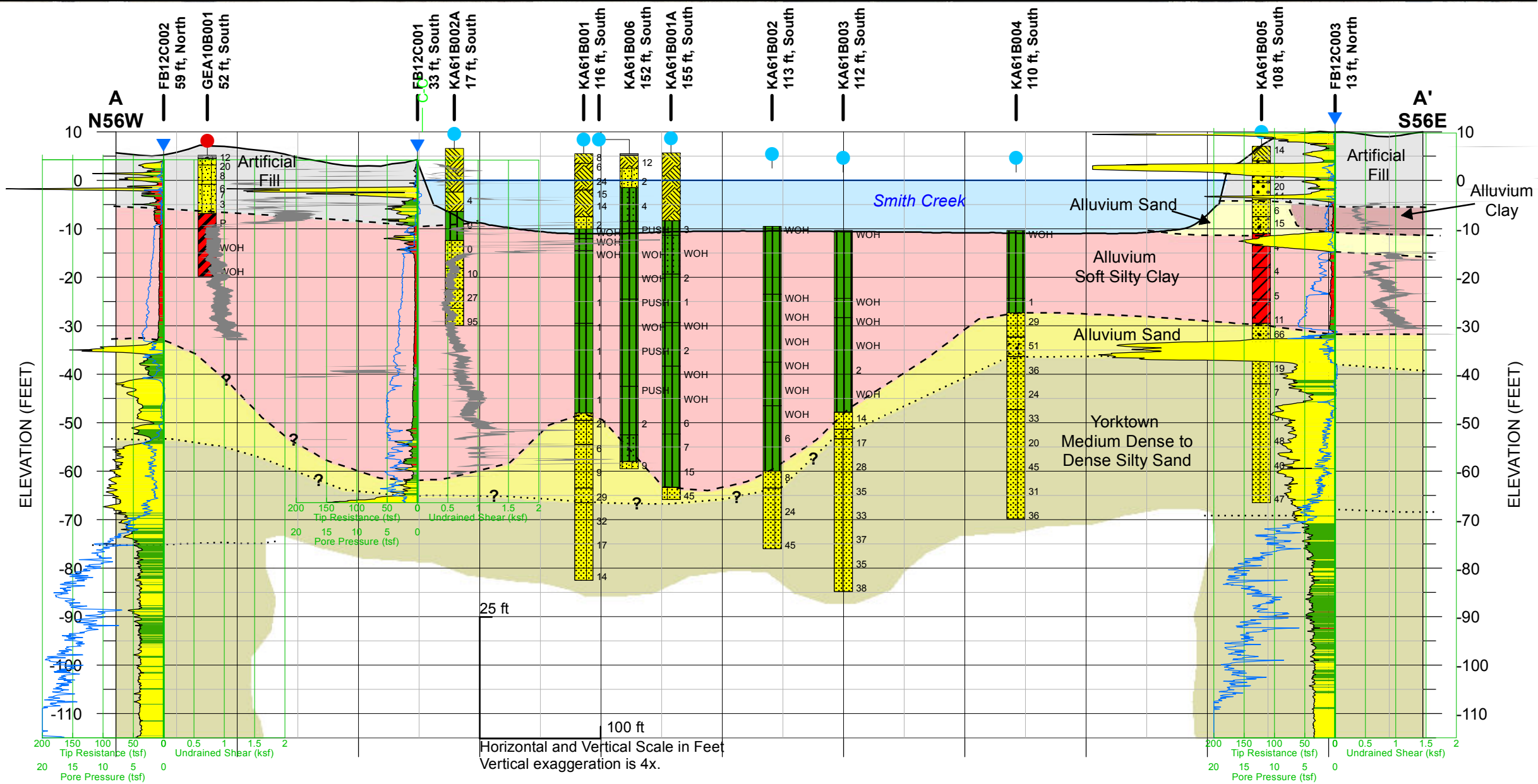
LEGEND

- 2012 Fugro CPT
- 2010 GET Solutions Inc. Boring
- 2006 GET Solutions Inc. Boring
- 1961 Knoeble & Graef Boring
- Cross Section Location

Soil Types

- Pavement
- FILL
- Poorly graded GRAVEL (GP)
- Poorly graded SAND (SP)
- SAND with gravel (SW)
- Silty SAND (SM)
- SAND with silt (SP-SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL)
- Elastic SILT (MH)
- SILT (ML)
- Sandy SILT (ML)
- Highly plastic ORGANICS (OH)

- Notes:
- Elevation datum is NAVD88.
 - Topography from 2009 Pictometry, Inc. LiDAR survey.
 - Bathymetry from December 2010 Fugro multibeam survey.
 - Stratigraphic contacts are approximate and are interpreted from CPT sounding and boring data. Conditions vary both along and perpendicular to the section line.
 - Refer to Figure 4-6 for key to symbolism used on cross section.



PRELIMINARY SUBSURFACE
CROSS SECTION A-A'
City-wide Coastal Flooding Study
Norfolk, Virginia

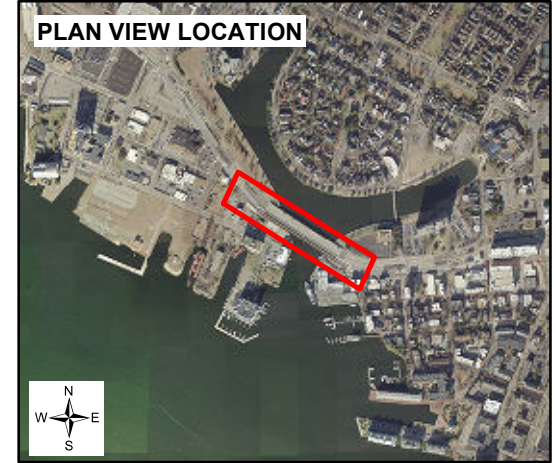
FIGURE 4-8



- LEGEND**
- 2012 Fugro CPT
 - 2010 GET Solutions Inc. Boring
 - 2006 GET Solutions Inc. Boring
 - 1961 Knoeble & Graef Boring
 - Cross Section Location

- Soil Types**
- Pavement
 - FILL
 - Poorly graded GRAVEL (GP)
 - Poorly graded SAND (SP)
 - SAND with gravel (SW)
 - Silty SAND (SM)
 - SAND with silt (SP-SM)
 - Fat CLAY (CH)
 - Lean CLAY (CL)
 - Sandy lean CLAY (CL)
 - Elastic SILT (MH)
 - SILT (ML)
 - Sandy SILT (ML)
 - Highly plastic ORGANICS (OH)

- Notes:**
- Elevation datum is NAVD88.
 - Topography from 2009 Pictometry, Inc. LiDAR survey.
 - Bathymetry from December 2010 Fugro multibeam survey.
 - Stratigraphic contacts are approximate and are interpreted from CPT sounding and boring data. Conditions vary both along and perpendicular to the section line.
 - Refer to Figure 4-6 for key to symbolism used on cross section.



**PRELIMINARY SUBSURFACE
CROSS SECTION B-B'**
City-wide Coastal Flooding Study
Norfolk, Virginia

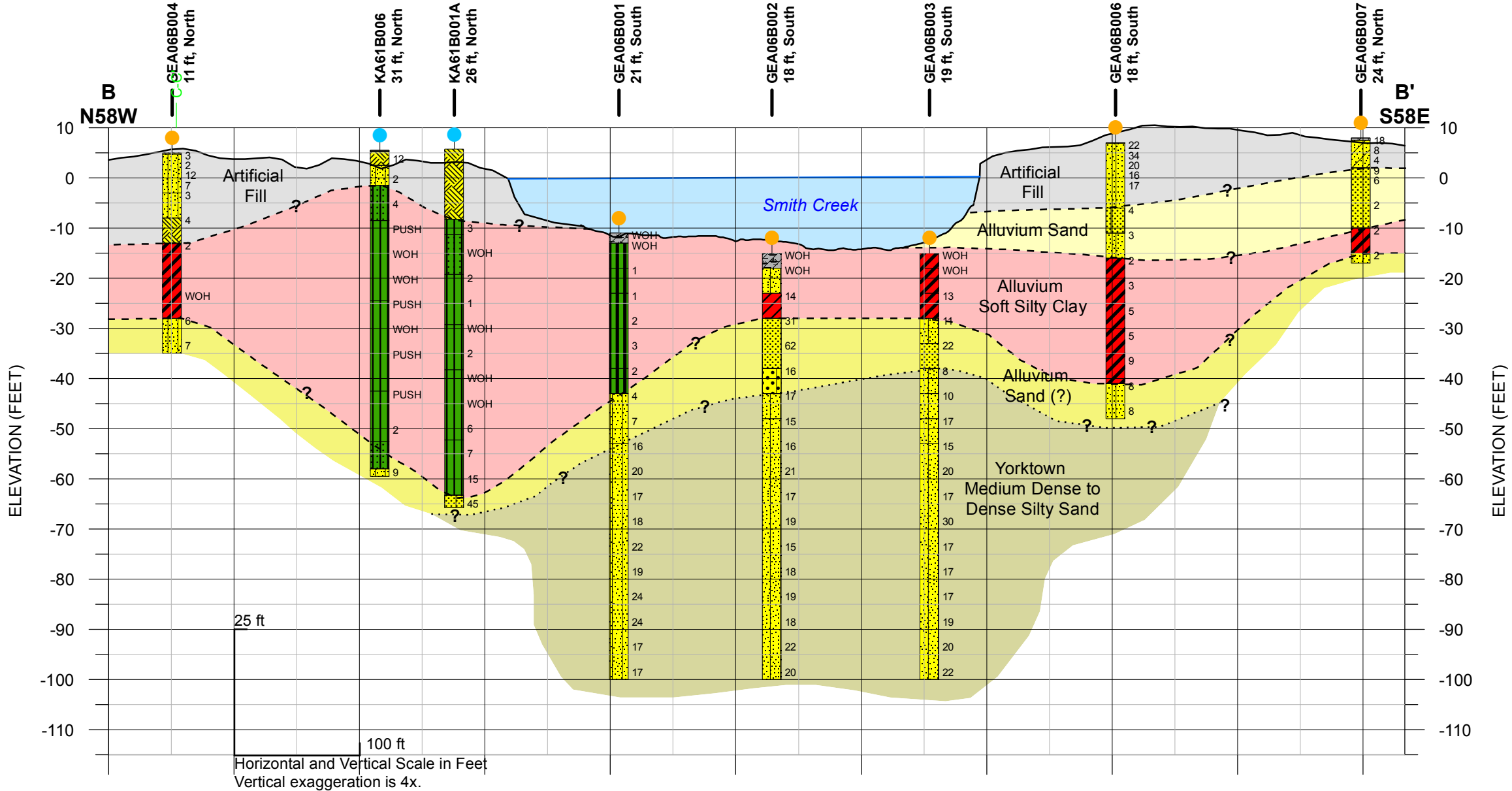


FIGURE 4-9



LEGEND

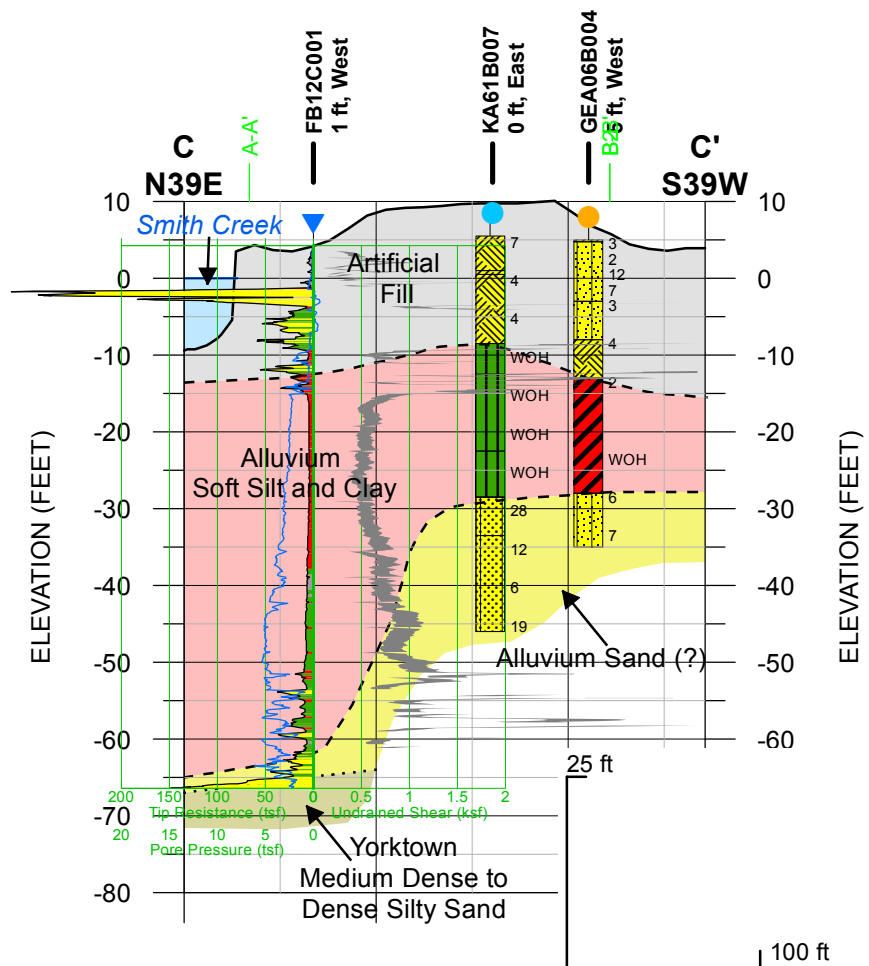
- ▼ 2012 Fugro CPT
- 2006 GET Solutions Inc. Boring
- 1961 Knoeble & Graef Boring
- Cross Section Location

Soil Types

- Pavement
- FILL
- Poorly graded GRAVEL (GP)
- Poorly graded SAND (SP)
- SAND with gravel (SW)
- Silty SAND (SM)
- SAND with silt (SP-SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL)
- Elastic SILT (MH)
- SILT (ML)
- Sandy SILT (ML)
- Highly plastic ORGANICS (OH)

Notes:

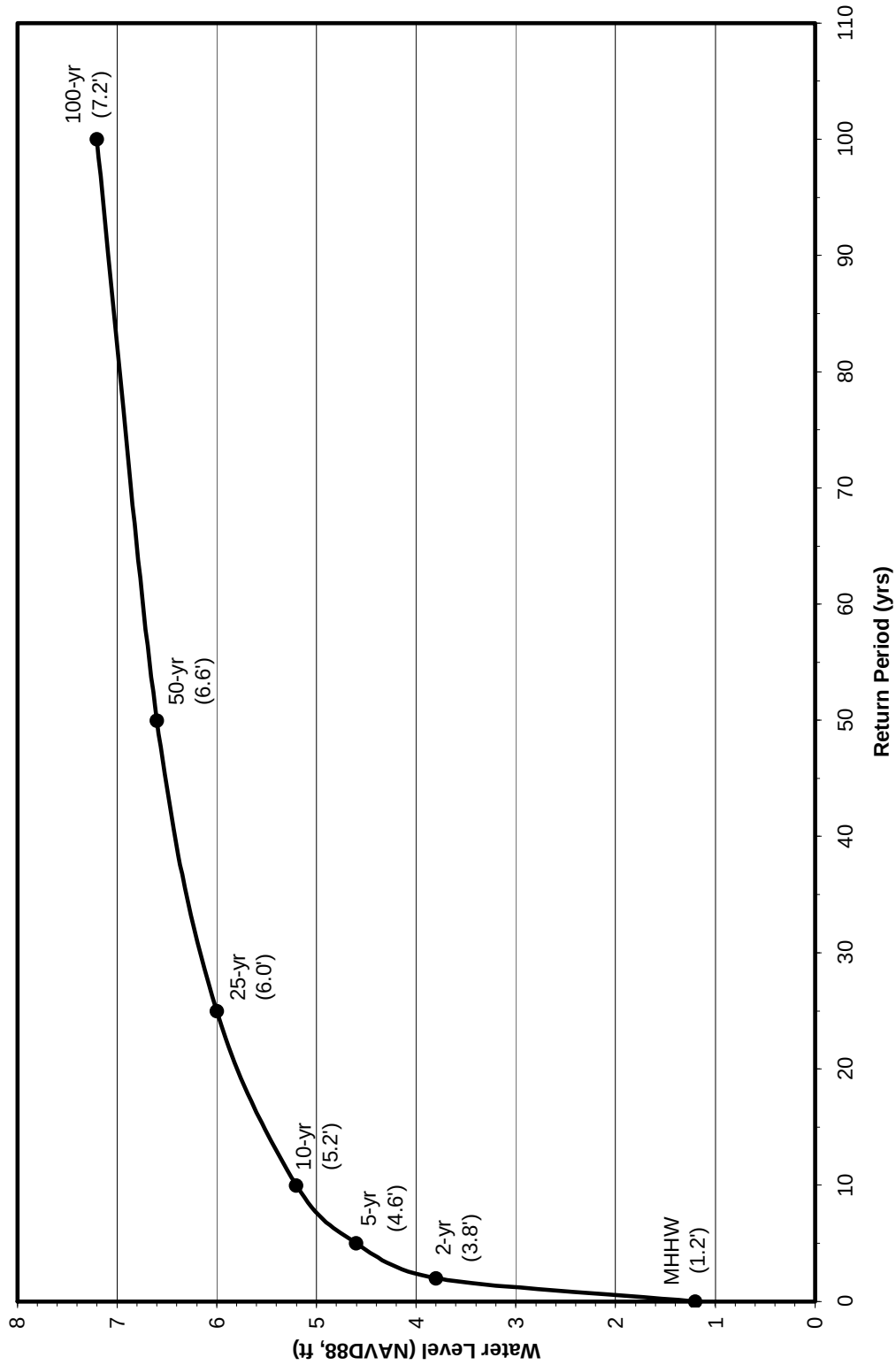
1. Elevation datum is NAVD88.
2. Topography from 2009 Pictometry, Inc. LiDAR survey.
3. Bathymetry from December 2010 Fugro multibeam survey.
4. Stratigraphic contacts are approximate and are interpreted from CPT sounding and boring data. Conditions vary both along and perpendicular to the section line.
5. Refer to Figure 4-6 for key to symbolism used on cross section.



**PRELIMINARY SUBSURFACE
CROSS SECTION C-C'**
City-wide Coastal Flooding Study
Norfolk, Virginia

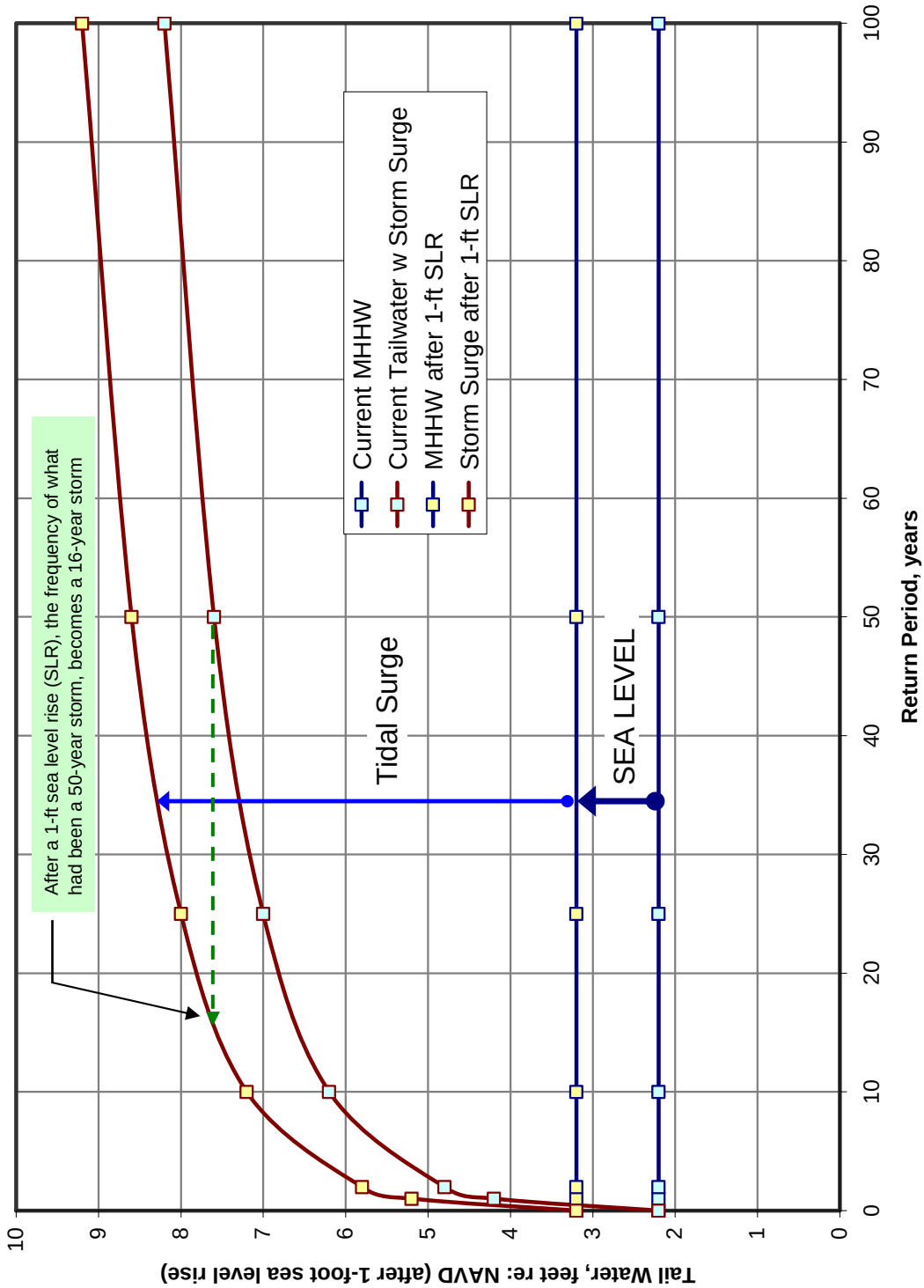
Horizontal and Vertical Scale in Feet
Vertical exaggeration is 4x.

FIGURE 4-10



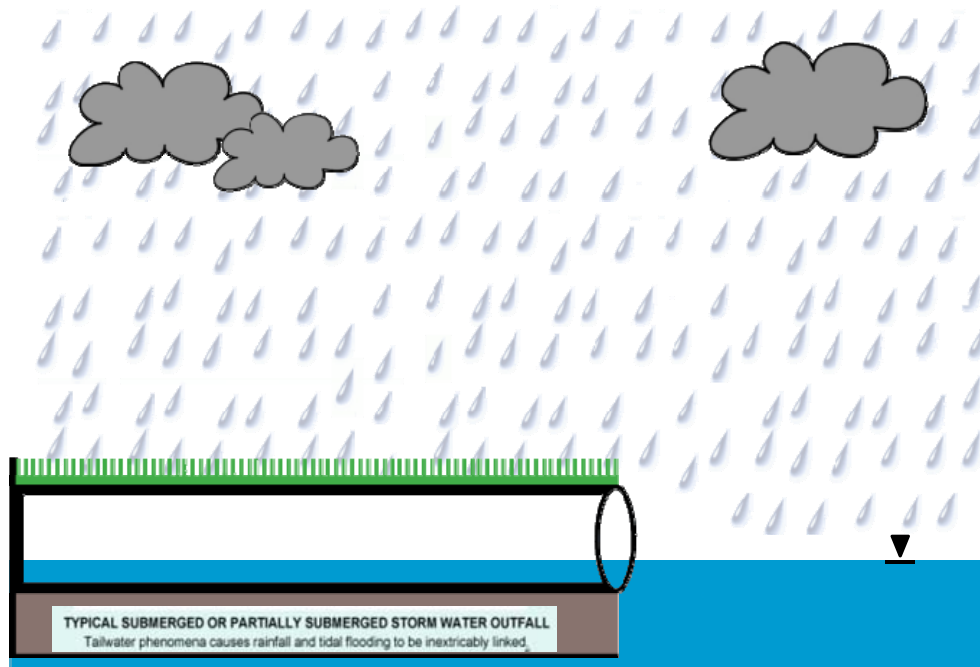
WATER LEVEL ELEVATIONS AT SEWELLS POINT FOR VARIOUS RETURN PERIODS
Based on Current Sea Level Elevation
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 5-1

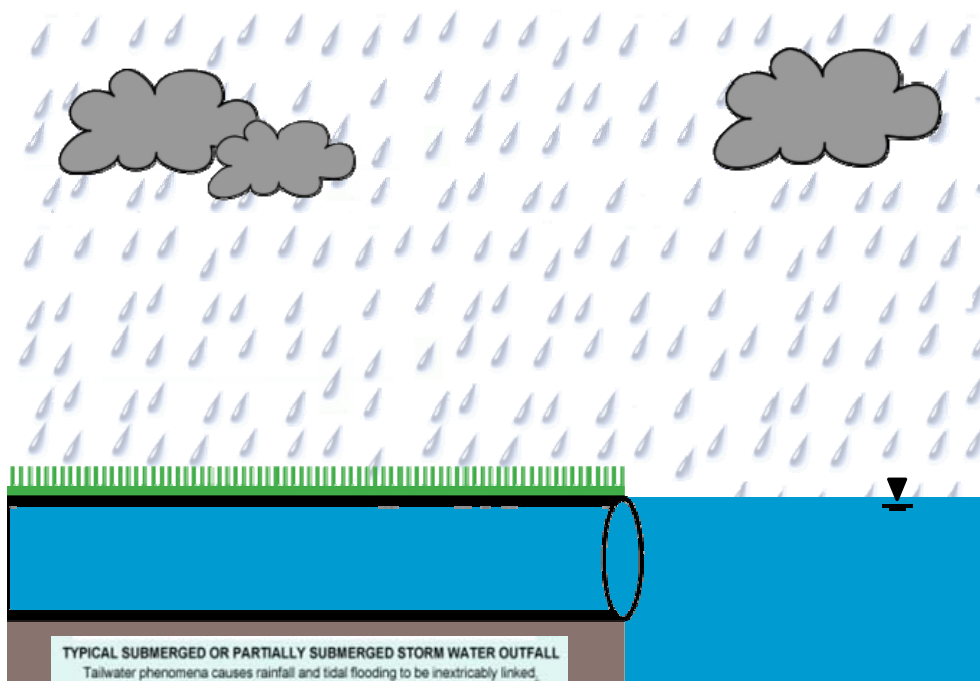


IMPLICATIONS OF FUTURE SEA LEVEL RISE
City-wide Coastal Flooding Study
Norfolk, Virginia

Rainfall Predominant



Coincident Surge Event



TAILWATER PHENOMENA

City-wide Coastal Flooding Study

Norfolk, Virginia



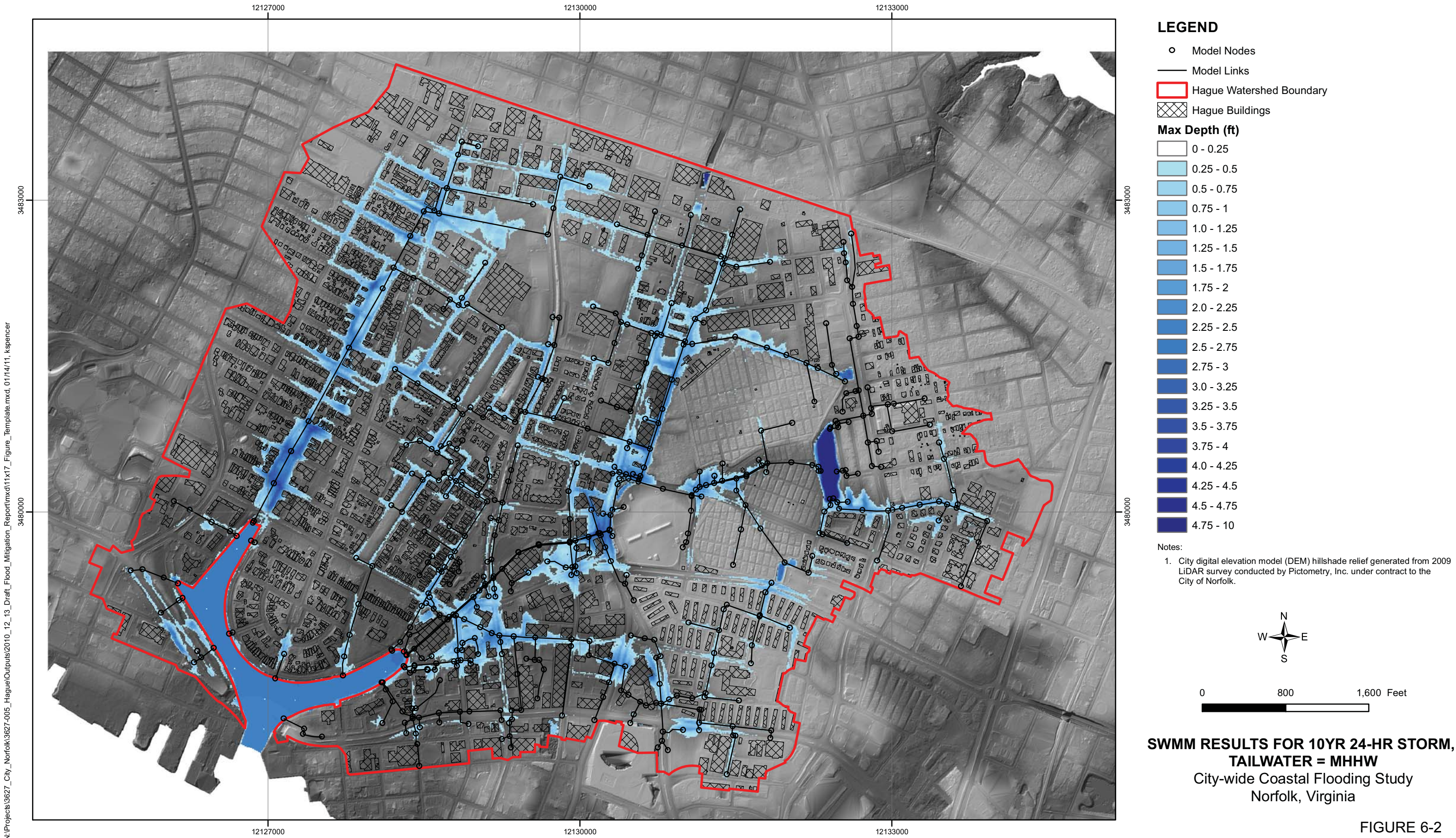


FIGURE 6-2

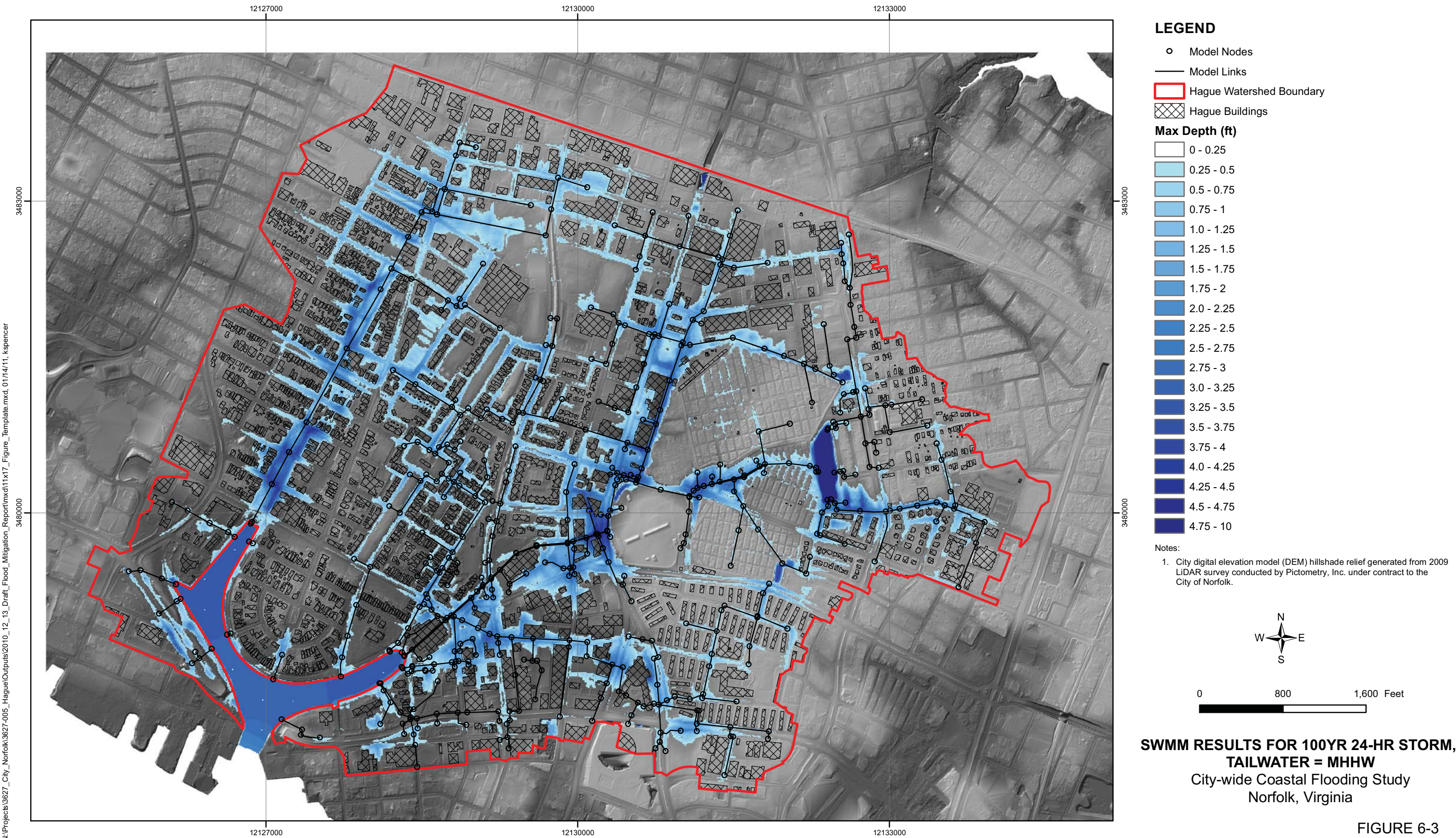


FIGURE 6-3

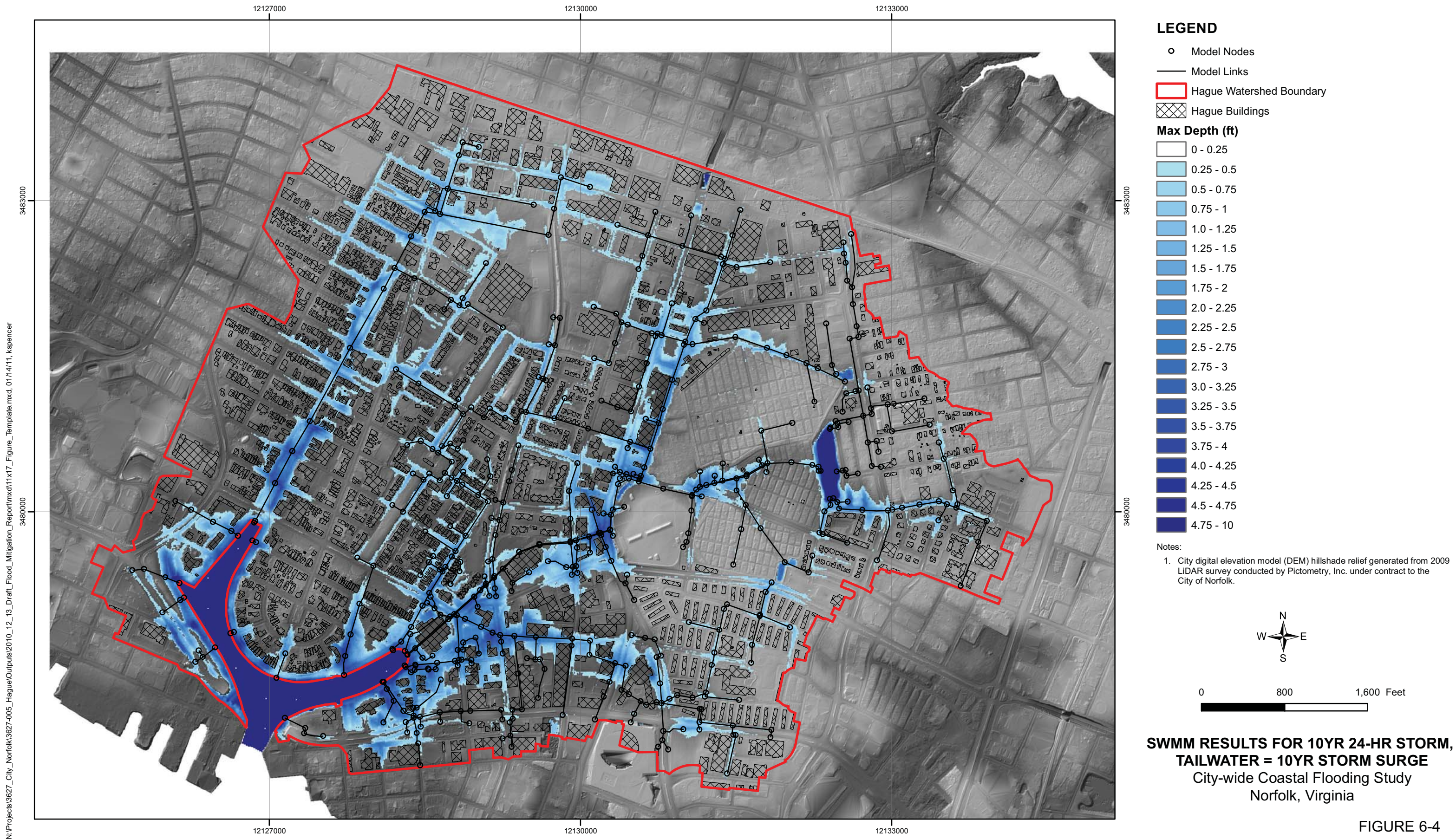


FIGURE 6-4

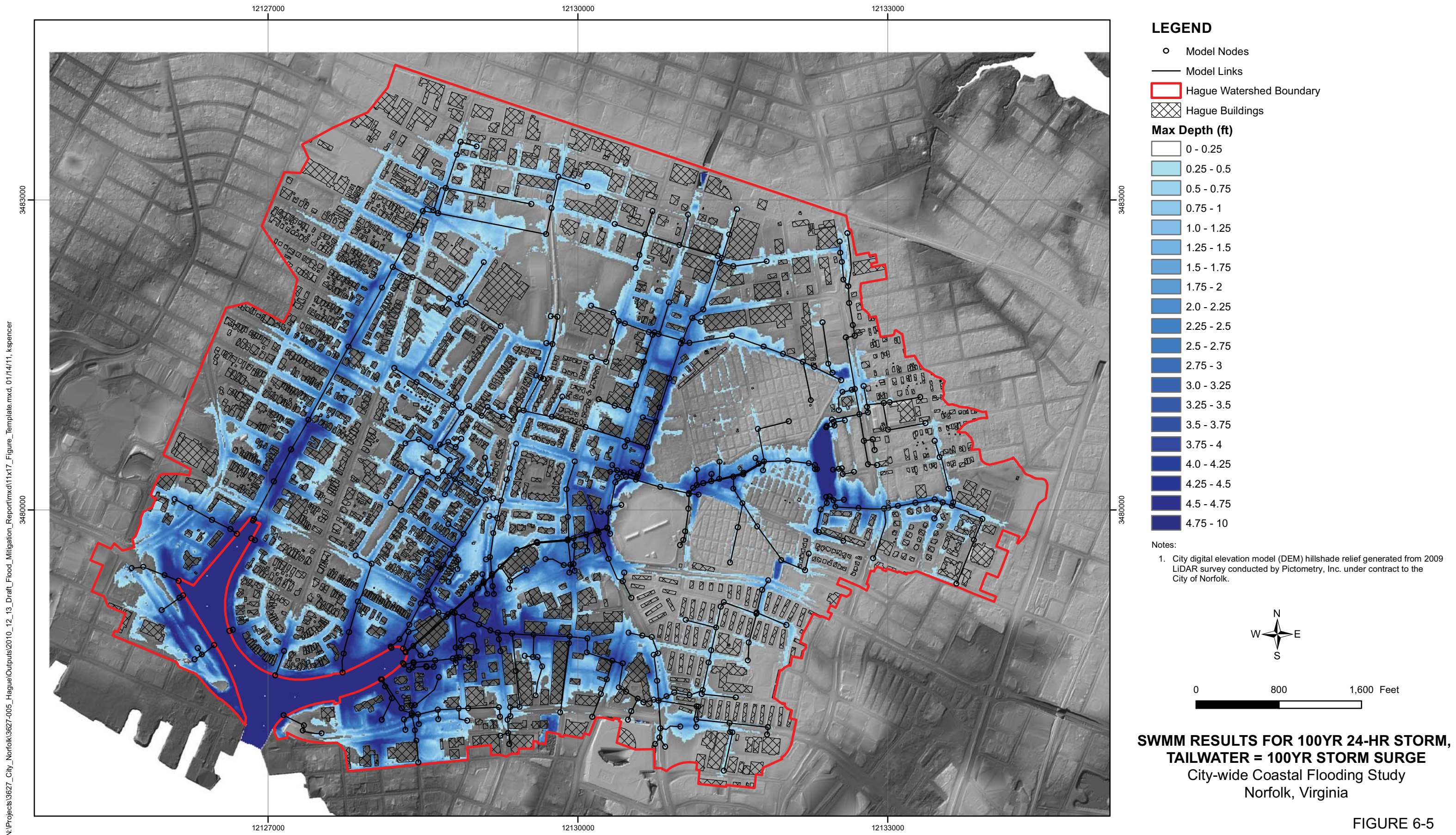


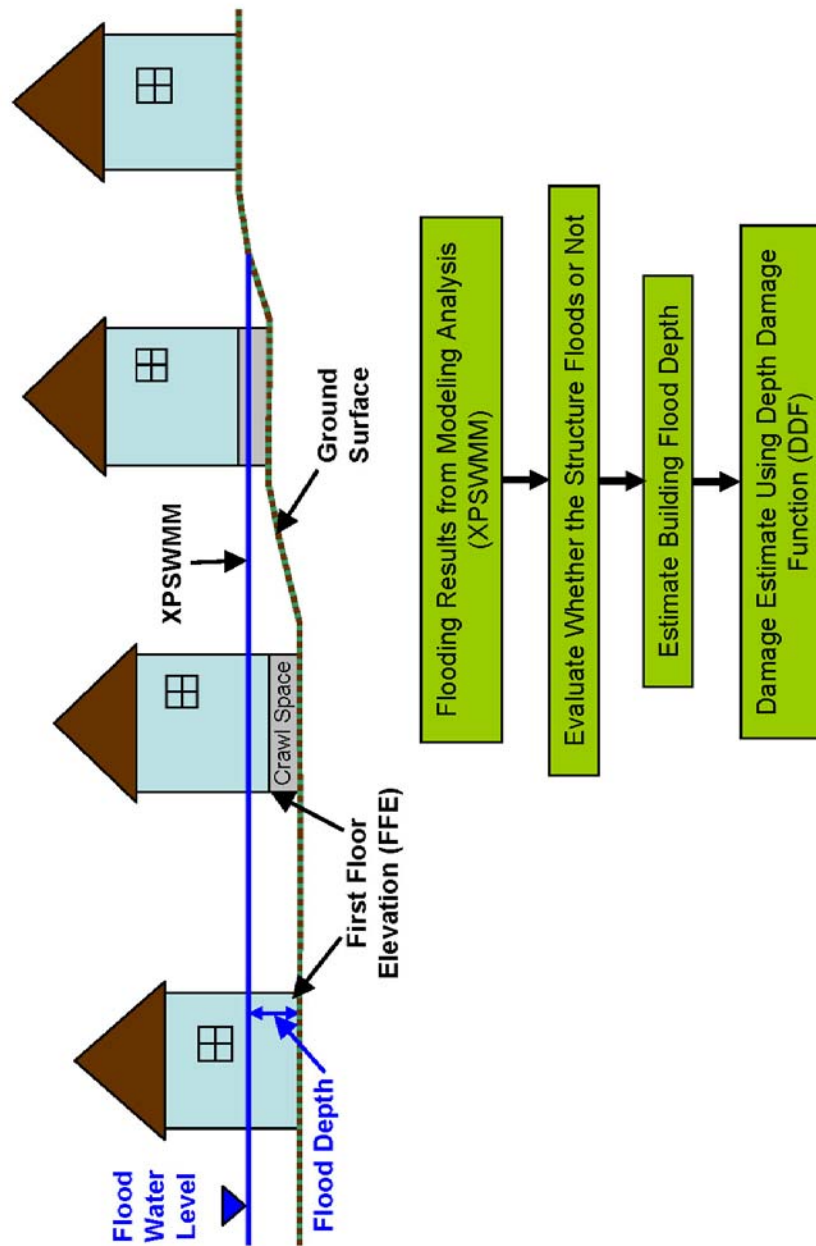
FIGURE 6-5



FIGURE 6-6



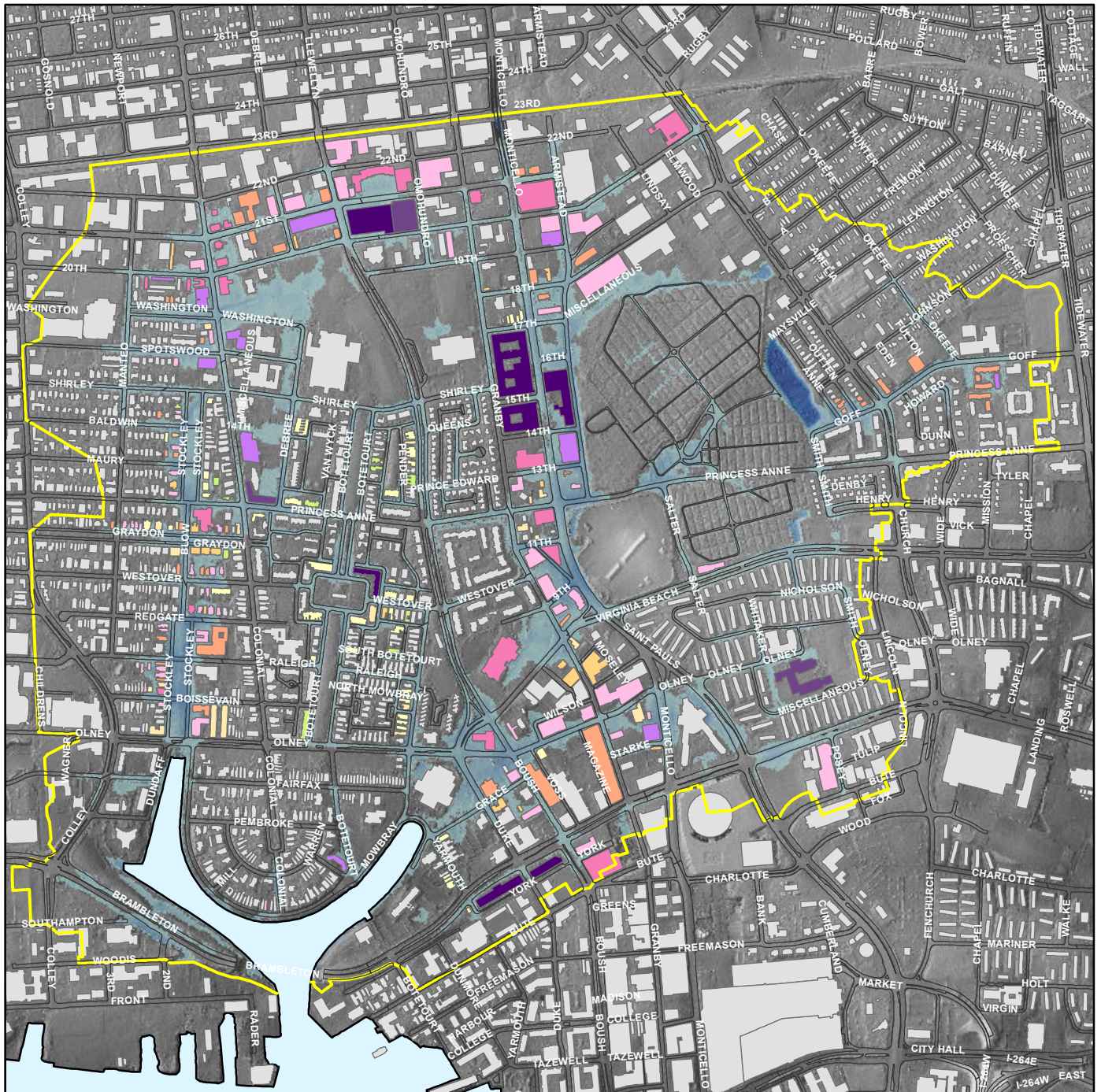
FIGURE 6-7



DEPTH DAMAGE FUNCTION CONCEPT

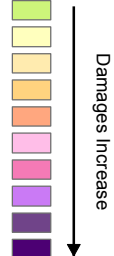
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 7-1



LEGEND

Total Estimated Damage
Buildings Not Damaged or
Not Included in Analysis

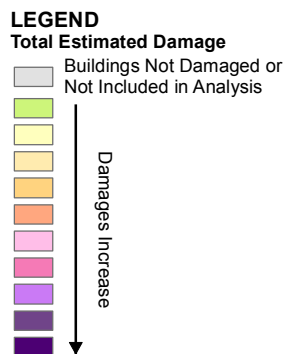


FLOOD DAMAGE ESTIMATES
10YR 24-HR STORM,
TAILWATER = MHHW
City-wide Coastal Flooding Study
Norfolk, Virginia



0 500 1,000
Feet

FIGURE 7-2



City-wide Coastal Flooding Study Norfolk, Virginia

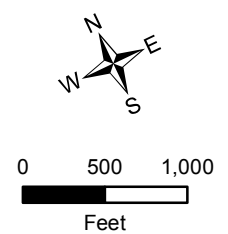
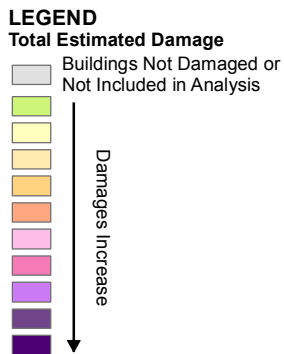
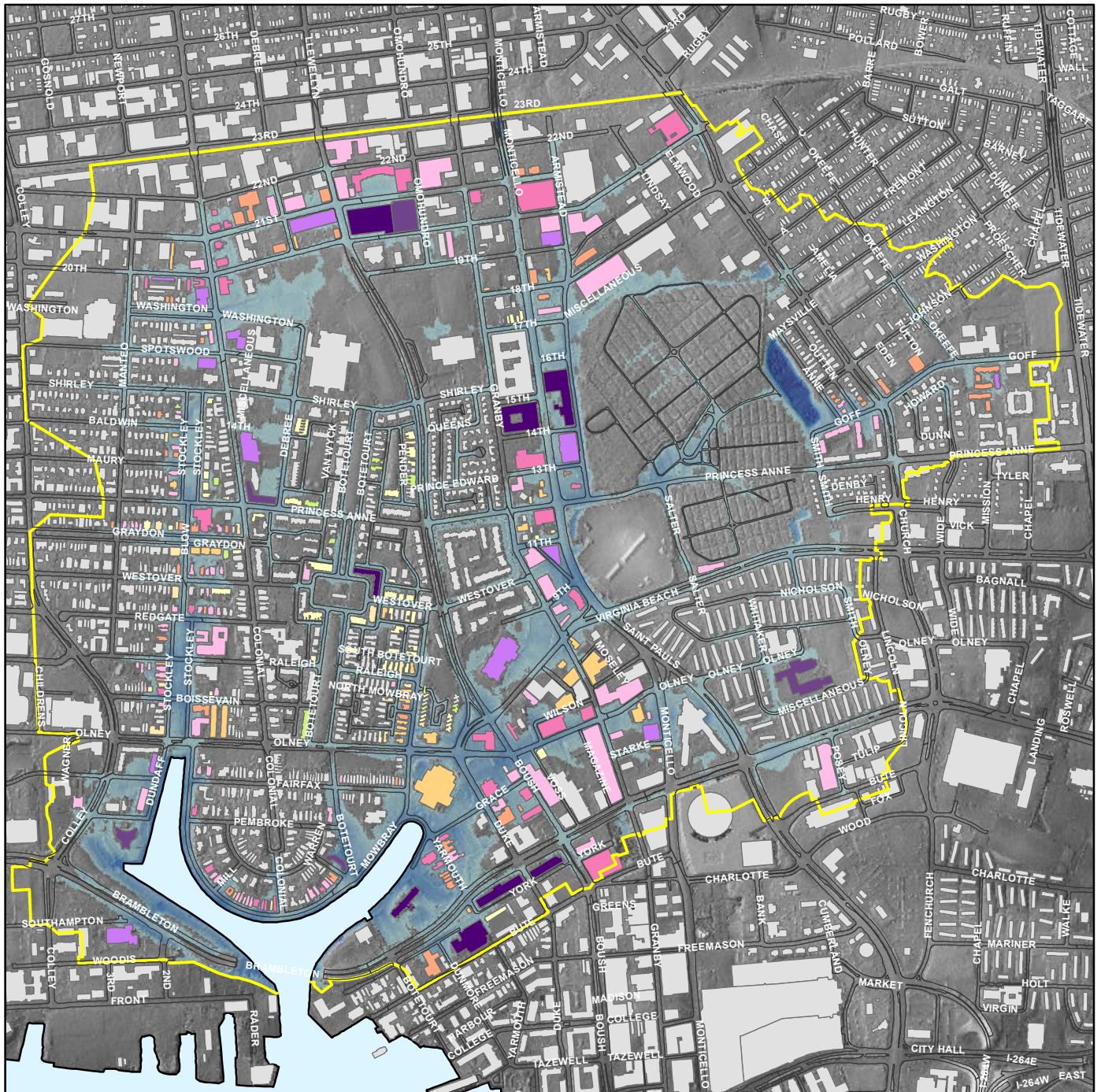


FIGURE 7-3



FLOOD DAMAGE ESTIMATES
10YR 24-HR STORM,
TAILWATER = 10YR STORM SURGE
City-wide Coastal Flooding Study
Norfolk, Virginia

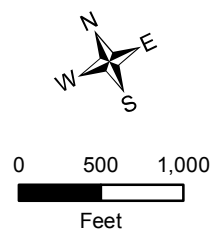
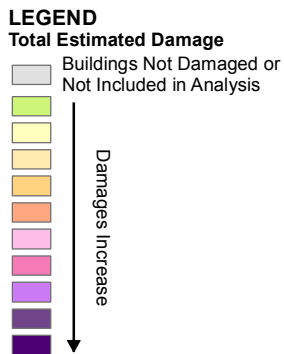
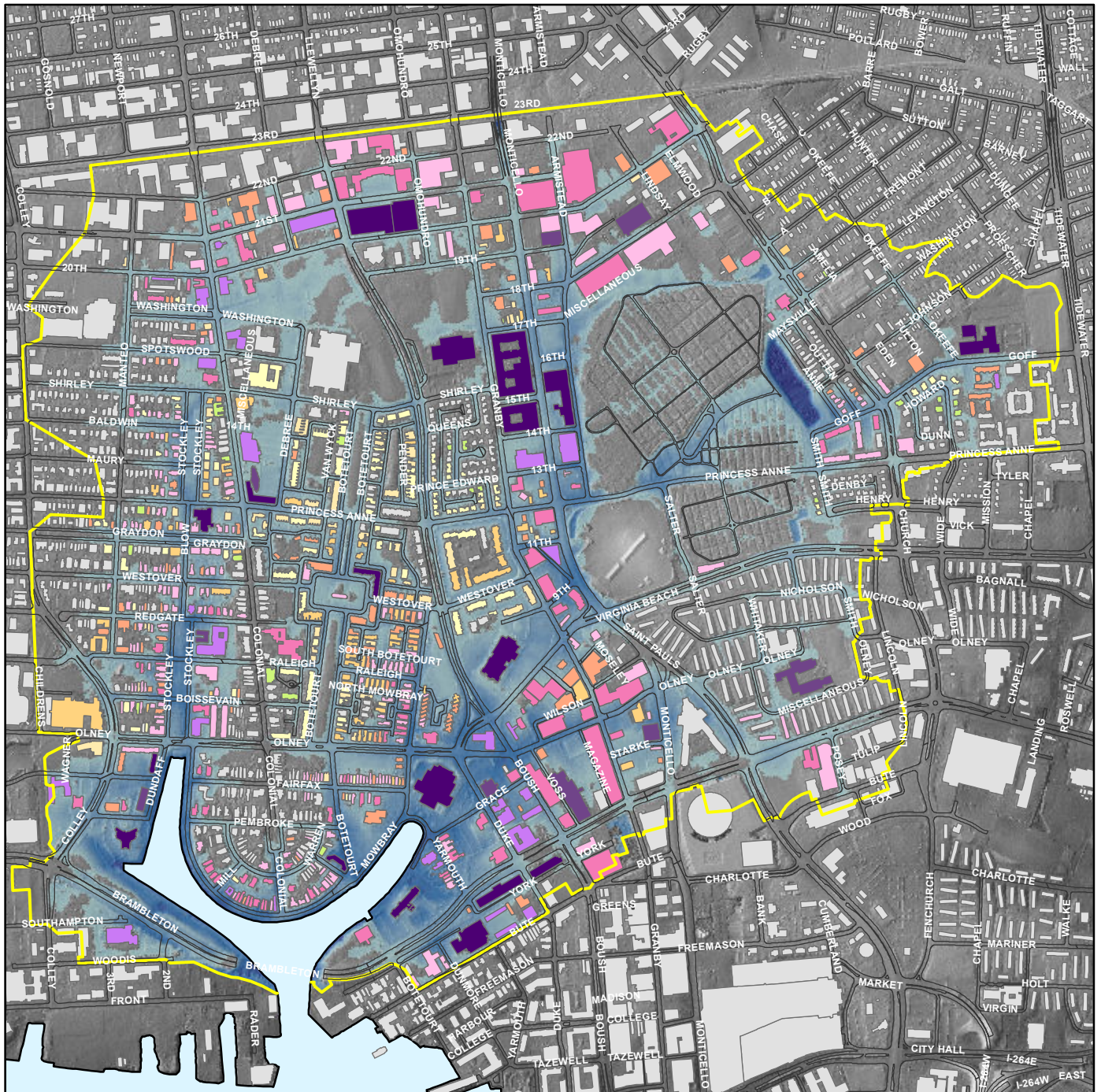


FIGURE 7-4

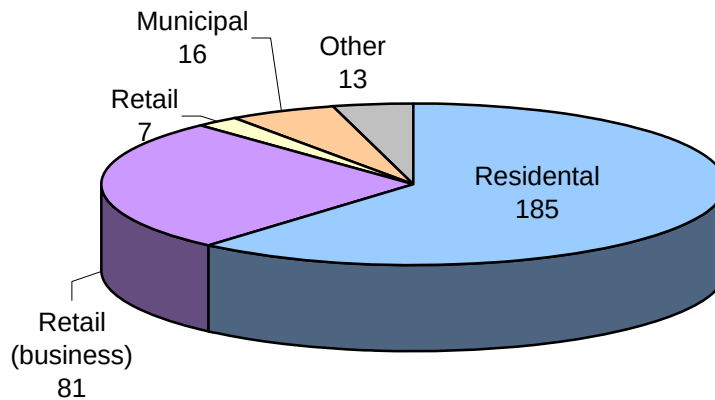


FLOOD DAMAGE ESTIMATES
100YR 24-HR STORM,
TAILWATER = 100YR STORM SURGE
City-wide Coastal Flooding Study
Norfolk, Virginia

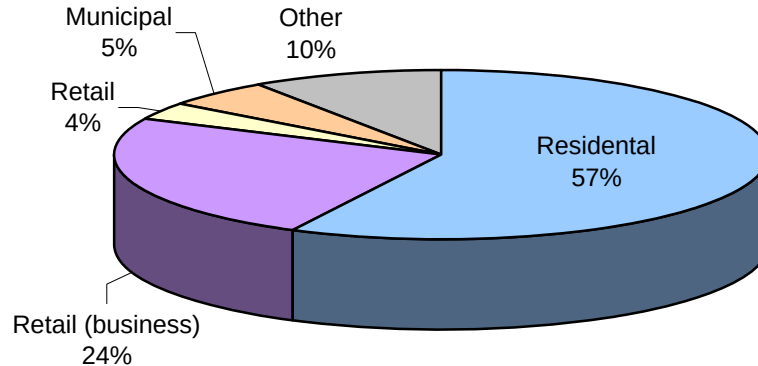


0 500 1,000
Feet

FIGURE 7-5



Number of Buildings Damaged by Flood



Cost Distribution of Flood Damage

Description and example subtypes for the above general categories:

Retail - Commodity-based businesses (department stores, grocery stores, convenience markets, gas stations, etc)

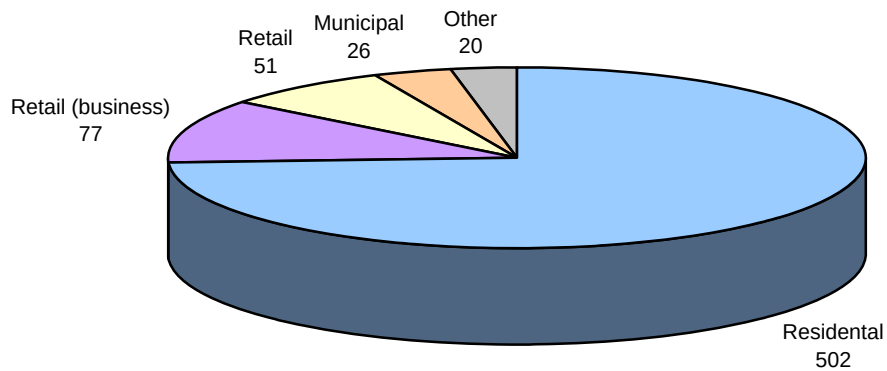
Retail (business) - Service-based businesses (office buildings, hotels, storage centers, service stations, funeral homes, exercise centers, etc)

Municipal - Government owned or operated facilities as well as health service facilities (hospitals, schools, parks, medical clinics/health centers, museums, etc)

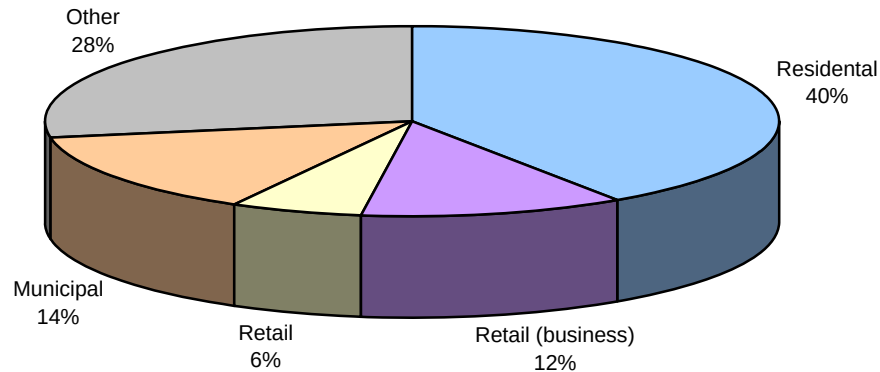
Residential - All private owned or government owned dwellings (single-family homes, condominiums, apartments, duplexes, etc)

Other - Unclassified property

SUMMARY OF FLOOD DAMAGE ESTIMATES
10YR 24-HR STORM, TAILWATER = 10YR STORM SURGE
 City-wide Coastal Flooding Study
 Norfolk, Virginia



Number of Buildings Damaged by Flood



Cost Distribution of Flood Damage

Description and example subtypes for the above general categories:

Retail - Commodity-based businesses (department stores, grocery stores, convenience markets, gas stations, etc)

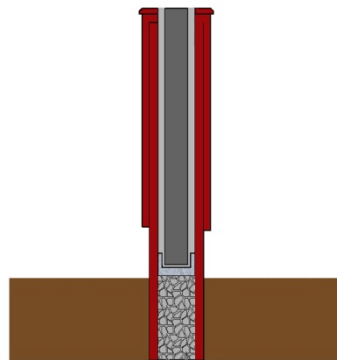
Retail (business) - Service-based businesses (office buildings, hotels, storage centers, service stations, funeral homes, exercise centers, etc)

Municipal - Government owned or operated facilities as well as health service facilities (hospitals, schools, parks, medical clinics/health centers, museums, etc)

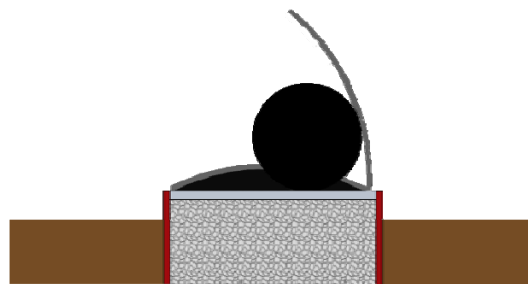
Residential - All private owned or government owned dwellings (single-family homes, condominiums, apartments, duplexes, etc)

Other - Unclassified property

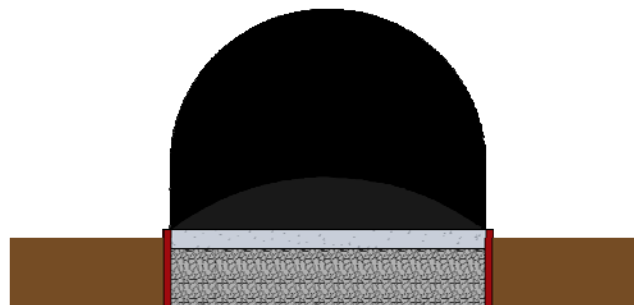
SUMMARY OF FLOOD DAMAGE ESTIMATES
100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE
 City-wide Coastal Flooding Study
 Norfolk, Virginia



Steel Gate Option



Obermeyer Gate Option

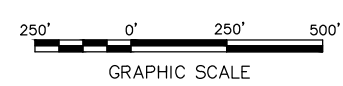


Inflatable Dam Option

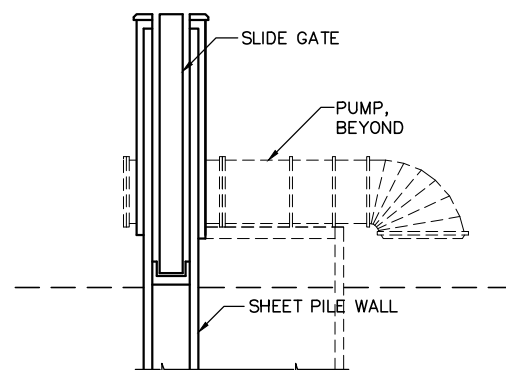
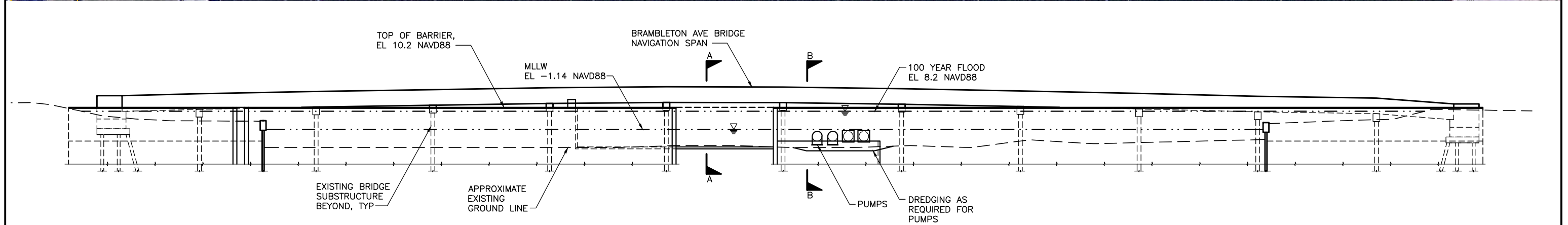
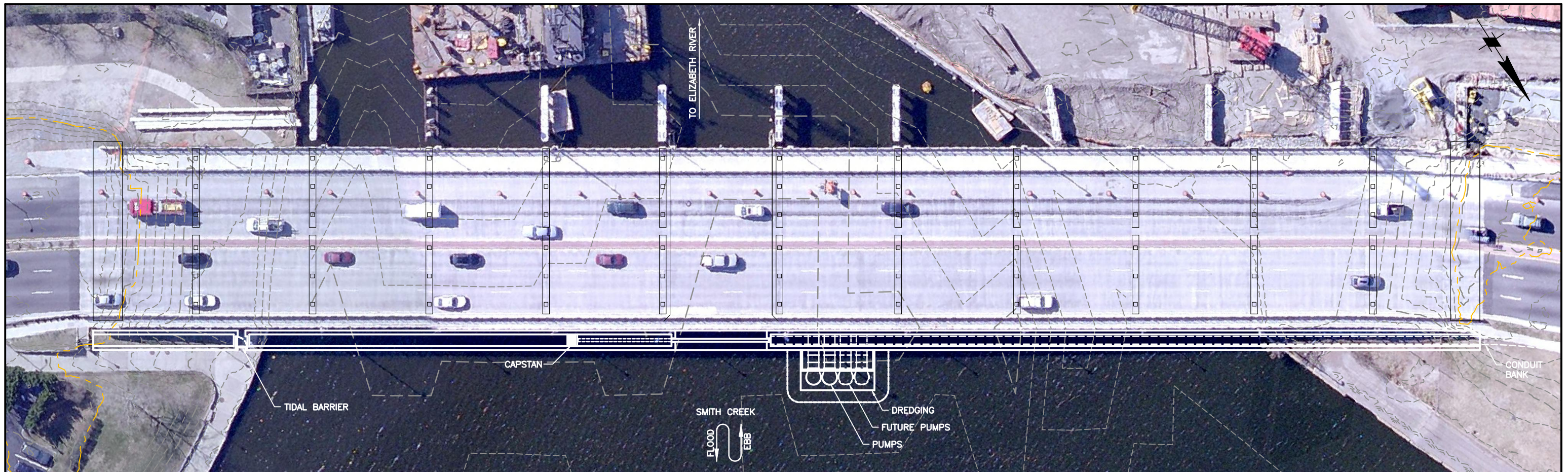
SCHEMATIC OF TIDE GATE TYPE OPTIONS
City-wide Coastal Flooding Study
Norfolk, Virginia



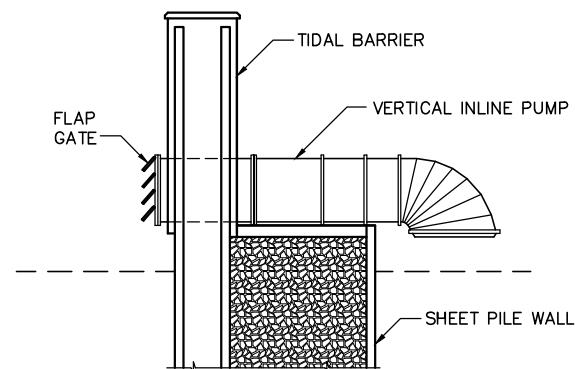
— PRIMARY OPTION
- - - - - ADDITIONAL OPTION



				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA		FIGURE 8-2
DSGN JOB NO. 6822-03		DR SCALE 1"=500' Koons		CHK DATE 12-14-2010	THE HAGUE TIDAL BARRIER WITH TIDE GATE, PUMPS, AND CLOSURE WALLS — BERMS	



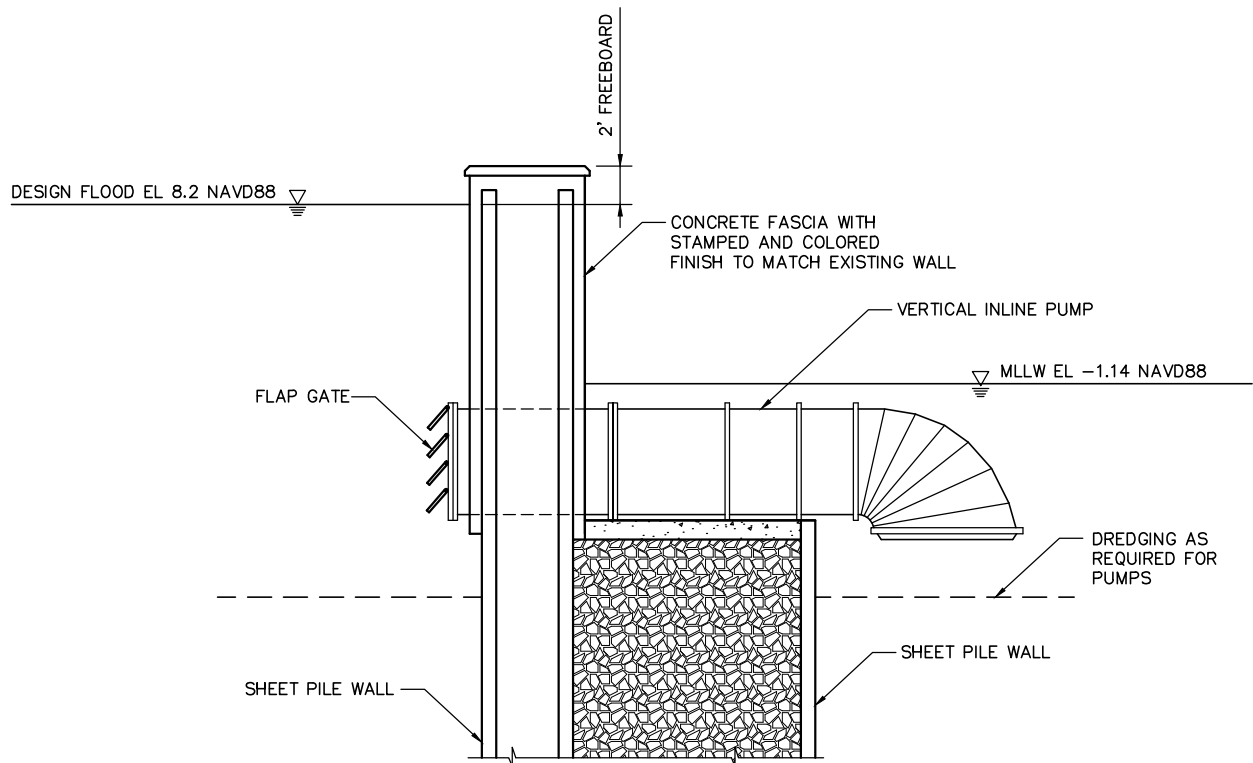
SECTION A-A



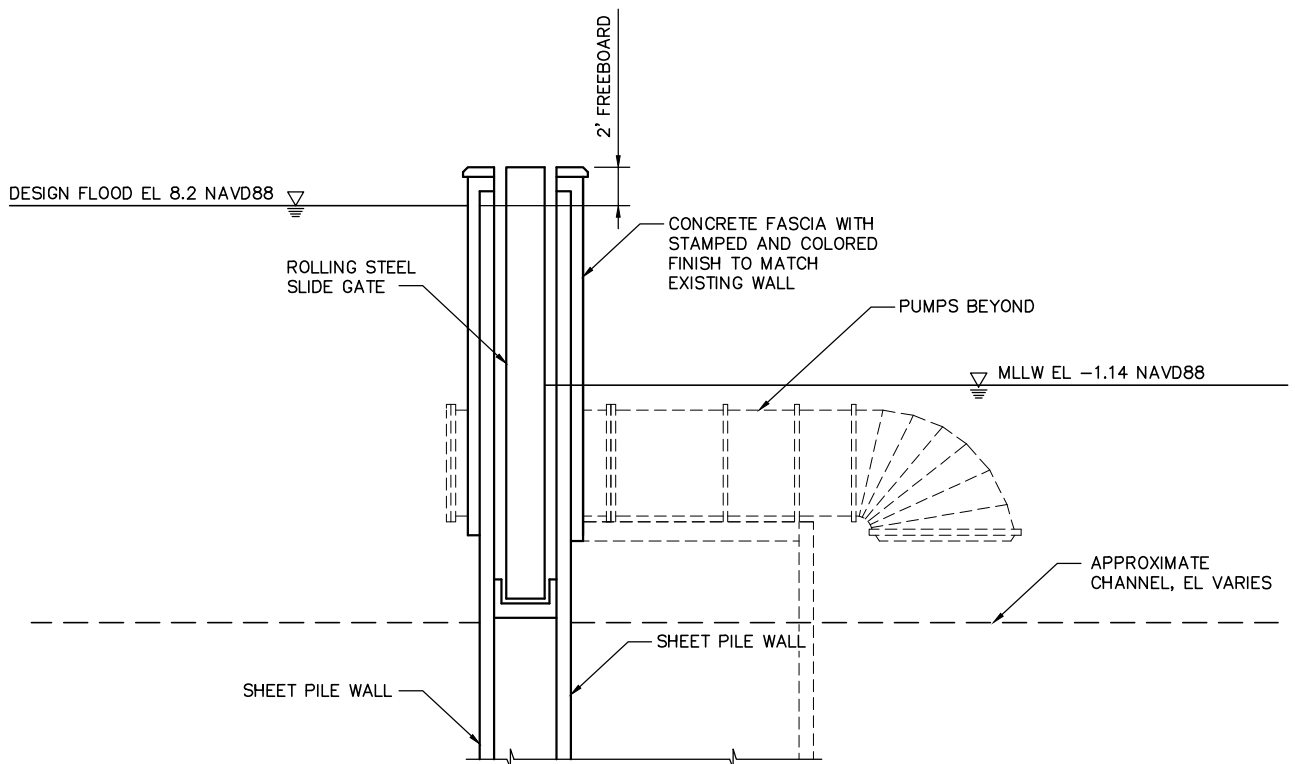
SECTION B-B



 moffatt & nichol					COASTAL FLOODING PROJECT NORFOLK, VIRGINIA		FIGURE 8-3
DSGN		DR	CHK		THE HAGUE		
JOB NO. 6822-03		SCALE 1"=50'	DATE 12-14-2010		BRAMBLETON AVENUE BRIDGE STEEL GATE OPTION		

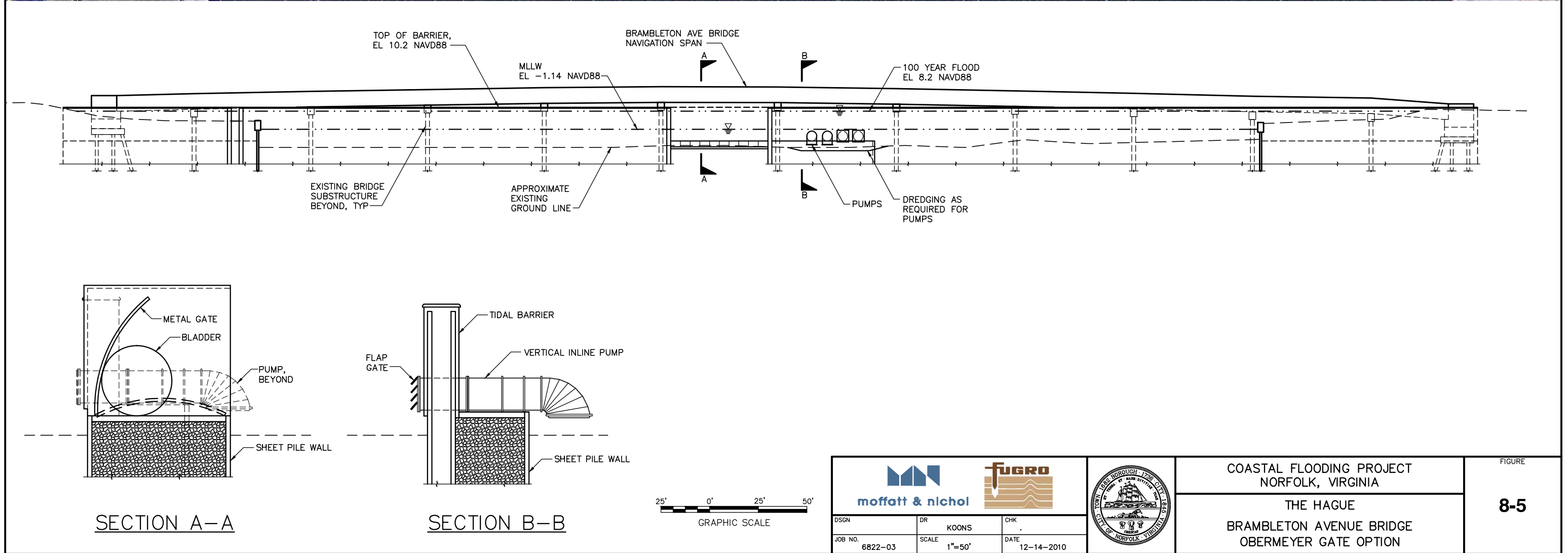
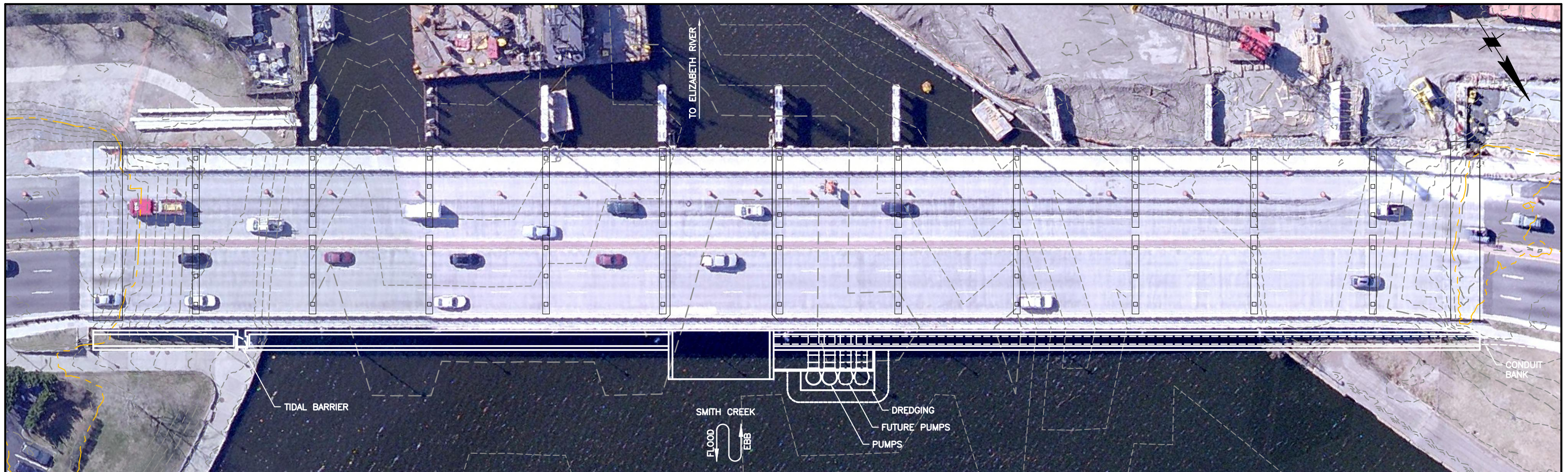


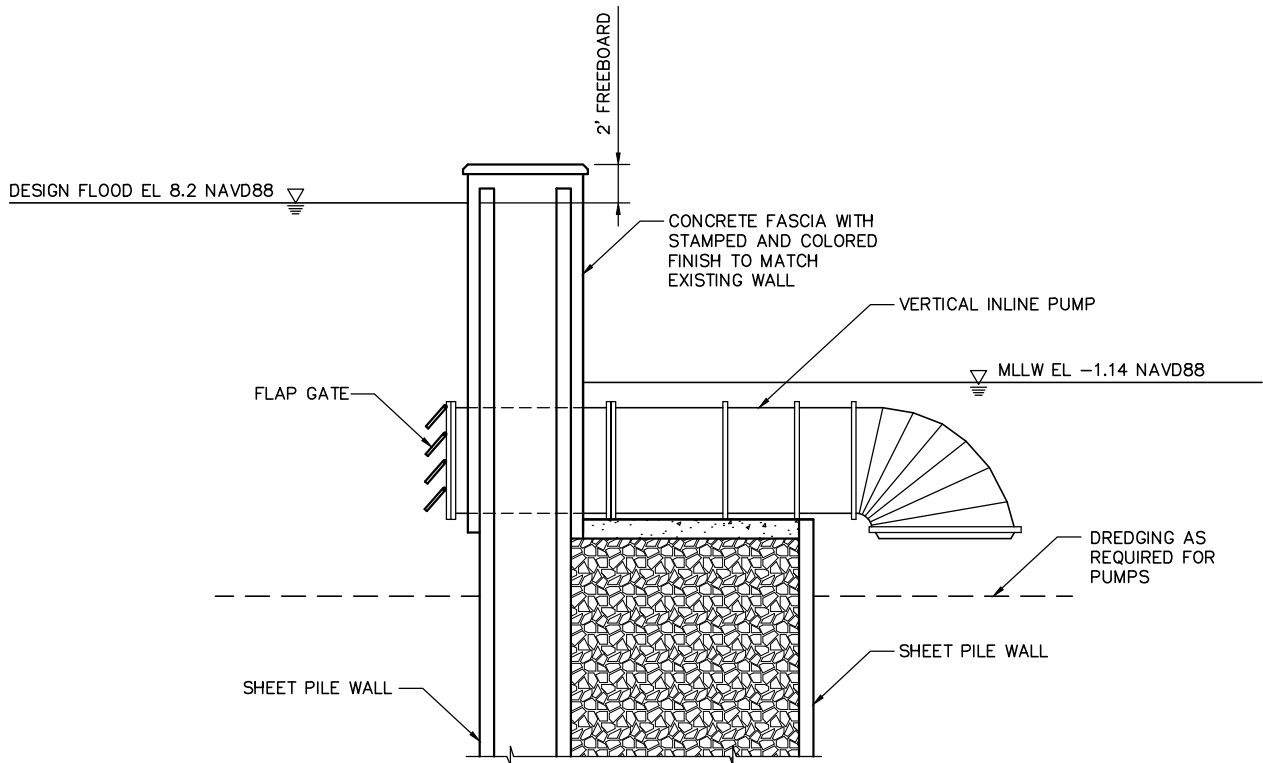
WALL SECTION AT PUMPS



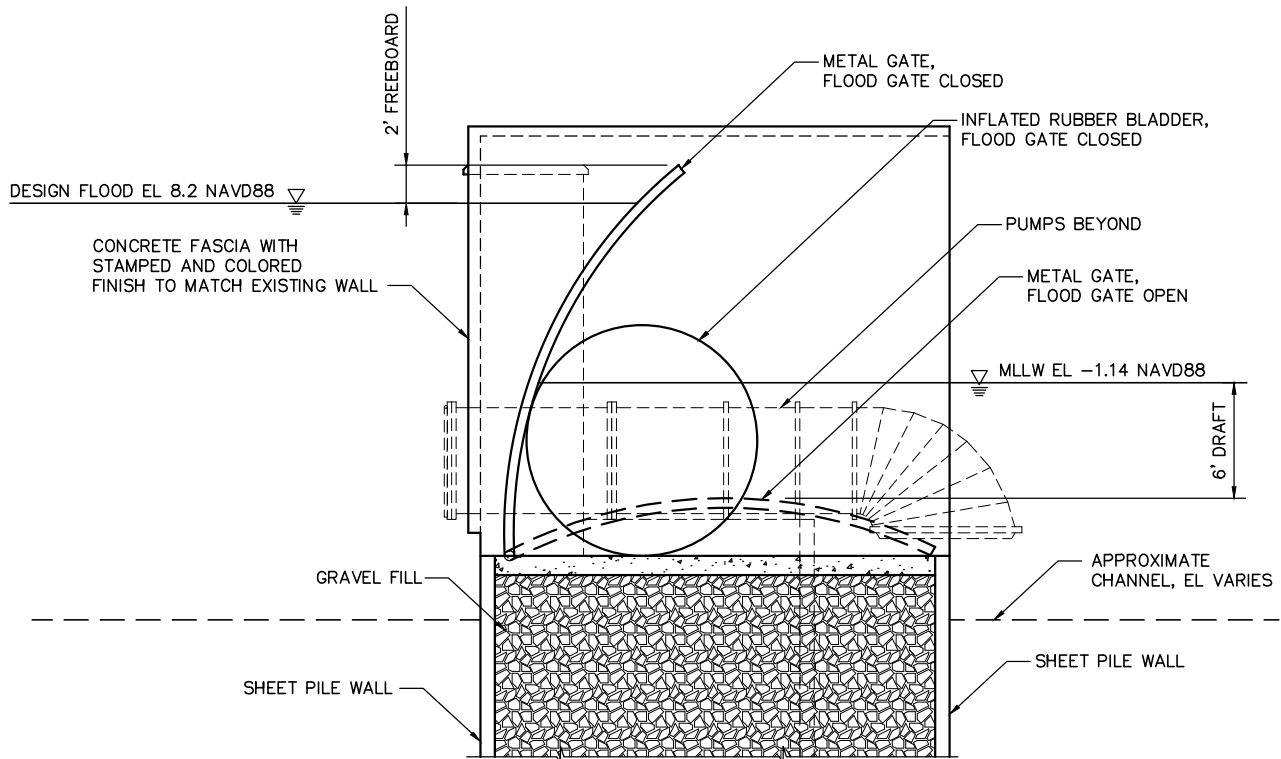
SECTION AT FLOOD STEEL GATE

 				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	FIGURE 8-4
DSGN	DR	CHK		THE HAGUE	
JOB NO. 6822-03	SCALE 1"=10'	DATE 12-14-2010		BRAMBLETON AVENUE BRIDGE STEEL GATE OPTION - SECTIONS	



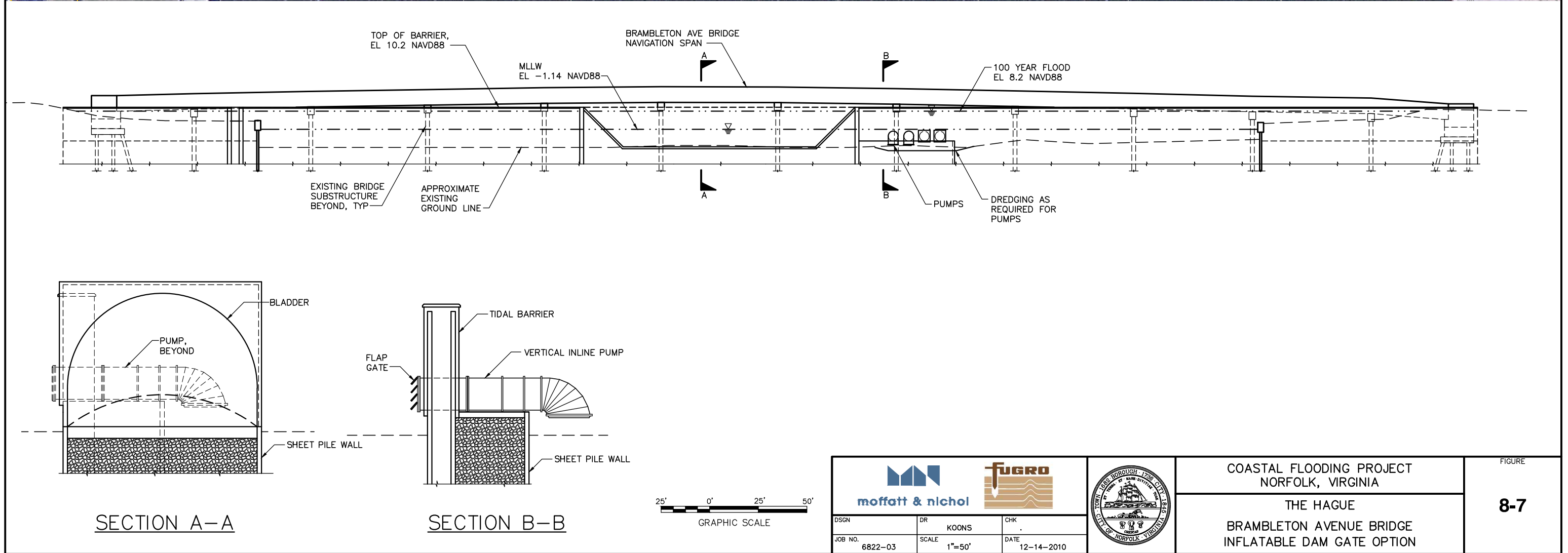
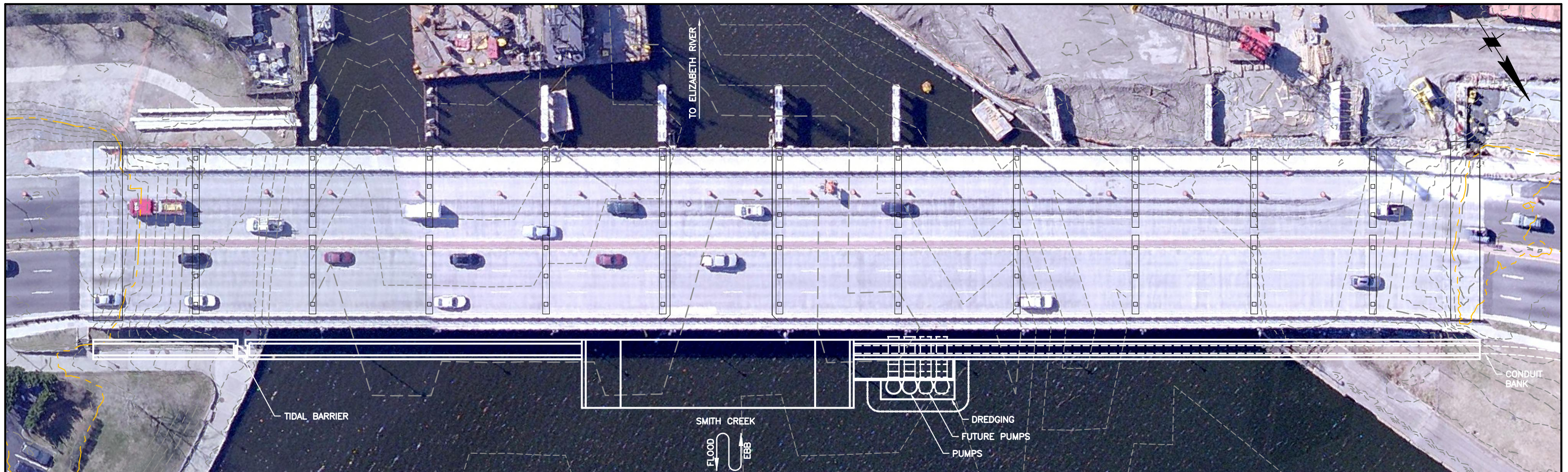


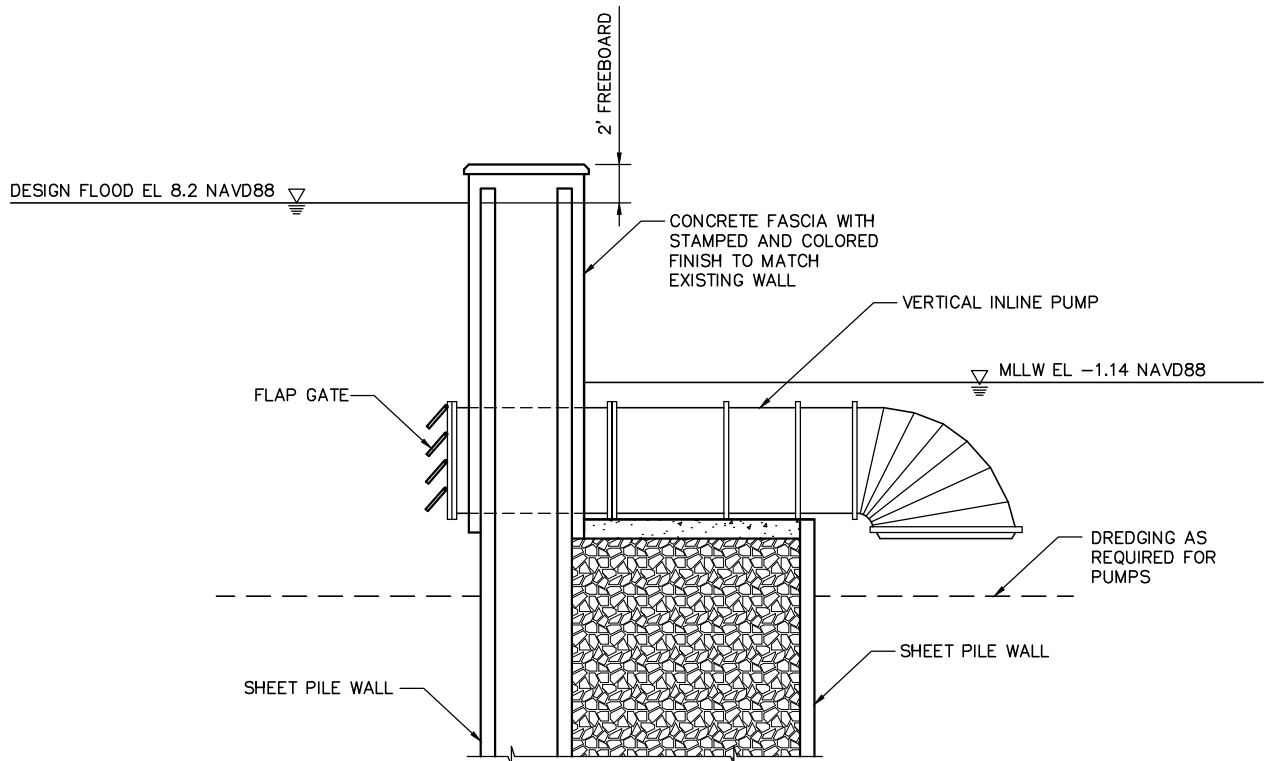
WALL SECTION AT PUMPS



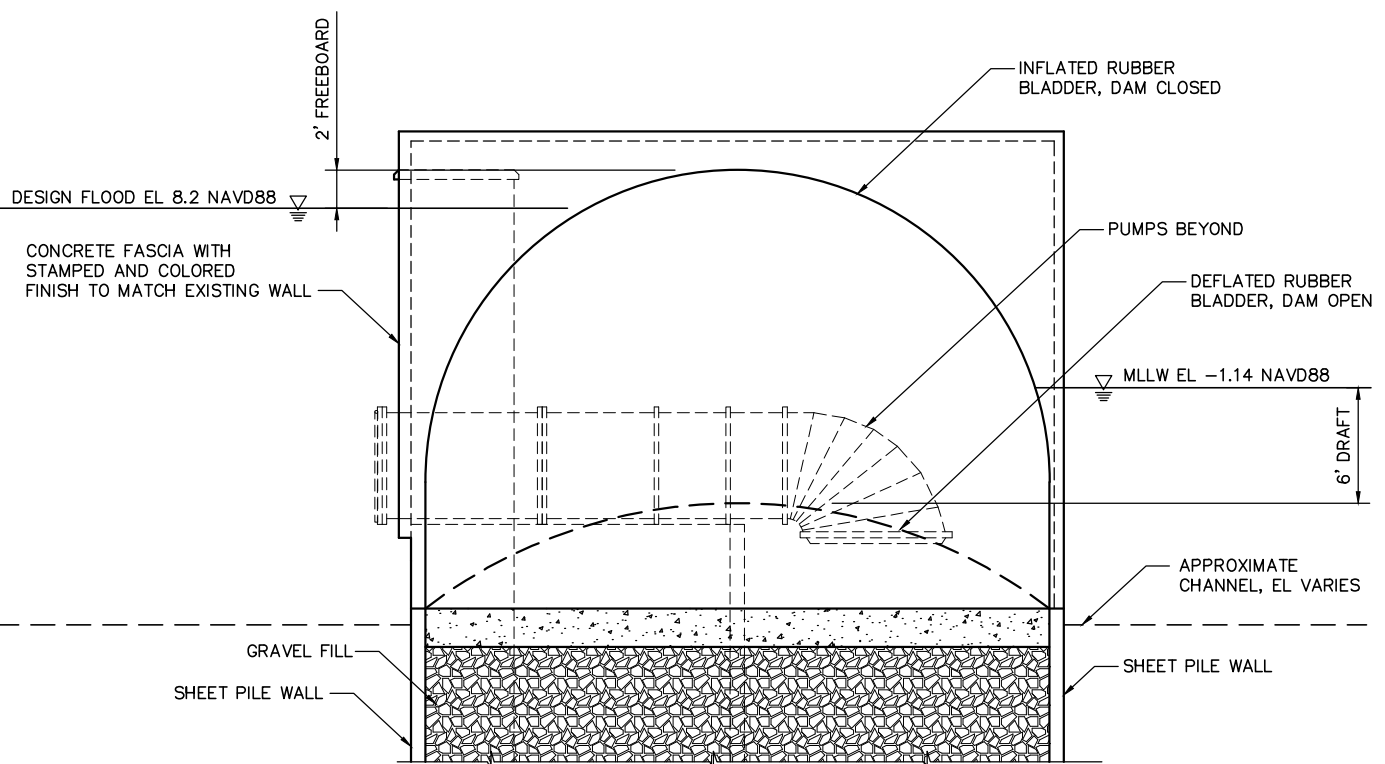
SECTION AT OBERMEYER GATE

 				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	FIGURE 8-6
DSGN	DR KOONS	CHK		THE HAGUE	
JOB NO. 6822-03	SCALE 1"=10'	DATE 12-14-2010		BRAMBLETON AVENUE BRIDGE OBERMEYER GATE OPTION - SECTIONS	





WALL SECTION AT PUMPS



SECTION AT INFLATABLE BLADDER



COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

THE HAGUE

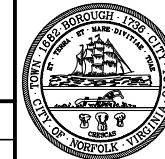
BRAMBLETON AVENUE BRIDGE
INFLATABLE DAM OPTION - SECTIONS

FIGURE

8-8

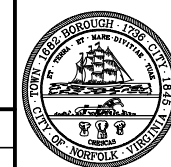
DSGN	DR	CHK
	KOONS	.
JOB NO.	SCALE	DATE
6822-03	1"=10'	12-14-2010



 moffatt & nichol					COASTAL FLOODING PROJECT NORFOLK, VIRGINIA		FIGURE 8-9
DSGN		DR	CHK		THE HAGUE SUBSTATION LOCATION		
JOB NO. 6822-03		SCALE 1"=50'	DATE 12-14-2010				



 BULKHEAD
 BERM

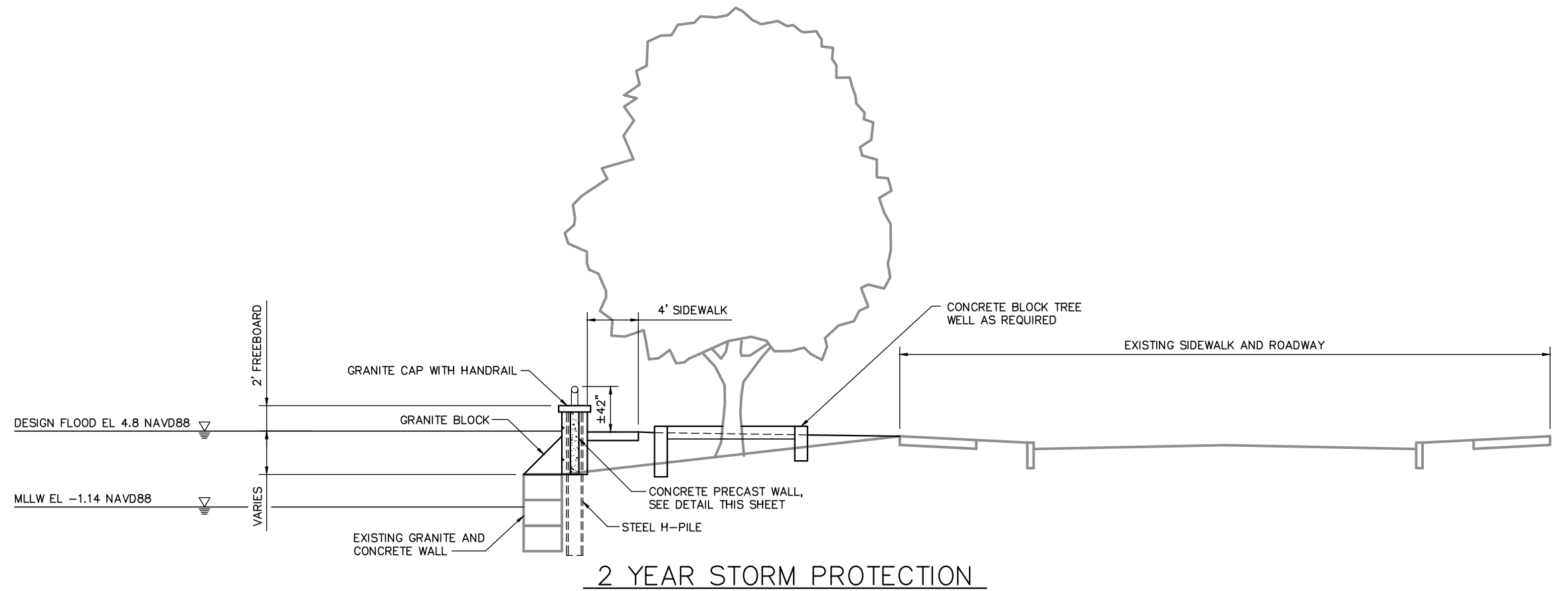
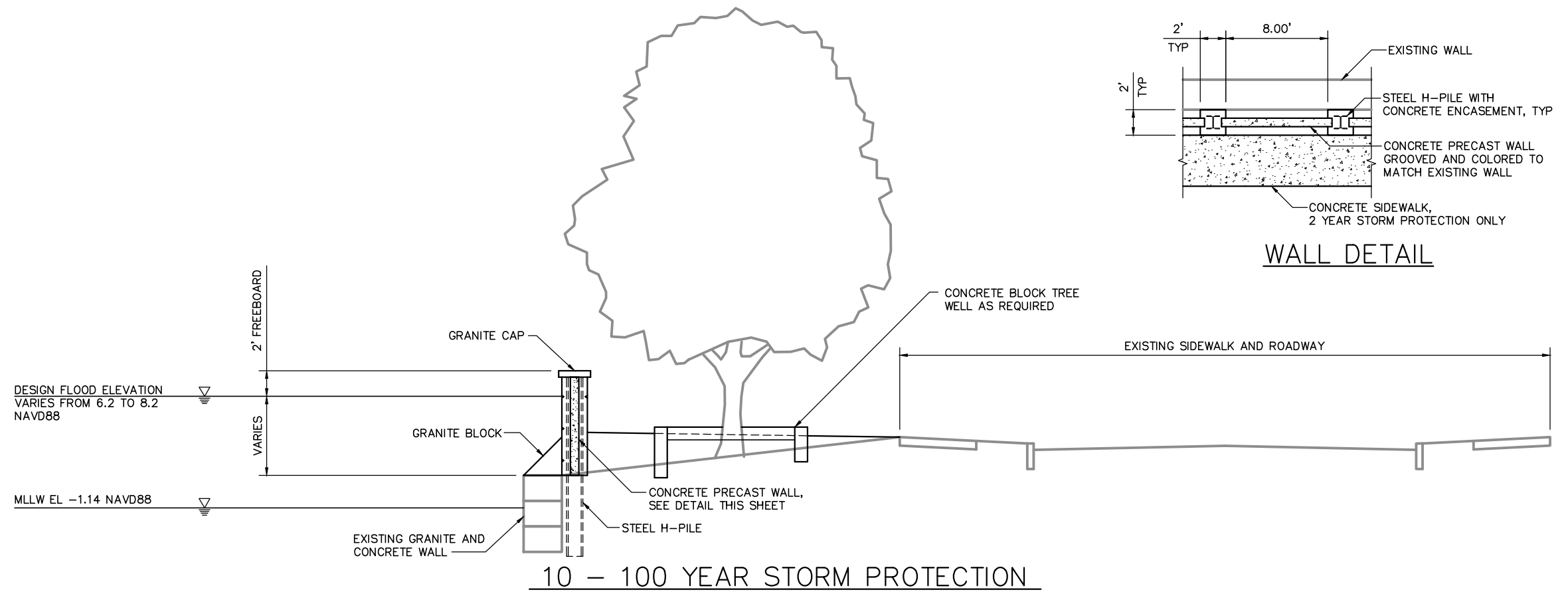


DSGN	DR	CHK
JOB NO. 6822-03	SCALE 1"=500'	DATE 12-14-2010

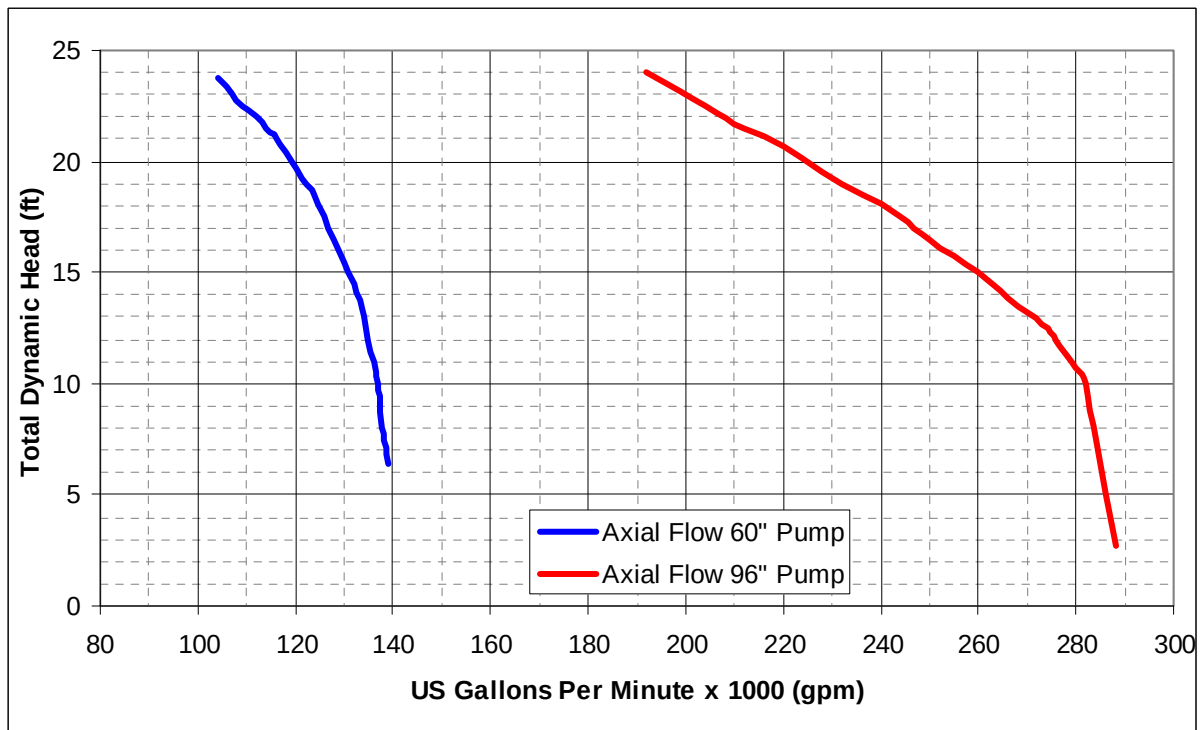
COASTAL FLOODING PROJECT
 NORFOLK, VIRGINIA
 THE HAGUE
 BULKHEAD WALL AND EARTHEN BERM

FIGURE

8-10



 					COASTAL FLOODING PROJECT NORFOLK, VIRGINIA THE HAGUE BULKHEAD WALL TYPICAL SECTIONS	FIGURE 8-11
DSGN	DR	CHK	JOB NO.	SCALE	DATE	
	KOONS	.	6822-03	1"=10'	12-14-2010	



PUMP CURVES FOR SWMM MODELS
City-wide Coastal Flooding Study
Norfolk, Virginia

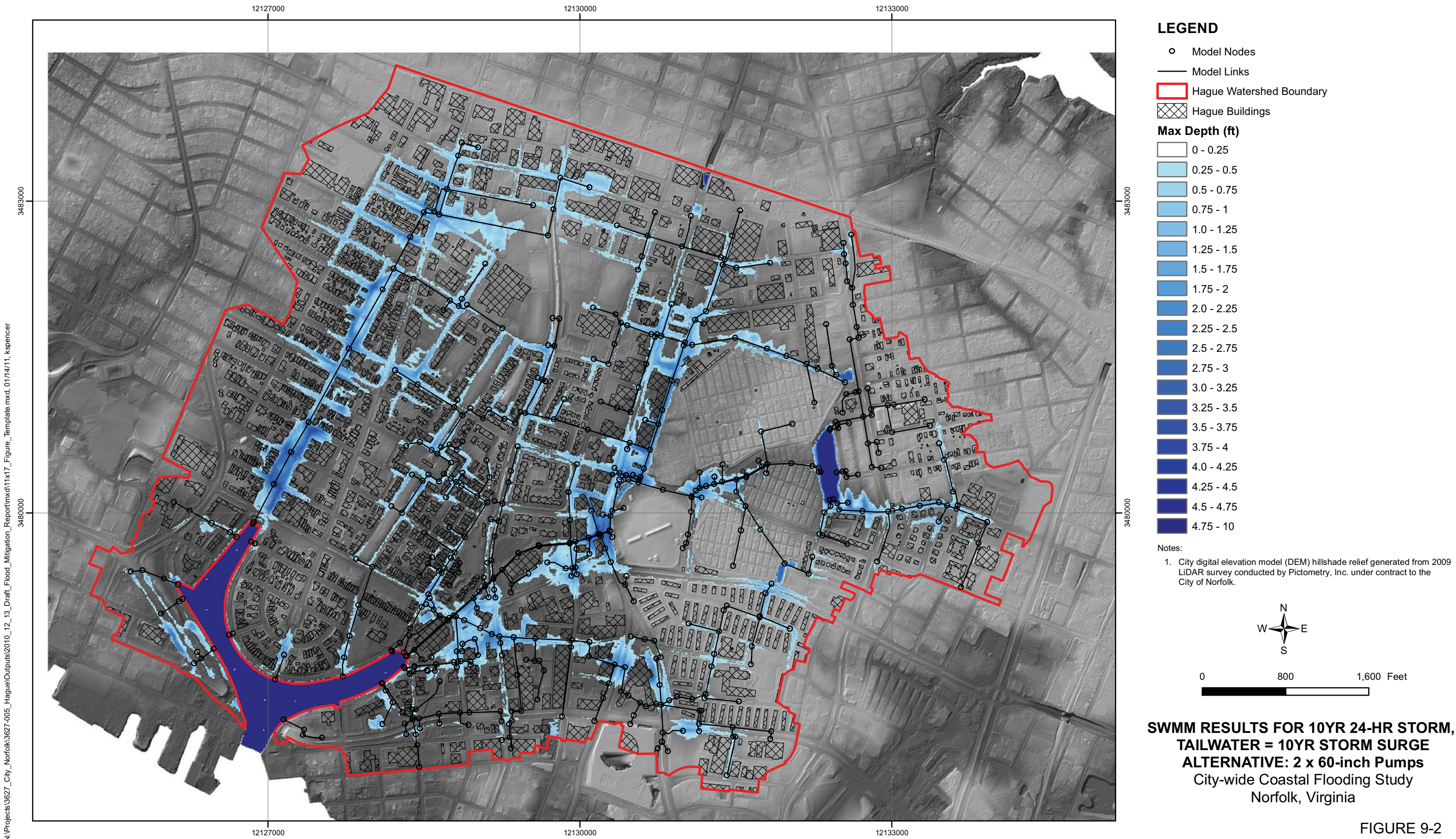


FIGURE 9-2



FIGURE 9-3



FIGURE 9-4

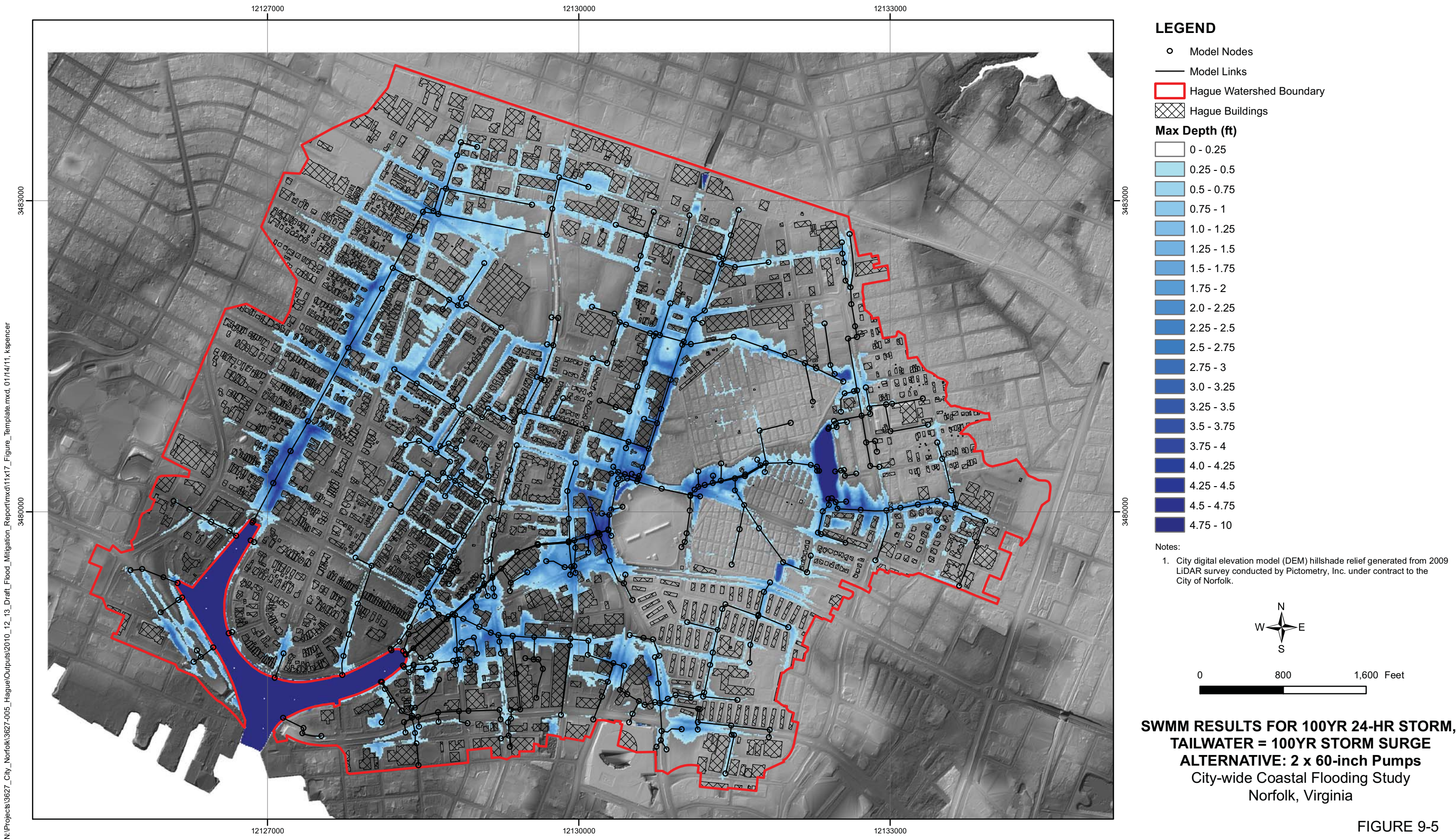


FIGURE 9-5

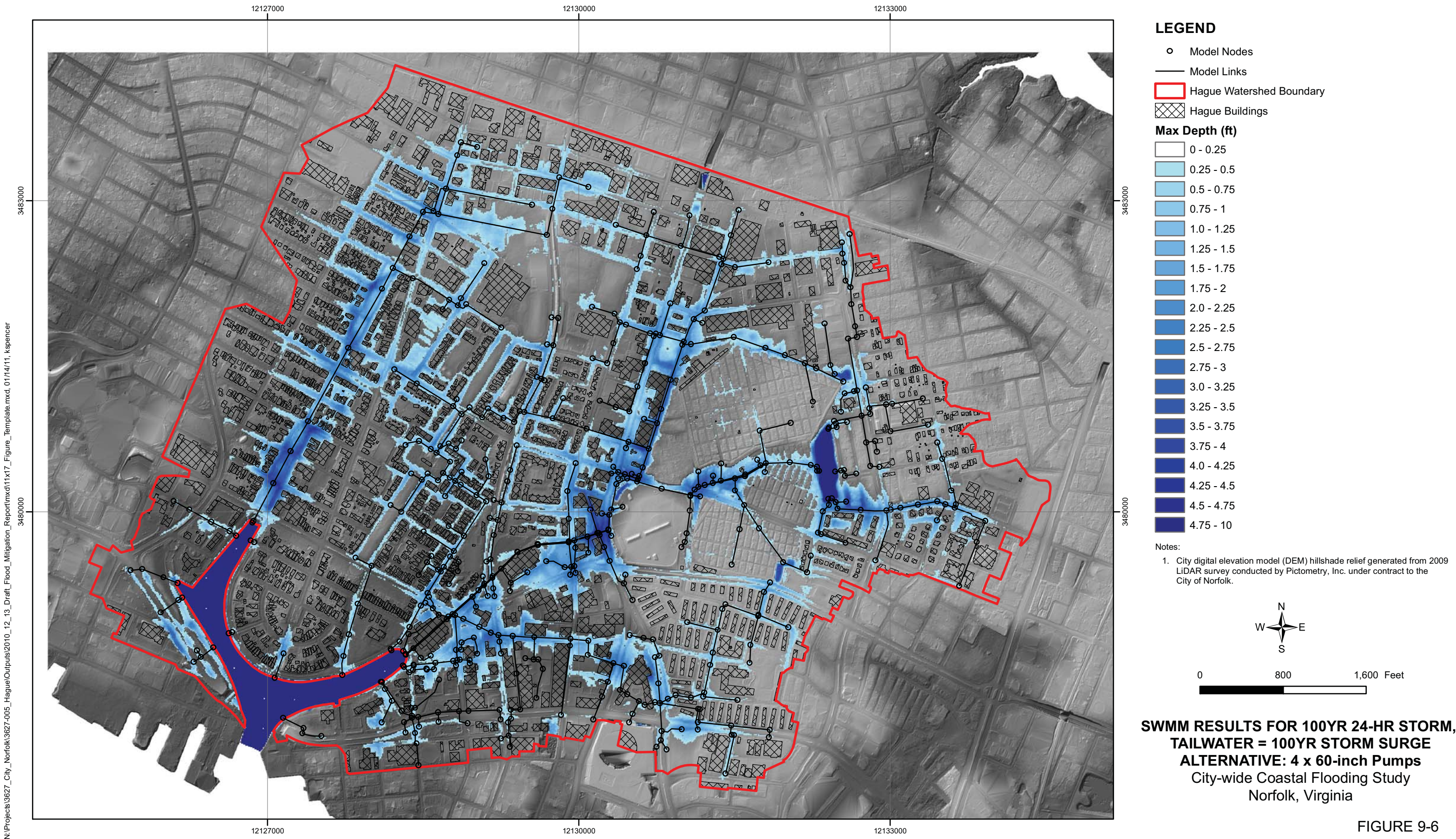


FIGURE 9-6



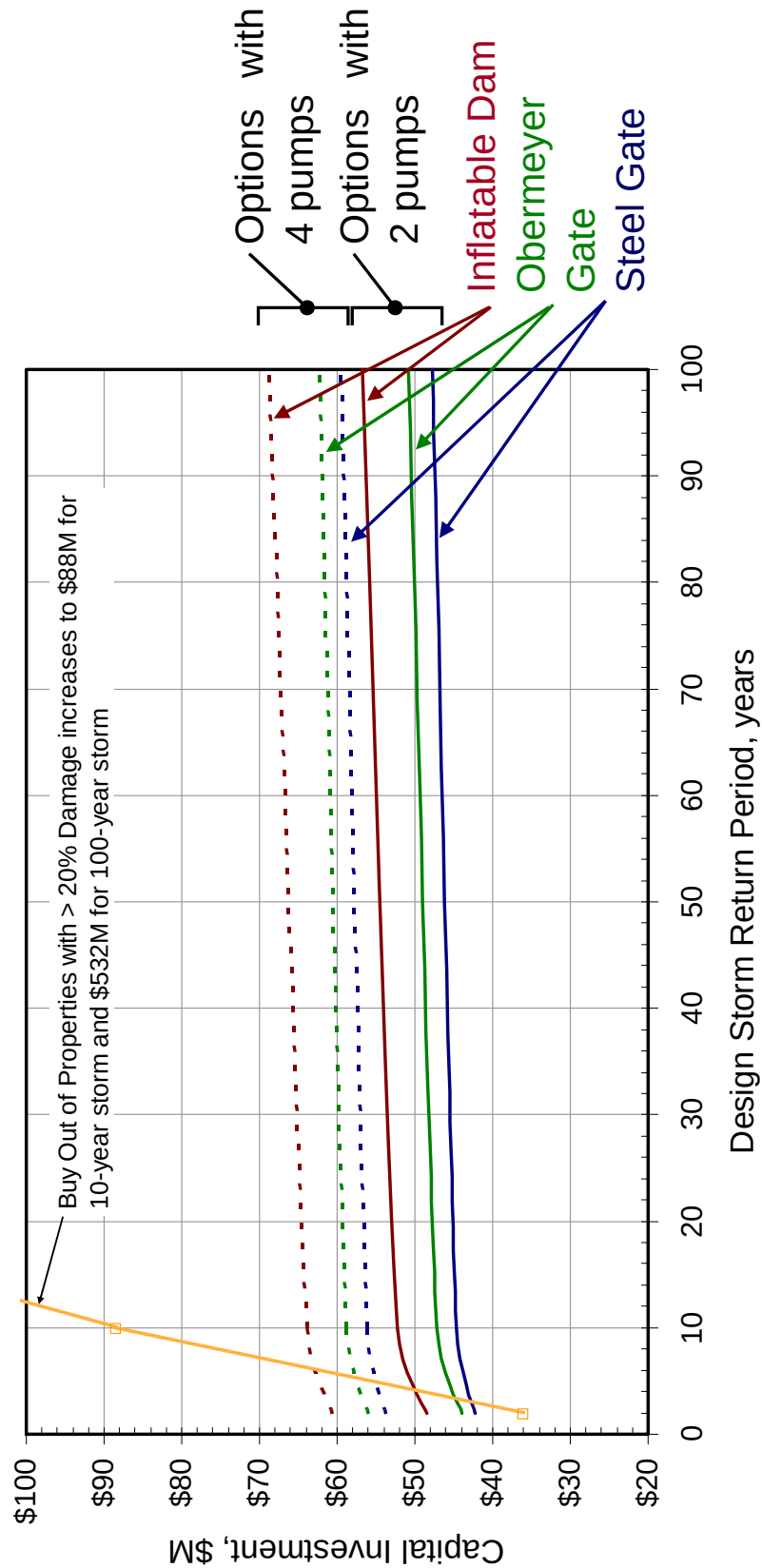
FIGURE 9-7



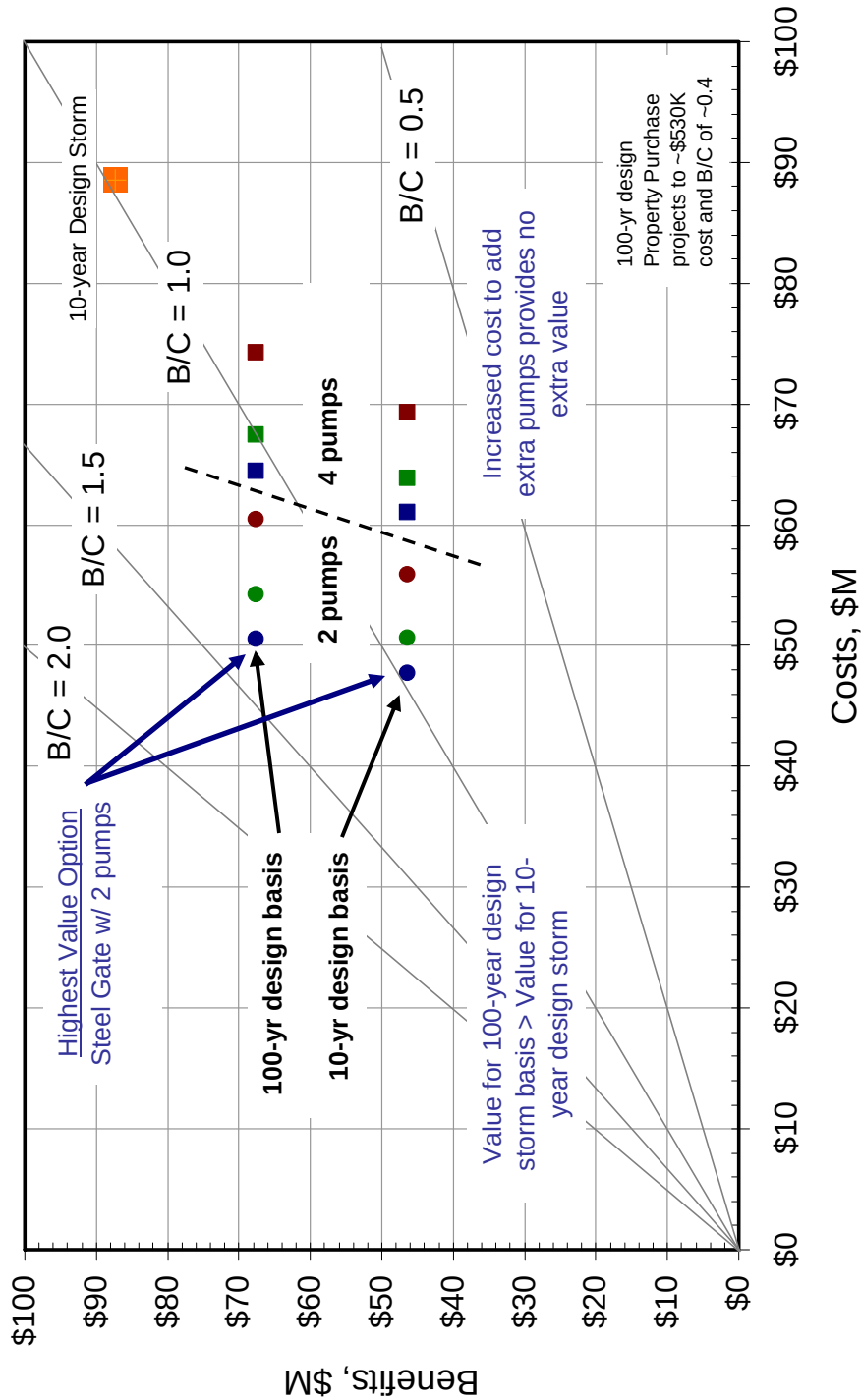
FIGURE 9-8



FIGURE 9-9



ESTIMATED PROJECT COSTS FOR FLOOD MITIGATION OPTIONS
City-wide Coastal Flooding Study
Norfolk, Virginia



BENEFIT/COST EVALUATION
City-wide Coastal Flooding Study
Norfolk, Virginia



LEGEND

- Hague Watershed Boundary
- Historic Buildings
- Historic Districts
- Parcels



0 500 1,000
Feet

HISTORIC DISTRICTS

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 14-1