FUGRO ATLANTIC

FLOOD MITIGATION ALTERNATIVES EVALUATION
THE HAGUE WATERSHED

City of Norfolk
City-wide Coastal Flooding Contract
Work Order No. 5

Prepared for:
CITY OF NORFOLK
DEPARTMENT OF PUBLIC WORKS

April 2011
Fugro Project No. 3627.005

FUGRO ATLANTIC

350 World Trade Center
Norfolk, Virginia 23510
Tel: (757) 625-3350

April 22, 2011
Project No. 3627.005

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

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

Subject: Flood Mitigation Alternatives Evaluation - The Hague Watershed, City of Norfolk, City-wide Coastal Flooding Project, Work Order No. 5

Dear Mr. White:

Enclosed is Fugro Atlantic's report documenting our flood mitigation alternatives evaluation for the Hague Watershed. This study and report were authorized by Work Order #5, dated July 9, 2010 of the City-wide Coastal Flooding contract (City of Norfolk Contract 1125). This report provides our technical assessment of flood mitigation options in The Hague. 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,

Kevin R. Smith
Senior Engineering Geologist/Project Manager

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

Enclosure:

Copies Submitted: (#)

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EXECUTIVE SUMMARY

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.

Our preliminary evaluations of coastal flooding susceptibility within the City and its implications for the design of future flood defense improvements were described in the *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.

The next phase of the City-Wide Coastal Flooding Contract begins 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 results of and recommendations developed during that evaluation are described in this current report.

COMMENTARY

When evaluating and using the information presented herein, 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 and natural processes 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.

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.

EVALUATION OF 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 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 cause 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. 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 gate can mitigate coastal flooding including much of the worst effects of extreme extra-tidal events from hurricanes and nor'easters. Because The Hague 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: 1st construct 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.

1.0 INTRODUCTION AND BACKGROUND

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.

CITY-WIDE COASTAL FLOODING PROGRAM

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).

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 for the Hague watershed in the City. The location of this drainage basin within the City is shown on Figure 1-1. Figure 1-2 shows the extent of the drainage basin and Figure 1-3 shows the area at the outlet of the basin.

AUTHORIZATION

Work Order No. 5 for the City-Wide Coastal Flooding Study was issued by the City on July 9, 2010. The intent of this current work order is to provide an Alternatives Evaluation 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 three levels of flood protection, specified as follows:

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

The "practical" design event is understood to be something that will be based on iterative, qualitative analyses that considers: low points in the project area, options to change highest potential elevation of protection, how that elevation compares to different return periods (based on current sea level), and how potential future sea level rise will change the level of protection.

PROJECT TEAM

The City-Wide Coastal Flooding contract studies and this report have been prepared by the Fugro Atlantic team that includes:

- Mr. Kevin Smith, the senior engineering geologist and GIS services manager with Fugro Atlantic is the Project Manager for the City-wide Coastal Flooding Contract,
- Mr. Thomas McNeilan, the general manager of Fugro Atlantic is Fugro's principal-in-charge and lead engineer for the contract,
- Mr. Kyle Spencer GIS analysts on Fugro Atlantic's staff has developed the GIS-based model and prepared the mapping used in the study,
- Mr. Johnny Martin, senior coastal/hydraulic engineer with Moffatt & Nichol has supervised Moffatt & Nichol's hydrological analyses efforts,
- Mr. Christopher Potter, coastal/hydraulic engineer with Moffatt & Nichol has assisted Mr. Martin,
- Dr. Mohamed Mekkawy, geotechnical engineer, of Fugro and Mr. Josh Hill, civil engineer with Moffatt & Nichol conducted the engineering evaluations for the various alternatives, and provided the opinions of probable cost for the various alternatives as reported herein.

Tom McNeilan and Johnny Martin are the primary authors of this report.

2.0 WATER LEVELS AND POTENTIAL FUTURE SEA LEVEL RISE IN THE CITY

WATER LEVEL ELEVATIONS AND RETURN PERIODS

The long-term data set provided by the Sewells Point tide gauge was analyzed using extremal 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 2-1 and the water levels for various return periods are listed in the following table.

Table 2-1. 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

WATER LEVELS WITHIN THE CITY

The city-wide coastal flooding contract included the installation of five tide gauges within various watersheds in May 2009. These gauges have provided quantitative data to measure and predict tides throughout the City relative to those at Sewells Point. Sewells Point, which has the longest history of tidal measurements, is the reference location used to communicate predicted tide levels to the City, the media, and to the population in general. 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 data include the November 2009 Nor'Ida nor'easter that produced the fourth highest recorded water level at the Sewells Point tide gauge, since it was established in 1928.

The tide gauge data measured over the last year are considered to provide a unique picture of the propagation of flood waters from Chesapeake Bay and the main stems of the Elizabeth River into the various water bodies within the City. The data set is unique in that no comparable data have been previously recorded within the Hampton Roads region. The data documents water levels at the five gauge locations that 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 provide an appropriate basis for estimating the difference between the water level in the Hague compared to that at Sewells Point. The statistical analyses of the measurements at this gauge relative to those at Sewells Point indicated that the peak and low water levels at this location are on average 0.5-foot above those at Sewells Point. In addition, 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 down town pump station tide gauge.

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 is recommended for the Hague watershed to account for temporal, local effects. Section five of this report discusses in detail the tailwater elevations used in this report.

FUTURE SEA LEVEL RISE CONSIDERATIONS

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. While the prediction of future sea level rise is a contentious subject of considerable scientific debate, 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 sea level rise at Sewells Point (relative sea level rise is considered to be the combined effects of sea level rise and subsidence) is 1.46 feet/century. To evaluate how a continuation of that rate of sea level rise will affect flooding in the City, we:

- Assumed a future 0.5-foot rise in sea level (if the rate of 1.46 feet/century continues this will equal the sea level in 35 years; i.e. 2045) and
- Recomputed the return period associated with various tide elevations at Sewells Point.

The return periods associated with different tide elevations at Sewells Point are summarized in the Table 2-2.

Table 2-2. 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

Examination of the data in the proceeding table implies that continuation of the current rate of sea level rise will increase the probability of seeing a particular flood water elevation by about 50% by 2045. This implies that the size of storms that can produce a specific flood water level will be less in the future than at the present. Figure 2-2 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.

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.

3.0 LOCATION AND WATERSHED (DRAINAGE BASIN) DESCRIPTION

WATERSHED DESCRIPTION

The Hague watershed is in the southwest portion of the City (Figure 1-1). 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).

Topography

The topography of the Hague watershed is generally flat and below elevation (El.) 12 feet NAVD88. Figure 3-1 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 3-1. A statistical summary of the ground surface elevation is provided on Figure 3-2 and Table 3-1. Approximately 30 percent of the study area lies below El. 8 feet NAVD88. The regional ground surface slopes gently to the southwest.

The watershed is bifurcated by to 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 3-1. 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%
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%

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 3-2. Figure 3-3 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 3-2. Hague Watershed Land Use Classifications

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

Receiving Water Body

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.

BASIN RIM

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 3-4. The number of locations along the basin rim and the length of the segments below different threshold elevations are summarized as in Table 3-3.

Table 3-3. 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.

4.0 BASIN OUTLET

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 4-1 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.

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 2-4 feet, relative to MLLW datum. That elevation corresponds to El.-4 to -6 feet re: NAVD88.

SUBSURFACE CONDITIONS

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. The boring log data were input into a GIS geotechnical database. Applications developed by Fugro were used to characterize the engineering and stratigraphic information in the database. Figures 4-2 and 4-3 present cross sections depicting interpreted subsurface conditions at the basin outlet.

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.

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.

5.0 DESIGN CRITERIA

TAILWATER ELEVATION AND COASTAL FLOODING CONSIDERATIONS

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.

Table 5-1. Tailwater Correction (re: Sewells Point) and Allowance for Sea Level Rise

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). This allowance may be accounted for in later design phases once the overall costs to meet the desired level of protection for current flooding levels are determined. Table 5.2 below details the recurrence interval tailwater elevations at Sewells Point and the design tailwater elevations for the Hague watershed (Fugro, 2010).

Table 5-2. 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	7.0
50	6.6	7.6
100	7.2	8.2

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 5-3 below.

Table 5-3. NOAA Return Frequency Rainfall Depths for Norfolk WSO 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

ELEVATION OF PROTECTION

The work scope definition for the alternatives evaluation includes the consideration of three different level of flood mitigation/defense. Those criteria were defined as follows:

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

10- and 100- Year Return Periods

As noted, the water level elevations at Sewells Point that are associated with the 100- and 10- year return periods are: Elevation +7.2 and +5.2 feet (re: NAVD88 datum). Those water levels at Sewells Point correspond to design water elevations in the Hague watershed equal to elevation +8.2 and +6.2 feet (re: NAVD88 Datum). 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, it was felt that the concept level designs should be completed with the current water levels given the uncertainty associated with accelerated sea level rise. Adjustments to wall heights and extents could always be made later and would be studied in the opinion of probable cost section of the report.

Practical Design Event

The "practical" design event in the return period or elevation criteria was defined to recognize that in some locations it might not be practical or cost-effective to provide flood mitigation/defense that met certain criteria for return period. Rather the "practical" design event was to be evaluated in the context of certain realities of the project locations, such as: low points in the project area, options to change highest potential elevation of protection, how that elevation compares to different return periods (based on current sea level), and how potential future sea level rise will change the level of protection. This was recognized to require iterative, qualitative analyses.

The protection associated with an elevation +8.2-ft (re: NAVD88 datum) 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 (re: NAVD88 datum) 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 foot or two for sea level rise quite easily. For the purposes of this study, it was determined that the designs of the floodwalls themselves would be designed with a 2 ft freeboard which would still provide 1 foot of freeboard with a sea level rise of 1 foot. This factor should be studied in more detail and optimized in final design. Based on the watershed basin rim elevations, it was also felt that the current water level of 8.2 ft could be designed for with requiring significant floodwall/levee systems to be installed all around the watershed perimeter.

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. Figure 5-1 illustrates the tailwater phenomena and the implications it has on storm water drainage systems. Discussions led to the conclusion that all rainfall conditions should be considered with a mean higher high water (MHHW) tide as well as coincident tailwater and rainfall events (i.e., 1-yr rainfall/1-yr storm surge, etc.) . These scenarios would help "bracket" the expected range of conditions that the proposed alternatives would have to ultimately face. The following combinations of tailwater elevation and precipitation, as shown in Table 5-4, have been considered in the alternative analyses presented herein.

Table 5-4. Design Combinations of Tailwater and Precipitation

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

6.0 EXISTING SYSTEM HYDROLOGIC/HYDRAULIC EVALUATION

SELECTION OF MODEL

XP-SWMM is a software package that utilizes the EPA Stormwater Management Model (SWMM) one-dimensional (1D) analytical engine for running rainfall-runoff simulations for single event or long-term simulations of runoff quantity and quality. 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 (2D) 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 2D in nature and would be difficult to appropriately represent using a 1D model. A powerful feature of TUFLOW is its ability to dynamically link to the 1D network of the SWMM engine. In XP-SWMM, the user sets up a model as a combination of 1D storm-drain network domains linked to 2D domains, i.e. the 2D and 1D domains are linked to form one model.

DEVELOPMENT OF MODEL INPUTS

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

Q (cfs) = coefficient × 2D cell depth (ft) ^ 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.

Rainfall Data

The precipitation frequency depths for the project were based on the published NOAA Atlas 14 values for the Norfolk WSO Airport (NOAA, 2004). The simulations were calculated using the SCS Type-II 24-hour rainfall distribution (USDA, 1986).

Subcatchments

The Hague drainage area was divided into 360 smaller subcatchments based the topographic Light Detection And Ranging (LiDAR) data collected by the City of Norfolk in 2009. Figure 6-1 shows the division of the drainage area into 18 larger catchment areas. Each subcatchment was analyzed to determine input parameters for 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.

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.

Conduits

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

Outfalls

The Hague inlet was included in the model as part of the 2D hydrodynamic grid. Therefore, the outfalls that drain water from the Hague into the Hague cove were set up as 1D nodes with their inverts linked to the 2D 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 2D model grid at the Brambleton Avenue Bridge, where the Hague cove outlets to the Elizabeth River. The boundary condition water surface elevation was based the recurrence interval tailwater elevations for the Downtown Pump Station, derived from the NOAA Station 8638610 at Sewells Point (Fugro, 2010) with the additional 1-ft increase due to basin and wind/cove effects.

Buildings

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

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 SWMM model for the Hague.

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.

EXISTING SYSTEM FLOODING DURING VARIOUS STORM EVENTS

Storm events of various return intervals were run in the 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 yr 24-hr return intervals from Norfolk International Airport precipitation frequency estimates, which were downloaded from NOAA. For the purpose of this report, only results for the 10yr and 100yr design storms will be presented. Results from the other design storms are presented in Appendix B.

MHHW Tailwater

The five design storms were simulated in the existing condition 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 10yr and 100yr design storms are presented in Figure 6-2 and Figure 6-3, respectively**.** Model results for each design storm are tabulated in Table 6-2 below.

Storm Surge Tailwater

The five design storms also were simulated in the existing condition SWMM model using the corresponding return period storm surge as the boundary condition water level. The recurrence interval storm surge levels used in the modeling were presented in Table 5-4 of the Design Criteria. Model results for the 10yr design storm with 10yr storm surge and the 100yr design storms with 100yr storm surge are presented in Figure 6-4 and Figure 6-5, respectively. Model results for each design storm scenario are tabulated in Table 6-1. For reference, the extent of flooding for the 10yr and 100yr storm surges without any coincident rainfall are presented in Figure 6-6 and Figure 6-7, 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 longer return periods (larger storms). Nonetheless, it is also apparent from the existing conditions modeling that the interior drainage system also is a serious constraint with the system at best able to carry a ~2yr, 24hr rainfall with the tailwater at MHHW.

Table 6-1. Existing Condition SWMM 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 Storm, MHHW Tide	129.7	66.8	94.0	0.71	1.2
2yr Storm, MHHW Tide	172.4	86.1	116.7	0.74	1.4
10yr Storm, MHHW Tide	304.7	148.6	175.5	0.85	2.4
25yr Storm, MHHW Tide	396.5	192.2	210.8	0.91	3.0
50yr Storm, MHHW Tide	479.8	230.1	236.6	0.97	3.5
100yr Storm, MHHW Tide	569.5	271.1	262.1	1.03	4.0
1yr Storm, 1yr Storm Surge	129.9	92.1	112.7	0.82	2.4
2yr Storm, 2yr Storm Surge	172.2	127.1	141.0	0.90	3.4
10yr Storm, 10yr Storm Surge	304.7	263.8	224.2	1.18	6.9
25yr Storm, 25yr Storm Surge	396.6	388.7	281.6	1.38	9.5
50yr Storm, 50yr Storm Surge	477.4	507.1	329.7	1.54	11.0
100yr Storm, 100yr Storm Surge	566.4	656.9	380.2	1.73	13.1

7.0 EXISTING CONDITION ESTIMATES OF DAMAGE COSTS

METHODOLOGY

Flood damage estimates were assessed for a range of flooding scenarios under existing conditions. However, these analyses would also be completed for many of the flood mitigation alternatives to aid in their assessment. The analysis focuses on direct damage to structures and contents of private and public buildings. The primary focus of this analysis is to estimate the economic damages associated with future flood events in the Hague watershed under existing conditions and to provide a basis for performing a benefit-cost comparison of flood mitigation alternatives. We note 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.

In general, 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. The function, referred to as a 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). This study used a building inventory file developed by the project team with assistance from the City, output flooding results from the 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. For the Hague watershed, damage assessments were conducted for all scenarios evaluated in XP-SWMM. The results of the damage assessment estimates for existing conditions can be found later in this section and the damage assessment estimates for flood mitigation alternatives are included in the benefit-cost summary tables which are discussed in Section 9.0 and included in Appendix D. A description of the procedure is provided in the following sections.

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 used in the Hague watershed.

After building footprints were updated, the buildings were classified by type. The building type was used to determine which depth damage function (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 the following table.

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			

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.

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.

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 in the "Catalog of Residential Depth-Damage Functions" (USACE 1992), USACE's EGM 01-03 (USACE, 2000) and EGM 04-01 (USACE, 2003) 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.

Damage Assessment Estimates

For this study, a GIS-based damage assessment tool was developed. The tool reads the flood water body outputs from the modeling runs described in a previous section of this report 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 damage assessments for existing conditions are provided in Table 7-2. The distribution of estimated damages for 10yr Storm with a MHHW tailwater and a 100yr Storm with a MHHW tailwater are presented in Figures 7-2 and 7-3 respectively. The distribution of estimated damages for 10yr Storm with 10yr Storm Surge and a 100yr Storm with Storm Surge are presented in Figures 7-4 and 7-5. The damage assessments for existing conditions are provided in Table 7-2.

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 Storm, MHHW Tide	148	10.3 (3.7)	6.3 (2.9)	16.6 (6.7)
2yr Storm, MHHW Tide	184	12.9 (4.6)	7.9 (3.7)	20.8 (8.3)
10yr Storm, MHHW Tide	334	22.3 (7.5)	13.8 (6.0)	36.2 (13.5)
25yr Storm, MHHW Tide	490	24.4 (9.0)	15.7 (7.0)	40.1 (16.1)
50yr Storm, MHHW Tide	623	28.9 (10.0)	18.9 (7.8)	47.9 (17.8)
100yr Storm, MHHW Tide	757	33.5 (11.0)	22.2 (8.5)	55.7 (19.6)
1yr Storm, 1yr Storm Surge	150	10.6 (3.9)	6.5 (3.1)	17.2 (7.0)
2yr Storm, 2yr Storm Surge	185	13.5 (4.9)	8.3 (3.9)	21.9 (8.9)
10yr Storm, 10yr Storm Surge	336	25.6 (8.7)	15.8 (6.8)	41.5 (15.5)
25yr Storm, 25yr Storm Surge	493	32.4 (11.4)	20.5 (8.7)	53.0 (20.2)
50yr Storm, 50yr Storm Surge	625	43.5 (13.3)	27.6 (10.1)	71.1 (23.5)
100yr Storm, 100yr Storm 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 PROJECT DEFINITION OR DEVELOPMENT OF ALTERNATIVES

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.

When evaluating and developing flood mitigation/defense projects in the City, 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.

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 (and their acceptability, or not), 4) an evaluation of alternatives, and 5) the development and implementation of a mitigation and risk management plans.

The flood hazard and risk 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, 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.

Often mitigation approaches include more than one of the above strategies. The following lists a number of types of flood mitigation elements.

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

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

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

FLOOD MITIGATION/DEFENSE OPTIONS ELIMINATED

Prior to defining the alternate flood mitigation/defense options for evaluation it was possible to eliminate some approaches due to either their technical feasibility or other intrinsic factors associated with the approach. Table 8-1 shows 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

Due to Technical Feasibility

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 negligible open areas 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 the coastal strip.
- Setbacks and Buffers - The area is too densely developed and there is negligible open areas for consideration of either setbacks or buffers.

CONCEPTS 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 three flood mitigation 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 are presented below and were evaluated under the various storm events. These alternatives are grouped into five categories and are presented in Table 8-2. The differentiation between alternatives subscripted Xa, subscripted Xb and subscripted Xc is as follows:

- Alternatives subscripted Xa included a tidal barrier with a steel tide gate
- Alternatives subscripted Xb included a tidal barrier with an Obermeyer gate, and
- Alternatives subscripted Xc included a tidal barrier with an Inflatable dam.

Table 8-2. Hague Alternatives

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

Each alternative was evaluated for: 2 Year, 10 Year, 25 Year, 50 Year and 100 Year storm events. The final wall elevations for the structures were calculated by adding 1.5 additional feet of freeboard to the analyzed storm event elevation. This would provide some protection from wave overtopping and provide the FEMA required 1' of freeboard (FEMA, 2009a). Table 8-3 below provides the analyzed wall elevation in addition to the final wall height for all scenarios.

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

Storm Event	Analyzed Storm Elevation (ft, NAVD88)	Final Height with Freeboard* (ft, NAVD88)
2yr, 2yr	4.8	6.3
10yr, 10yr	6.2	7.7
25yr, 25yr	7.0	8.5
50yr, 50yr	7.6	9.1
100yr, 100yr	8.2	9.7

*Heights for the Steel Gate Bulkhead is 2.3' higher than heights shown

A description of each alternative is provided below. The Opinion of Probable Cost for each alternative and their respective storm events are provided in the "Opinion of Probable Cost" section of the report (Section 10.0). A summary of the typical expected service life is also provided in the "Opinion of Probable Cost" section. 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.

Alternatives 1 through 3 - Tidal Barrier with Tide Gate, Pumps, and Closure Walls and Berms

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

- Tidal barrier structures with a tide gate to protect against inundation from tidal surge
- Pumps to remove rainfall runoff when the tide gate is closed, and
- Closure walls and berms across low lying areas of the basin/watershed's perimeter

Tidal Barrier Structures with Tide Gate

The tidal barrier and tide gate will be constructed on the upstream side of the Brambleton Bridge in the Hague. The overall length of the barrier is approximately 750 LF and will tie into the existing elevations of the surrounding environment. Given the soil conditions within this area of the Hague, the proposed barrier wall will consist of two AZ-14 steel sheetpile walls separated approximately six feet apart and constructed parallel to the bridge. Between these two bulkheads, aggregate base will be used to fill the bulkhead to final wall elevation where a tremie concrete slab will be placed. A decorative fascia wall will be installed on the upstream side of the barrier structure for aesthetics.

The gate assembly which will be located in-line with the existing navigational channel and fender system of the bridge will range in width from 50 linear feet for the steel gate and Obermeyer Gate to 110 linear feet for the inflatable dam. At the gate location, the top of the bulkhead will be located at Elevation -4 (NAVD 88) which will allow small boat traffic to access Smith Creek through this section of the barrier. Tide gate options are provided below and a schematic drawing of these can be found on 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 (3) 60-inch diameter pumps (2 operational & 1 back-up), Alternative 2 scenarios will utilize five (5) 60-inch diameter pumps (4 operational & 1 back-up) and

Alternative 3 scenarios will utilize five (5) 96-inch pumps (4 operational & 1 back-up). For all three alternative scenarios, the intake lines of the pumps will be located upstream of the tide gate and the discharge lines will be mounted to the downstream side of the tidal barrier wall. Flap gates will be installed on the discharge side of the pumps to prevent water infiltration back-into the pump system. The pumps will be powered via a substation with electric; however, emergency back-up generators will be located on-site to allow operation during power outages. Given the aesthetics of the Hague community, all electrical components including the generators will be housed in an aesthetically pleasing 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.

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.

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 ALTERNATIVES ANALYSES

EVALUATION METHODOLOGIES

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 2D model grid and Hague watershed boundary acted as the 2D grid boundary. The outfalls which drain from the Hague were given tide-gates preventing backflow, and each was assigned a fixed 1D 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.

PREDICTED FLOODING WITH MITIGATION DURING VARIOUS STORM EVENTS

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 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 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 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 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 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 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 SWMM Results for Pump vs. Bulkhead Wall Alternatives

Hague Scenario	Change in Max Flood Volume (vs. Existing)		Change in Max Flooded Area (vs. Existing)		Change in Average Max Flood Depth (vs. Existing)		Change in Average Duration of Flooding (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%

FLOOD DAMAGE 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 Damages

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 OPINION OF PROBABLE COSTS - FLOOD MITIGATION OPTIONS

A total of 11 alternatives were evaluated under the various storm events. These alternatives are defined in Section 8 and are presented in Table 8-1.

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 Cost

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 Opinion of Probable Cost breakdown, 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.

Operational & Maintenance (O&M) Costs with Respect to Design Life

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 ALTERNATIVE

BENEFIT - COST (B/C) ANALYSIS RATIO

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).

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 = ½ = 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.

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).

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).

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 foot or so is within 5-15% of the total project cost. Therefore, the 100-yr 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 steel gate, 2 -60" pump option should be selected and designed for the 100-yr event.

In conclusion, based on the analyses completed to date the preferred alternative is the construction of a floodwall, tide gate, a pump station (with 2 - 60" pumps and 1 - 60" spare) and closure walls with a total capital cost of $47.4M. The alternative can be broken down into two phases:

Phase 1 - Construction of Steel Closure & Pump Station ($36M)

Phase 2 - Construction of Floodwall south of Brambleton ($11M)

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

This option will provide protection today for a 100-yr coastal 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 the Hague.

12.0 IMPLICATION OF POTENTIAL FUTURE SEA LEVEL RISE

The analyses results as presented hereto are based 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.

- 1st 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 ___.
- 2nd 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.
- 3rd Convert today's tailwater versus return period for a 1-foot rise in sea level as shown on Figure _d,
- 4th 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
- 5th 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.

13.0 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 cause 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 for 2-60" 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 2 - 60" pumps and 1 - 60" spare) and closure walls with a total capital cost of $47.4M. The preferred alternative has a B/C ratio of 1.34 (economically justified)
- This alternative can be split into two phases with construction of the floodwall, steel gate and pump station first ($36M) followed by the floodwall south of Brambleton ($11M). The floodwall south of Brambleton 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 protection 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 delta costs for building the floodwall higher for sea level rise concerns will 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 design.

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 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: 1st construct 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.

14.0 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.

15.0 REFERENCES

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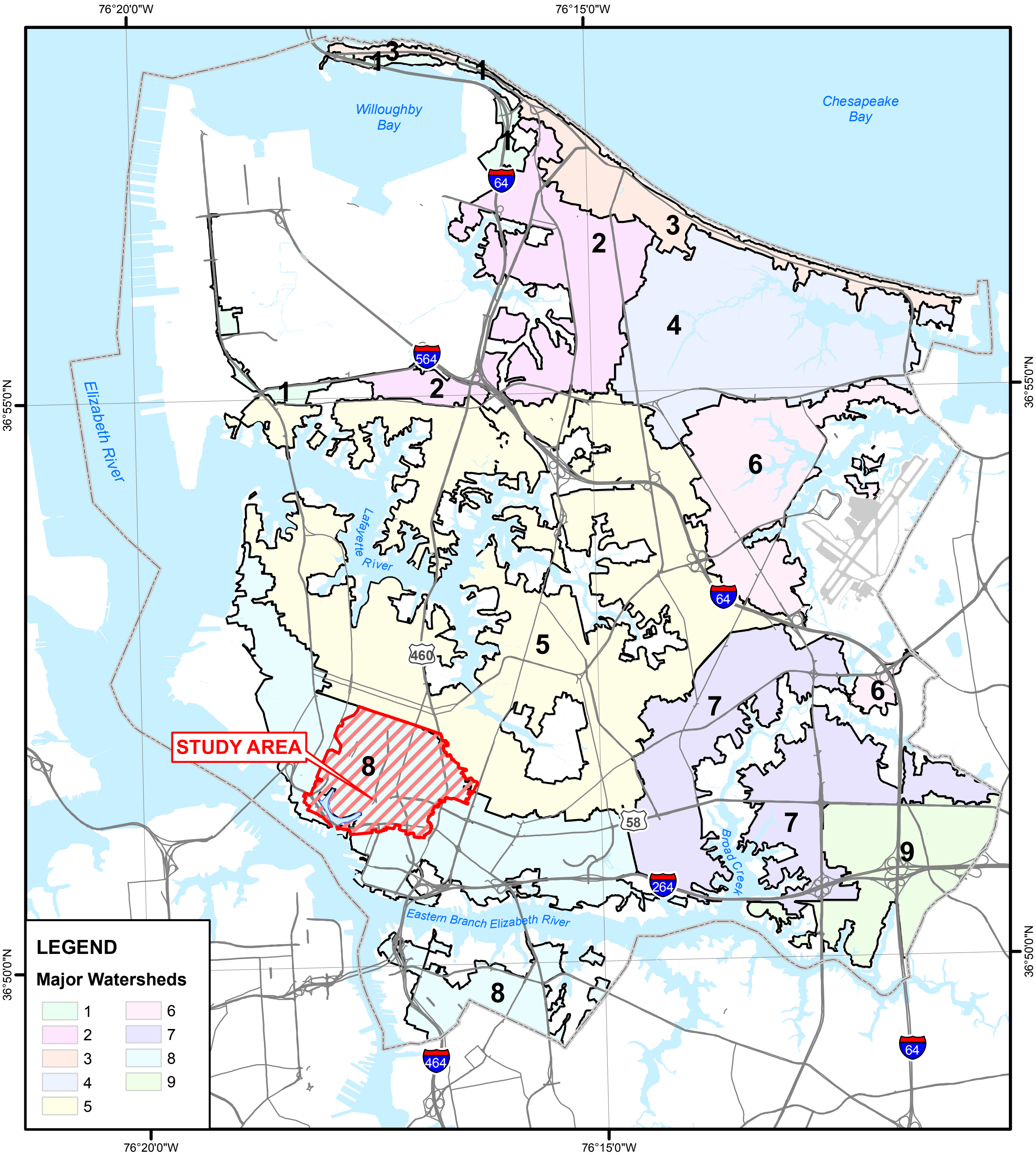
FIGURES

City of Norfolk, Department of Public Works
Project No. 3627.005



N
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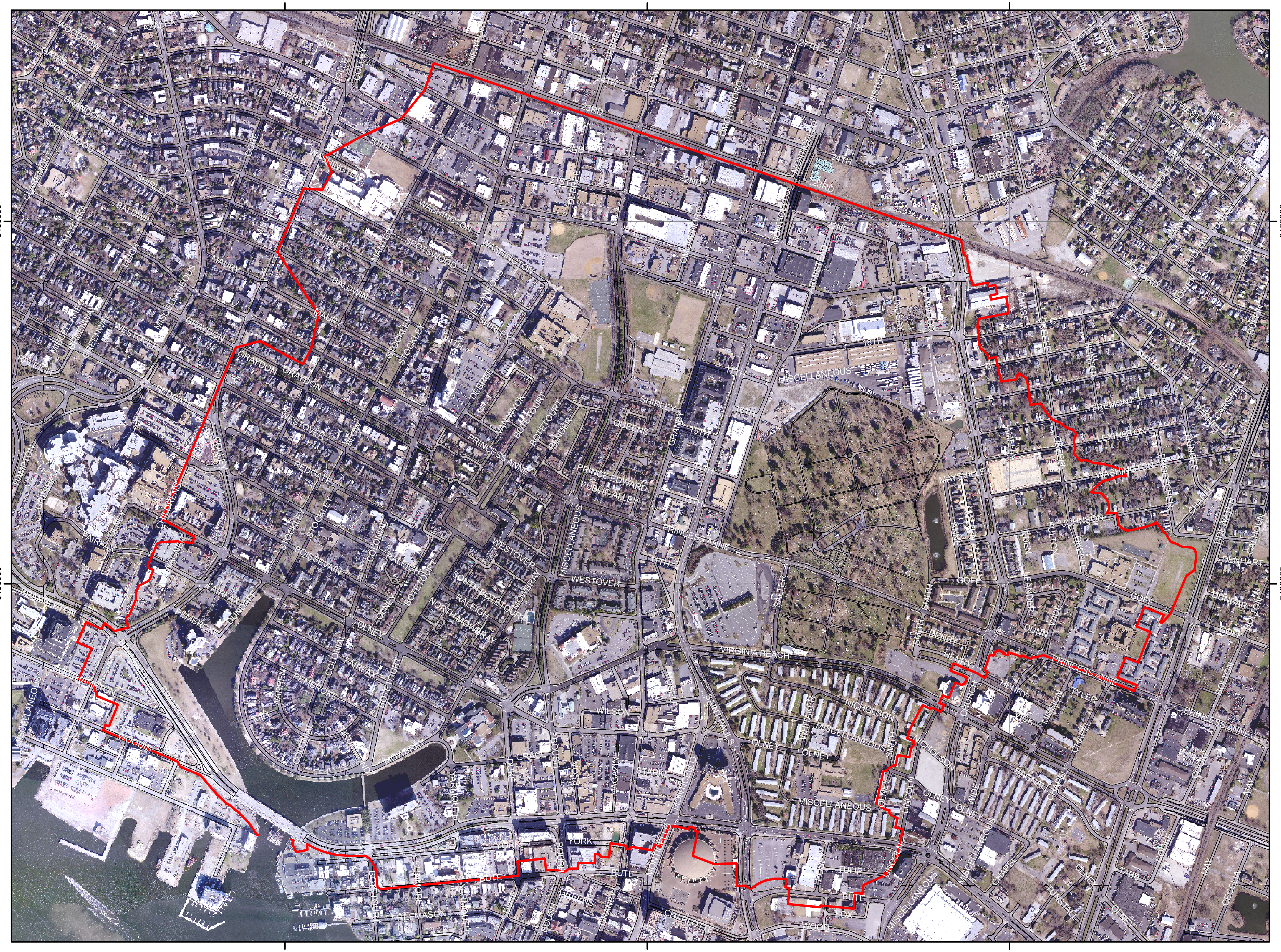
PROJECT AREA
City-wide Coastal Flooding Study
Norfolk, Virginia

0 1 Miles
0 1 Kilometers

FIGURE 1-1

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_04_20_Draft_Flood_Mitigation_Report\mxd\Fig-1-1_Project_Area.mxd, 04/21/11, jfisher



LEGEND

Hague Watershed Boundary

Note:

1. City 2009 aerial photograph mosaic provided by City of Norfolk GIS Department.

N
W E
S

0 800 1,600 Feet

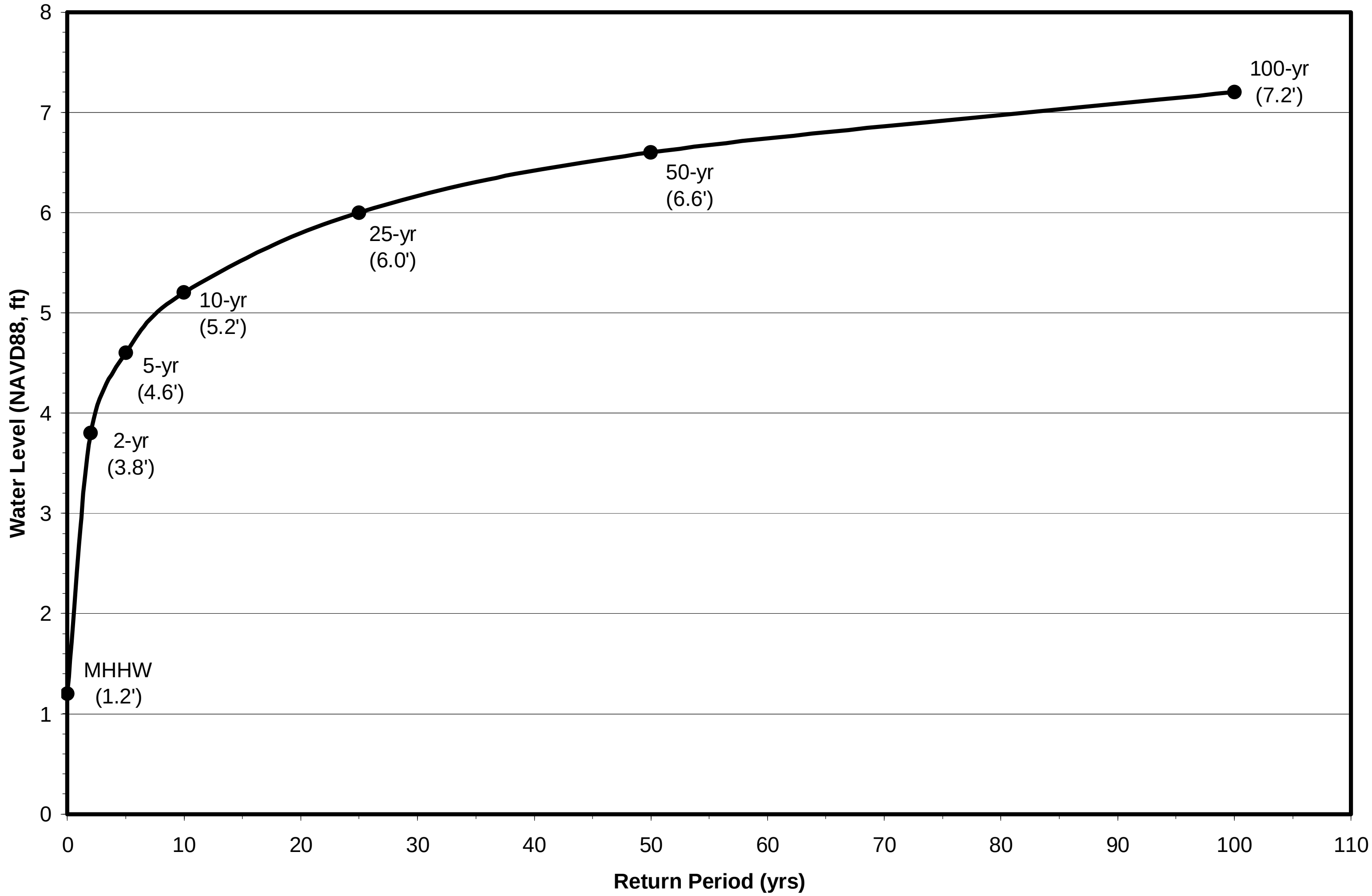
HAGUE WATERSHED
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 1-2

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_04_20_Draft_Flood_Mitigation_Report\mxd\Fig-1-2_Hague_Watershed.mxd, 04/21/11, jfisher

BASIN OUTLET
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 1-3



WATER LEVEL ELEVATIONS AT SEWELLS POINT FOR VARIOUS RETURN PERIODS

Based on Current Sea Level Elevation

City-wide Coastal Flooding Study

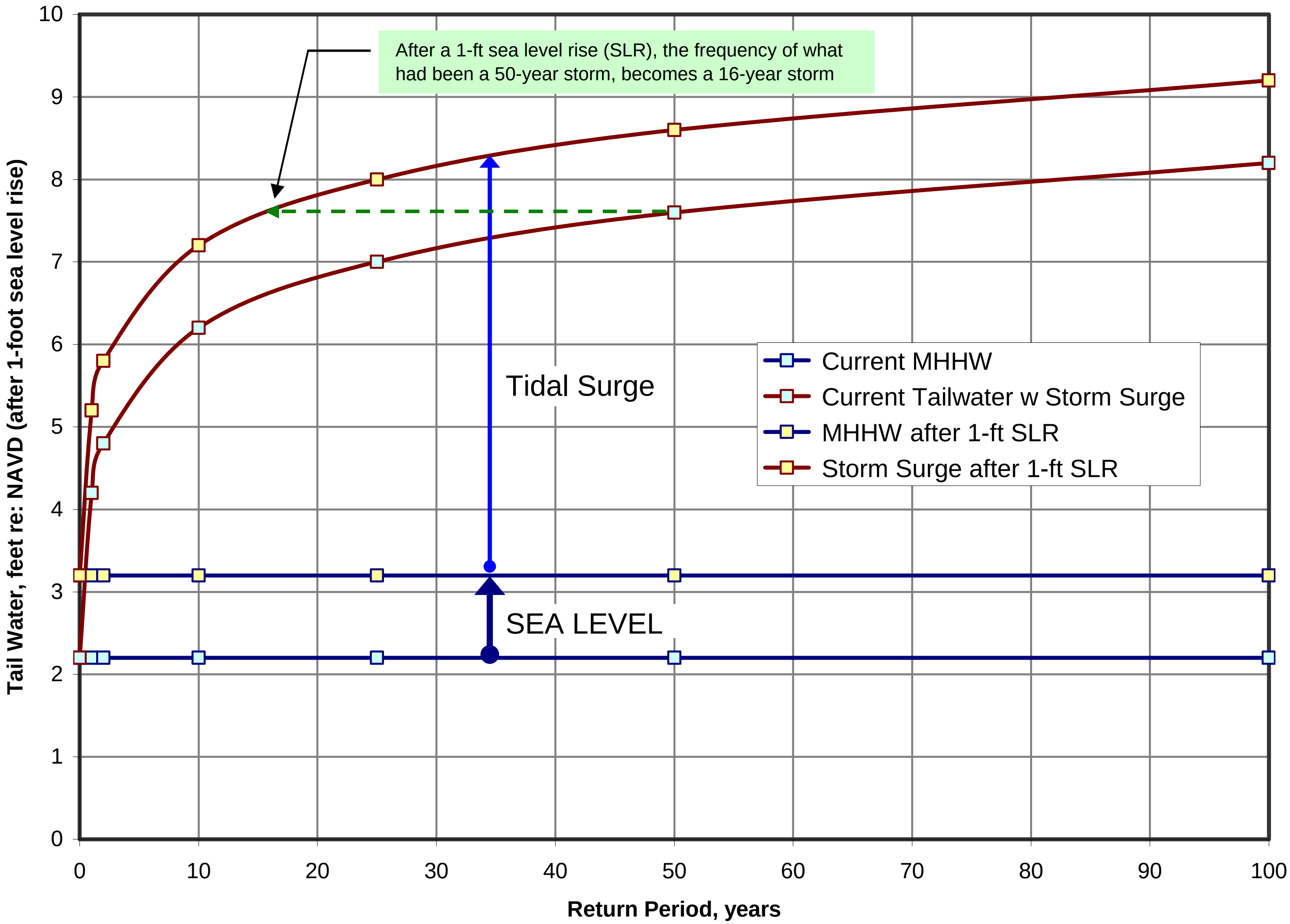
Norfolk, Virginia

FIGURE 2-1

City of Norfolk, Department of Public Works
Project No. 3627.005

N:\PROJECTS\3627_CITY_NORFOLK\3627-005_HAGUE\OUTPUTS\2011_04_20_DRAFT_FLOOD_MITIGATION_REPORT\DOC\FIG-2-2_IMPLICATIONSSLR.DOC

City of Norfolk, Department of Public Works
Project No. 3627.005



IMPLICATIONS OF FUTURE SEA LEVEL RISE

City-wide Coastal Flooding Study

Norfolk, Virginia

FIGURE 2-2

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

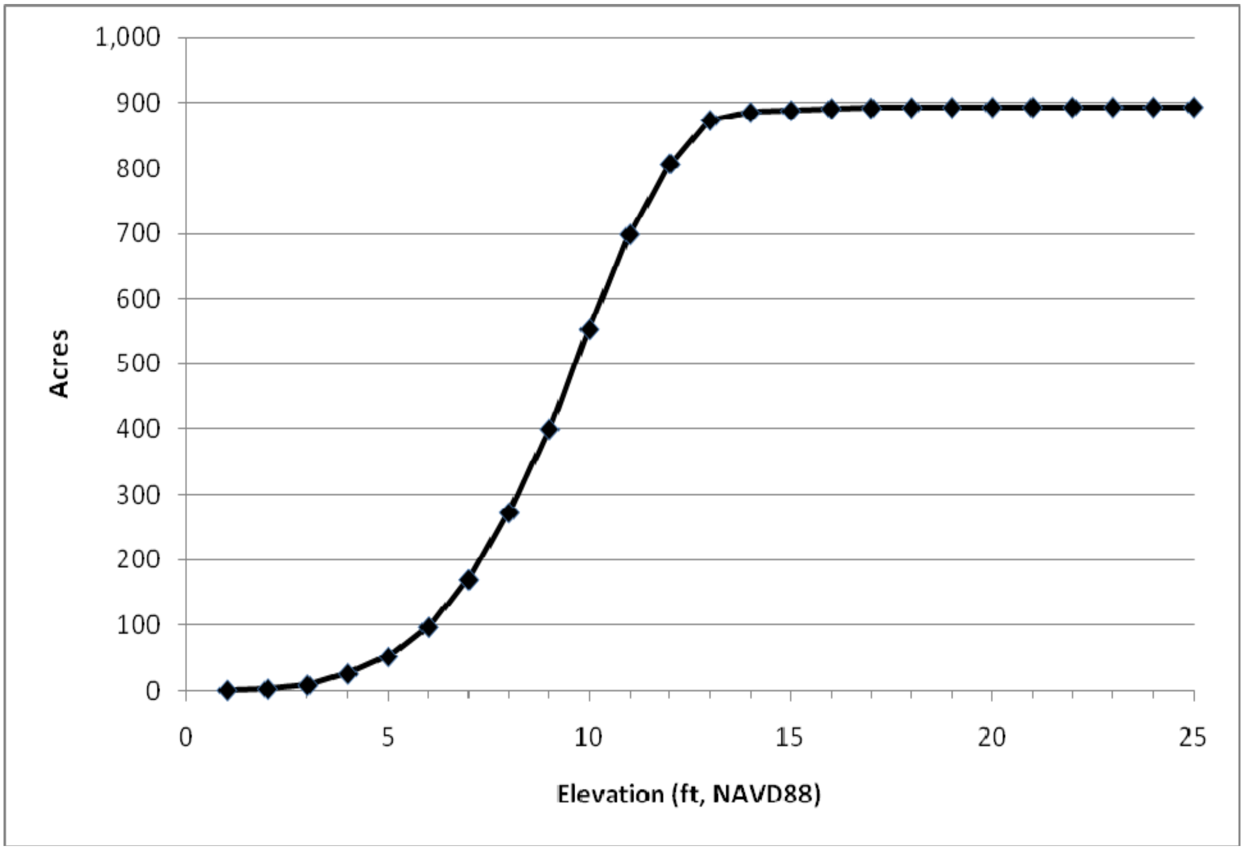
N E S W

0 500 1,000
Feet

TOPOGRAPHY
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 3-1

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_04_20_Draft_Flood_Mitigation_Report\mxd\Fig-3-1_Topography.mxd, 04/20/11, jfisher



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:

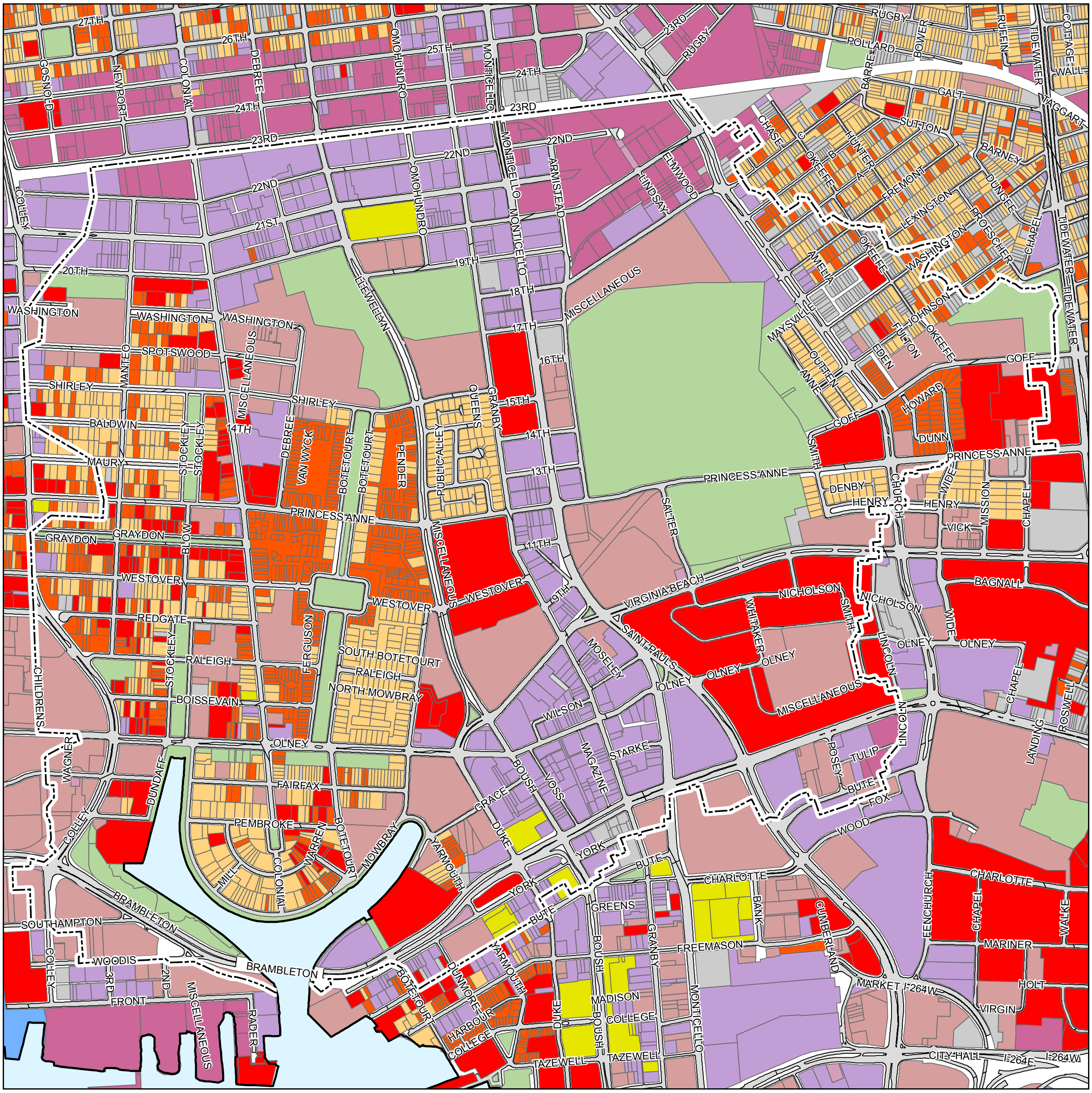
1) 50 acres of the study area is equal to or below elevation 5 feet (NAVD88).
2) 400 acres of the study area is equal to or below elevation 9 feet (NAVD88).

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

N E S W

0 500 1,000
Feet

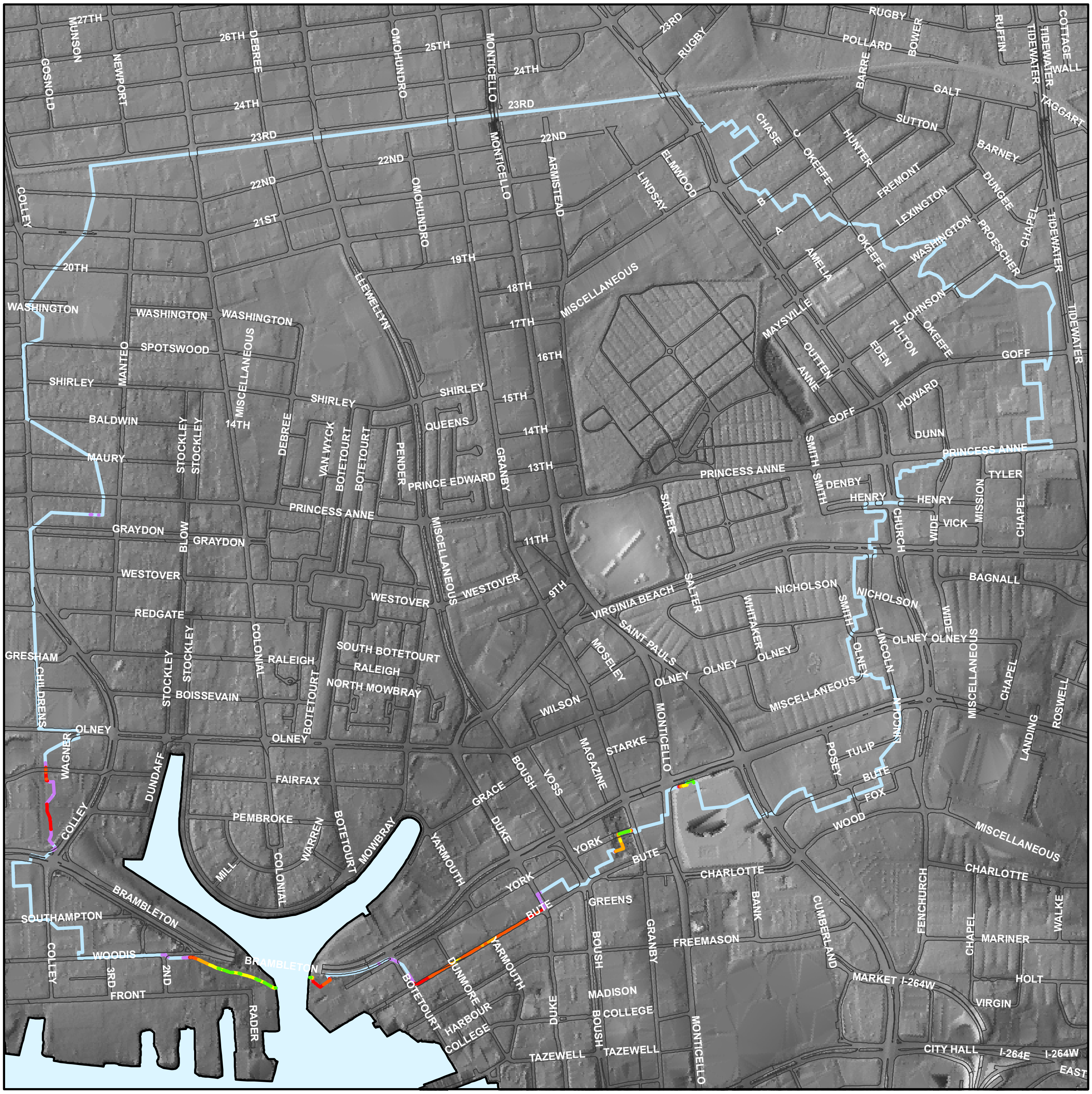
EXISTING LAND USE AREAS

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 3-3

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_04_20_Draft_Flood_Mitigation_Report\mxd\Fig-3-3_Hague_Landuse.mxd, 04/20/11, jfisher

City of Norfolk, Department of Public Works
Project No. 3627.005



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

N
E
W
S

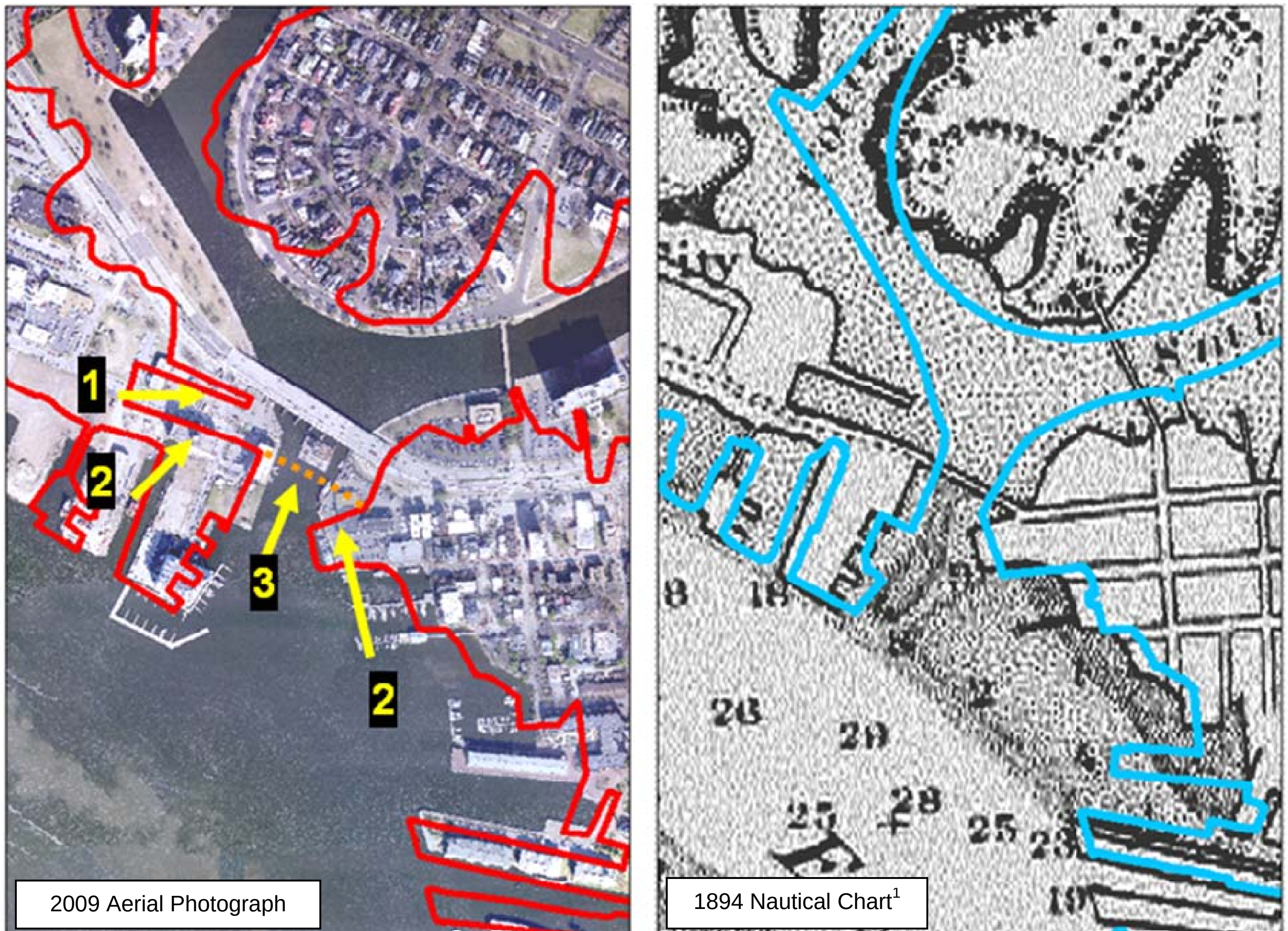
0 500 1,000
Feet

WATERSHED RIM ELEVATION

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 3-4

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_04_20_Draft_Flood_Mitigation_Report\mxd\Fig-3-4_Rim_Elevations.mxd, 04/20/11, jfisher



[1]NOAA Chart BiC-45, Hampton Roads

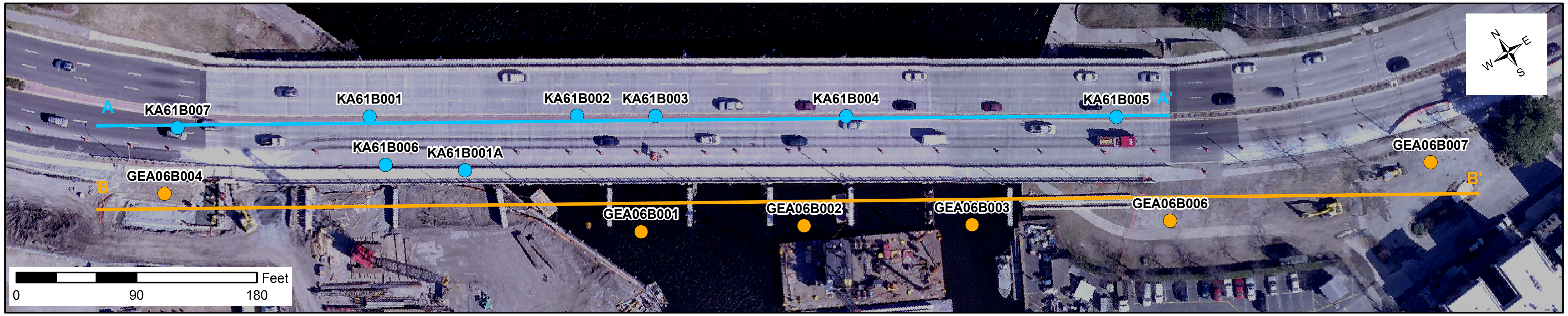
Numerous former structures may be present and challenging to tie new structures into shore
1 - Former Pier
2 - Former Hard Shoreline; possible concrete or piling structures
3 - Former Bridge

FORMER SHORELINE STRUCTURES
Basin Outlet
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 4-1

City of Norfolk, Department of Public Works
Project No.3627.005

N:\PROJECTS\3627_CITY_NORFOLK\3627-005_HAGUE\OUTPUTS\2011_04_20_DRAFT_FLOOD_MITIGATION_REPORT\DOC\FIG-4-1_FORMERSHORELINESTRUCTURES.DOC



LEGEND

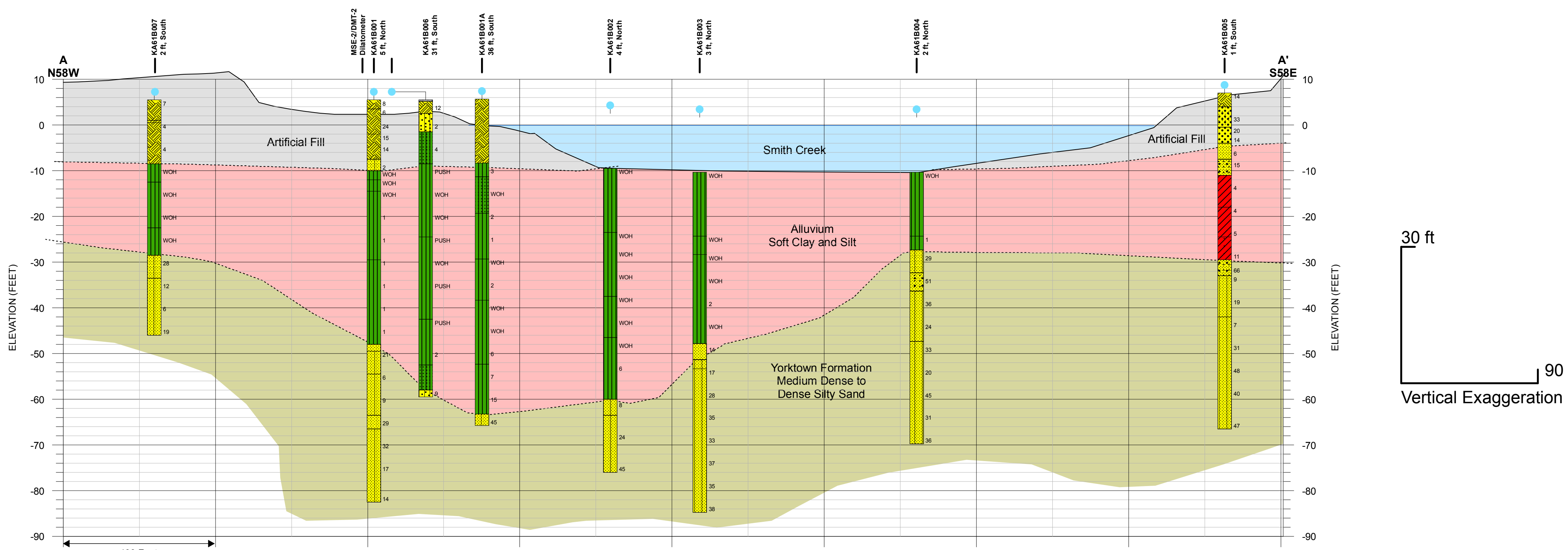
- 1961 Geotechnical Boring
- 2006 Geotechnical Boring
- Cross Section Location AA'
- Cross Section Location BB'

Stratigraphic Units

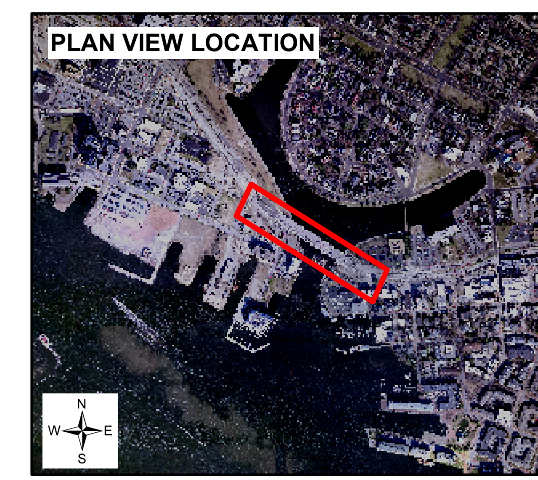
- af — Artifical Fill
- Qals — Sand Alluvium
- Qal — Soft Silt and Clay Alluvium
- Ty — Yorktown Formation - Silty Sand

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)



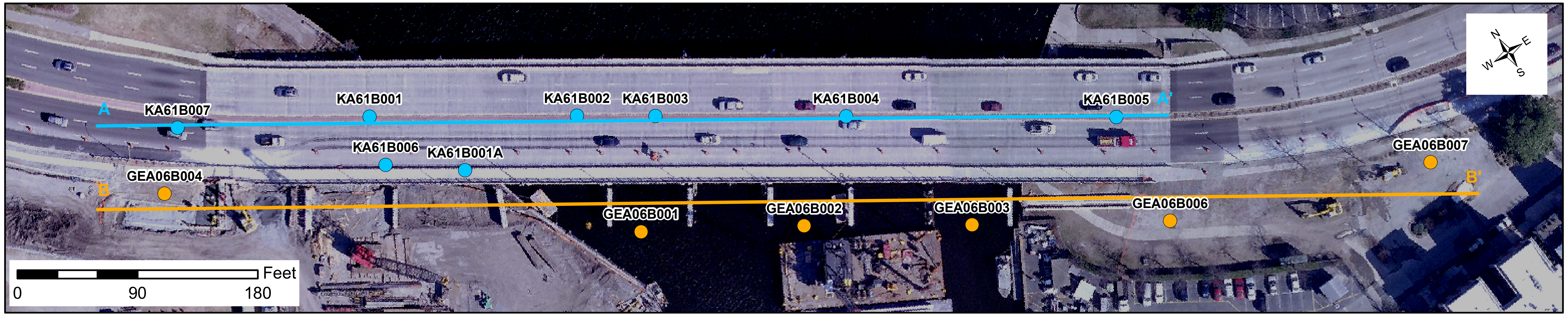
Vertical Exaggeration is 3x.



BRAMBLETON BRIDGE PRELIMINARY SUBSURFACE CROSS SECTION
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 4-2

City of Norfolk, Department of Public Works
Project No. 3627.005



LEGEND

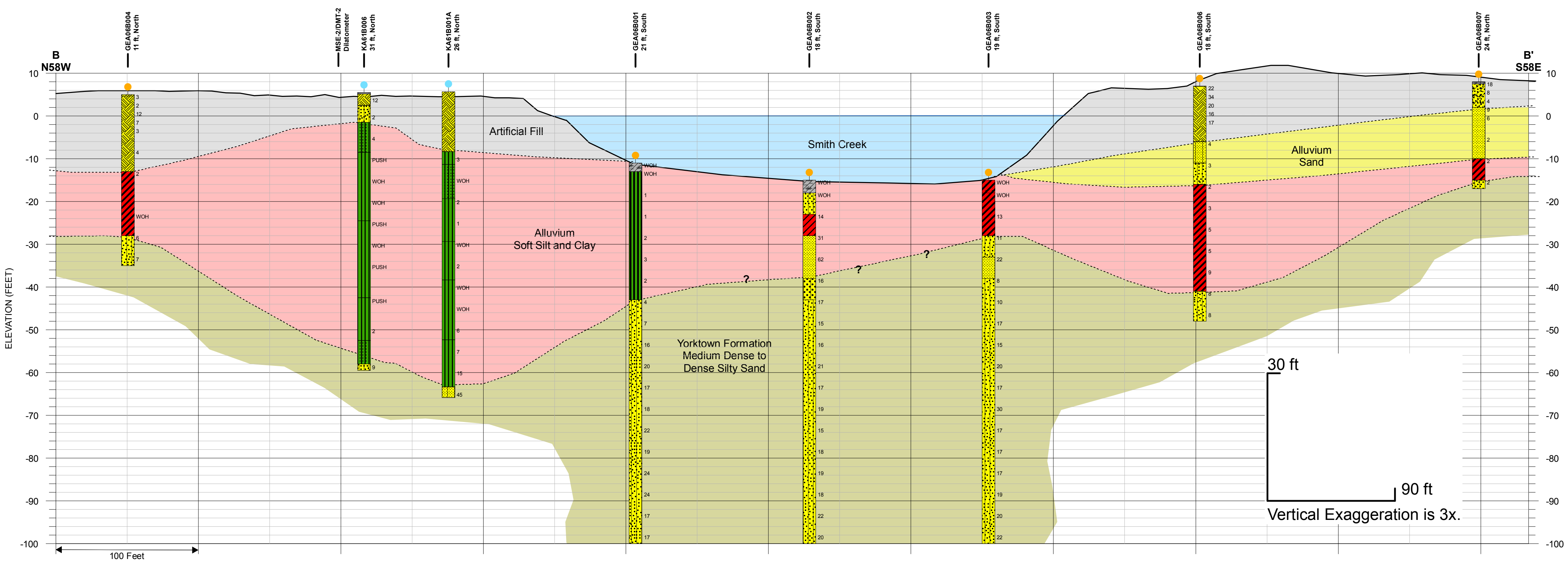
- 1961 Geotechnical Boring
- 2006 Geotechnical Boring
- Cross Section Location AA'
- Cross Section Location BB'

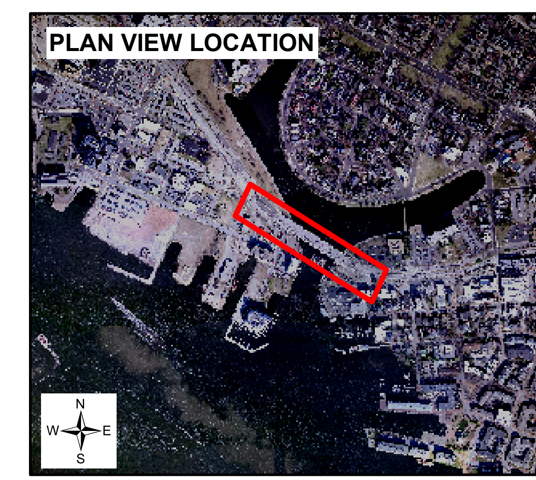
Stratigraphic Units

- af — Artifical Fill
- Qals — Sand Alluvium
- Qal — Soft Silt and Clay Alluvium
- Ty — Yorktown Formation - Silty Sand

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)





LIGHT RAIL BRIDGE PRELIMINARY SUBSURFACE CROSS SECTION

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 4-3

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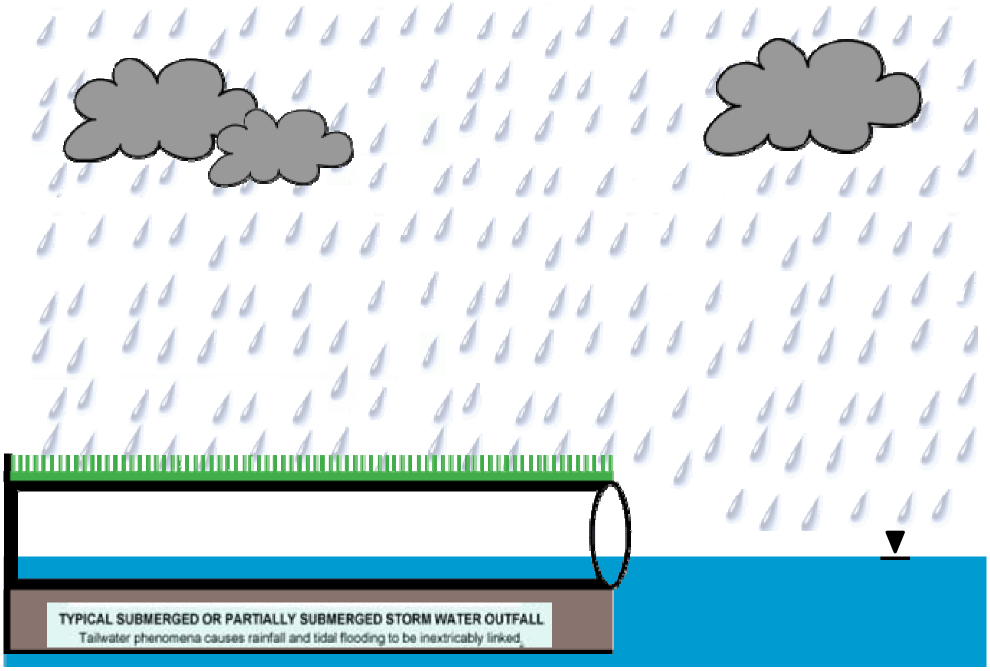
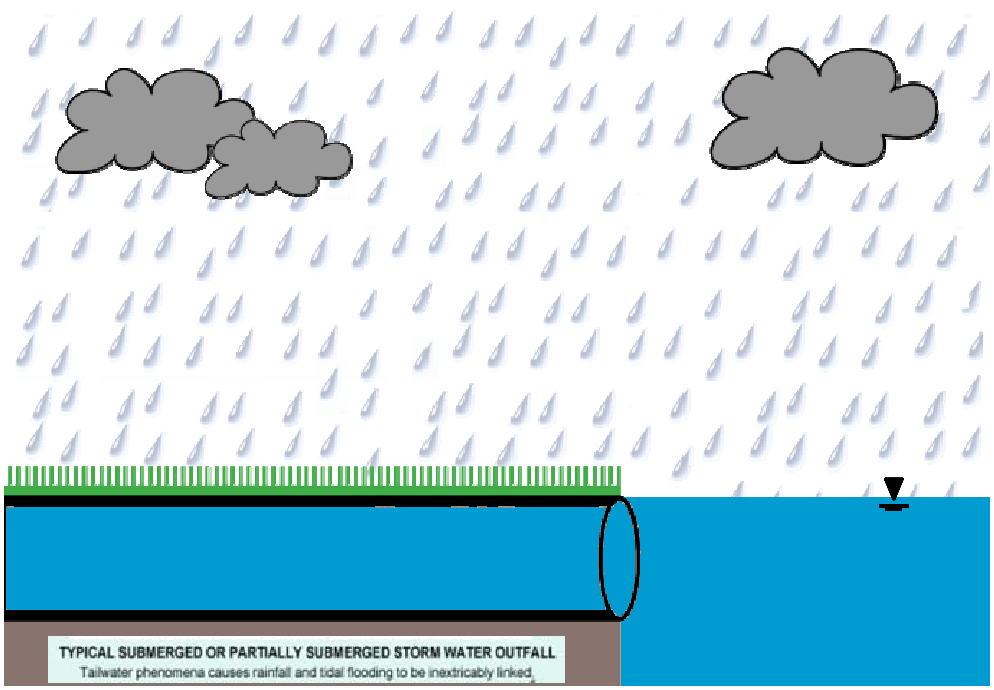
Rainfall Predominant



Coincident Surge Event



TAILWATER PHENOMENA
City-wide Coastal Flooding Study
Norfolk, Virginia

LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Subcatchments (with Acreages Labeled)

Notes:

1. City 2008 aerial photograph mosaic provided by City of Norfolk GIS Department.

N
W E
S

0 800 1,600 Feet

SWMM SUBCATCHMENTS FOR THE HAGUE DRAINAGE AREA

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-1

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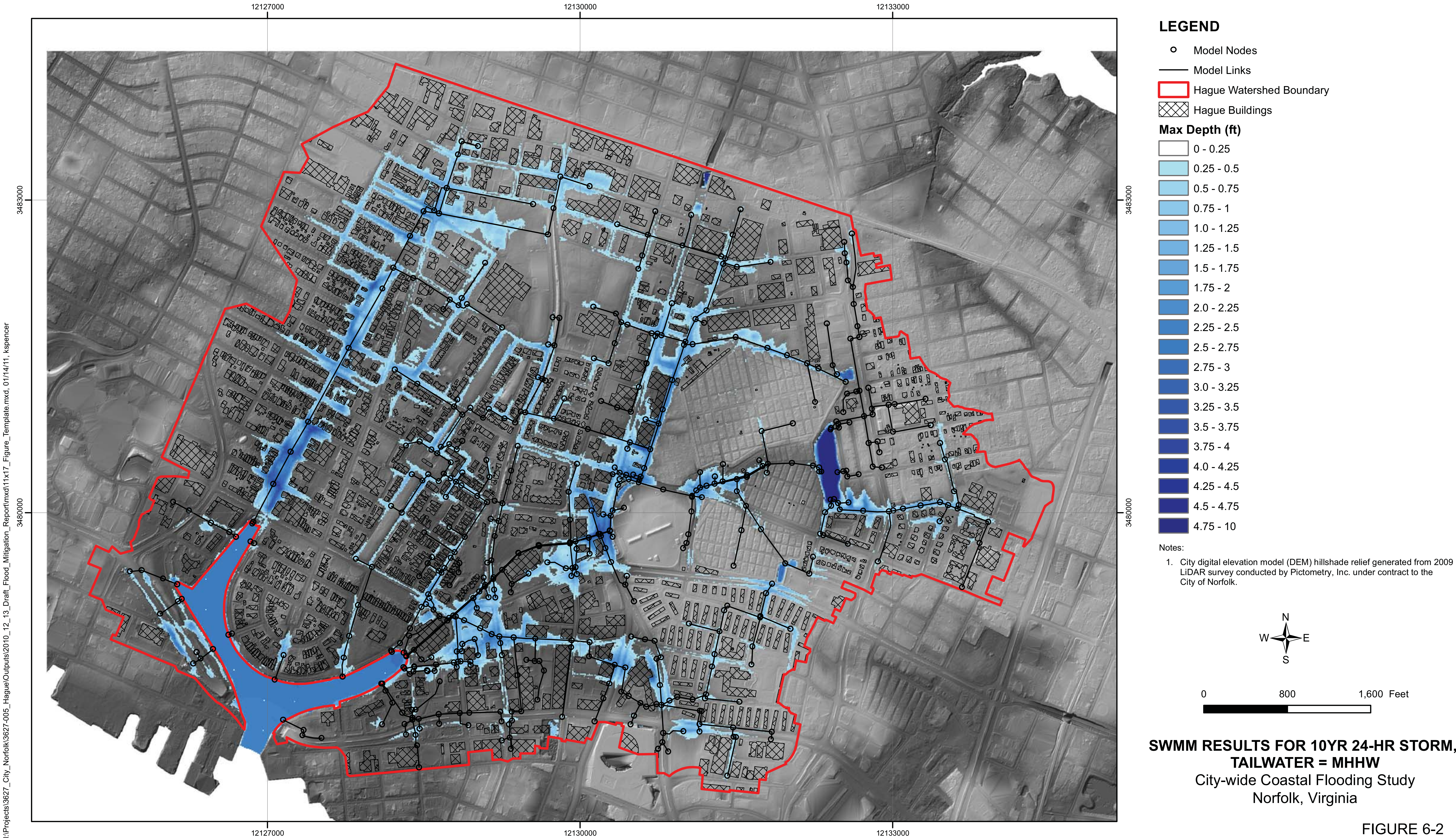

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

SWMM RESULTS FOR 10YR 24-HR STORM, TAILWATER = MHHW

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-2

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LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
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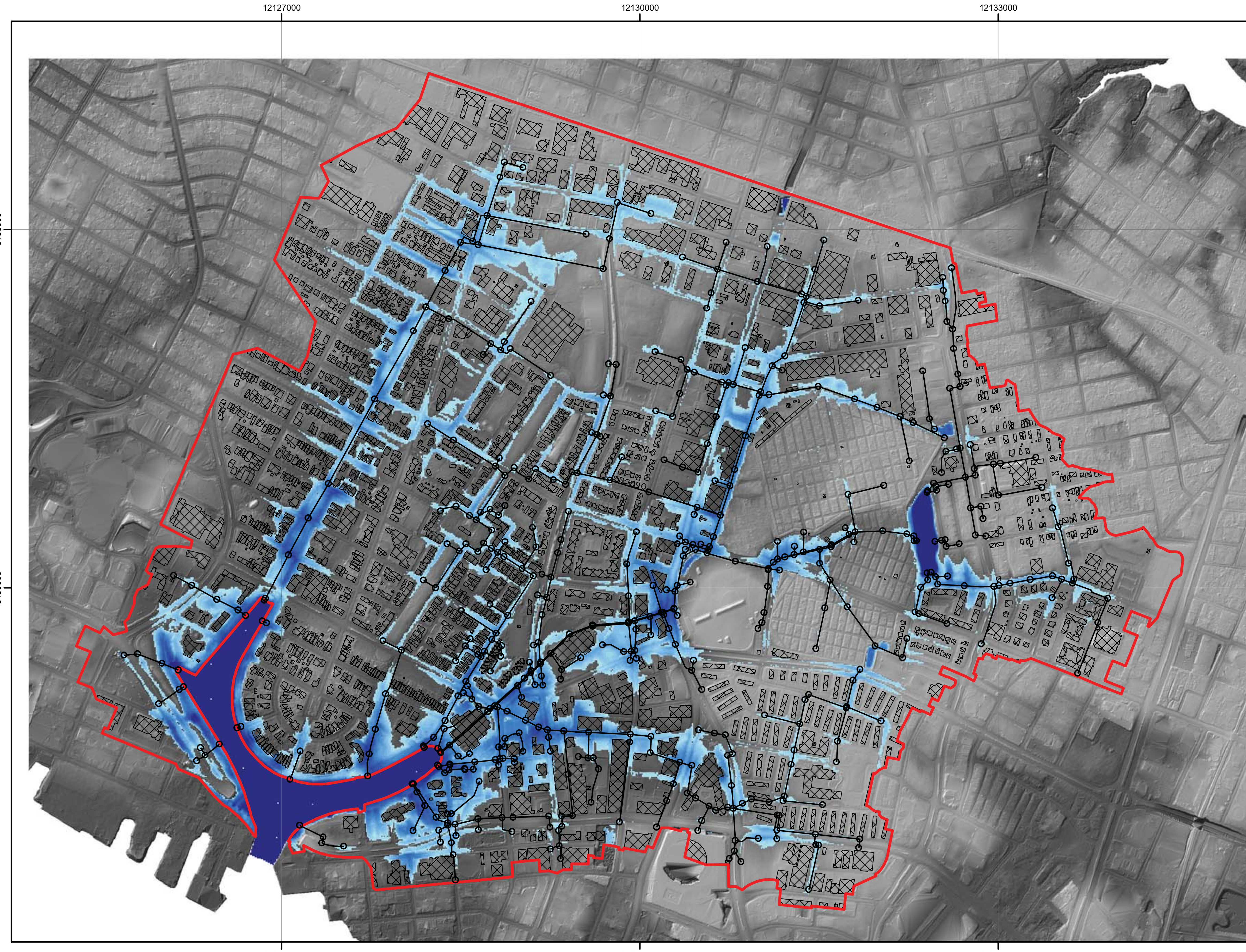
0 800 1,600 Feet

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = MHHW

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-3

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

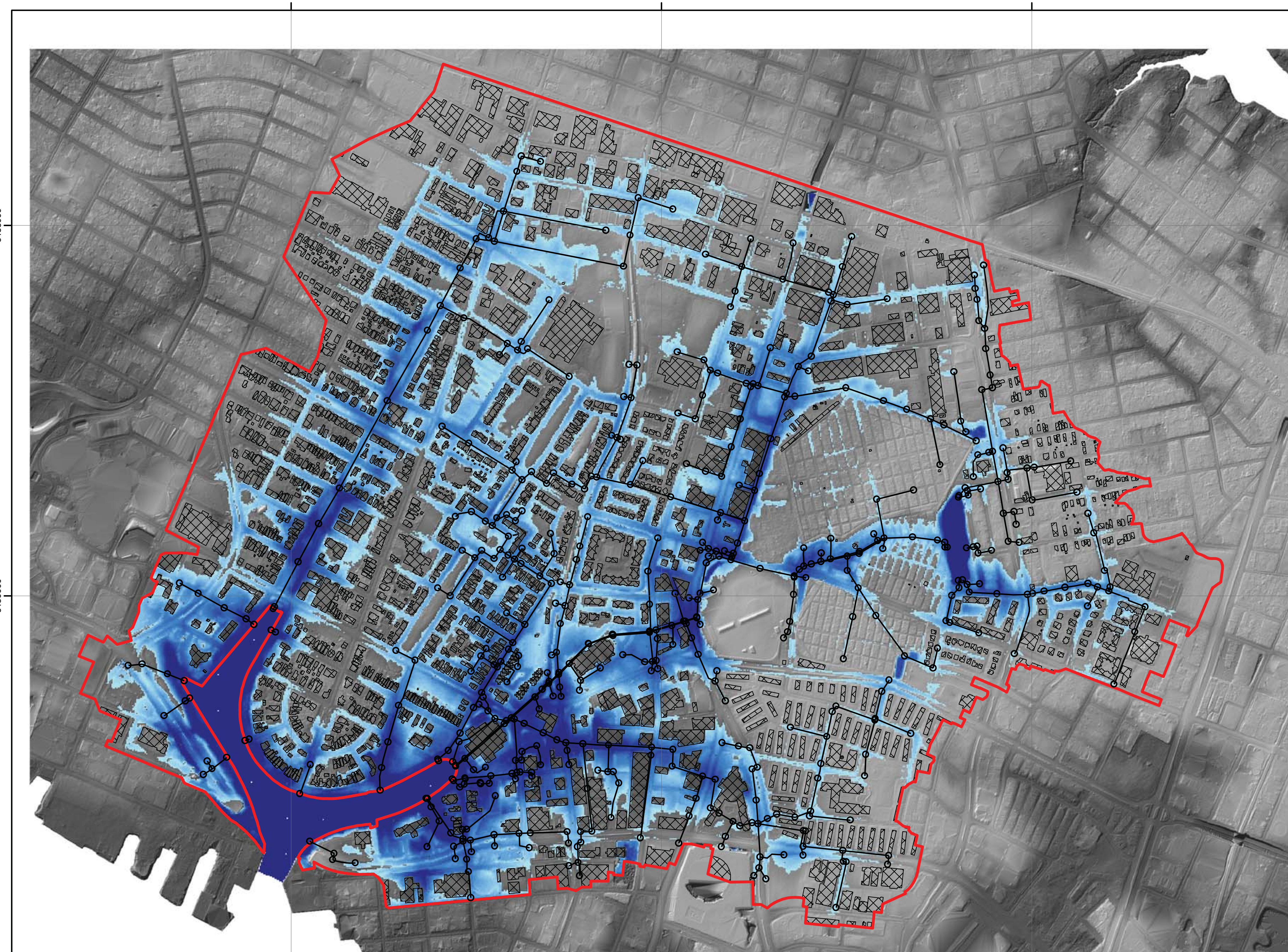
0 800 1,600 Feet

SWMM RESULTS FOR 10YR 24-HR STORM, TAILWATER = 10YR STORM SURGE

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-4

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

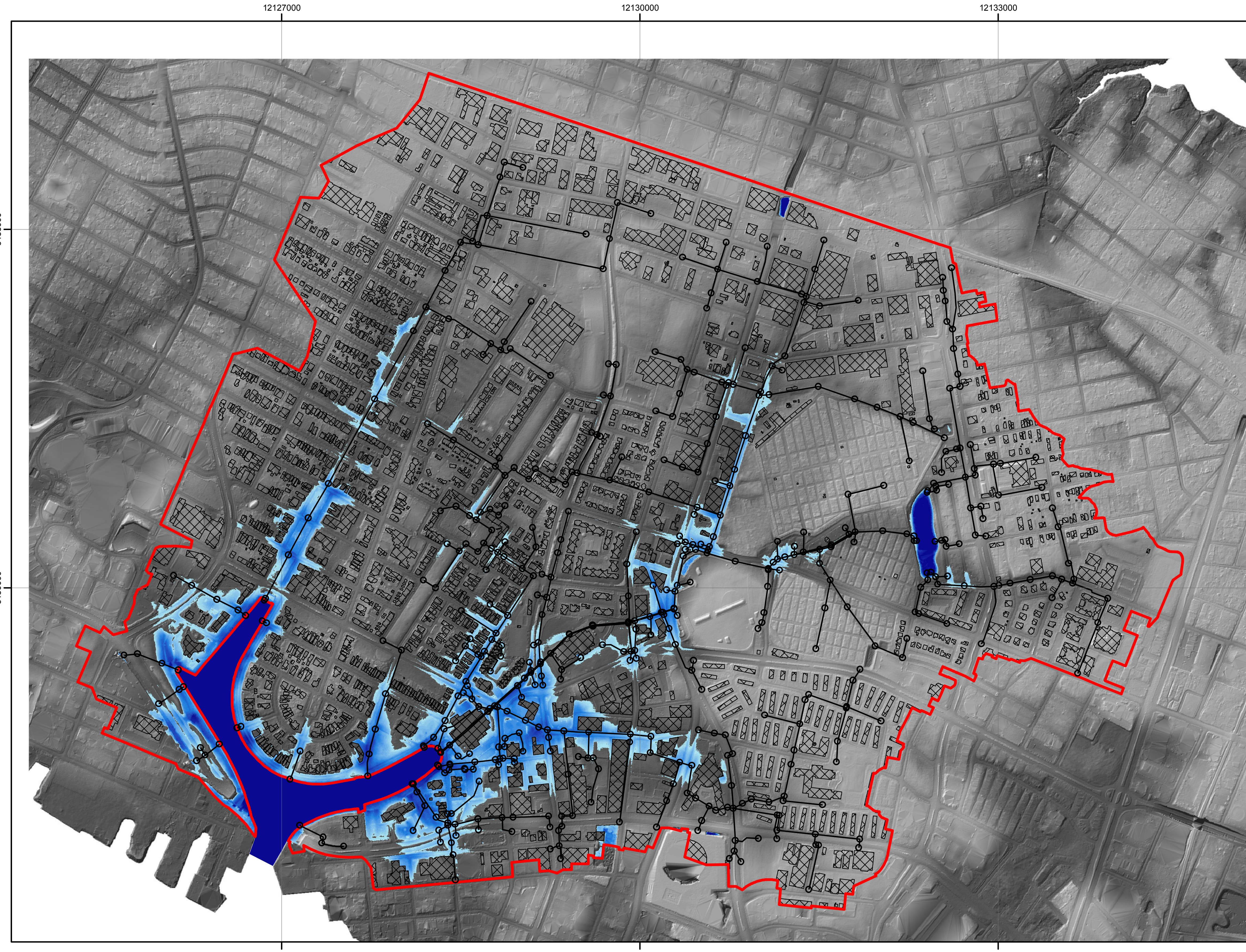
0 800 1,600 Feet

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-5

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

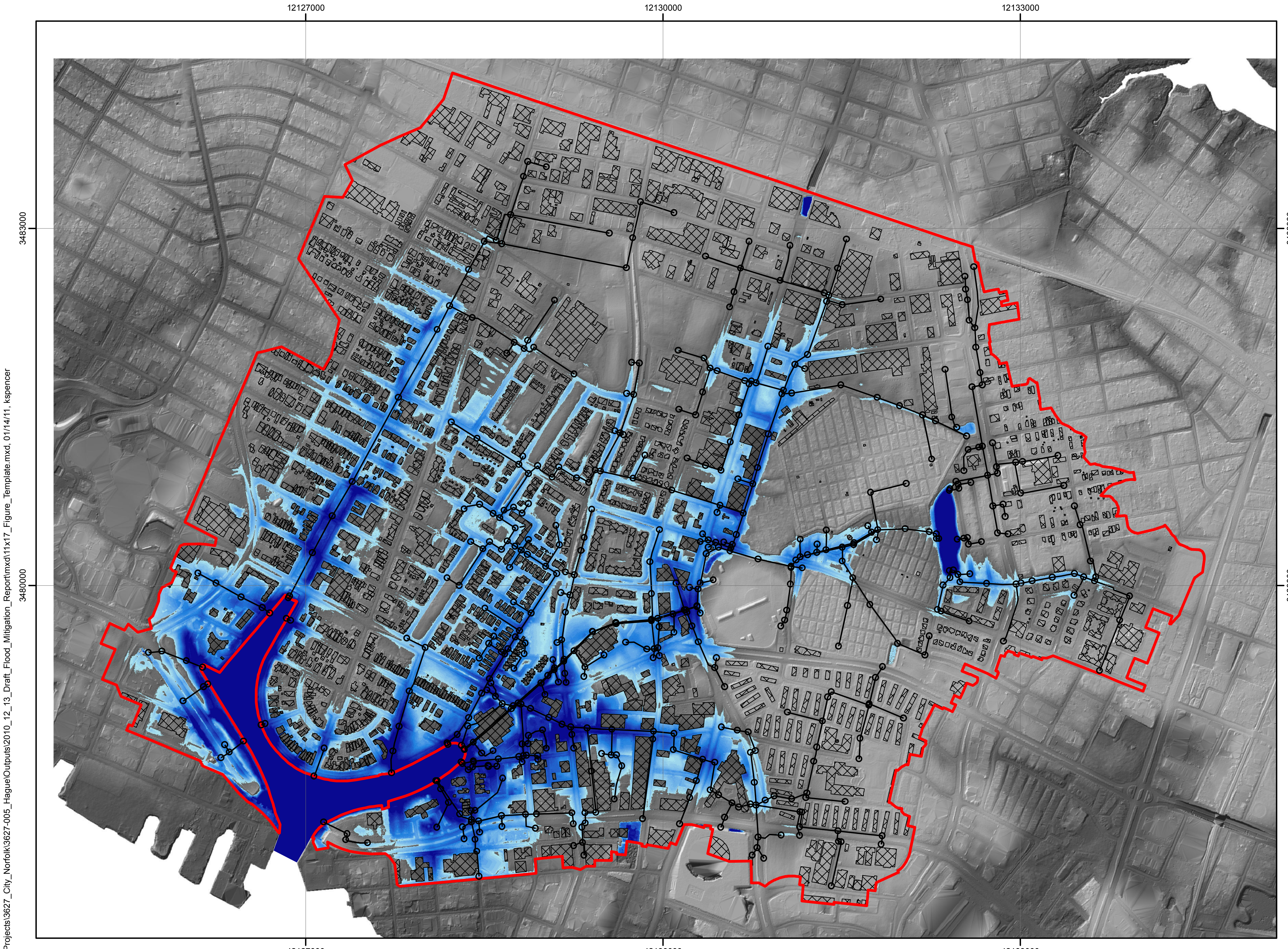
0 800 1,600 Feet

EXTENT OF 10YR STORM SURGE
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-6

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer

City of Norfolk, Department of Public Works
Project No. 3627.005



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

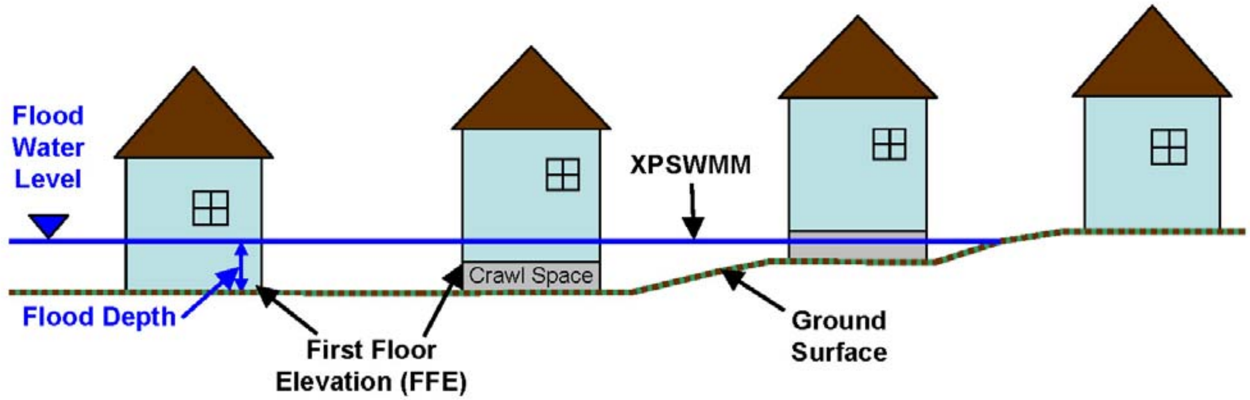
1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

0 800 1,600 Feet

EXTENT OF 100YR STORM SURGE
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 6-7



Flooding Results from Modeling Analysis (XPSWMM)

↓

Evaluate Whether the Structure Floods or Not

↓

Estimate Building Flood Depth

↓

Damage Estimate Using Depth Damage Function (DDF)

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
Damages Increase

FLOOD DAMAGE ESTIMATES
10YR 24-HR STORM, TAILWATER = MHHW

City-wide Coastal Flooding Study
Norfolk, Virginia

N E S W

0 500 1,000
Feet

FIGURE 7-2

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_02_09_Draft_Flood_Mitigation_Report\mxd\Fig-7-2_10yrMHHW_ExCond_DamageEst.mxd, 02/08/11, kspencer

LEGEND

Total Estimated Damage

Buildings Not Damaged or Not Included in Analysis

Damages Increase

FLOOD DAMAGE ESTIMATES
100YR 24-HR STORM,
TAILWATER = MHHW

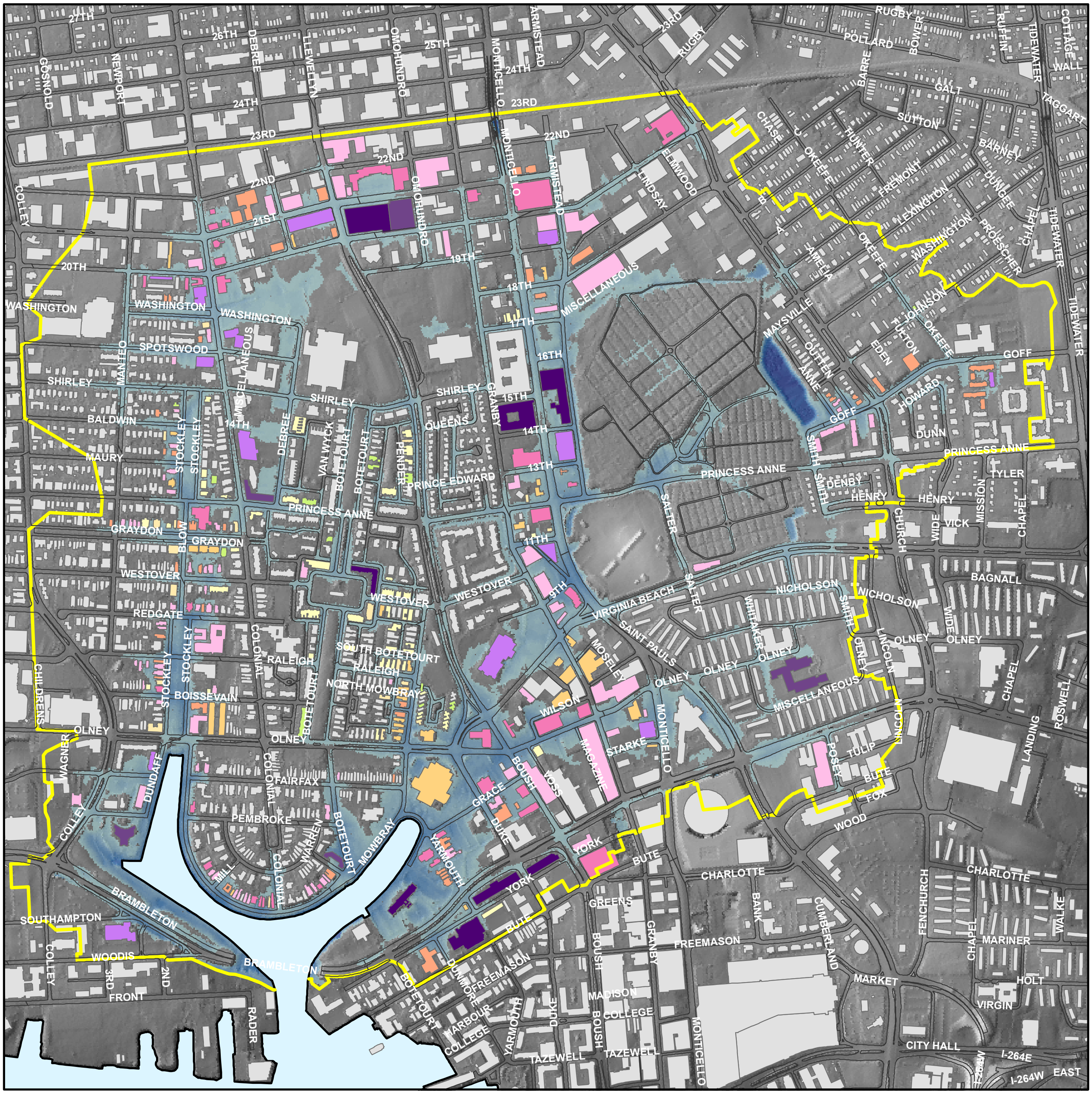
City-wide Coastal Flooding Study
Norfolk, Virginia

N
E
S
W

0 500 1,000
Feet

FIGURE 7-3

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_02_09_Draft_Flood_Mitigation_Report\mxd\Fig-7-3_100yrMHHW_ExCond_DamageEst.mxd, 02/08/11, kspencer



LEGEND

Total Estimated Damage

Buildings Not Damaged or Not Included in Analysis

Damages Increase

FLOOD DAMAGE ESTIMATES
10YR 24-HR STORM,
TAILWATER = 10YR STORM SURGE

City-wide Coastal Flooding Study

Norfolk, Virginia

N E S W

0 500 1,000

Feet

FIGURE 7-4

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_02_09_Draft_Flood_Mitigation_Report\mxd\Fig-7-4_10yr10yr_ExCond_DamageEst.mxd, 02/08/11, kspencer

LEGEND

Total Estimated Damage

Buildings Not Damaged or Not Included in Analysis

Damages Increase

FLOOD DAMAGE ESTIMATES
100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE

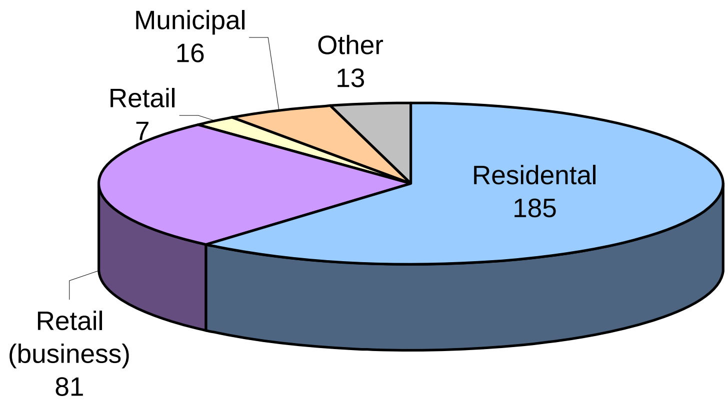
City-wide Coastal Flooding Study
Norfolk, Virginia

N E S W

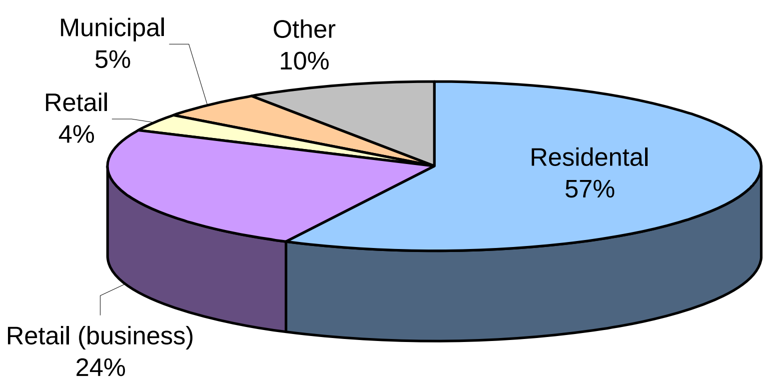
0 500 1,000
Feet

FIGURE 7-5

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2011_02_09_Draft_Flood_Mitigation_Report\mxd\Fig-7-5_100yr100yr_ExCond_DamageEst.mxd, 02/08/11, kspencer



Number of Buildings Damaged by Flood

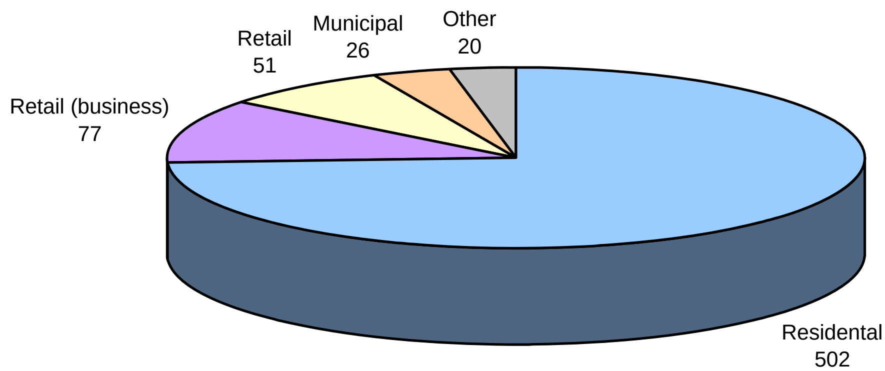


Cost Distribution of Flood Damage

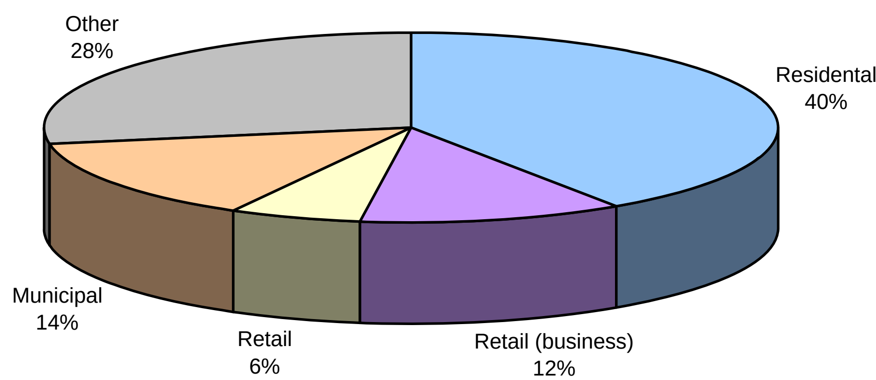
Description and example subtypes for the above general categories:
Retail - Commodity-based businesses (department stores, grocery stores, convenience markets, gas stations, etc)
Retail (business) - Service-based businesses (office buildings, hotels, storage centers, service stations, funeral homes, exercise centers, etc)
Municipal - Government owned or operated facilities as well as health service facilities (hospitals, schools, parks, medical clinics/health centers, museums, etc)
Residental - 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

FIGURE 7-6



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)
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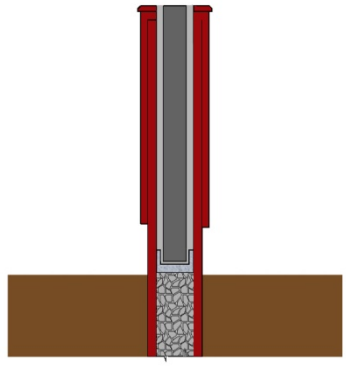
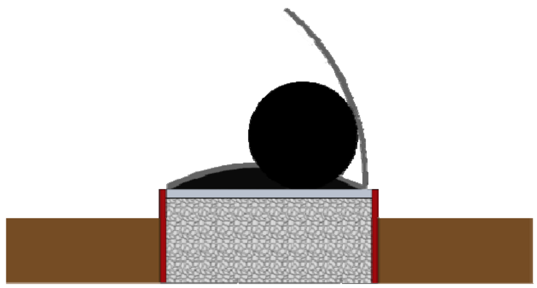
Other - Unclassified property

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

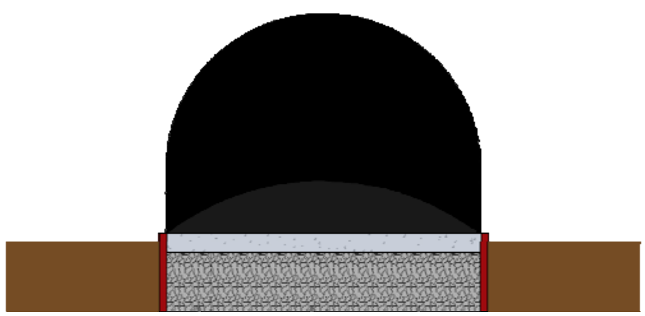
FIGURE 7-7



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

250' 0' 250' 500'

GRAPHIC SCALE

moffatt & nichol

FUGRO

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

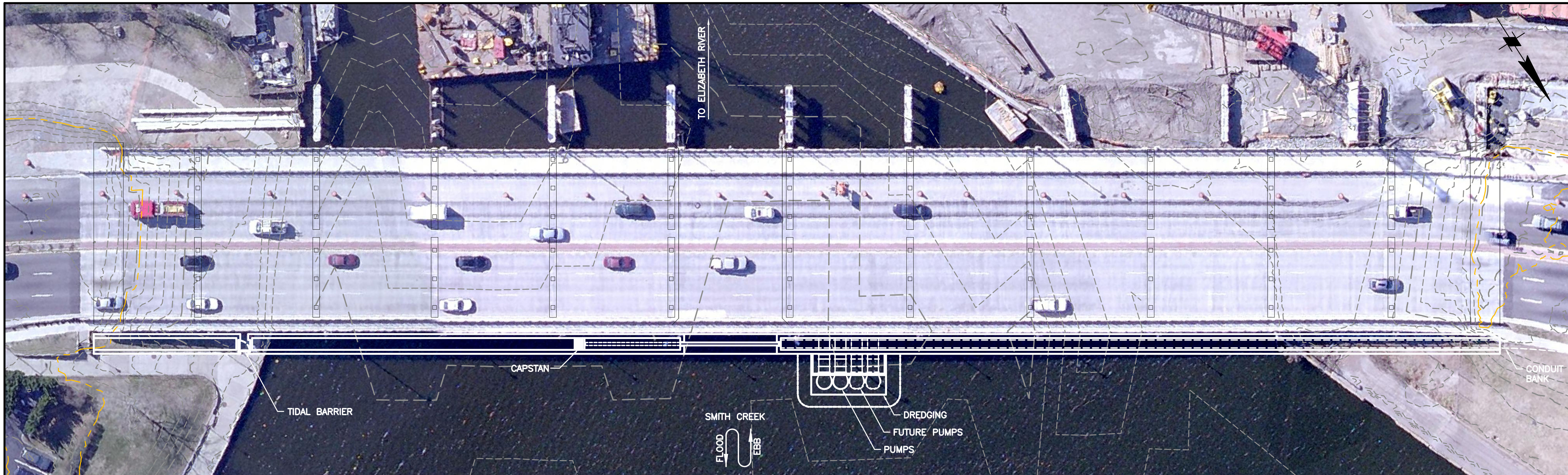
COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

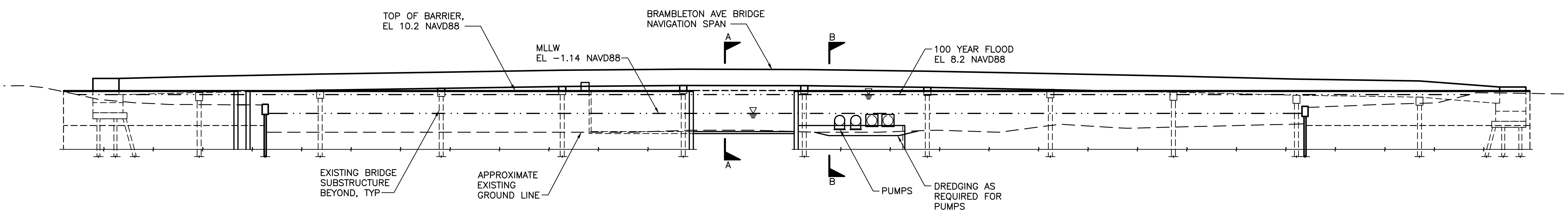
THE HAGUE

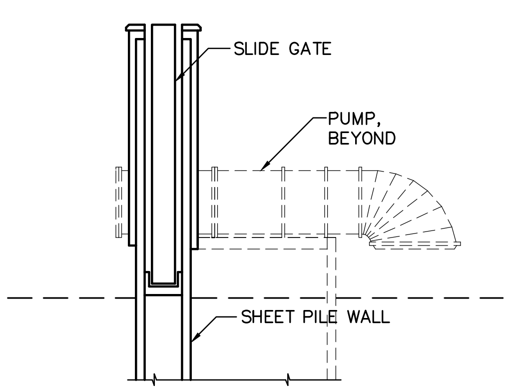
TIDAL BARRIER WITH TIDE GATE, PUMPS, AND CLOSURE WALLS – BERMS

FIGURE

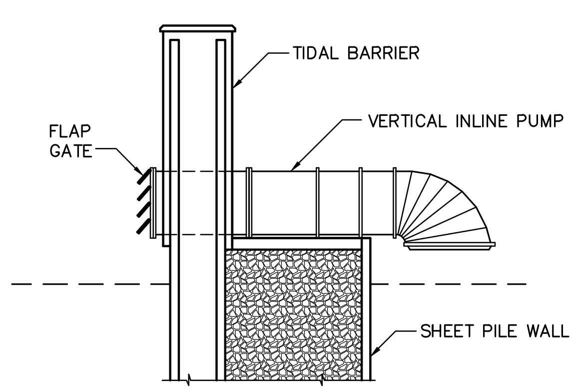
8-2







SECTION A–A



SECTION B–B

25' 0' 25' 50'

GRAPHIC SCALE

moffatt & nichol

FUGRO

DSGN	DR KOONS	CHK
JOB NO. 6822–03	SCALE 1"=50'	DATE 12–14–2010

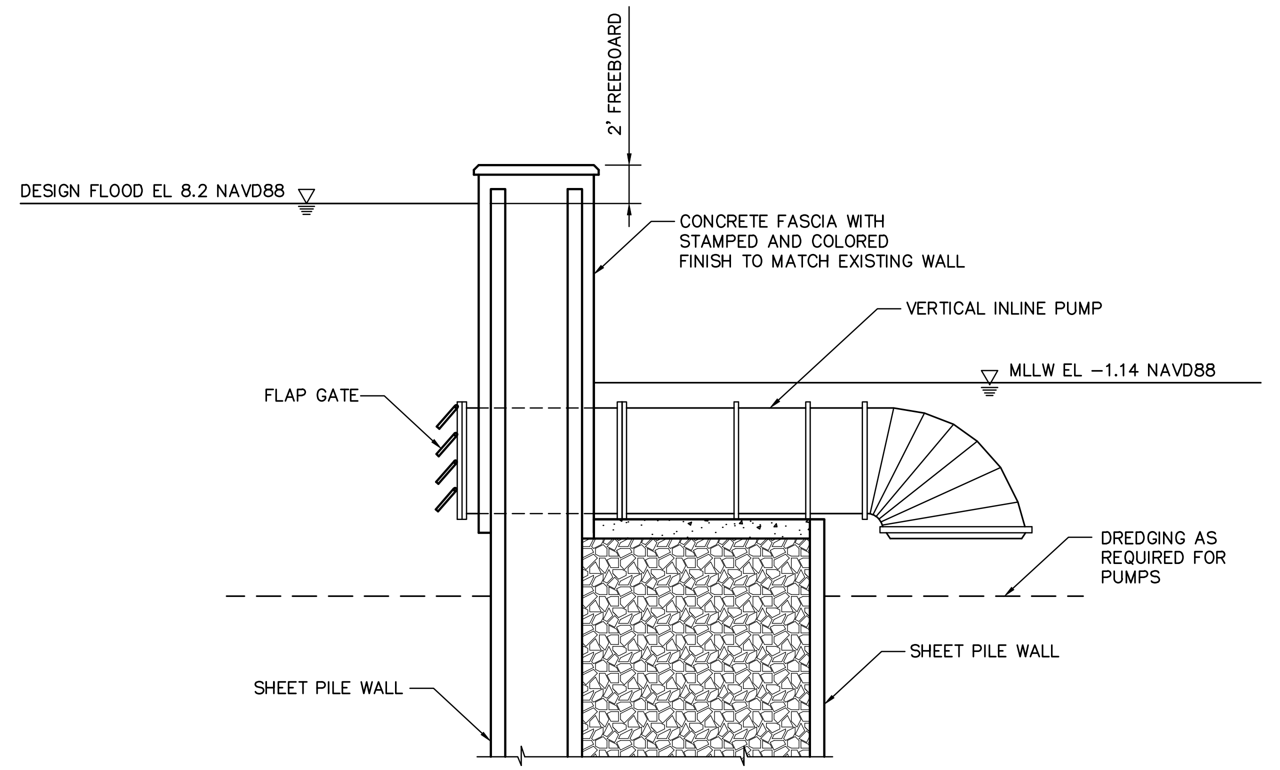
COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

THE HAGUE

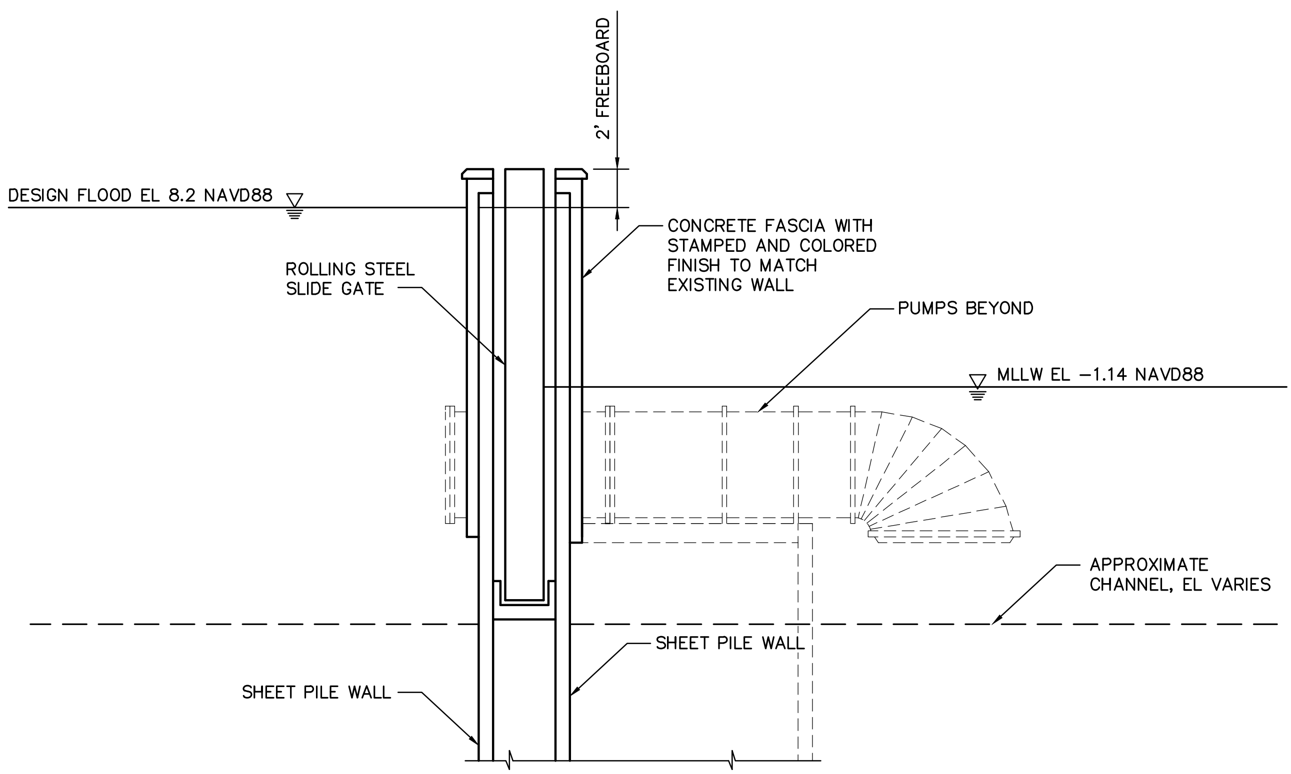
BRAMBLETON AVENUE BRIDGE
STEEL GATE OPTION

FIGURE

8-3



WALL SECTION AT PUMPS



SECTION AT FLOOD STEEL GATE

moffatt & nichol

FUGRO

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

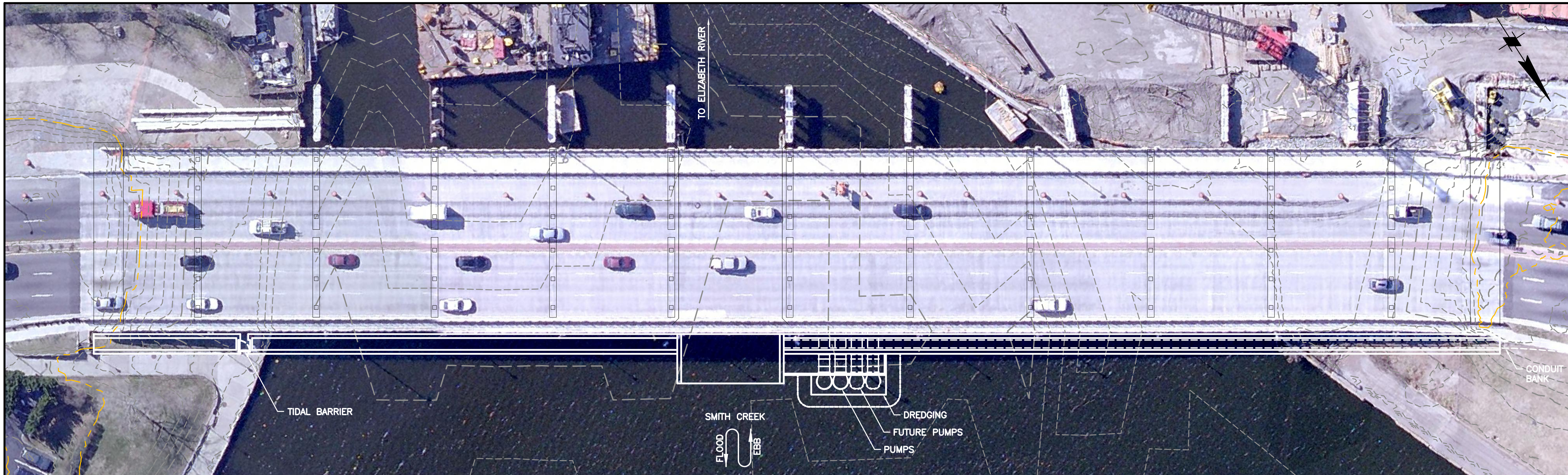
COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

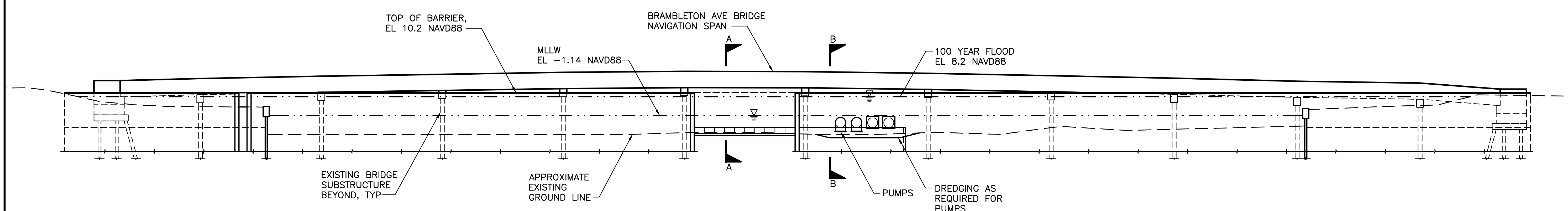
THE HAGUE

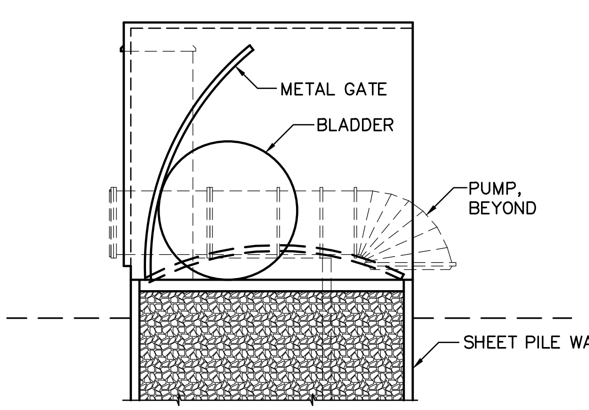
BRAMBLETON AVENUE BRIDGE
STEEL GATE OPTION – SECTIONS

FIGURE

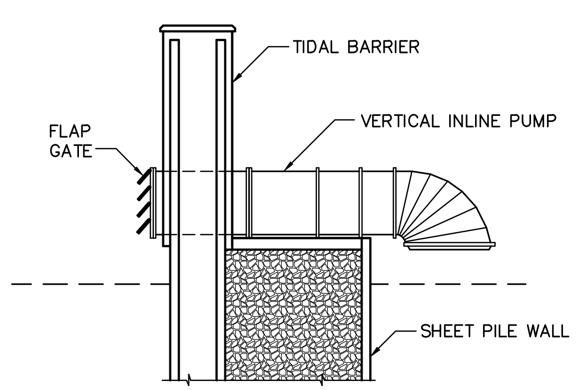
8-4







SECTION A–A



SECTION B–B

25' 0' 25' 50'
GRAPHIC SCALE

moffatt & nichol
FUGRO

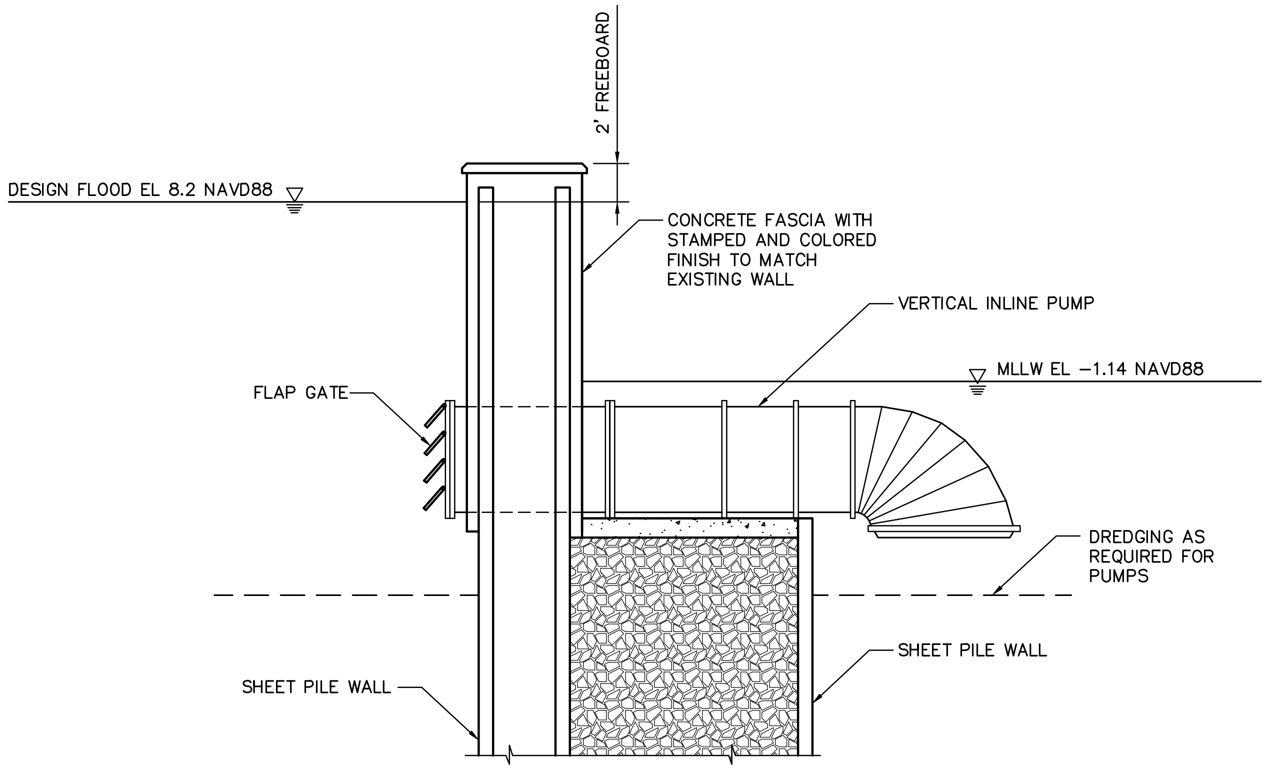
DSGN	DR KOONS	CHK
JOB NO. 6822–03	SCALE 1"=50'	DATE 12–14–2010

COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

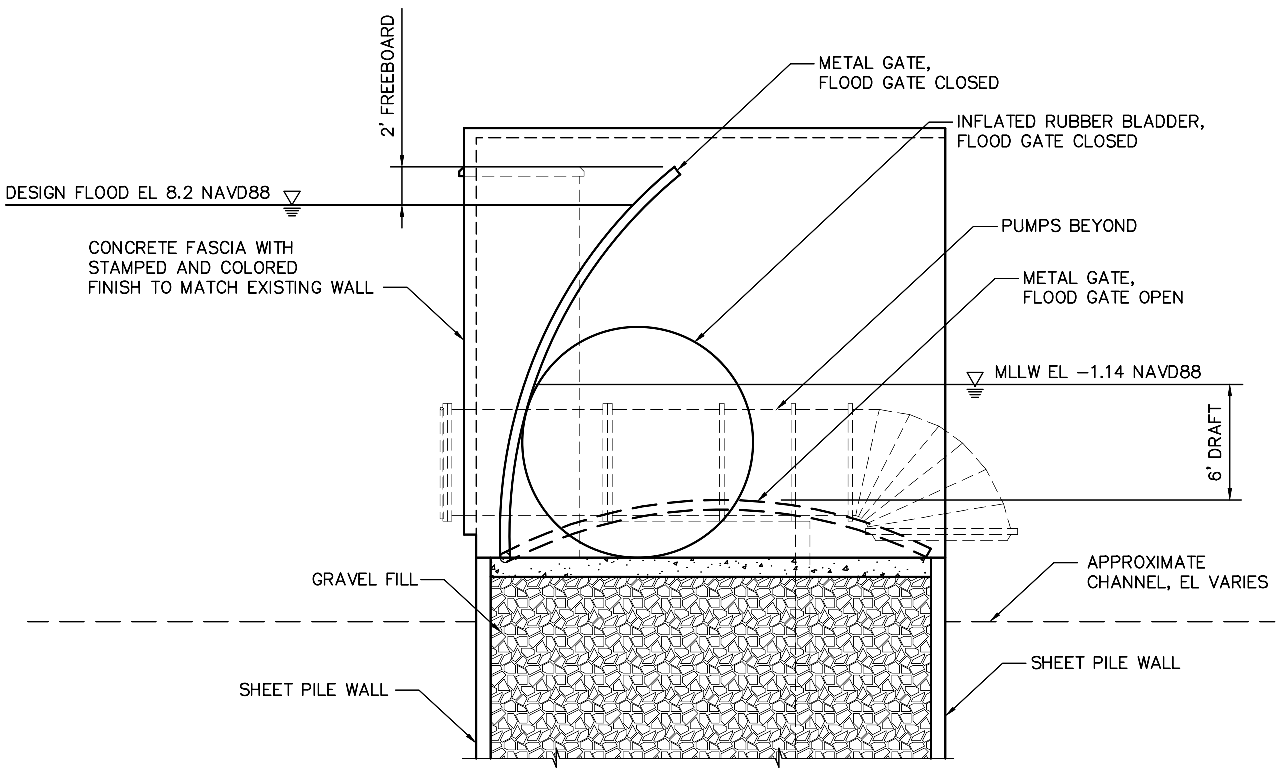
THE HAGUE

BRAMBLETON AVENUE BRIDGE
OBERMEYER GATE OPTION

FIGURE
8-5



WALL SECTION AT PUMPS



SECTION AT OBERMEYER GATE

moffatt & nichol

FUGRO

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

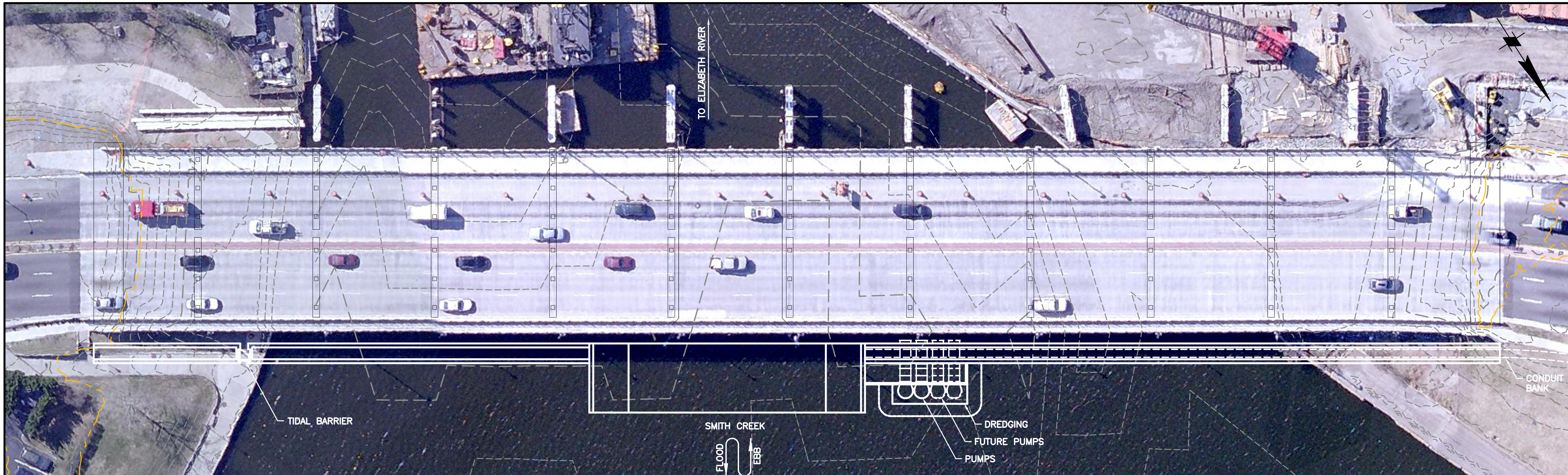
COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

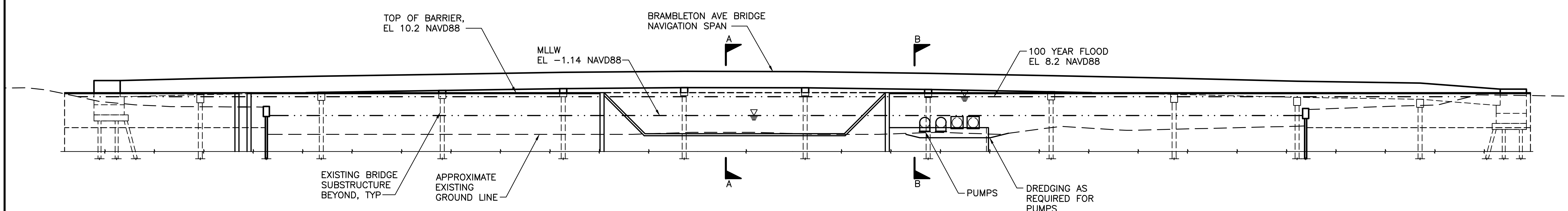
THE HAGUE

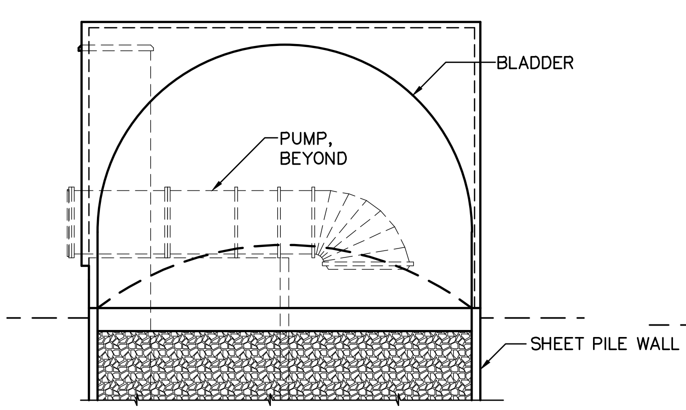
BRAMBLETON AVENUE BRIDGE
OBERMEYER GATE OPTION – SECTIONS

FIGURE

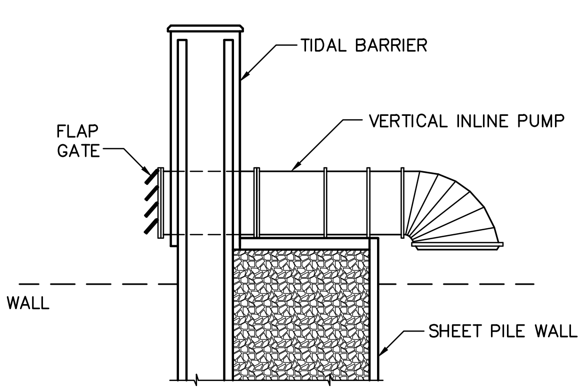
8-6







SECTION A–A



SECTION B–B

25' 0' 25' 50'
GRAPHIC SCALE

moffatt & nichol
FUGRO

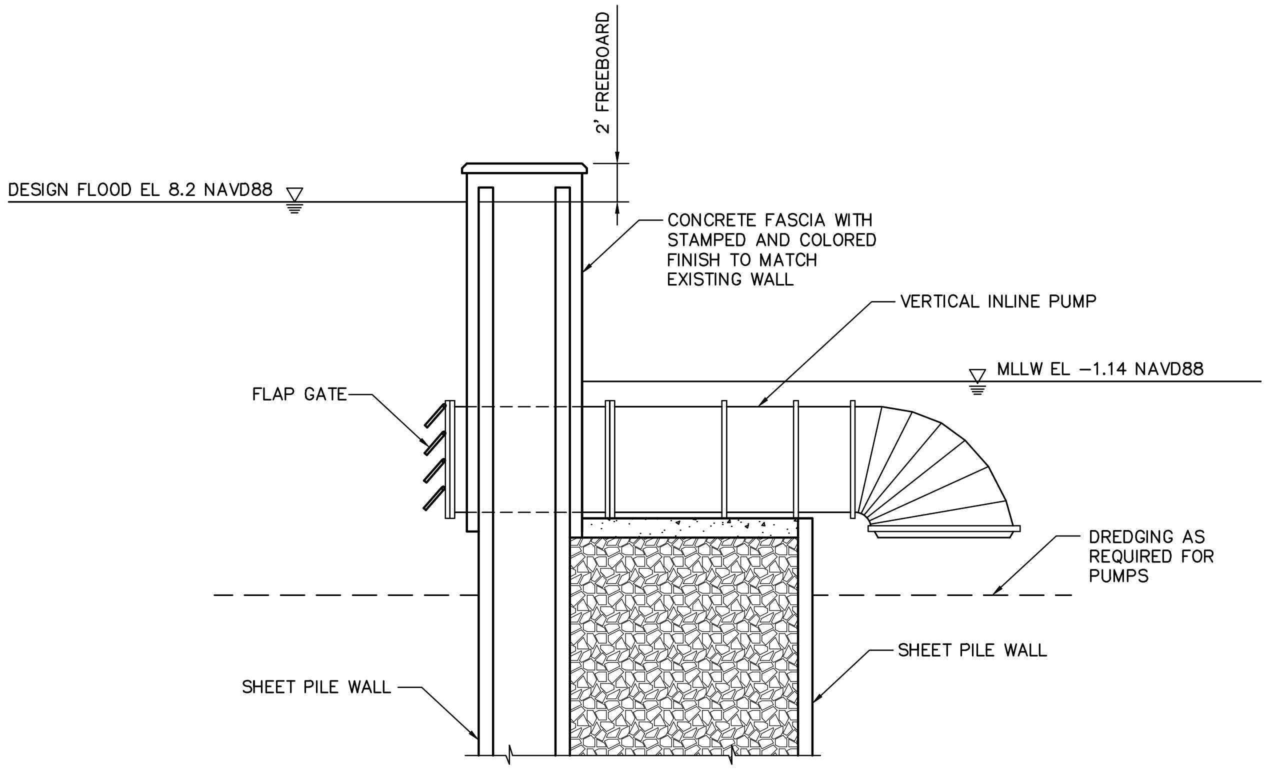
DSGN	DR KOONS	CHK
JOB NO. 6822–03	SCALE 1"=50'	DATE 12–14–2010

COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

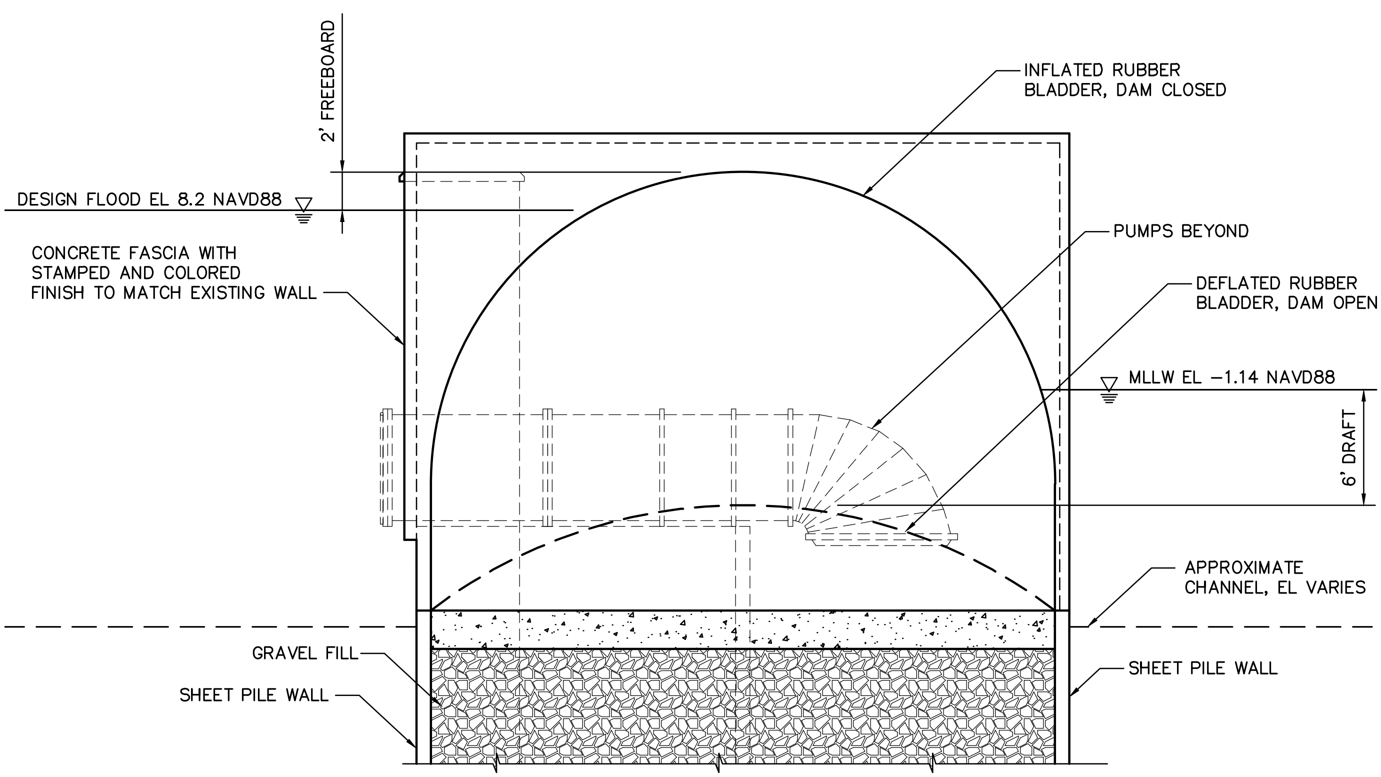
THE HAGUE

BRAMBLETON AVENUE BRIDGE
INFLATABLE DAM GATE OPTION

FIGURE
8-7



WALL SECTION AT PUMPS



SECTION AT INFLATABLE BLADDER

moffatt & nichol

FUGRO

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

TOWN · 1682 · BOROUGH · 1736 · CITY · 1845 · CITY OF NORFOLK · VIRGINIA · ET TERRA ET MARE DIVITIAE TUAE · CRESCAS

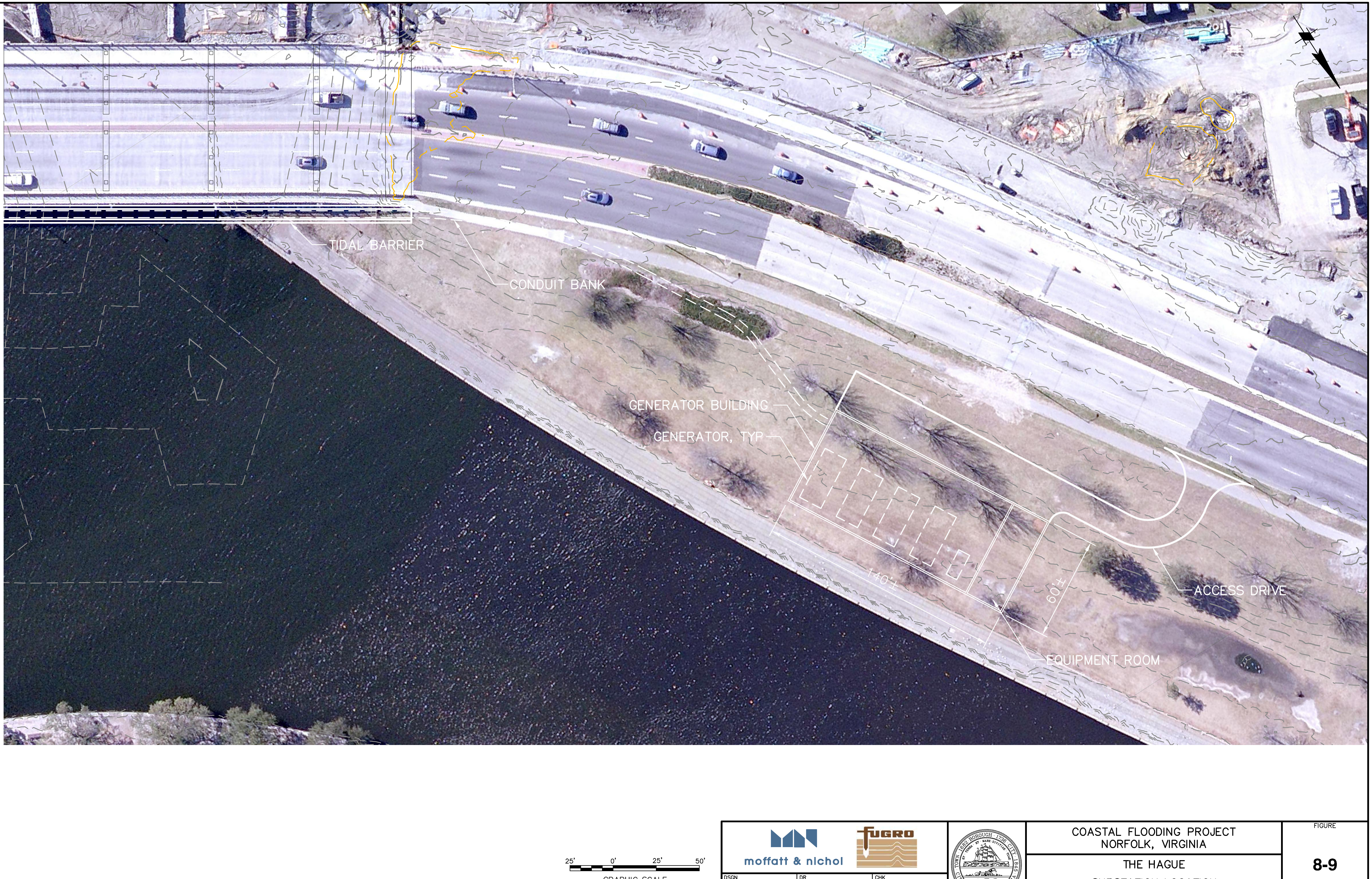
COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

THE HAGUE

BRAMBLETON AVENUE BRIDGE
INFLATABLE DAM OPTION – SECTIONS

FIGURE

8-8



moffatt & nichol

FUGRO

DSGN	DR KOONS	CHK .
JOB NO. 6822-03	SCALE 1"=50'	DATE 12-14-2010

COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

THE HAGUE
SUBSTATION LOCATION

FIGURE
8-9

BULKHEAD
BERM

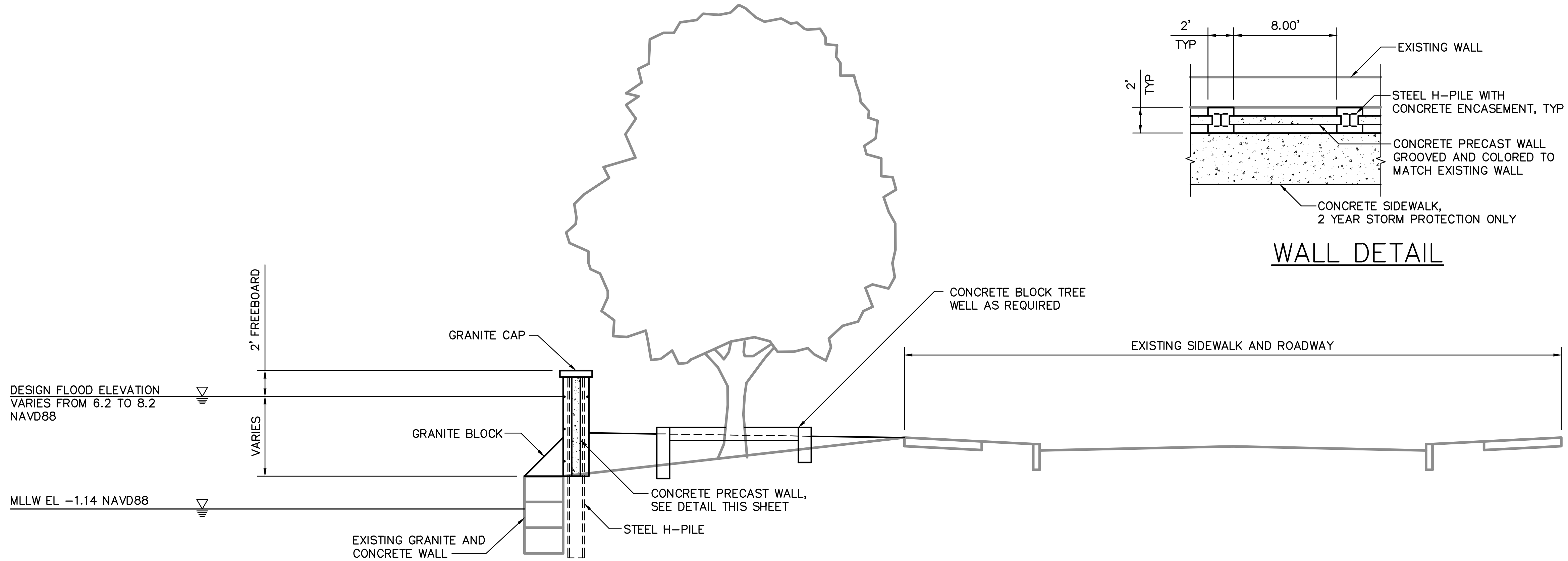
moffatt & nichol
FUGRO

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

COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

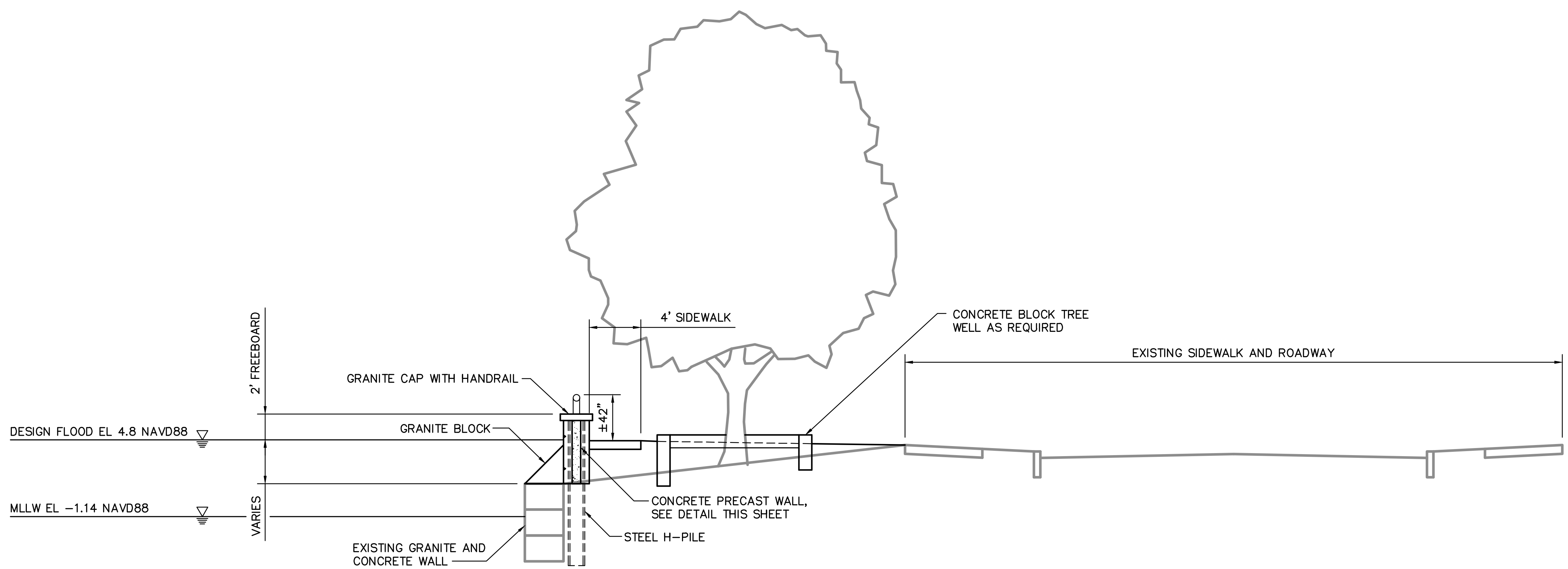
THE HAGUE
BULKHEAD WALL AND EARTHEN BERM

FIGURE
8-10



WALL DETAIL

10 – 100 YEAR STORM PROTECTION



2 YEAR STORM PROTECTION

5' 0' 5' 10'
GRAPHIC SCALE

moffatt & nichol

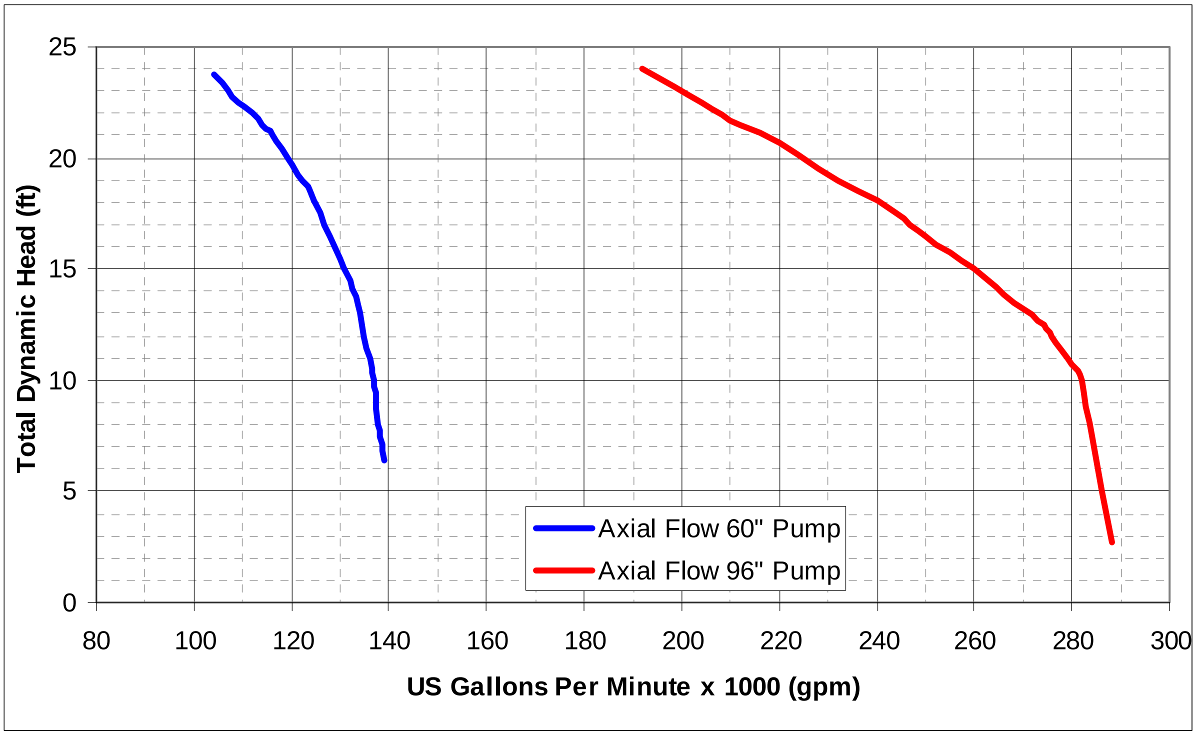
FUGRO

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

COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

THE HAGUE
BULKHEAD WALL TYPICAL SECTIONS

FIGURE
8-11



PUMP CURVES FOR SWMM MODELS

City-wide Coastal Flooding Study

Norfolk, Virginia

FIGURE 9-1

LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N W E S

0 800 1,600 Feet

SWMM RESULTS FOR 10YR 24-HR STORM, TAILWATER = 10YR STORM SURGE ALTERNATIVE: 2 x 60-inch Pumps

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-2

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer

City of Norfolk, Department of Public Works
Project No. 3627.005

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer

LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
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- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

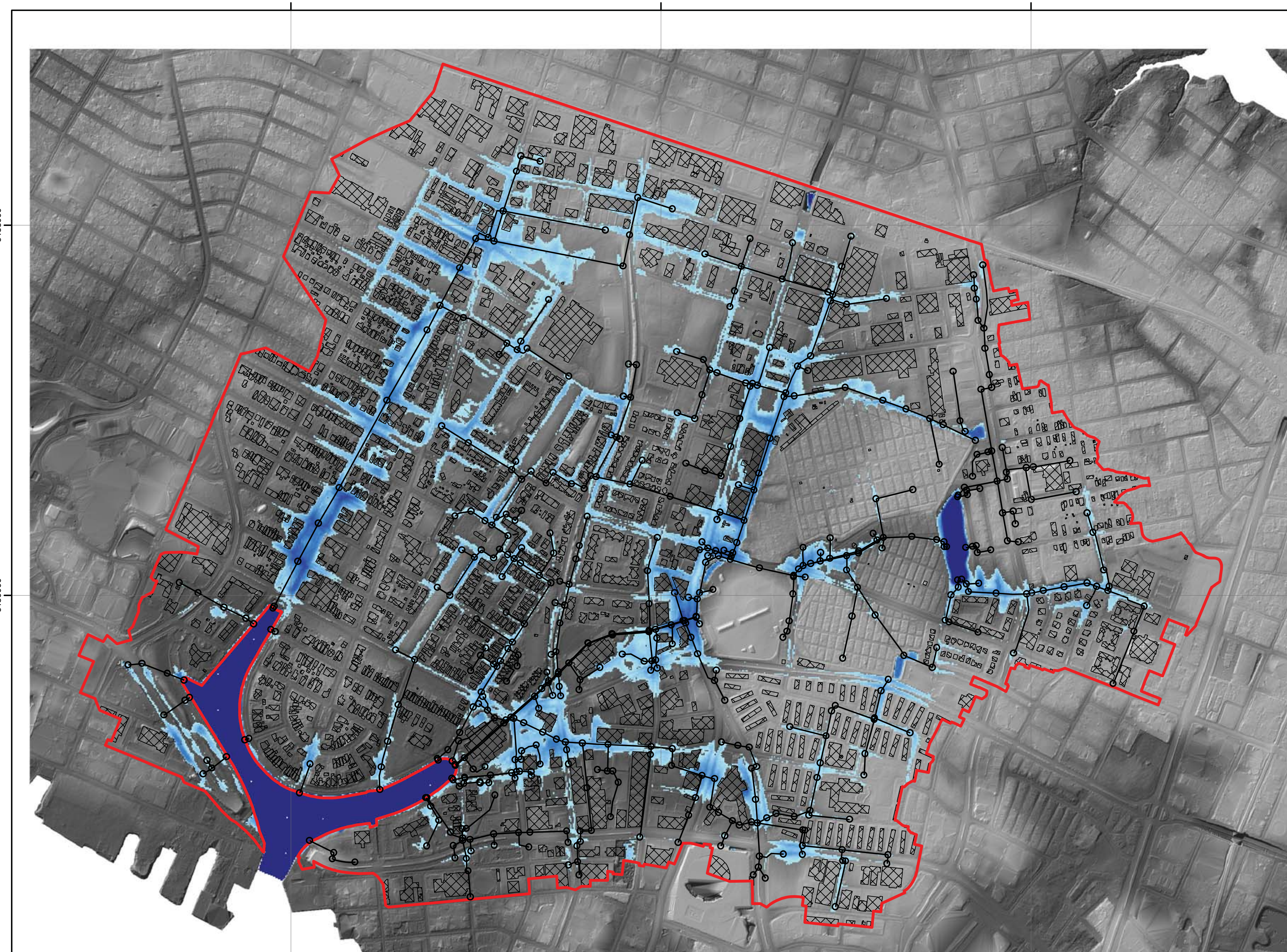
0 800 1,600 Feet

SWMM RESULTS FOR 10YR 24-HR STORM, TAILWATER = 10YR STORM SURGE ALTERNATIVE: 4 x 60-inch Pumps

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-3

City of Norfolk, Department of Public Works
Project No. 3627.005



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LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

0 800 1,600 Feet

SWMM RESULTS FOR 10YR 24-HR STORM,
TAILWATER = 10YR STORM SURGE
ALTERNATIVE: 4 x 96-inch Pumps
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-4

City of Norfolk, Department of Public Works
Project No. 3627.005

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE
ALTERNATIVE: 2 x 60-inch Pumps
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-5

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer

City of Norfolk, Department of Public Works
Project No. 3627.005

LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

0 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1.0 - 1.25
1.25 - 1.5
1.5 - 1.75
1.75 - 2
2.0 - 2.25
2.25 - 2.5
2.5 - 2.75
2.75 - 3
3.0 - 3.25
3.25 - 3.5
3.5 - 3.75
3.75 - 4
4.0 - 4.25
4.25 - 4.5
4.5 - 4.75
4.75 - 10

Notes:
1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

0 800 1,600 Feet

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE
ALTERNATIVE: 4 x 60-inch Pumps
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-6

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer

LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

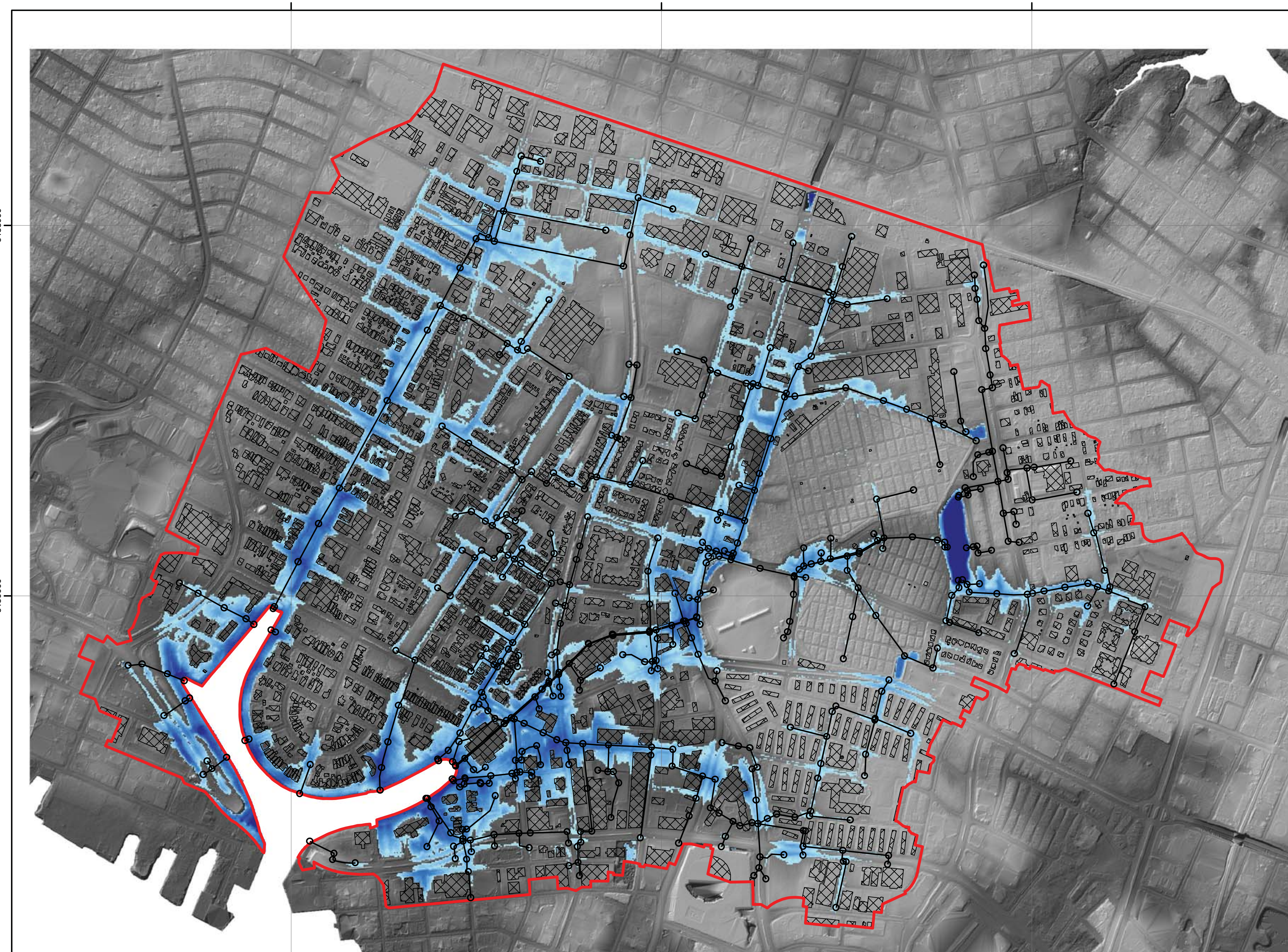
0 800 1,600 Feet

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE ALTERNATIVE: 4 x 96-inch Pumps

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-7

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

0 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1.0 - 1.25
1.25 - 1.5
1.5 - 1.75
1.75 - 2
2.0 - 2.25
2.25 - 2.5
2.5 - 2.75
2.75 - 3
3.0 - 3.25
3.25 - 3.5
3.5 - 3.75
3.75 - 4
4.0 - 4.25
4.25 - 4.5
4.5 - 4.75
4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

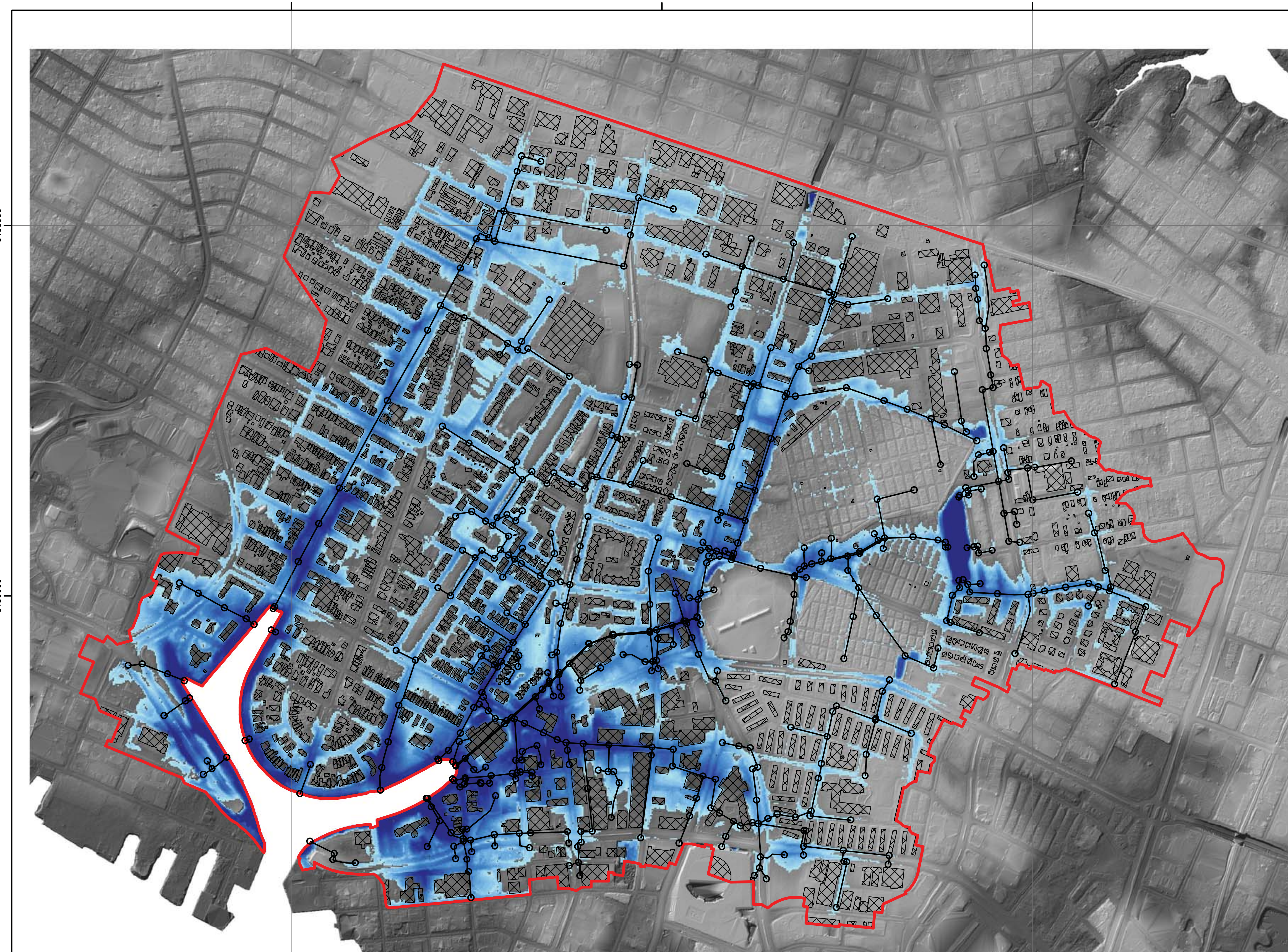
N W E S

0 800 1,600 Feet

SWMM RESULTS FOR 10YR 24-HR STORM, TAILWATER = 10YR STORM SURGE ALTERNATIVE: BULKHEAD WALL

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-8



LEGEND

- Model Nodes
- Model Links
- Hague Watershed Boundary
- Hague Buildings

Max Depth (ft)

- 0 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1
- 1.0 - 1.25
- 1.25 - 1.5
- 1.5 - 1.75
- 1.75 - 2
- 2.0 - 2.25
- 2.25 - 2.5
- 2.5 - 2.75
- 2.75 - 3
- 3.0 - 3.25
- 3.25 - 3.5
- 3.5 - 3.75
- 3.75 - 4
- 4.0 - 4.25
- 4.25 - 4.5
- 4.5 - 4.75
- 4.75 - 10

Notes:

1. City digital elevation model (DEM) hillshade relief generated from 2009 LiDAR survey conducted by Pictometry, Inc. under contract to the City of Norfolk.

N
W E
S

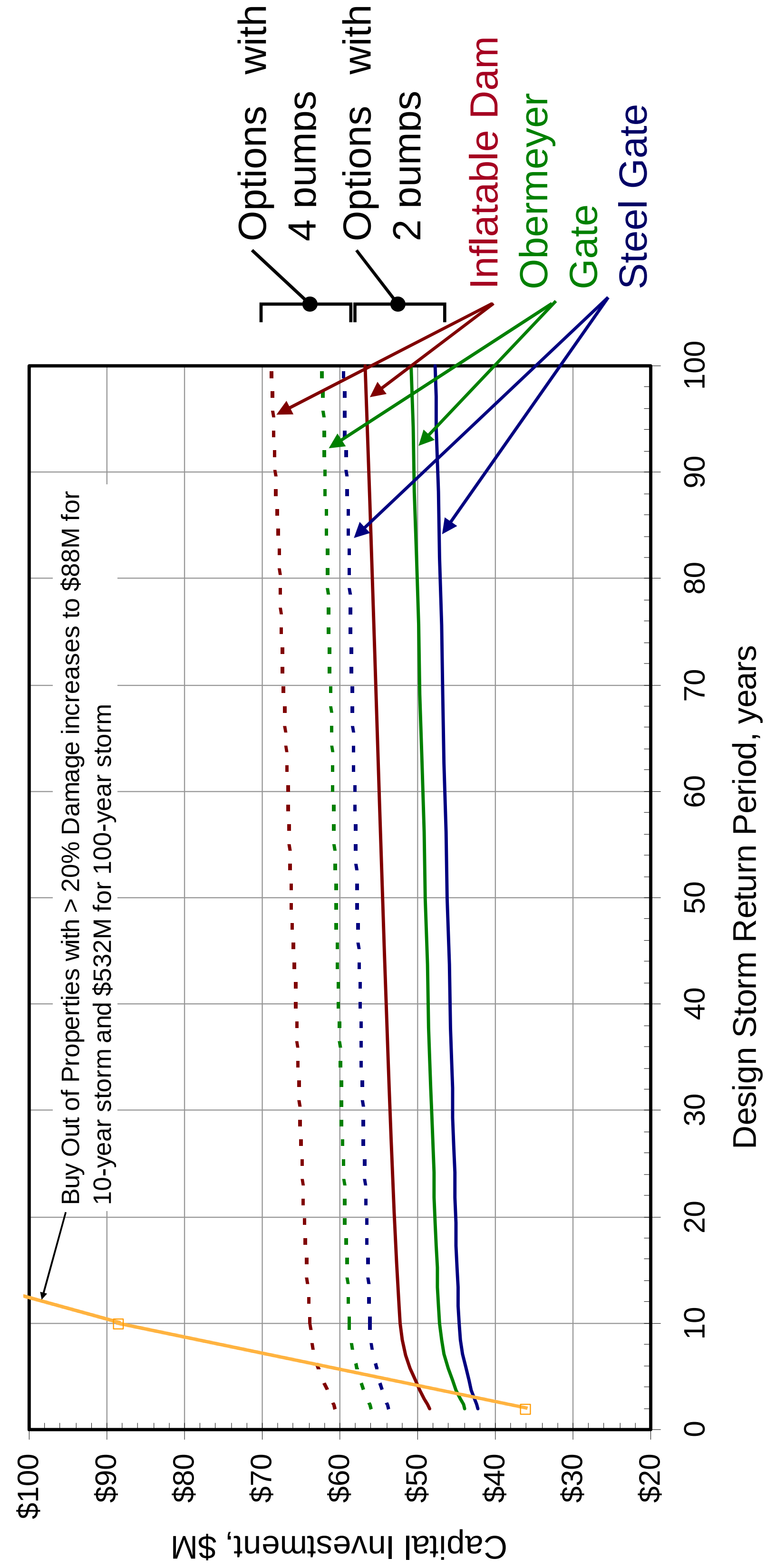
0 800 1,600 Feet

SWMM RESULTS FOR 100YR 24-HR STORM, TAILWATER = 100YR STORM SURGE ALTERNATIVE: BULKHEAD WALL

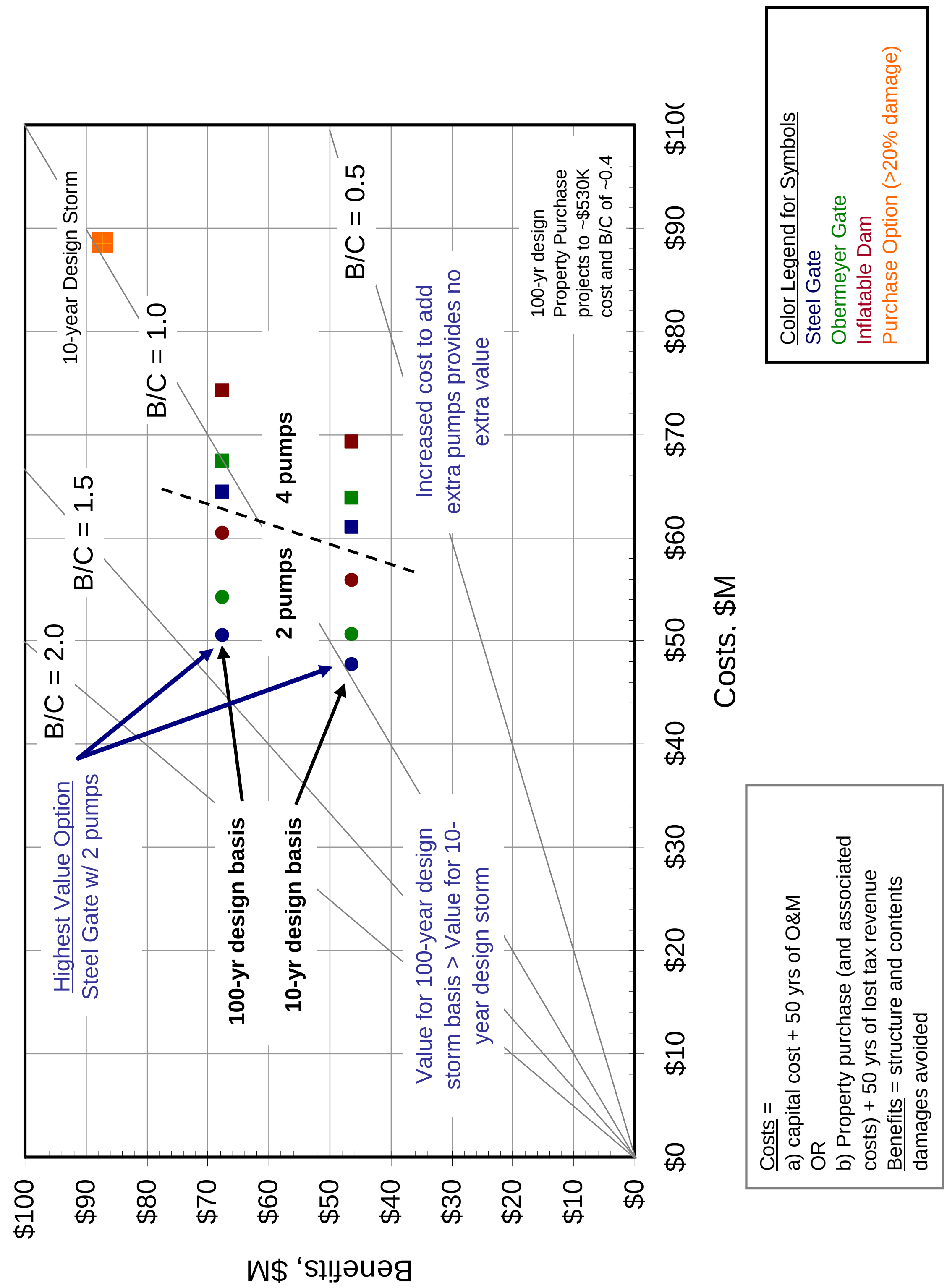
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 9-9

N:\Projects\3627_City_Norfolk\3627-005_Hague\Outputs\2010_12_13_Draft_Flood_Mitigation_Report\mxd\11x17_Figure_Template.mxd, 01/14/11, kspencer



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



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

FIGURE 11-2