

FLOOD MITIGATION ALTERNATIVES EVALUATION PRETTY LAKE WATERSHED

City of Norfolk City-wide Coastal Flooding Contract Work Order No. 6

Prepared for:
CITY OF NORFOLK
DEPARTMENT OF PUBLIC WORKS

April 2011
Fugro Project No. 3627.006





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

Subject: Flood Mitigation Alternatives Evaluation - Pretty Lake Watershed, City of Norfolk,
City-wide Coastal Flooding Project, Work Order No. 6

Dear Mr. White:

Enclosed is Fugro Atlantic's report documenting our flood mitigation alternatives evaluation for the Pretty Lake Watershed. This study and report were authorized by Work Order #6, 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 Pretty Lake. 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 Pretty Lake area of the City.

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



Sincerely,

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

Kevin R. Smith
Senior Engineering Geologist/Project Manager

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

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

Enclosure:

Copies Submitted: (#)

Draft

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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 Pretty Lake 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 PRETTY LAKE

The Pretty Lake watershed includes the East Ocean View residential/commercial community, Bayview neighborhoods, and the Camellia neighborhoods. The area borders a tidal estuary known as Pretty Lake that is the western tributary to Little Creek. The watershed (catchment area) from which storm water runoff discharges into Pretty Lake is hereinafter referred to as "The Pretty Lake Watershed".

Flooding in The Pretty Lake Watershed is caused by the combined effects of "high tides" and heavy precipitation. The effects of these "high tides" (coastal flooding) are expected to worsen over time as mean sea level rises. This study demonstrates that a flood gate on the upstream (western side) of Shore Drive, in combination with raising grades along the perimeter of the Pretty Lake Watershed, could effectively mitigate about two thirds of the estimated flood damage risk over the design life of the project. The projected benefits of the recommended flood defense improvements are estimated to be more than twice the estimated cost.

Other methods that were considered to mitigate the risk of flood damage include property buyout and elevation of structures. Except for a few isolated locations associated with raising street and road elevations, elevation of structures was not found to be a viable mitigation option. Likewise, the costs of property buyout would render this option impractical. Improvements to the existing storm sewerage system were also considered. However, the hydrologic modeling performed as part of this study shows that the existing system is hydraulically adequate for a 10-year precipitation event. Some flooding mitigation could be achieved by improving the existing storm sewerage system; however, the existing system appears to meet current City standards for collection and discharge of at least the amount of runoff from a 10-year storm.

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 evaluations 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 Pretty Lake 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. 6 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.

The tide gauge at the Little Creek Recreation Center is located within the Pretty Lake drainage. 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 0.1-foot below those at Sewells Point.

The differences of the tide level offset between the local tide gauge and Sewells Point can be due to many local factors, such as wind driven setup (which varies with wind direction and location), localized storm water discharge effects, and local geometric amplifications the effects of wind direction and local geometric amplification (e.g., cove effects). For design applications it is appropriate to consider those temporally variations between the local tide and those at Sewells Point. A 0.4-foot decrease in tailwater elevations is recommended for the Pretty Lake 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 Pretty Lake watershed is in the northeast portion of the City (Figure 1-1). The watershed includes 7,721 parcels within the 2,545 acres of land in the watershed. Approximately 22,650 residents of the City live within the drainage basin (as defined by the City's Planning Department).

Topography

The topography of the Pretty Lake 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 22 percent of the study area lies below El. 8 feet NAVD88. The southern and eastern portions of the watershed's ground surface slopes gently to the north into Pretty Lake. The northern area of the watershed is predominantly low lying and flat with the exceptions of a few high mounds.

The watershed is made up of several surface drainage systems that trend north and northeast. The low lying areas in the south are adjacent to the small drainages feeding Pretty Lake. In the north area of the watershed the topography is primarily flat between Pretty Lake and the Chesapeake Bay, with the exception of a few high knolls. In general, the ground surface slope varies throughout the watershed.

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	4,756	4,756	4.6	4.6
3 to 4	2,864	7,620	2.8	7.4
4 to 5	3,665	11,285	3.6	11.0
5 to 6	3,283	14,568	3.2	14.2
6 to 7	3,298	17,866	3.2	17.4
7 to 8	4,581	22,447	4.5	21.9
8 to 9	6,791	29,237	6.6	28.5
9 to 10	10,186	39,423	9.9	38.4
10 to 11	15,209	54,633	14.8	53.2
11 to 12	17,878	72,511	17.4	70.6
12 to 13	17,080	89,591	16.6	87.3
13 to 14	9,309	98,900	9.1	96.3
14 to 15	2,863	101,763	2.8	99.1
15 to 25	847	102,610	0.8	99.9

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 Pretty Lake watershed. As can be seen from the table below, the watershed is primarily residential, and low density residential is the majority land use type. Commercial, Open Space/Recreational, and Vacant land uses are fairly equal.

Table 3-2. Pretty Lake Watershed Land Use Classifications

Usage	Number of Acres	Percent of Watershed
Low Density Residential	1330	60.4
Medium Density Residential	130	5.9
High Density Residential	149	7.4
Commercial	148	6.8
Institutional	50	2.3
Open Space/Recreational	192	8.7
Transportation/Utility	5	0.2
Military	76	3.4
Vacant	109	4.9

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

Receiving Water Body

The Pretty Lake, formally known as Little Creek, is the receiving body of water which subsequently feeds into the Chesapeake Bay. Both bodies of water are tidally influenced and subject to storm surges.

BASIN RIM

The perimeter of the watershed is about 69,200 feet (13.2 miles). The perimeter is delineated by the Shore Drive Bridge on the east and the sand dunes along the Ocean View beaches to the north. On the western perimeter the watershed runs roughly along Chesapeake Boulevard and surrounding side streets through the neighborhoods. The southern rim of the watershed roughly follows Little Creek Road

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)
1.6	4	97
3.6	9	206
4.2	10	146
5.0	13	265
5.6	11	344
6.4	26	668
7.0	24	956
7.6	38	974

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 a hundred to almost a thousand 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 Shore Drive Bridge over Pretty Lake. The shoreline along the outlet has been slightly modified by the construction of the bridge and the marina on the southeastern shore and the condominiums on the southwestern shore. Figure 4-1 compares conditions at the basin outlet depicted in an historical aerial photograph from 1937 and a 2009 aerial photograph. The existing bridge was built in 2002, but there may be remnants of the former present in the subsurface and present obstructions for future subsurface structures (e.g. piles, sheetpile walls, etc.).

NAVIGATION REQUIREMENTS

Pretty Lake is actively used by small craft. The City has specified that the channelized entrance to Pretty Lake should provide a minimum draft of 4 feet, relative to MLLW Datum. That elevation corresponds to El. -6.7 feet re: NAVD88 Datum.

SUBSURFACE CONDITIONS

Fugro compiled and reviewed available information relative to the subsurface conditions. Primary sources of information were 1999 boring logs from Shore Drive Bridge. 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. Figure 4-2 presents the cross section 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, fine to coarse-grained alluvium, Pliocene age Yorktown Formation. The artificial fill is assumed to be associated with development and construction of the Shore Drive Bridge. Exploration logs suggest the material is primarily sand soils. The artificial fill is less than 10 feet thick underneath Pretty Lake but is thicker to the north and south.

Fine to coarse-grained alluvium underlies the artificial fill. The alluvium has two distinct layers; a loose to dense sand layer over a fine-grained-very loose sand layer. The loose to dense sand layer is considered as fluvial-estuarine sediment and varies in thickness between 15 and 30 feet. The fine-grained sand layer varies in thickness between 5 and 25 feet and is encountered at a depth of 25 to 30 feet.

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.

Design Subsurface Profiles for Concept Evaluation

In order to conceptually evaluate possible flood mitigation systems at Pretty Lake, 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;
- Design strength parameters including undrained shear strength and friction angles;
- 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 clay and silt layer.

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

Draft

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	-0.1	-0.1
Wind Direction and/or Cove Offset	0.5	0.4
Allowance for Future Sea Level Rise	1.0	1.4

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 Pretty Lake watershed (Fugro, 2010).

Table 5-2. Tailwater Elevations at Sewells Point and the Pretty Lake Watershed

Return Period (years)	Sewells Point Water Level (ft, NAVD88)	Pretty Lake Watershed Design Tailwater Elevation (ft, NAVD88)
MHHW	1.2	1.6
1	3.2	3.6
2	3.8	4.2
5	4.6	5.0
10	5.2	5.6
25	6	6.4
50	6.6	7.0
100	7.2	7.6

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 Pretty Lake watershed equal to elevation +7.6 and +5.6feet (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 +7.6-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 +7.6-ft crest elevation corresponds to approximately a 31-year return period event.

The protection associated with an elevation +5.6-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 +5.6-ft crest elevation corresponds to approximately a 3-year return period event.

Given the watershed topography for Pretty Lake, 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 1.5 ft freeboard. 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 7.6 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	1.6
2yr Storm, MHHW Tide	3.57	1.6
10yr Storm, MHHW Tide	5.51	1.6
25yr Storm, MHHW Tide	6.82	1.6
50yr Storm, MHHW Tide	7.96	1.6
100yr Storm, MHHW Tide	9.21	1.6
1yr Storm, 1yr Storm Surge	2.93	3.6
2yr Storm, 2yr Storm Surge	3.57	4.2
10yr Storm, 10yr Storm Surge	5.51	5.6
25yr Storm, 25yr Storm Surge	6.82	6.4
50yr Storm, 50yr Storm Surge	7.96	7.0
100yr Storm, 100yr Storm Surge	9.21	7.6

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:

$$Q \text{ (cfs)} = \text{coefficient} \times 2D \text{ cell depth (ft)} ^ \text{exponent}$$

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

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

- Rainfall: time series of rainfall,
- Subcatchment Data: area, overland flow, % slope, % impervious, curve number,
- Junction Data: inverts, depth, ponded area,
- Conduit Data: shape, size, length, roughness, inverts, loss coefficients,
- Outfall-inverts, tide gate, tidal boundary condition,
- Building footprints within the Pretty Lake 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 Pretty Lake drainage area was divided into 116 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 individual subcatchments. 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 Pretty Lake waterbody was included in the model as part of the 2D hydrodynamic grid. Therefore, the outfalls that drain water from the watershed into Pretty Lake 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 Shore Drive Bridge (US HWY 60), where Pretty Lake outlets to the Chesapeake Bay. The boundary condition water surface elevation was based on the recurrence interval tailwater elevations for the Little Creek Recreation Center (-0.1 ft compared to Sewells Point), derived from the NOAA Station 8638610 at Sewells Point (Fugro, 2010) with the additional 0.5-ft increase due to 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 20-ft x 20-ft grid size DEM that was used in the SWMM model for Pretty Lake.

MODEL CALIBRATION

Detailed calibration data were not available for the Pretty Lake 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 Pretty Lake 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 Pretty Lake outlets at the Shore Drive Bridge equal to MHHW. MHHW for Pretty Lake was determined to be +1.6-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 6-1 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 ~10yr, 24hr rainfall with the tailwater at MHHW.

Table 6-1. Existing Condition SWMM Results

Pretty Lake 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	404.7	58.9	113.1	0.52	1.1
2yr Storm, MHHW Tide	536.0	86.7	160.9	0.54	1.3
10yr Storm, MHHW Tide	954.0	175.3	287.6	0.61	1.7
25yr Storm, MHHW Tide	1240.4	237.5	361.5	0.66	2.1
50yr Storm, MHHW Tide	1493.1	287.7	408.1	0.70	2.4
100yr Storm, MHHW Tide	1771.0	343.7	456.4	0.75	2.8
1yr Storm, 1yr Storm Surge	404.7	112.7	154.8	0.73	4.3
2yr Storm, 2yr Storm Surge	537.3	183.7	222.8	0.82	5.5
10yr Storm, 10yr Storm Surge	954.0	466.5	400.7	1.16	9.8
25yr Storm, 25yr Storm Surge	1246.9	692.6	497.3	1.39	10.9
50yr Storm, 50yr Storm Surge	1500.7	896.0	570.9	1.57	11.8
100yr Storm, 100yr Storm Surge	1772.2	1126.3	645.0	1.75	12.8

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 Pretty Lake 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 Pretty Lake 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 Pretty Lake watershed. The project team coordinated with the City to update building footprints based on 2009 aerial photography. Approximately 11,400 buildings were used in the Pretty Lake 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

Pretty Lake Scenario	Number of Buildings Impacted	Structural Damage ^a (\$, millions)	Contents Damage ^a (\$, millions)	Total Damage ^a (\$, millions)
1yr Storm, MHHW Tide	474	2.71 (0.7)	1.7 (0.5)	4.47 (1.3)
2yr Storm, MHHW Tide	698	3.56 (0.9)	2.3 (0.7)	5.86 (1.7)
10yr Storm, MHHW Tide	1,098	7.23 (1.8)	4.61 (1.4)	11.8 (3.33)
25yr Storm, MHHW Tide	1,454	8.95 (2.3)	5.69 (1.8)	14.6 (4.20)
50yr Storm, MHHW Tide	1,706	11.5 (3.08)	7.31 (2.4)	18.8 (5.48)
100yr Storm, MHHW Tide	2,159	15.4 (4.23)	9.76 (3.2)	25.1 (7.52)
1yr Storm, 1yr Storm Surge	474	3.31 (0.8)	2.1 (0.6)	5.43 (1.5)
2yr Storm, 2yr Storm Surge	698	4.88 (1.2)	3.1 (0.9)	7.99 (2.1)
10yr Storm, 10yr Storm Surge	1,364	13.8 (2.9)	8.52 (2.2)	22.3 (5.19)
25yr Storm, 25yr Storm Surge	1,591	20.9 (4.16)	12.5 (3.2)	33.4 (7.45)
50yr Storm, 50yr Storm Surge	1,861	29.7 (5.49)	17.5 (4.34)	47.2 (9.84)
100yr Storm, 100yr Storm Surge	2,159	39.7 (6.28)	23.4 (4.97)	63.1 (11.2)

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

Flood Mitigation Alternative Options	Options Deemed Technically/ Economically Unfeasible	Potentially Feasible Options	Feasibility Explanation
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 Pretty Lake, 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.
- Beach Nourishment - The area in question along Pretty Lake 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 Pretty Lake 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 subscribed Xa, subscribed Xb and subscribed Xc is as follows:

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

Table 8-2. Pretty Lake Alternatives

Alternative	Category
1a, 1b, 1c	Tidal Barrier with Tide Gate, 2- 60" Dia. Pumps , and Road Raise
2a, 2b, 2c	Tidal Barrier with Tide Gate, 4 - 60" Dia. Pumps, and Road Raise
3a, 3b, 3c	Tidal Barrier with Tide Gate, 4 - 96" Dia. Pumps, and Road Raise
4	Bulkhead Wall and Earthen Berm and Road Raise
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	4.2	5.7
10yr, 10yr	5.6	6.1
25yr, 25yr	6.4	7.9
50yr, 50yr	7.0	8.5
100yr, 100yr	7.6	9.1

*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 Road Raise

Alternatives 1 through 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
- Road raise at low lying areas of the basin/watershed perimeter

Tidal Barrier Structures with Tide Gate

The tidal barrier and tide gate will be constructed on the upstream side of the Shore Drive Bridge. The overall length of the barrier is approximately 400 LF and will tie into the existing elevations of the surrounding environment. Given the soil conditions within this area of Pretty Lake, 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 -6 (NAVD 88) which will allow small boat traffic to access Pretty Lake 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 adjacent bulkhead to be an additional 2.3' higher for the pockets than Table 8-2 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 Pretty Lake community, all electrical components including the generators will be housed in an aesthetically pleasing structure.

Road Raise

Road raising and utility relocation is anticipated along 2800 linear feet of road on Shore Drive, Pretty Lake Road, and Dunning Road. This work will provide a barrier of Pretty Lake from the bay side of the Chesapeake Bay. In addition to the road raise, several homes between Dunning Road and Pretty Lake are proposed to be raised which will allow their existing floor elevation to be above the flood plain.

Alternative 4 – Bulkhead Wall, Earthen Berm and Road Raise

This alternative includes installing a steel bulkhead and earthen berm along the shoreline at specific locations of Pretty Lake. The location of these structures and elevations for the final height are dependent on the storm event scenarios and its relative surge elevation. Figure 8-10 and 8-11 provide general placement of structures for Alternative 4 for a 10-year and 100-yr surge event. Breakdown in costs per storm event are provided in the “Opinion of Probable Cost” section of the report.

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 Pretty Lake. 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 Pretty Lake watershed during storm events. For the first three alternatives, an artificial barrier was placed in the model where Pretty Lake outlets at the Shore Drive Bridge. Then either two 60-inch pumps, four 60-inch pumps, or four 96-inch pumps were used to drain flood waters out of the lake. These pump sizes were selected based on the magnitude of the pipe flows discharging into Pretty Lake 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 0.8-ft NAVD88 and stopped when the water level fell below -0.1-ft NAVD88. For reference, MTL at the Sewells Point tide gage is roughly -0.3-ft NAVD88.

The fourth alternative simulated the construction of a bulkhead wall and earthen berm around Pretty Lake, which prevented storm surges from flooding onto the lower-lying areas adjacent to the water. In this scenario, the lake was removed from the 2D model grid and the shoreline acted as the 2D grid boundary. The outfalls which drain into Pretty Lake 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 to carry at best approximately a 2-yr to 10-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 Pretty Lake 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

Pretty Lake 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"	954.0	175.6	291.7	0.60	2.0
10yr, 10yr 4x60"	954.0	175.1	291.5	0.60	2.0
10yr, 10yr 2x60"	954.0	180.7	293.6	0.62	2.1
100yr, 100yr 4x96"	1772.2	336.4	453.8	0.74	3.3
100yr, 100yr 4x60"	1772.2	361.7	464.2	0.78	3.7
100yr,100yr 2x60"	1772.2	390.8	473.6	0.83	4.6
Change vs. Existing Conditions					
10yr, 10yr 4x96"	-	-64.4%	-27.2%	-48.3%	-79.9%
10yr, 10yr 4x60"	-	-62.5%	-27.2%	-48.4%	-80.0%
10yr,10yr 2x60"	-	-61.3%	-26.7%	-47.1%	-78.8%
100yr, 100yr 4x96"	-	-70.1%	-29.6%	-57.6%	-74.5%
100yr, 100yr 4x60"	-	-67.9%	-28.0%	-55.4%	-71.4%
100yr, 100yr 2x60"	-	-65.3%	-26.6%	-52.7%	-64.5%

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

Pretty Lake 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	954.0	789.9	526.4	1.50	3.4
100yr, 100yr	1772.2	1477.5	785.9	1.88	5.5
Change vs. Existing Conditions					
10yr, 10yr	-	69.3%	31.4%	28.9%	-65.2%
100yr, 100yr	-	31.2%	21.8%	7.7%	-57.2%

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 and actually worsened flooding compared to existing condition due to storm water accumulating behind the wall. The pump alternatives blocked storm surges at the Shore Drive Bridge with a tidal barrier, but also affected the tailwater condition at the outfalls of the storm drain system by allowing Pretty Lake to be pumped down to elevations within normal tidal range. During the pump-alternative SWMM simulations, the water level at the pump-intakes was maintained at an elevation 1 to 1.5 feet below MHHW (0 to 0.5 ft NAVD88), and the center of Pretty Lake was maintained at an elevation of 1 to 2 ft NAVD88. This reduction in tailwater elevation, compared to the corresponding storm surge elevation for the simulated event, 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

Pretty Lake Scenario	Change in Max Flood Volume		Change in Max Flooded Area		Change in Average Max Flood Depth		Change in Average Duration of Flooding	
	(vs. Existing)		(vs. Existing)		(vs. Existing)		(vs. Existing)	
	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall	Pumps	Bulkhead Wall
10yr, 10yr	-62%	69%	-27%	31%	-48%	29%	-80%	-65%
100yr, 100yr	-68%	31%	-28%	22%	-55%	8%	-70%	-57%

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)	6.01 (1.5)	13.6 (3.50)	-56%	-65%
2a, 2b, 2c (4 x 60" Pumps)	6.11 (1.6)	13.1 (3.41)	-56%	-67%
3a, 3b, 3c (4 x 96" Pumps)	6.11 (1.6)	12.8 (3.34)	-56%	-68%
4	8.68 (2.2)	20.2 (5.06)	-38%	-49%
5	N/A	N/A	N/A	N/A
Estimated Contents Damages, millions				
1a, 1b, 1c (2 x 60" Pumps)	3.84 (1.2)	8.66 (2.7)	-55%	-63%
2a, 2b, 2c (4 x 60" Pumps)	3.90 (1.2)	8.35 (2.6)	-54%	-66%
3a, 3b, 3c (4 x 96" Pumps)	3.90 (1.2)	8.19 (2.5)	-54%	-65%
4	5.52 (1.7)	12.7 (3.9)	-35%	-46%
5	N/A	N/A	N/A	N/A
Estimated Structure and Contents Damages, millions				
1a, 1b, 1c (2 x 60" Pumps)	9.86 (2.80)	22.3 (6.20)	-56%	-65%
2a, 2b, 2c (4 x 60" Pumps)	10.0 (2.85)	21.4 (6.04)	-55%	-67%
3a, 3b, 3c (4 x 96" Pumps)	10.0 (2.85)	21.0 (5.92)	-55%	-46%
4	14.2 (4.06)	32.9 (8.97)	-36%	-48%
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	\$34.9	\$38.4
1b	\$37.6	\$41.9
1c	\$41.3	\$46.1
2a	\$46.9	\$50.5
2b	\$49.4	\$53.8
2c	\$53.3	\$57.6
3a	\$81.5	\$85.0
3b	\$84.1	\$88.5
3c	\$90.4	\$94.1
4	\$94.7	\$189.7
5	\$174.24	\$473.7

Based on the Opinion of Probable Cost breakdown, the tidal barrier options relative to the type of tide gate had a variance of approximately \$8 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 4 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, and Road Raise	\$232K	\$3.2M
Alt 1b: Tidal Barrier with Obermeyer Gate, 2 - 60" Dia. Pumps, and Road Raise	\$236K	\$3.3M
Alt 1c: Tidal Barrier with Inflatable Dam, 2 - 60" Dia. Pumps, and Road Raise	\$260K	\$3.6M
Alt 2a: Tidal Barrier with Steel Gate, 4 - 60" Dia. Pumps, and Road Raise	\$361K	\$4.9M
Alt 2b: Tidal Barrier with Obermeyer Gate, 4 - 60" Dia. Pumps, and Road Raise	\$365K	\$5.0M
Alt 2c: Tidal Barrier with Inflatable Dam, 4 - 60" Dia. Pumps, and Road Raise	\$389K	\$5.4M
Alt 3a: Tidal Barrier with Steel Gate, 4 - 96" Dia. Pumps, and Road Raise	\$450K	\$6.2M
Alt 3b: Tidal Barrier with Obermeyer Gate, 4 - 96" Dia. Pumps, and Road Raise	\$470K	\$6.5M
Alt 3c: Tidal Barrier with Inflatable Dam, 4 - 96" Dia. Pumps, and Road Raise	\$494K	\$6.8M
Alt 4: Bulkhead Wall, Earthen Berm and Road Raise	\$772K	\$10.6M

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	\$7.65
20% Damage Buyout - 10 Year Storm Event	\$26.45
20% Damage Buyout - 25 Year Storm Event	\$40.29
20% Damage Buyout - 50 Year Storm Event	\$54.16
20% Damage Buyout - 100 Year Storm Event	\$71.91

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 \$38.4M (Steel Gate and 2-60" Pumps) to \$473.7M (Property Buyout) for the 100-Year storm events. 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 BENEFIT - COST (B/C) ANALYSIS FEMA GUIDANCE

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 if the City solicits FEMA funding. We calculated the benefit-cost ratio for the eleven flood mitigation options described in Section 9.0 of the report. Major factors considered in the analyses are:

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

As described in Section 7.0 Flood Damage Estimates, only risk of direct damages to structures and contents were calculated for this particular assessment. If the City solicits FEMA funding, then the risk of damage 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

The probability of damage from a storm event in any given year is inversely related to the return period of the event. For example, a 2-yr event has a factor of 0.5 given that it has an annual probability of occurrence of $1/R = 1/2 = 0.5$. Likewise, a 100-yr event has a probability of $1/100 = 0.01$ of happening in a given year. These probabilities can be used probabilistically to derive annualized damage estimates for pre- and post-project conditions. The difference between the two, in any given year, is the estimated annualized project benefit. For the analyses and evaluations described in this report, estimated annualized benefits are assumed to be constant with respect to the hazard analyses, but are adjusted for the time value of money (inflation) as described below.

Design life of the Mitigation Option

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

Present Value of Project

Based on FEMA and OMB direction a 7% interest rate was used for the present value analyses of benefits and costs. For each mitigation alternative, initial capital costs and ongoing O&M costs were brought to present value. Likewise, using a 7% inflation factor, present-value analysis was applied to the estimated annualized 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 imply

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 Road Raise - is the most cost effective alternative with a Benefit Ratio of 2.14.

Table 11-1. Summary of Benefit-Cost Ratios

Alternative	Estimated Benefit to Cost Ratio	
	10yr, 10yr	100yr, 100yr
1a	1.80	2.14
1b	1.67	1.97
1c	1.52	1.79
2a	1.32	1.61
2b	1.26	1.55
2c	1.17	1.42
3a	0.78	0.98
3b	0.76	0.94
3c	0.70	0.88
4	0.38	0.26
5	0.49	0.25

When interpreting the B/C values presented in Table 11-1, it should be emphasized that a design that anticipates a hydrogeologic event of some perceived probability of occurrence produces benefits that do not directly correspond to prevention of the damages associated with the occurrence of that single event. These B/C values are based on the present value of annualized probabilities of damage, not the damages expected from a single event.

Costs and benefits of the various alternatives

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 1.5 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 results of our analyses show that coastal defense improvements can be used to mitigate the effects of extreme high tides in the Pretty Lake watershed. 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 road raise with a total capital cost of \$38.4. This option will provide protection today for a 100-yr coastal surge level and approximately a 10-yr rainfall event. Variants of this coastal defense alternative were considered and were found to be variously somewhat more costly, no more effective, or were considered to be less reliable. The property buyout alternative can be expected to be unreasonably expensive.

Figure 11-2 shows the relative cost vs. return period for a coastal event. Estimated costs increase only slightly for the range of design storm return periods. These structures are so deep (because of geotechnical considerations) below the existing mudline relative to height above mean high water that the incremental cost of greater height is small relative to overall cost. Accordingly, it makes sense to consider the relatively small costs that would be needed to mitigate the effects of future storm surges that could be higher than those that have occurred in the relatively short historical record used to develop the hydrologic models described in this report.

With the exception of increased height of gate, wall, road, and berm, other alternatives besides the recommended alternative 1a, including a greater number of pumps and different methods to protect against tidal surges, were estimated to cost more without significant improvement in flood hazard mitigation. 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. The higher B/C value for alternative 1a reflects lower costs rather than increased benefits over other alternatives. The hydrologic models developed during this study indicate that the existing upland storm sewerage system is adequate for approximately the 10-yr rainfall event before the inlet and pipe systems become overwhelmed and floodwaters cannot reach Pretty Lake in a hydraulically efficient manner. 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 would have negligible benefit with respect to flood damage mitigation unless improvements are made to the upland storm water collection system. Since the existing system appears to be adequate for the 10-year precipitation event, storm sewerage improvements in the Pretty Lake Watershed would imply a higher standard of storm water management throughout the entire City.

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.

Draft

15.0 REFERENCES

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FIGURES

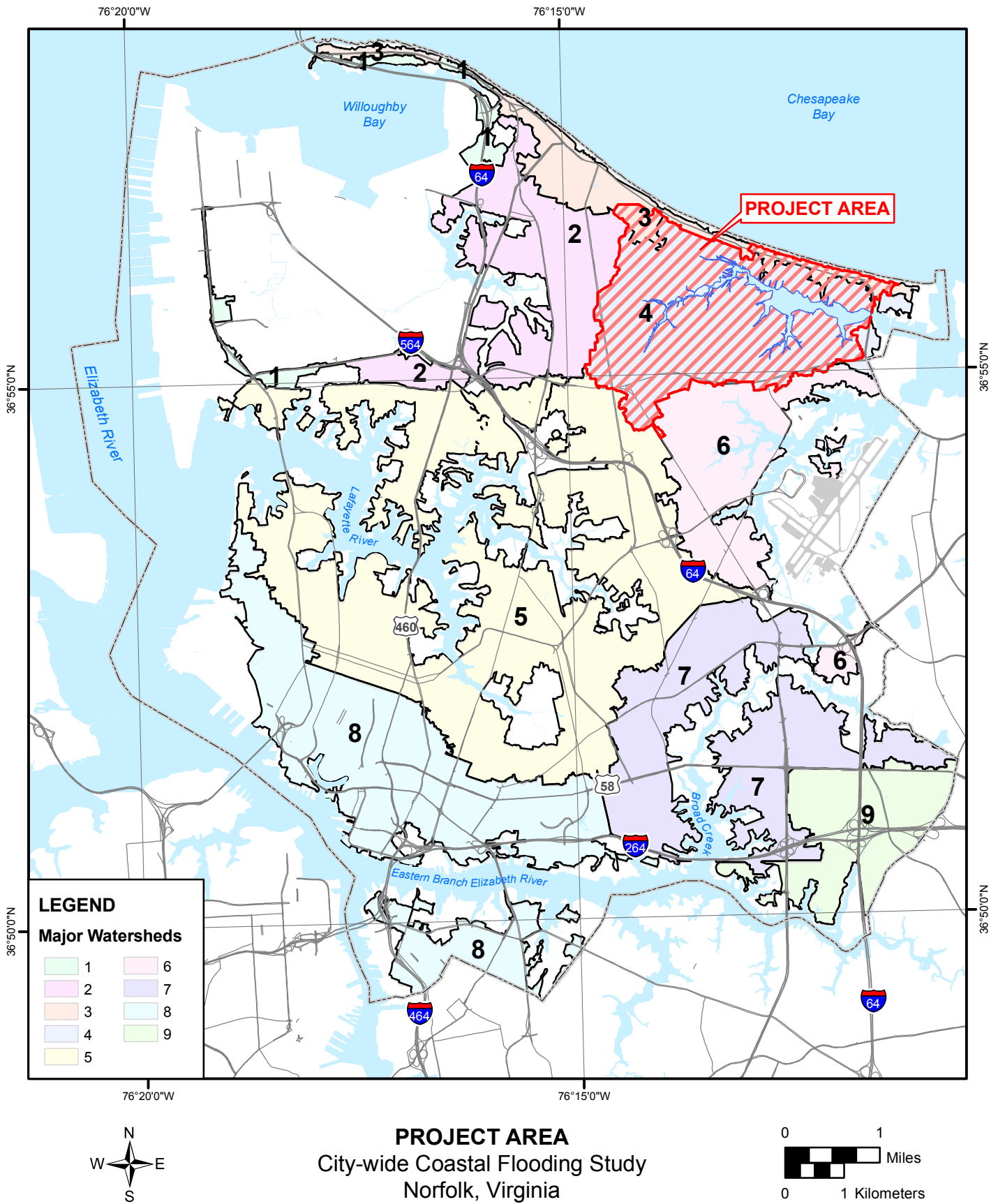
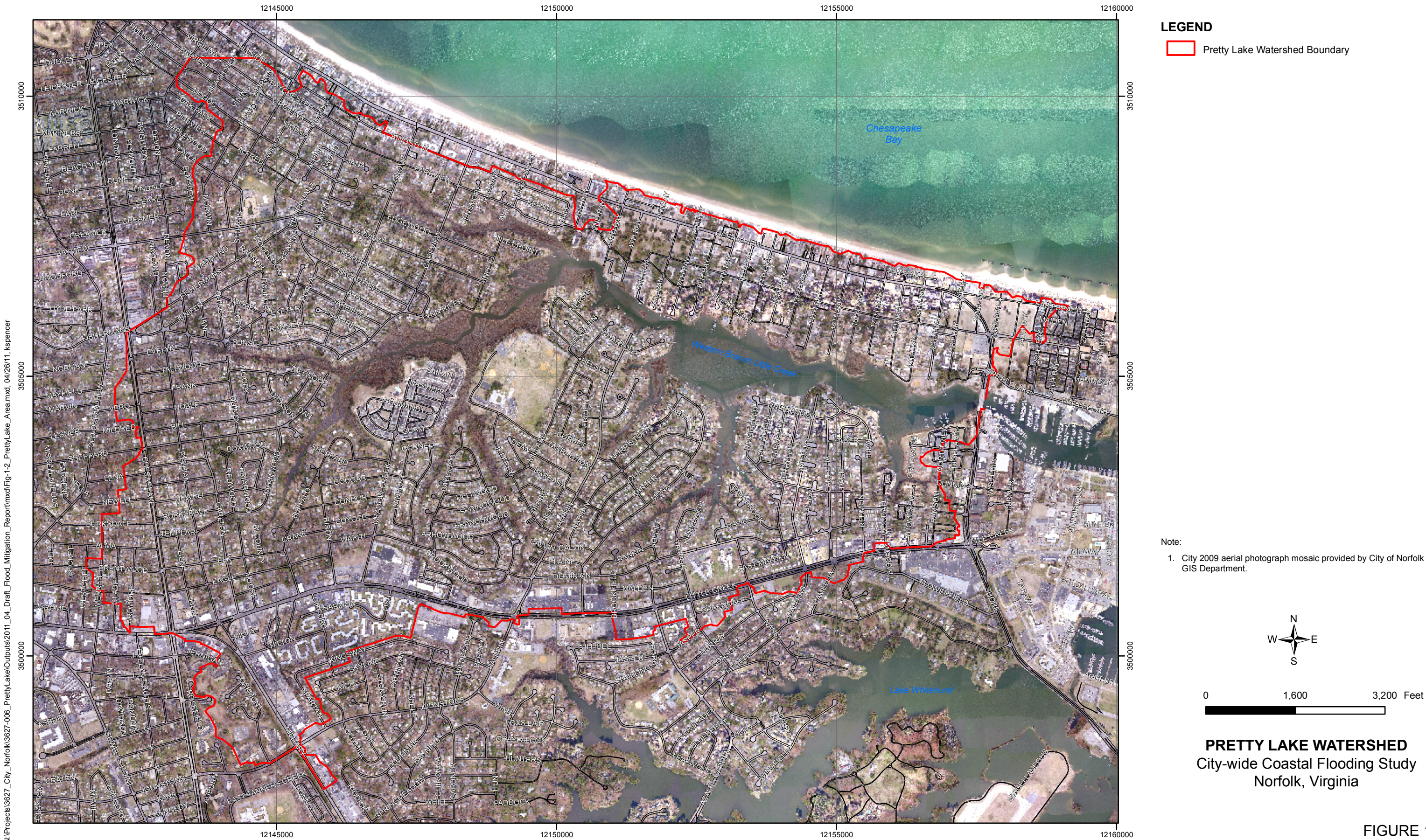


FIGURE 1-1

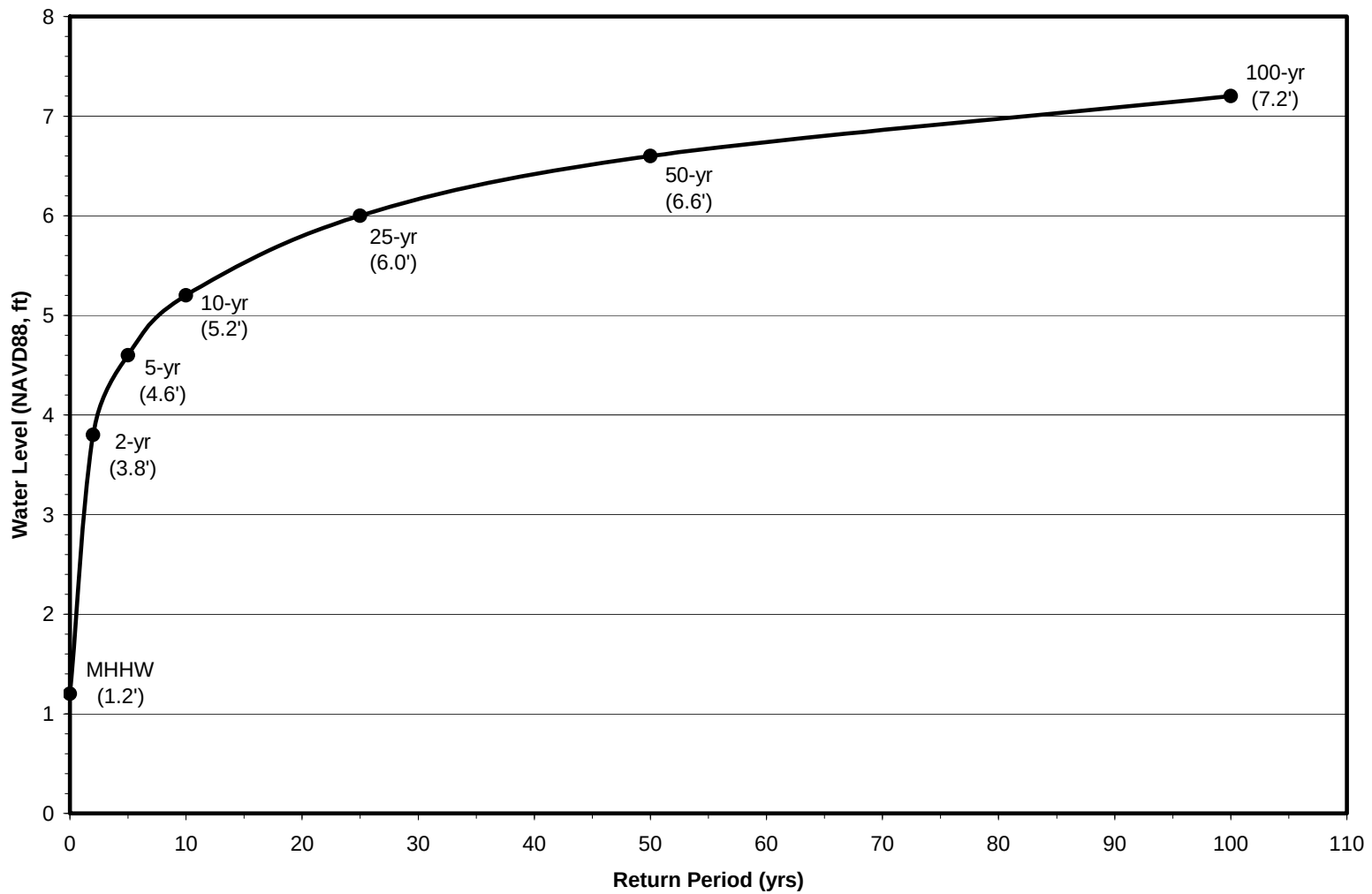




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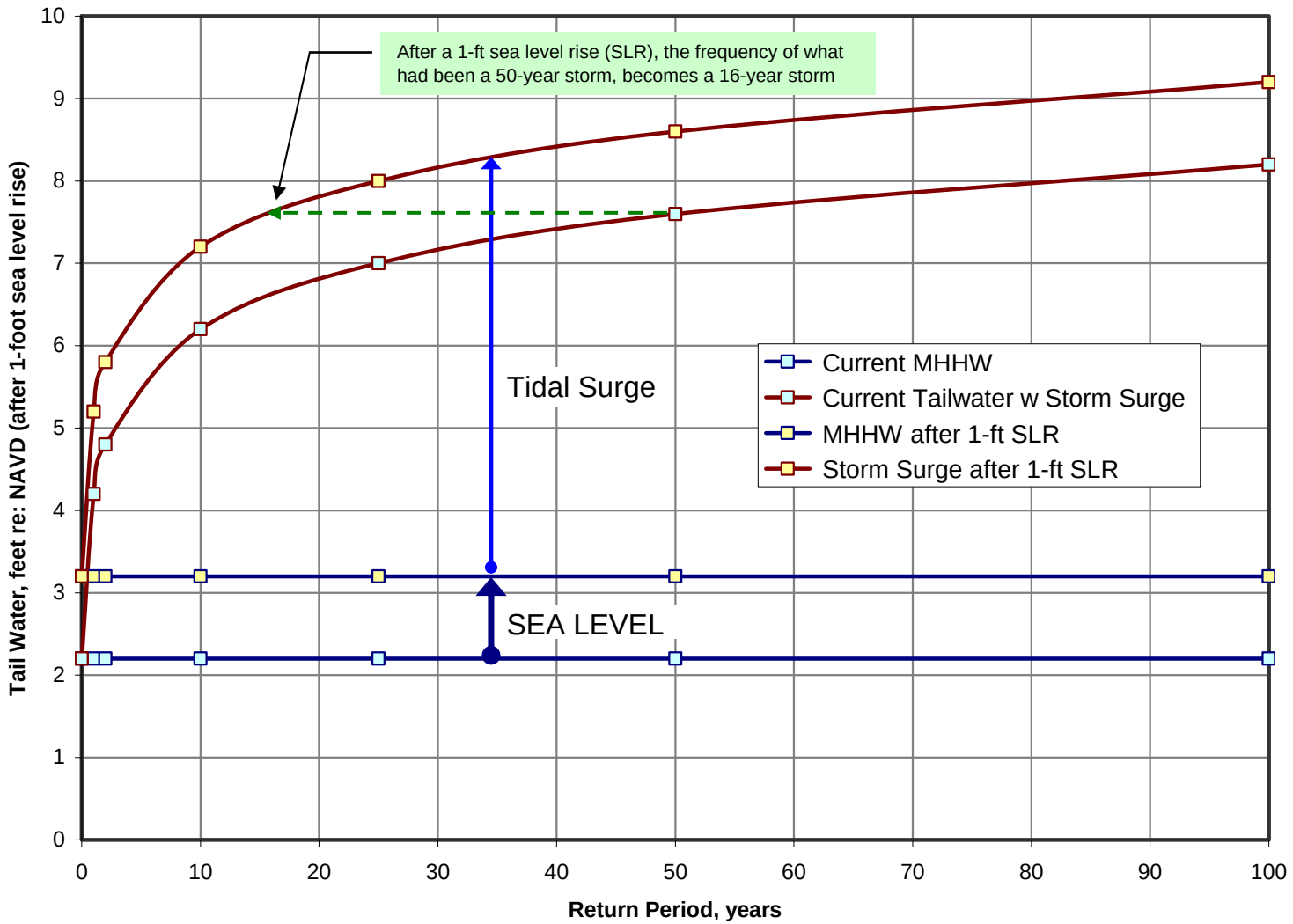
BASIN OUTLET
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 1-3

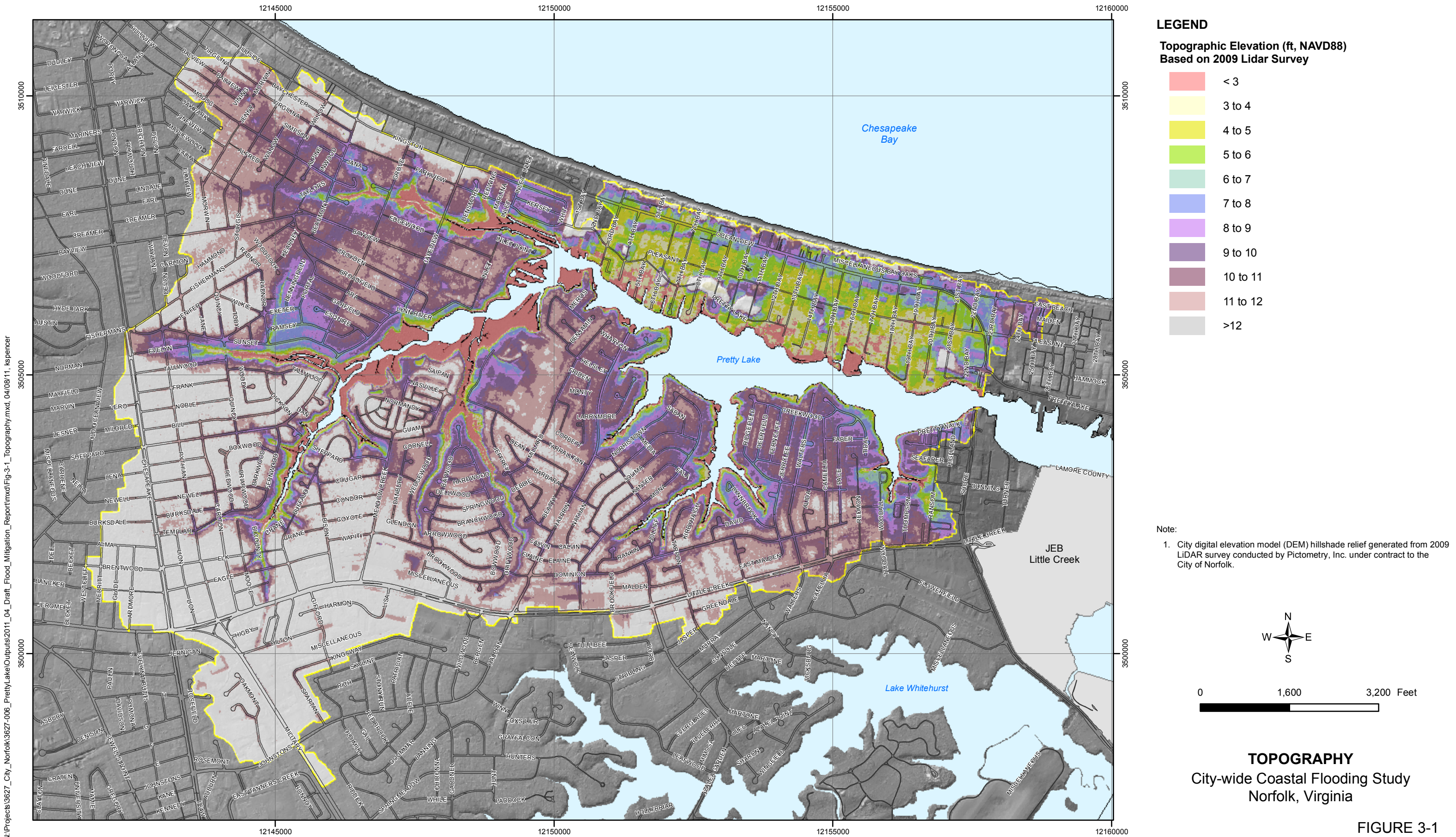


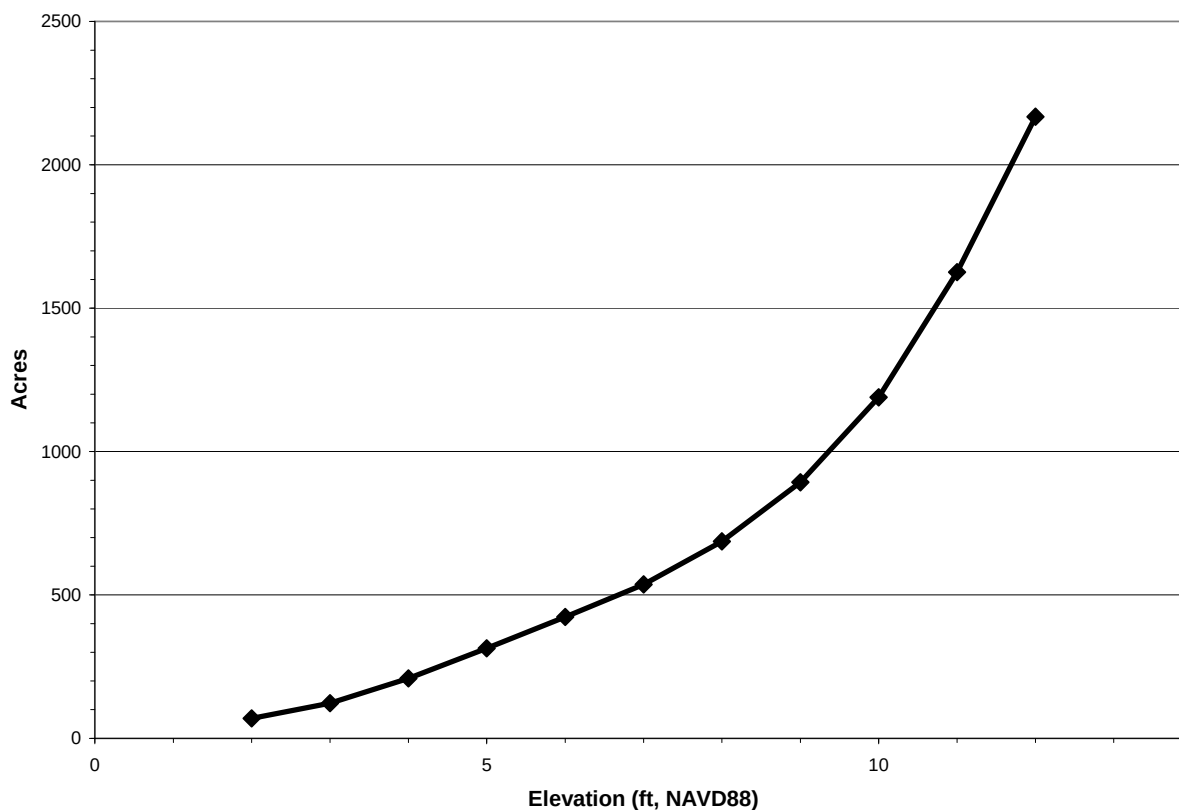
WATER LEVEL ELEVATIONS AT SEWELLS POINT FOR VARIOUS RETURN PERIODS
Based on Current Sea Level Elevation
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 2-1



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



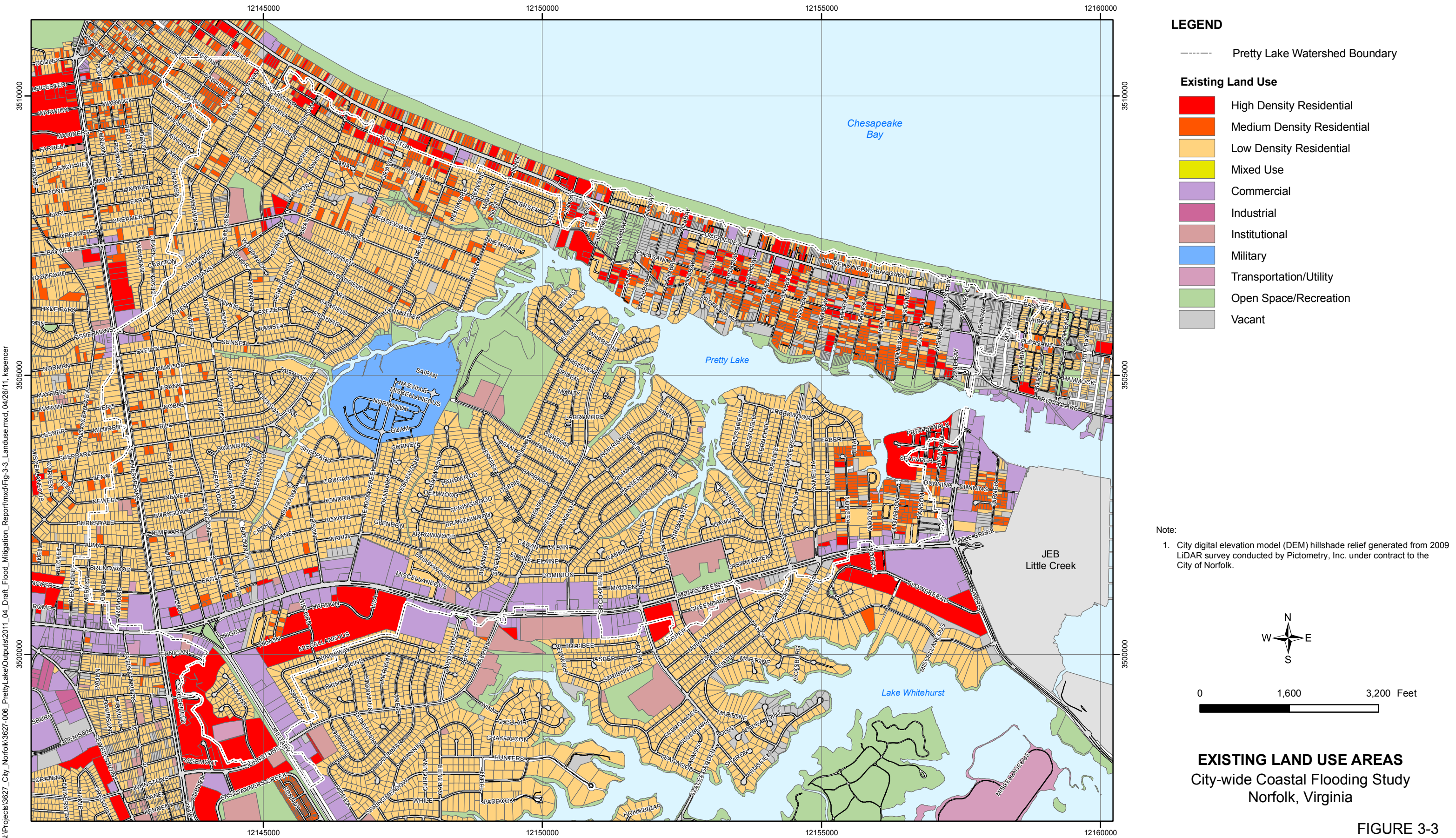


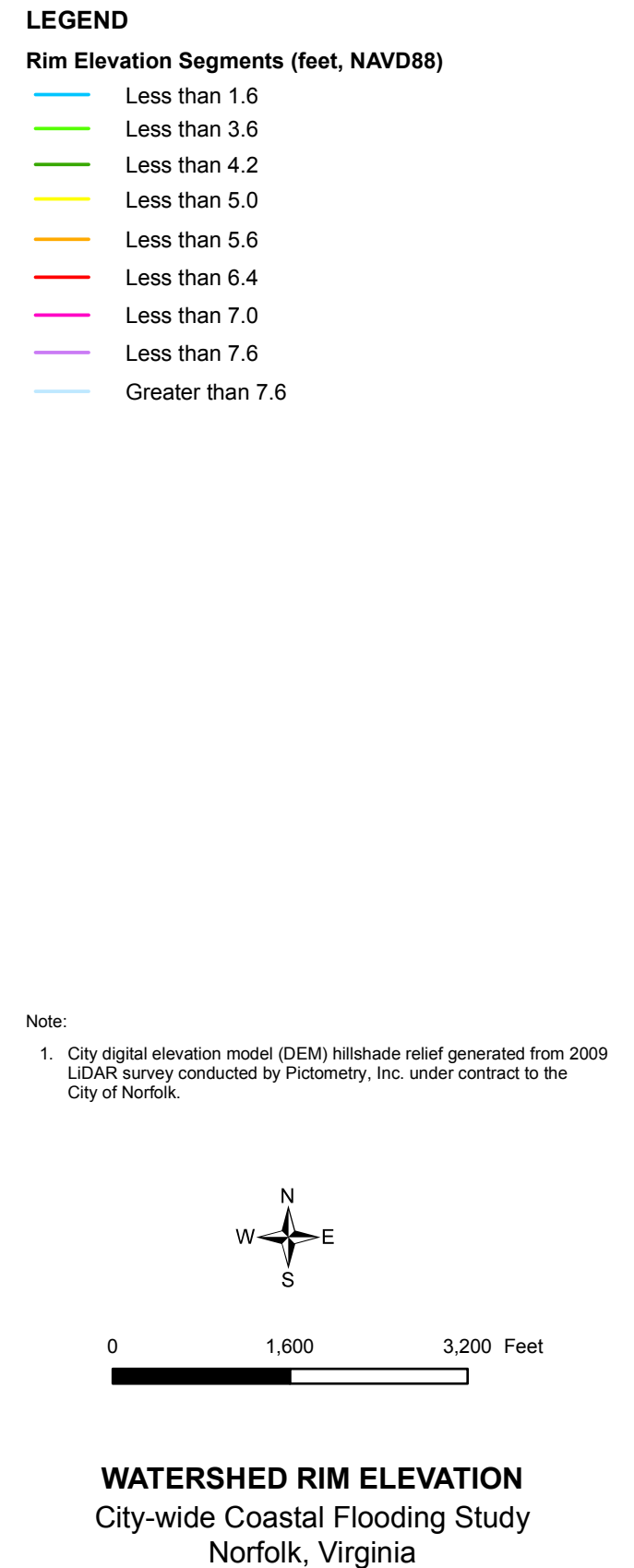
This graph represents a statistical characterization of the ground surface elevation within the Pretty Lake watershed. This cumulative frequency graph is based on the 2009 LiDAR survey data that has a 3-ft by 3-ft bin size (horizontal footprint is 3-ft by 3-ft). The watershed encompasses approximately 2200 acres. Acreage estimates in this graph do not include the Pretty Lake water body.

Examples of how this graph may be interpreted:

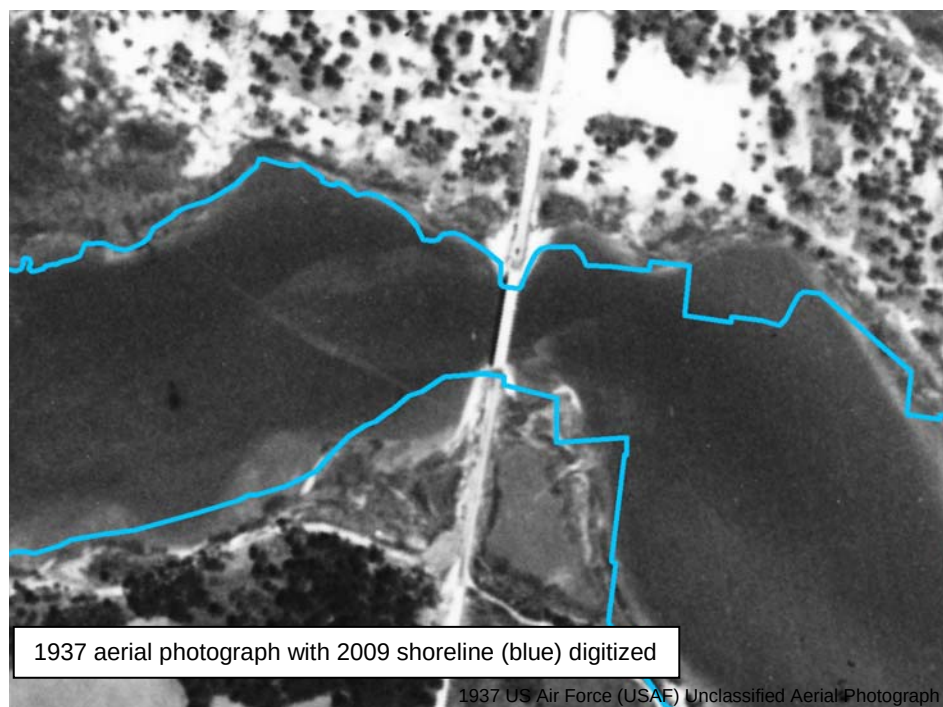
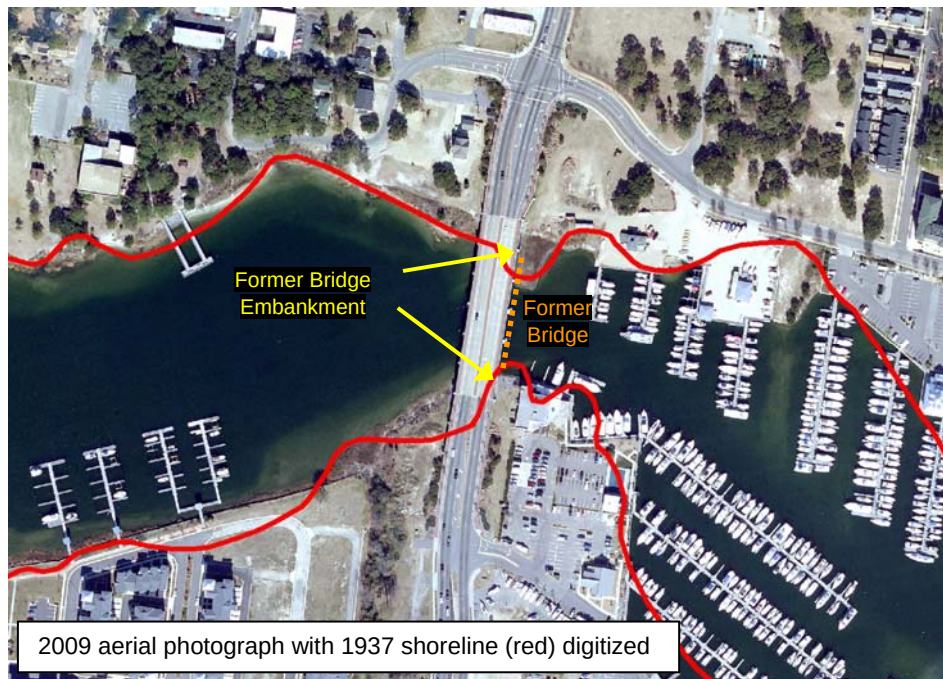
- 1) Roughly 300 acres of the study area is equal to or below elevation 5 feet (NAVD88).
- 2) Roughly 900 acres of the study area is equal to or below elevation 9 feet (NAVD88).

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





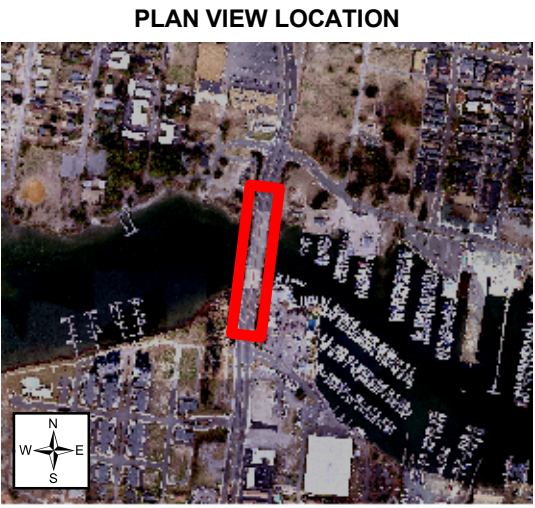
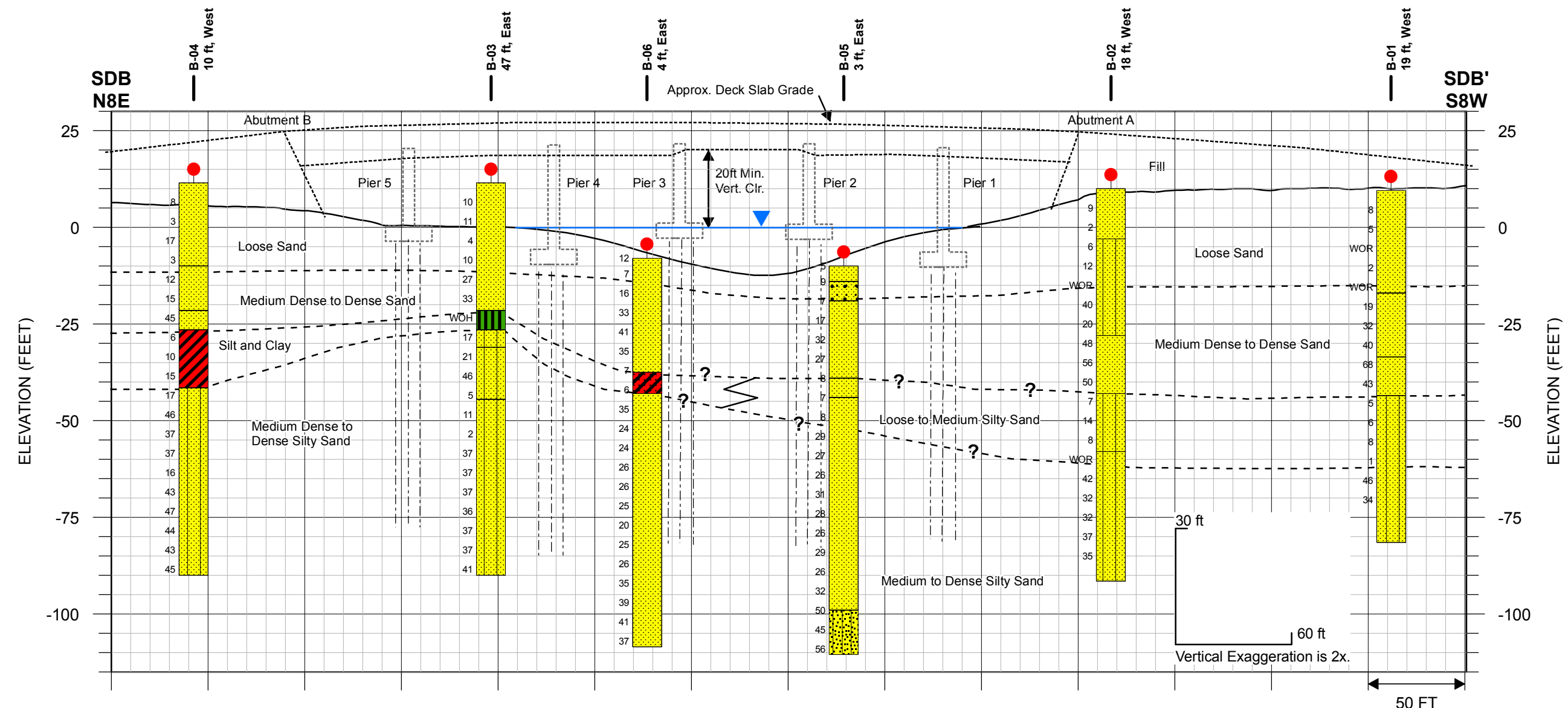
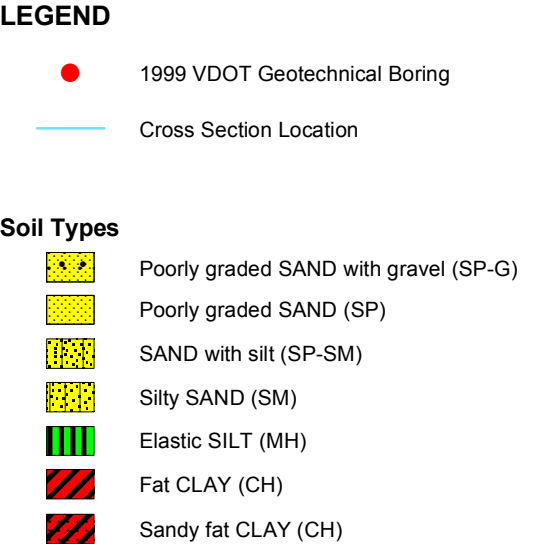
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FORMER SHORELINE STRUCTURES

Basin Outlet

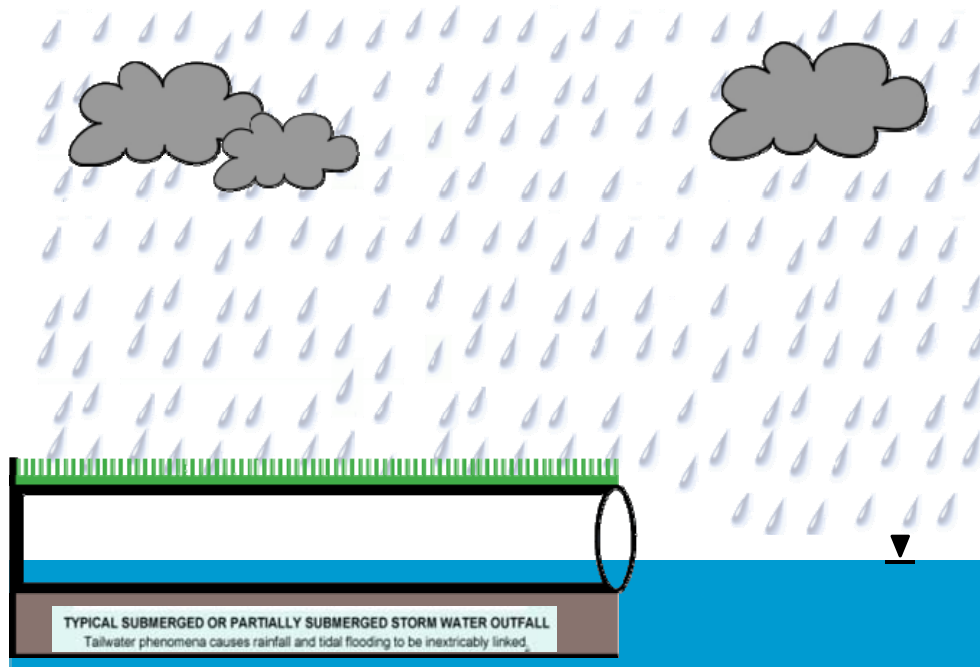
City-wide Coastal Flooding Study
Norfolk, Virginia



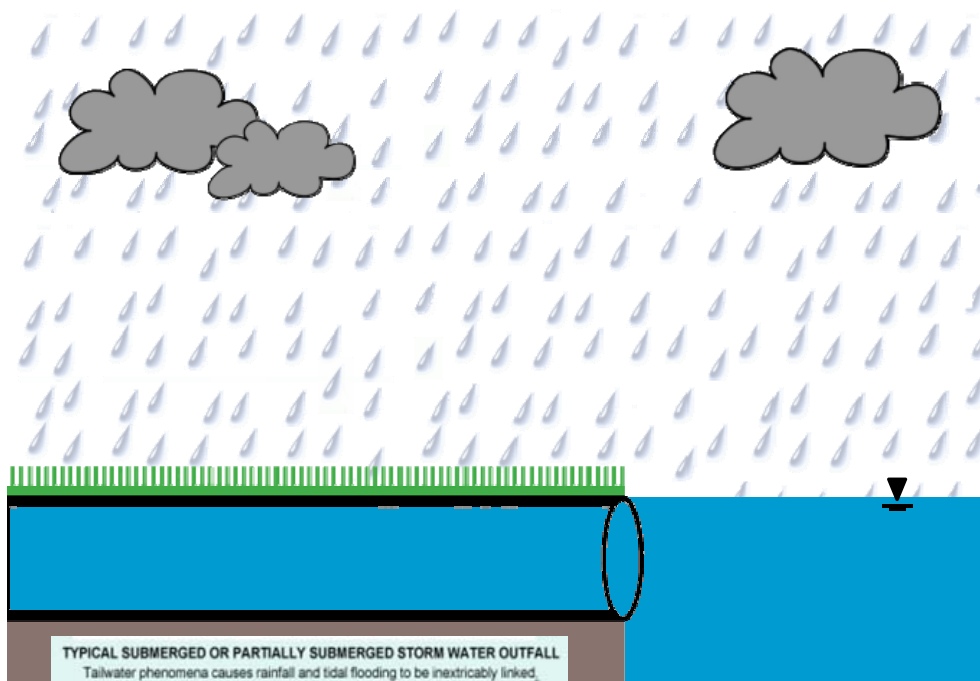
**PRELIMINARY SUBSURFACE
CROSS SECTION**
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE 4-2

Rainfall Predominant



Coincident Surge Event



TAILWATER PHENOMENA

City-wide Coastal Flooding Study

Norfolk, Virginia



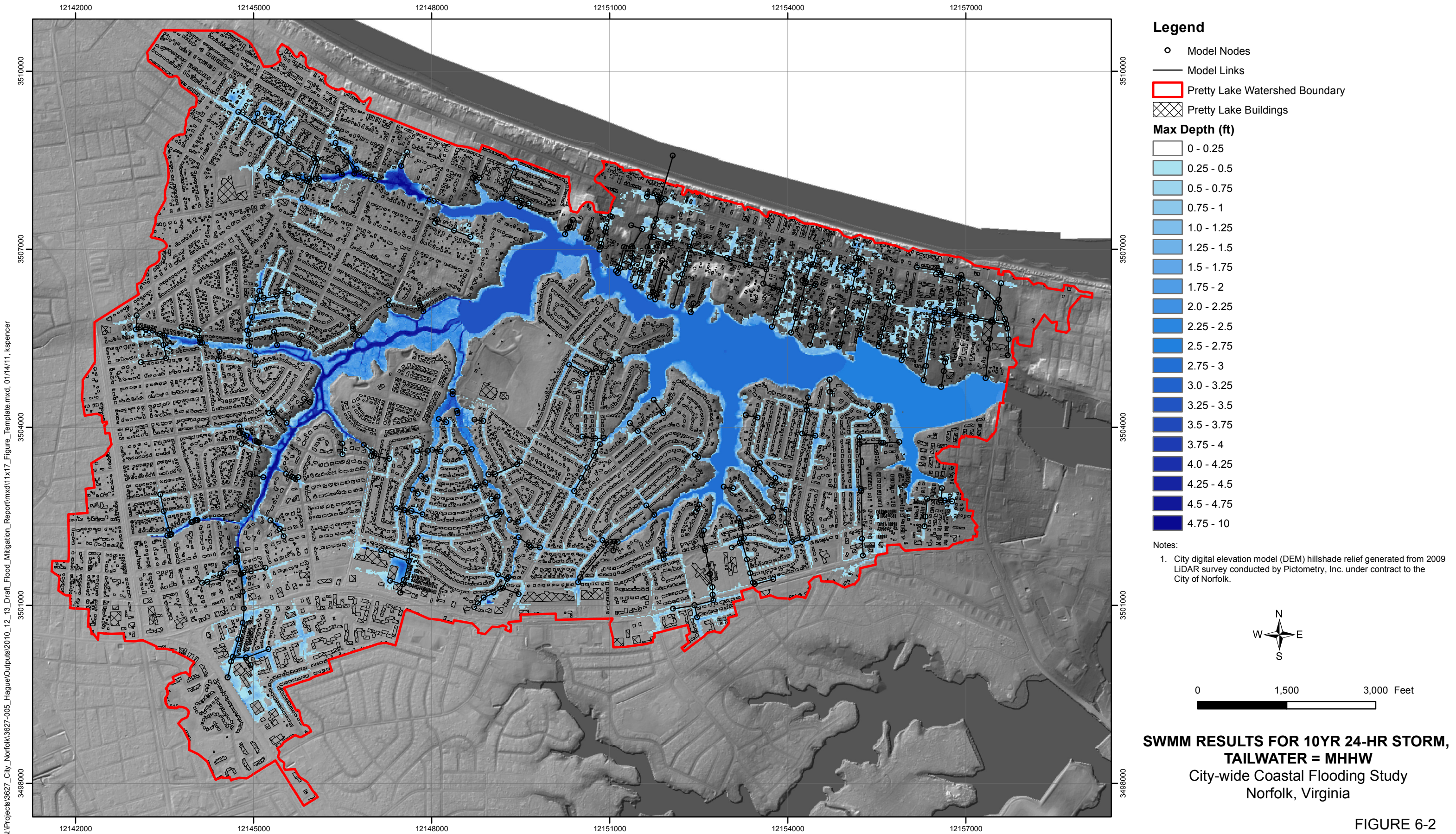


FIGURE 6-2

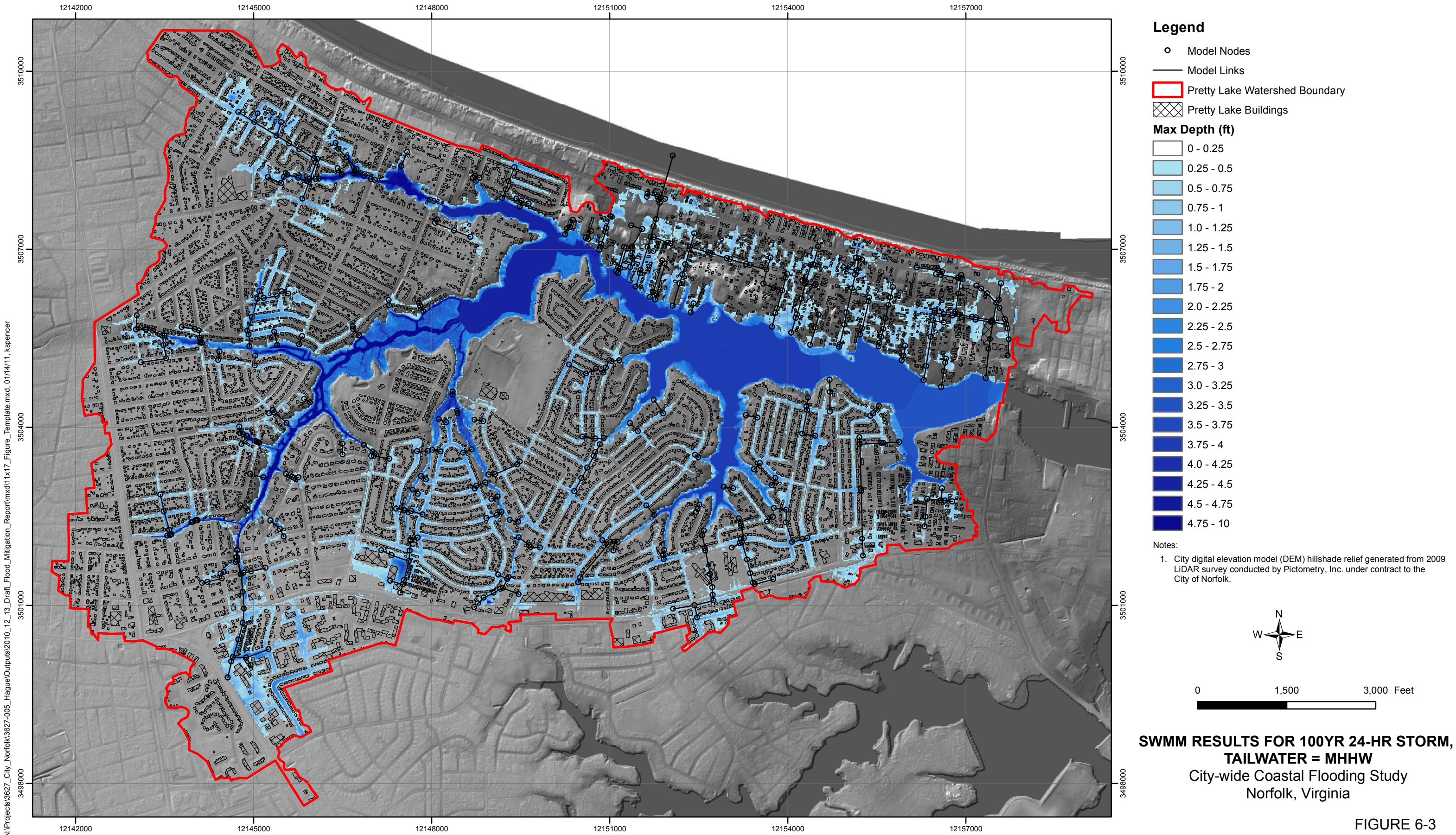


FIGURE 6-3

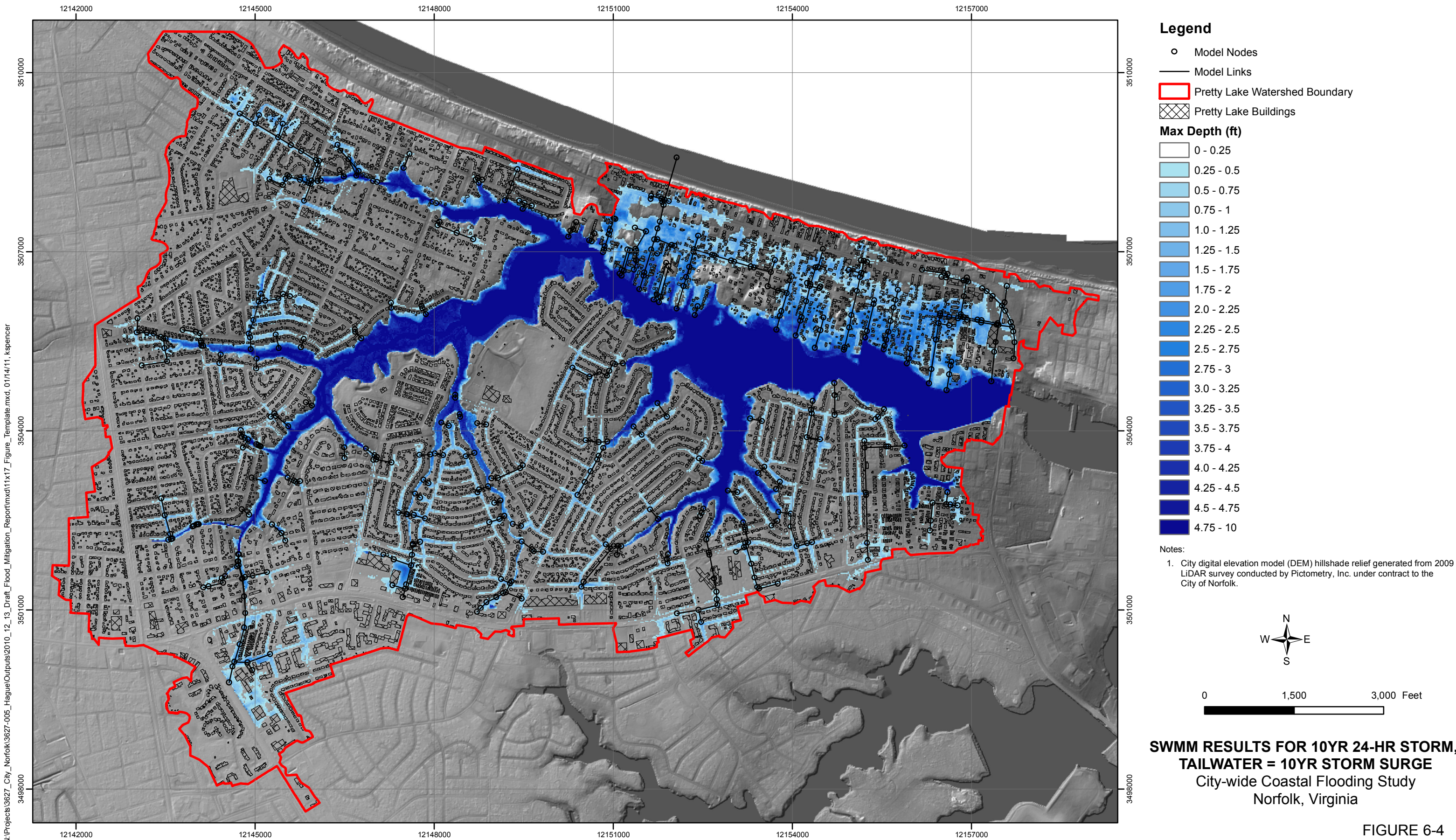


FIGURE 6-4

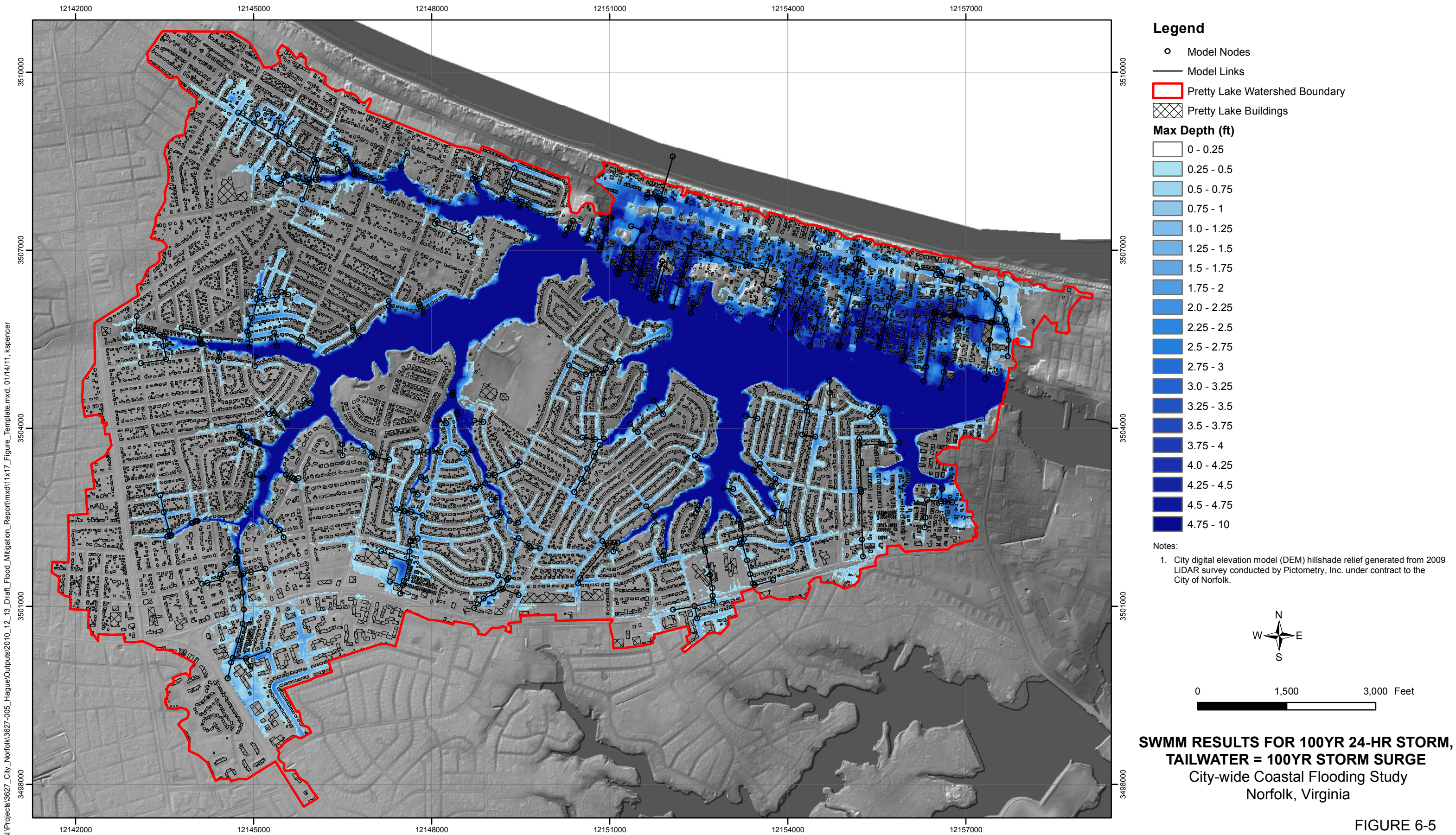


FIGURE 6-5

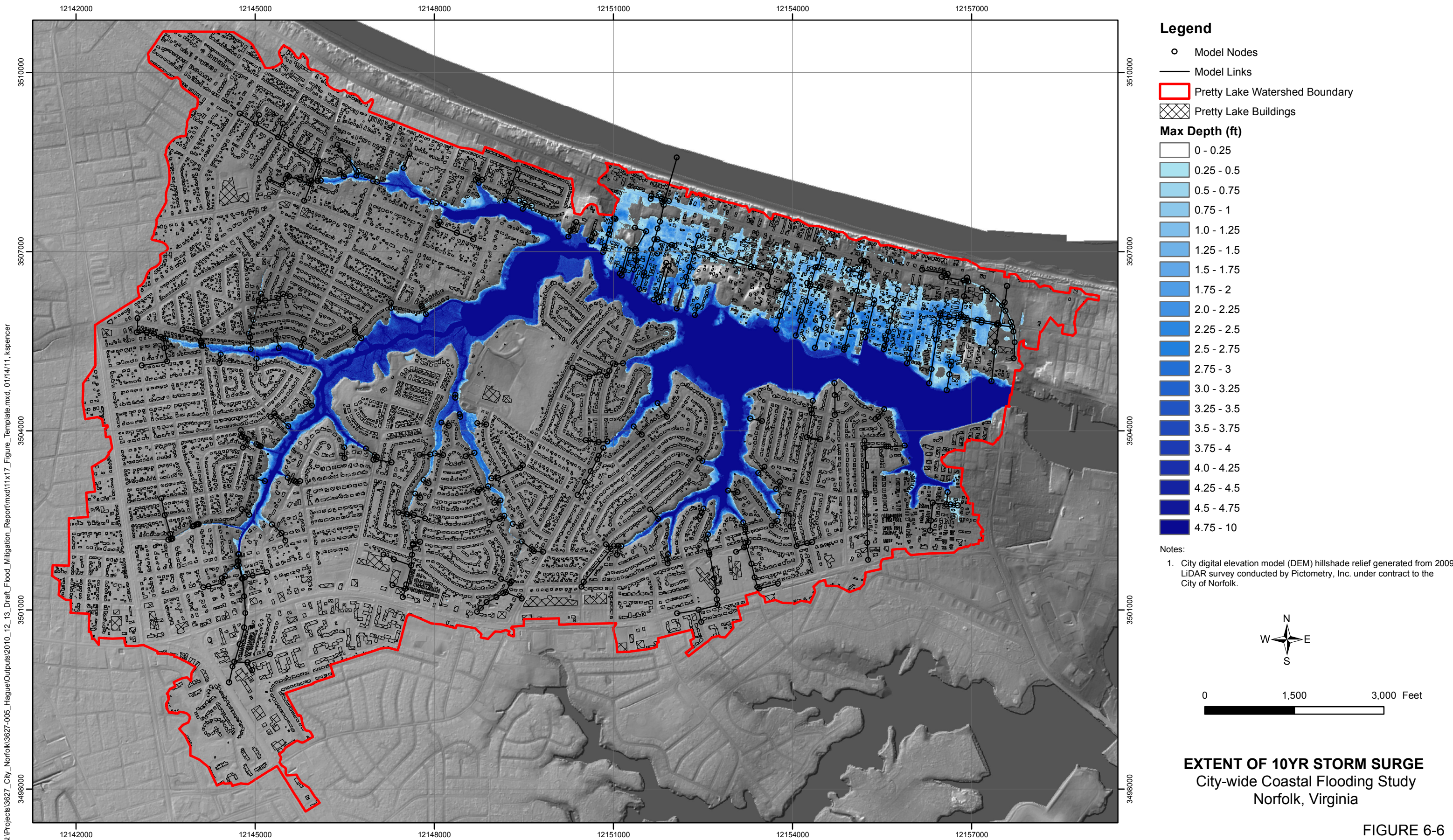


FIGURE 6-6

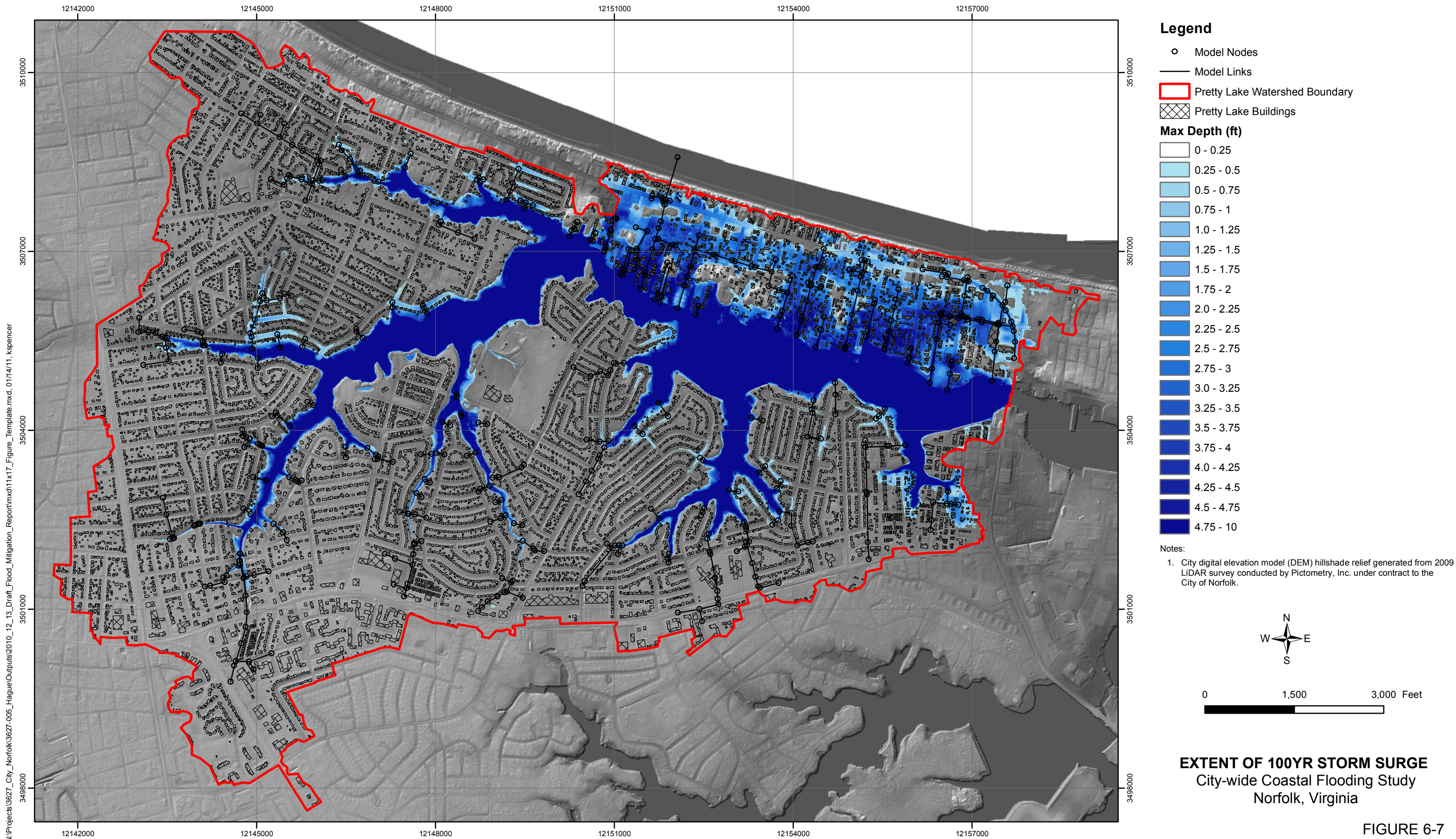
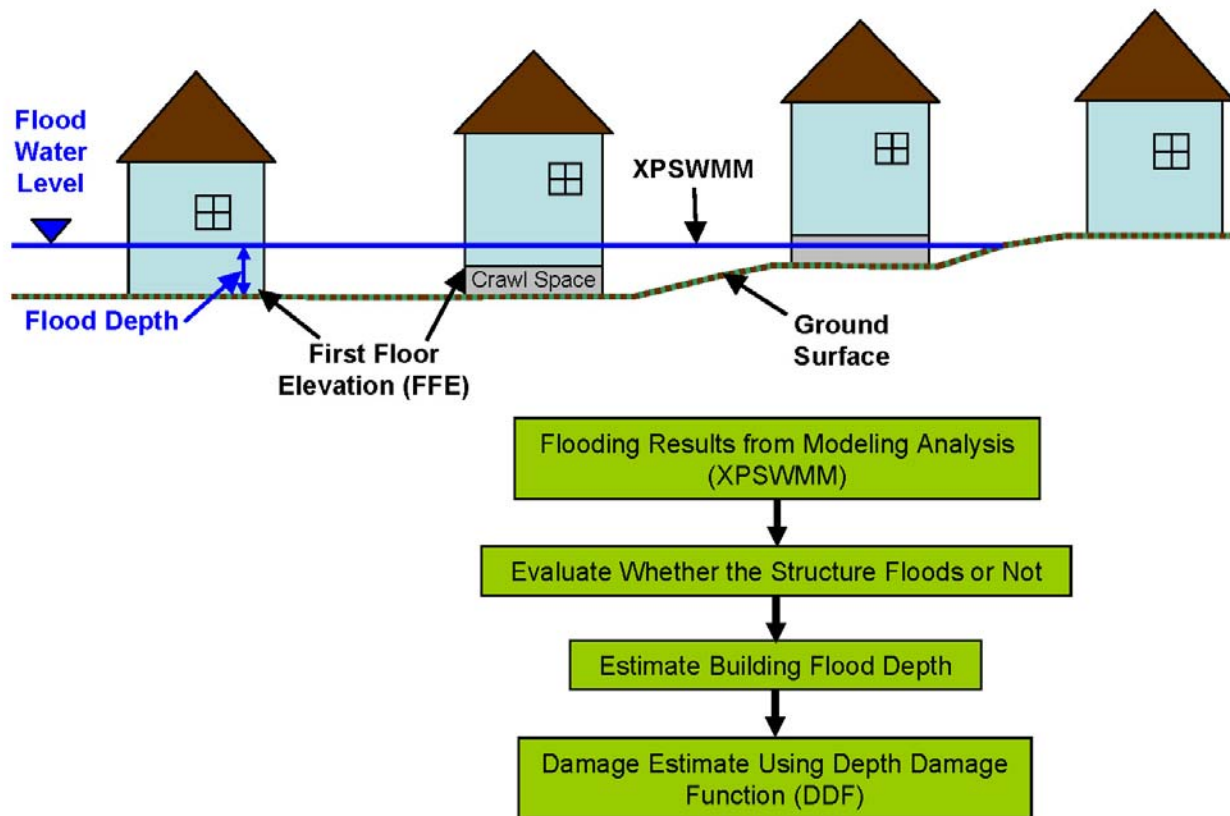
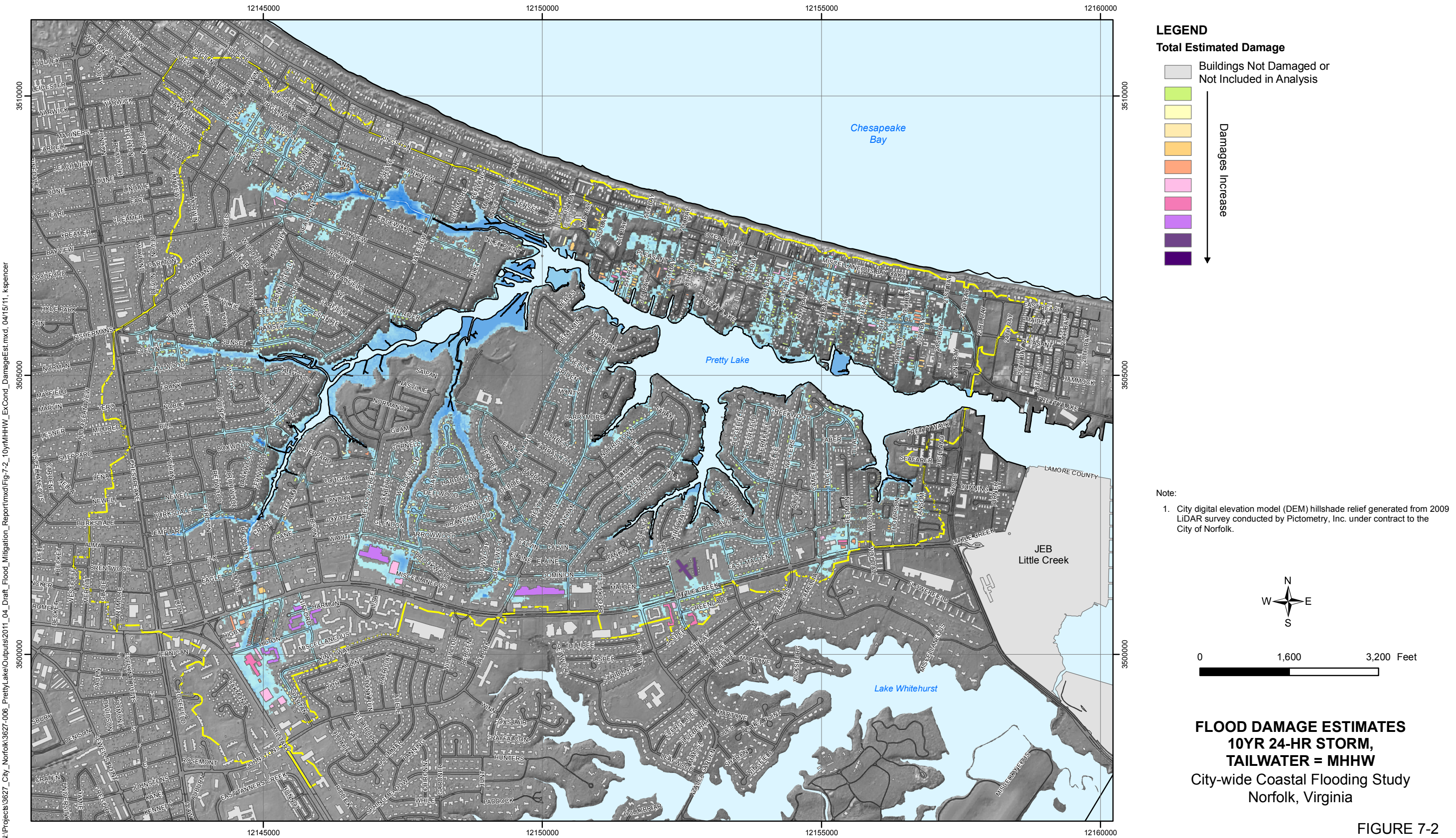


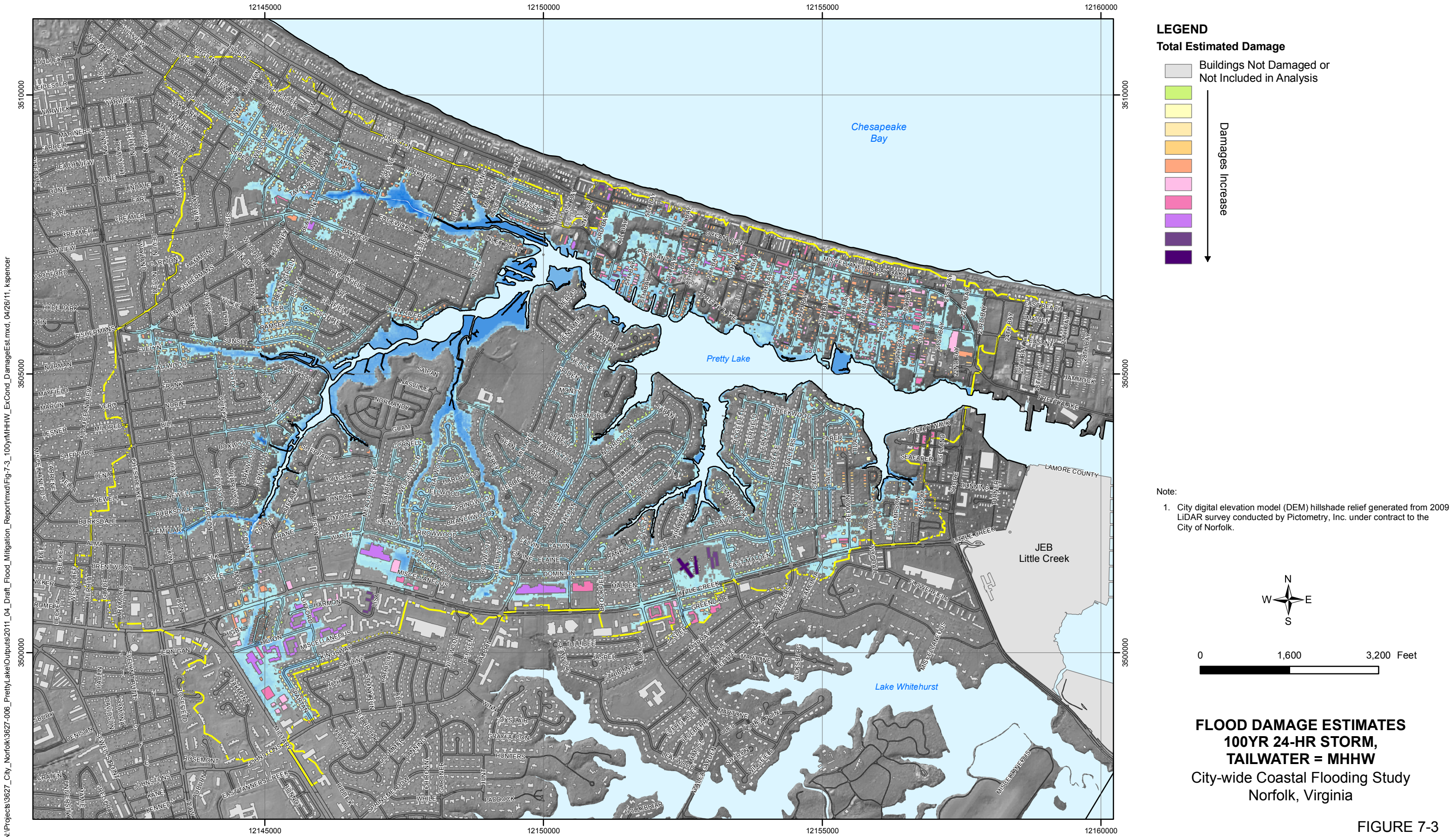
FIGURE 6-7

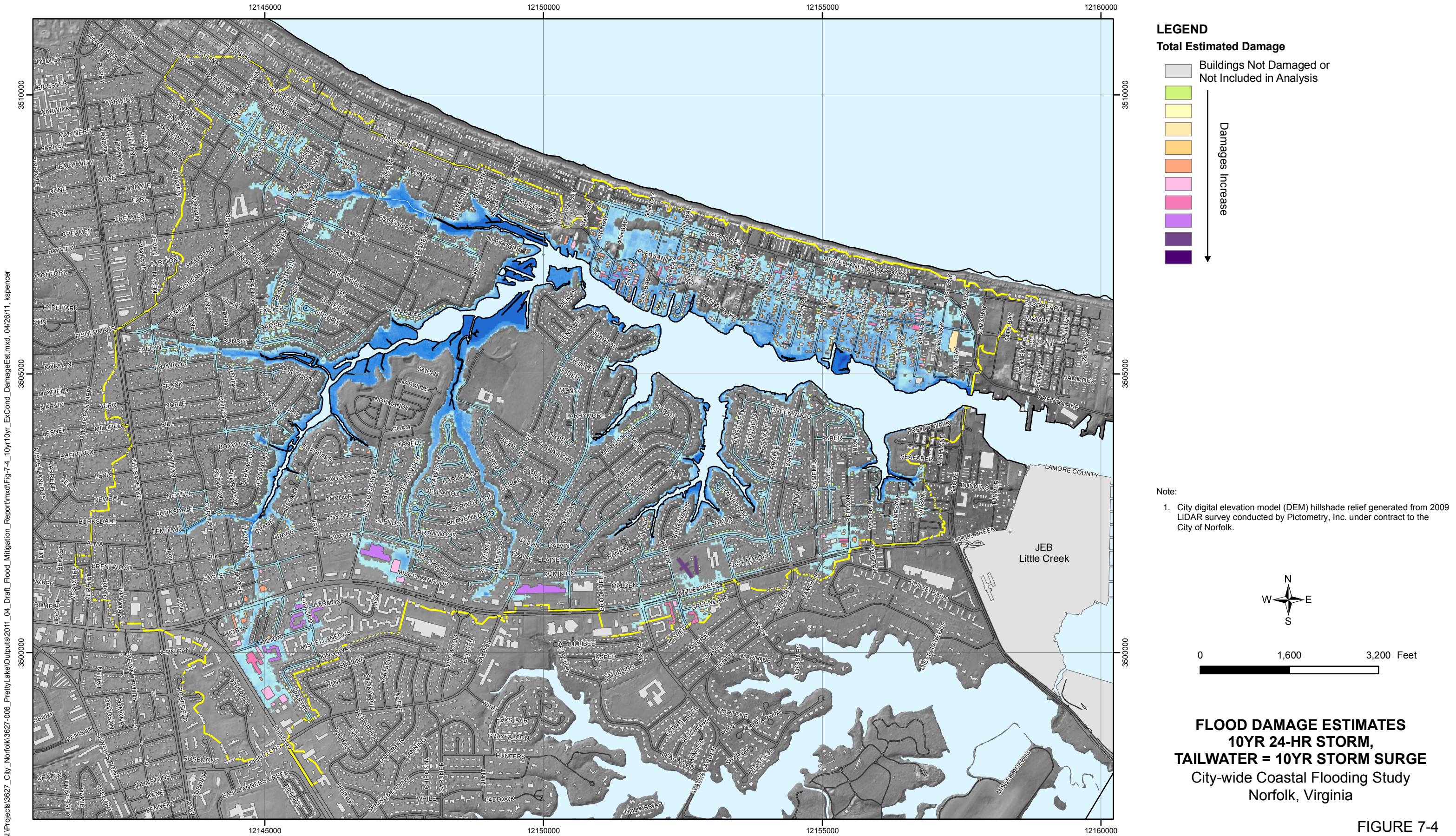


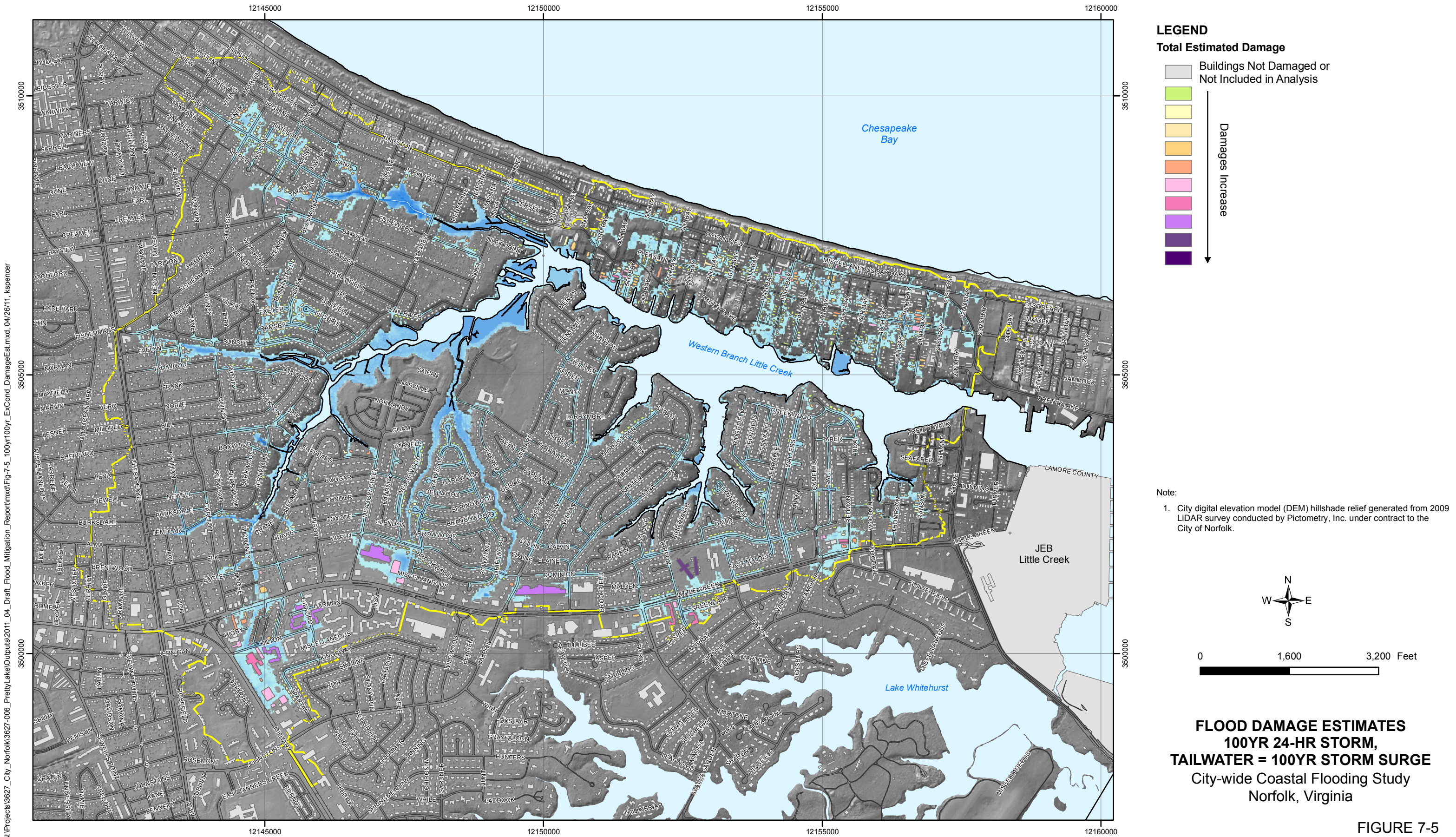
DEPTH DAMAGE FUNCTION CONCEPT
City-wide Coastal Flooding Study
Norfolk, Virginia

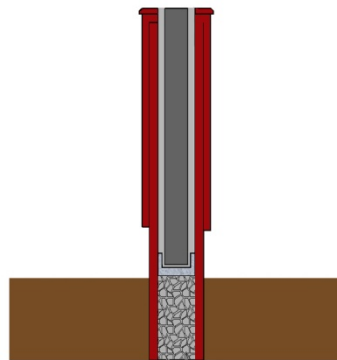
FIGURE 7-1



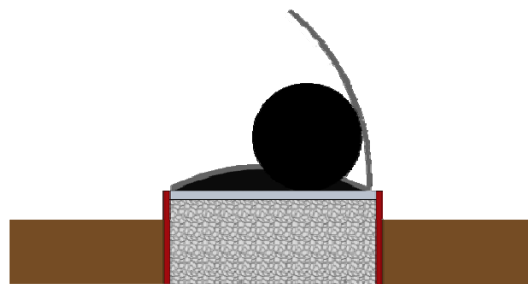




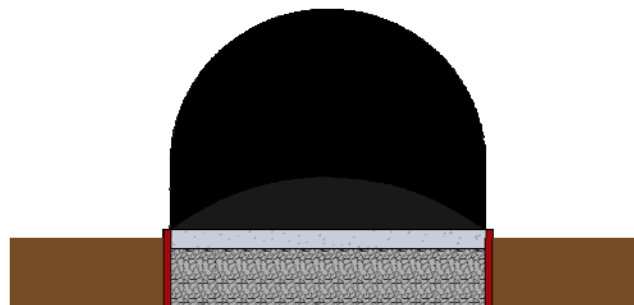




Steel Gate Option



Obermeyer Gate Option



Inflatable Dam Option

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



— TOTAL ROAD LENGTH = 2,800 LF

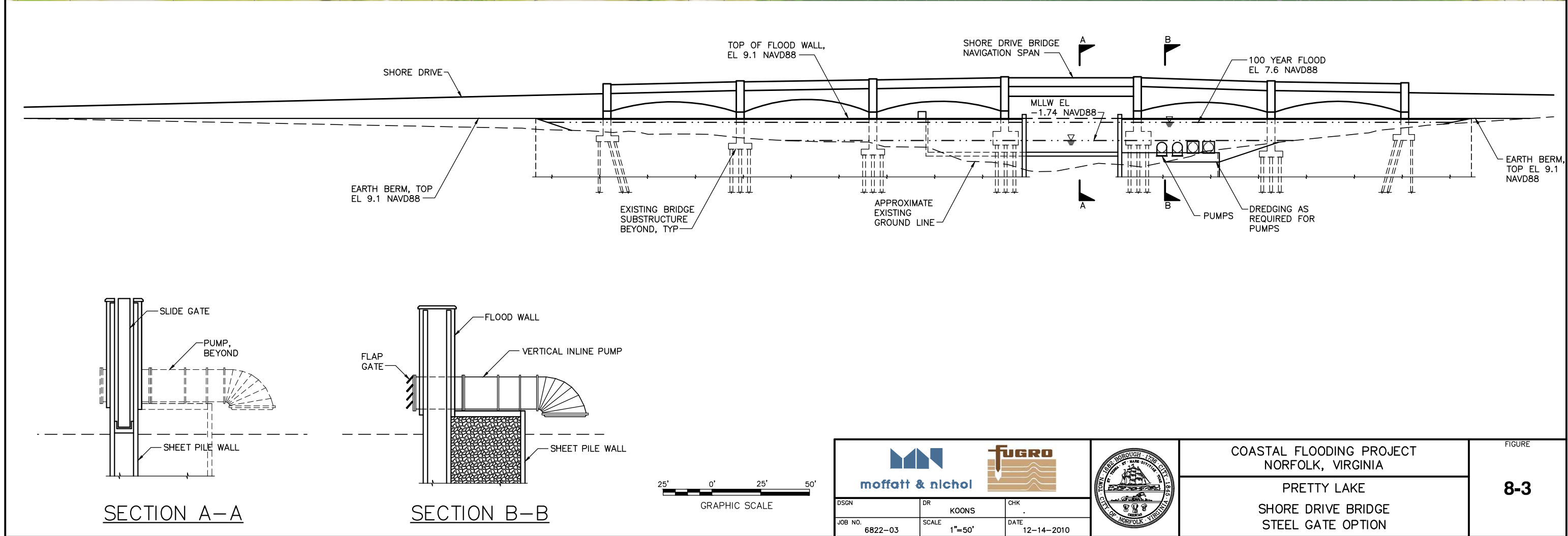
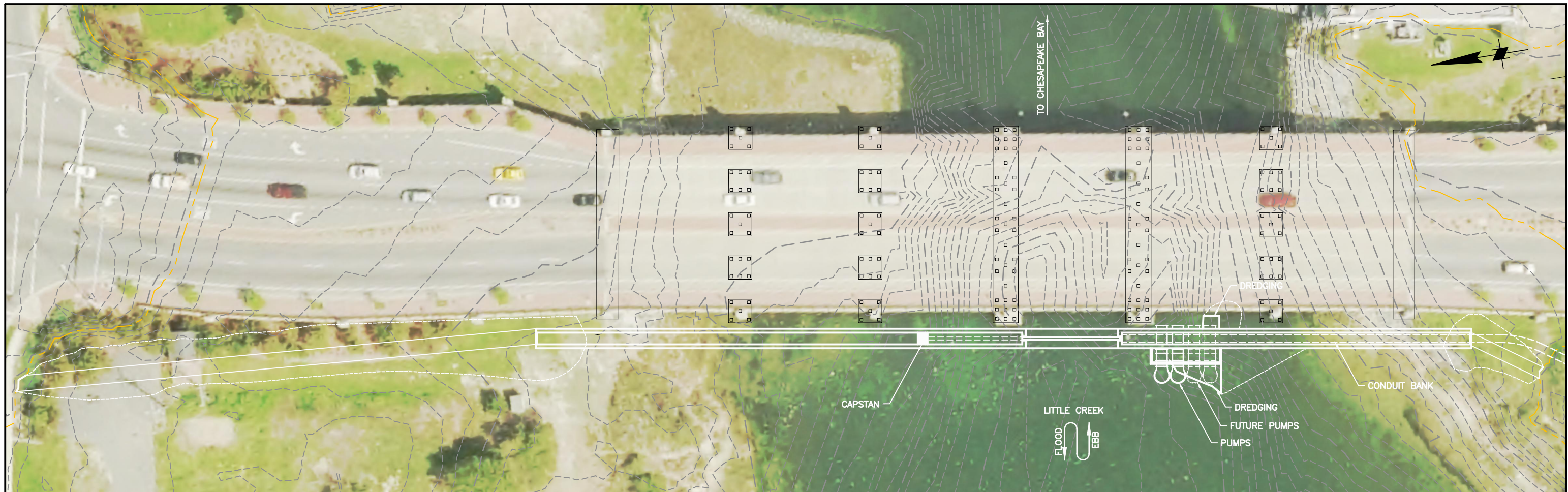


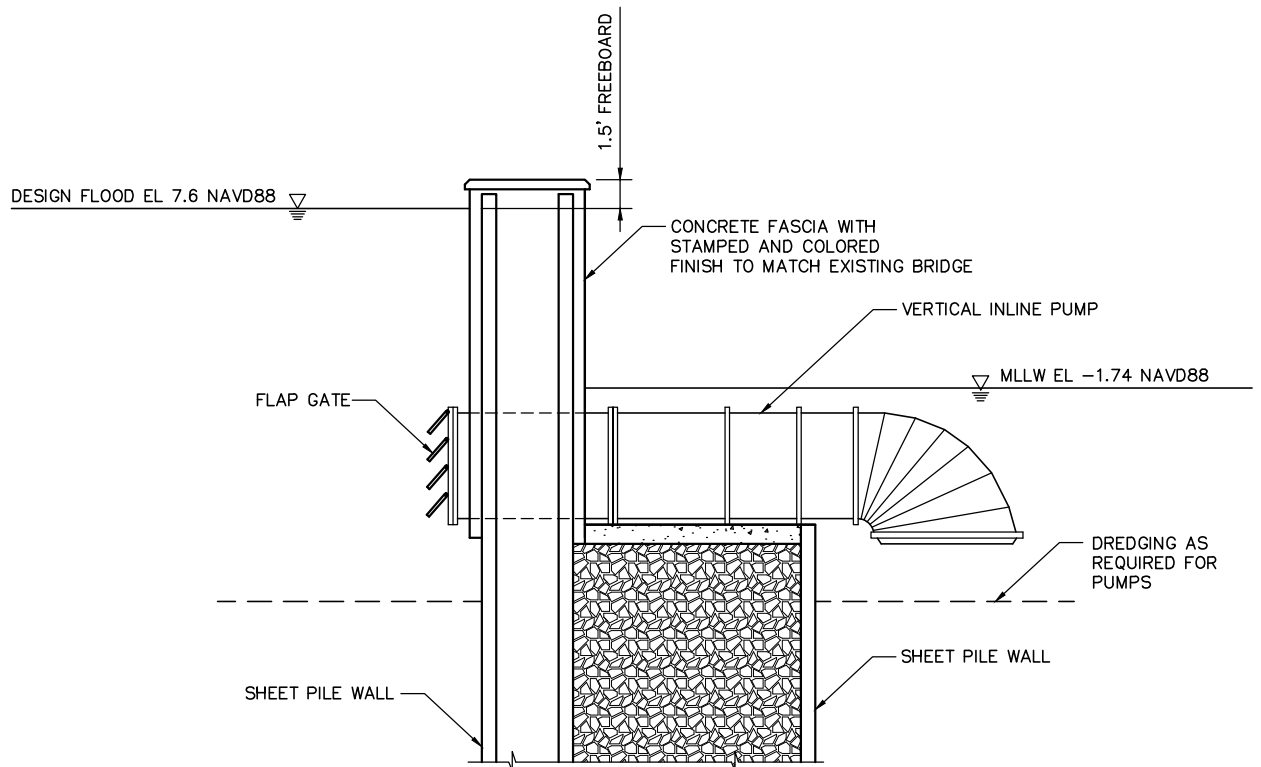
			
moffatt & nichol			
DSGN	DR	CHK	
JOB NO.	SCALE	DATE	
6822-03	1"=250'	12-14-2010	



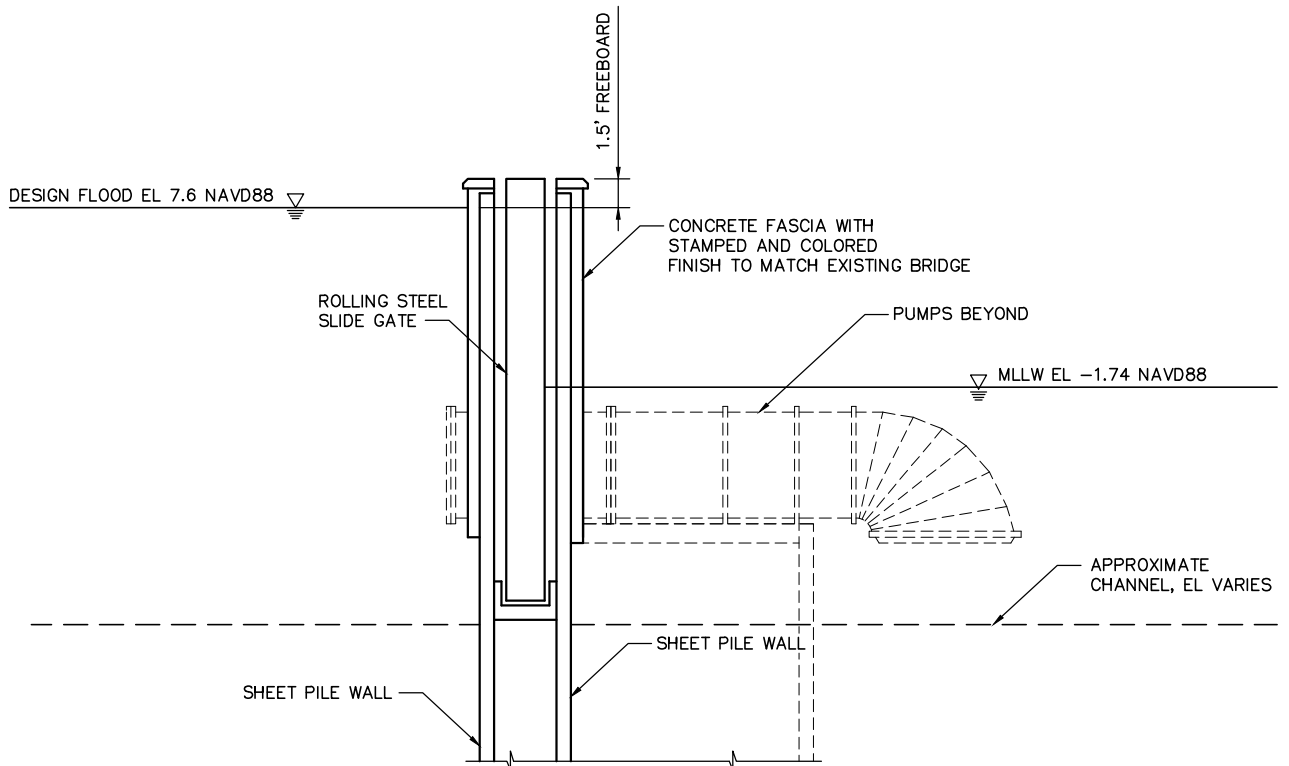
COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	
PRETTY LAKE TIDAL BARRIER WITH TIDE GATE, PUMPS AND ROAD RAISE	

FIGURE
8-2



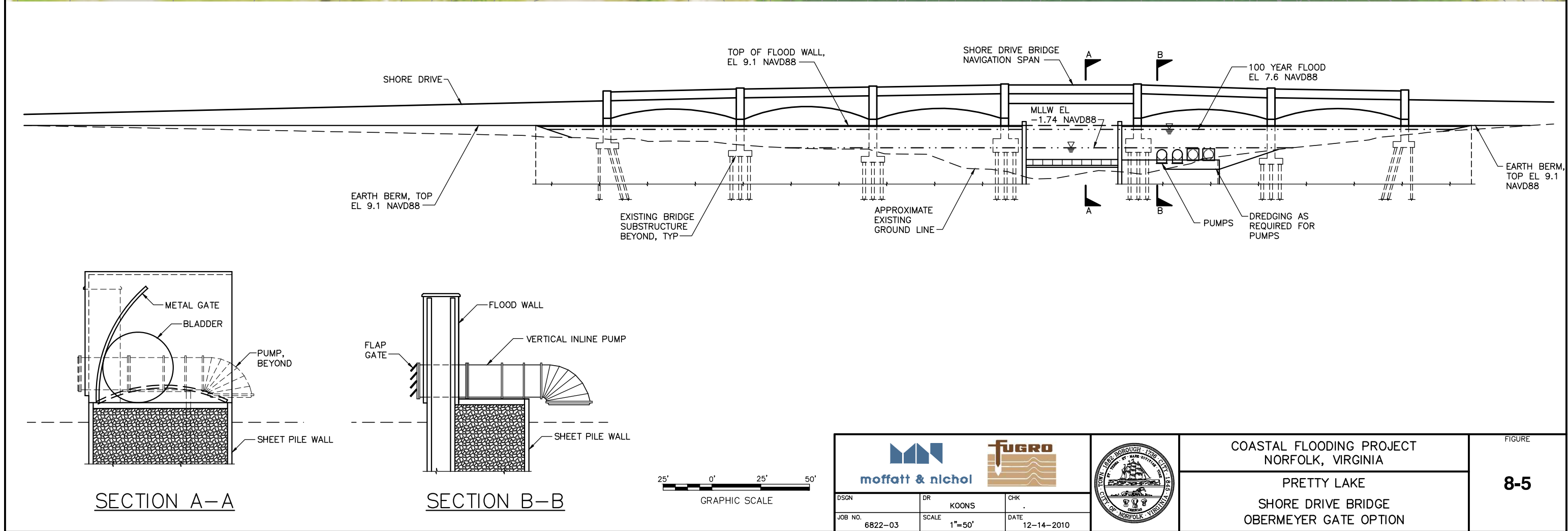
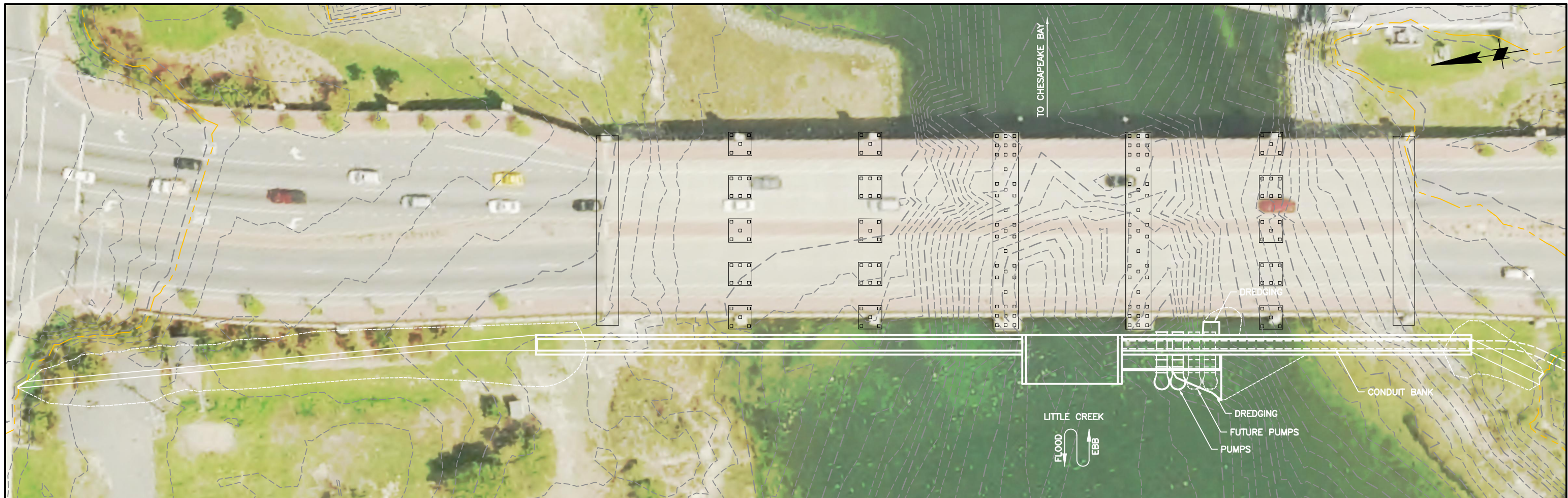


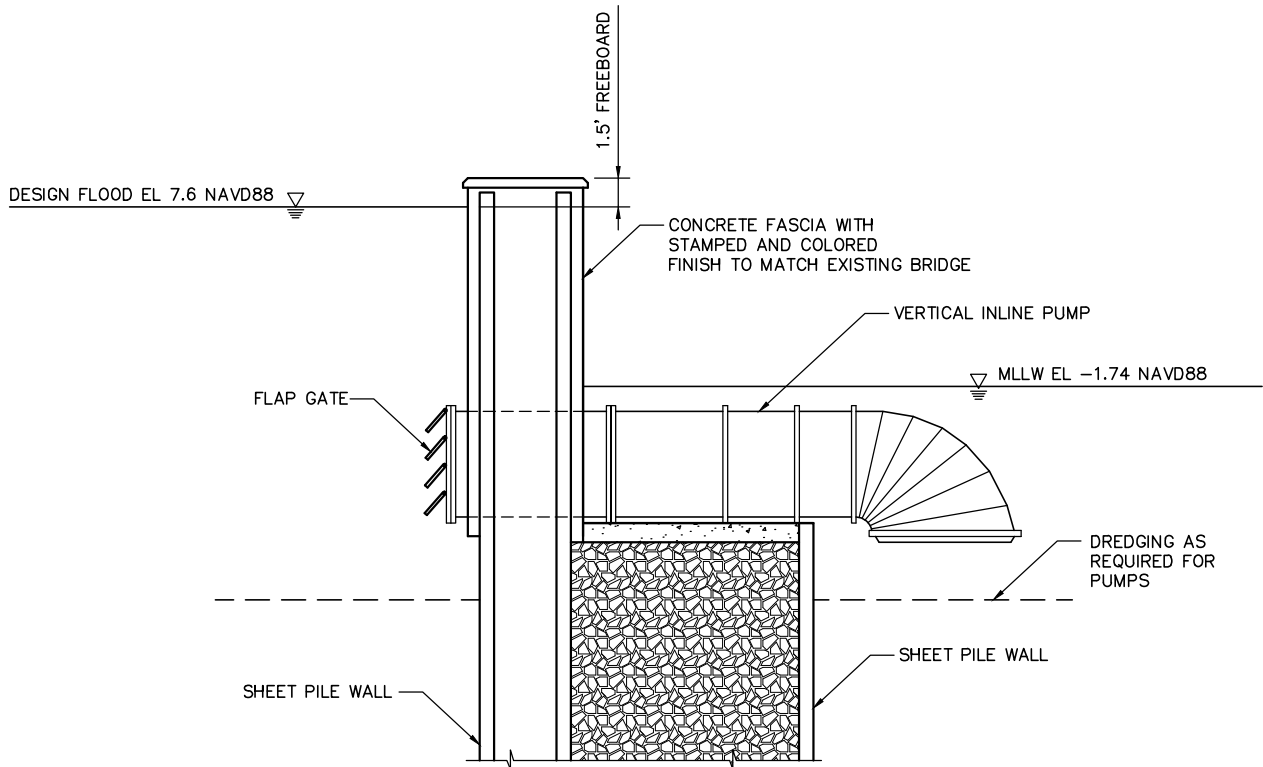
WALL SECTION AT PUMPS



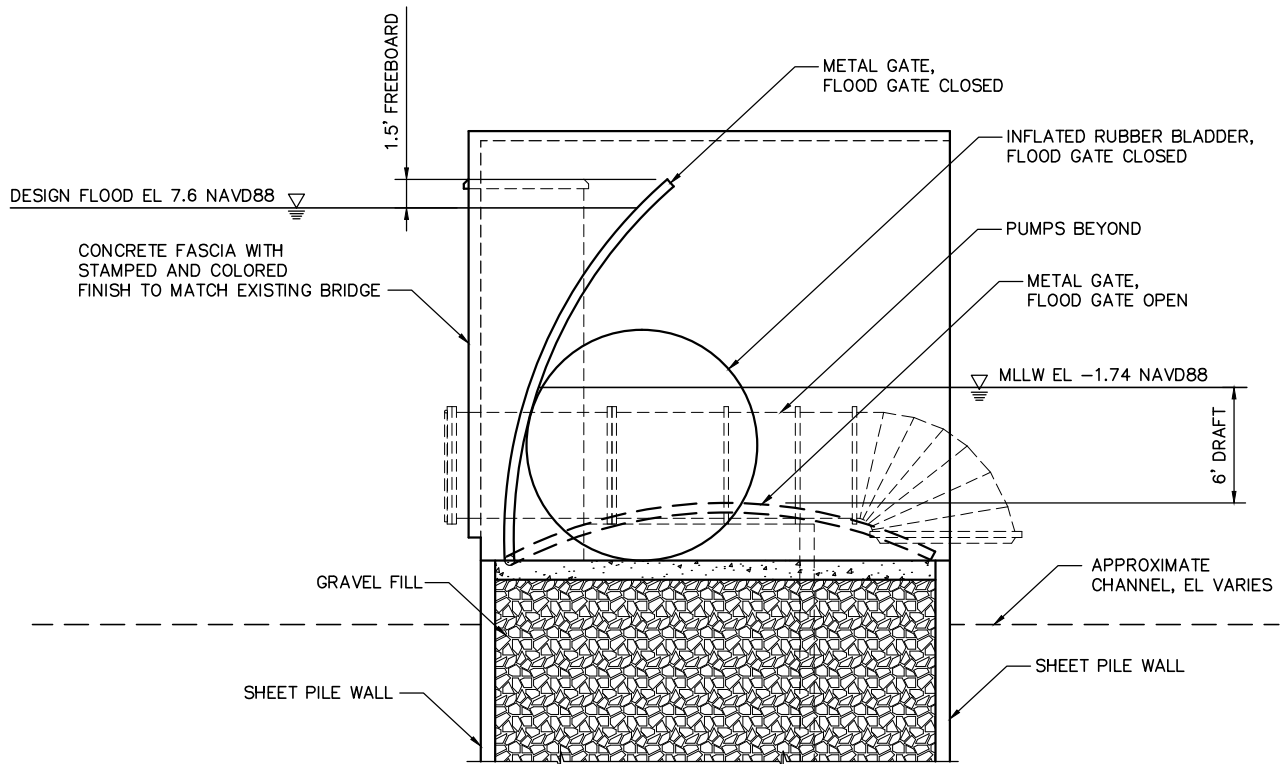
SECTION AT FLOOD STEEL GATE

 				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	FIGURE 8-4
DSGN	DR	CHK		PRETTY LAKE SHORE DRIVE BRIDGE STEEL GATE OPTION – SECTIONS	
JOB NO. 6822-03	SCALE 1"=10'	DATE 12-14-2010			



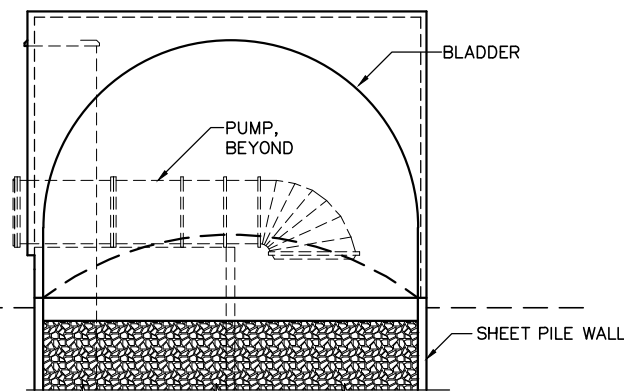
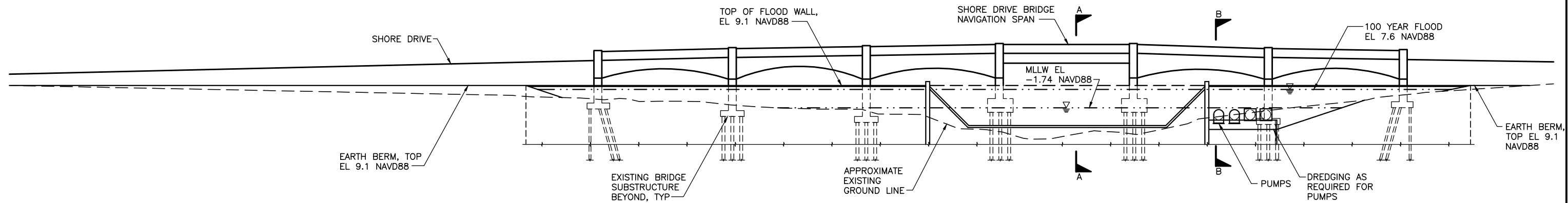
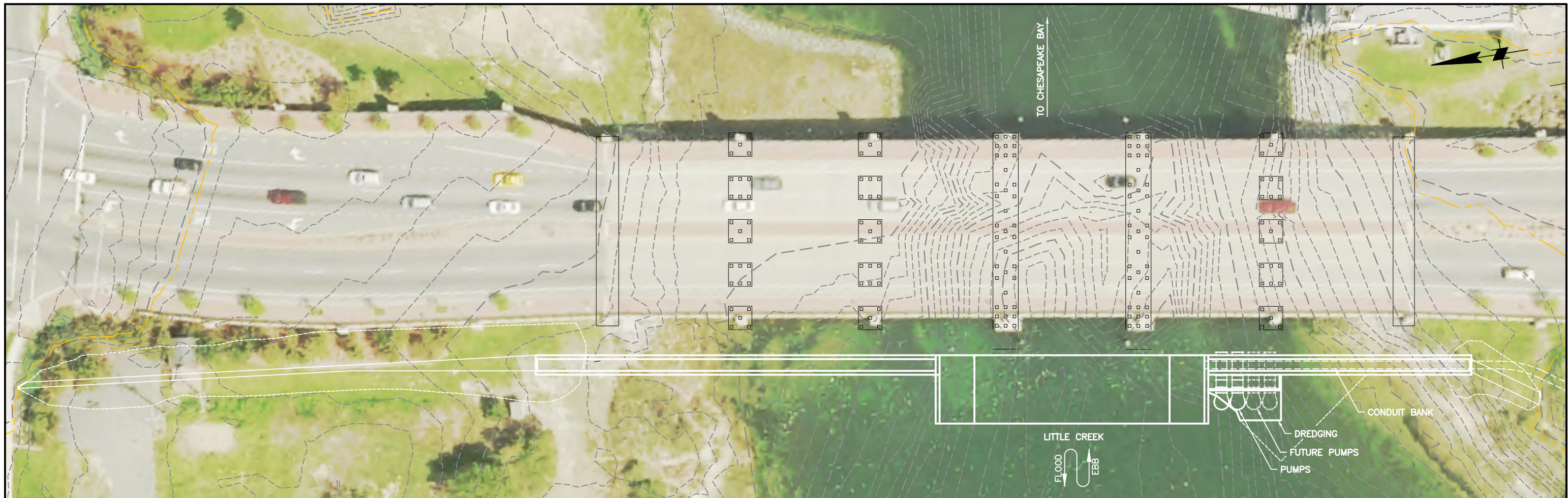


WALL SECTION AT PUMPS

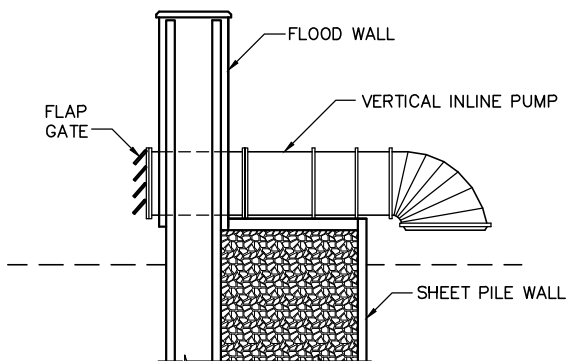


SECTION AT OBERMEYER GATE

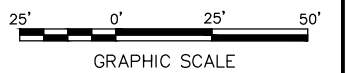
 moffatt & nichol					COASTAL FLOODING PROJECT NORFOLK, VIRGINIA		FIGURE 8-6
DSGN	DR KOONS	CHK .	PRETTY LAKE SHORE DRIVE BRIDGE OBERMEYER GATE OPTION – SECTIONS				
JOB NO. 6822-03	SCALE 1"=10'	DATE 12-14-2010					

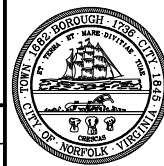


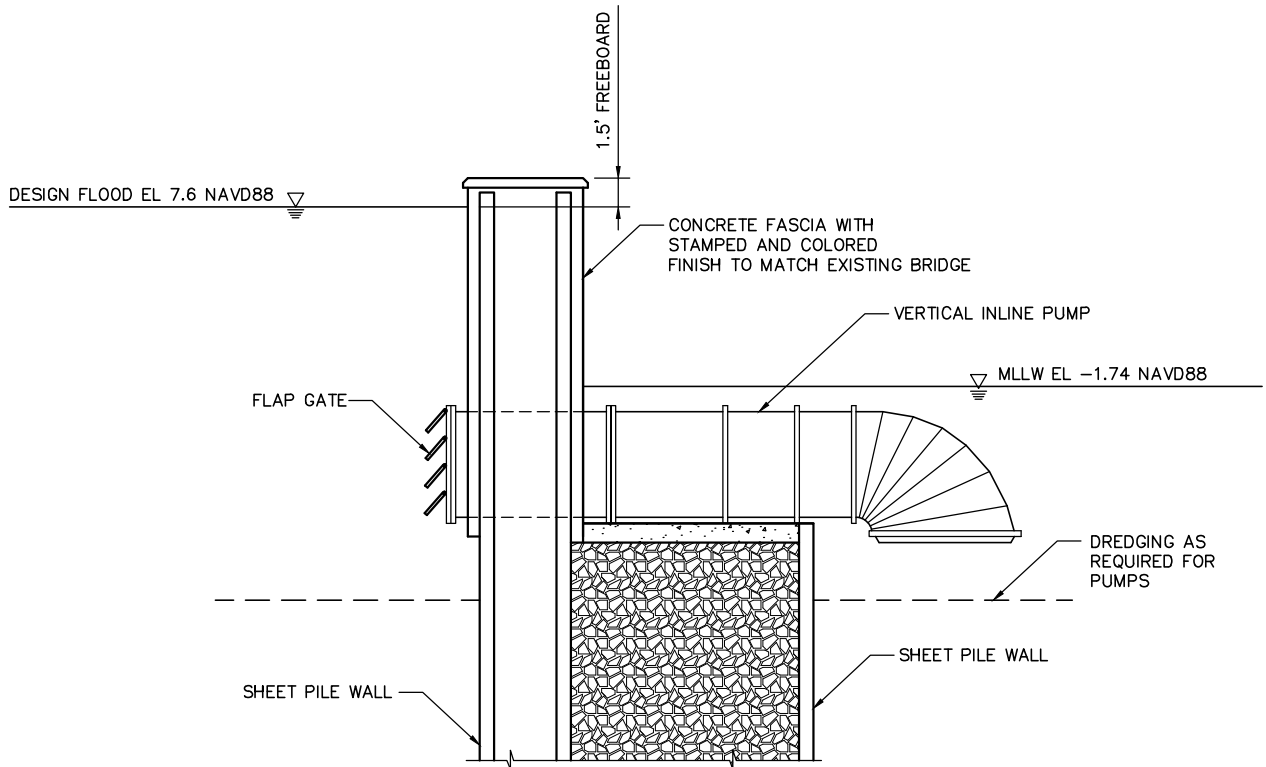
SECTION A-A



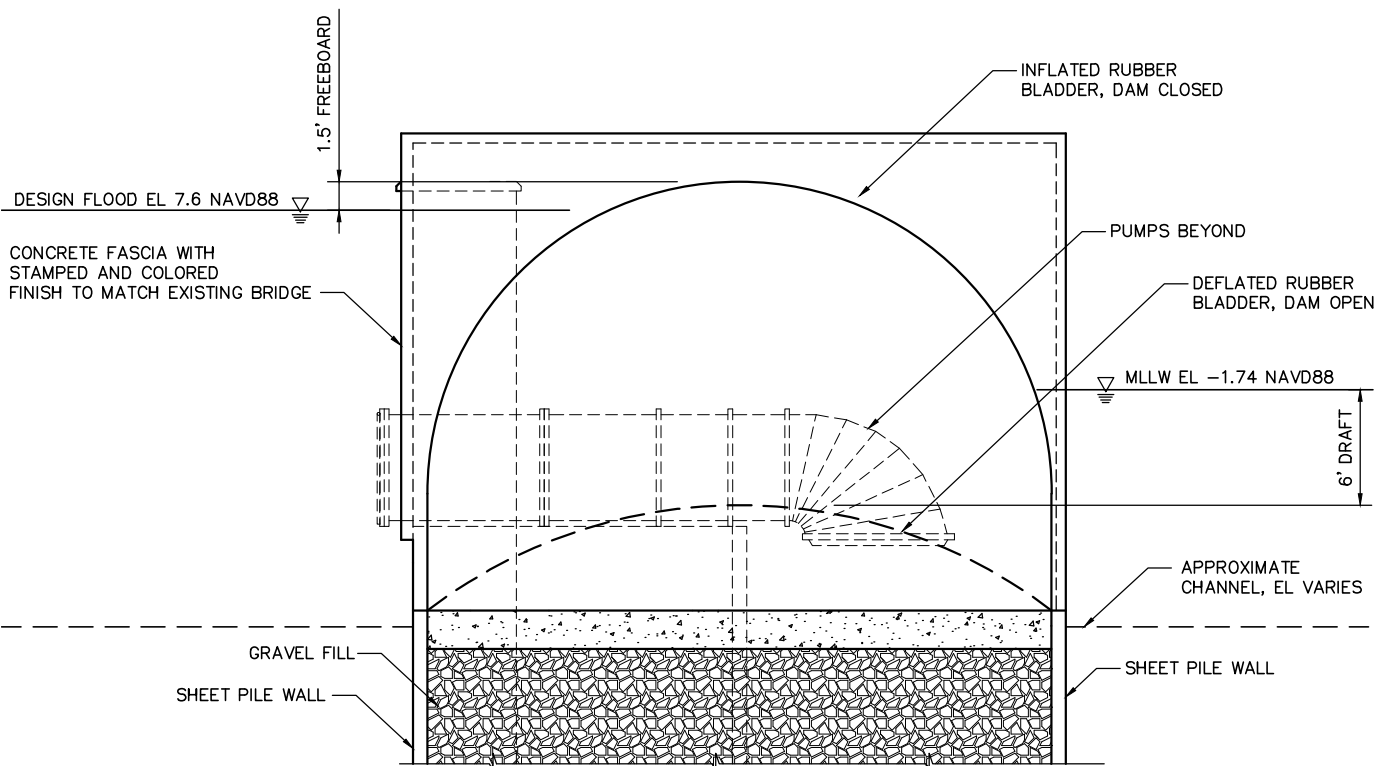
SECTION B-B



 				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA		FIGURE 8-7
				PRETTY LAKE		
				SHORE DRIVE BRIDGE INFLATABLE DAM/GATE OPTION		
DSGN	DR	CHK				
JOB NO. 6822-03	SCALE 1"=50'	DATE 12-14-2010				



WALL SECTION AT PUMPS



SECTION AT INFLATABLE BLADDER



COASTAL FLOODING PROJECT
NORFOLK, VIRGINIA

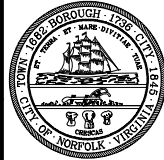
PRETTY LAKE
SHORE DRIVE BRIDGE
INFLATABLE DAM/GATE OPTION - SECTIONS

FIGURE

8-8

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



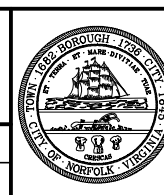
 moffatt & nichol 				COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	FIGURE 8-9
DSGN JOB NO. 6822-03	DR SCALE 1"=30'	CHK DATE 12-14-2010		PRETTY LAKE SHORE DRIVE BRIDGE SUBSTATION LOCATION	



- TOTAL BERM LENGTH = 2,465 LF
- TOTAL UPSTREAM BULKHEAD LENGTH = 20,760 LF
- TOTAL ROAD LENGTH = 3,900 LF



 moffatt & nichol		 fugro
DSGN	DR	CHK
JOB NO. 6822-03	SCALE 1"=1000'	DATE 12-14-2010



COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	
PRETTY LAKE	
BULKHEAD WALL, EARTHEN BERM AND ROAD RAISE OPTION – 10YR STORM SURGE	

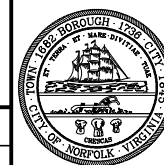
FIGURE
8-10



- TOTAL BERM LENGTH = 2,465 LF
- TOTAL UPSTREAM BULKHEAD LENGTH = 39,000 LF
- TOTAL ROAD LENGTH = 4,500 LF

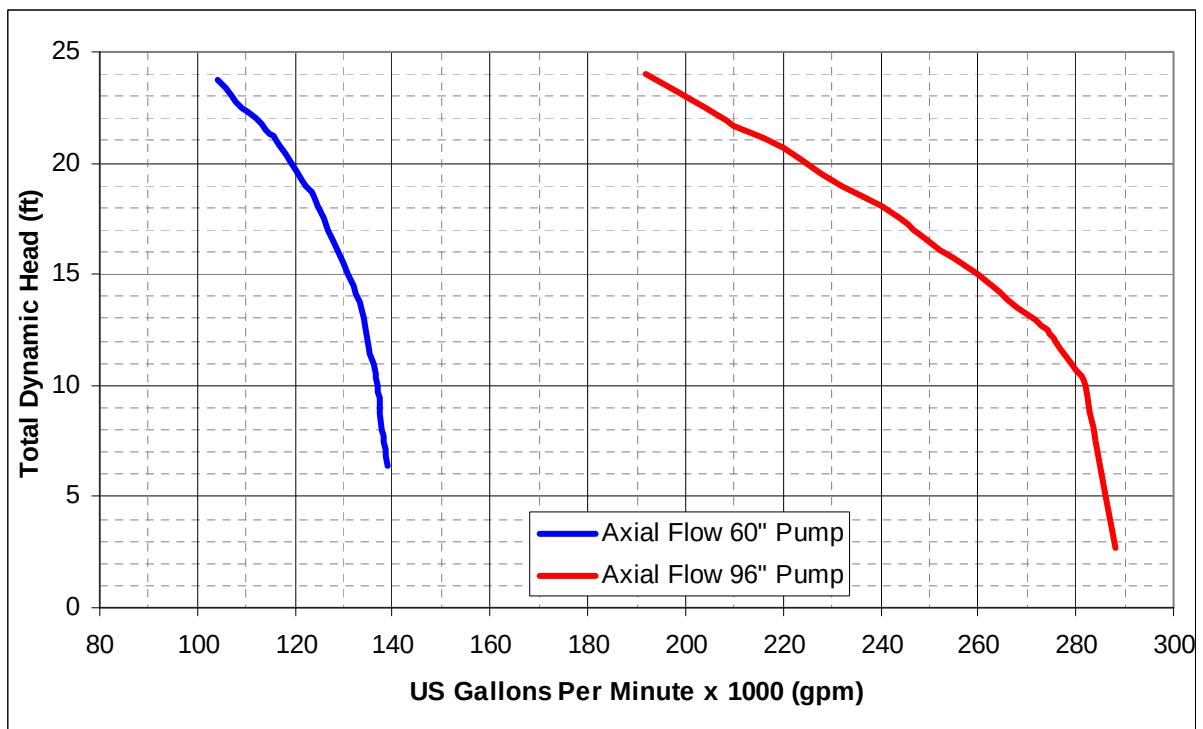


 		
DSGN	DR	CHK
JOB NO. 6822-03	SCALE 1"=1000'	DATE 12-14-2010



COASTAL FLOODING PROJECT NORFOLK, VIRGINIA	
PRETTY LAKE	
BULKHEAD WALL, EARTEN BERM AND ROAD RAISE OPTION – 100YR STORM SURGE	

FIGURE
8-11



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

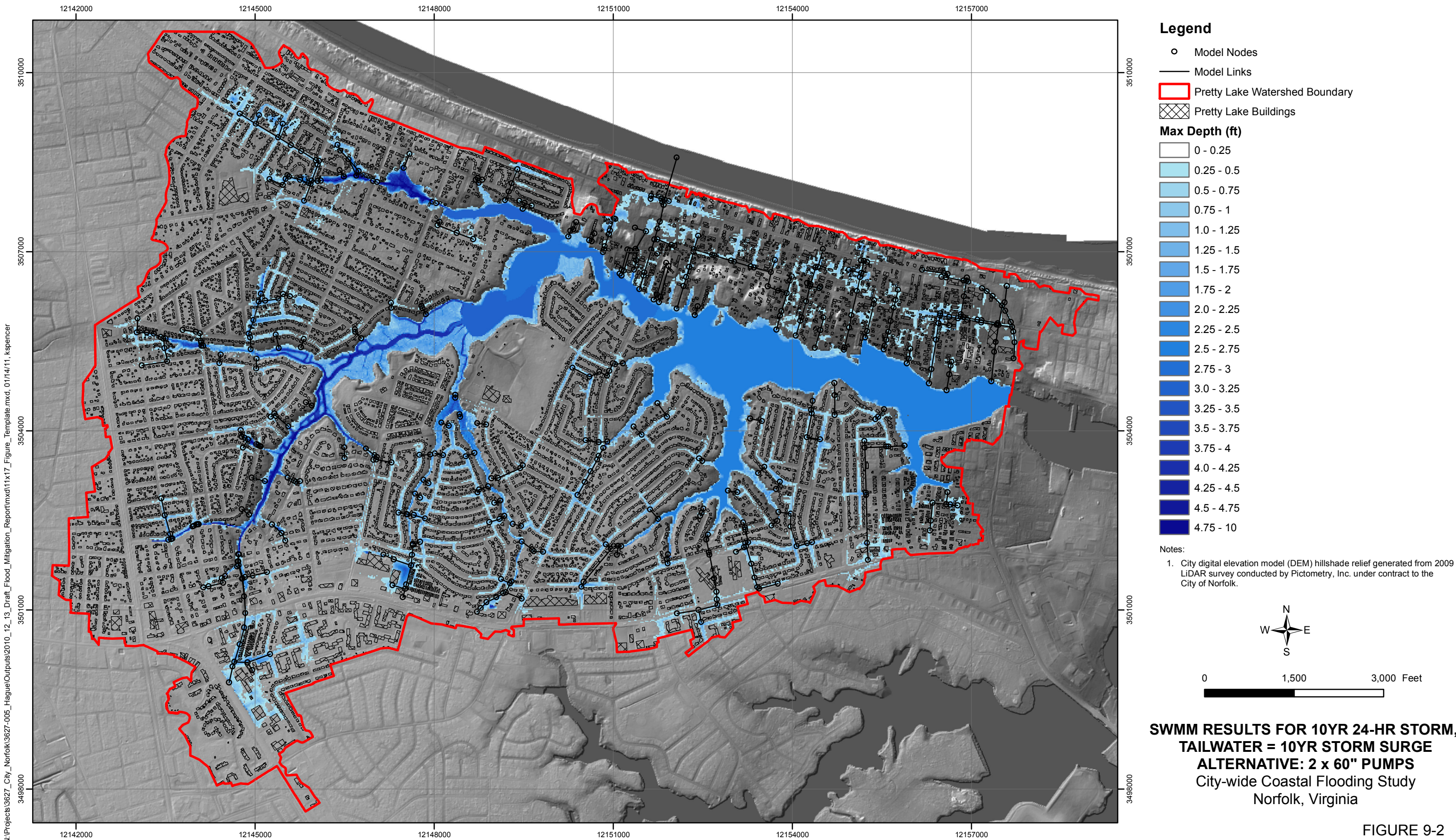


FIGURE 9-2

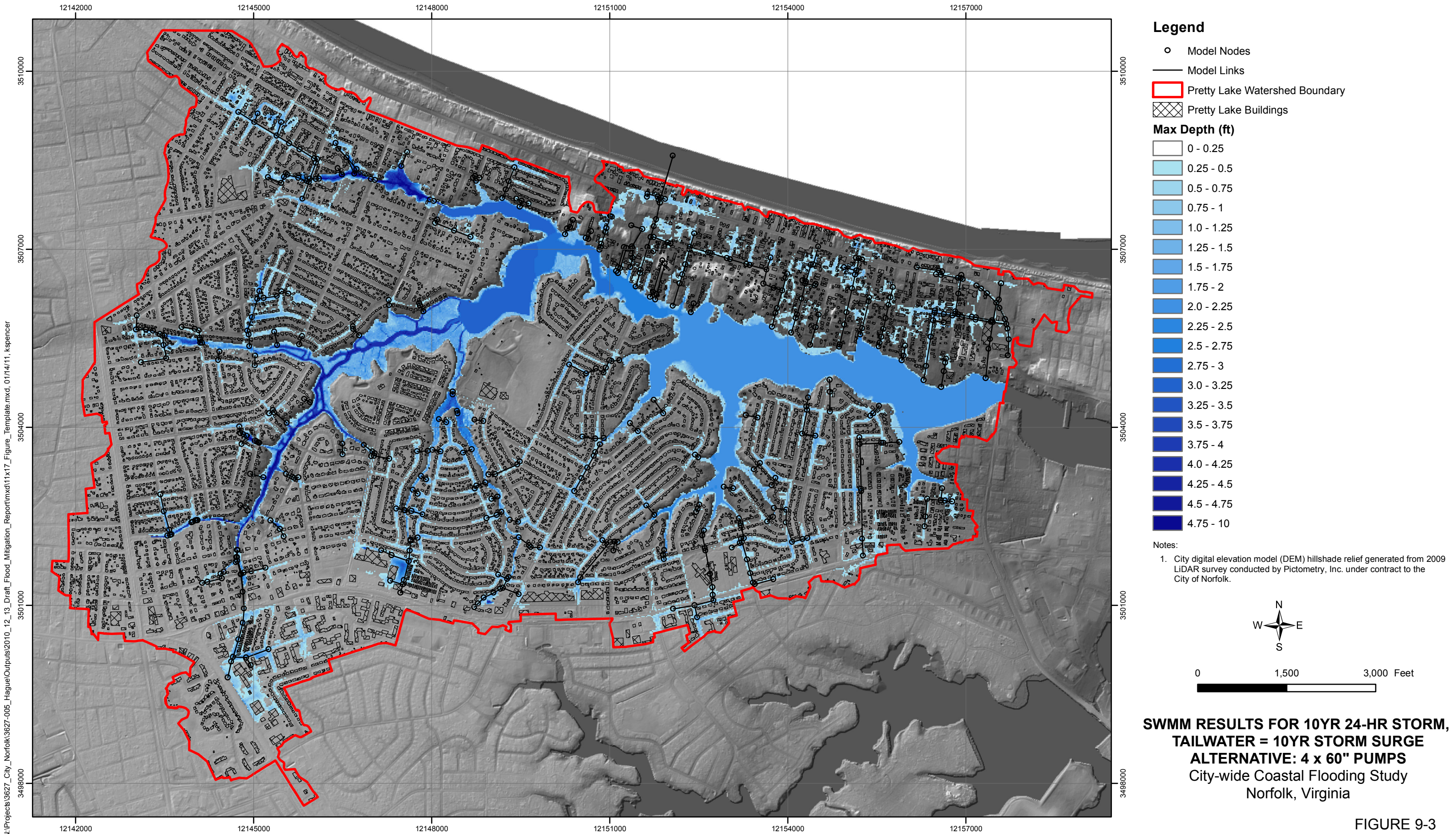


FIGURE 9-3

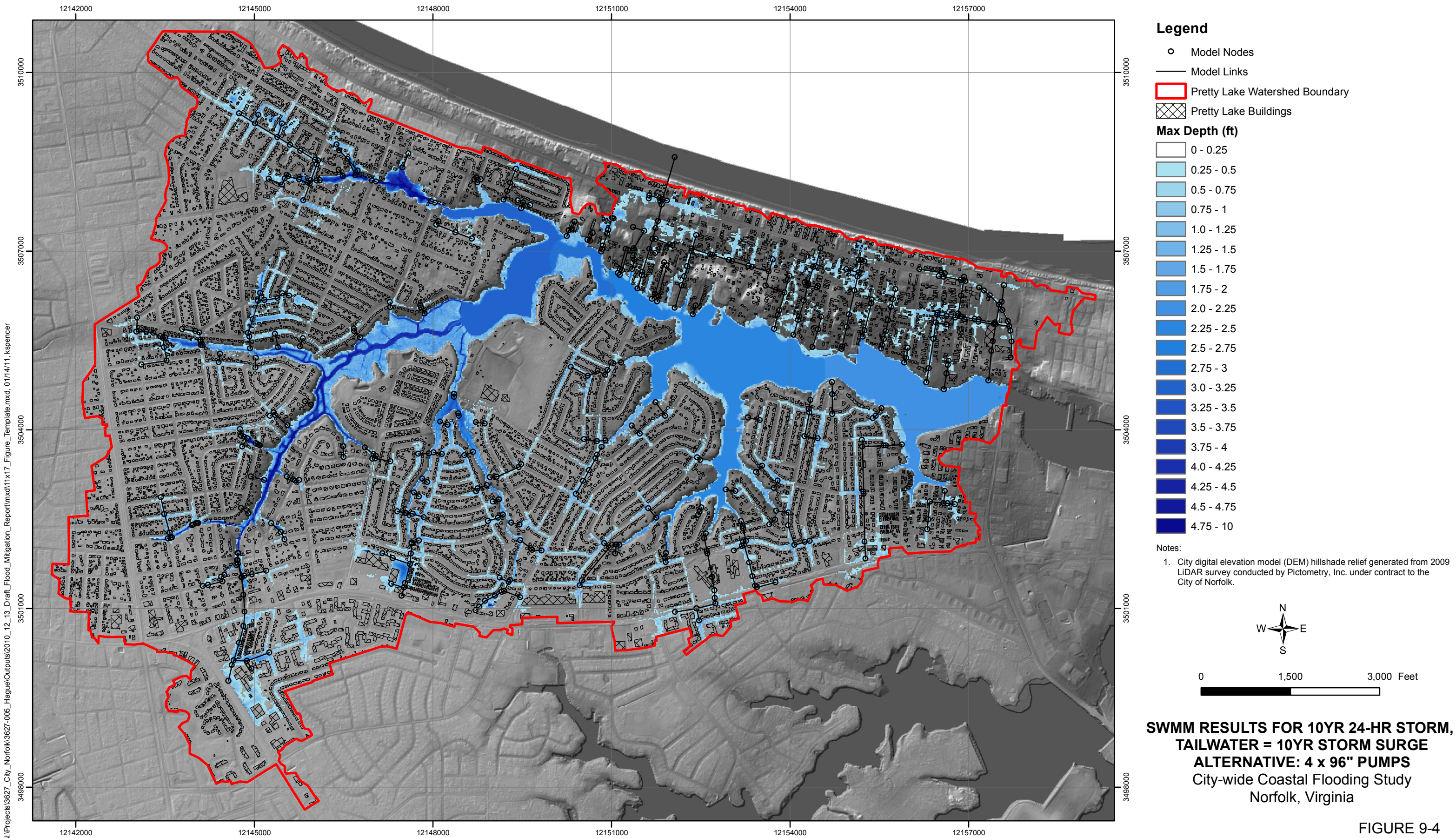


FIGURE 9-4

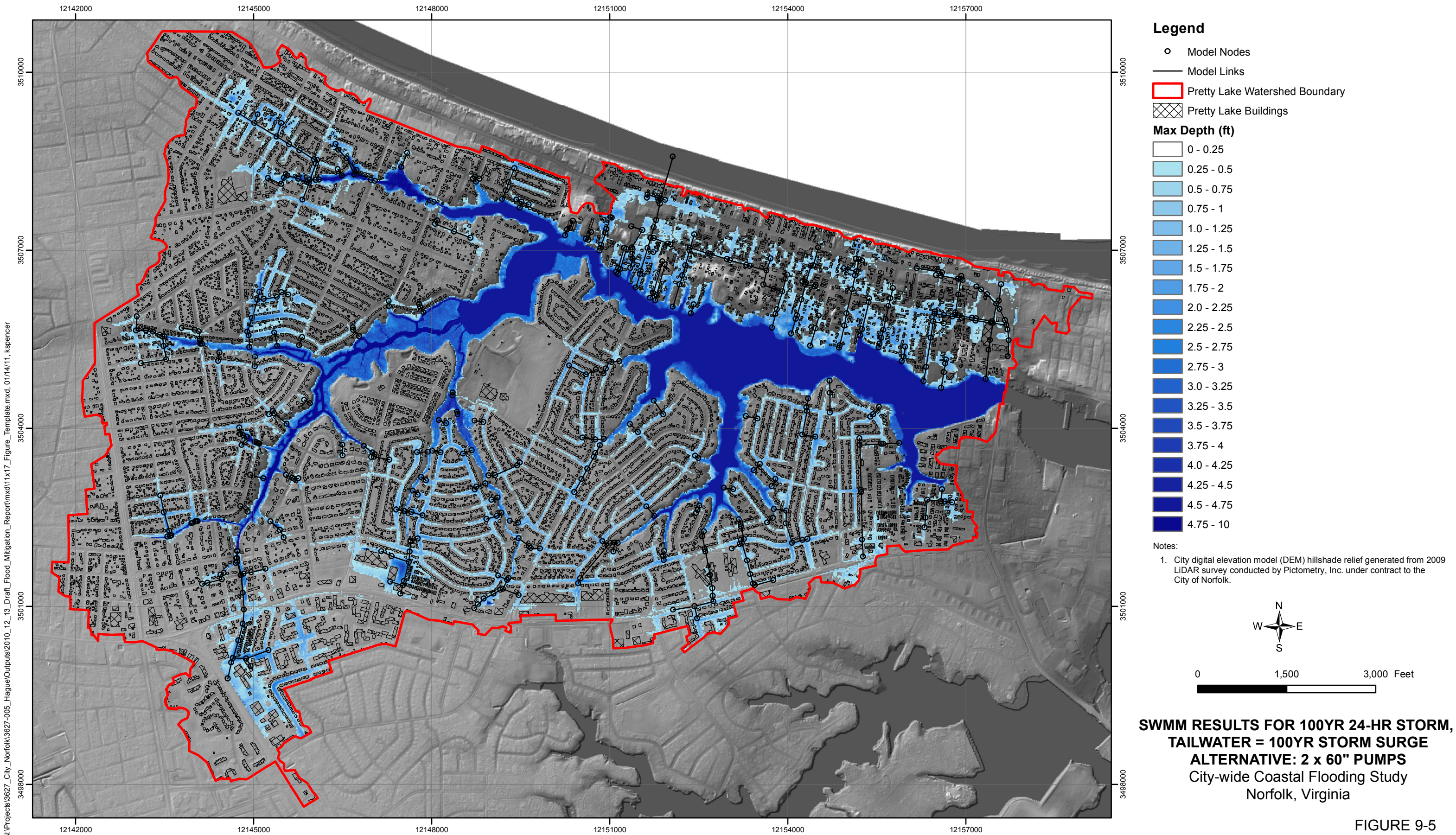


FIGURE 9-5

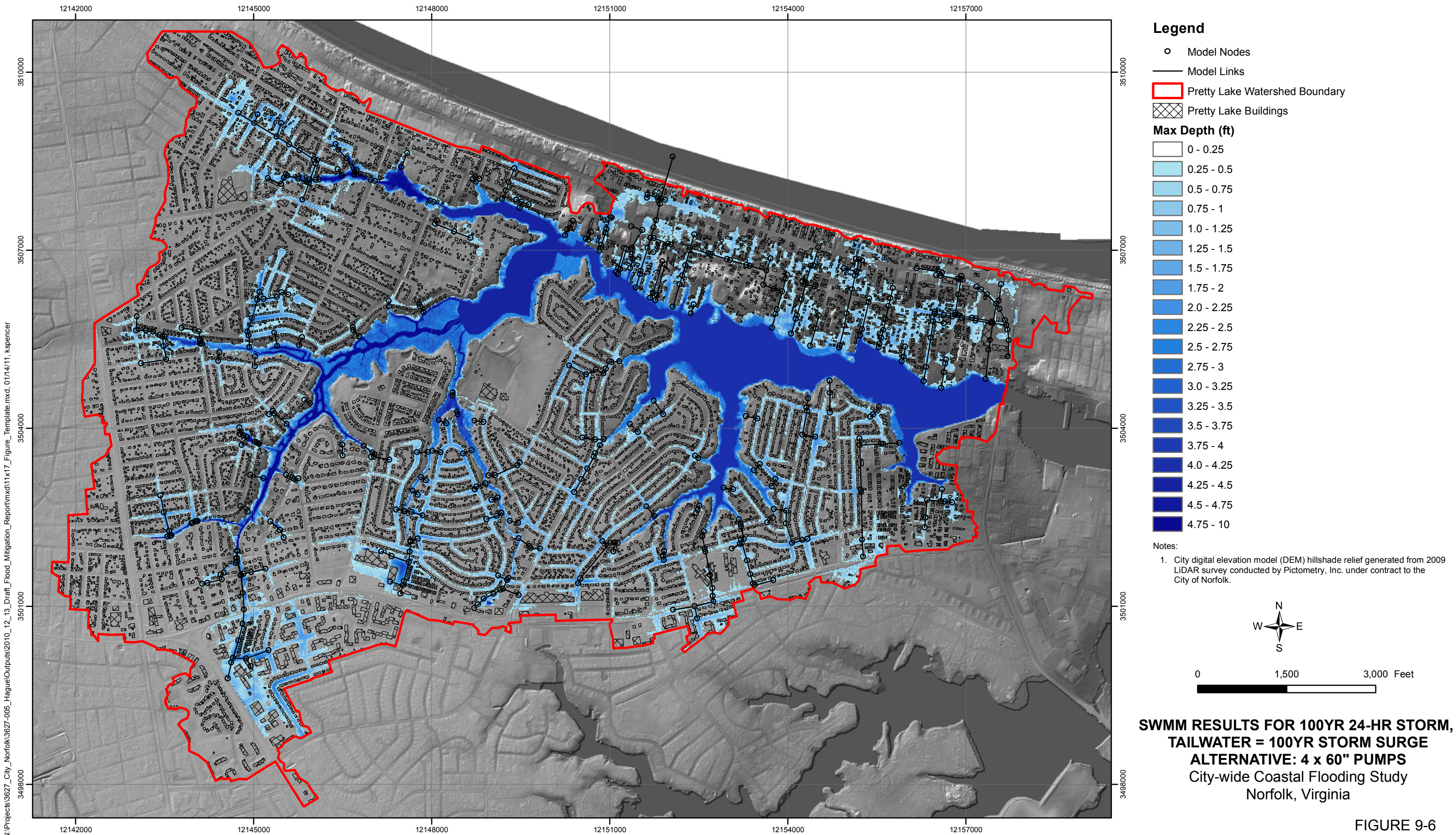


FIGURE 9-6

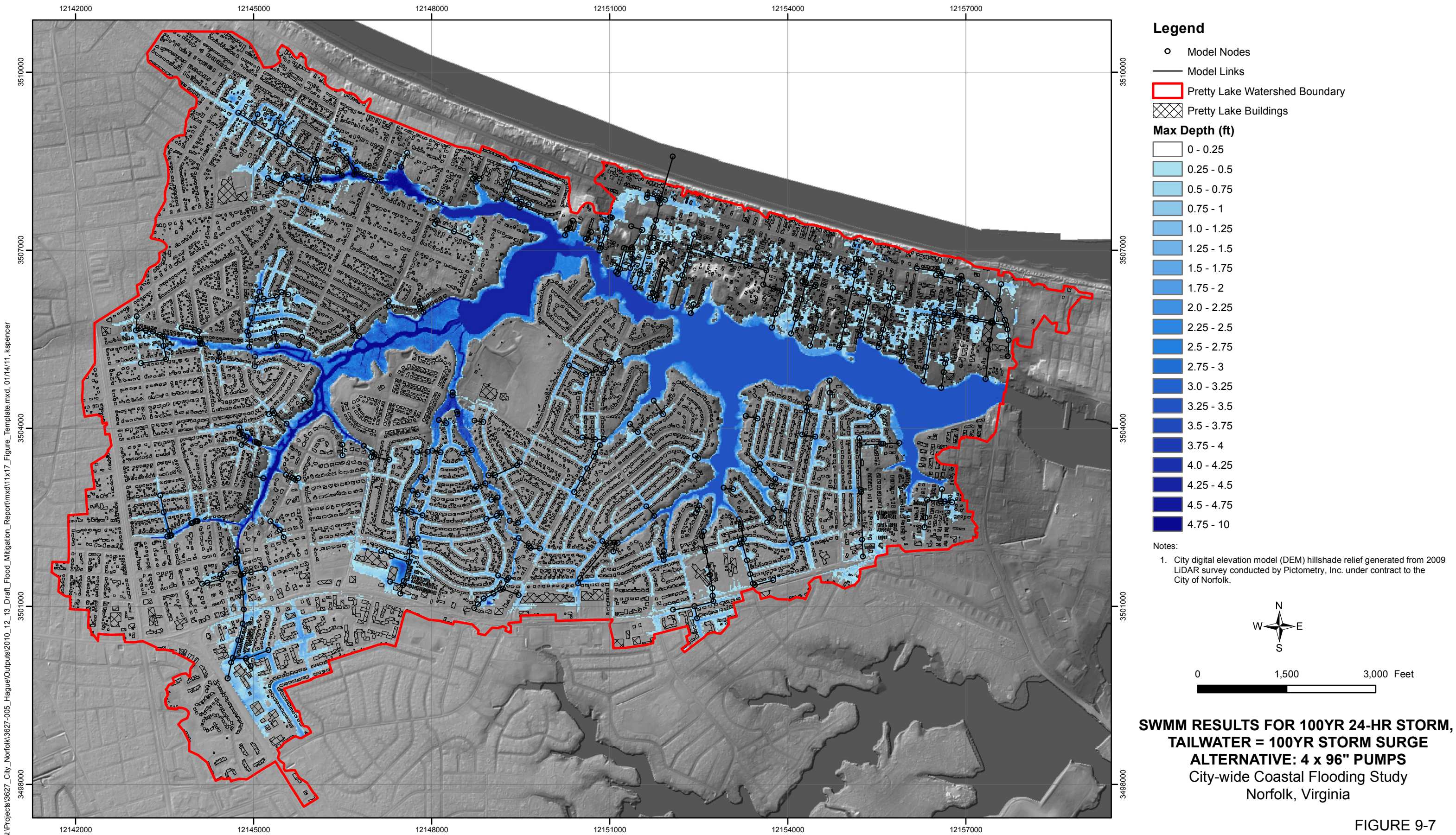


FIGURE 9-7

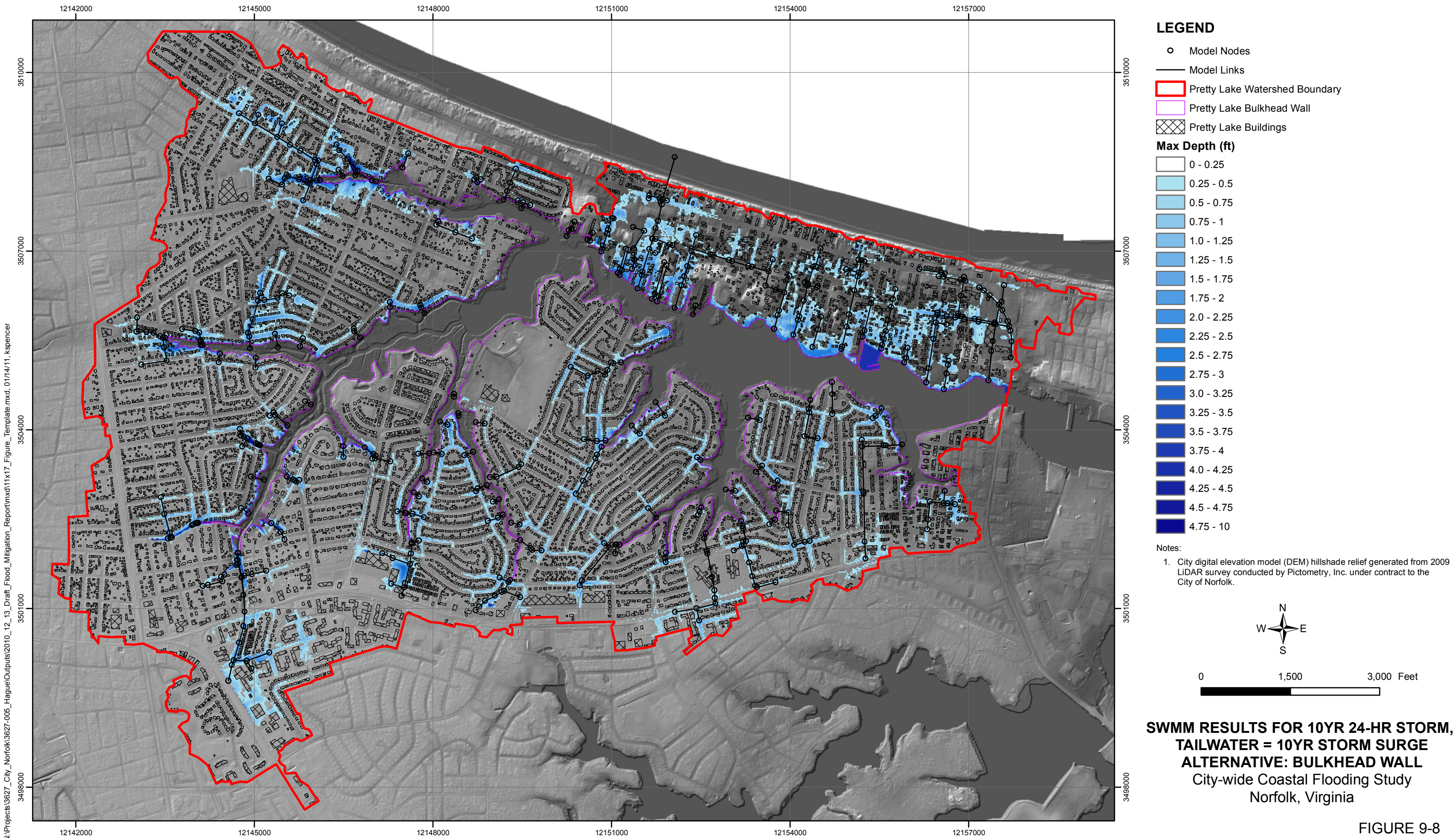


FIGURE 9-8

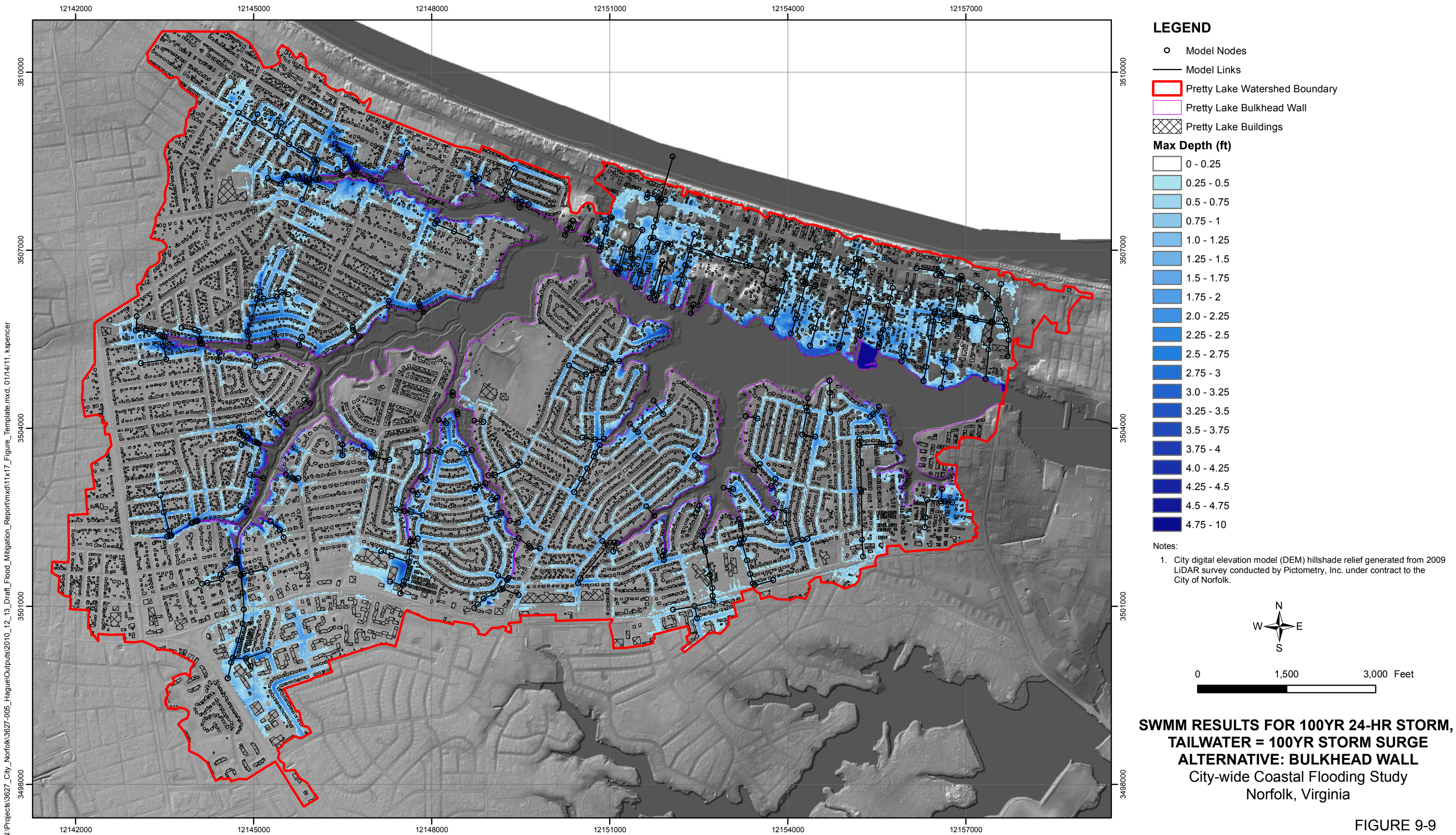
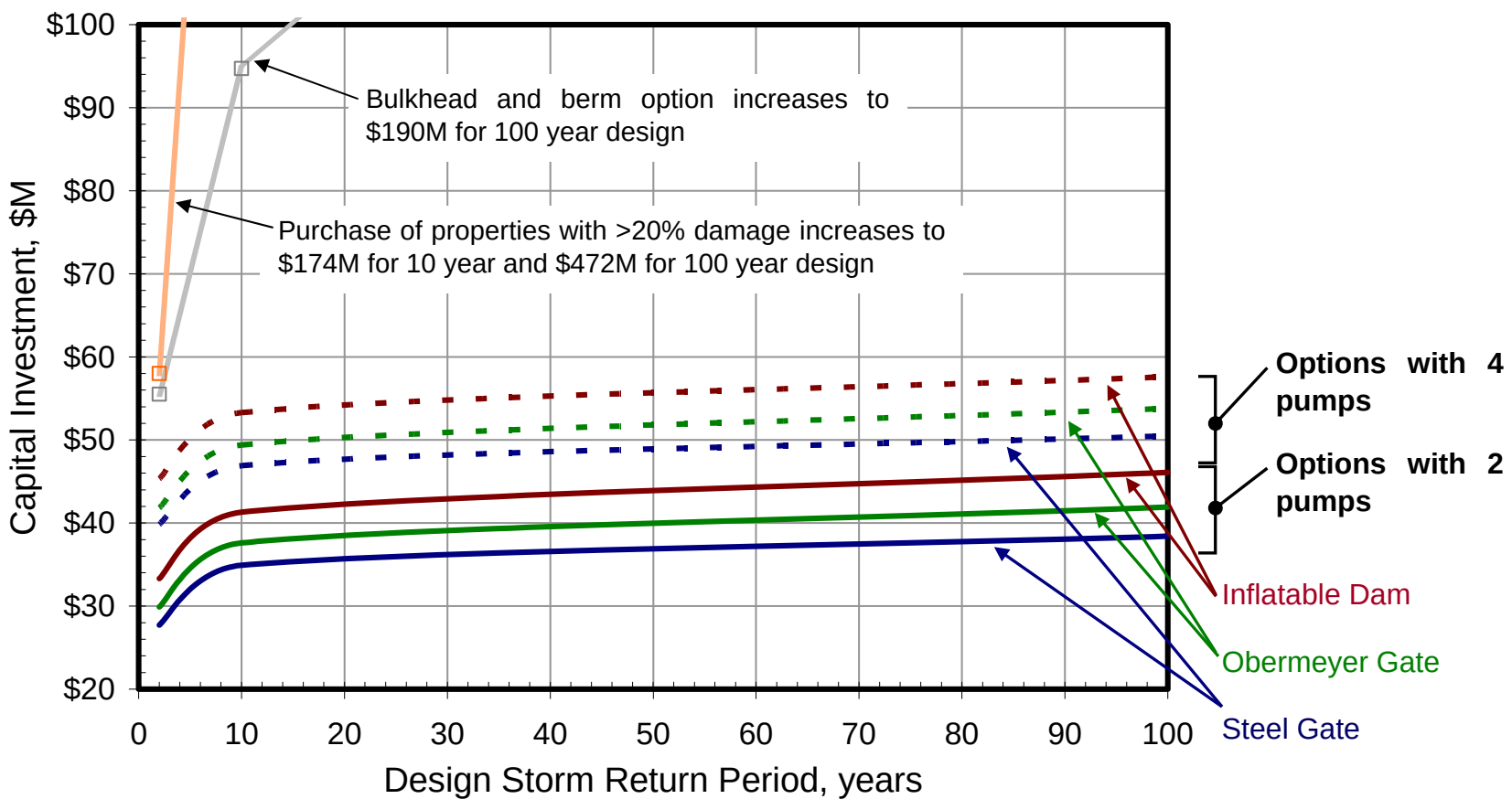
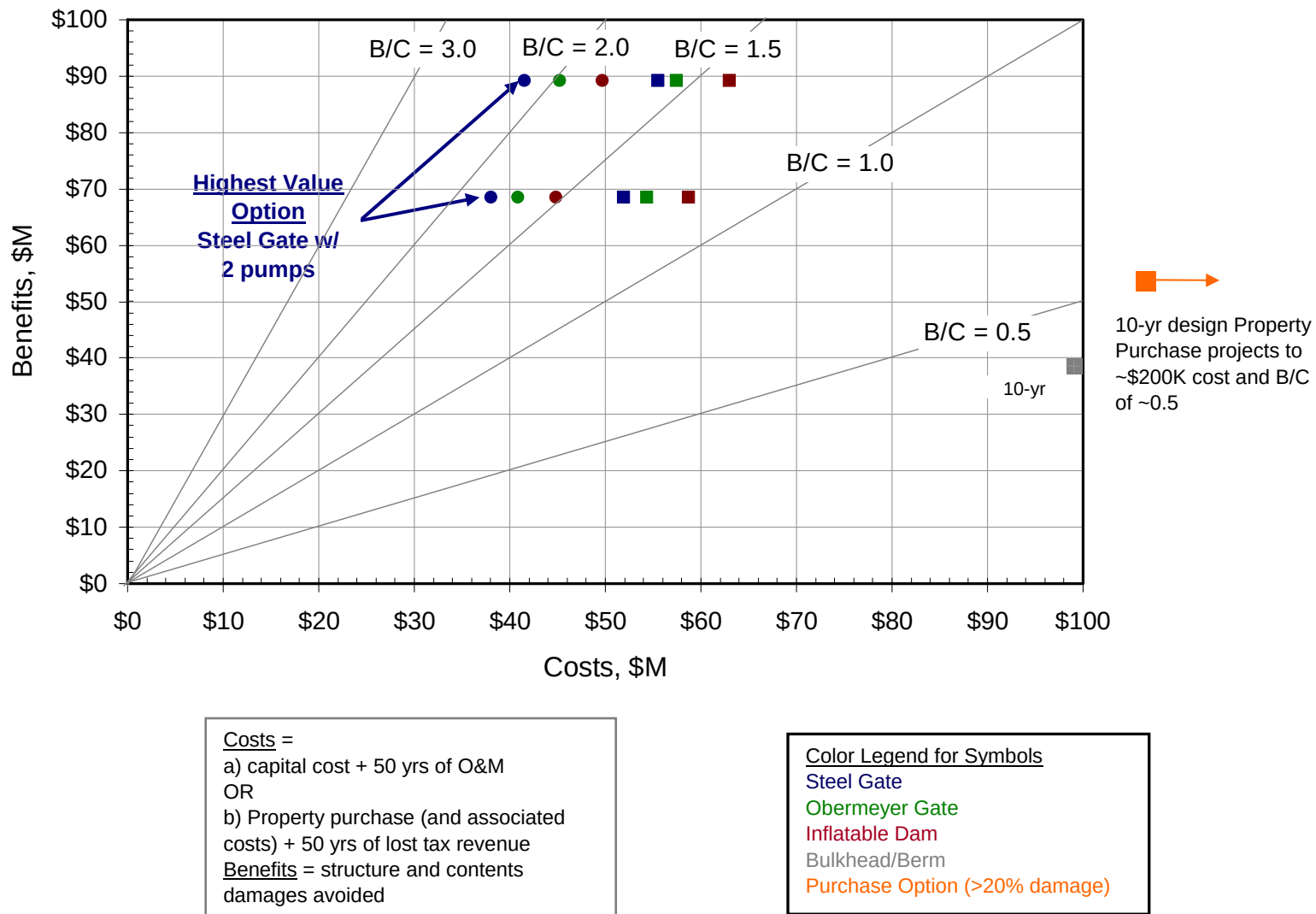


FIGURE 9-9



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



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