

# **PRELIMINARY ENGINEERING FEASIBILITY REPORT PRETTY LAKE WATERSHED**

## **City of Norfolk City-wide Coastal Flooding Contract Work Order No. 2**

Prepared for:  
CITY OF NORFOLK  
DEPARTMENT OF PUBLIC WORKS

March 2012  
Fugro Project No. 04.8111024





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

March 23, 2012  
Project No. 04.81110024

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

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

Subject: Preliminary Engineering Feasibility Report – Pretty Lake Watershed, City of Norfolk, City-wide Coastal Flooding Project, Work Order No. 2

Dear Mr. White:

Enclosed is Fugro Atlantic's report documenting our preliminary engineering feasibility report for the Pretty Lake Watershed. This report was authorized by Work Order #2, dated October 28, 2011 of the City-wide Coastal Flooding contract (City of Norfolk Contract 13062). This report provides our technical assessment of flood mitigation options in Pretty Lake and a preliminary engineering feasibility analysis of the preferred design alternative. Our report considers various options for mitigation approach, screens those options relative to their technical merit, flexibility, and projected costs. The report also includes consideration of several different criteria for flood mitigation in terms of severity of storm and potential future sea level rise.

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



Kevin R. Smith  
Senior Engineering Geologist/Project Manager



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

Copies Submitted:    ()

Draft

## CONTENTS

|                                                                                                                | Page |
|----------------------------------------------------------------------------------------------------------------|------|
| 1.0 EXECUTIVE SUMMARY .....                                                                                    | 1    |
| 1.1 Background .....                                                                                           | 1    |
| 1.2 Evaluation of Flood Mitigation Options for Pretty Lake .....                                               | 2    |
| 1.3 Preliminary (10% Level) Design and Feasibility of the Preferred Coastal Flood Mitigation Alternative ..... | 3    |
| 1.4 Potential Federal Participation .....                                                                      | 3    |
| 1.5 Potential Environmental Impacts and NEPA Process .....                                                     | 3    |
| 2.0 INTRODUCTION AND BACKGROUND .....                                                                          | 4    |
| 2.1 Project Background .....                                                                                   | 4    |
| 2.2 City-wide Coastal Flooding Program .....                                                                   | 4    |
| 2.2.1 Previous Phases .....                                                                                    | 4    |
| 2.2.2 Current Phase .....                                                                                      | 5    |
| 2.3 Authorization .....                                                                                        | 5    |
| 2.4 Incorporated Documents .....                                                                               | 6    |
| 3.0 ORGANIZATION OF REPORT AND LIST OF ACRONYMS .....                                                          | 7    |
| 3.1 Report Organization .....                                                                                  | 7    |
| 3.2 List of Acronyms .....                                                                                     | 7    |
| 4.0 THE PRETTY LAKE WATERSHED LOCATION AND DESCRIPTION .....                                                   | 8    |
| 4.1 Watershed Description and Receiving Water Body .....                                                       | 8    |
| 4.2 Topography .....                                                                                           | 8    |
| 4.3 Bathymetry .....                                                                                           | 9    |
| 4.4 Land Use .....                                                                                             | 9    |
| 4.5 Basin Rim Description .....                                                                                | 10   |
| 4.6 Site Conditions at Basin Outlet .....                                                                      | 10   |
| 4.7 Navigation Requirements .....                                                                              | 11   |
| 4.8 Subsurface Geotechnical Conditions (C-4) .....                                                             | 11   |
| 4.8.1 Geology and Subsurface Stratigraphy .....                                                                | 11   |
| 4.8.2 Design Subsurface Profiles for Concept Evaluation .....                                                  | 12   |
| 4.8.3 Future Considerations for Foundation Design .....                                                        | 13   |
| 5.0 COASTAL FLOODING: TIDE- / SURGE-DRIVEN TAILWATER ELEVATIONS (C-2) .....                                    | 14   |
| 5.1 Previous Interpretive Report and Study Implications .....                                                  | 14   |
| 5.2 Tides and Surge-Driven Water Level Elevations .....                                                        | 14   |
| 5.2.1 Long-term Measured Water Levels at Sewells Point .....                                                   | 14   |
| 5.2.2 Short-term Water Level Measurements in Other Parts of the City .....                                     | 15   |
| 5.3 Consideration of Future Sea Level Rise .....                                                               | 15   |
| 5.4 Coastal Tailwater Elevations for the Pretty Lake Watershed .....                                           | 16   |
| 6.0 COASTAL FLOODING: PRECIPITATION HYDROLOGY AND HYDRAULICS (C-2) .....                                       | 18   |
| 6.1 Rainfall and Precipitation .....                                                                           | 18   |

|        |                                                                                                 |    |
|--------|-------------------------------------------------------------------------------------------------|----|
| 6.2    | Elevation of Protection.....                                                                    | 18 |
| 6.2.1  | 10-Year and 100-Year Return Periods .....                                                       | 18 |
| 6.2.2  | Summary.....                                                                                    | 19 |
| 6.3    | Design Combinations of Coastal Water Elevation and Precipitation.....                           | 19 |
| 6.4    | EXISTING SYSTEM HYDROLOGIC/HYDRAULIC EVALUATION .....                                           | 20 |
| 6.4.1  | Selection of Model: XPSWMM .....                                                                | 20 |
| 6.4.2  | Development of Model Inputs.....                                                                | 20 |
| 6.4.3  | Rainfall Data .....                                                                             | 21 |
| 6.4.4  | Subcatchments .....                                                                             | 21 |
| 6.4.5  | Junctions.....                                                                                  | 21 |
| 6.4.6  | Conduits.....                                                                                   | 21 |
| 6.4.7  | Outfalls.....                                                                                   | 21 |
| 6.4.8  | Buildings .....                                                                                 | 22 |
| 6.4.9  | Topographic Data.....                                                                           | 22 |
| 6.4.10 | Model Calibration .....                                                                         | 22 |
| 6.4.11 | Existing System Flooding During Various Storm Events.....                                       | 22 |
| 7.0    | EXISTING CONDITION ESTIMATES OF DAMAGE COSTS .....                                              | 24 |
| 7.1    | Methodology .....                                                                               | 24 |
| 7.1.1  | Building Inventory Methodology .....                                                            | 24 |
| 7.1.2  | Building Values .....                                                                           | 25 |
| 7.1.3  | First Floor Elevations.....                                                                     | 25 |
| 7.1.4  | Depth Damage Functions - Structures and Contents .....                                          | 26 |
| 7.1.5  | Damage Assessment Estimates.....                                                                | 26 |
| 8.0    | DEVELOPMENT OF FLOOD DAMAGE MITIGATION ALTERNATIVE CONCEPTS.....                                | 28 |
| 8.1    | Introduction.....                                                                               | 28 |
| 8.2    | Flood Mitigation/Defense Strategies and Options .....                                           | 28 |
| 8.3    | Flood Damage Mitigation Options Eliminated .....                                                | 30 |
| 8.4    | Alternatives Selected for Further Evaluation .....                                              | 31 |
| 8.4.1  | Alternatives 1 through 3: Tidal Barrier with Tide Gate, Pumps, and Road Raise.....              | 32 |
| 8.4.2  | Alternative 4: Bulkhead Wall, Earthen Berm and Road Raise.....                                  | 34 |
| 8.4.3  | Alternative 5: Property Buyout.....                                                             | 34 |
| 9.0    | EVALUATION OF CONCEPTUAL ALTERNATIVES .....                                                     | 35 |
| 9.1    | Hydrologic / Hydraulic Modeling Evaluations .....                                               | 35 |
| 9.1.1  | Induced Flooding With Mitigation Alternatives .....                                             | 35 |
| 9.1.2  | Residual Flooding With Mitigation Alternatives.....                                             | 36 |
| 9.2    | Flood Damage Reduction Estimates.....                                                           | 38 |
| 10.0   | CONCEPTUAL LEVEL OPINION OF PROBABLE COSTS FOR FLOOD DAMAGE MITIGATION ALTERNATIVES (C-19)..... | 39 |
| 10.1.1 | Capital Costs.....                                                                              | 39 |
| 10.1.2 | Operational & Maintenance (O&M) Costs with Respect to Design Life (C-15).....                   | 40 |
| 10.1.3 | Summary.....                                                                                    | 40 |

|                                                                                                                          |    |
|--------------------------------------------------------------------------------------------------------------------------|----|
| 11.0 SELECTION OF PREFERRED CONCEPTUAL ALTERNATIVE .....                                                                 | 42 |
| 11.1 Benefit - Cost (B/C) Analysis Ratio (C-19) .....                                                                    | 42 |
| 11.1.1 Probability of Storm Events and Their Re-Occurrence Related to Damages .....                                      | 42 |
| 11.1.2 Design life of the Mitigation Option .....                                                                        | 42 |
| 11.1.3 Present Value of Project .....                                                                                    | 42 |
| 11.1.4 B/C Ratio .....                                                                                                   | 42 |
| 12.0 PRELIMINARY CIVIL AND STRUCTURAL ENGINEERING DESIGN OF PREFERRED ALTERNATIVE 1A (ITEMS C-6, C-7,C-8) .....          | 44 |
| 12.1 Functional Design Requirements and Design Criteria .....                                                            | 44 |
| 12.1.1 Functional design requirements .....                                                                              | 44 |
| 12.1.2 Applicable codes and standards .....                                                                              | 45 |
| 12.1.3 Design loads and load combinations .....                                                                          | 45 |
| 12.2 Overview of the Preferred Preliminary Design Alternative .....                                                      | 45 |
| 12.2.1 Overview of Appendix E Drawings .....                                                                             | 46 |
| 12.3 Design of Primary Smith Creek Tidal Barrier and Gate .....                                                          | 47 |
| 12.3.1 Fixed Wall .....                                                                                                  | 47 |
| 12.3.2 Steel Gate .....                                                                                                  | 48 |
| 12.4 Design of Pumping Facilities .....                                                                                  | 49 |
| 12.4.1 General description .....                                                                                         | 49 |
| 12.4.2 Electrical and mechanical requirements .....                                                                      | 50 |
| 12.5 Design of Additional Perimeter Barriers .....                                                                       | 50 |
| 12.5.1 General description .....                                                                                         | 50 |
| 12.5.2 East: along Little Creek waterfront to Cobb's Marina .....                                                        | 51 |
| 12.5.3 South-east: along Little Creek JEB boundary .....                                                                 | 51 |
| 12.5.4 South: Shore Drive / Little Creek Road intersection west along Little Creek Road .....                            | 52 |
| 12.5.5 Berm: north of Pretty Lake Ave. ....                                                                              | 52 |
| 12.6 Corrosion Mitigation .....                                                                                          | 52 |
| 12.7 Real Estate Requirements .....                                                                                      | 53 |
| 12.8 Facility / Utility Relocation Requirements .....                                                                    | 53 |
| 13.0 PRELIMINARY DESIGN LEVEL OPINION OF PROBABLE COSTS AND BENEFIT-COST ANALYSIS FOR PREFERRED DESIGN ALTERNATIVE ..... | 55 |
| 13.1 Purpose of This Section .....                                                                                       | 55 |
| 13.2 Capital Costs .....                                                                                                 | 55 |
| 13.3 Operational & Maintenance Costs with Respect to Design Life (C-15) .....                                            | 59 |
| 13.4 Benefit-Cost Ratio Analysis .....                                                                                   | 59 |
| 14.0 ENVIRONMENTAL OBJECTIVES AND REQUIREMENTS (ITEM C-13) .....                                                         | 60 |
| 14.1 Overview of NEPA and Regulatory Requirements .....                                                                  | 60 |
| 14.2 Coordination with Federal, State, and Local Stakeholders .....                                                      | 61 |
| 14.3 NOTABLE AREA FEATURES and Likelihood of Potential Impacts .....                                                     | 62 |
| 14.3.1 Water Quality .....                                                                                               | 62 |
| 14.3.2 Shoreline and Shallow Bottom Habitat .....                                                                        | 63 |
| 14.3.3 Wetlands .....                                                                                                    | 64 |

|                                                                      |    |
|----------------------------------------------------------------------|----|
| 14.3.4 Marina and Recreational Boating .....                         | 64 |
| 14.3.5 Little Creek Joint Expeditionary Base .....                   | 64 |
| 14.3.6 Local Community .....                                         | 64 |
| 15.0 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN (ITEM C-10)..... | 65 |
| 15.1 Main Floodwall and Gate .....                                   | 65 |
| 15.2 Pumping Facilities.....                                         | 65 |
| 15.3 Additional Perimeter Walls.....                                 | 65 |
| 15.4 Access and Staging Areas.....                                   | 65 |
| 16.0 IMPLICATION OF POTENTIAL FUTURE SEA LEVEL RISE .....            | 66 |
| 17.0 CONCLUSIONS AND RECOMMENDATIONS .....                           | 67 |
| 18.0 LIMITATIONS.....                                                | 69 |
| 19.0 REFERENCES.....                                                 | 70 |

Draft



## TABLES (Presented Within Text)

|                                                                                                                                                          | Page |
|----------------------------------------------------------------------------------------------------------------------------------------------------------|------|
| Table 4-1: Summary of Watershed Topography .....                                                                                                         | 8    |
| Table 4-2: Pretty Lake Watershed Land Use Classifications .....                                                                                          | 9    |
| Table 4-3: Low Ground Surface Conditions along Watershed Perimeter.....                                                                                  | 10   |
| Table 5-1: Tide Elevations at Sewells Point for Various Return Periods.....                                                                              | 14   |
| Table 5-2: Predicted Storm Surge Levels and Return Periods, Current Sea Level Elevation and after 0.5- and 1.0-Foot Increases in Relative Sea Level..... | 16   |
| Table 5-3: Tailwater Correction from Sewells Point and Allowance for Sea Level Rise at the Pretty Lake Watershed.....                                    | 17   |
| Table 5-4: Tailwater Elevations at Sewells Point and the Pretty Lake Watershed .....                                                                     | 17   |
| Table 6-1: NOAA Return Frequency Rainfall Depths for Norfolk KORF Airport .....                                                                          | 18   |
| Table 6-2: Design Combinations of Tailwater and Precipitation.....                                                                                       | 19   |
| Table 6-3: Existing Condition XP-SWMM Model Results.....                                                                                                 | 23   |
| Table 7-1: Typical Building Classifications .....                                                                                                        | 25   |
| Table 7-2: Existing Condition Structure and Contents Flood Damage Estimates.....                                                                         | 27   |
| Table 8-1: Flood Mitigation Alternatives Feasibility Assessment.....                                                                                     | 30   |
| Table 8-2: Flood Damage Mitigation Alternatives Evaluated .....                                                                                          | 32   |
| Table 8-3: Elevation of Structures Based on Storm Events .....                                                                                           | 32   |
| Table 9-1: Summary of XP-SWMM Results for Pump Alternatives Modeling .....                                                                               | 36   |
| Table 9-2: Summary of XP-SWMM Results for Bulkhead Wall Alternatives Modeling .....                                                                      | 37   |
| Table 9-3: Comparison of XP-SWMM Results for Pump vs. Bulkhead Wall Alternatives.....                                                                    | 37   |
| Table 9-4: Estimated Flood Damage Reductions .....                                                                                                       | 38   |
| Table 10-1: Opinion of Probable Capital Costs for Flood Mitigation Alternatives .....                                                                    | 39   |
| Table 10-2: Alternative Operational & Maintenance Costs .....                                                                                            | 41   |
| Table 10-3: Property Buyout Revenue Loss.....                                                                                                            | 41   |
| Table 11-1: Benefit-Cost Ratio (relative to damage to structure and contents).....                                                                       | 43   |
| Table 12-1: Design Water Surface Elevations for Components of Alternative 1A Preliminary Design .....                                                    | 44   |
| Table 12-2: Known Likely Utility Conflicts with Preferred Preliminary Alternative.....                                                                   | 54   |
| Table 13-1: Opinion of Probable Capital Costs for Preferred Preliminary Design Alternative....                                                           | 56   |
| Table 13-2: Operational & Maintenance Costs for Preferred Preliminary Design Alternative ....                                                            | 59   |

## FIGURES (Presented After Text)

|                                                                                     | Figure |
|-------------------------------------------------------------------------------------|--------|
| Project Area .....                                                                  | 2-1    |
| Pretty Lake Watershed Boundary.....                                                 | 2-2    |
| Basin Outlet .....                                                                  | 2-3    |
| Topography.....                                                                     | 4-1a   |
| Bathymetry.....                                                                     | 4-1b   |
| Statistical Distribution of Topography .....                                        | 4-2    |
| Existing Land Use .....                                                             | 4-3    |
| Basin Rim Elevation.....                                                            | 4-4    |
| Former Shoreline Structures .....                                                   | 4-5    |
| Photograph of Cone Penetration Test.....                                            | 4-6    |
| Key to Subsurface Cross Sections .....                                              | 4-7    |
| Preliminary Subsurface Cross Section A-A' .....                                     | 4-8    |
| Preliminary Subsurface Cross Section B-B' .....                                     | 4-9    |
| Water Levels for Various Return Periods .....                                       | 5-1    |
| Implications of Future Sea Level Rise .....                                         | 5-2    |
| Tailwater Phenomena .....                                                           | 5-3    |
| SWMM Model Subcatchments .....                                                      | 6-1    |
| SWMM Existing Conditions 10yr 24-hr Storm, Tailwater = MHHW .....                   | 6-2    |
| SWMM Existing Conditions 100yr 24-hr Storm, Tailwater = MHHW .....                  | 6-3    |
| SWMM Existing Conditions 10yr 24-hr Storm, Tailwater = 10yr Storm Surge .....       | 6-4    |
| SWMM Existing Conditions 100yr 24-hr Storm, Tailwater = 100yr Storm Surge.....      | 6-5    |
| Extent of 10yr Storm Surge .....                                                    | 6-6    |
| Extent of 100yr Storm Surge .....                                                   | 6-7    |
| Depth Damage Function Concept.....                                                  | 7-1    |
| Flood Damage Estimates 10yr 24-hr Storm, Tailwater = MHHW .....                     | 7-2    |
| Flood Damage Estimates, 100yr 24-hr Storm, Tailwater = MHHW .....                   | 7-3    |
| Flood Damage Estimates 10yr 24-hr Storm, Tailwater = 10yr Storm Surge.....          | 7-4    |
| Flood Damage Estimates, 100yr 24-hr Storm, Tailwater = 100yr Storm Surge.....       | 7-5    |
| Summary of Flood Damage Estimates, 10yr 24-hr Storm, Tailwater = 10yr Storm Surge.. | 7-6    |
| Summary of Flood Damage Estimates, 100yr 24-hr Storm, Tailwater = 100yr Storm Surge | 7-7    |
| Schematic of Tide Gate Type Options .....                                           | 8-1    |
| Tidal Barrier with Tide Gate, Pumps, and Road Raise .....                           | 8-2    |
| Shore Drive Bridge Steel Gate Option.....                                           | 8-3    |
| Shore Drive Bridge Steel Gate Option - Sections .....                               | 8-4    |
| Shore Drive Bridge Obermeyer Gate Option .....                                      | 8-5    |
| Shore Drive Bridge Obermeyer Gate Option - Sections .....                           | 8-6    |
| Shore Drive Bridge Inflatable Dam/Gate Option .....                                 | 8-7    |
| Shore Drive Bridge Inflatable Dam/Gate Option - Sections .....                      | 8-8    |
| Pretty Lake Substation Location.....                                                | 8-9    |
| Bulkhead Wall, Earthen Berm and Road Raise Option = 10yr Storm Surge.....           | 8-10   |
| Bulkhead Wall, Earthen Berm and Road Raise Option = 100yr Storm Surge.....          | 8-11   |
| Pump Curves for SWMM Models .....                                                   | 9-1    |



|                                                                                   |      |
|-----------------------------------------------------------------------------------|------|
| SWMM 2 x 60-inch Pumps Condition 10yr 24-hr, Tailwater = 10yr Storm Surge .....   | 9-2  |
| SWMM 4 x 60-inch Pumps Condition 10yr 24-hr, Tailwater = 10yr Storm Surge .....   | 9-3  |
| SWMM 4 x 96-inch Pumps Condition 10yr 24-hr, Tailwater = 10yr Storm Surge .....   | 9-4  |
| SWMM 2 x 60-inch Pumps Condition 100yr 24-hr, Tailwater = 100yr Storm Surge ..... | 9-5  |
| SWMM 4 x 60-inch Pumps Condition 100yr 24-hr, Tailwater = 100yr Storm Surge ..... | 9-6  |
| SWMM 4 x 96-inch pumps Condition 100yr 24-hr, Tailwater = 100yr Storm Surge ..... | 9-7  |
| SWMM Bulkhead Wall Condition 10yr 24-hr, Tailwater = 10yr Storm Surge.....        | 9-8  |
| SWMM Bulkhead Wall Condition 100yr 24-hr, Tailwater = 100yr Storm Surge.....      | 9-9  |
| Estimated Project Costs for Flood Mitigation Options.....                         | 11-1 |
| Benefit/Cost Evaluation.....                                                      | 11-2 |
| Alternative Not Considered .....                                                  | 12-1 |

## APPENDICES

|            |                                                   |
|------------|---------------------------------------------------|
| APPENDIX A | SUBSURFACE CONDITIONS                             |
| APPENDIX B | MODELING RESULTS                                  |
| APPENDIX C | OPINION OF PROBABLE COST AND OPERATIONAL COST     |
| APPENDIX D | BENEFIT/COST RATIO CALCULATIONS                   |
| APPENDIX E | PRELIMINARY (10% LEVEL) DESIGN PLANS AND SECTIONS |



## 1.0 EXECUTIVE SUMMARY

### 1.1 BACKGROUND

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

In recognition of those considerations, the City initiated a City-Wide Coastal Flooding Evaluation via Contract 13062. The information from the City-wide Coastal Flooding study is considered relevant for not only developing design criteria and designs of public works improvements but also provides important information for various planning studies and emergency response plans within the City.

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

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

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

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

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

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

## **1.2 EVALUATION OF 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 infrastructure improvements consisting of a flood wall – with a gate to be closed during coastal surge events – can mitigate coastal flooding including much of the worst effects of extreme extra-tidal events from hurricanes and nor'easters. This primary flood barrier on the upstream (western side) of Shore Drive, in combination with additional, shorter walls around portions of the watershed's perimeter, selective street grade raising, and pumps to pass excess storm water over the flood barrier, 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 individual 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.

Some flooding mitigation could be achieved by improving the existing storm sewerage system. The existing system appears to meet current City standards for collection and discharge of at least the amount of runoff from a 10-year return period rainfall event.

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

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

The opinion of probable capital cost for the preferred alternative, as presently formulated, is approximately \$46.4 million, in present value dollars. A detailed breakdown of line items, quantities, and unit costs is provided in Table 13-1.

### **1.4 POTENTIAL FEDERAL PARTICIPATION**

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

### **1.5 POTENTIAL ENVIRONMENTAL IMPACTS AND NEPA PROCESS**

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



## **2.0 INTRODUCTION AND BACKGROUND**

### **2.1 PROJECT BACKGROUND**

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

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

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

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

### **2.2 CITY-WIDE COASTAL FLOODING PROGRAM**

#### **2.2.1 Previous Phases**

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



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

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

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

## **2.2.2 Current Phase**

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

## **2.3 AUTHORIZATION**

Work Order No. 2 for the City-Wide Coastal Flooding Study was issued by the City on October 28, 2011. The intent of this current work order is to provide a Preliminary Engineering Feasibility Report that can be used by the City for evaluation, budgeting and project development scheduling. The Fugro team's work scope included the following activities:

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

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

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

## 2.4 INCORPORATED DOCUMENTS

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

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

Draft

### **3.0 ORGANIZATION OF REPORT AND LIST OF ACRONYMS**

#### **3.1 REPORT ORGANIZATION**

This section will be updated prior to final report submittal.

#### **3.2 LIST OF ACRONYMS**

This section will be updated prior to final report submittal.

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

## 4.0 THE PRETTY LAKE WATERSHED LOCATION AND DESCRIPTION

### 4.1 WATERSHED DESCRIPTION AND RECEIVING WATER BODY

The Pretty Lake watershed is in the northeast portion of the City of Norfolk (Figure 2-1). 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).

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.

### 4.2 TOPOGRAPHY

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

The watershed is 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. The ground surface slope varies throughout the watershed.

**Table 4-1: Summary of Watershed Topography**

| Elevation (ft, NAVD88) | Number of Acres | Cumulative Number of Acres | Percent of Watershed | Cumulative Percent |
|------------------------|-----------------|----------------------------|----------------------|--------------------|
| Lower than 3           | 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%              |

| Elevation (ft, NAVD88) | Number of Acres | Cumulative Number of Acres | Percent of Watershed | Cumulative Percent |
|------------------------|-----------------|----------------------------|----------------------|--------------------|
| 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                    | 0.3%                 | 99.4%              |
| 15 to 25               | 847             | 102,610                    | 0.6%                 | 100.0%             |

### 4.3 BATHYMETRY

The bathymetric elevation of the area near Shore Drive bridge is shown on Figure 4-1b. Bathymetry contours shown on Figure 4-1b was created from a compilation of bathymetry data. Bathymetry contours west of the bridge are based on a single beam survey conducted in 1999. Bathymetry contours beneath the bridge and to the east were created using sounding data digitized from NOAA Chart 12255 (2008).

Bathymetry appears to be lower on the west side of the bridge than on the east or downstream side of the bridge. The area west of the bridge, which has been subjected to dredging in the past, is as low as El. -16 to -18 feet NAVD88 in the center channel. East of the bridge, the center channel appears to be as low as El. -10 feet NAVD88.

### 4.4 LAND USE

The number of acres and percent of the watershed with the following land use classification (as defined by the City's Planning Department) is summarized in Table 4-2. Figure 4-3 presents a map of the land use in the 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 4-2: 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                  |
| Industrial                 | 76              | 3.4                  |
| Mixed Use                  | 109             | 4.9                  |
| Vacant                     | 1330            | 60.4                 |

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

#### 4.5 BASIN RIM DESCRIPTION

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

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

| Elevation (ft, NAVD88) | Number of Low Segments | Length of Low Segments (ft) |
|------------------------|------------------------|-----------------------------|
| 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                         |

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, berms, or road raising will be needed, and the required lengths can range from 100 to almost 1,000 feet. Based on review of the available data however, it would appear that mitigation can be afforded up to and beyond the 100-yr surge event.

#### 4.6 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 4-5 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 bridge in the subsurface, which may present obstructions for future subsurface structures (e.g. piles, sheetpile walls, etc.).

## **4.7 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.

## **4.8 SUBSURFACE GEOTECHNICAL CONDITIONS (C-4)**

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. After the initial review of the existing geotechnical data, three cone penetration tests were conducted by Fugro in March 2012 to provide additional geotechnical data for characterization and validate the older geotechnical information. Figures 4-8 and 4-9 present cross sections that depict interpreted subsurface conditions at the basin outlet. Locations of the CPT soundings are included on the plan view of Figure 4-8 and 4-9. The CPT logs are provided in Appendix A.

### **4.8.1 Geology and Subsurface Stratigraphy**

#### Regional Geology

The Pretty Lake area is located in the Coastal Plain physiographic province. Flat-lying plains and terraces dominate the landscape. The Coastal Plain is underlain by a wedge of Cretaceous to Holocene age sediments that thicken to the east and pinch out at the Fall Line approximately 70 miles west of the project area. Jurassic-Triassic age basement rocks lie approximately 1,800 feet beneath the site. The wedge of Cretaceous and younger sediments were deposited as a result of multiple marine transgressions and regressions. Sediments within the upper 150 feet beneath the site are Pliocene to Recent in age. The Pliocene and younger sediments have been deposited and subsequently eroded in places during the rising and falling sea levels that resulted from glacial and interglacial periods.

#### Historical Development

The project area has been modified by man's activities during the last several decades. The historical development of the Pretty Lake outlet and change in the Shore Drive bridge alignment are evident. The potential for encountering remnants of historical construction should be recognized when planning flood mitigation projects in the project area. Figure 4-5 shows the location of the former bridge observed in a 1937 aerial photograph. Figure 4-5 also shows a comparison of the shorelines between 2009 and 1937 aerial photographs. The shoreline comparison indicates where artificial fill was placed to create the 2009 shoreline and former, buried shoreline (and possible buried shoreline features such as rock, concrete, or other debris) remnants may be located.

#### Site Conditions

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, and Pliocene age Yorktown formation. Interpreted subsurface conditions are shown on Figures 4-8 and 4-9.



**Artificial Fill.** The artificial fill is depicted above the section and is assumed to be associated with shoreline development and construction of the Shore Drive Bridge. Exploration logs suggest the artificial fill is primarily sand soils and between 5 and 15 feet thick.

**Quaternary Alluvium.** Fine to coarse-grained alluvium underlies the artificial fill. The alluvium has two distinct layers; a loose to dense sand interbedded with layers of fine-grained material over a predominately fine-grained layer interbedded with layers of loose sand. The loose to dense sand layer varies in thickness between 15 and 50 feet and extends down to about El. -35 to 43 feet (Figure 4-8). The sand layer is interbedded with fine-grained sediments that are about 2 to 10 feet thick. Lateral continuity of the fine-grained layers is somewhat variable. They appear to extend laterally 10s to 100s of feet and do not appear to represent a ubiquitous, low permeability layer.

The fine-grained layer was encountered by the explorations between El. -35 and -75 feet. The fine-grained layer varies in thickness from about 15 to 25 feet. The 2012 Fugro CPT soundings provided confirmation that our assumption of low blow counts reported in the 1999 borings in this unit are indicative of fine-grained, weak materials.

**Yorktown Formation.** Pliocene age Yorktown formation sediments underlie the fine-grained alluvium at an elevation of about El. -60 to -73 feet. Elevation of the Yorktown surface is inferred to represent an erosion surface and thus can vary in elevation over short distances (e.g. Figure 4-9). The Yorktown formation is generally comprised of marine silty sand deposits. Regionally, this unit is commonly the end-bearing strata for many piled foundations. CPT C-3 penetrated the base of the Yorktown formation sand unit at El. -136 feet. A stiff clay unit was encountered beneath the Yorktown formation sand unit in CPT C-3 and is about 48 feet thick. CPT C-3 terminated in dense material inferred to be sandy sediments at El. -187 ft.

#### 4.8.2 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.

#### 4.8.3 Future Considerations for Foundation Design

Preliminary evaluation of the subsurface conditions indicates that the sheet pile will need to extend below the silt and clay layer. For future foundation design however, high quality in situ and laboratory data will be needed to better evaluate the following:

- Variability of the subsurface condition at the project location especially at areas farther away from the Shore Drive bridge where no geotechnical data is available;
- Shear strength properties that govern sheet pile wall design such as soil internal friction angle and interface friction angle to better determine earth pressure coefficients.
- Sheet pile penetration depth at the proposed area;
- Lateral earth pressure on the sheet pile wall system and hence better predict lateral deflection and lateral yielding of the sheet pile wall;
- The need for anchored blocks or piles.

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

### 5.1 PREVIOUS INTERPRETIVE REPORT AND STUDY IMPLICATIONS

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

### 5.2 TIDES AND SURGE-DRIVEN WATER LEVEL ELEVATIONS

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

#### 5.2.1 Long-term Measured Water Levels at Sewells Point

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

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

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

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

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

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

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

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 temporal variations between the local tide and those at Sewells Point. ***A 0.4 foot decrease in tailwater elevations, below the base Sewells Point value, is recommended for the Pretty Lake watershed to account for temporal, local effects.***

### 5.3 CONSIDERATION OF FUTURE SEA LEVEL RISE

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

Published data and evaluations (NOAA, 2010) interpret that the recent rate of **relative sea level rise** at Sewells Point is 1.46 feet/century. To evaluate how a continuation of that rate of sea level rise will affect flooding in the City, the return periods associated with various tide elevations at Sewells Point have been computed assuming a 0.5 foot and a 1.0 foot rise in future sea level. At the NOAA estimated rate of 1.46 feet/century, these rises correspond approximately to years 2045 and 2080, respectively.

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

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

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

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

#### **5.4 COASTAL TAILWATER ELEVATIONS FOR THE PRETTY LAKE WATERSHED**

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

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

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

| 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).

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

**Table 5-4: Tailwater Elevations at Sewells Point and the 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                    | 6.0                                    | 6.4                                                           |
| 50                    | 6.6                                    | 7.0                                                           |
| 100                   | 7.2                                    | 7.6                                                           |

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

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

Coastal flooding events with high tailwater elevations in Little Creek are highly likely to be associated with intense and/or prolonged rainfall over the entire Pretty Lake watershed. Any engineered solution for mitigating coastal flooding in the Pretty Lake watershed must account for this interaction between the storm water system and the elevated water surface in the receiving waters.

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

### 6.1 RAINFALL AND PRECIPITATION

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

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

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

### 6.2 ELEVATION OF PROTECTION

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

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

#### 6.2.1 10-Year and 100-Year Return Periods

As noted, the water level elevations at Sewells Point that are associated with the 100-year and 10-year return periods are elevation +7.2 and +5.2 feet NAVD88, respectively. Those



water levels at Sewells Point correspond to design water elevations in the Pretty Lake watershed equal to elevation +7.6 and +5.6 feet NAVD88.

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

### 6.2.2 Summary

The protection associated with an elevation +7.6-ft NAVD88 is approximately equivalent to a 100-year return period design based on current sea level. After a future 1-foot sea level rise, the +7.6-ft crest elevation corresponds to approximately a 31-year return period event.

The protection associated with an elevation +5.6-ft NAVD88 is approximately equivalent to a 10-year return period design based on current sea level. After a future 1-foot sea level rise, the +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 one to two feet to accommodate sea level rise. For the purposes of the conceptual alternatives evaluation study, it was determined that the designs of the primary, over water floodwalls would be designed with a 1.5 ft freeboard above the 100-year return period elevation of +7.6-ft NAVD88. This elevation would allow for 0.5 foot of freeboard in a future scenario with 1 foot of sea level rise. This factor should be confirmed with the City and other stakeholders at each future, subsequent design phase.

## 6.3 DESIGN COMBINATIONS OF COASTAL WATER ELEVATION AND PRECIPITATION

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

**Table 6-2: Design Combinations of Tailwater and Precipitation**

| Design Case                 | 24-hr Design Storm Precipitation (in) | Tailwater Elevation (ft, NAVD88) |
|-----------------------------|---------------------------------------|----------------------------------|
| 1yr RP rainfall, MHHW tide  | 2.93                                  | +1.6                             |
| 2yr RP rainfall, MHHW tide  | 3.57                                  | +1.6                             |
| 10yr RP rainfall, MHHW tide | 5.51                                  | +1.6                             |

| Design Case                               | 24-hr Design Storm Precipitation (in) | Tailwater Elevation (ft, NAVD88) |
|-------------------------------------------|---------------------------------------|----------------------------------|
| 25yr RP rainfall, MHHW tide               | 6.82                                  | +1.6                             |
| 50yr RP rainfall, MHHW tide               | 7.96                                  | +1.6                             |
| 100yr RP rainfall, MHHW tide              | 9.21                                  | +1.6                             |
| 1yr RP rainfall, 1yr RP coastal surge     | 2.93                                  | +3.6                             |
| 2yr RP rainfall, 2yr RP coastal surge     | 3.57                                  | +4.2                             |
| 10yr RP rainfall, 10yr RP coastal surge   | 5.51                                  | +5.6                             |
| 25yr RP rainfall, 25yr RP coastal surge   | 6.82                                  | +6.4                             |
| 50yr RP rainfall, 50yr RP coastal surge   | 7.96                                  | +7.0                             |
| 100yr RP rainfall, 100yr RP coastal surge | 9.21                                  | +7.6                             |

## 6.4 EXISTING SYSTEM HYDROLOGIC/HYDRAULIC EVALUATION

### 6.4.1 Selection of Model: XPSWMM

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

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

### 6.4.2 Development of Model Inputs

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

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

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

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

- Rainfall: time series of rainfall,
- Subcatchment Data: area, overland flow, % slope, % impervious, curve number,
- Junction Data: inverts, depth, ponded area,
- Conduit Data: shape, size, length, roughness, inverts, loss coefficients,
- Outfall-inverts, tide gate, tidal boundary condition,
- Building footprints within the 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.

#### **6.4.3 Rainfall Data**

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

#### **6.4.4 Subcatchments**

The Pretty Lake drainage area was divided into 116 smaller subcatchments based the Light Detection And Ranging (LiDAR) topographic data collected by the City of Norfolk in 2009. Figure 6-1 shows the division of the drainage area into 18 larger catchment areas. Each subcatchment was analyzed to determine input parameters for XP-SWMM. Percent imperviousness and curve number were estimated from USGS data sets representing land use and imperviousness provided by the City. Percent slope was estimated from topography. Other model inputs were simply left as the default values.

#### **6.4.5 Junctions**

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

#### **6.4.6 Conduits**

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

#### **6.4.7 Outfalls**

The 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 recurrence interval tailwater elevations in Table 5-4.

#### **6.4.8 Buildings**

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

#### **6.4.9 Topographic Data**

In 2009 Pictometry, Inc., under contract to the City of Norfolk, performed a LiDAR survey which provided topographic data at a 3-ft by 3-ft horizontal resolution. Those survey data provide the basis for the 20-ft x 20-ft grid size DEM that was used in the XP-SWMM model for Pretty Lake.

#### **6.4.10 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.

#### **6.4.11 Existing System Flooding During Various Storm Events**

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

#### **MHHW Tailwater**

The five design rainfall events were simulated in the existing condition XP-SWMM model using a boundary condition water level where 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 10-year and 100-year return period design rainfall events with a MHHW tailwater condition are presented in Figure 6-2 and Figure 6-3, respectively. Model result statistics for each simulation are presented in Table 6-3 below.

#### **Storm Surge Tailwater**

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

For perspective, the extent of flooding for the 10-year and 100-year coastal storm surges ***without any coincident rainfall*** are presented in Figure 6-6 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. The existing storm water conveyance system appears to be able, at best, to carry a ~10-year return period, 24hr duration design rainfall with the tailwater at MHHW.

**Table 6-3: Existing Condition XP-SWMM Model 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 RP rainfall, MHHW tide                | 404.7                             | 58.9                     | 113.1                 | 0.52                            | 1.1                                |
| 2yr RP rainfall, MHHW tide                | 536.0                             | 86.7                     | 160.9                 | 0.54                            | 1.3                                |
| 10yr RP rainfall, MHHW tide               | 954.0                             | 175.3                    | 287.6                 | 0.61                            | 1.7                                |
| 25yr RP rainfall, MHHW tide               | 1240.4                            | 237.5                    | 361.5                 | 0.66                            | 2.1                                |
| 50yr RP rainfall, MHHW tide               | 1493.1                            | 287.7                    | 408.1                 | 0.70                            | 2.4                                |
| 100yr RP rainfall, MHHW tide              | 1771.0                            | 343.7                    | 456.4                 | 0.75                            | 2.8                                |
| 1yr RP rainfall, 1yr RP coastal surge     | 404.7                             | 112.7                    | 154.8                 | 0.73                            | 4.3                                |
| 2yr RP rainfall, 2yr RP coastal surge     | 537.3                             | 183.7                    | 222.8                 | 0.82                            | 5.5                                |
| 10yr RP rainfall, 10yr RP coastal surge   | 954.0                             | 466.5                    | 400.7                 | 1.16                            | 9.8                                |
| 25yr RP rainfall, 25yr RP coastal surge   | 1246.9                            | 692.6                    | 497.3                 | 1.39                            | 10.9                               |
| 50yr RP rainfall, 50yr RP coastal surge   | 1500.7                            | 896.0                    | 570.9                 | 1.57                            | 11.8                               |
| 100yr RP rainfall, 100yr RP coastal surge | 1772.2                            | 1126.3                   | 645.0                 | 1.75                            | 12.8                               |

## **7.0 EXISTING CONDITION ESTIMATES OF DAMAGE COSTS**

### **7.1 METHODOLOGY**

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

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

Damage assessments were conducted for all 11 of the existing condition scenarios evaluated in XP-SWMM. This section of the report describes the procedure and inputs utilized and presents the results of the damage assessment estimates for existing conditions. Detailed outputs are included in Appendix D.

#### **7.1.1 Building Inventory Methodology**

A GIS file of the building footprints was developed for this study and was used to define the spatial locations of buildings in the Pretty Lake watershed. The project team coordinated with the City to update building footprints based on 2009 aerial photography. Approximately 11,400 buildings were identified within the Pretty Lake watershed for use in the damage assessments.

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



**Table 7-1: Typical Building Classifications**

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

### 7.1.2 Building Values

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

### 7.1.3 First Floor Elevations

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

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



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

#### **7.1.4 Depth Damage Functions - Structures and Contents**

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

#### **7.1.5 Damage Assessment Estimates**

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

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

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

<sup>a</sup> Number in parentheses represents one standard deviation based on recommended depth damage function (DDF) percentage.

## **8.0 DEVELOPMENT OF FLOOD DAMAGE MITIGATION ALTERNATIVE CONCEPTS**

### **8.1 INTRODUCTION**

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

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

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

### **8.2 FLOOD MITIGATION/DEFENSE STRATEGIES AND OPTIONS**

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

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

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

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

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

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

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

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

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

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

### 8.3 FLOOD DAMAGE MITIGATION OPTIONS ELIMINATED

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

**Table 8-1: Flood Mitigation Alternatives Feasibility Assessment**

| Flood Mitigation Alternative Options          | Options Deemed Technically/ Economically Unfeasible | Potentially Feasible Options | Feasibility Explanation               |
|-----------------------------------------------|-----------------------------------------------------|------------------------------|---------------------------------------|
| <b>Drainage &amp; Conveyance Improvements</b> |                                                     |                              |                                       |
| Channelization                                |                                                     |                              | Lack of land availability             |
| Storm Drainage Improvements                   |                                                     |                              | Based on Benefit/Cost Analysis        |
| <b>Elevation of Ground Surface</b>            |                                                     |                              |                                       |
| Building Elevation                            |                                                     |                              | Historical Buildings/Expensive        |
| Grade Raise                                   |                                                     |                              | Based on Benefit/Cost Analysis        |
| <b>Flood Barriers</b>                         |                                                     |                              |                                       |
| 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        |
| <b>Impoundment &amp; Storage</b>              |                                                     |                              |                                       |
| Permanent Retention Ponds                     |                                                     |                              | Lack of land availability             |
| Temporary Use of Land                         |                                                     |                              | Lack of land availability             |
| <b>Adaptive Land Use</b>                      |                                                     |                              |                                       |
| 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            |
| <b>Public Policy</b>                          |                                                     |                              |                                       |
| Building Codes                                |                                                     |                              | Protect newly built structures        |
| Zoning & Land Use                             |                                                     |                              | Limit structures in flood-prone areas |
| Education                                     |                                                     |                              | Enhance understanding of flood risks  |
| Warning Systems                               |                                                     |                              | Attempt to limit potential damage     |

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

- Storm Water Channelization - There are no open storm water channels in the 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 insufficient open area for consideration of either permanent or temporary retention ponds.
- Beach Nourishment - The area in question along Pretty Lake is not located along a beach.
- Setbacks and Buffers - The area is too densely developed and there is negligible open area for consideration of either setbacks or buffers.

#### 8.4 ALTERNATIVES SELECTED FOR FURTHER EVALUTION

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

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

Within these collective elements, several different types of alternatives for flood barriers and drainage improvements were considered to reduce flooding. A total of **11 alternatives** were conceptualized as presented below and were evaluated under the various design storm events. These alternatives are grouped into five categories as presented in

Table 8-2. The differentiation between alternatives subscripted Xa, Xb, and Xc has to do with the nature of the gate in the fixed tidal barrier:

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

**Table 8-2: Flood Damage Mitigation Alternatives Evaluated**

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

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

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

| Storm Event                               | Analyzed Storm Elevation<br>(ft, NAVD88) | Barrier Elevation with<br>Freeboard* (ft, NAVD88) |
|-------------------------------------------|------------------------------------------|---------------------------------------------------|
| 2yr RP rainfall, 2yr RP coastal surge     | +4.2                                     | +5.7                                              |
| 10yr RP rainfall, 10yr RP coastal surge   | +5.6                                     | +6.1                                              |
| 25yr RP rainfall, 25yr RP coastal surge   | +6.4                                     | +7.9                                              |
| 50yr RP rainfall, 50yr RP coastal surge   | +7.0                                     | +8.5                                              |
| 100yr RP rainfall, 100yr RP coastal surge | +7.6                                     | +9.1                                              |

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

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

#### 8.4.1 Alternatives 1 through 3: Tidal Barrier with Tide Gate, Pumps, and Road Raise

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

- Fixed tidal barrier structures with a movable tide gate to protect against inundation from tidal surge
- Pumps to remove rainfall runoff when the tide gate is closed, and
- Road raise at low lying areas of the basin/watershed perimeter.



### Tidal Barrier Structures with Tide Gate

The tidal barrier and tide gate would be constructed on the upstream side of the Shore Drive Bridge. The overall length of the barrier would be approximately 400 LF and would tie into the existing elevations of the surrounding environment. Given the soil conditions within this area of Pretty Lake, the proposed barrier wall would 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 would be placed. A decorative fascia wall would be installed on the upstream side of the barrier structure for aesthetics.

The gate assembly would be located in-line with the existing navigational channel and fender system of the bridge. The opening width of the gate would vary with gate type, from approximately 50 LF for the sliding steel gate and Obermeyer gate to 110 LF for the inflatable dam, due to abutment angle requirements. At the gate location, a navigation clearance to at least elevation -6 ft NAVD 88 would be necessary to allow small boat traffic to access Pretty Lake through the navigation span of the existing bridge. The three conceptual options for the tide gate are described below and in Figure 8-1:

**Steel Gate.** The steel gate utilizes steel framing and roll on a guide which would 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 would be stored in a pocket located on one of the opening. Because the steel gates are required to be stored in a pocket this option requires the bulkhead to be an additional 2.3 feet higher than Table 8-3 indicates.

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

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

### Pumps

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

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

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

#### Road Raise

Road raising and utility relocation was evaluated along 2,800 linear feet of road on Shore Drive, Pretty Lake Road, and Dunning Road. This work would complete the barrier between Pretty Lake and the Chesapeake Bay. In addition to the road raise, several homes between Dunning Road and Pretty Lake are proposed to be raised which would allow their existing floor elevation to be above the flood plain.

#### **8.4.2 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-year return period surge event. Breakdown in costs per storm event are provided in the opinion of probable cost (Section 10.0 of this report).

#### **8.4.3 Alternative 5: Property Buyout**

Alternative 5 includes purchasing the property with structures that are identified as high damage risks. Since FEMA does not have an established buy-out criteria for this mitigation option, review of the depth damage function was completed to determine the most feasible correlation. Based on this function, it was determined that a depth damage function of 20% would provide the City an optimal characterization of the required property buyout within 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, and
- Loss of City Property Tax.

## 9.0 EVALUATION OF CONCEPTUAL ALTERNATIVES

### 9.1 HYDROLOGIC / HYDRAULIC MODELING EVALUATIONS

Five alternatives were considered in order to reduce flooding of the 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 2-D model grid and the shoreline acted as the 2-D grid boundary. The outfalls which drain into Pretty Lake were given tide-gates preventing backflow, and each was assigned a fixed 1-D water-surface boundary condition associated with the model-scenario.

In the analysis, the 1, 2, 10, 25, 50, and 100-yr 24-hr design storms were run in XPSWMM for each alternative for both the MHHW and coincident surge events. The corresponding design event storm surge was used as the tailwater elevation at the pump-outlet or at the outfalls. For the purpose of this report, only results for the 10 year and 100 year design storms will be presented in Figures 9-2 through 9-9. Results from the other design storms are presented in Appendix B. It is important to note that the XPSWMM models show that the upland piping system is adequate 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.

#### 9.1.1 Induced Flooding With Mitigation Alternatives

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

### 9.1.2 Residual Flooding With Mitigation Alternatives

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

**Table 9-1: Summary of XP-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                                |
| <b>Change vs. Existing Conditions</b> |                                   |                          |                       |                                 |                                    |
| 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 XP-SWMM results. The bulkhead wall alternative prevented storm surges from flooding inland, but also resulted in storm water accumulating behind the wall. Figures 9-8 and 9-9 present the

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

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

| 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                                |
| <b>Change vs. Existing Conditions</b>       |                                   |                          |                       |                              |                                    |
| 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. The table shows 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 it 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 XP-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%          |

## 9.2 FLOOD DAMAGE REDUCTION ESTIMATES

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

**Table 9-4: Estimated Flood Damage Reductions**

| Alternative                                               | Estimated Structure Damages (\$ Millions) |                           |                                |              |
|-----------------------------------------------------------|-------------------------------------------|---------------------------|--------------------------------|--------------|
|                                                           | 10yr, 10yr <sup>a</sup>                   | 100yr, 100yr <sup>a</sup> | 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          |
| <b>Estimated Contents Damages, millions</b>               |                                           |                           |                                |              |
| 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          |
| <b>Estimated Structure and Contents Damages, millions</b> |                                           |                           |                                |              |
| 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          |

<sup>a</sup> Number in parentheses represents one standard deviation based on recommended depth damage function (DDF) percentage



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

### 10.1.1 Capital Costs

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

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

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

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

| Alternative | Opinion of Probable Costs (\$ Millions) |                                        |
|-------------|-----------------------------------------|----------------------------------------|
|             | 10-year RP Rainfall and Coastal Surge   | 100-year RP Rainfall and Coastal Surge |
| 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 conceptual opinion of probable cost breakdowns, 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.



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

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)

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.

### 10.1.3 Summary

The 11 alternatives varied in cost from \$38.4M (Steel Gate and 2-60" Pumps) to \$473.7M (Property Buyout) for the 100-year RP 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.

**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.

**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.

## **11.0 SELECTION OF PREFERRED CONCEPTUAL ALTERNATIVE**

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

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

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

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

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

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

#### **11.1.2 Design life of the Mitigation Option**

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

#### **11.1.3 Present Value of Project**

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

#### **11.1.4 B/C Ratio**

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

alternatives. The B/C ratio of the alternatives analyzed indicates that Alternative 1a - Tidal Barrier with Steel Gate, 2 - 60" Pumps, and Road Raise - is the most cost effective alternative with a Benefit Ratio of 2.14 for a 100-year storm event. Figures 11-1 illustrates the relationship of the various alternatives for 10-year versus 100-year design events.

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

| Alternative | Estimated Benefit to Cost Ratio |              |
|-------------|---------------------------------|--------------|
|             | 10yr, 10yr                      | 100yr, 100yr |
| 1a          | 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.

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

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

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

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

One project alternative for the Pretty Lake area not considered for the conceptual design phase of the project was a constructed closure at the mouth of Little Creek Inlet where it enters the Chesapeake Bay. Such a project would preclude need for the "preferred alternative" at Shore Drive bridge. This alternative project would require a great deal of regional support and collaboration with U.S. Navy, USACE, and the City of Virginia Beach.

The Little Creek Inlet Federal Navigation Channel is maintained to a depth of about 22 feet (Re. MLLW). The shore to shore width range is approximately 800 to 1,500 feet. Figure 12-1 outlines the area such a project could possibly be constructed.

### 12.1 FUNCTIONAL DESIGN REQUIREMENTS AND DESIGN CRITERIA

#### 12.1.1 Functional design requirements

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

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

| Location                                                                              | Barrier Elevation<br>(ft, NAVD88) | Allowance for<br>Wave Setup,<br>Sea Level Rise | Remaining<br>Freeboard |
|---------------------------------------------------------------------------------------|-----------------------------------|------------------------------------------------|------------------------|
| Primary tidal barrier and tide gate across Pretty Lake entrance at Shore Drive Bridge | +10.6                             | 0.5 ft + 1 ft                                  | 1.5 ft                 |
| East overland flood barrier                                                           | +9.1                              | 0 ft + 0.5 ft                                  | 1 ft                   |
| South-east overland flood barrier (along boundary with Little Creek Amphibious Base)  | +9.1                              | 0 ft + 0.5 ft                                  | 1 ft                   |

| Location                                                         | Barrier Elevation<br>(ft, NAVD88) | Allowance for<br>Wave Setup,<br>Sea Level Rise | Remaining<br>Freeboard |
|------------------------------------------------------------------|-----------------------------------|------------------------------------------------|------------------------|
| Little Creek Drive overland flood barrier                        | +9.1                              | 0 ft + 0.5 ft                                  | 1 ft                   |
| Berm on City-owned property on north side of<br>Pretty Lake Ave. | +9.1                              | 0 ft + 0.5 ft                                  | 1 ft                   |
| Start-up of pumps at primary tidal barrier (min.)                | +1.0                              | -                                              | -                      |
| Shut-off of pumps at primary tidal barrier (max.)                | --1.0                             | -                                              | -                      |
| 100-year return period tailwater level (Table 5-4)               | +7.6                              | -                                              | -                      |

At the +9.1 ft NAVD88 design water level, the overland flood walls and berm allow for 0.5 ft of future sea level rise while maintaining 1 ft of freeboard in the design flood event. NFIP regulations require at least 2 ft of freeboard above FEMA's effective Base Flood Elevation (BFE), which is currently published as +7.6 ft NAVD88 for this site. Designing to a higher water elevation would increase the linear extent of overland flood wall required in the Pretty Lake project area, and a significantly higher design elevation for the overland barriers would likely require gates across several businesses' vehicle entrances where relatively simple existing grade raising is presently envisioned.

The elevation of the primary tidal barrier and gate across the entrance to Pretty Lake is set at a higher elevation than the overland barriers. This is to build in more reserve allowance for future sea level rise and to maintain greater freeboard in future conditions, since this primary barrier would be significantly more expensive to modify (raise) in the future, compared to the overland barriers.

Pretty Lake is utilized by small water craft, and the City has specified that the entrance to the cove – through the Shore Drive bridge and the tidal barrier gate – should provide a minimum draft of 4 feet relative to MLLW datum (equivalent to elevation -6 ft NAVD88 datum). This requirement has been incorporated into the preliminary design by setting the gate sill at an elevation of -7 ft NAVD88.

The XPSWMM modeling of the Pretty Lake watershed shows that the pumping system at the Shore Drive Bridge tidal barrier needs to be able to convey approximately 620 cubic feet per second (cfs) – or 278,275 gallons per minute (gpm) – during the peak of the 100-year return period 24-hr rainfall event, against a 100-year return period coastal tailwater elevation of +7.6 ft NAVD88.

### 12.1.2 Applicable codes and standards

This section intentionally left blank.

### 12.1.3 Design loads and load combinations

This section intentionally left blank.

## 12.2 OVERVIEW OF THE PREFERRED PRELIMINARY DESIGN ALTERNATIVE

A total of 11 alternatives were evaluated in the conceptual stage of project development, covering a range of coastal surge and rainfall event magnitudes. From that evaluation, the design alternative with the highest Benefit-Cost (B/C) ratio was selected, and further



engineering effort has been put into developing that Alternative 1a to a 10% preliminary level of design.

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

Additional fixed barriers, in the form of short to moderately high steel and concrete walls, are required to enclose low-lying adjacent areas and prevent flood penetration by flanking of the primary tidal barrier. The present development of Alternative 1a consists of approximately 1,170 ft of overland flood wall running east from Shore Drive to existing high ground. A second, longer overland flood wall (approximately 1,850 ft) would run along the existing boundary line between the City of Norfolk and the Little Creek property of the military's Joint Expeditionary Base (JEB) Little Creek - Fort Story, terminating at the east side of Shore Drive at Little Creek Road.

The intersection of Shore Drive and Little Creek Road would need to be raised approximately 1 ft to 1.5 ft above existing grade, and vehicle entrances at a few business along Little Creek Road (and at one on the Little Creek waterfront immediately east of the bridge) would need to be raised approximately 0.5 ft to 1 ft. It is currently proposed to continue with street grade raising along Little Creek Road to a point approximately 290 ft west of the intersection, where the existing street grade appears to tie into the +9.1 ft NAVD88 design elevation. Raising the intersection and this segment of Little Creek Road would form the southern flood barrier of this project.

Finally, a berm is proposed along the southern end of a presently undeveloped lot north of Pretty Lake Ave. This berm would close a gap between areas of existing higher ground in this area to prevent flood waters from flanking the primary tidal barrier to the north.

Most of the rainfall-runoff captured within the Pretty Lake watershed drains to Little Creek through the Shore Drive bridge, thence to Chesapeake Bay. With the primary Pretty Lake tidal barrier gate closed, stormwater discharges to Little Creek would be impounded landward of the Shore Drive bridge. The completed Alternative 1a project therefore requires the inclusion of a pump station at the primary barrier, with associated equipment including back-up power generators.

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

### **12.2.1 Overview of Appendix E Drawings**

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



Sheet G-101 gives an overview of the locations and alignments of the various project elements. The main tidal barrier across Little Creek runs on the Pretty Lake (landward) side of the Shore Drive bridge to existing high ground at the south end of the bridge. Starting at Shore Drive, the east overland barrier runs along the Pretty Lake waterfront, connecting to existing but limited area of higher ground at Cobb's Marina. From the south-east side of the Cobb's Marina property, the south-east overland barrier runs along the line of the Little Creek JEB boundary fence to the intersection of Shore Drive, Little Creek Road, and Midway Road. The project includes raising this intersection and from 100 ft to 300 ft of the intersecting streets. The raised portion of the Little Creek Road west of the intersection completes the connection to existing higher ground at the design level of +9.1 ft NAVD88. It is currently proposed to locate a building on City-owned property at the south end of the Shore Drive bridge tidal barrier, to house back-up power generators and electrical equipment for the pump and gate motors. More detailed descriptions of each of the project elements are given in the sections below.

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

Sheets GR-101 illustrates the locations of various historical geotechnical borings noted in Section 4.8 above.

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

### **12.3 DESIGN OF PRIMARY SMITH CREEK TIDAL BARRIER AND GATE**

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

#### **12.3.1 Fixed Wall**

The tidal barrier and tide gate would be constructed on the upstream side of the Shore Drive bridge on Pretty Lake. It must be constructed far enough landward of the bridge to allow sufficient room for construction access and operations, primarily considering requirements for batter pile driving angles. The overall length of the conceptual barrier would be approximately 470 linear feet (LF).

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

During preliminary (10% level) design development, structural engineering calculations were conducted. These calculations, based in part upon the geotechnical information discussed in Section 4.8, indicated a change in the wall section type (Sheet S-201) would be necessary to

prevent unacceptable deflection under the 100-year return period design water level condition described above. It is particularly important to limit deflection in sections where the sliding steel gate will interact with the fixed wall.

In the center reaches of the barrier, where water depths are greater and geotechnical conditions generally poorer, the preliminary design section consists of a combination of 60-inch diameter by 0.75-inch thick steel pipe piles (PP 60"  $\phi$ x3/4"), spaced 9.8 ft on center, connected by AZ 26-700 steel sheet piles over the 4.8 ft gap between pipe piles. The pipe piles provide significant additional structural resistance to overturning and deflection than would be feasible with sheet piles alone. Required pipe pile lengths range from approximately 70 to 90 ft; at this 10% level of design, the sheet piles are considered to extend to the same depth as the pipe piles.

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

Pipe pile and sheet pile sizes and embedment depths depend heavily upon the available geotechnical data and derived engineering properties of the existing soil layers. No additional field geotechnical data has been collected and analyzed to date along the proposed alignments of any of the project elements. As such, this preliminary design of required pile penetration is subject to substantial revision at later stages of design development, pending required additional field data collection. These revisions may include value-engineering to optimize the lengths of steel sheet piles between the steel pipe piles. As noted above, sheet pile lengths are presently considered to be the same as adjacent pipe pile lengths.

The crest of the wall along most of its length is set at +10.6 ft NAVD88 as shown in Table 12-1 above. The crest is elevated by an additional 2.25 ft in the gate abutment sections, to accommodate the sliding gate.

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

### **12.3.2 Steel Gate**

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

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

The gate assembly would be located in-line with the existing navigational channel and fender system of the bridge. The opening width of the gate is 65 ft, coinciding with the width and position of the Shore Drive bridge navigation span. At the gate location, the gate sill

elevation has been set to -7 ft (NAVD 88) to allow small boat traffic to pass between Pretty Lake and Little Creek.

## **12.4 DESIGN OF PUMPING FACILITIES**

### **12.4.1 General description**

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

Each of the three pumps would be mounted behind and immediately against the combination wall. The pumps must be mounted at an elevation that allows at least 4 ft of water depth over the intake pipe elevation at pump startup. The intake bell entrance must be at least 3 ft above the bed or sump elevation. For both criteria, greater depths are allowed and preferred. Setting the pumps at a relatively lower elevation than absolutely necessary will allow the pumps to be started at lower water levels, for example during maintenance testing or earlier during the rising leg of a storm event. Sheet S-201 shows the pump intake elevation as -4.0 ft NAVD88, which would allow a minimum water surface elevation of +0 ft NAVD88 for pump startup.

At present, it is considered that the most appropriate location for the pumps is to the north of the gate opening, where slightly greater existing water depth is available under the pump intake bells. This would also position the pump discharges near existing rip rap scour protection under the bridge and on the bayward side of the bridge. The pumps are shown supported on a typical strainer bar stand (e.g. by MWI) and a large roller. The strainer stand provides vertical support and prevents large trash or debris objects from impinging on the pump impellers. The roller also provides vertical support and will allow for some movement of the assembly perpendicular to the wall, if the wall deflects slightly during a flood event. The stand and roller are supported by a concrete slab on a concrete pile foundation; the slab will help to avoid bed scour during operation of these high-capacity pumps. Due to shallow existing bed elevations at the locations of one or more of the pumps, the preliminary design includes dredging of up to 2 ft under the proposed footprint of the pump support structure, and the support slab includes sidewalls to help mitigate sediment deposition near the pump intake bells.

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

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

#### **12.4.2 Electrical and mechanical requirements**

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

Sheet E-001 shows the proposed location of the generator and equipment building in relation to Shore Drive and the proposed tidal barrier across Pretty Lake. Preliminary building dimensions and major equipment arrangement are shown on Sheet E-101 and Sheet E-201.

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

### **12.5 DESIGN OF ADDITIONAL PERIMETER BARRIERS**

#### **12.5.1 General description**

In order to make the main tidal barrier and gate across Pretty Lake effective, by preventing inundation through various low lying areas around the overland perimeter of the adjacent area, additional fixed closure walls and/or berms would be required to the north, east, and south of the Shore Drive bridge as shown on Sheet S-101. Along the north and south boundaries of the protected area, the barrier takes the form of an earthen berm and street grade raising, respectively.

The concepts for the additional perimeter barriers are very different than those evaluated at the conceptual design stage described in Sections 8.0 through 11.0. During preliminary design development, it was found that the specific street grade raising envisioned at the conceptual design evaluation stage was not as technically feasible as originally believed. Consequently, the preliminary design of Alternative 1a now involves fixed overland flood walls and an earthen berm, in addition to street grade raising at a different location than proposed in the conceptual Alternative 1a.

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

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

#### **12.5.2 East: along Little Creek waterfront to Cobb's Marina**

The overland barrier east of the Shore Drive bridge (Sheets S-105 and S-106) is primarily a steel and concrete wall (with some segments replaced by existing grade raising), extending from existing higher ground approximately 100 ft east of Shore Drive, along the Little Creek waterfront to existing higher ground at Cobb's Marina. The total length of the barrier is approximately 1,170 ft, as shown in the elevation profiles on Sheet S-107.

At the +9.1 ft NAVD design elevation, the barrier could avoid most utility conflicts – based on known available utility information as described in Section 12.8 below. The steel sheet pile-supported concrete wall segment of the barrier would cross a single 30-inch concrete storm drain pipe approximately 100 ft east of Captain's Galley.

Where it crosses an existing business entrance (Captain's Galley / Marine Concepts / Surf Rider Taylor's Landing) at its starting point, the barrier would take the form of vehicle entrance and sidewalk strip regrading to achieve the +9.1 ft NAVD88 level.

The portion of the wall located approximately between stations 5+00 and 9+00 runs between an existing boat yard and dry stack building and the Little Creek waterfront. To mitigate impacts to this business, the existing grade behind the wall would need to be raised so that boat launching operations could continue. Alternatively, the fixed wall may be replaced, in this limited section, with demountable flood walls; this type of wall consists of rigid panels and vertical supports that are manually installed into pre-cut holes or slots in the ground before a flood event. The specifics of this design element would need to be developed in consultation with the business owner.

#### **12.5.3 South-east: along Little Creek JEB boundary**

The south-east overland flood barrier consists of a steel sheet pile-supported concrete wall, beginning at the south-east side of the Cobb's Marina property and running along the Little Creek JEB boundary. The wall is expected to run along the existing security fence line behind the existing private property parcels. Pending consultation with JEB management, the wall may be allowed to replace the existing fence. In that case, its crest elevation may need to be a few feet higher than proposed, or additional chain link and barbed / razor wire may need to be added on top of the concrete flood wall cap. This possibility and the required design details would need to be done in consultation with JEB management. The total length of the barrier is approximately 1,170 ft, as shown in the elevation profiles on Sheet S-108.

No utility conflicts are known to exist for the south-east overland flood wall, based on the known available utility information. However, no utility information is held at present for the Little Creek JEB, and the possibility of utility conflicts will need to be investigated during subsequent design stages.



#### 12.5.4 South: Shore Drive / Little Creek Road intersection west along Little Creek Road

The intersection of Shore Drive, Little Creek Road, and Midway Road currently lies at elevations between 6.5 ft to 7.5 ft NAVD88. The existing grade of Little Creek Road ties into the design elevation of +9.1 ft NAVD88 within 290 ft west of the intersection. Raising the intersection and segments of adjacent streets to maintain no greater than 2% grade will involve (but will not be limited to) the following primary activities over approximately 700 linear feet of multi-lane divided roadway and approximately 160 ft of single lane road:

- Sidewalk, curb and gutter demolition and reconstruction
- Additional layers of asphalt to raise the road surfaces
- Regrading of raised median islands
- Regrading eight existing driveway entrances
- Construction of a low height concrete retaining wall (Little Creek Road east of the intersection)
- Drainage structure and manhole elevation increases, plus potential additional storm water drainage pipes and catchments
- Possibly raising the existing overhead, cantilevered traffic signals if vertical clearances are not sufficient

#### 12.5.5 Berm: north of Pretty Lake Ave.

The conceptual level alternatives evaluation assumed that a segment of Pretty Lake Ave. would be raised to enclose the northern boundary of the project area. During the preliminary (10% level) design review, it was observed that the property along the north side of Pretty Lake Ave. is (1) close in existing grade to the design elevation of +9.1 ft NAVD88, and (2) presently vacant. It was decided that placing a berm along the boundary of this vacant lot, tying into existing grades of approximately +9.2 ft NAVD88 on 24th Bay St, would provide a less expensive and impactive alternative to raising the grade of Pretty Lake Ave. A plan outline and typical section of the berm are shown in Sheet S-109.

Alternatively, the existing grade of the presently vacant lot could be **raised to a level 1 to 2 ft above the design level of +9.1 ft NAVD88**. Existing grade of the property across 24th Bay St. is approximately 11 ft NAVD88. Taking this step at present, while the lot is undeveloped, could allow for a higher design level to be implemented in the future (if necessary) by simply raising the elevation of 24th Bay St. at its crown approximately 125 ft north of Pretty Lake Ave. It is noted that this presently vacant lot is owned primarily by the NH&RA, but that Clark Investments LLC may own a portion of the south end of the lot (approximately 1700 square feet according to the City of Norfolk parcels GIS layer).

#### 12.6 CORROSION MITIGATION

This section intentionally left blank.

## 12.7 REAL ESTATE REQUIREMENTS

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

The proposed berm north of Pretty Lake Ave. would be located on property listed as owned by the Norfolk Housing and Redevelopment Authority (NH&RA), and it is assumed that the City would not require an easement to construct that berm. The proposed east, south-east, and south overland flood barriers cover a total of approximately 3,570 linear feet. It is assumed at present that the City would need to obtain easements of width approximately 15 ft in order to permanently install the walls on private property. The opinion of probable cost presented in Section 13.2 of this report assumes a value of \$8.00 per square foot for easement acquisition, based on an analysis of average property values along the proposed wall alignments.

## 12.8 FACILITY / UTILITY RELOCATION REQUIREMENTS

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

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

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

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

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

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

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



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

| Project Element                                                 | Location of Utility Conflict                    | Description of Utility Conflict                 |
|-----------------------------------------------------------------|-------------------------------------------------|-------------------------------------------------|
| <b>Tidal barrier across Smith Creek, and associated systems</b> | none                                            | none                                            |
|                                                                 |                                                 |                                                 |
| <b>Berm along north side of Pretty Lake Ave.</b>                | none                                            | none                                            |
|                                                                 |                                                 |                                                 |
| <b>Shore Drive / Little Creek Drive intersection raising</b>    | various                                         | new storm water drainage features required      |
|                                                                 |                                                 |                                                 |
| <b>Southeast overland barrier</b>                               | none, but need to consult with Little Creek JEB | none, but need to consult with Little Creek JEB |
|                                                                 |                                                 |                                                 |
| <b>East overland barrier</b>                                    | none                                            | none                                            |

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

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

#### **13.1 PURPOSE OF THIS SECTION**

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

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

#### **13.2 CAPITAL COSTS**

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

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

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

The opinion of probable capital cost for the preferred alternative, as presently formulated, is approximately \$46.4. A detailed breakdown of line items, quantities, and unit costs is provided in Table 13-1.

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

|                                                                                   |          |                           |                      |                            |  |
|-----------------------------------------------------------------------------------|----------|---------------------------|----------------------|----------------------------|--|
|  |          | Opinion of Probable Cost  |                      | DATE PREPARED<br>23-Mar-12 |  |
|                                                                                   |          |                           |                      | DRAFT Submittal            |  |
| ACTIVITY AND LOCATION                                                             |          | CONSTRUCTION CONTRACT NO. |                      | IDENTIFICATION NUMBER      |  |
| City of Norfolk                                                                   |          |                           |                      |                            |  |
| Norfolk, Virginia                                                                 |          | ESTIMATED BY              |                      | CATEGORY CODE NUMBER       |  |
| PROJECT TITLE                                                                     |          | Moffatt & Nichol          |                      |                            |  |
| Pretty Lake - Tidal Barrier with Steel Gate, 3-60" Dia. Pumps, and Closure Walls  |          | STATUS OF DESIGN          |                      | JOB ORDER NUMBER           |  |
| For 100 Year Event @ ELEV +7.6' (Design Water Surface ELEV +9.1')                 |          | Preliminary (10%)         |                      | 7607-02                    |  |
| ITEM DESCRIPTION                                                                  | QUANTITY |                           | ENGINEERING ESTIMATE |                            |  |
|                                                                                   | NUMBER   | UNIT                      | UNIT COST            | TOTAL                      |  |
| LITTLE CREEK (Shore Drive Bridge) Flood Wall with Steel Gate                      |          |                           |                      |                            |  |
| Flood Wall                                                                        |          |                           |                      |                            |  |
| AZ26 (50 VLF)                                                                     | 54       | EA                        | \$4,300.00           | \$232,200                  |  |
| AZ26 (70 VLF)                                                                     | 26       | EA                        | \$6,020.00           | \$156,520                  |  |
| AZ26 (90 VLF)                                                                     | 42       | EA                        | \$7,740.00           | \$325,080                  |  |
| PP60x0.75 (70 VLF, Coated w/ Connectors)                                          | 16       | EA                        | \$42,213.00          | \$675,408                  |  |
| PP60x0.75 (90 VLF, Coated w/ Connectors)                                          | 21       | EA                        | \$50,885.00          | \$1,068,585                |  |
| Connectors (assume 20% of total steel)                                            | 1        | LS                        | \$491,558.60         | \$491,559                  |  |
| Concrete cap / fascia                                                             | 1,575    | CY                        | \$1,000.00           | \$1,574,580                |  |
| Handrail                                                                          | 934      | LF                        | \$178.00             | \$166,252                  |  |
| Steel Gate and Equipment                                                          |          |                           |                      |                            |  |
| 24" SQ Conc Batter Piles (24 piles)                                               | 24       | EA                        | \$9,961.09           | \$239,066                  |  |
| W33x152                                                                           | 38       | TON                       | \$10,125.00          | \$382,442                  |  |
| 3/4" Thick Outside Plate                                                          | 19       | TON                       | \$10,125.00          | \$193,737                  |  |
| 1/2" Thick Plate @3'-0"                                                           | 11       | TON                       | \$10,125.00          | \$116,434                  |  |
| 3/4" Thick Bottom Plate                                                           | 4        | TON                       | \$10,125.00          | \$36,175                   |  |
| Roller Guide                                                                      | 136      | LF                        | \$298.00             | \$40,528                   |  |
| Flanged Steel Wheel Assembly                                                      | 8        | EA                        | \$3,120.00           | \$24,960                   |  |
| Gate Rail                                                                         | 136      | LF                        | \$298.00             | \$40,528                   |  |
| Capstan & Cabling                                                                 | 1        | LS                        | \$193,500.00         | \$193,500                  |  |
| UHMW Rollers                                                                      | 16       | EA                        | \$2,587.00           | \$41,392                   |  |
| Pocket Seal                                                                       | 35       | LF                        | \$4,650.00           | \$162,750                  |  |
| LITTLE CREEK (Shore Drive Bridge) Flood Wall Pump Station                         |          |                           |                      |                            |  |
| Pumps and Mounting                                                                |          |                           |                      |                            |  |
| 60" pumps                                                                         | 3        | EA                        | \$1,380,000.00       | \$4,140,000                |  |
| Support Structure - piles,header,rods, etc.                                       | 3        | EA                        | \$70,000.00          | \$210,000                  |  |
| Misc 60" Pipe Sections                                                            | 3        | EA                        | \$8,000.00           | \$24,000                   |  |
| 60" Flapgates                                                                     | 3        | EA                        | \$19,200.00          | \$57,600                   |  |
| Generator Building                                                                |          |                           |                      |                            |  |
| Concrete Block Building for Generator Equipment                                   | 3,878    | SF                        | \$200.00             | \$775,500                  |  |
| Aesthetic Features of Pump Station                                                | 1        | LS                        | \$100,000.00         | \$100,000                  |  |
| Clearing, Grubbing & Grading                                                      | 667      | SY                        | \$10.00              | \$6,667                    |  |

|                                                                        |       |     |                |             |
|------------------------------------------------------------------------|-------|-----|----------------|-------------|
| <b>Electrical</b>                                                      |       |     |                |             |
| Dominion Power Installation Costs                                      | 1     | LS  | \$200,000.00   | \$200,000   |
| Installation Equipment                                                 | 1     | LS  | \$20,228.20    | \$20,228    |
| Site Work                                                              | 1     | LS  | \$9,943.24     | \$9,943     |
| Switchboard                                                            | 1     | LS  | \$74,231.00    | \$74,231    |
| Conduits & Fittings                                                    | 1     | LS  | \$191,405.00   | \$191,405   |
| VFD Drive                                                              | 3     | EA  | \$150,000.00   | \$450,000   |
| 2500 KW Standby Generator                                              | 2     | EA  | \$1,245,875.00 | \$2,491,750 |
| Paralleling Switchgear                                                 | 1     | LS  | \$429,800.00   | \$429,800   |
| # 500 kcmil XHHW                                                       | 7,500 | LF  | \$14.10        | \$105,750   |
| Other Electrical Equipment                                             | 1     | LS  | \$181,582.84   | \$181,583   |
| Insurance & Taxes for Electrical                                       | 1     | LS  | \$159,404.00   | \$159,404   |
| Sales Tax for Electrical                                               | 1     | LS  | \$173,859.00   | \$173,859   |
| <b>EAST and SOUTHEAST Overland Flood Wall</b>                          |       |     |                |             |
| <b>Wall Section Type A</b>                                             |       |     |                |             |
| Demo Concrete / Asphalt                                                | 266   | SY  | \$15.00        | \$3,986     |
| AZ 12-700 (12 VLF)                                                     | 350   | EA  | \$1,032.00     | \$361,200   |
| Concrete Wall                                                          | 286   | CY  | \$1,000.00     | \$286,062   |
| <b>Wall Section Type B</b>                                             |       |     |                |             |
| Demo Concrete / Asphalt                                                | 578   | SY  | \$15.00        | \$8,667     |
| AZ 12-700 (18 VLF)                                                     | 761   | EA  | \$1,548.00     | \$1,178,028 |
| Concrete Wall                                                          | 895   | CY  | \$1,000.00     | \$894,917   |
| Concrete Footing                                                       | 293   | CY  | \$1,000.00     | \$293,333   |
| <b>Wall Section Type D</b>                                             |       |     |                |             |
| Demo Concrete / Asphalt                                                | 36    | SY  | \$15.00        | \$542       |
| AZ 12-700 (27 VLF)                                                     | 48    | EA  | \$2,322.00     | \$111,456   |
| Concrete Wall                                                          | 14    | CY  | \$1,000.00     | \$13,597    |
| Concrete Footing                                                       | 18    | CY  | \$1,000.00     | \$18,333    |
| <b>Entrances and Working Waterfront</b>                                |       |     |                |             |
| Regrade 2 Driveway Entrances:<br>asphalt (30 by 10 by 2)               | 135   | TON | \$110.00       | \$14,850    |
| Regrade access from marina dry stack to<br>bulkhead: concrete pavement | 1,181 | CY  | \$700.00       | \$826,389   |
| <b>EAST and SOUTHEAST Overland Flood Wall Utility Conflicts</b>        |       |     |                |             |
| 8" Ductile Iron Water Main Relocation                                  | 1     | EA  | \$22,800.00    | \$22,800    |
| 8" Clay or Ductile Iron Sewer Line Relocation                          | 1     | EA  | \$19,200.00    | \$19,200    |
| 30" Concrete Stormwater Line Relocation                                | 1     | EA  | \$28,800.00    | \$28,800    |
| Gas Line Relocation                                                    | 1     | EA  | \$18,000.00    | \$18,000    |
| Underground Electric Ductbank Relocation                               | 3     | EA  | \$18,000.00    | \$54,000    |
| Allowance for unknown utility conflicts                                | 1     | LS  | \$71,400.00    | \$71,400    |



| <b>Raise Intersection at SHORE DRIVE and LITTLE CREEK ROAD</b>    |               |     |                |                     |
|-------------------------------------------------------------------|---------------|-----|----------------|---------------------|
| Raise Shore Drive north of intersection                           | 1             | LS  | \$175,950.00   | \$175,950           |
| Raise Shore Drive south of intersection                           | 1             | LS  | \$125,350.00   | \$125,350           |
| Raise Little Creek Road west of intersection                      | 1             | LS  | \$612,950.00   | \$612,950           |
| Raise Little Creek Road east of intersection                      | 1             | LS  | \$202,400.00   | \$202,400           |
| Raise Midway Road east of intersection                            | 1             | LS  | \$365,700.00   | \$365,700           |
| Raise Intersection: asphalt                                       | 2,478         | TON | \$110.00       | \$272,576           |
| Elevate Traffic Signals /                                         |               |     |                |                     |
| Electric Utility Coordination                                     | 1             | LS  | \$100,000.00   | \$100,000           |
| Drainage Modifications, various locations                         | 1             | LS  | \$93,000.00    | \$93,000            |
| <b>BERM north of PRETTY LAKE AVE</b>                              |               |     |                |                     |
| Clearing and Grubbing                                             | 0.3           | AC  | \$1,600.00     | \$480               |
| Select Fill                                                       | 354           | CY  | \$24.00        | \$8,488             |
| Fine Grading                                                      | 661           | SY  | \$10.00        | \$6,610             |
| Topsoil                                                           | 110           | CY  | \$28.00        | \$3,085             |
| Seed                                                              | 661           | SY  | \$1.25         | \$826               |
| <b>Easements and Mitigation</b>                                   |               |     |                |                     |
| City of Norfolk Easement                                          | 50,845        | SF  | \$8.00         | \$406,760           |
| Temporary Easement (8% of CoN Easement)                           | 1             | LS  | \$32,540.80    | \$32,541            |
| Wetland Mitigation                                                | 1.1           | AC  | \$500,000.00   | \$535,000           |
| <b>SUBTOTAL - CONSTRUCTION ELEMENTS</b>                           |               |     |                | <b>\$23,130,690</b> |
| <b>Contractor Markups and Contingency</b>                         |               |     |                |                     |
| Contractor - Overhead & Profit                                    |               | %   | 15%            | \$3,469,603.43      |
| Contractor - Mobilization/Demobilization                          |               | %   | 12%            | \$2,775,682.75      |
| Contractor - Difficult Waterside Conditions                       | est. lump sum | LS  | \$1,000,000.00 | \$1,000,000         |
| Contractor - Erosion/Sediment Control                             |               | %   | 5%             | \$1,156,534         |
| Contractor - Traffic Control                                      |               | %   | 2%             | \$462,614           |
| Construction Contingency                                          |               | %   | 30%            | \$6,939,207         |
| <b>SUBTOTAL - CONSTRUCTION, WITH MARKUPS</b>                      |               |     |                | <b>\$38,934,331</b> |
| <b>Planning, Design, Permitting, and Construction Observation</b> |               |     |                |                     |
| USACE Recon study                                                 |               | LS  | \$200,000.00   | \$200,000           |
| USACE Feasibility study                                           |               | LS  | \$2,000,000.00 | \$2,000,000         |
| USACE EIS and NEPA coordination                                   |               | LS  | \$500,000.00   | \$500,000           |
| Engineering, P&S, Const. Obs.                                     |               | %   | 12%            | \$4,672,119.70      |
| <b>SUBTOTAL - PLANNING AND ENGINEERING</b>                        |               |     |                | <b>\$7,372,120</b>  |
| <b>TOTAL</b>                                                      |               |     |                | <b>\$46,306,451</b> |
|                                                                   |               |     | <b>SAY</b>     | <b>\$46,400,000</b> |

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

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

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

Assumptions for the operational and maintenance costs included:

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

Operational and maintenance costs for the preliminary design alternative are provided in Table 13-2. The estimated O&M cost for Alternative 1a is unchanged from the conceptual level analysis, and it consists of an expected annual cost of \$253,000, or a total of approximately \$3.5 million in present value over a 50 year design life. Based on FEMA and OMB direction, a 7% interest rate was utilized for the present value analysis.

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

| Alternatives                                                                      | Annual Operational Costs (\$) | 50-yr Operational Costs (\$) Present Worth |
|-----------------------------------------------------------------------------------|-------------------------------|--------------------------------------------|
| Alt 1a: Tidal Barrier with Steel Gate, 2 - 60" Dia. Pumps, Closure Walls and Berm | \$253K                        | \$3.5M                                     |

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

### 13.4 BENEFIT-COST RATIO ANALYSIS

The benefit cost ratios were updated based on the opinion of probably cost developed for the preliminary (10%) design of the preferred design alternative. Table 11-1 summarizes the B/C ratios for the various alternatives. The B/C ratio of the preferred 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 for a 100-year storm event.



## **14.0 ENVIRONMENTAL OBJECTIVES AND REQUIREMENTS (ITEM C-13)**

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

### **14.1 OVERVIEW OF NEPA AND REGULATORY REQUIREMENTS**

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

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

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

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

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

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

## **14.2 COORDINATION WITH FEDERAL, STATE, AND LOCAL STAKEHOLDERS**

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

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

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

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



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

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

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

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

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

### **14.3 NOTABLE AREA FEATURES AND LIKELIHOOD OF POTENTIAL IMPACTS**

#### **14.3.1 Water Quality**

Pretty Lake and the adjacent connected waters of Little Creek are the primary open water bodies to be considered, and they are populated by numerous aquatic species. Pretty Lake exchanges water directly with the tidal Little Creek and Chesapeake Bay through the Shore Drive bridge opening.

Pretty Lake is populated by numerous aquatic species. Specific species inhabiting Pretty Lake are not presently documented, but it is assumed that species may include benthic organisms, insects, fish of all life stages, aquatic birds, and water-dependent mammals. Documentation of species actually inhabiting Smith Creek will need to be developed, and potential specific impacts will need to be considered, during the environmental assessment phase of project development.

The primary concern with regard to water quality is that the proposed tidal barrier may act to limit tidal exchanges between Pretty Lake and bayward waters of Little Creek. Limited flushing may result in undesirable dissolved oxygen, nutrient, and /or temperature levels in Pretty Lake basin.

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

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

The hydrodynamic model was developed in the DELFT3D modeling software, and a typical tidal water level time series (based on Sewells Point tide records) was input as the boundary condition outside the cover. Model simulations used a conservative (non-decaying) “tracer” constituent deployed uniformly throughout Pretty Lake – behind the tidal barrier – to evaluate flushing times for both existing conditions and proposed Alternative 1a conditions. These screening-level simulations do not indicate that the proposed tidal barrier would increase flushing times in Pretty Lake (at the barrier or at any point further within the lake).

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

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

#### **14.3.2 Shoreline and Shallow Bottom Habitat**

Vertical walls are reflective of incident wave energy, and they may also alter the flow patterns and velocities of tidal currents. The proposed tidal barrier would block any sediment movement between Pretty Lake and the area bayward of the wall. The extent of sediment transport through the existing bridge opening is presently unknown.

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

On the discharge (Little Creek) side of the wall, the pump discharge may generate vertical and horizontal eddies in the immediate vicinity of the wall. These may impact on the adjacent shoreline and/or on the bed. However, except during maintenance inspection when the pumps may be run at a lower flow rate, the pumps will run at the design flow rates only during significant coastal flooding events. The natural events themselves would also have flow- and wave-induced impacts on the shorelines and bed, and it is not expected that the pump discharges will have significant additional impacts in addition to the natural impacts of the storm event.

### **14.3.3 Wetlands**

Shallow water areas exist on the Pretty Lake side of the primary tidal barrier. The nature of these areas, and whether they would be considered jurisdictional wetland areas, is not documented.

### **14.3.4 Marina and Recreational Boating**

Pretty Lake is also used by small recreational water craft, but it is not expected that the proposed project will significantly impact recreational boating activity.

Several marinas and significant length of hardened shoreline (vertical bulkheads) currently exist immediately bayward of the primary tidal barrier. The proposed overland flood walls would be located in already highly developed areas. The proposed berm north of Pretty Lake Ave. would be located on property listed as owned by the NH&RA. The lot is presently vacant, but it is understood that it may be slated for development of residential dwellings.

The east overland barrier is presently proposed to run along the Little Creek waterfront, and it could impact the operations of an existing marina business by impeding access between the waterfront and the marina's yard and dry stack facility. The barrier would need to be designed to limit these impacts and/or realigned to go around (landward of) parts of the facility. However, this level of design / realignment should be done during and after consultation with the property owner(s).

### **14.3.5 Little Creek Joint Expeditionary Base**

The boundary between the City of Norfolk and the US Navy's Joint Expeditionary Base Little Creek-Fort Story (formerly Little Creek Amphibious Base) is presently marked and secured by a chain link fence. An appropriately designed wall – solid instead of chain link – may actually improve the visual aesthetics for the residents along the south-east wall alignment.

From a base security standpoint, the presence of the solid wall would block sightlines of security patrols along this boundary, except from elevated watch positions. It is unknown whether this would be a significant security concern for the Navy, and this and other logistics issues should be discussed with base management.

### **14.3.6 Local Community**

The floodwalls would extend primarily through commercial districts, and the overland wall along the Little Creek base boundary would run behind a residential district. Though it is not likely to present problems, the socio-economic impacts of the overland wall segments will have to be considered during NEPA compliance. The socio-economic make-up of the community will also need to be documented, in order to show that the project will not have socio-economically disproportionate adverse impacts.

Along some segments, the presence of the wall may create areas that are not visible from a distance, or it may block light from existing sources, thus creating pedestrian safety issues. For example, dark areas or areas that cannot be easily observed may encourage criminal activity or hinder police response to criminal activity.

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

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

### **15.1 MAIN FLOODWALL AND GATE**

### **15.2 PUMPING FACILITIES**

### **15.3 ADDITIONAL PERIMETER WALLS**

### **15.4 ACCESS AND STAGING AREAS**

Draft



## 16.0 IMPLICATION OF POTENTIAL FUTURE SEA LEVEL RISE

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

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

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

- 1<sup>st</sup> plot the cost and damage curves versus return period for the design to be evaluated. For example, Figure \_\_a shows such a plot for the Alternate \_\_.
- 2<sup>nd</sup> Convert the costs versus return period to costs versus tailwater elevation, using the tailwater versus return period plot shown on Figure \_\_b to create the costs and damage curve shown on Figure \_\_c.
- 3<sup>rd</sup> Convert today's tailwater versus return period for a 1-foot rise in sea level as shown on Figure \_\_d,
- 4<sup>th</sup> add the "after 1-foot" of sea level rise tailwater versus design period to plot compare the relationship between those two variable for the current conditions, as shown on Figure \_\_e, and
- 5<sup>th</sup> 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 0.5 ft to 1 ft of freeboard. Nonetheless, raising the structures should be further investigated during the next design phase and a final design elevation selected. In many ways, it would be prudent to include an allowance for sea level rise since adding elevation will be more difficult after the fact, than the added (delta) cost associated with raising the top of the structure by another foot. The estimated delta cost to raise the crest of the floodwall by an additional 1- is ~5-15% of the initial cost. Where this relationship would breakdown is when the flood levels approach elevations where significant portions of the watershed rim would have to be raised - the costs would then likely underweigh the benefits.



## 17.0 CONCLUSIONS AND RECOMMENDATIONS

The 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.

The primary conclusions and recommendations from the current study include:

- The study results show that coastal defense improvements can be used to mitigate the effects of extreme high tides in the Pretty Lake watershed.
- The preferred alternative is the construction of a floodwall, tide gate, a pump station (with two 60" pumps operating simultaneously and one 60" pump as a backup ) and closure barriers with a total capital cost of approximately \$46.4 million. The preferred alternative has a B/C ratio of approximately 1.15, and is therefore economically justified.
- This option will provide coastal flood mitigation today for a 100-yr 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 but no more effective, or were considered to be less reliable. The property buyout alternative can be expected to be unreasonably expensive.
- 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 existing upland storm water piping 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 of two MWI SEA360 1(60" discharge diameter) pumps is adequate to address the flow rates which can be delivered by the existing storm water piping system. Additional pumping capacity 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.
- 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.

- At +10.6 ft NAVD 88, the crest elevation of the primary tidal barrier across Pretty Lake at the Shore Drive bridge allows for greater future sea level rise and greater freeboard than the currently proposed +9.1 ft NAVD88 design elevation of the overland flood walls, berm, and street grade raising. Designing for an elevation significantly higher than +9.1 ft NAVD88 would change the length and height of overland wall required, and it may necessitate the inclusion of several gates similar to the vehicle gates in the existing Norfolk Downtown Floodwall. A final decision concerning what height should control the design of the primary tidal barrier and overland barriers should be made during the next design phase.
- At the time of the report, the USACE has approved a study to evaluate whether there is Federal interest in the Pretty Lake project. The study is planned to occur during the summer of 2012 and expected to take six months to complete. If the USACE deems there is Federal interest in the project, then the project may be eligible to pursue Federal funding through a partnership with the Federal government.
- The proposed project will be required to go through the NEPA process. This study has initiated some of the steps necessary to evaluate potential environmental impacts. This study conducted preliminary hydrodynamic analysis to evaluate the impact to tidal flushing of a wall and gate structure near Shore Drive bridge. These screening-level simulations do not indicate that the proposed tidal barrier would increase flushing times in Pretty Lake (at the barrier or at any point further within the lake). The proposed structure will impact subaqueous bottomlands and potentially limited wetland areas along the shoreline area.

## 18.0 LIMITATIONS

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

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

Draft

## 19.0 REFERENCES

- American Petroleum Institute (2000), "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms - Working Stress Design," API Recommended Practice 2A-WSD (RP 2A-WSD), 21st Ed., December 2000, API, Washington, D.C.
- Duncan, J. M. (2000), "Factors of Safety and Reliability in Geotechnical Engineering," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 4, pp. 307-316.
- GET Solutions, Inc. (2006), "Norfolk Light Rail Transit Project," Volume 1 - Norfolk Southern Railroad Company Track Alignment.
- Fugro Atlantic (2009), "Final Marine Geotechnical Site Characterization Report," Craney Island Eastward Expansion, Portsmouth, Virginia, dated October 14, 2009.
- Fugro Atlantic (2010), "Preliminary Coastal Flooding Evaluation and Implications for Flood Defense Design," City-Wide Coastal Flooding Study, Norfolk, Virginia, dated July 2010.
- Liao, S. C., and Whitman, R. V. (1986), "Overburden Correction Factors for SPT," Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 3, pp. 373-377.
- Meyerhof, G. G. (1963), "Some Recent Research on the Bearing Capacity of Foundations," Canadian Geotechnical Journal, Vol. 1, No. 4, pp. 57-64.
- National Oceanic and Atmospheric Administration (NOAA) (2010a) "Sea Level Trends." <http://tidesandcurrents.noaa.gov/sltrends/sltrends.shtml>
- National Oceanic and Atmospheric Administration (NOAA) (2010b), NOAA Tides & Currents, "Mean Sea Level Trend, 8638610 Sewells Point, Virginia." [http://tidesandcurrents.noaa.gov/sltrends/sltrends\\_station.shtml?stnid=8638610](http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8638610)
- National Oceanic and Atmospheric Administration (NOAA) (2004). "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 3, G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley. NOAA, National Weather Service, Silver Spring, Maryland
- National Resource Conservation Service (NRCS) (1986), U.S. Department of Agriculture (USDA) Technical Release 55. "Urban Hydrology for Small Watersheds." Washington, D.C.
- Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974), "Foundation Engineering," 2nd Edition, John Wiley, New York.
- U.S. Army Corps of Engineers (USACE) (2010), "Bulkhead Replacement NOAA Marine Operations Center Norfolk, Virginia," Report of Subsurface Explorations, Preliminary Geotechnical Engineering Evaluation.
- U.S. Army Corps of Engineers (USACE) (2003), Economic Guidance Memorandum (EGM) 04-01, "Generic Depth-Damage Relationships for Residential Structures with Basements," Washington, D.C.

U.S. Army Corps of Engineers (USACE) (2000), Economic Guidance Memorandum (EGM) 01-03, "Generic Depth-Damage Relationships," Washington, D.C.

U.S. Army Corps of Engineers (USACE) (1992), IWR Report 92-R-3, "Catalog of Residential Depth-Damage Functions," Washington, D.C.

Draft

## FIGURES

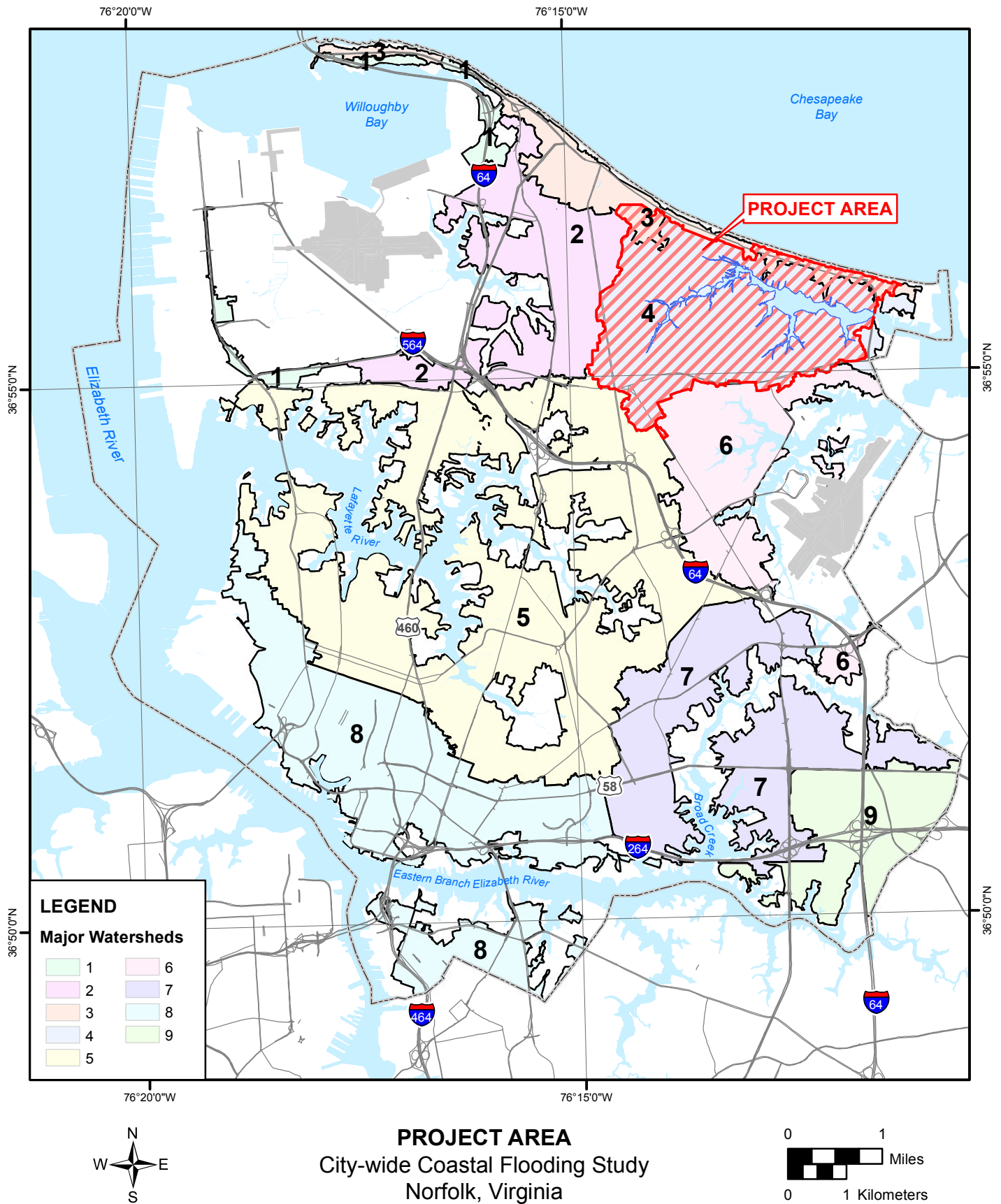


FIGURE 2-1







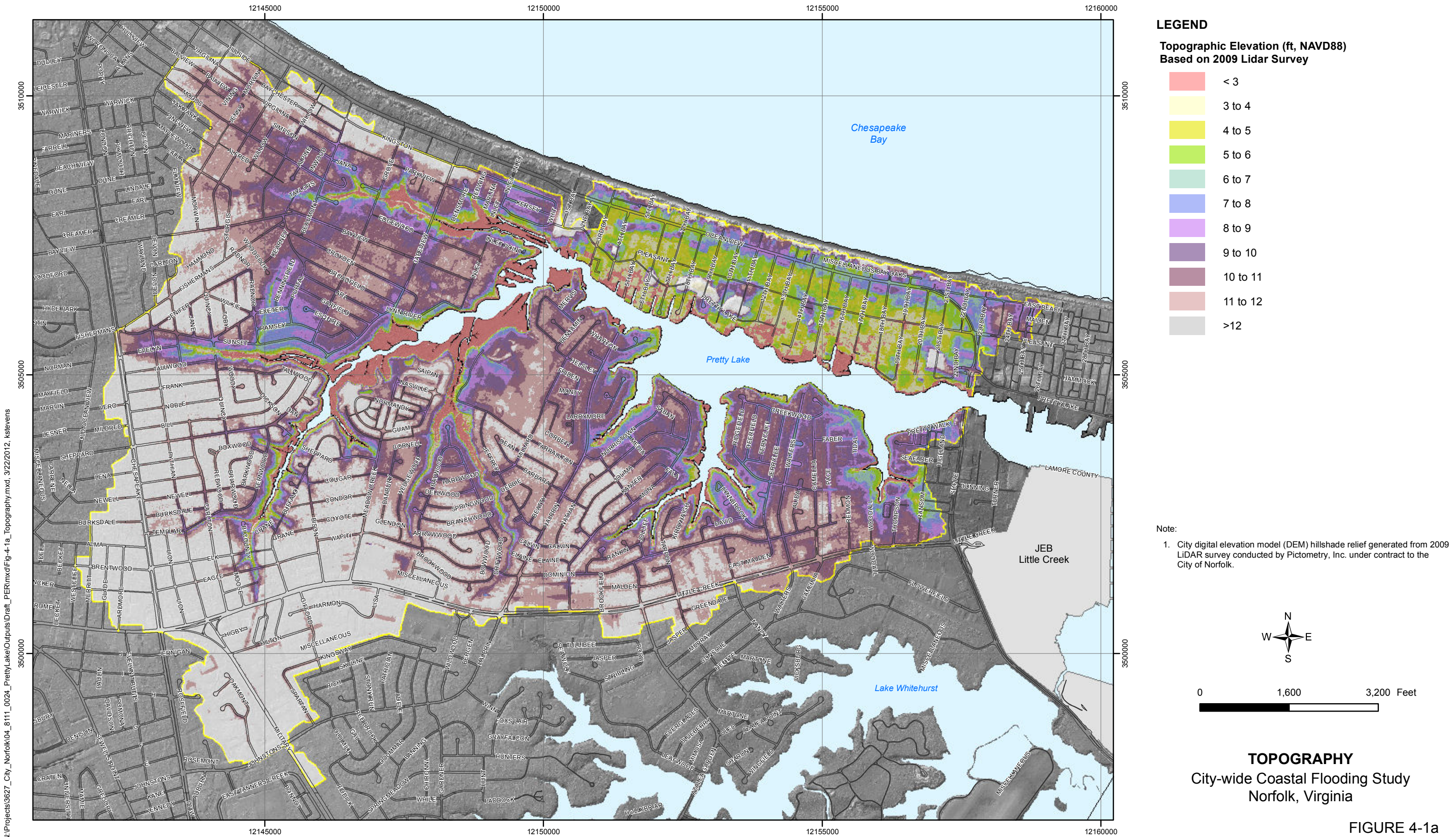


0 100 200  
Feet

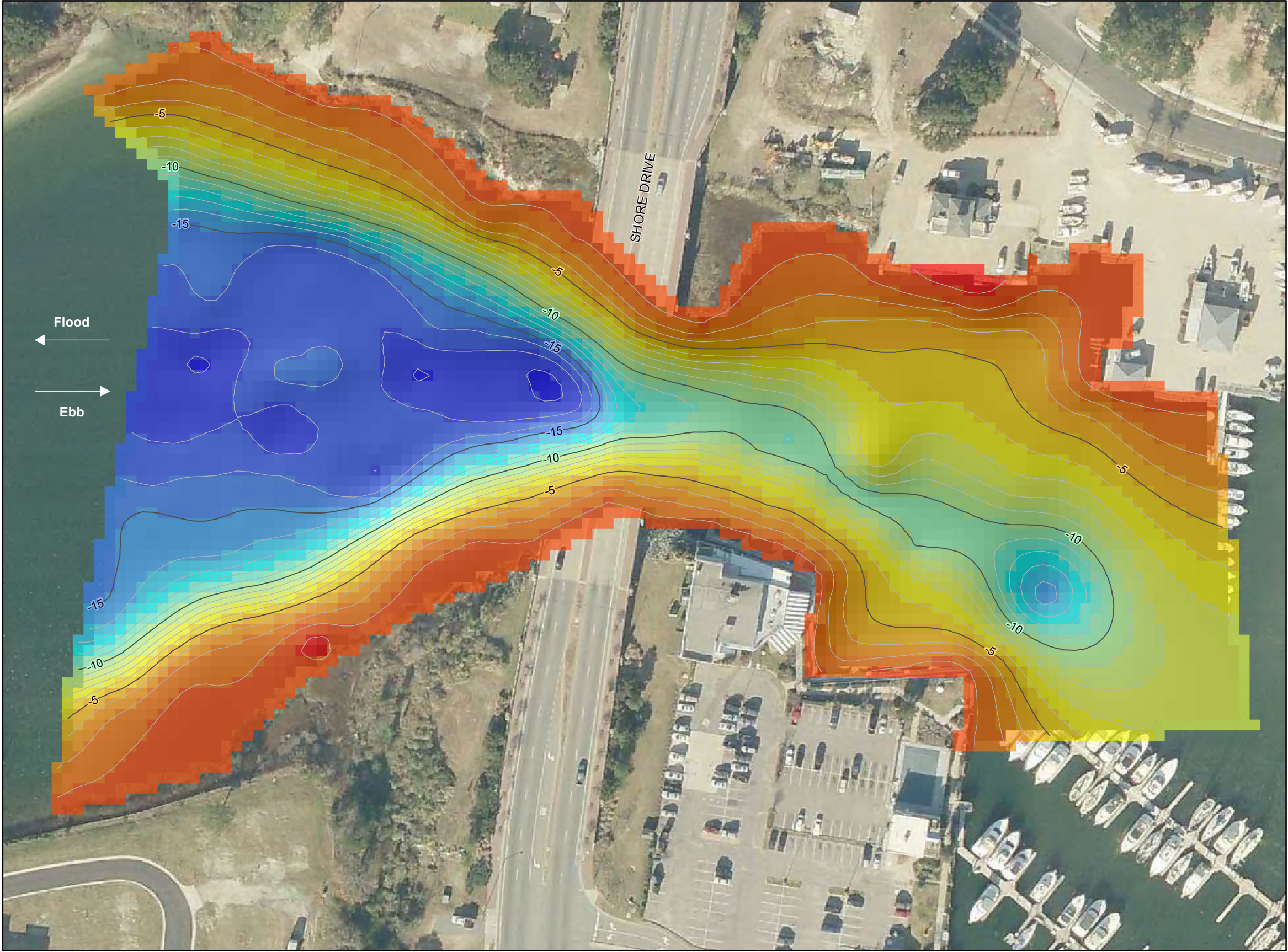
**BASIN OUTLET**  
City-wide Coastal Flooding Study  
Norfolk, Virginia

**FIGURE 2-3**









LEGEND

**Bathymetric Elevation (re: NAVD88)**

— Contour interval is 5 feet

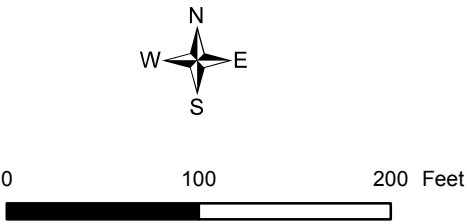
— Contour interval is 1 foot

**Bathymetric Elevation (re: NAVD88)**

0 feet

-18.5 feet

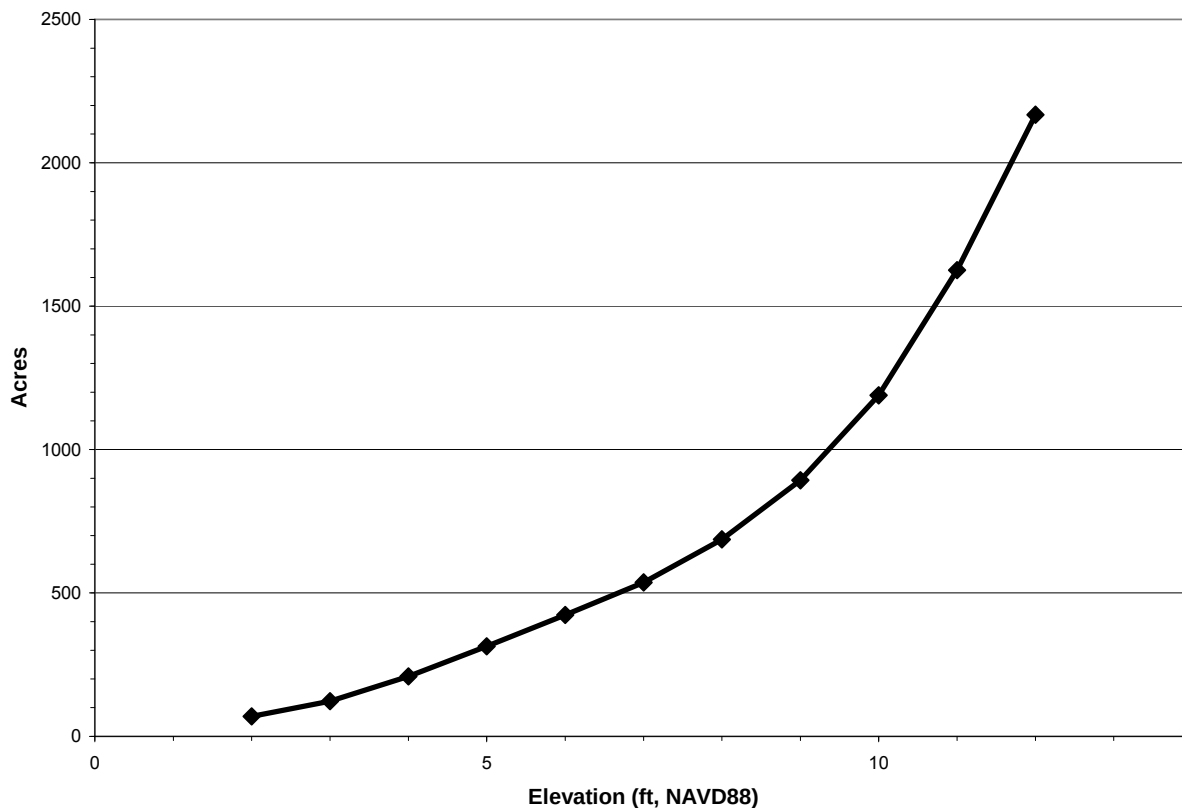
- Notes:
1. Bathymetry data represent a compilation of bathymetric data sources. West of the bridge, data are from a single beam survey conducted in 1999 by Waterway Surveys and Engineering. The survey boundary is the western side of the bridge. Bathymetry data east of the bridge are based on soundings from NOAA Chart 12255 Little Creek Inlet (2008).
  2. Norfolk 2010 aerial photograph provided by the City of Norfolk GIS Department.



**BATHYMETRY**  
City-wide Coastal Flooding Study  
Norfolk, Virginia

FIGURE 4-1b





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.

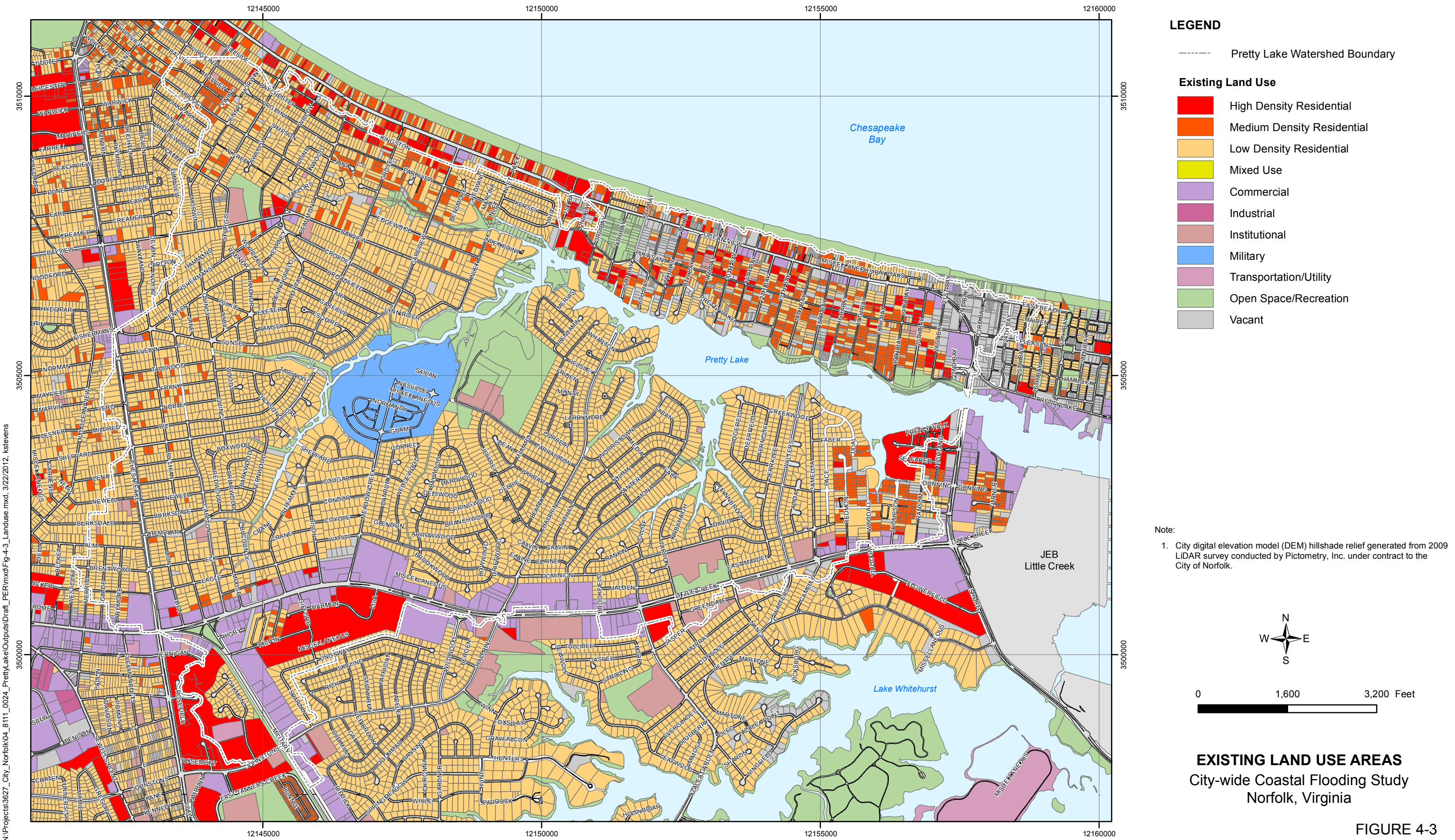
Examples of how this graph may be interpreted:

- 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



DRAFT





- Less than 1.6
- Less than 3.6
- Less than 4.2
- Less than 5.0
- Less than 5.6
- Less than 6.4
- Less than 7.0
- Less than 7.6
- Greater than 7.6

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

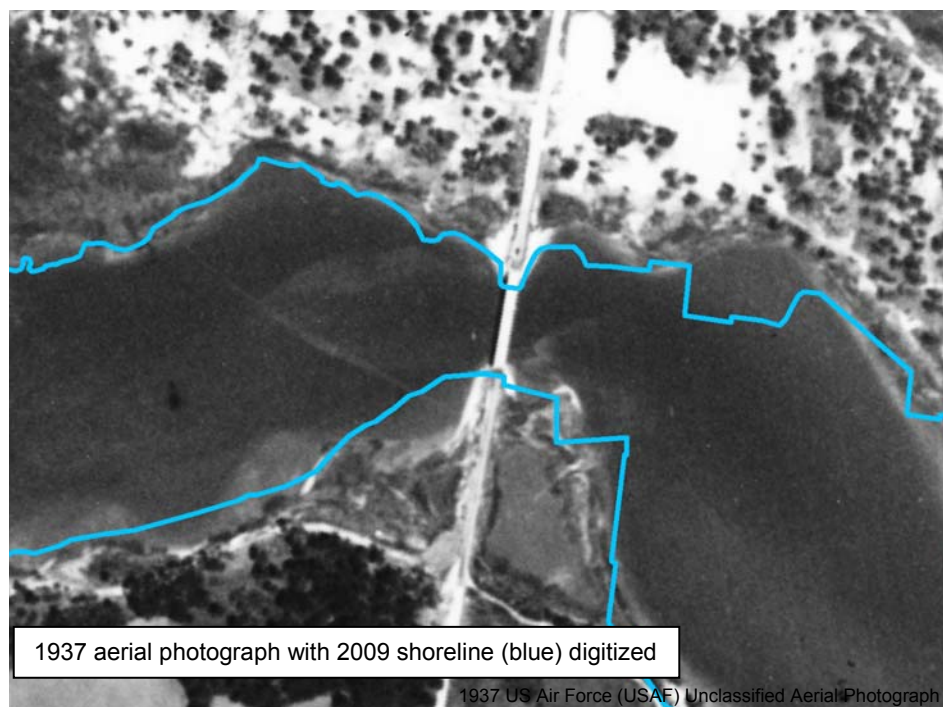
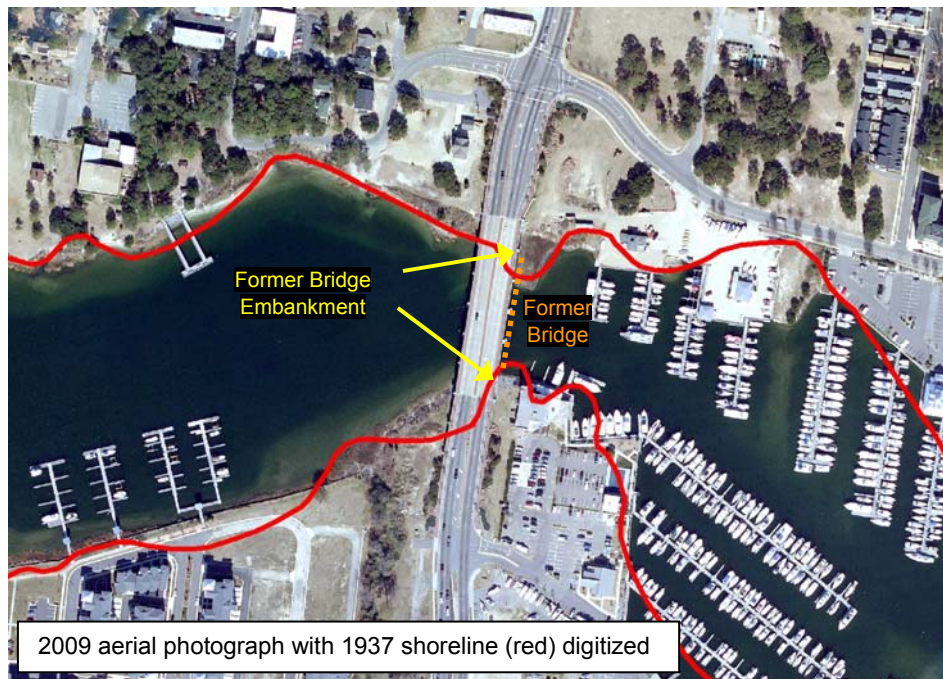


0                      1,600                      3,200 Feet

City-wide Coastal Flooding Study  
Norfolk, Virginia

FIGURE 4-4





## FORMER SHORELINE STRUCTURES

### Basin Outlet

City-wide Coastal Flooding Study  
Norfolk, Virginia



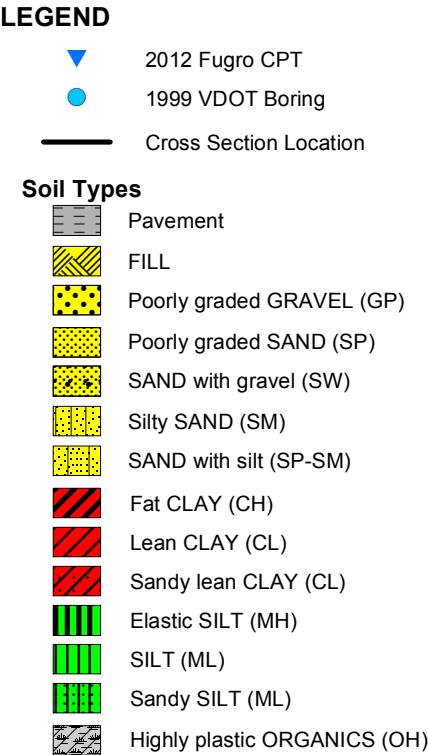
Photograph is of a cone penetration test (CPT) being conducted during the City-wide flooding study. Three CPTs were conducted at the Pretty Lake project area to collect geotechnical data. Photograph was taken of work performed at the Hague project area.

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

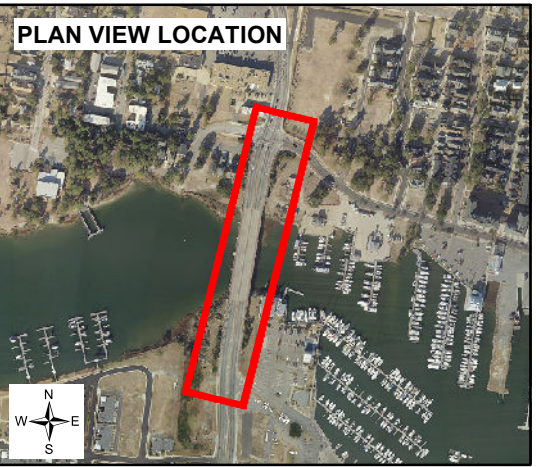
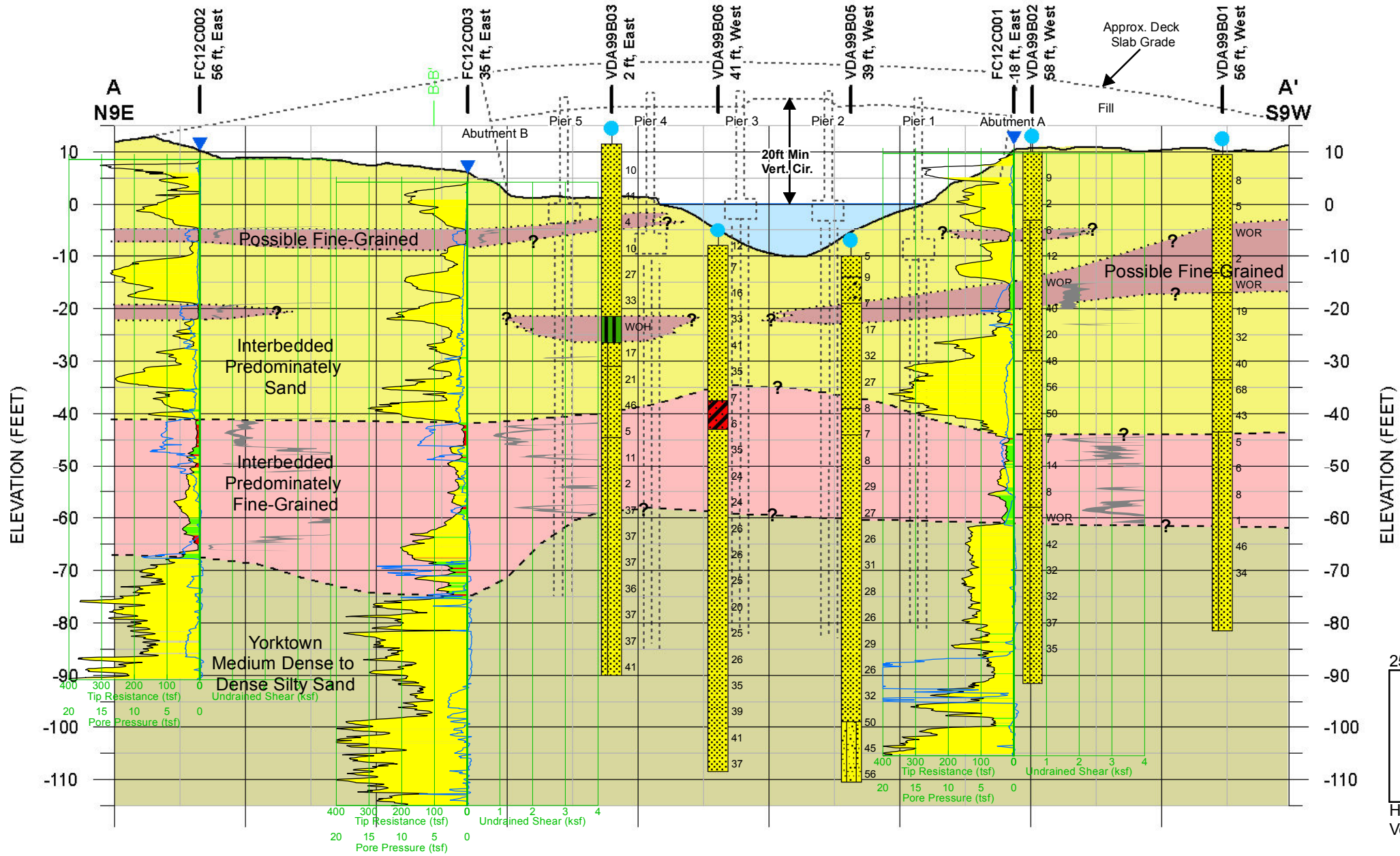
FIGURE 4-7



N:\Projects\3627\_City\_Norfolk\04\_8111\_0024\_PrettyLake\Outputs\Draft\_PER\mxd\Fig-4-8\_PrettyLake\_Prel\_Sub\_Xsect.mxd, 3/22/2012, ksteven



- Notes:
- Elevation datum is NAVD88.
  - Topography from 2009 Pictometry, Inc. LiDAR survey.
  - Bathymetry from 1999 Waterway Surveys and Engineering single beam survey.
  - Stratigraphic contacts are approximate and are interpreted from CPT sounding and boring data. Conditions vary both along and perpendicular to the section line.
  - Refer to Figure 4-7 for key to symbolism used on cross section.



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

FIGURE 4-8





### LEGEND

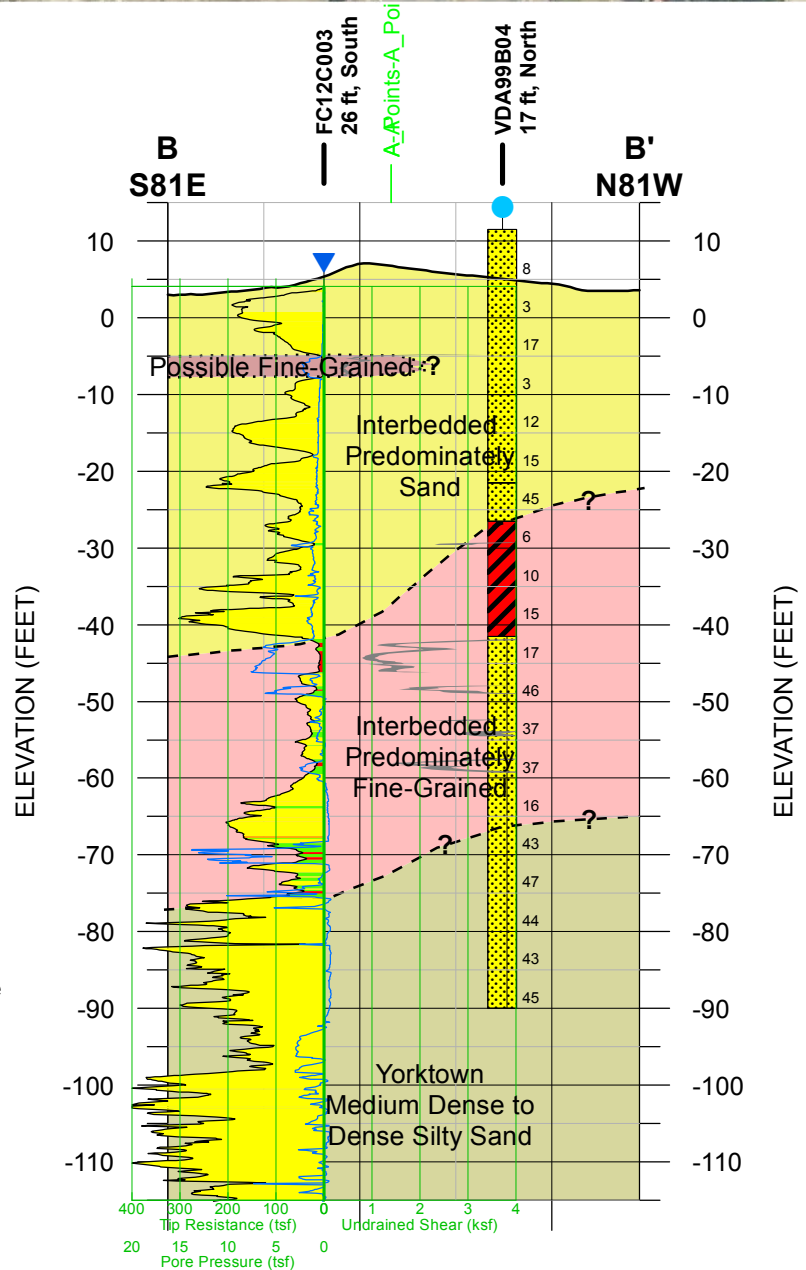
- ▼ 2012 Fugro CPT
- 1999 VDOT Boring
- Cross Section Location

### Soil Types

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

### Notes:

1. Elevation datum is NAVD88.
2. Topography from 2009 Pictometry, Inc. LiDAR survey.
3. Stratigraphic contacts are approximate and are interpreted from CPT sounding and boring data. Conditions vary both along and perpendicular to the section line.
4. Refer to Figure 4-7 for key to symbolism used on cross section.

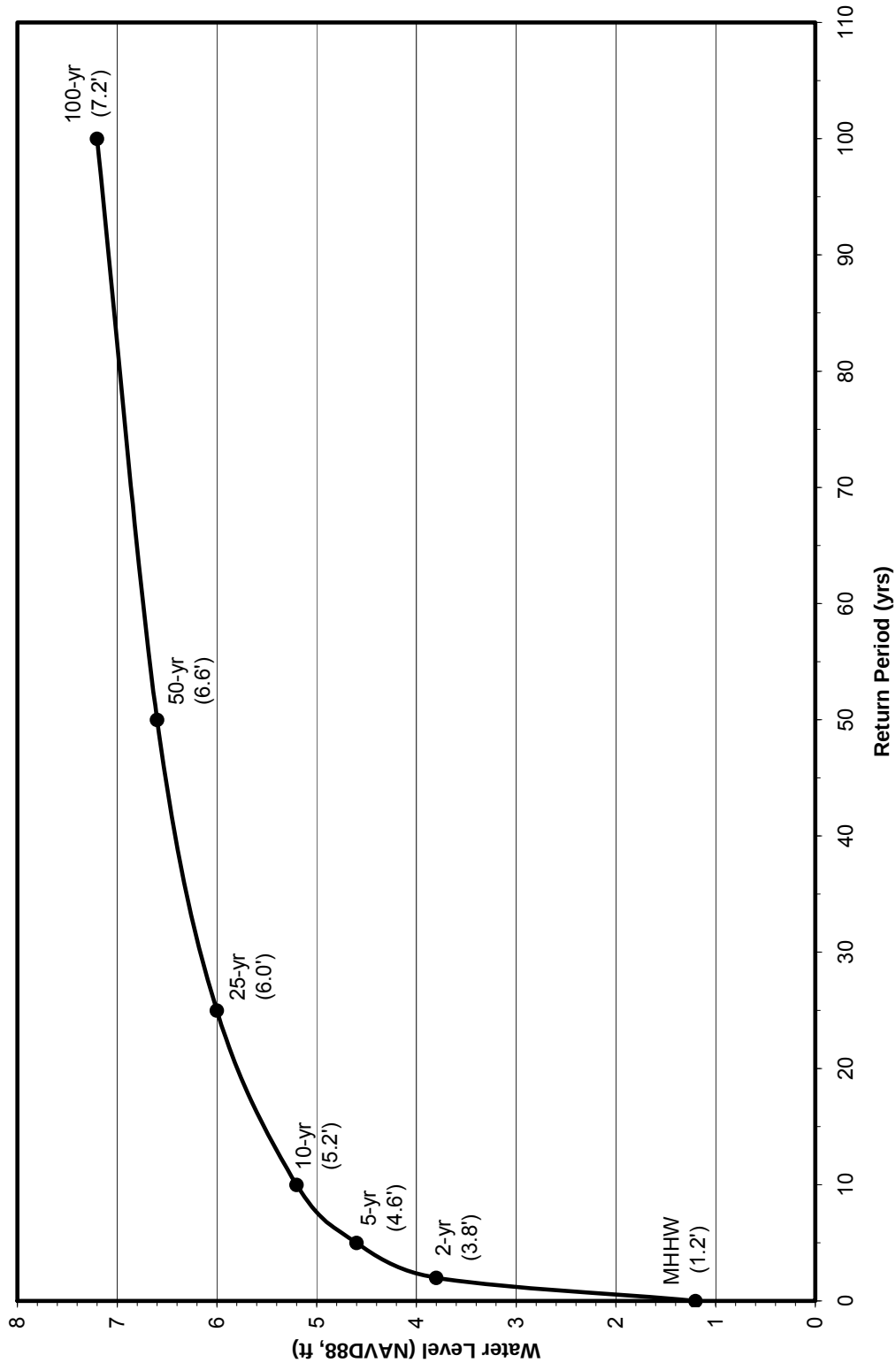


## PRELIMINARY SUBSURFACE CROSS SECTION B-B'

City-wide Coastal Flooding Study  
Norfolk, Virginia

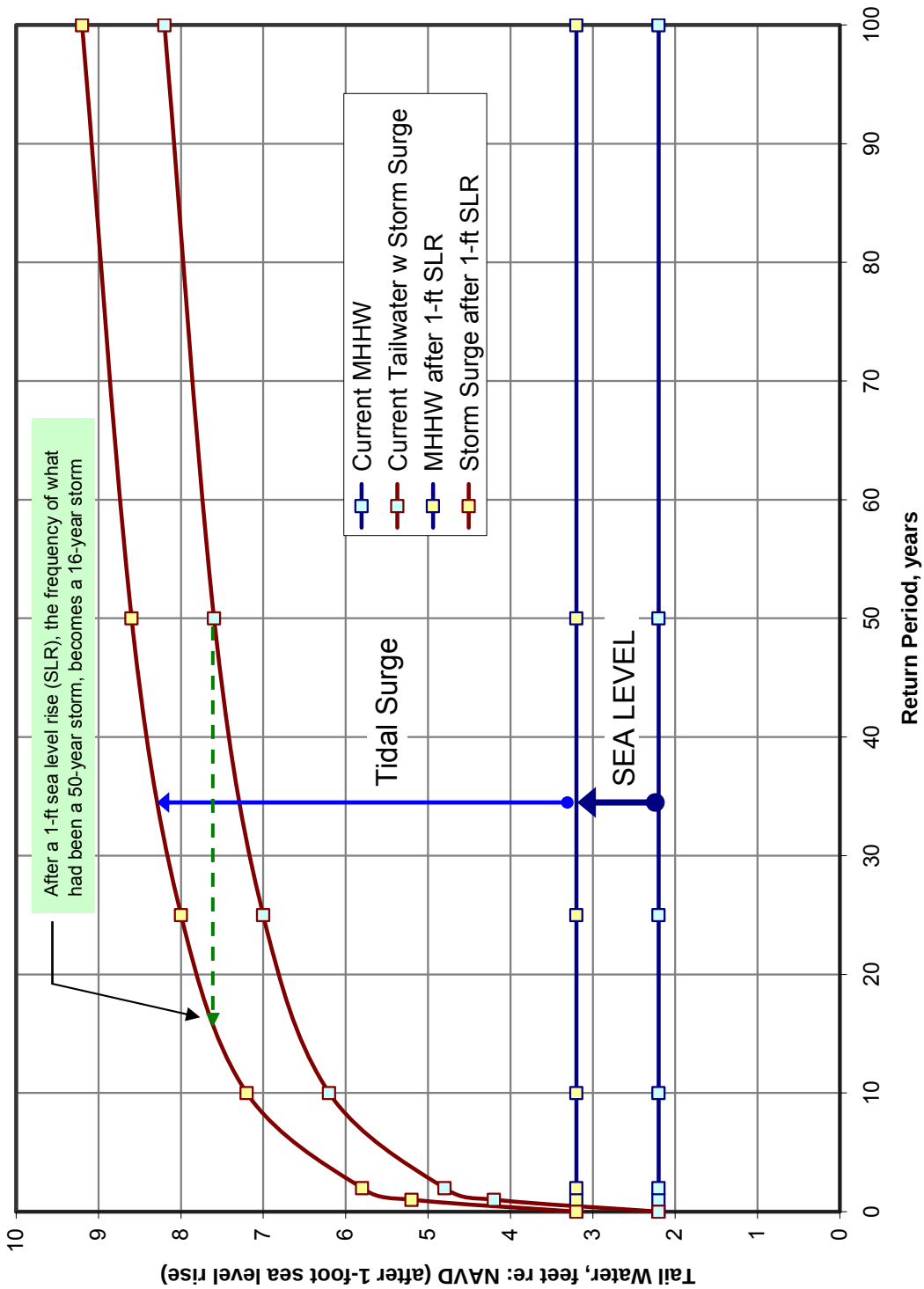
FIGURE 4-9

25 ft  
100 ft  
Horizontal and Vertical Scale in Feet  
Vertical exaggeration is 4x.



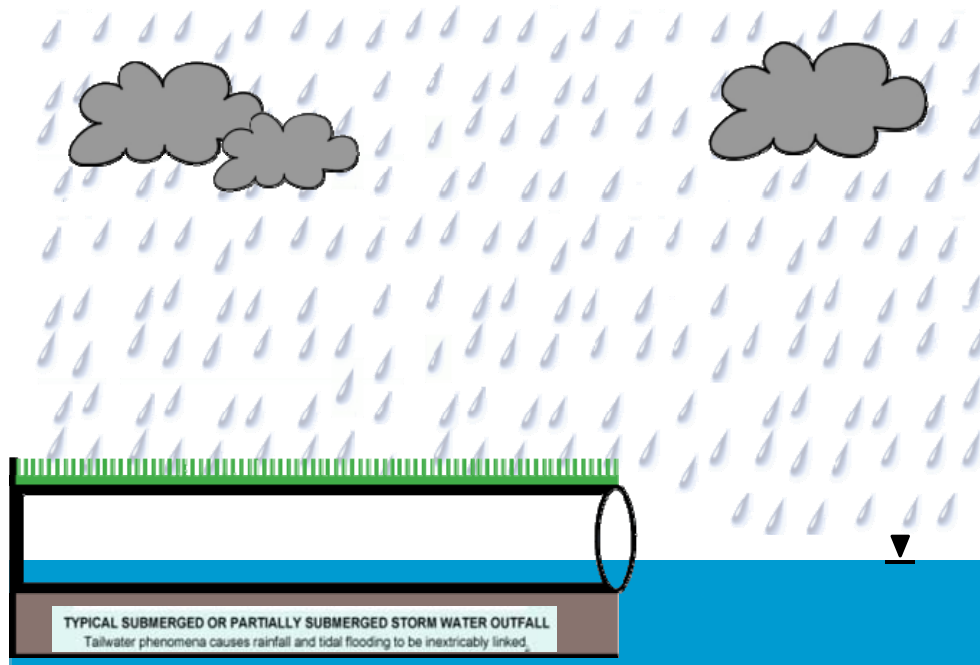
**WATER LEVEL ELEVATIONS AT SEWELLS POINT FOR VARIOUS RETURN PERIODS**  
Based on Current Sea Level Elevation  
City-wide Coastal Flooding Study  
Norfolk, Virginia

FIGURE 5-1

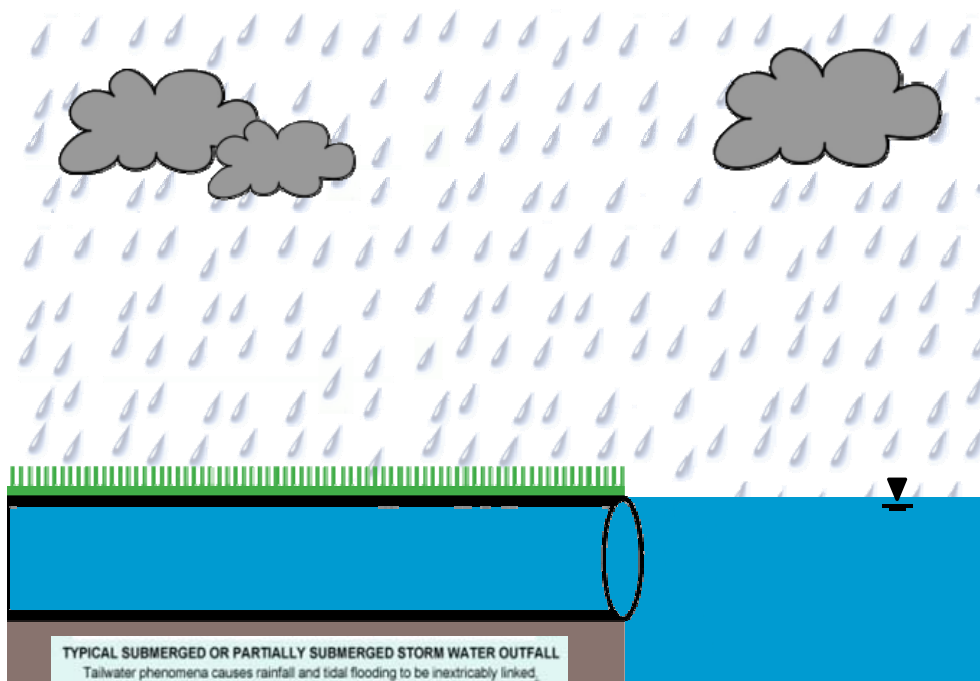


**IMPLICATIONS OF FUTURE SEA LEVEL RISE**  
City-wide Coastal Flooding Study  
Norfolk, Virginia

### Rainfall Predominant



### Coincident Surge Event



## TAILWATER PHENOMENA

### City-wide Coastal Flooding Study

### Norfolk, Virginia



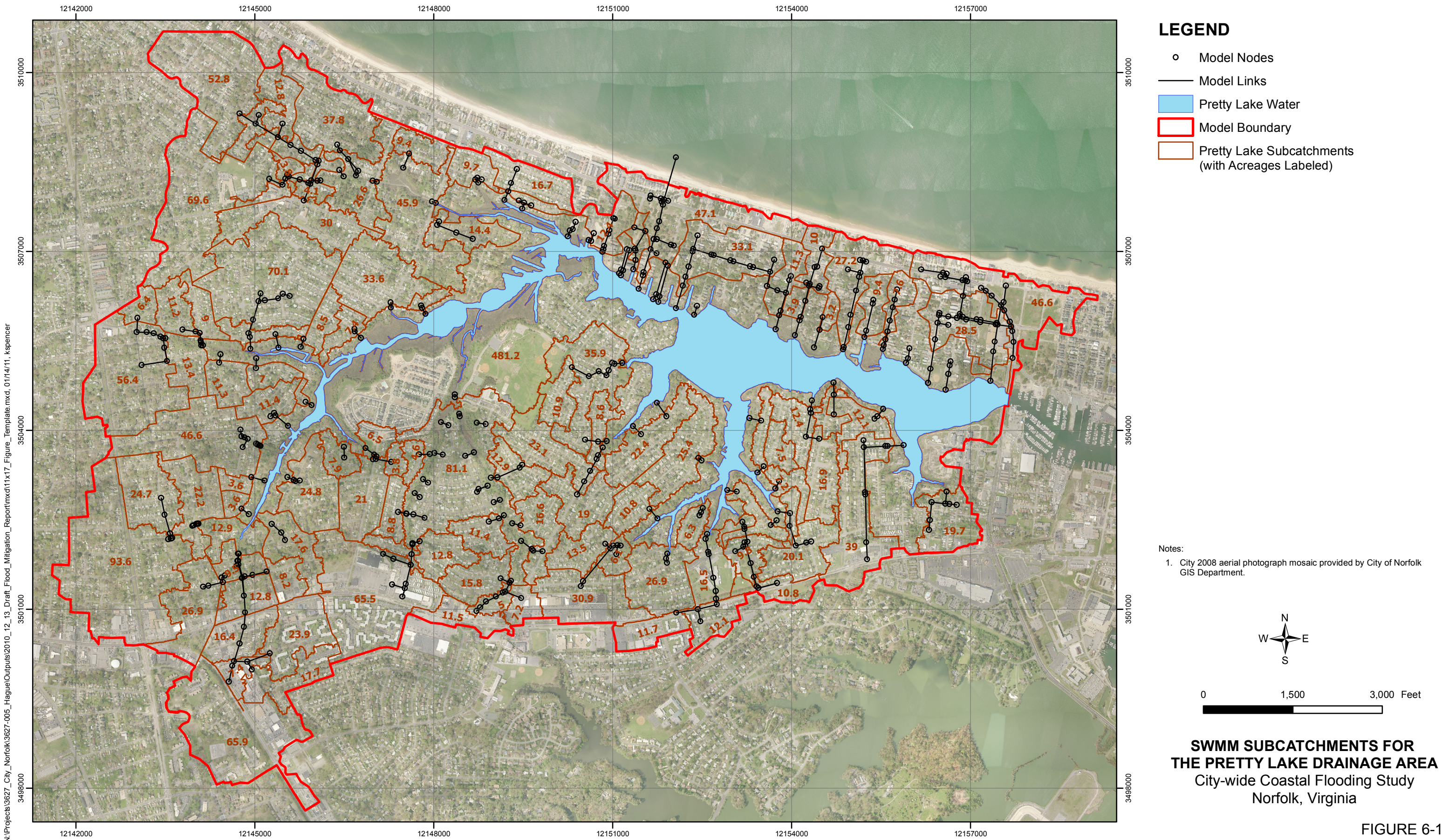


FIGURE 6-1



DRAFT

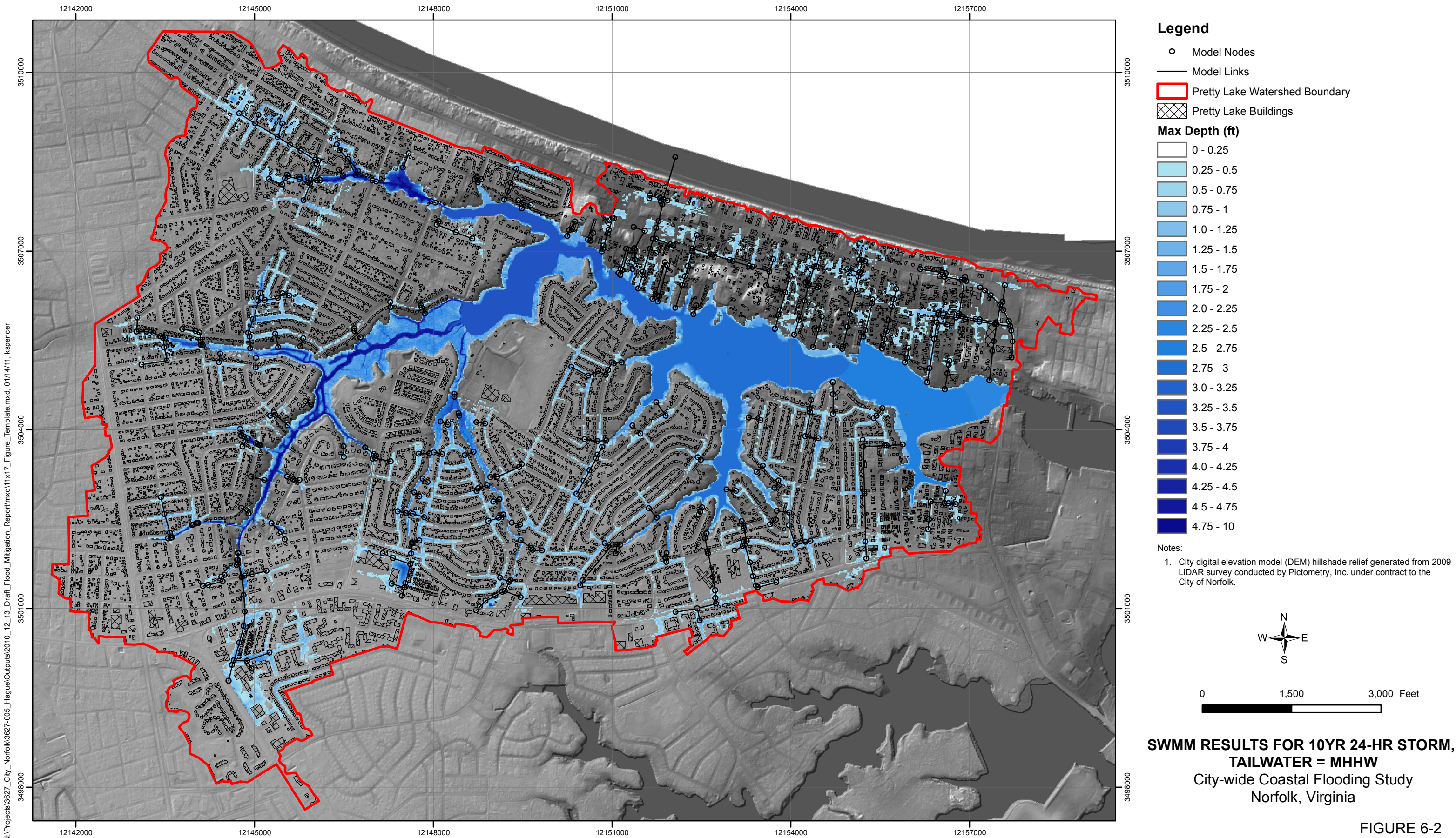


FIGURE 6-2



DRAFT

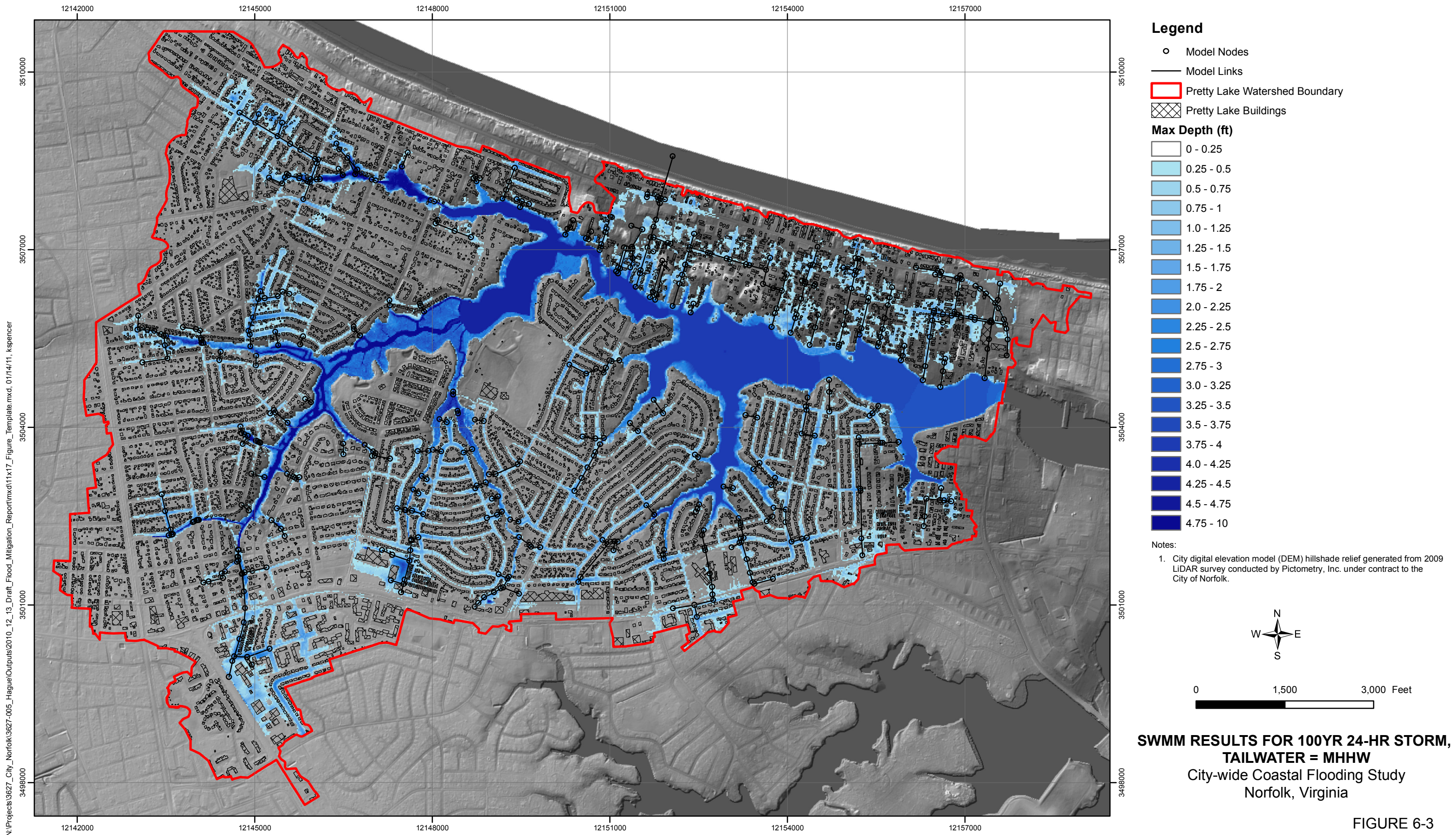


FIGURE 6-3



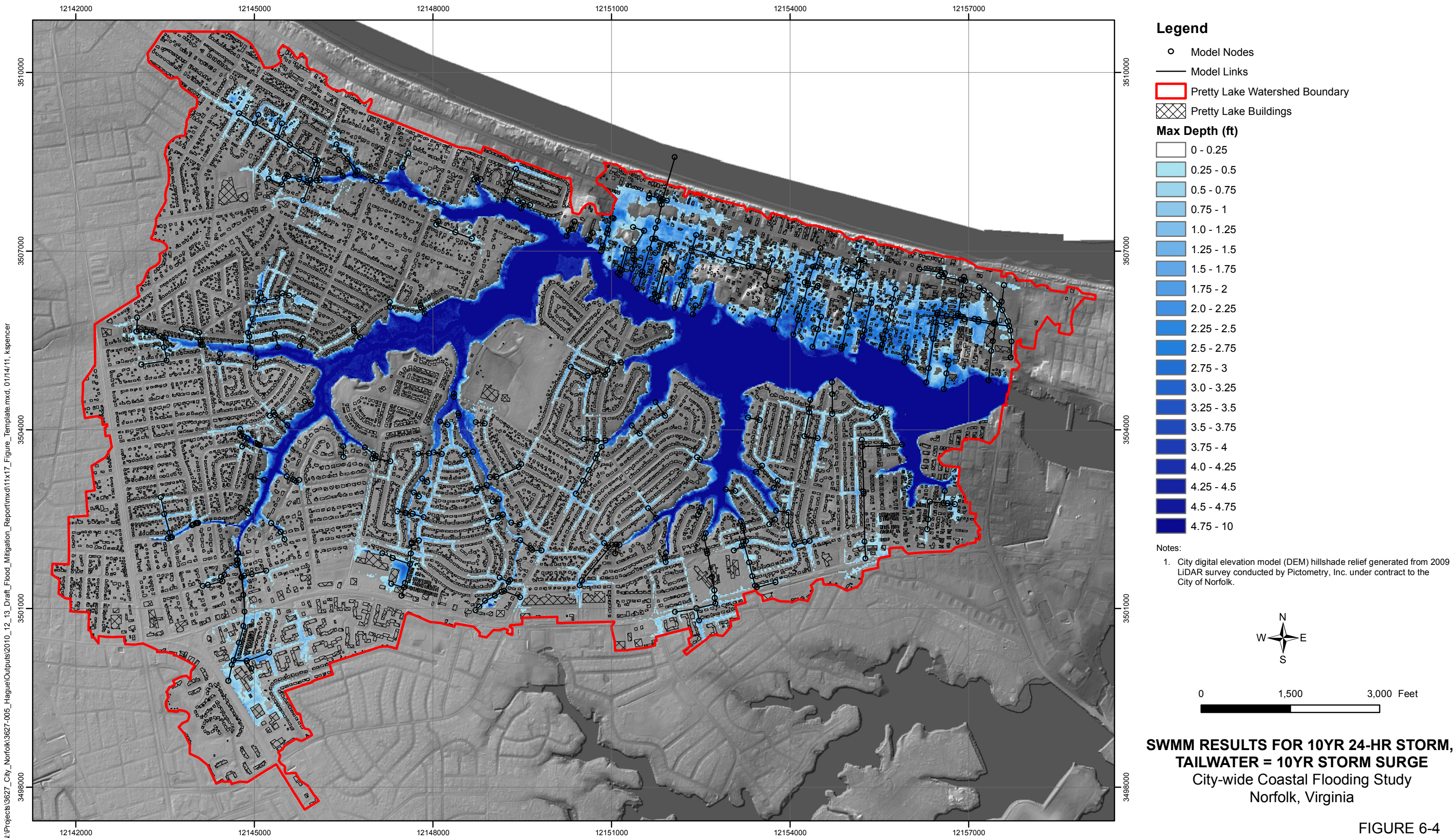


FIGURE 6-4



DRAFT

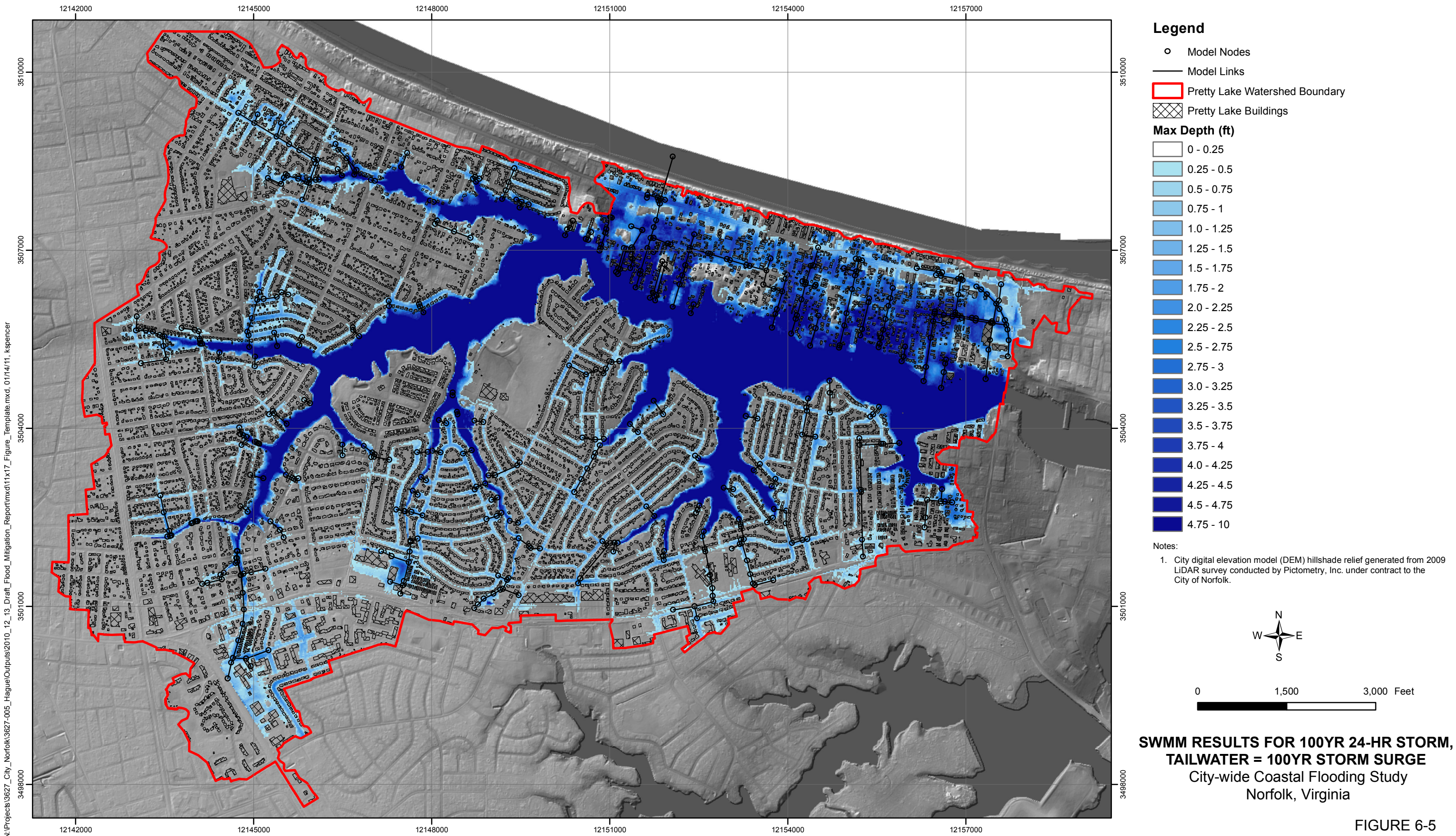


FIGURE 6-5



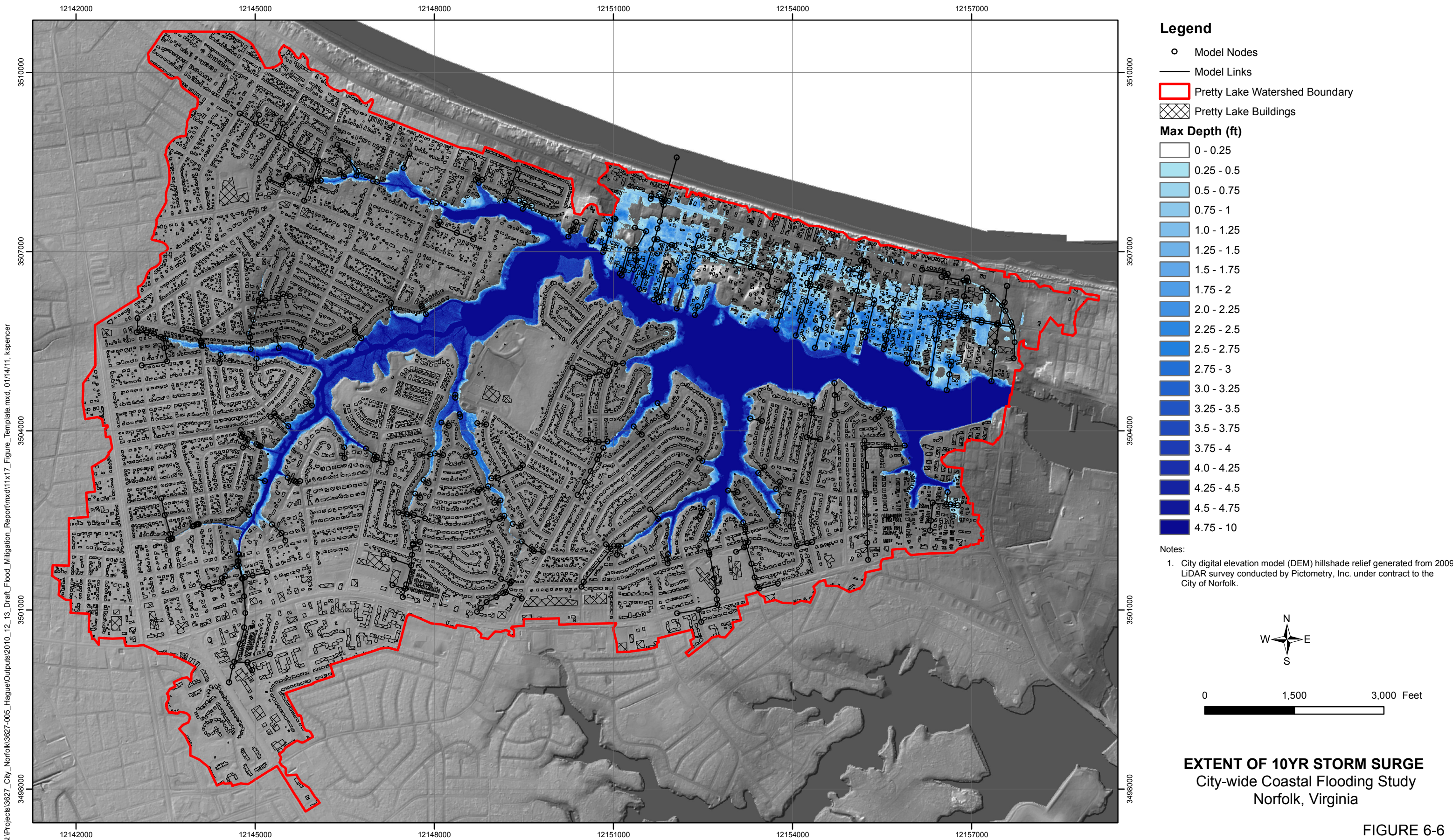


FIGURE 6-6



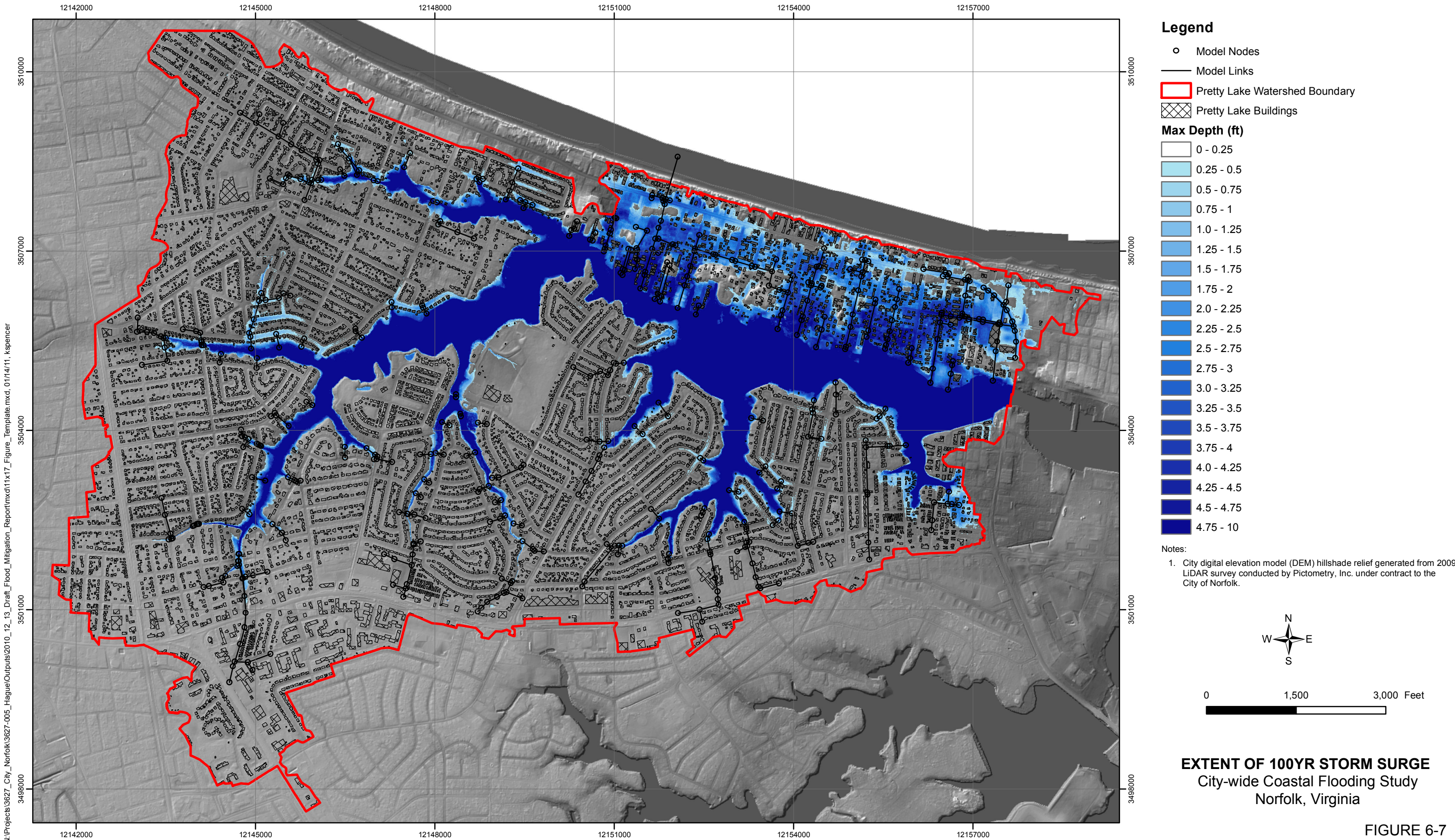
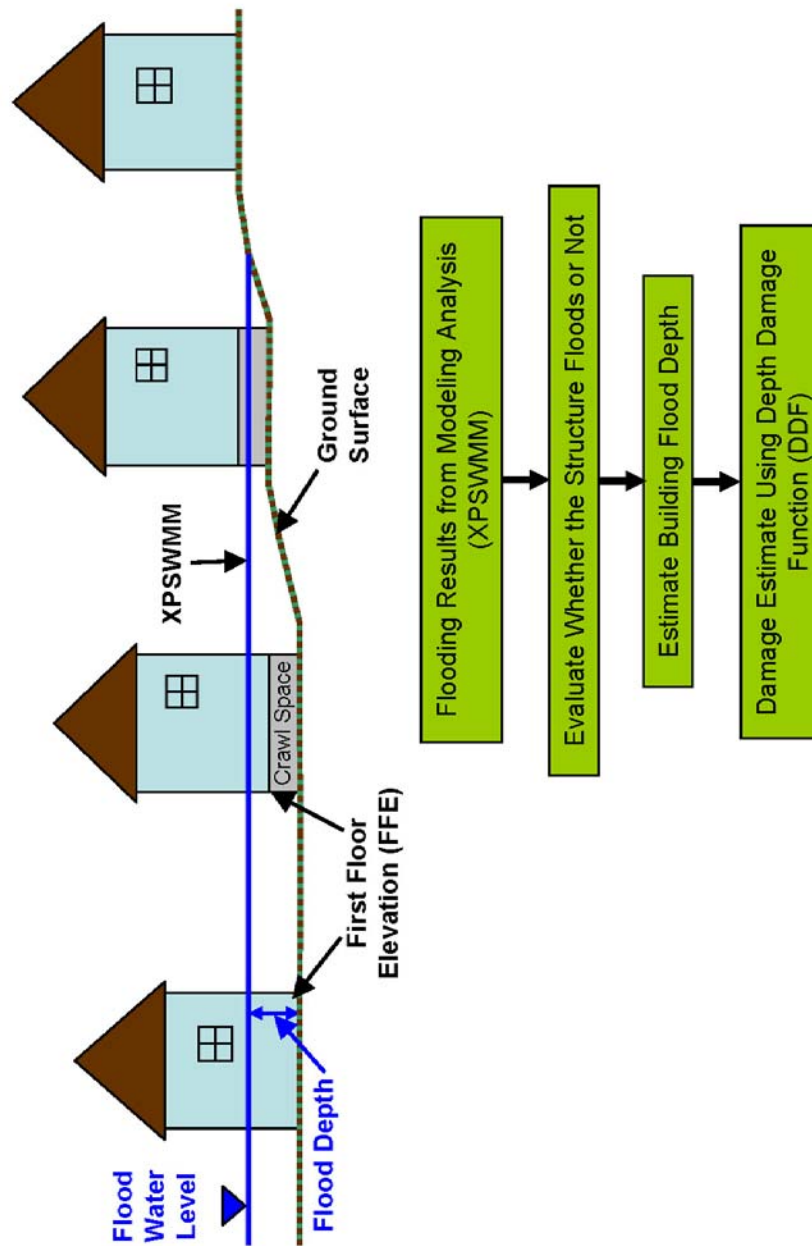


FIGURE 6-7

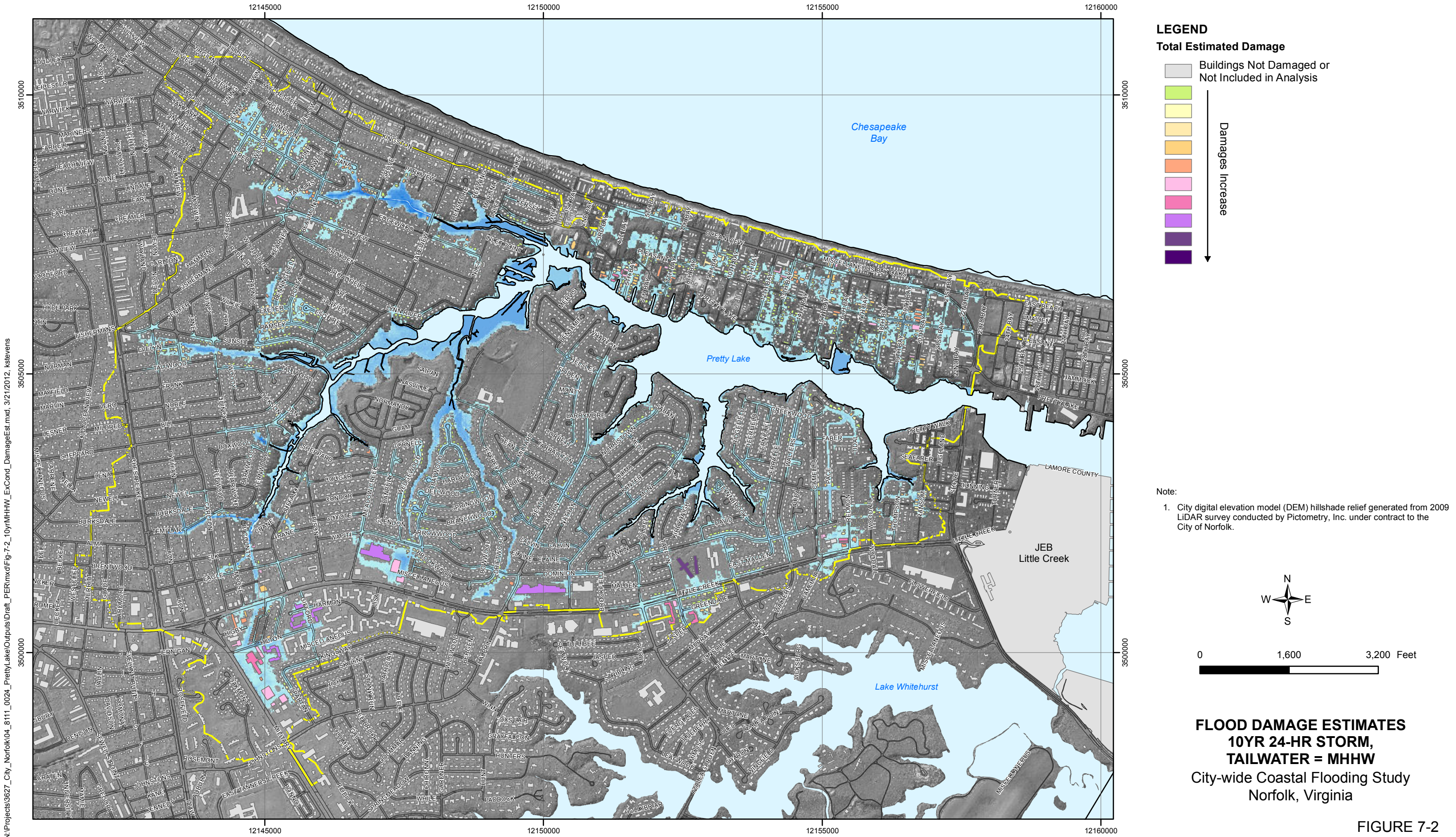




**DEPTH DAMAGE FUNCTION CONCEPT**  
City-wide Coastal Flooding Study  
Norfolk, Virginia

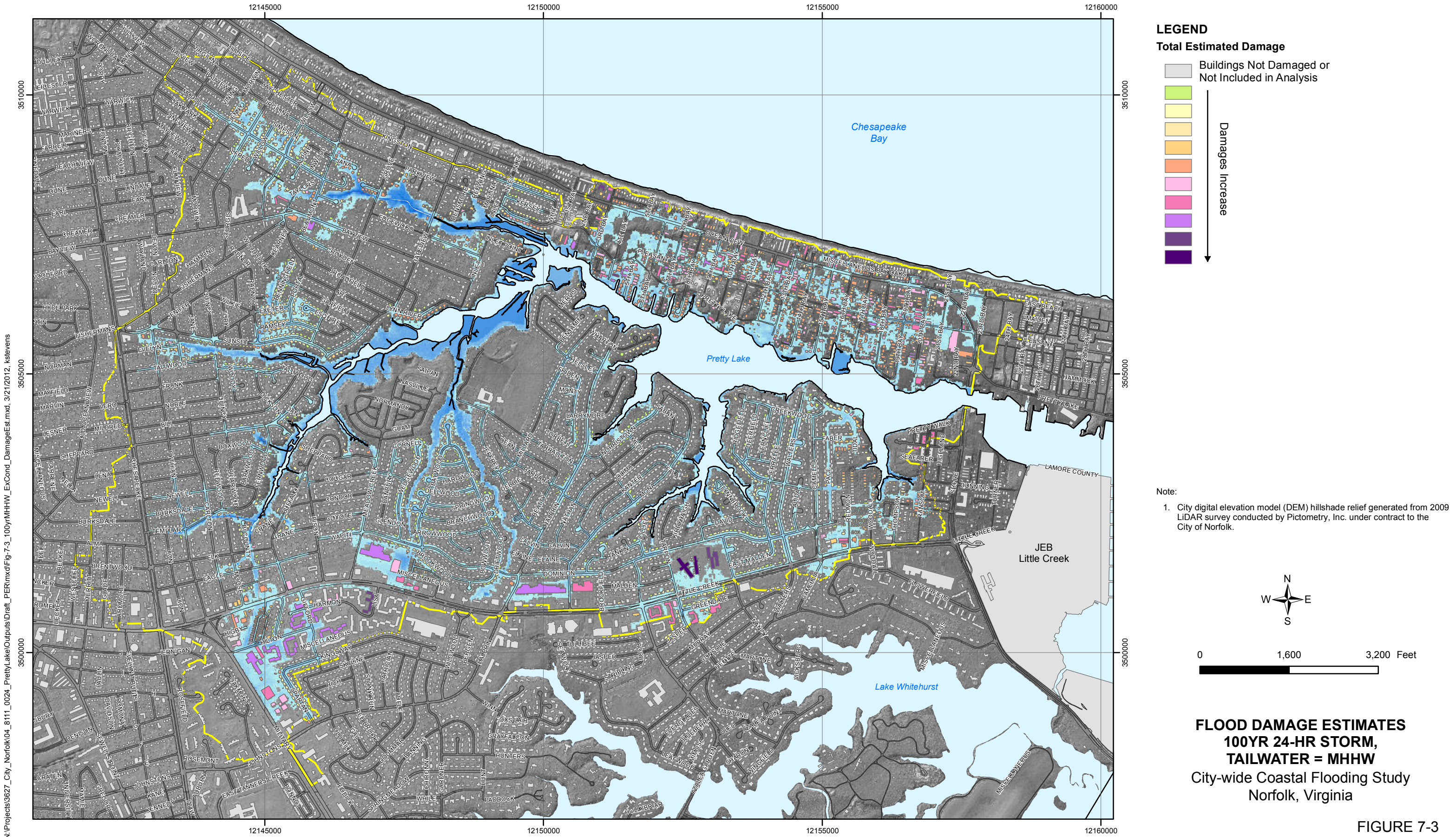


DRAFT



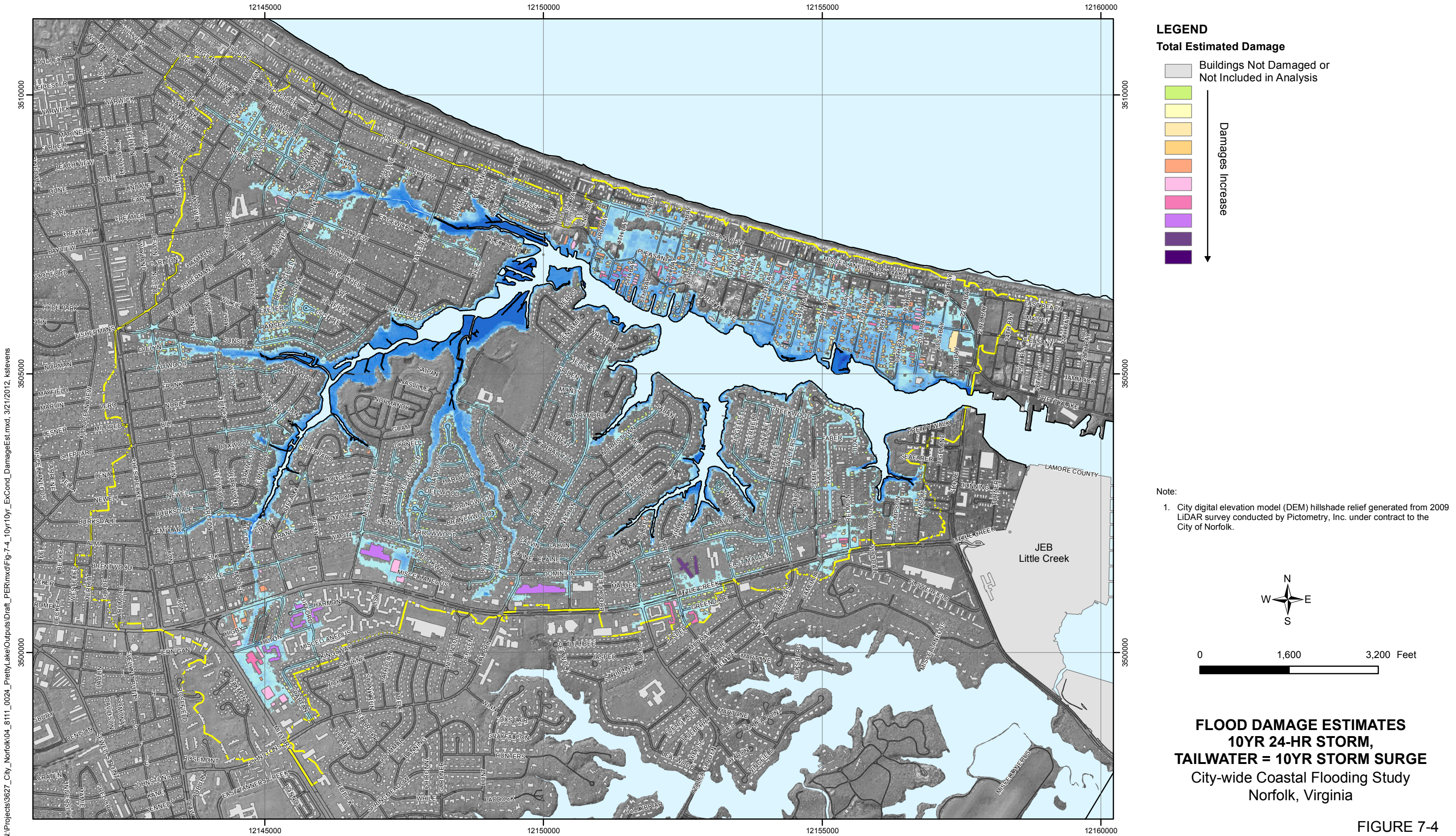


DRAFT





DRAFT





DRAFT

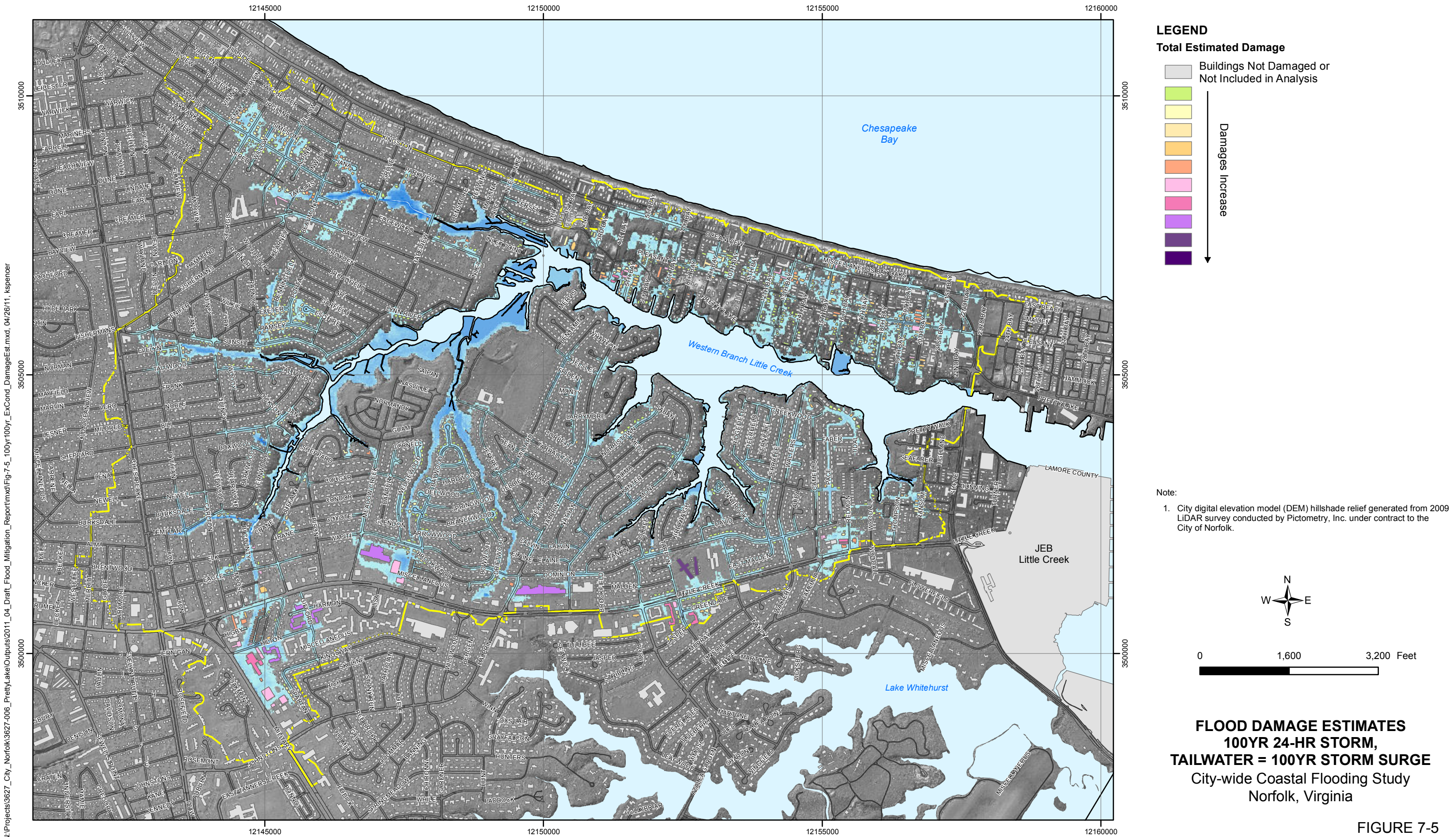
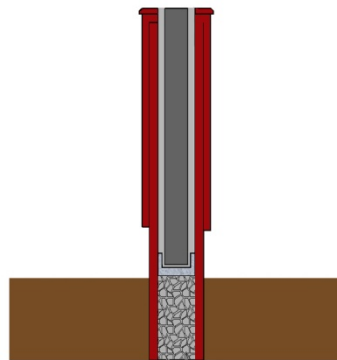
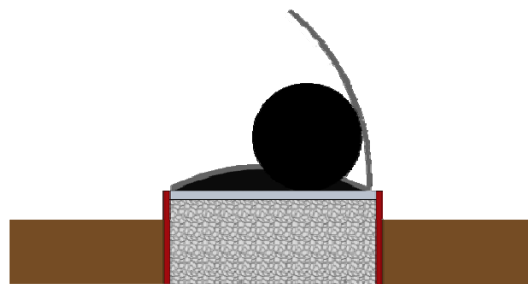


FIGURE 7-5

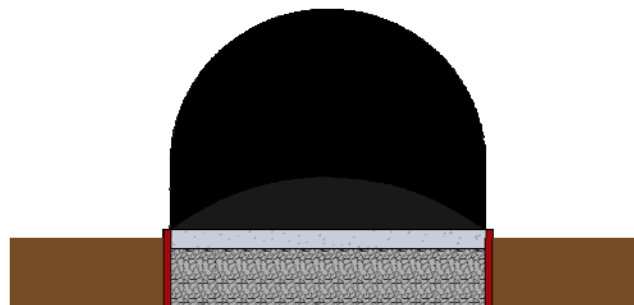




**Steel Gate Option**



**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



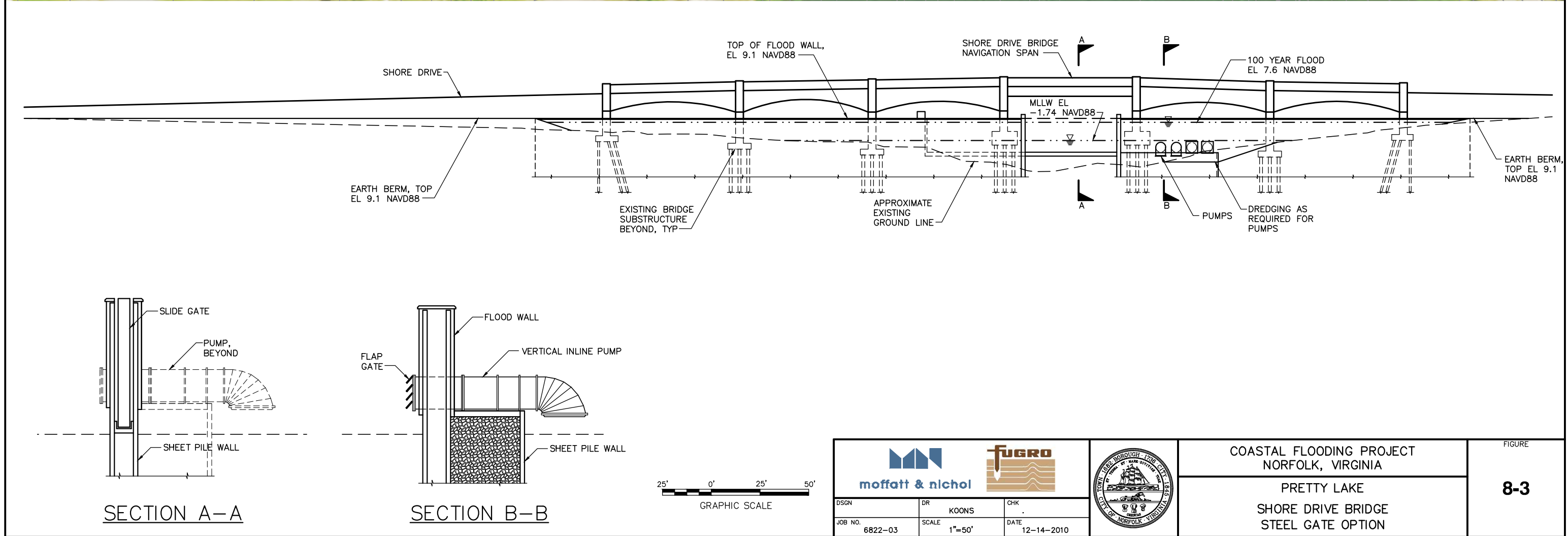
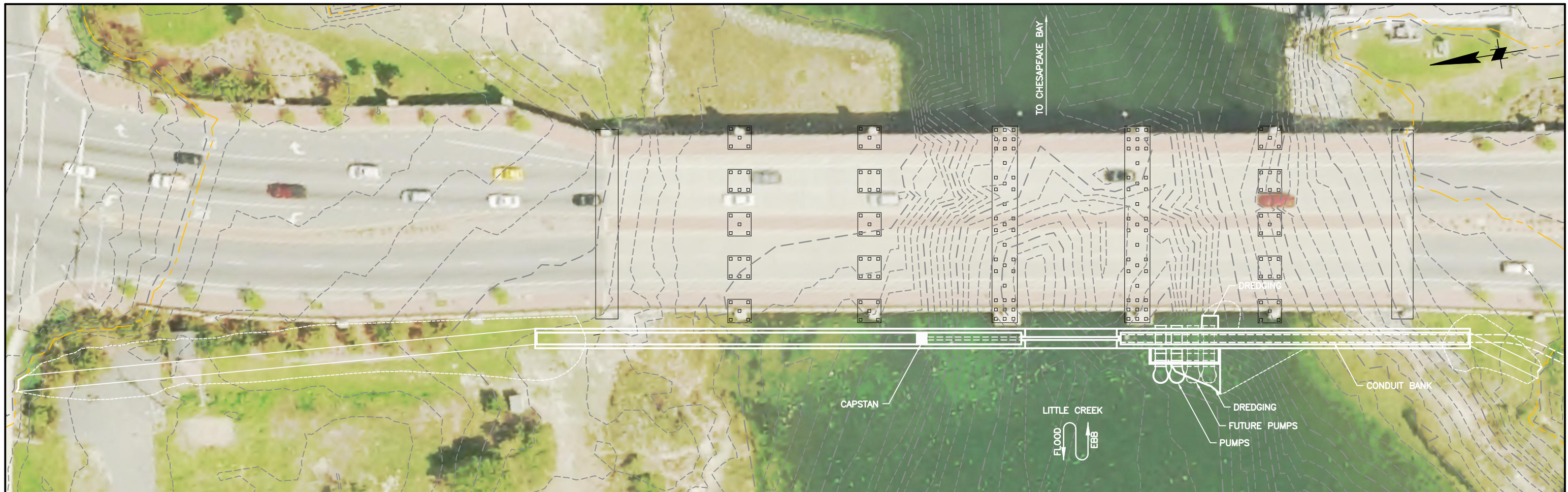
|                                                                                       |         |                                                                                       |  |
|---------------------------------------------------------------------------------------|---------|--------------------------------------------------------------------------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

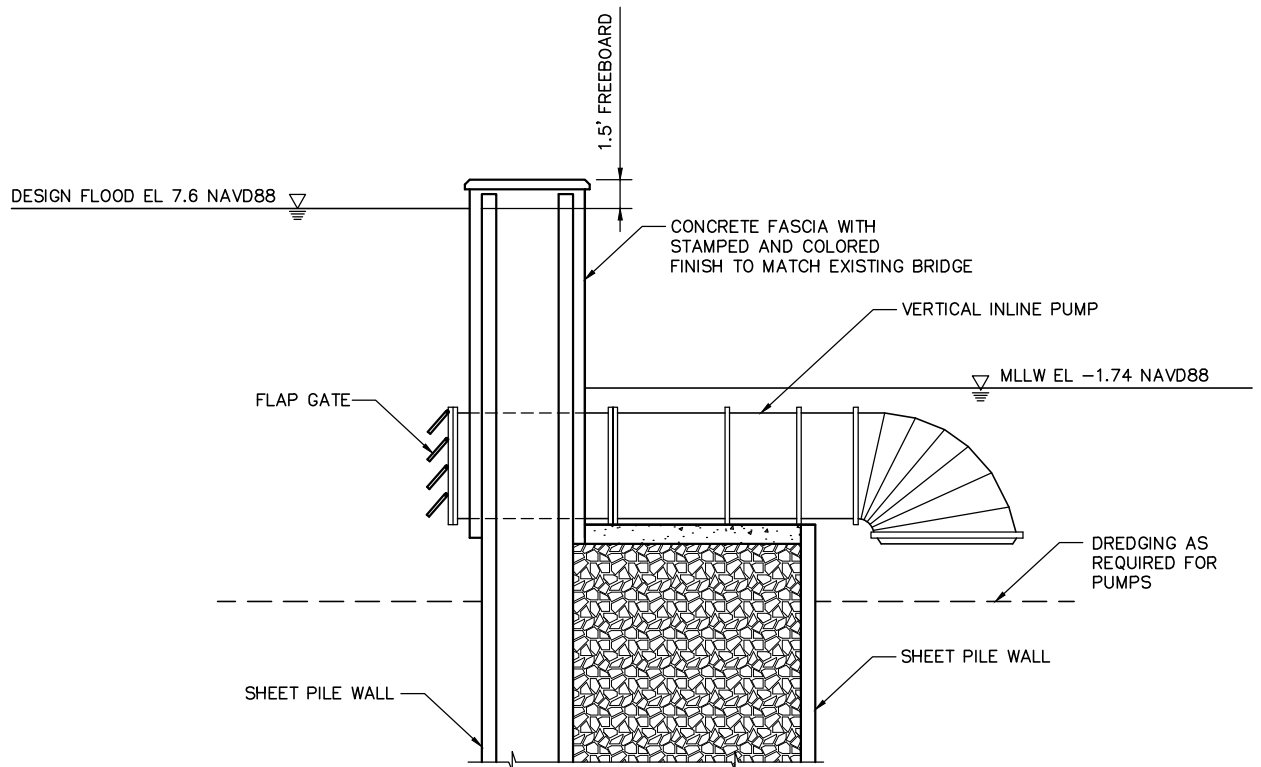


|                                                                      |  |
|----------------------------------------------------------------------|--|
| COASTAL FLOODING PROJECT<br>NORFOLK, VIRGINIA                        |  |
| PRETTY LAKE<br>TIDAL BARRIER WITH TIDE GATE,<br>PUMPS AND ROAD RAISE |  |

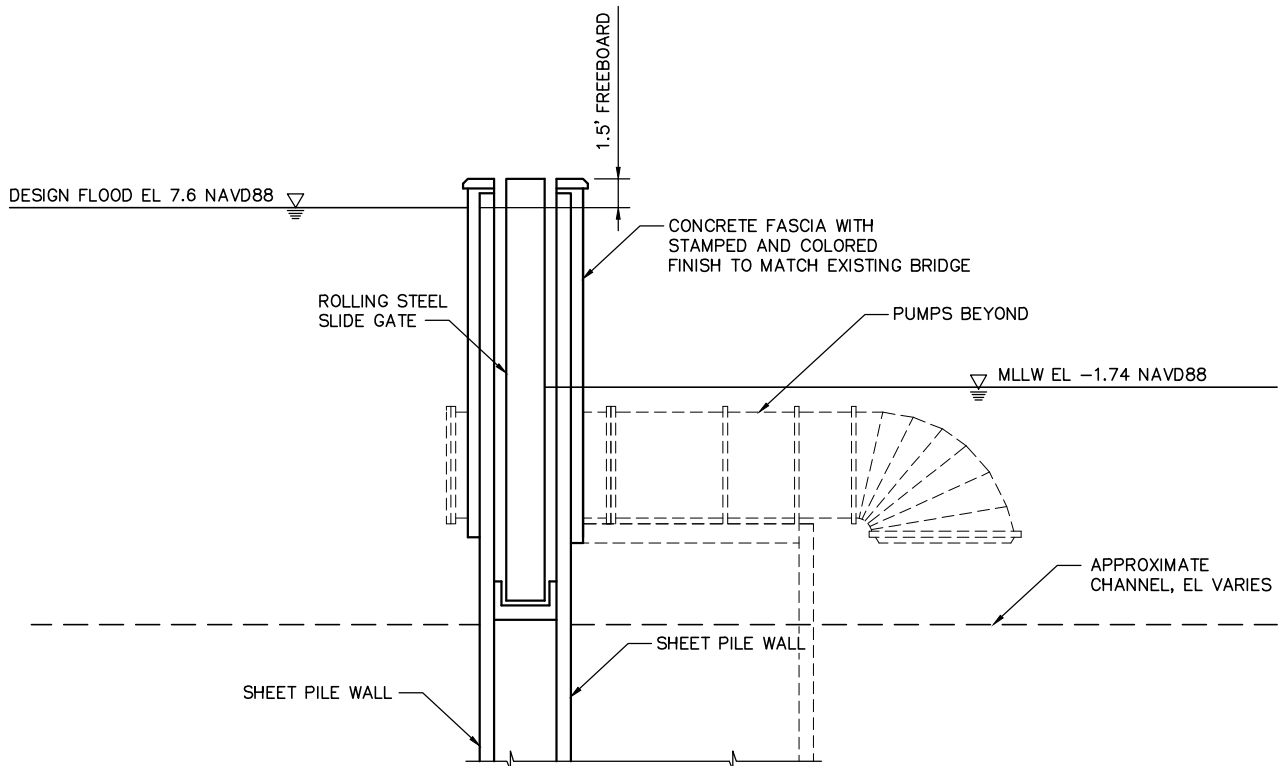
|        |
|--------|
| FIGURE |
| 8-2    |







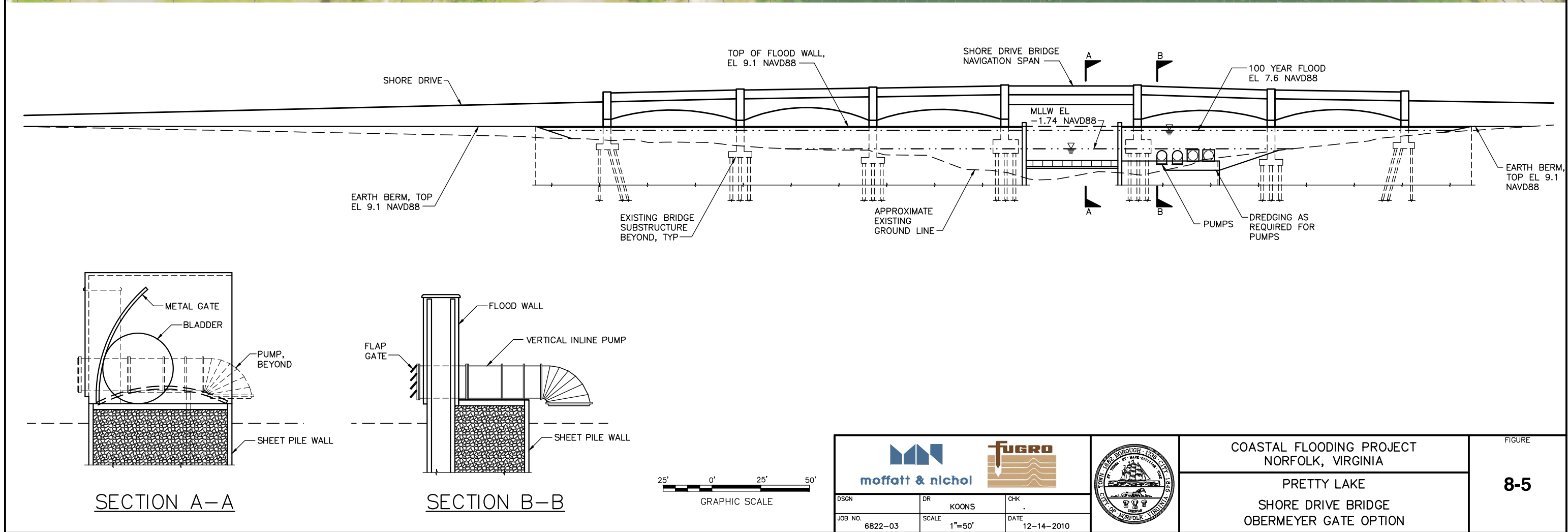
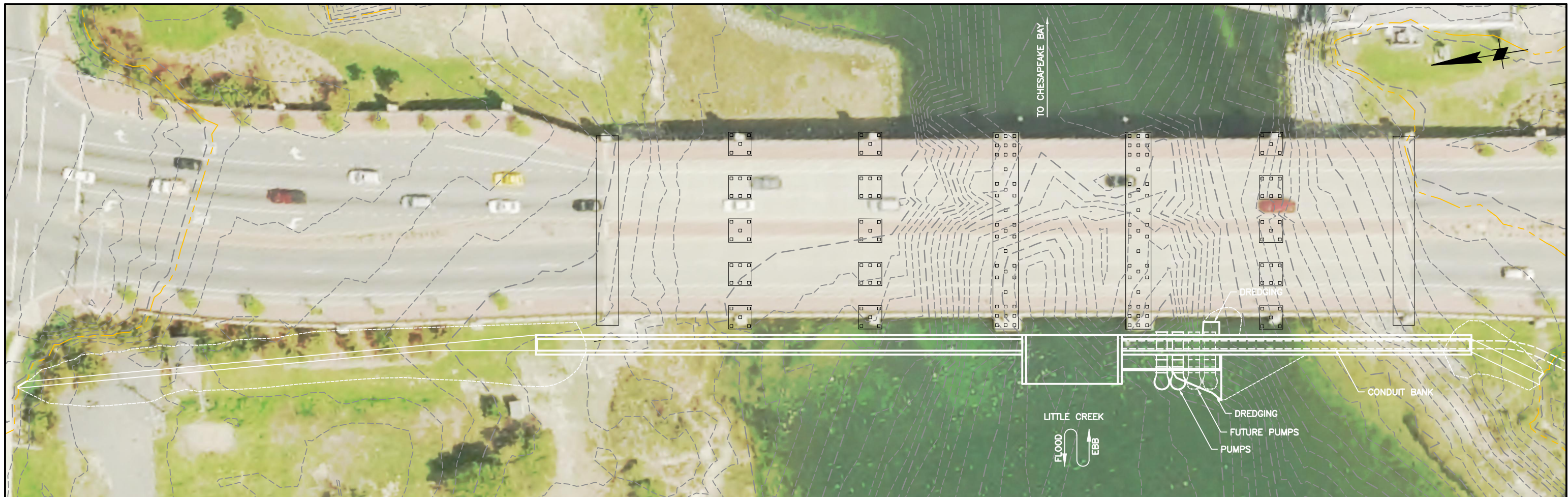
WALL SECTION AT PUMPS

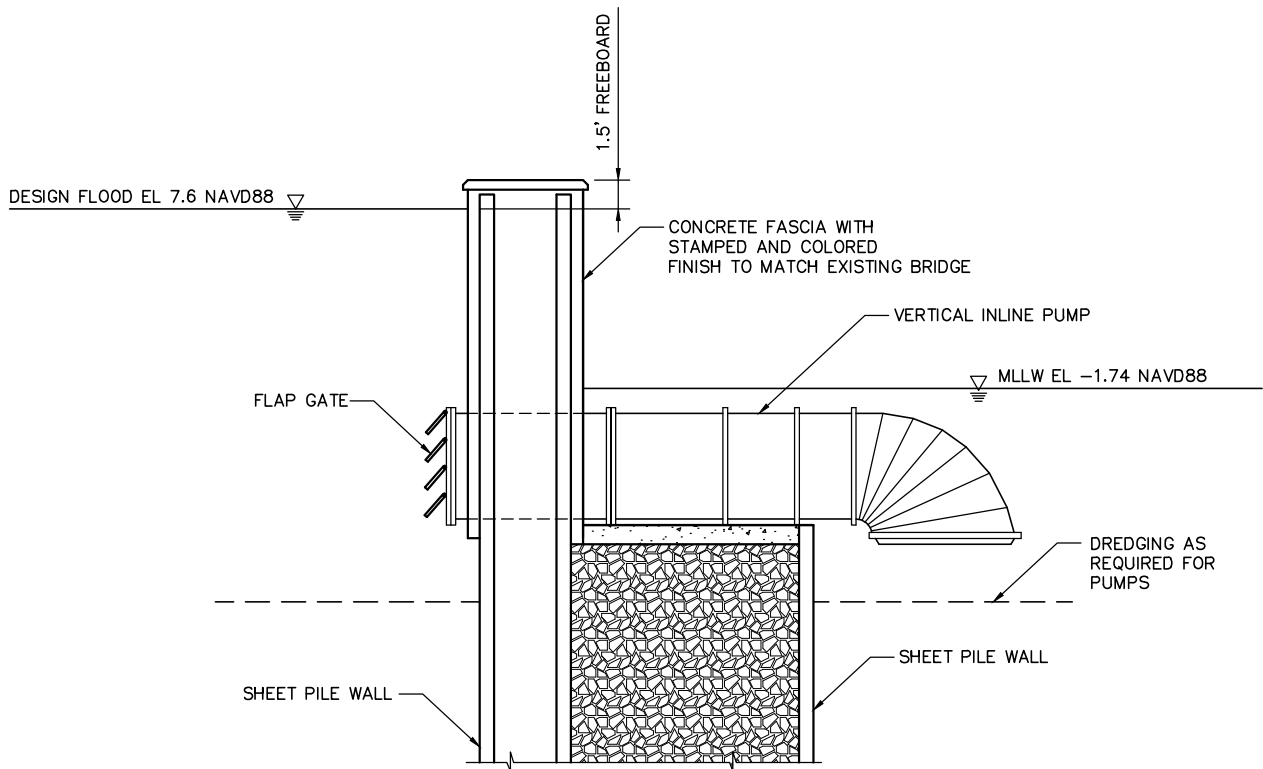


SECTION AT FLOOD STEEL GATE

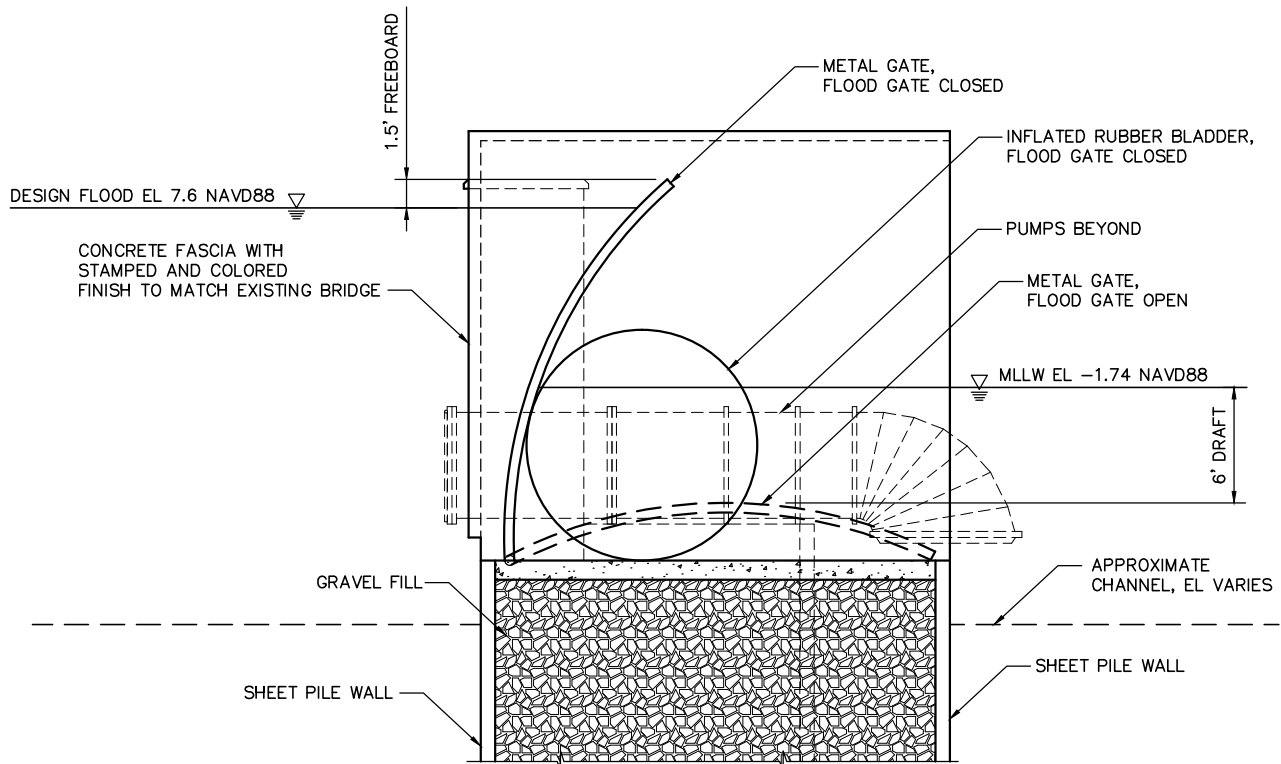
|                                                                                                                                                                         |                 |                    |                                                                                     |    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







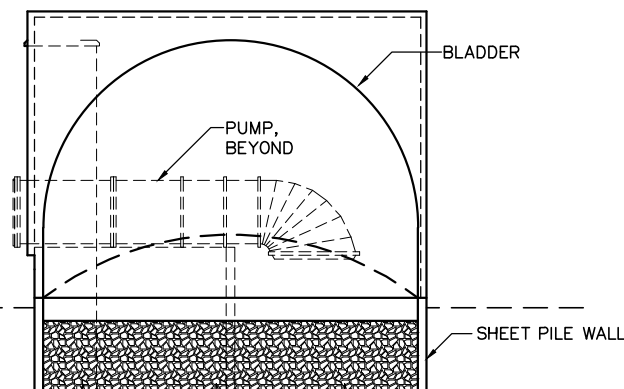
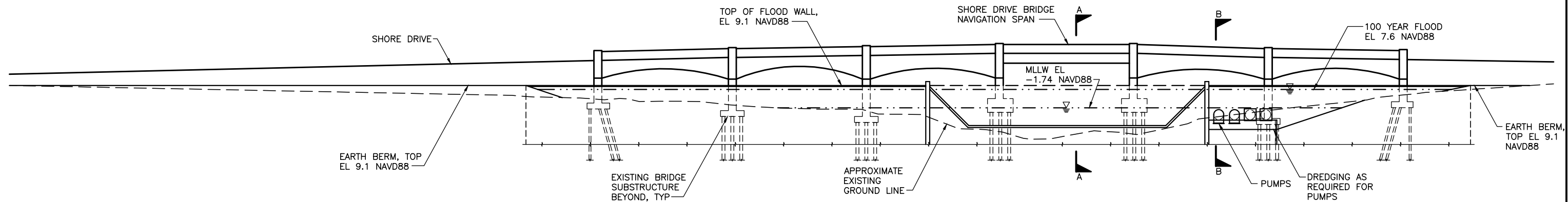
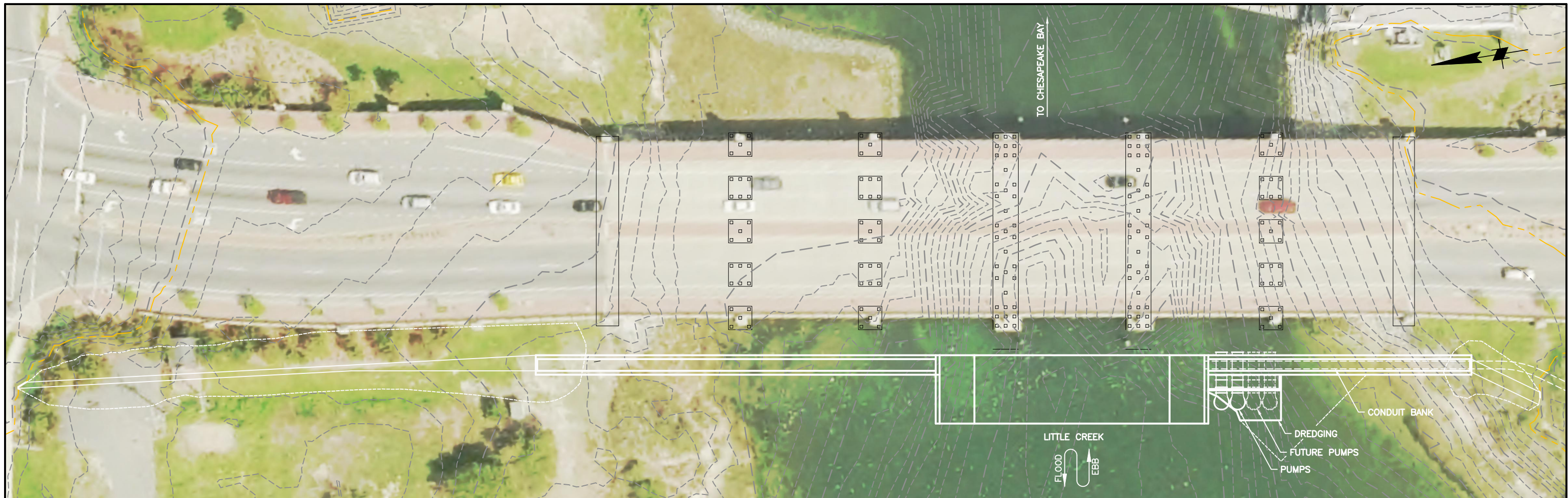
WALL SECTION AT PUMPS



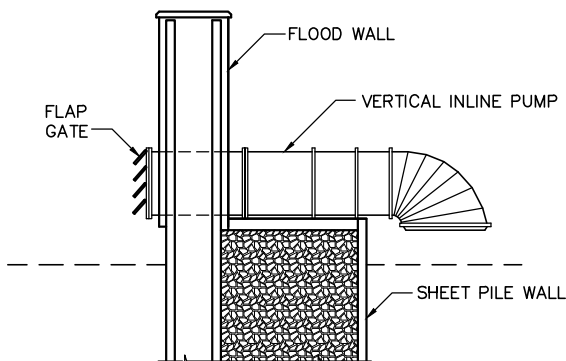
SECTION AT OBERMEYER GATE

|                                                                                                                                                                         |                 |                    |                                                                                     |                                                                       |                          |
|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------|--------------------|-------------------------------------------------------------------------------------|-----------------------------------------------------------------------|--------------------------|
|   |                 |                    |  | COASTAL FLOODING PROJECT<br>NORFOLK, VIRGINIA                         | FIGURE<br><br><b>8-6</b> |
| DSGN                                                                                                                                                                    | DR              | CHK                |                                                                                     | PRETTY LAKE<br>SHORE DRIVE BRIDGE<br>OBERMEYER GATE OPTION – SECTIONS |                          |
| JOB NO.<br>6822-03                                                                                                                                                      | SCALE<br>1"=10' | DATE<br>12-14-2010 |                                                                                     |                                                                       |                          |

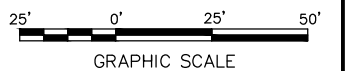




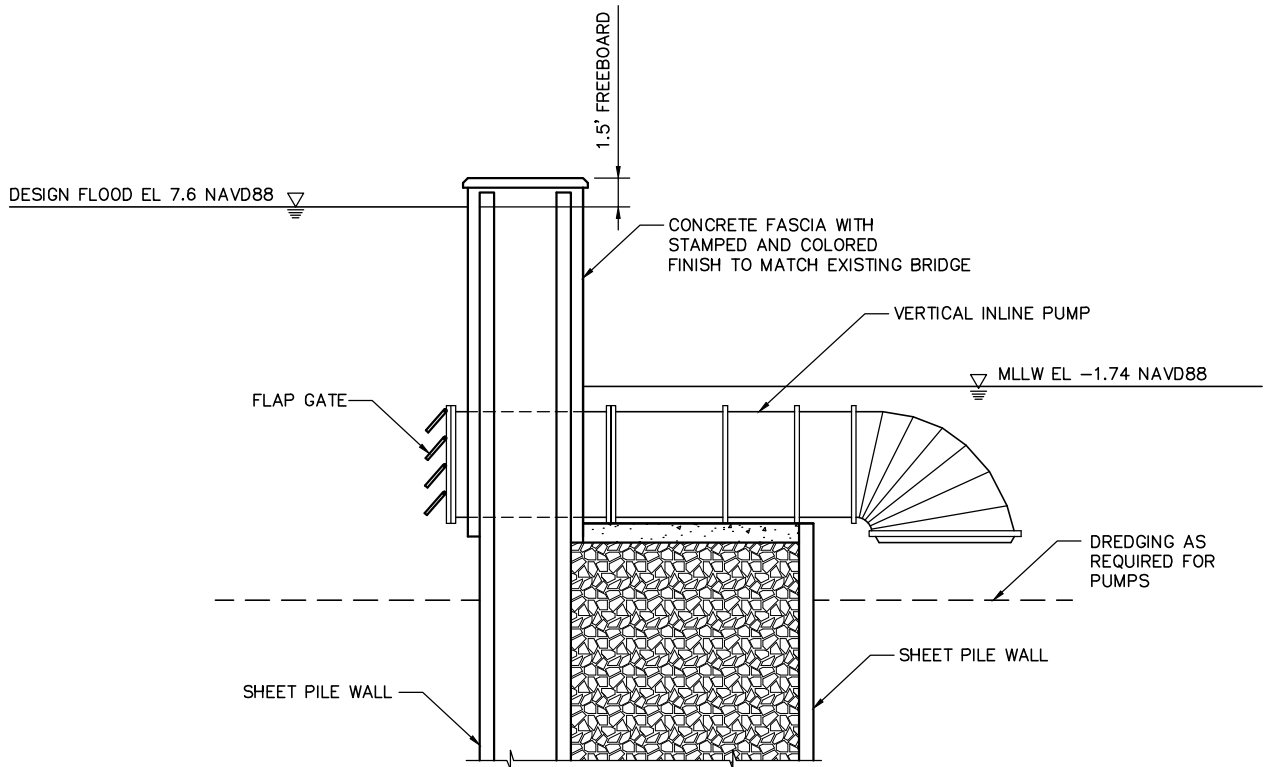
SECTION A-A



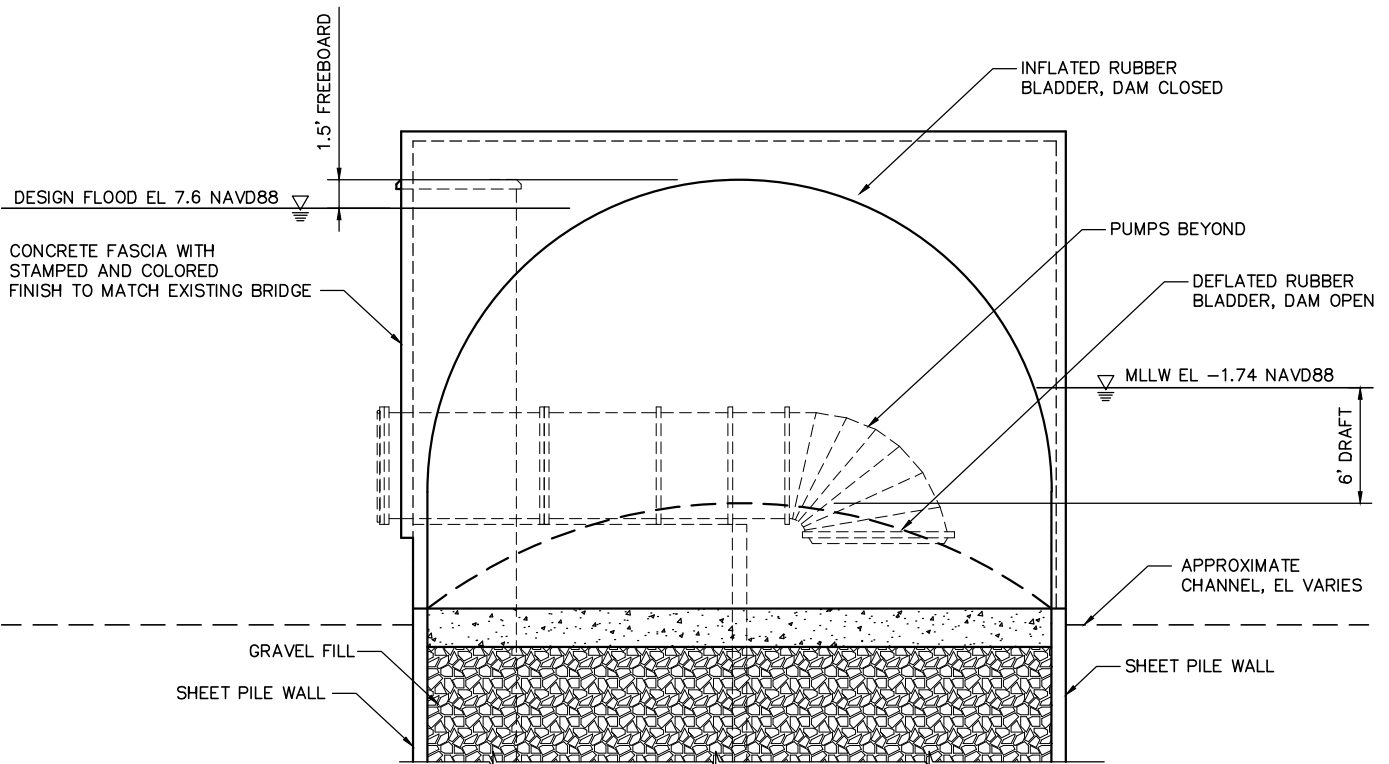
SECTION B-B



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

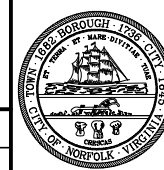
**8-8**

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





|                                                                                                                                                                                       |                                                     |                                                     |                                                                                                                                 |  |                      |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------|-----------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------|--|----------------------|
| <div></div> |                                                     |                                                     | <div>COASTAL FLOODING PROJECT<br/>NORFOLK, VIRGINIA</div> <div>PRETTY LAKE<br/>SHORE DRIVE BRIDGE<br/>SUBSTATION LOCATION</div> |  | FIGURE<br><b>8-9</b> |
| <div>DSGN</div> <div>JOB NO.<br/>6822-03</div>                                                                                                                                        | <div>DR<br/>KOONS</div> <div>SCALE<br/>1"=30'</div> | <div>CHK<br/>.</div> <div>DATE<br/>12-14-2010</div> |                                                                                                                                 |  |                      |



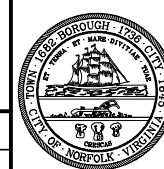




- TOTAL BERM LENGTH = 2,465 LF
- TOTAL UPSTREAM BULKHEAD LENGTH = 20,760 LF
- TOTAL ROAD LENGTH = 3,900 LF



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



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

|             |
|-------------|
| FIGURE      |
| <b>8-10</b> |

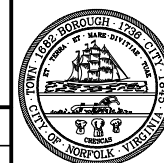




- TOTAL BERM LENGTH = 2,465 LF
- TOTAL UPSTREAM BULKHEAD LENGTH = 39,000 LF
- TOTAL ROAD LENGTH = 4,500 LF

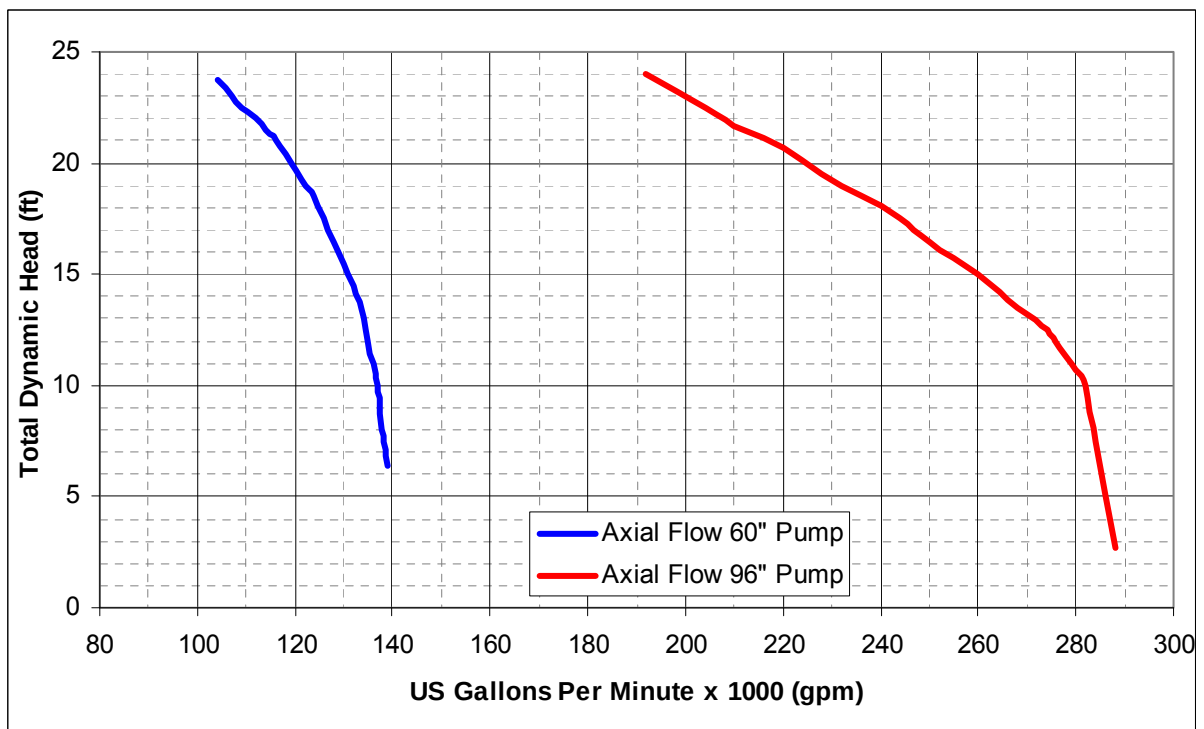


|                                                                                       |          |                                                                                       |  |
|---------------------------------------------------------------------------------------|----------|------------------------------------------------------------------------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



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





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



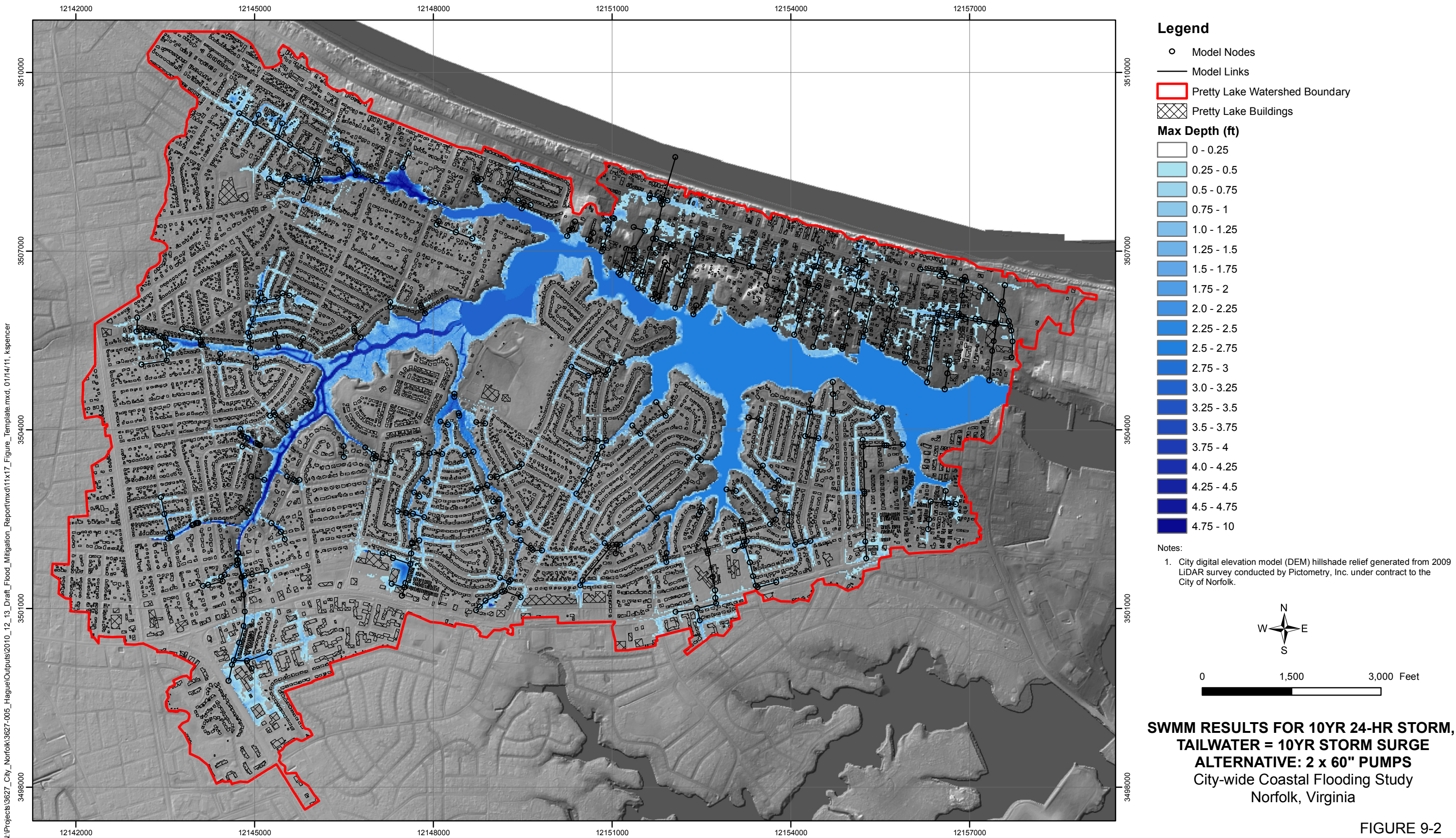


FIGURE 9-2



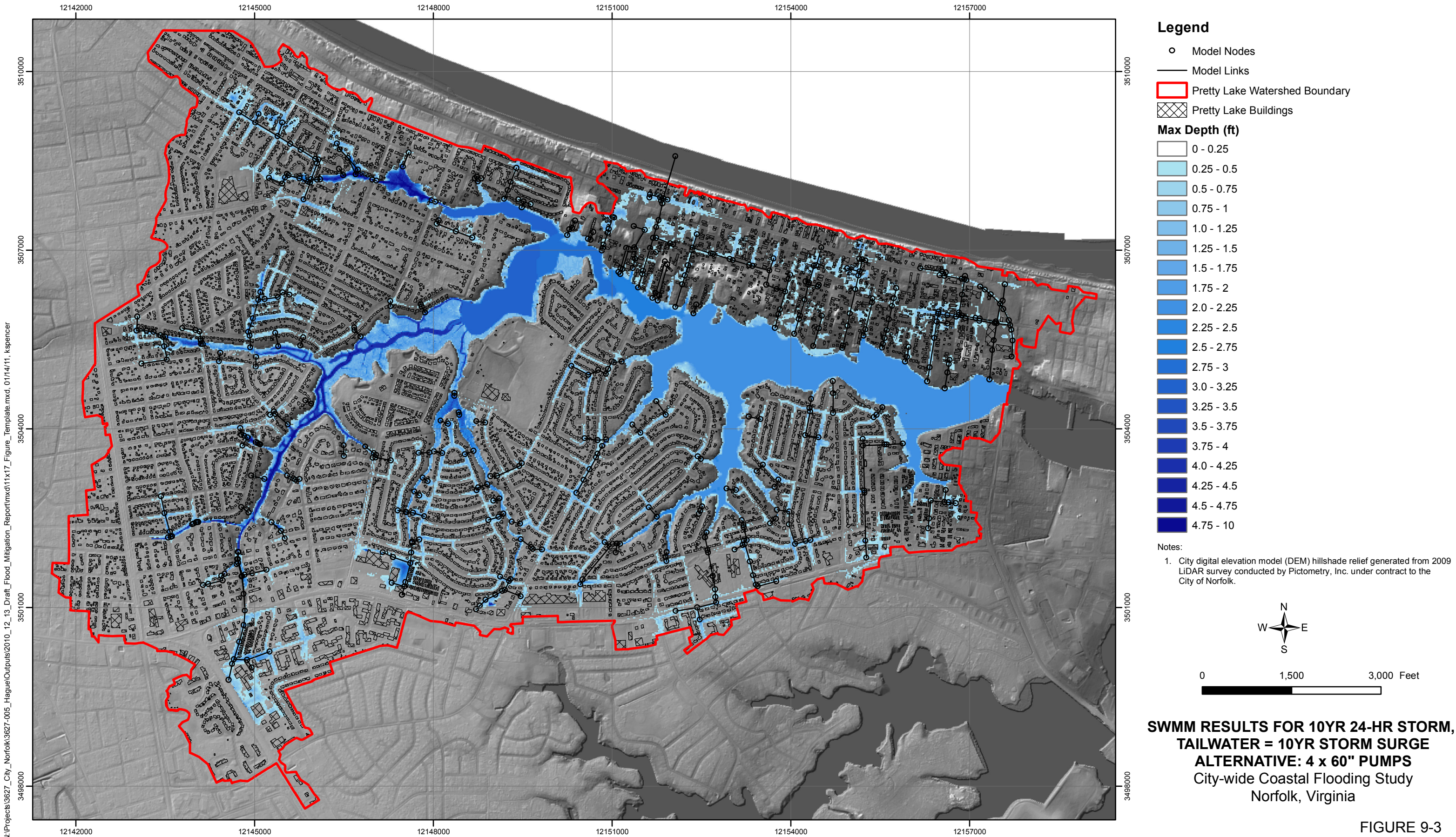


FIGURE 9-3



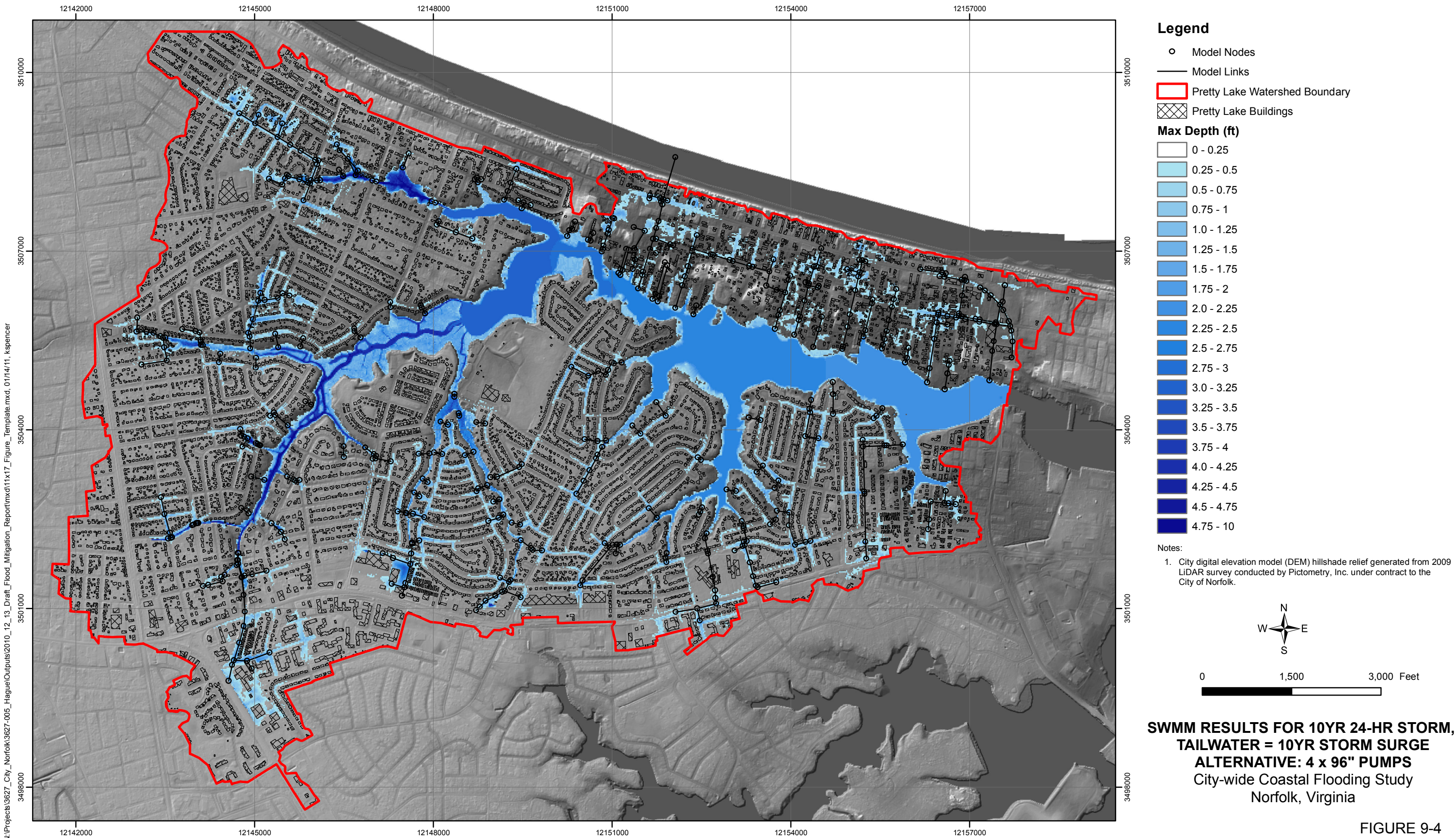


FIGURE 9-4



DRAFT

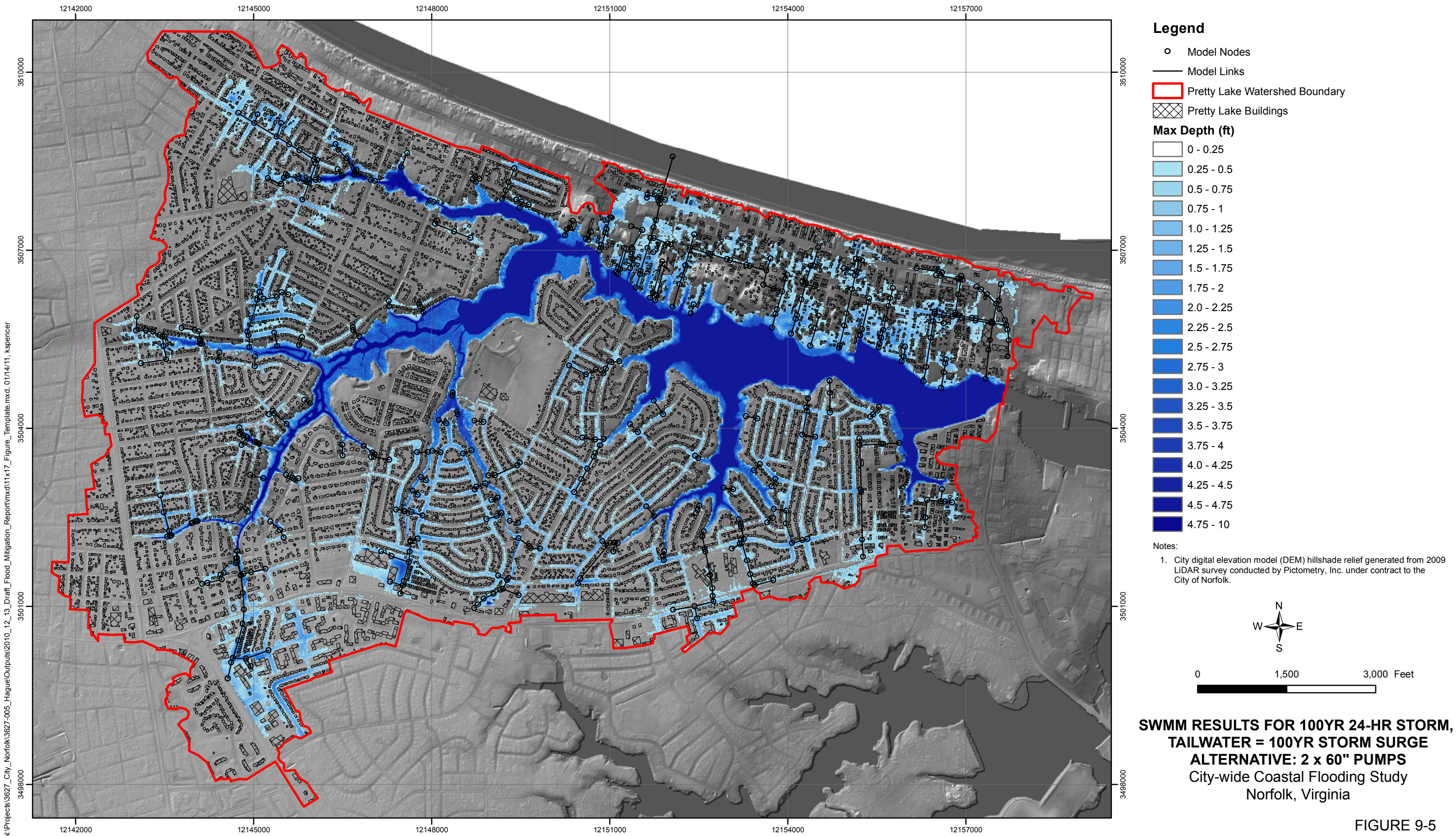


FIGURE 9-5



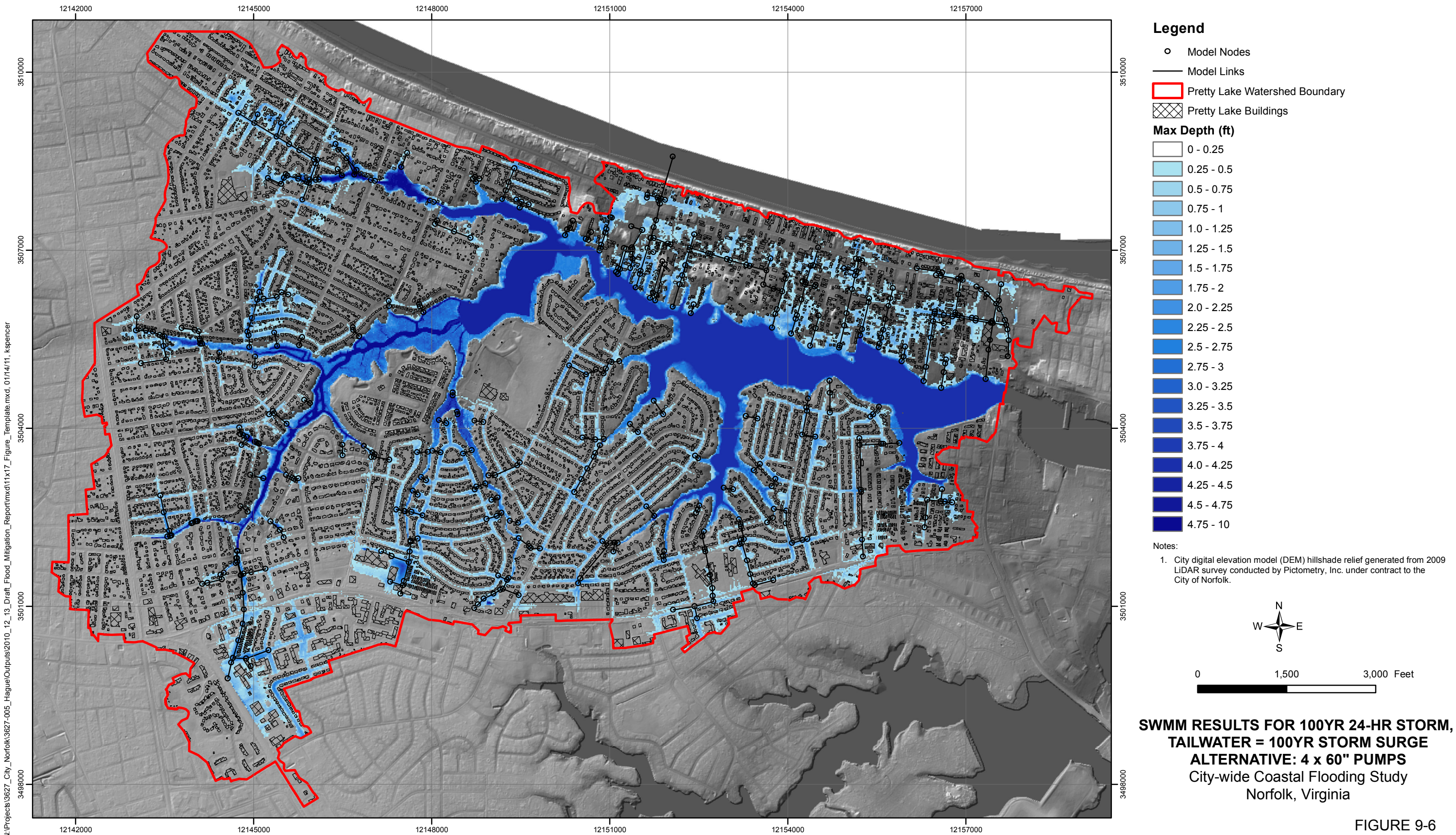


FIGURE 9-6



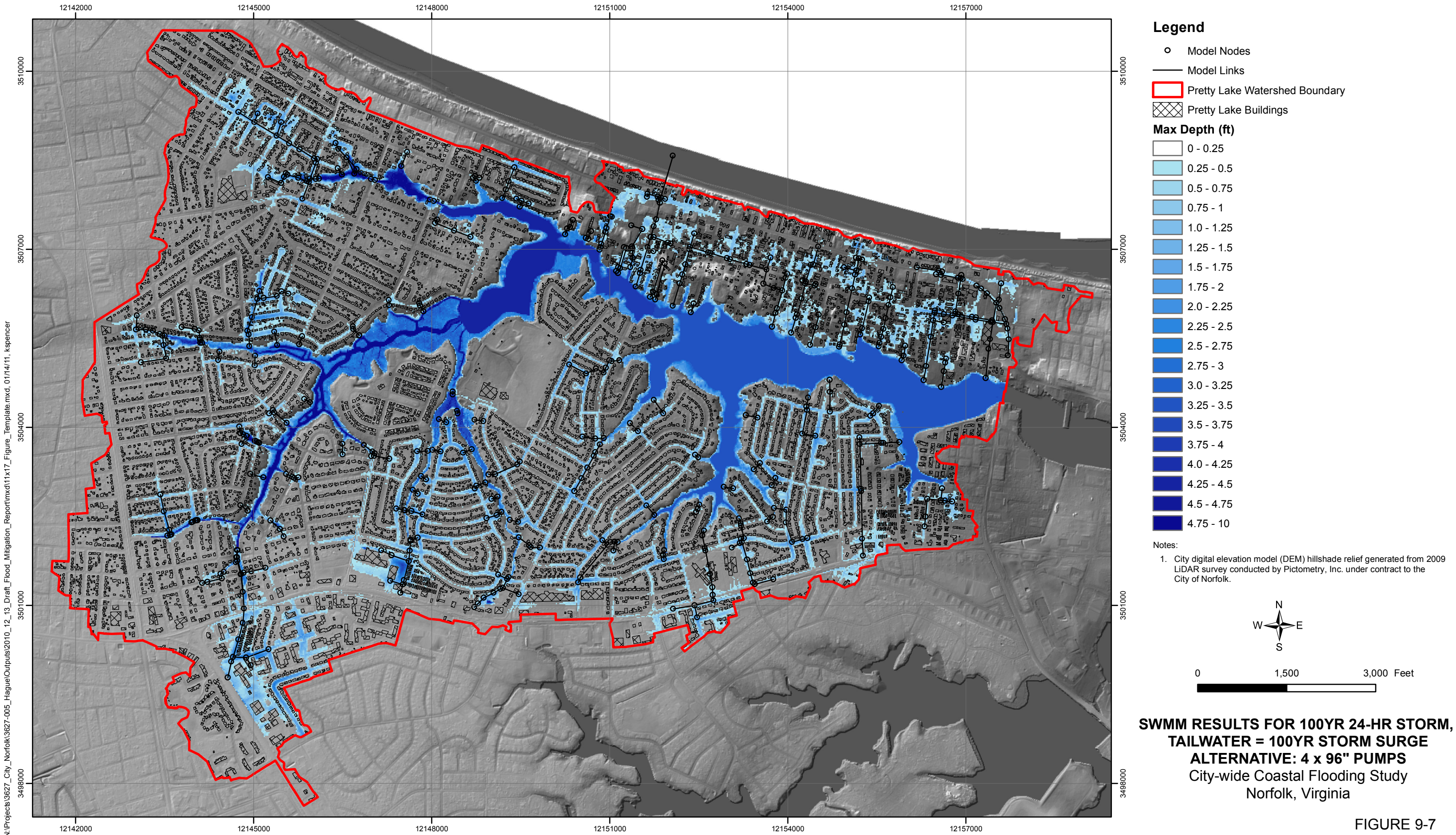


FIGURE 9-7



DRAFT

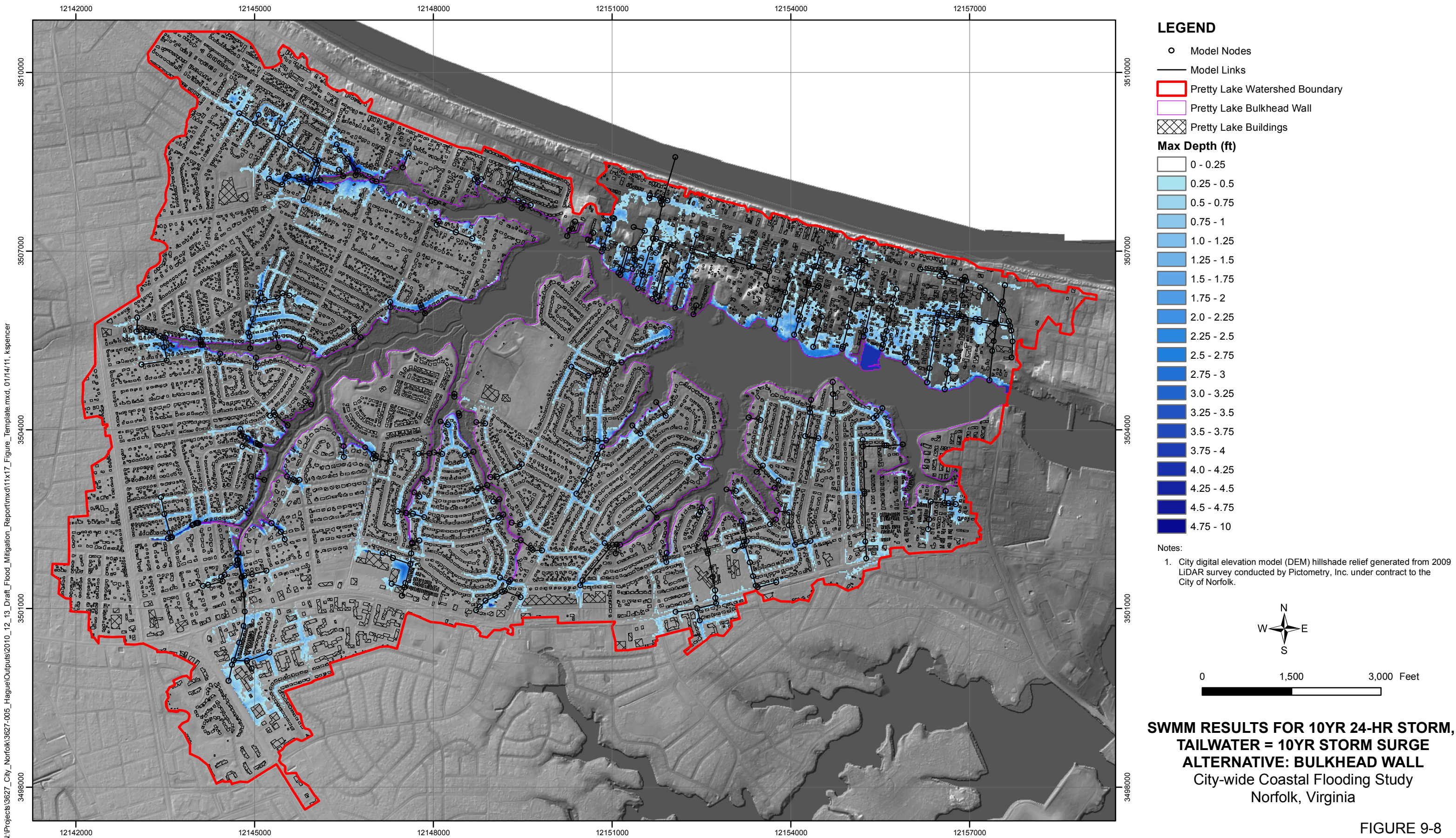


FIGURE 9-8



DRAFT

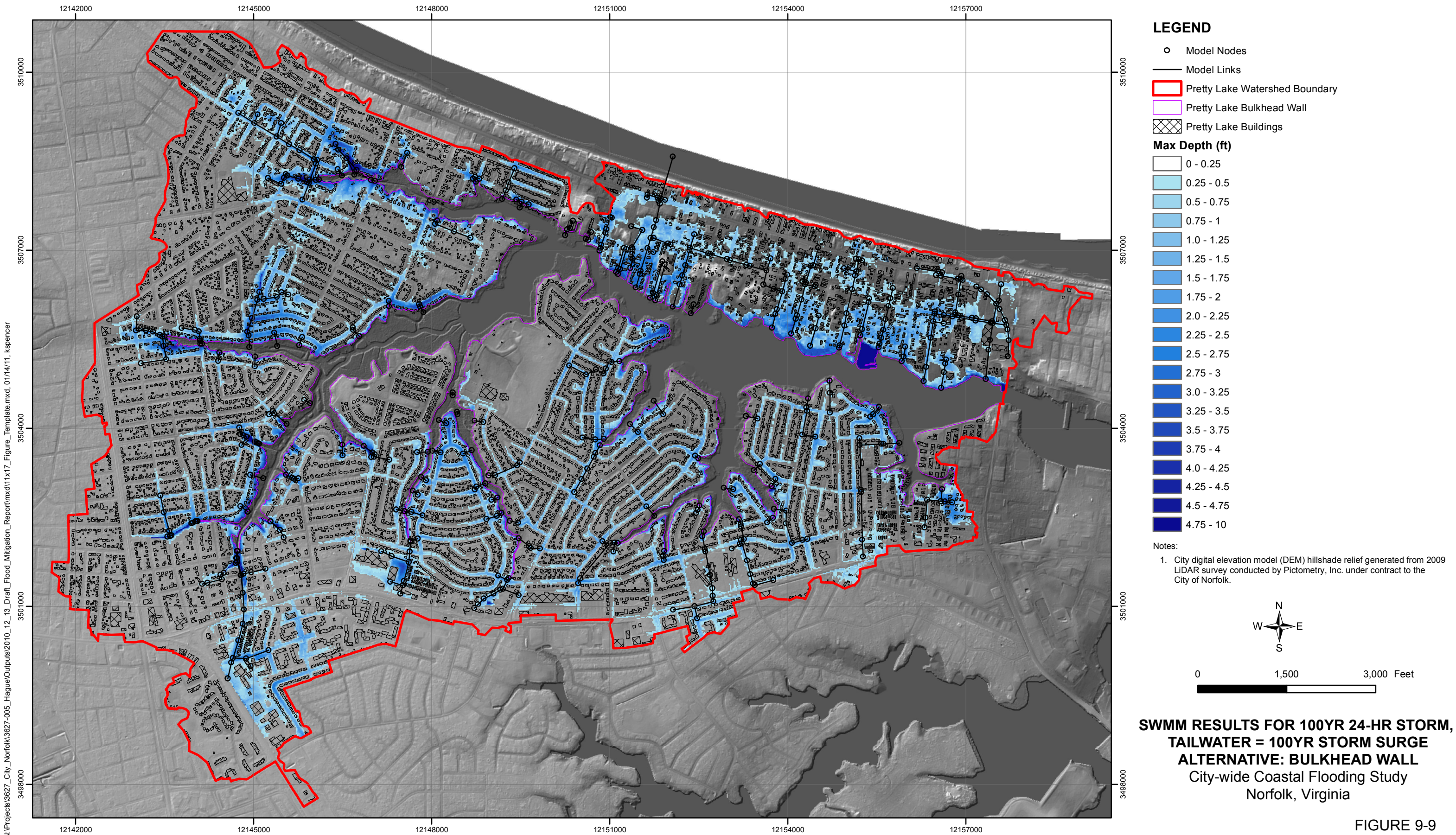
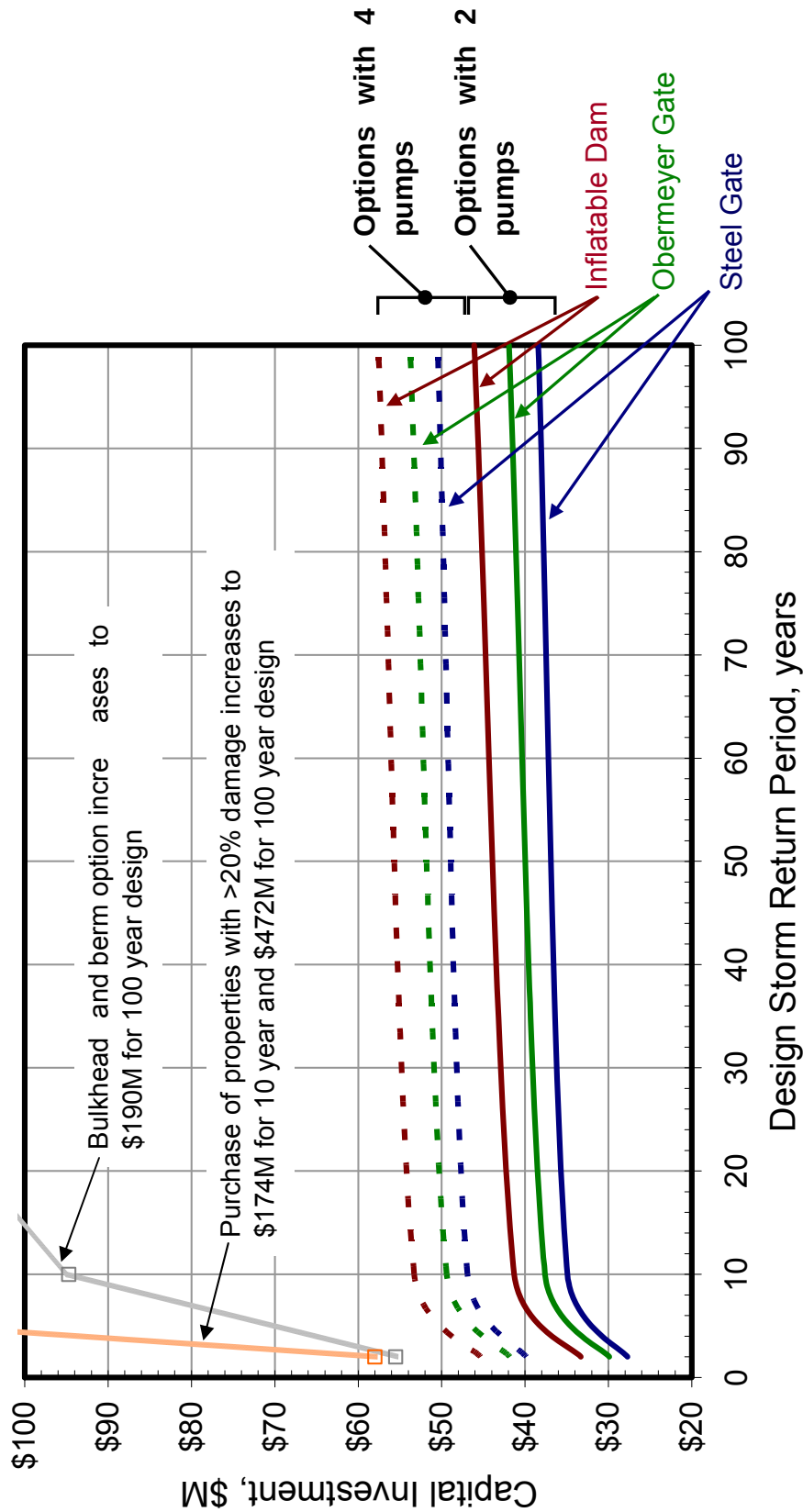


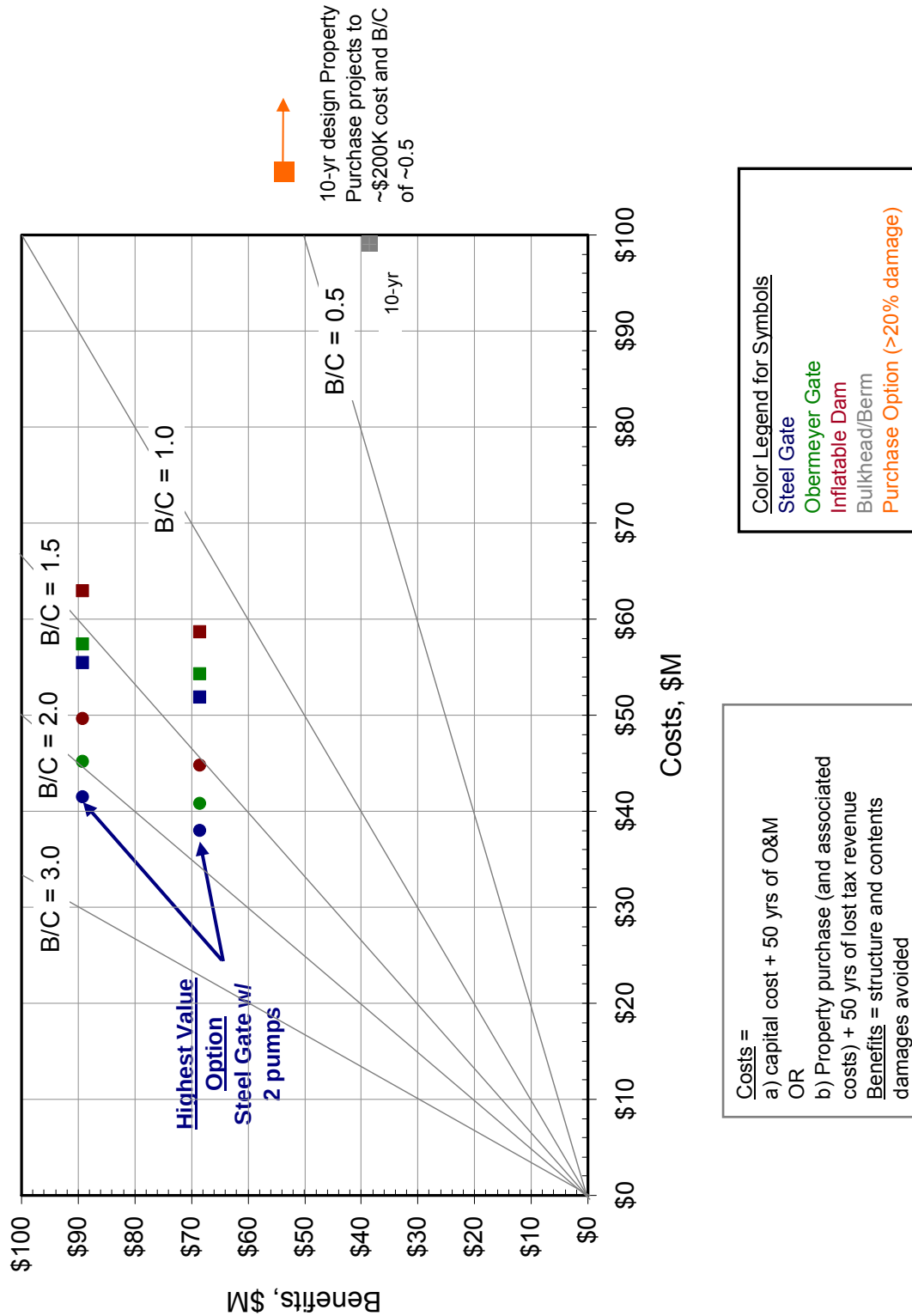
FIGURE 9-9





ESTIMATED PROJECT COSTS FOR FLOOD MITIGATION OPTIONS

City-wide Coastal Flooding Study  
Norfolk, Virginia



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



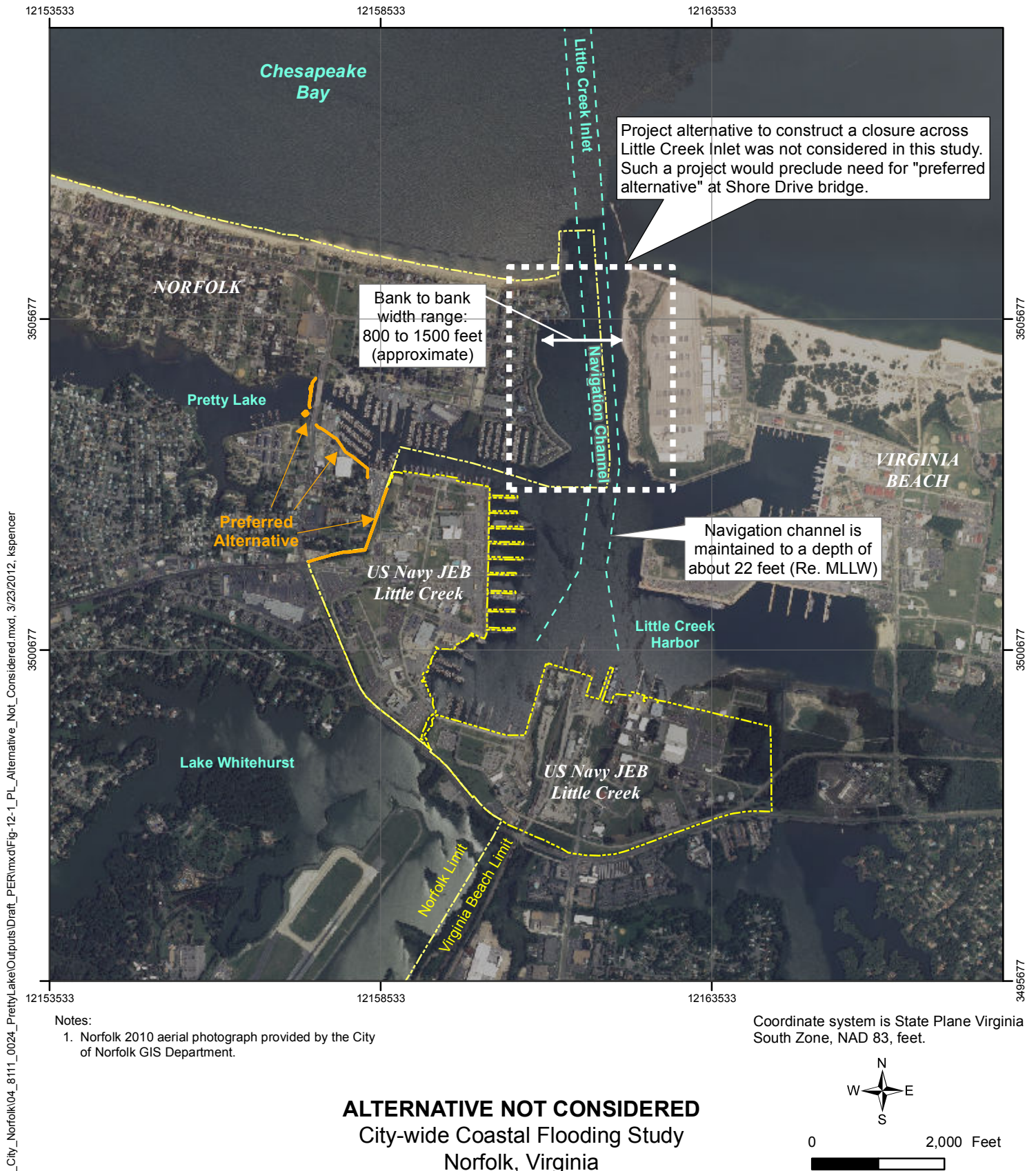


FIGURE 12-1