VIRGINIA ELECTRIC AND POWER COMPANY
RICHMOND, VIRGINIA 23261

March 12, 2015

U.S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, DC 20555

Serial No. 15-107
NL&OS/GDM R4
Docket Nos.: 50-280/281
License Nos.: DPR-32/37

VIRGINIA ELECTRIC AND POWER COMPANY (DOMINION)
SURRY POWER STATION UNITS 1 AND 2
FLOOD HAZARD REEVALUTION REPORT IN RESPONSE TO MARCH 12, 2012 INFORMATION REQUEST REGARDING FLOODING ASPECTS OF RECOMMENDATION 2.1

On March 12, 2012, the Nuclear Regulatory Commission (NRC) issued, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," to all power reactor licensees and holders of construction permits in active or deferred status. Flooding Recommendation 2.1 requires licensees to perform a flood hazard reevaluation and provide a final report documenting the results, as well as pertinent site information and detailed analysis.

For Flooding Recommendation 2.1, Enclosure 2 of the letter states that in accordance with the NRC's prioritization plan, within 1-3 years from the date of the information request, submit the Hazard Reevaluation Report and include the interim action plan. In a letter dated May 11, 2012, the NRC issued their prioritization plan. The response date for Surry Power Station (SPS) Units 1 and 2 flood hazard reevaluation was identified as March 12, 2015, which corresponds to three years from the date of the March 12, 2012 Information Request.

Attachment 1 provides the SPS Flooding Hazard Reevaluation Report. The report is being provided to the Document Control Desk in hard-copy form and to the other letter recipients in electronic form on a compact disc. Attachment 2 provides the SPS interim actions.

If you have any questions regarding this information, please contact Mr. Gary D. Miller at (804) 273-2771.

Sincerely,

Dan Stoddard

Daniel G. Stoddard
Senior Vice President – Nuclear Operations

COMMONWEALTH OF VIRGINIA)
)
COUNTY OF HENRICO)

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Daniel G. Stoddard, who is Senior Vice President – Nuclear Operations of Virginia Electric and Power Company. He has affirmed before me that he is duly authorized to execute and file the foregoing document in behalf of that company, and that the statements in the document are true to the best of his knowledge and belief.

A010
NRR

Acknowledged before me this 12th day of March, 2015.

My Commission Expires: January 31, 2016

Wanda D. Craft
Notary Public

(SEAL)

WANDA D. CRAFT
Notary Public
Commonwealth of Virginia
Reg. # 7520495
My Commission Expires January 31, 2016

Commitments made in this letter: No new regulatory commitments.

Attachments:
1. Flooding Hazard Reevaluation Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Flooding, March 2015
2. Surry NTTF 2.1: Flooding Hazard Reevaluation Interim Actions Plan

cc: U.S. Nuclear Regulatory Commission, Region II
Regional Administrator
Marquis One Tower
245 Peachtree Center Ave. NE
Suite 1200
Atlanta, Georgia 30303-1257

Ms. K. R. Cotton Gross
Project Manager
U.S. Nuclear Regulatory Commission
One White Flint North, Mail Stop 08 G-9A
11555 Rockville Pike
Rockville, MD 20852-2738

Dr. V. Sreenivas
Project Manager
U.S. Nuclear Regulatory Commission
One White Flint North, Mail Stop 08 G-9A
11555 Rockville Pike
Rockville, MD 20852-2738

NRC Senior Resident Inspector
Surry Power Station

Mr. J. E. Reasor, Jr.
Old Dominion Electric Cooperative
Innsbrook Corporate Center, Suite 300
4201 Dominion Blvd.
Glen Allen, Virginia 23060

ATTACHMENT 1

FLOODING HAZARD REEVALUATION REPORT FOR RESOLUTION OF FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

MARCH 2015

VIRGINIA ELECTRIC AND POWER COMPANY (DOMINION)
SURRY POWER STATION UNITS 1 AND 2

ZACHRY

Zachry Nuclear Engineering, Inc.

ENGINEERING EVALUATION
14-E15

DOMINION FLOODING HAZARD REEVALUATION REPORT FOR SURRY POWER STATION UNITS 1 AND 2

IN RESPONSE TO 50.54(F) INFORMATION REQUEST REGARDING NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

REVISION 1

QA CLASSIFICATION: SAFETY RELATED

Prepared by:	Derek H. Andersen	03/11/15 Date
Co-Prepared by:	David M. Leone / Michael A. Mobile	03/11/2015 Date
Reviewed by:	Stephen F. Superson	3/11/15 Date
Co-Reviewed by:	Daniel C. Stapleton / Peter H. Baril	03/11/2015 Date
Approved by:	Addison G. Ricker	3.11.15 Date

Client: Dominion/Surry Power Station
Zachry Nuclear, Inc. Job No. : 6029

REVISION HISTORY

Revision	Revision Description
0	Original Issue: Derek H. Andersen was responsible for preparing the front matter of the EE, Section 1, Section 4, and Section 5. David M. Leone and Michael A. Mobile were responsible for co-preparing Section 2 and Section 3. Stephen F. Superson was the overall responsible reviewer, and in particular reviewed the front matter of the EE, Section 1, Section 3, Section 4, and Section 5. Daniel C. Stapleton and Peter H. Baril were responsible for co-reviewing Section 2 and Section 3. This Engineering Evaluation, while in accordance with Zachry Procedure N0302, Rev. 01, is formatted and presented in such a manner as to be consistent with the expectations of Dominion and the Nuclear Regulatory Commission (NRC). This re-formatting will include header, footer, and page number adjustments that will allow for easy topic recognition while not violating any Zachry branding guidelines. The Engineering Evaluation Verification Form will not be included as an attachment to this document, but will instead be kept in records with this EE, as a separate document.
1	This Engineering Evaluation was revised in order to address follow-up comments. These comments were minor in nature, but provided clarity, were editorial, or corrected cross-references throughout the entire Engineering Evaluation. Parts of the document that were updated are tracked with revision bars on the right side of the document; additionally the entire document was reviewed. The authorship and review of the report is identical to Revision 0. All comments in the Revision 0 Description still apply.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS AND ABBREVIATIONS

Acronym/Abbreviation	Description
ADCIRC	Advanced Circulation Model computer program
AEP	Annual Exceedance Probability
ANS	American Nuclear Society
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASCII	American Standard Code for Information Interchange
AWL	Antecedent Water Level
CDB	Current Design Basis
CDF	Cumulative Density Function
CEM	Coastal Engineering Manual
CFR	Code of Federal Regulations
cfs	cubic feet per second
CLB	Current License Basis
CVV	Cumbre Vieja Volcano
DAD	Depth-Area-Duration
DEM	Digital Elevation Model
DBFL	Design Basis Flood Level
DTM	Digital Terrain Model
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FSAR	Final Safety Analysis Report

Acronym/Abbreviation	Description
ft	feet
GPD	Generalized Pareto Distribution
gpm	Gallons per minute
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
HUC	Hydrologic Unit Code
HURDAT2	Hurricane Database (NOAA)
ISFSI	Independent Spent Fuel Storage Installation
ISG	Interim Staff Guidance (NRC)
JPM	Joint Probability Method
JPM-OS	Joint Probability Method – Optimum Sampling
kt	Knots
LIDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
MHW	Mean High Water
MHHW	Mean High High Water
MLW	Mean Low Water
MLLW	Mean Low Low Water
MSL	Mean Sea Level (Local Site Datum)
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center

Acronym/Abbreviation	Description
NED	National Elevation Dataset
NGDC	National Geophysical Data Center
NGVD29	National Vertical Datum of 1929
NID	National Inventory of Dams
nm	Nautical miles
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTHMP	National Tsunami Hazard Mitigation Program
NTTF	Near-Term Task Force
NWS	National Weather Service
OBE	Operating Basis Earthquake
PA	Protected Area
PDF	Probability Density Function
PDH	Probability Density Histogram
PMF	Probable Maximum Flood
PMH	Probable Maximum Hurricane
PMP	Probable Maximum Precipitation
PMS	Probable Maximum Seiche
PMSS	Probable Maximum Storm Surge
PMT	Probable Maximum Tsunami
PMWE	Probable Maximum Water Elevation
RMSE	Root Mean Square Error

Acronym/Abbreviation	Description
SOCA	Security Owner Controlled Area
SCS	Soil Conservation Service
SLOSH	Sea, Lakes, and Overland Surges from Hurricanes
SLR	Sea Level Rise
SMF	Submarine Mass Failure
SPAS	Storm Precipitation Analysis System
SPS	Surry Power Station
SSCs	Structures, Systems and Components
SSE	Safe Shutdown Earthquake
SWAN	Simulation WAves Nearshore computer program
TAW	Technical Advisory Committee for Water Retaining Structures
UFSAR	Updated Final Safety Analysis Report
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geological Survey
VBS	Vehicle Barrier System
WMO	World Meteorological Organization
WRT	WindRisk Tech LLC

INTRODUCTION

Following the accident at the Fukushima Dai-ichi nuclear power plant, caused by the Great Tōhoku Earthquake and subsequent tsunami that occurred on March 11, 2011, the Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF). The NTTF was tasked with performing a comprehensive review of the NRC processes and regulations to determine if additional measures or improvements were required.

The NTTF concluded that an accident with consequences similar to the Fukushima accident is unlikely to occur in the United States and provided a set of recommendations to the NRC. The NRC directed the staff to determine which of the recommendations should be implemented without unnecessary delay.

In turn, the NRC issued a Request for Information (RFI) pursuant to 10 CFR 50.54(f). Enclosure 2 of this RFI addressed NTTF Recommendation 2.1 and requested a written response from licensees with the following purposes:

- To gather information with respect to NTTF Recommendation 2.1, as amended by staff requirements memoranda (SRM) associated with SECY -11-0124 and SECY -11-0137, and the Consolidated Appropriations Act, for 2012 *(Pub Law* 112-74), Section 402, to reevaluate seismic and flooding hazards at operating reactor sites
- To collect information to facilitate NRC's determination if there is a need to update the design basis and systems, structures, and components (SSCs) important to safety to protect against the updated hazards at operating reactor sites
- To collect information to address Generic Issue (GI) 204 regarding flooding of nuclear power plant sites following upstream dam failures

Surry Power Station (SPS), located in Surry County, Virginia, is one of the sites being required to submit information compliant with this RFI. Surry Power Station consists of two operational units (Units 1 and 2). In response to the RFI this Report is being generated to address NTTF Recommendation 2.1 with respect to Units 1 and 2.

The methodology of this report follows the Hierarchical Hazard Assessment (HHA) approach, as described in NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characteristic at Nuclear Power Plants in the United States of America" and its supporting reference documentation. Any assumptions used in this report are stated and justified in the body of the report.

SPS has adopted the U.S. Coast & Geologic Survey Mean Sea Level (MSL) datum as its reference for elevations. Parts of this report may mention the North American Vertical Datum of 1988 (NAVD 88). The difference in Elevation between these datums at the site is 1.44 ft (NAVD88 0.00 ft = 1.44 ft MSL). Unless specified, this report will refer to all elevations in MSL.

Section 1 of this report provides detailed site information related to flooding hazards. Section 2 includes the reevaluation of flood hazards for each reevaluated flood-causing mechanism. A comparison of the reevaluated flood hazards to those in the current licensing basis is provided in Section 3. Interim flood protection measures and actions for higher flood hazards identified are summarized in Section 4. Additional Actions required are concluded in Section 5. There are three Appendices attached, two of which are discussed in Section 2. The third Appendix, Appendix C, is a third party review of the hurricane/storm surge calculations performed in support of this Evaluation.

Introduction References

1. **NRC 2012**. U.S. Nuclear Regulatory Commission, "Request for information pursuant to Title 10 of the Code of Federal Regulations 50.54(f) regarding Recommendations 2.1, 2.3, and 9.3, of the Near- Term Task Force Review of Insights from the Fukushima Dai-ichi Accident", March 12, 2012

2. **NRC 2011a**. U.S. Nuclear Regulatory Commission, "Recommended Actions to be Taken Without Delay from the Near Term Task Force Report", SECY-11-0124, September 9, 2011

3. **NRC 2011b**. U.S. Nuclear Regulatory Commission, "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned", SECY-11-0137, October 3, 2011

4. **Dominion 2014**. Surry Power Station, "Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)", Rev. 46.02

5. **NRC 2011c**. U.S. Nuclear Regulatory Commission, NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America", November 2011

6. **Dominion 2013**. Project Letter to Michael Kerst from John B. Lee, "SPS Units 1 & 2 Beyond Design Basis (BDB) Project Vertical Datum Conversion Information", dated 07/10/13

1.0 SITE INFORMATION RELATED TO FLOODING HAZARDS

This section provides detailed site information related to flooding hazards as requested in Enclosure 2 of the NRC RFI letter pursuant to Title 10 CFR 50.54(f) dated March 12, 2012. Under this enclosure, Recommendation 2.1 requests the following with respect to site information:

- Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets
- Current design basis flood elevations for all flood causing mechanisms
- Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance
- Changes to the watershed and local area since license issuance
- Current licensing basis flood protection and pertinent flood mitigation features at the site
- Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)

The requested information is presented in Sections 1.1 through 1.6.

1.1. Detailed Site Information

The Surry Power Station is located in Surry County, Virginia, on a point of land called Gravel Neck that juts into the James River from the south, as shown in Figure 1.1-1. The site is at the end of Route 650 and south of and adjacent to the Hog Island State Wildlife Management Area. It is bordered by the James River on either side of the peninsula.

Regionally, the site is 4.5 miles west-north-west of Fort Eustis, 7 miles south of Colonial Williamsburg, and 8 miles east-north-east of the town of Surry. Jamestown Island, part of the Colonial National Historical Park, is to the northwest on the northern shore of the James River. The area within 10 miles of the site covers parts of Surry, Isle of Wight, York, and James City Counties, and parts of the cities of Newport News and Williamsburg. Surry and Isle of Wight Counties are predominantly rural and characterized by farmland, wood tracts of land, and marshy wetlands. York and James City Counties and the cities of Newport News and Williamsburg are more urban and are characterized by recreational areas and growing population centers. The region 10 to 30 miles east and southeast of the site is comprised of the Hampton, Newport News, Norfolk, and Portsmouth, Virginia urban areas. Additionally, the site is 44 miles southeast of Richmond, Virginia and approximately 40 miles west of the Atlantic Ocean.

The plant site comprises approximately 830 acres. The plant property lines, which are the same as the site boundary lines, are shown on Figure 1.1-2. Virginia Electric and Power Company (Virginia Power), owns, in fee simple, all of the land within the site boundary, both above and beneath the surface, with the exception of state route 650, which passes through the site to the Hog Island State Wildlife Management Area to the north. The Plot Plan for Surry is shown on Figure 1.1-3 and the site topography is shown in Figures 1.1-4 and 1.1-5.

The ground surface at the site is generally flat, with steep banks sloping down to the river and to the low-level waterfowl refuge to the north. Station ground grade has been established at an elevation of 26.5 feet MSL at Hampton Roads, Virginia. Beyond the site boundaries, maximum land elevations within a 5-mile radius are generally in the range of 40 to 60 feet. Much of the region is characterized by marshes, extensive swamps, small streams, and pocosins. Water tables are very near to the surface throughout the entire area, accounting for the large amount of surface waters. Drainage throughout the area is toward Hampton Roads, on the Atlantic Ocean and near the mouth of Chesapeake Bay.

The Power Block buildings which house SSCs are located at the general site grade of 26.5 ft MSL. Within these buildings the SSCs are located at various elevations. For further information for SSC elevations refer to Table 1.5-1. This table discusses the current flood protection levels, which is primarily provided by site grade.

The James River is formed by the junction of the Cowpasture and Jackson Rivers in Botetourt County, Virginia, and flows easterly 340 miles before emptying into Hampton Roads at Newport News, Virginia.

The flow of water in the James River at the site consists of three components:

- Fresh water discharge from the James River watershed
- Flow due to the oscillatory ebb and flood of the tide

- FLOW due to the circulation pattern caused by intrusion of saline water within the estuary

The James River watershed at Surry has an area of 9,521 square miles. This watershed consists of three subbasins as shown in Figure 2.2-1. The mean monthly discharge of the James River at the station site is averaged at 10,229 cfs, based on mean monthly data between October 1935 and September of 1993.

The tide in the James River is a semidiurnal tide, with two high waters and two low waters each lunar day of 24.84 hours. The oscillatory ebb and flood of this tide constitute the dominant motion in the waterway in the vicinity of the site. The volume rate of flow associated with net non-tidal circulation pattern caused by intrusion of saline water within the estuary, while small compared to the oscillatory tidal flows, is several-fold larger than the volume rate of river discharge.

1.1.1. References

1.1.1-1 Dominion 2014. Surry Power Station, "Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)", Rev. 46.02

1.1.1-2 ESRI 2009. "USA Topo Maps", December 2009

1.1.1-3 Zachry 2014a. Zachry Calculation 13-009 "Probable Maximum Precipitation (PMP) at Surry Power Station", Rev. 0

1.1.1-4 Zachry 2014b. Zachry Calculation 13-010 "Local Intense Precipitation – Generated Flood Flow and Elevations at Surry Power Station", Rev. 0

1.1.1-5 McKim & Creed 2012. Surry Nuclear Generating Station Aerial Mapping Validation Report (Native AutoCAD files), 12/1/12

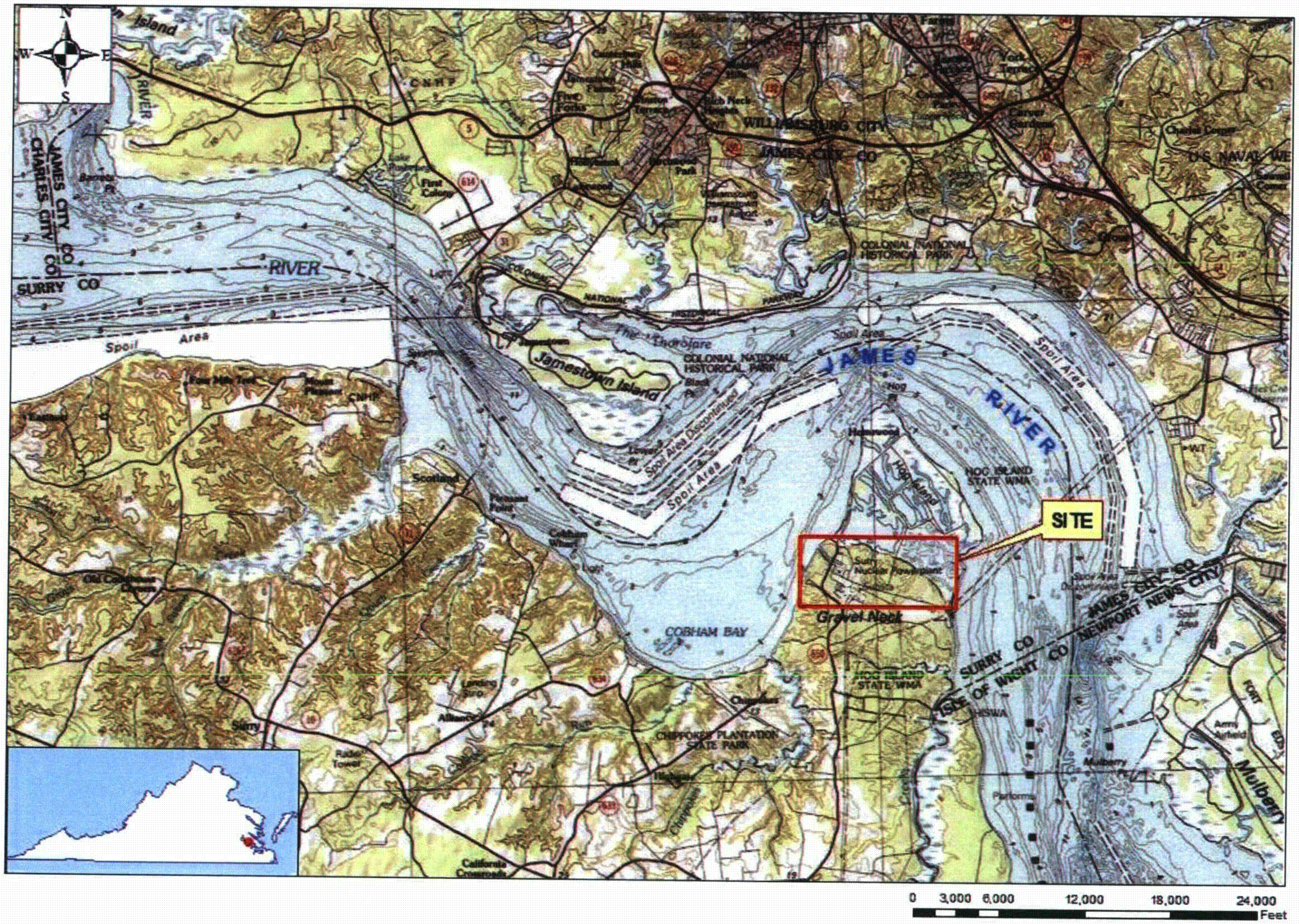


Figure 1.1-1: Site Locus Map (ESRI 2009)

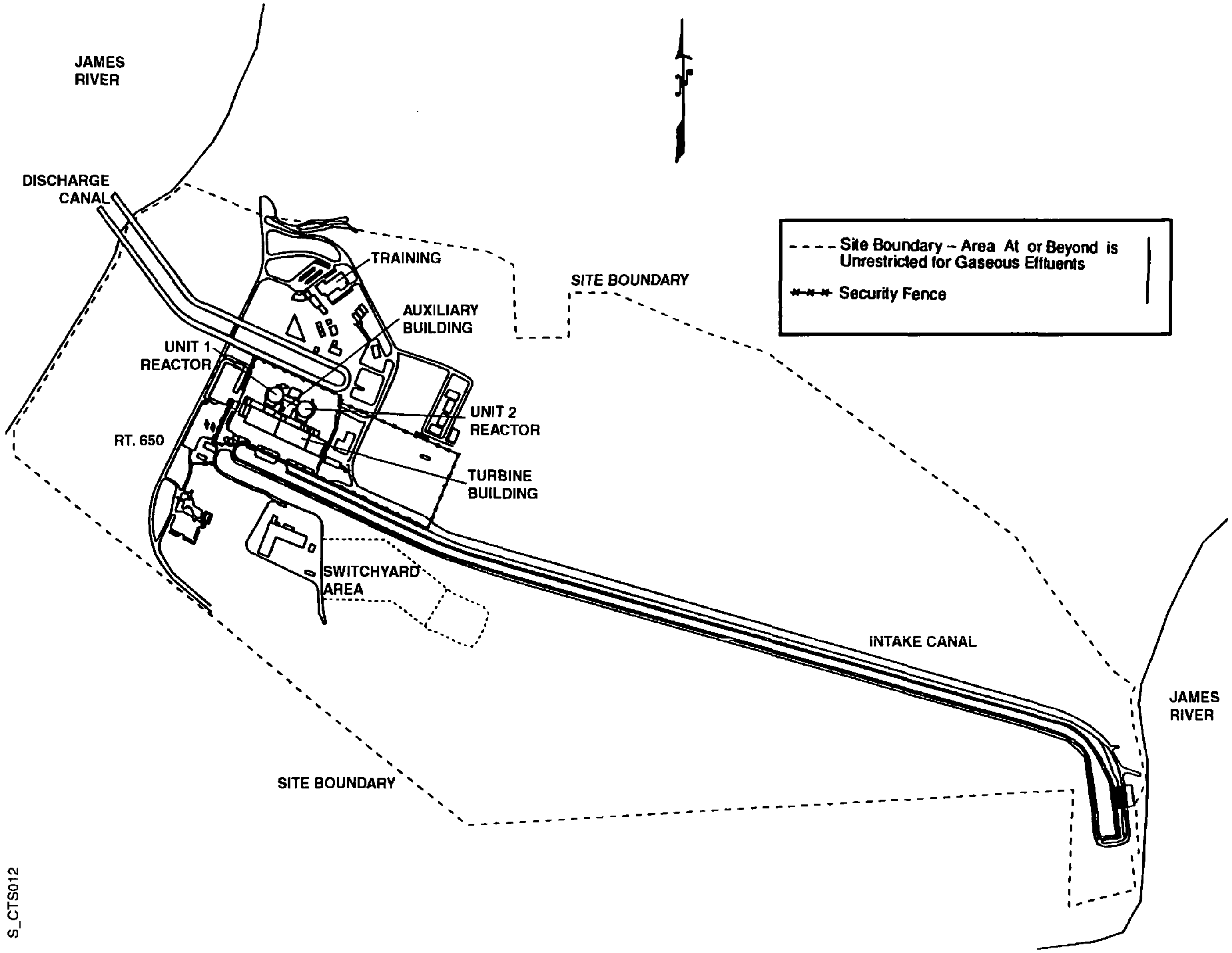
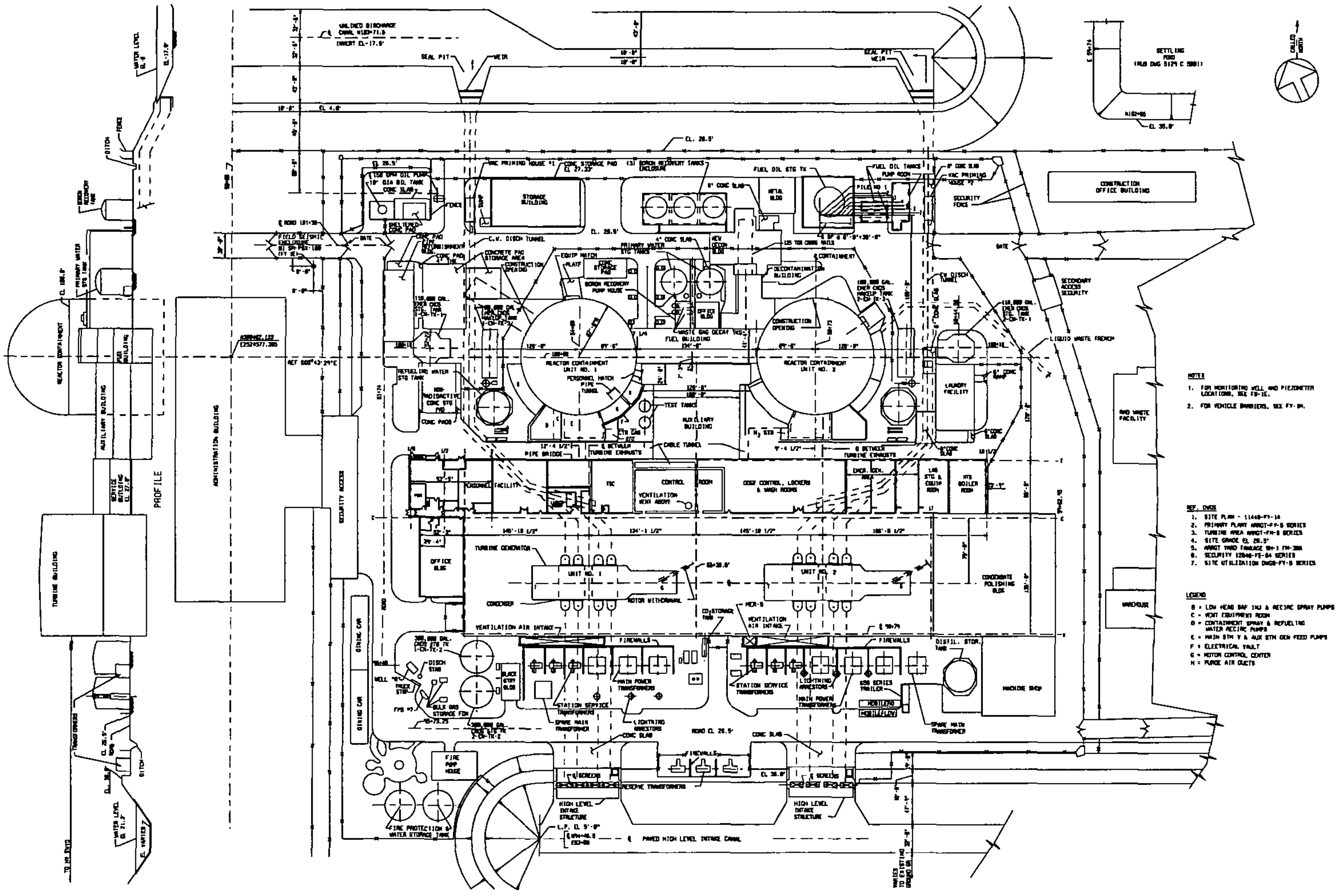


Figure 1.1-2: SITE BOUNDARY AND MAJOR STRUCTURES (Dominion 2014)



C:\usr\dgn\S1501002DGNmf1

Figure 1.1-3: PLOT PLAN (Dominion 2014)

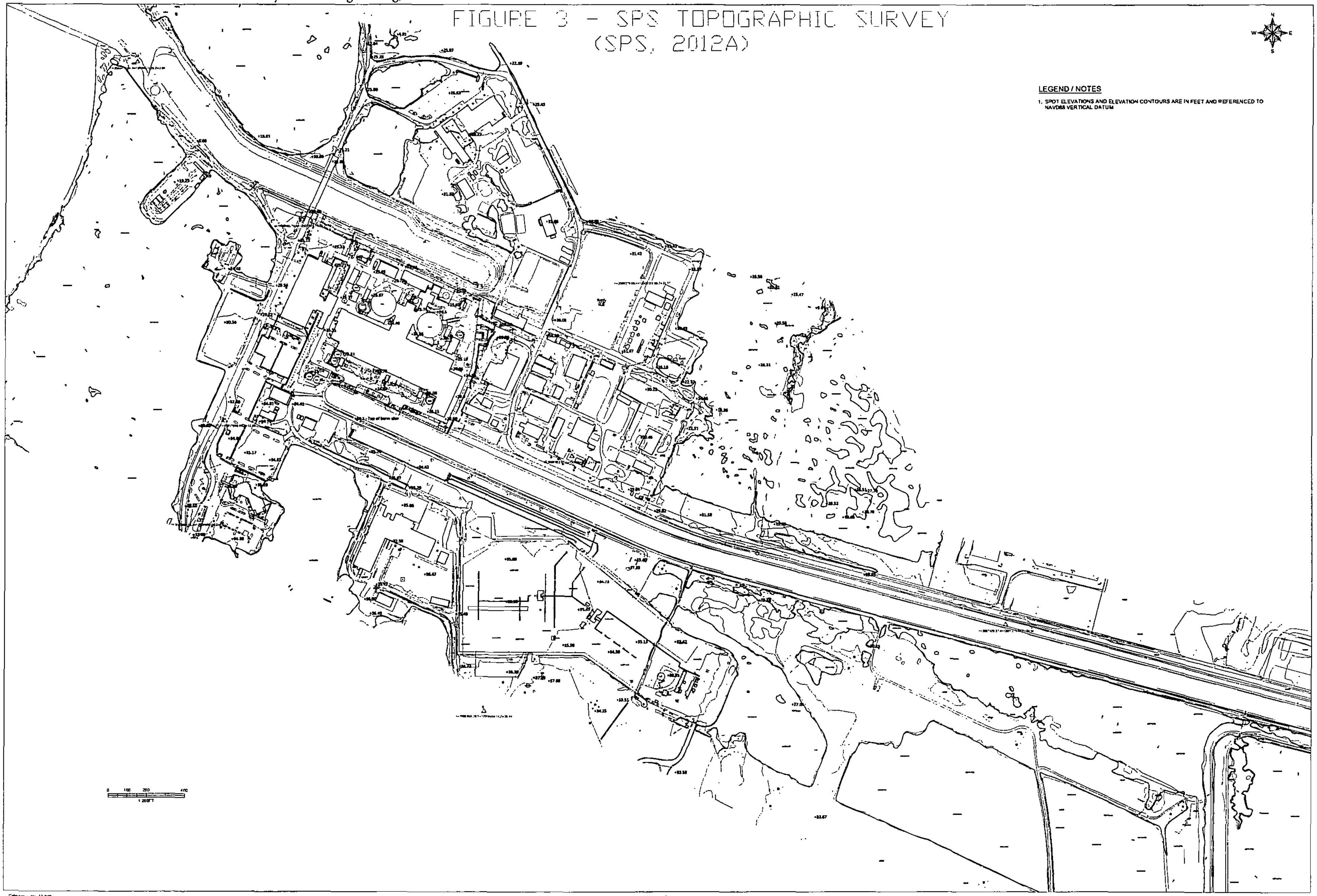


Figure 1.1-4: Site Topography (Zachry 2014b)

Figure 1.1-5: Site Topography Near the Low Level Intake Structure (McKim & Creed 2012)

1.2. Current Design Basis Flood Elevations

As discussed in the Flooding Walkdowns Results Report, related to NTTF Recommendation 2.3; both Units at Surry were licensed to the draft General Design Criterion (GDC) 2 dated July 10, 1967. This met the intent of Appendix A to 10 CFR Part 50, GDC 2. As a result, the draft wording (not current) is the following:

> Those systems and components of reactor facilities that are essential to the prevention of accidents that could affect the public health and safety or to the mitigation of their consequences are designed, fabricated, and erected in accordance with performance standards that enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, ice, and other local site effects. The design bases so established reflect:
>
> 1) Appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and the surrounding area, and
>
> 2) An appropriate margin for withstanding forces greater than those recorded, in view of uncertainties about the historical data and their suitability as a basis for design.

In accordance with this, the Current Design/Licensing Basis flood elevations for the different flood causing mechanisms are addressed in the SPS UFSAR, Chapter 2, Section 2.3.1.2. Based on this section, there are two applicable events that could cause external flooding at Surry. The first is James River Flooding due to watershed runoff and/or Surge due to severe storms. The second is the Probable Maximum Hurricane (PMH) surge (including effects on the intake structure and intake canal), which is generated from the HUR 7-97 guidance document, listed in the UFSAR. In addition to the flood causing events identified, the UFSAR discusses precipitation on site, but does not explicitly address it as a flooding concern.

A statistical analysis of river discharge data is performed based on James River discharge data from 1935 to 1993. The analysis concluded that a flood with a recurrence interval of 1 in 50 years results in a river water level rise of no more than 1 foot above normal mean river level. Based on this the CLB qualitatively concludes that James River Flooding is not a concern at the site. The flood elevations due to PMH, along with other flood elevations requested by the NRC in the Current Licensing Basis (CLB) are summarized in Table 1.2-1.

1.2.1. References

1.2.1-1 Dominion 2012. Virginia Electric and Power Company, Surry Power Station Units 1 and 2 "Flooding Walkdowns Results Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", November 2012

1.2.1-2 Dominion 2014. Surry Power Station, "Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)", Rev. 46.02

1.2.1-3 Dominion 1994. Virginia Electric and Power Company, Surry Power Station Units 1 and 2 "Individual Plant Examinations of Non-Seismic External Events and Fires" December 1994

Table 1.2-1: Current Design Basis Flood Elevations

Flooding Mechanism	Design Basis Flood Level (MSL)	UFSAR Section
Storm Surge (including wave-runup)	28.6 ft (East Side)	2.3.1.2.2
Storm Surge (including wave-runup)	24.0 ft (West Side)	2.3.1.2.2
Local Intense Precipitation[1]	No flooding expected	2.3.1.2
Flooding in Streams and Rivers	No flooding expected	2.3.1.2
Dam Failure[1,2]	No flooding expected	2.3.1.2
Seiche[1]	No flooding expected	2.3.1.2
Tsunami[1]	No flooding expected	2.3.1.2
Ice Induced Flooding[3]	No flooding expected	2.3.1.2.1
Channel Migration or Diversion[1]	No flooding expected	2.3.1.2

Notes:
1. These flooding events are not addressed in section 2.3.1.2 of the UFSAR, therefore it is inferred that no flooding is expected from these hazards.
2. Dam Failure considers both upstream dam failure scenarios, as well as onsite dam failure (The Intake canal in this scenario).
3. Section 2.3.1.2 of the UFSAR qualitatively precludes flooding due to downstream ice formations, but does not address upstream ice dams. Therefore it is inferred that no flooding is expected from this hazard.

1.3. Flood-Related Changes to the Licensing Basis and Any Flood Protection Changes (Including Mitigation)

There has been no change to the CLB flooding elevations beyond what is described in the SPS UFSAR. The Flooding Walkdowns Results Report identified multiple minor deficiencies with the current flood protection at the site. These deficiencies included material condition (most identified deficiencies), a few missing conduit seals and the condition of the storm drains. These deficiencies were addressed by Condition Reports (CRs) and if applicable were entered into the Corrective Action Process (CAP). Other flood protection features were found available, functional, and fairly well maintained. Any new flood protection measures planned will be addressed by the licensee after review of this Flooding Hazard Reevaluation Report.

1.3.1. References

1.3.1-1 Dominion 2014. Surry Power Station, "Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)", Rev. 46.02

1.3.1-2 Dominion 2012. Virginia Electric and Power Company, Surry Power Station Units 1 and 2 "Flooding Walkdowns Results Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", November 2012

1.4. Changes to the Watershed and Local Area

The James River watershed at Surry is discussed in Section 1.1 and in more detail in the SPS UFSAR. There are no known or planned river control structures on the James River. Several small impoundments on tributaries in the upper reaches of the river do exist; however, their size and location would preclude any effect or danger to the safety-related structures at the site. There have been no changes to the watershed in the vicinity of the site that impact flood hazards.

The local area is discussed in Section 1.1. In addition to the regional information, it is important to note that Hampton, Newport News, Norfolk, and Portsmouth, Virginia all comprise areas that are a part of a major Atlantic Coast seaport, a U.S. naval base, and have the largest industry in shipbuilding. While there have been some inevitable small local area changes due to population growth, there have been no site changes pertinent to flood hazards.

1.4.1. References

1.4.1-1 Dominion 2014. Surry Power Station, "Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)", Rev. 46.02

1.5. Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

As discussed in Section 1.2 there are two events considered that could cause external flooding in the CLB. The first is Probable Maximum Flood (PMF) of the James River due to watershed runoff and/or Surge due to severe storms. The second is the Probable Maximum Hurricane (PMH) surge (including effects on the intake structure and intake canal). In response to these flooding events the CLB addresses different flood protection and mitigation features available. Table 1.5-1 provides a list of Maximum Probable Flood Protection Levels for Class 1 Structures and Table 1.5-2 summarizes flood protection/mitigation features.

Based on the CLB, the PMF event results in only a one foot rise in the normal river level and is therefore bounded by the more limiting PMH. There are no specific flood protection and/or mitigation features considered for this event because the PMF does not produce any limiting flood levels for any of the Class I structures.

The PMH Stillwater level for Surry is calculated to be 22.7 feet. The PMH will cause a wave runup elevation of 28.6 feet on the east side of the plant and 24.0 feet on the west side of the plant. The plant configuration (modes of operation) is not specifically discussed in the CLB and therefore it would be assumed that the function of flood protection from this event would be applicable to all modes of plant operation.

1.5.1. Specific Flood Protection and Mitigation Features (Summarized from the Flooding Walkdowns Results Report)

Site (Power Block)

As described in the CLB, the typical site grade at the SPS power block is at 26.5 feet, which is above the wave runup elevation on the west side of the plant. Therefore critical equipment in this area is protected by site grade.

Intake Structure

The intake structure is located approximately 1.25 miles east of the main plant buildings (the power block). The emergency service water pumping equipment (pumps, diesel-driven pump motors, fuel oil tanks, etc.) is housed in a reinforced concrete structure above the deck of the circulating water intake structure. The floor and walls of the emergency service water pump house (ESPH) are watertight. The sill of the ESPH door and air intake louver openings are located at Elevation 21 ft. 2 in. During a short duration of time in the PMH, the Stillwater level is expected to be above this elevation, leading to the possibility of surging water entering into the ESPH.

To limit a buildup of water in the ESPH, the air intake louvers are equipped with exterior covers which, when installed, limit water ingress into the ESPH. The exterior covers on these louvers prevent surging water from overtopping the watertight wells, which were constructed inside the louvers inside the ESPH for additional flood protection. These watertight wells provide protection up to 24 ft. MSL. For both ESPH doors and the intake louver openings, the corresponding seal plates (flood gates) and exterior covers, respectively, are required to be installed whenever hurricane conditions exist, or are forecasted to exist, to preclude significant water ingress. The seal plates provide protection up to 24 ft. MSL. The door seal plates and louver opening covers are procedurally installed.

With the normal air intake louvers covered, air for operation of the diesel-driven emergency service water pumps would be provided through the motor-operated dampers located in the top of the ESPH structure. The positioning of these dampers under the exhaust hood precludes any significant water entry into the ESPH from wave overtopping or runup on the structure. The elevation of the exhaust centerline is 36.5 ft. This is sufficiently above mean sea level to prevent flooding. It is possible that occasional waves may cause splash and spray up the walls of the structure. These storm conditions would not affect the integrity of the intake structure, as the roof is watertight and the exhaust outlet is at an elevation above wave generated splash, spray or flow and is configured to prevent rainwater flow into the exhaust in such an event.

Intake Canal

The normal water elevation at the power station end of the canal will vary between 26.0 feet and 30.0 feet. A minimum freeboard of greater than four feet is maintained between the canal water surface and the berm at 36.0 feet during hurricane flooding of the river thus providing a flood mitigation feature (Therefore the maximum intake canal level is 32 ft MSL). This freeboard is procedurally maintained and is adequate to contain surges within the canal.

Site Protection against a Precipitation Event

The SPS site grading and the storm drains are the flood protection features for a maximum precipitation event. These flood protection features are part of the design bases of SPS but are not specifically discussed in the UFSAR. Although not explicitly discussed in the SPS UFSAR, walls, doors, and roofs of certain buildings are also understood to protect site equipment from the direct effect of the precipitation event.

1.5.2. Weather Conditions or Flood Levels that Trigger Procedures and Associated Actions for Providing Flood Protection and Mitigation (Summarized from the Flooding Walkdowns Results Report)

SPS Site-Specific Abnormal Operations Procedure

SPS Procedure 0-AP-37.01, *Abnormal Weather Condition Preparation* is a site-specific Operations department procedure that is initiated when certain abnormal environmental conditions are expected at the site, including flooding and hurricane winds. Prior to arrival of the hurricane, this procedure requires the start of hurricane preparations such as closing doors, putting flood protection barriers in place, and preparing equipment required for shutdown. Both ESPH doors are required to be closed. The doors and the corresponding seal plates, and intake louver exterior covers, are required to be closed or installed whenever hurricane conditions exist, or are forecast to exist, which would require their use to preclude significant water ingress. Part 8 of this procedure directs SPS Maintenance to initiate Procedure GMP-031, *Emergency Service Water (ESW) Pump House Stop Log Installation and Removal,* to install the pump house door seal plates (stop logs) to the two flood protection doors and install the intake damper air louver covers which are also flood protection features.

Procedure 0-AP-37.01 also confirms the intake canal water elevation to be between 28 feet and 30 feet which ensures the CLB minimum freeboard of greater than four feet is maintained between the canal water surface and the berm elevation of 36.0 feet.

Procedure OP-ZZ-021, *Severe Weather Preparation,* confirms that the ESW pump house doors are closed and checks that all manholes are returned to functional condition.

Corporate Hurricane Response Plan

Dominion corporate hurricane response plan CO-PROC-OOO-HRP-NUCLEAR, *Hurricane Response Plan (Nuclear)*, provides a corporate-level assessment of station operational status and delineation of corporate responsibilities and support staff requirements. The plan provides for an assessment of pre-storm preparedness and implementation of associated contingency activities. The plan also establishes post-storm guidelines, and addresses emergency staffing in terms of management, supervision and support personnel. The plan is intended to provide general guidelines for management to prepare for and recover from a hurricane. The plan contains activity checklists developed to expedite preparations for impending severe weather, as well as post-storm response actions. A management decision to implement the plan would be made when the projected onsite arrival time of hurricane force winds is greater than 36 hours.

1.5.3. References

1.5.3-1 Dominion 2012. Virginia Electric and Power Company, Surry Power Station Units 1 and 2 “Flooding Walkdowns Results Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding”, November 2012

1.5.3-2 Dominion 2014a. Surry Power Station, “Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR)”, Rev. 46.02

1.5.3-3 Dominion 2014b. SPS Procedure 0-AP-37.01, “Abnormal Environmental Conditions”, Revision 64

1.5.3-4 Dominion 2014c. SPS Procedure GMP-031, “Emergency Service Water (ESW) Pump House Stop Log Installation and Removal”, Revision 3

1.5.3-5 Dominion 2014d. SPS Procedure OP-ZZ-021, “Severe Weather Preparation”, Revision 8

1.5.3-6 Dominion 2014e. Dominion Procedure, CO-PROC-OOO-HRP-NUCLEAR, “Hurricane Response Plan (Nuclear)”, Revision 12

Table 1.5-1: Maximum-Probable-Flood Protection Levels for Class I Structures

Class I Structure	Flood Protection Level (ft – MSL)
Containment structure	26.5
Cable vault and cable tunnel	26.5
Pipe tunnel between containment and auxiliary building	26.5
Main steam and feedwater isolation valve cubicle	27.5
Recirculation spray and low-head safety injection pump cubicle	26.5
Safeguards ventilation room	26.5
Auxiliary building	26.5
Fuel building	26.5
Control room	27.0
Emergency switchgear and relay room	26.5
Relay room	26.5
Battery room	26.5
Air-conditioning equipment room	26.5
Reactor trip breaker cubicle	45.25
Emergency diesel-generator room	26.5
Circulating water intake structure (emergency service water pump house)	24.0
High-level intake structure	36.0
Seal pit	Not Applicable

Table 1.5-2: Flood Protection/Mitigation Features

Event	Location/Structure	Feature	Feature Notes
PMF	Site (Power Block)	Power block grade Elevation 26.5'	Passive feature
PMF	Intake Structure	Intake shoreline grade	Passive feature
PMH	Site (Power Block)	Power block grade Elevation 26.5'	Passive feature
PMH	Intake Structure	Water-tight walls and roof	Passive feature
PMH	Intake Structure	Air intake louver covers and wells	Procedurally installed feature
PMH	Intake Structure	Flood gates at doors	Procedurally installed feature
PMH	Intake Structure	Exhaust elevation and configuration	Passive feature
PMH	Intake Structure	Alternate air intake position	Passive feature
PMH	Intake Structure	Freeboard (minimum 4')	Procedurally maintained
Precipitation	Site (Power Block)	Walls, doors and site grading	Passive feature
Precipitation	Site (Power Block)	Storm drains	Passive feature

1.6. Additional Site Details

Additional site details, such as bathymetry, will be provided as required in Section 2.

2.0 FLOODING HAZARD REEVALUATION

This section provides a reevaluation of the flood hazards for each flood causing mechanism identified in Enclosure 2 of the NRC RFI letter pursuant to Title 10 CFR 50.54(f) dated March 12, 2012. Under this enclosure, Recommendation 2.1 requests that an analysis be performed for each of the following flood causing mechanisms:

- Local intense precipitation and site drainage
- Flooding in streams and rivers
- Dam breaches and failures
- Storm surge and seiche
- Tsunami
- Ice-induced flooding
- Channel migration or diversion
- Combined effects

Mechanisms that are not applicable to the site will be screened out and justified. A basis will be provided for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.

The requested information is presented in sections 2.1 through 2.9. The information presented in these sections will be a summary of the following Zachry Calculations:

- Zachry Calculation 13-009, Rev. 0, "Probable Maximum Precipitation (PMP) at Surry Power Station"
- Zachry Calculation 13-010, Rev. 0, "Local Intense Precipitation – Generated Flood Flow and Elevations at Surry Power Station"
- Zachry Calculation 13-011, Rev. 0, "Probable Maximum Flood (PMF) at Surry Power Station"
- Zachry Calculation 13-025, Rev. 0, "Probable Maximum Seiche (PMS) at Surry Power Station"
- Zachry Calculation 13-028, Rev. 0, "Evaluation of Channel Migration/Diversion at Surry Power Station"
- Zachry Calculation 13-075, Rev. 0, "Evaluation of Ice Effects at Surry Power Station"
- Zachry Calculation 13-131, Rev. 0, "Probable Maximum Tsunami (PMT) at Surry Power Station (Regional and Site Screening Analysis)"
- Zachry Calculation 13-143, Rev. 0, "Evaluation of Dam Failures for Surry Power Station"

- Zachry Calculation 14-028, Rev. 0, "Probable Maximum Hurricane for Surry Power Station"
- Zachry Calculation 14-047, Rev. 0, "Detailed Tsunami Modeling for Surry Power Station (SPS)"
- Zachry Calculation 14-116, Rev. 0, "Deterministic Probable Maximum Storm Surge for Surry Power Station"
- Zachry Calculation 14-117, Rev. 0, "Probabilistic Storm Surge for Surry Power Station"
- Zachry Calculation 14-207, Rev. 0, "Site Specific Probable Maximum Precipitation (PMP) at Surry Power Station"
- Zachry Calculation 14-221, Rev. 0, "Local Intense Precipitation Flooding Using Site Specific Precipitation Information – Surry Power Station"
- Zachry Calculation 14-224, Rev. 0, "Combined Effect Flood Analysis at Surry Power Station"

2.1. Local Intense Precipitation

This section summarizes the evaluation of flooding at SPS due to the Local Intense Precipitation (LIP) event. The Local Intense Precipitation is the Probable Maximum Precipitation (PMP) centered over the site area and the local watershed.

All elevations in this section refer to the SPS vertical datum, mean sea level (MSL), unless otherwise noted. The elevations in the topographic survey (McKim & Creed, 2012a), which was an input to this calculation, are referenced to the North American Vertical Datum of 1988 (NAVD88). To convert NAVD88 to MSL at SPS, add 1.44 ft to the NAVD88 elevation (SPS, 2013a).

2.1.1 Site Description

SPS is located on Gravel Neck, adjacent to the James River in Surry County, Virginia. A general locus map is presented in Figure 2.1-1. SPS is a two-unit facility bordered on the east and west sides by the James River, and on the south and north sides by the Intake and Discharge Canals respectively. SPS Unit 1 is on the west side of the site and SPS Unit 2 is on the east side of the site.

Average site elevation around SPS Unit 1 and Unit 2 buildings is 26.5 feet MSL (SPS, 2014). The local site configuration is shown in Figure 2.1-2. Site grades are relatively flat and drainage is normally accomplished through a system of catch basins and underground storm drains. Surface drainage is constricted by the Intake Canal and perimeter security barriers to the east and west of Units 1 and 2. The Intake Canal is located to the south of the buildings and partially impounded by an earthen berm with a concrete liner and a top elevation of 36.0 feet MSL. The Intake Canal water surface elevation is maintained between elevations 26 and 30 feet MSL. The Intake Canal has a small drainage area and is not directly connected to the James River. The Discharge Canal is located north of the buildings. The Discharge Canal Control Structure is an uncontrolled structure that contains water, conservatively considered at a minimum elevation of 6 feet MSL in the Discharge Canal. Concrete security barriers on the west and east of Units 1 and 2 are generally about 2.7 feet high with minimal openings for runoff flow conveyance, except at vehicle and pedestrian openings.

2.1.2 Method

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC, 2011, Section 2) was used for the evaluation of the LIP and resultant water surface elevation at SPS. Due to anticipated unconfined flow characteristics, a two-dimensional hydrodynamic computer model, FLO-2D (FLO-2D, 2014a), was used for this calculation. A technical description of the FLO-2D model is included in Appendix A.

The HHA for LIP used the following steps:

1. Define FLO-2D model limits for LIP analysis.
2. Develop the FLO-2D computer model with site features.
3. Develop LIP/PMP inputs:

a. Using the National Oceanic and Atmospheric Administration (NOAA) and U.S. Army Corps of Engineers (USACE) Hydrometeorological Report Nos. 51 (HMR-51) and 52 (HMR-52);

b. Refining generic HMR-51 and HMR-52 rainfall values through a site-specific meteorological study at SPS: The PMP values provided in HMR-51 for SPS provide values starting at 6-hours and 10-square-miles. There are no explicit values provided at the 1- and 6-hour durations for 1-square-mile. HMR-52 provides information to derive the 1-hour 1- and 10-mi2 values based on HMR-51 6-hour 10-square-mile PMP. Unfortunately, the most recent storm evaluated in HMR 51 occurred in 1972. In addition, because HMR-51 and 52 cover large domains, generalization and conservatism were employed in the development of their respective PMP values that do not necessarily reflect the site-specific characteristics of SPS.

4. Perform flood simulations in FLO-2D and estimate maximum water surface elevations at SPS.

NUREG/CR-7046 recommends that runoff losses be ignored during the LIP event to maximize runoff (NRC, 2011). Therefore, infiltration is conservatively not simulated in FLO-2D, even in relatively undeveloped areas such as woods. Time of concentration or lag time was not needed by the FLO-2D, because overland flow is directly calculated based on ground surface conditions, such as elevations and roughness coefficients, within FLO-2D. Storm drain effects were also excluded from the LIP simulations.

NUREG/CR-7046 also recommends that nonlinearity adjustments be considered for PMF calculations when using unit hydrograph methodology (NRC, 2011). Unit hydrograph rainfall-runoff translation parameters (i.e., infiltration potential, time of concentration) were not calculated or used by the FLO-2D model in this calculation. This ensures that rainfall will be directly translated into a runoff hydrograph, and thus obviating the need for non-linearity adjustments to comply with NUREG/CR-7046.

2.1.3 Results

2.1.3.1 FLO-2D Model Development

FLO-2D is a physical process model that routes flood hydrographs and rainfall-runoff over unconfined flow surfaces or in channels using the dynamic wave approximation to the momentum equation (FLO-2D, 2014b). The watershed applicable for the LIP analysis was computed internally within FLO-2D based on the digital terrain model (DTM) input into FLO-2D (McKim & Creed, 2012a). The FLO-2D model includes topography, site location, and building structures. Grid elements along the model computational boundary were selected as outflow grid elements.

The FLO-2D model was developed using the following steps:

Step 1: Delineate FLO-2D Model Boundary and Establish Grid Element Dimensions

The project computational (i.e. local drainage area) boundary was established based on the location of the site, anticipated location of model boundary conditions, and the local drainage area anticipated to contribute to and / or influence discharge to SPS. The local drainage area was delineated based on elevation data from the United States Geological Survey (USGS) National Elevation Dataset (NED) (USGS, 2013). The final model computational boundary is shown in Figure 2.1-3. The selected grid element size for the model was 15 feet by 15 feet, based on the level of detail judged appropriate for the project. A walkdown of the island of buildings adjacent to the Discharge Canal was performed and measurements were taken of the openings to confirm the judgment that the 15 feet by 15 feet grid was of acceptable resolution to capture the flow characteristics of the site. The local drainage area of the SPS Plant site encompasses the SPS site including the Intake Canal and the Discharge Canal.

Step 2: Assign Elevation Data to Grid Elements

Grid element elevation for the project area was calculated based on the site topographic survey for SPS (McKim & Creed, 2012a) prepared by photogrammetric methods using aerial photography and field survey. For areas within the local drainage area of the site but outside the domain of the site topographic survey for SPS (McKim & Creed, 2012a), elevation data from the USGS NED (USGS, 2013) was used.

The topographic survey (McKim & Creed, 2012a) performed in 2012 at SPS was required to meet the American Society for Photogrammetry and Remote Sensing (ASPRS) Class I Accuracy Standard for 1" = 50' planimetrics and 1-foot contour intervals, with +/- 0.5 feet horizontal accuracy, +/- 0.33 feet Root Mean Square Error (RMSE) vertical accuracy for 1-foot contours and +/- 0.17 feet RMSE vertical accuracy for spot elevations, at well-defined points (McKim & Creed, 2012b). The methodology of the topographic survey was aerial photogrammetric mapping of the site with sufficient control points for calibration meeting the mapping standard.

A minimum of two closest DTM points within the vicinity of a grid element was used in computing grid elevations. FLO-2D interpolated elevations for grid elements were spot checked for accuracy and modified as necessary based on site survey. The FLO-2D interpolated grid elements were modified to account for buildings as described below and the elevations of the top of the Intake Canal embankments were set to elevation 36 feet, MSL (SPS, 2014). The final interpolated elevations for the grid elements, including the elevations assigned to the building grid elements and Intake Canal embankments are shown in Appendix B1.

Step 3: Define Surface Roughness Parameters

Manning's n-values used in FLO-2D are composite values that represent flow resistance. Grid element Manning's n-values were assigned based on land cover types at the site, and recommended Manning's roughness coefficients contained in Table 1 of the FLO-2D Reference Manual (FLO-2D, 2014b) and based on the upper end of the range of Manning's n-values typically used in one-dimensional models (Chow, 1959).

Table 2.1-1 shows the relationship between Manning's n values and selected land cover categories for the floodplain areas. The Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete and asphalt surfaces to 0.3 for areas with tree/shrub cover. Figure 2.1-4 shows the Manning's coefficients selection for each land cover. The land cover type upon which the Manning's roughness coefficients assignments were made was based on the high resolution orthoimagery of the site, obtained as part of the SPS topographical survey (McKim & Creed, 2012a). For areas outside the domain of the SPS topographical survey, orthoimagery from ESRI (ESRI, 2013) was used to determine the land cover type.

The Intake Canal is paved with a 4.5 inch thick concrete (SPS, 1986). A uniform Manning's roughness coefficient of 0.02 was assigned to the channel segment representing the Intake Canal (FLO-2D, 2014b). The Discharge Canal is lined with Fabriform concrete liner (SPS, 1986). A uniform Manning's roughness coefficient of 0.04 was assigned to the channel segment representing the Discharge Canal. The Manning's roughness coefficient assigned to the channel segment representing the Discharge Canal is higher than that assigned to the channel segment representing the Intake Canal because of the rough surface of the Fabriform concrete liner.

Step 4: Represent Building Rooftops and Other Flow Obstructions

Buildings at SPS were modeled as elevated grid elements in the FLO-2D model. Grid elements representing buildings were assigned an elevation value higher than the surrounding topography to ensure that rainfall runs off the building to the surrounding areas. Elevating the building grid elements also ensures that buildings act as potential obstructions to overland flow. The building grid elements were elevated for modeling purposes and the assigned elevations to the building grid elements are not the actual roof top elevations. The locations of the buildings were based on the high resolution orthoimagery (McKim & Creed, 2012a) and are shown in Figure 2.1-10. Grid elements that were completely within the aerial extent of a building were assigned elevations at least 5 feet higher than the surrounding topography. Uniform elevations were assigned to grid elements representing a single building. This ensures that runoff from a rooftops are uniformly distributed to the surrounding topography. For buildings with different rooftop elevations adjacent to each other (as estimated based on aerial photographs), the relative change in rooftop elevations were represented as a 2-feet difference in building grid element elevations. The peak 1-hour duration LIP depths are less than the relative change in elevation of 2 feet. Hence, water is unlikely to build-up high enough to drive flow from rooftops with lower elevations to adjacent rooftops with higher elevations. This ensures that general flow directions of runoff from rooftops are considered.

Step 5: Define Channel Segments and Inflows

Channel segments to represent the Intake and Discharge Canals were created. The channel segment representing the Intake Canal has a total length of about 9,500 feet and extends from the Intake Structure at the shore end of the canal to the high level intake structures at the plant end of the canal. The channel segment representing the Discharge Canal has a total length of about 2,500 feet and extends from the plant end of the canal to the shore line of the James River. The channel segments are shown on Figure 2.1-3.

Channel segment geometries for the Intake and Discharge Canals were defined based on the SPS topographic survey data (McKim & Creed, 2012a) and information contained in the SPS Updated Final Safety Analysis Report (UFSAR) (SPS, 2014) and the SPS Life Extension Evaluation reports for the Intake and Discharge Canals (SPS, 1986). The Intake and Discharge Canals were modeled as trapezoidal channels. The base width, depth, and side slopes of the Intake Canal were calculated to be 32 ft, 31 ft, and 1.5H: 1V, respectively. The top of berm elevation for the Intake Canal was set to elevation 36 feet, MSL (SPS, 2014). The base width, depth, and side slopes of the Discharge Canal were calculated to be 65 ft, 23.5 ft, and 2H:1V, respectively. Channel slope is calculated in FLO-2D as the difference between the channel element thalweg elevations divided by half the sum of the channel lengths within the channel elements. A minimum water level of 6' MSL was assumed in the Discharge Canal based on the top elevation of the Discharge Channel Control Structure. The channel bed elevation at the downstream end of the Discharge Canal was set at elevation 6 feet, MSL, which is equal to the top elevation of the Discharge Channel Control Structure to ensure that water does not exit the Discharge Canal until the water levels exceed the top elevation of the Discharge Channel Control Structure.

Initial water levels and flow into and out of the canals were modeled based on the scenarios/cases being considered. Two cases/scenarios were considered as described in Section 2.3.3. Under both cases, the initial water level in the Intake Canal was set at the maximum operating water level of 30 feet, MSL (SPS, 2014). Under the first case, a discharge of 3,921 cfs (1,760,000 gpm) from the Intake Canal representing the Plant circulating cooling flow rate was routed from the Intake Canal once the water level exceeds the maximum operating level of 30 feet, MSL. The discharge of 3,921 cfs represents the flow supplied by the four 220,000 gpm pumps for each of the two power generating units at SPS (SPS, 2014). Under the second case, a discharge of 980 cfs (440,000 gpm) representing the minimum discharge from the Intake Canal required to limit the maximum water elevation in the Intake Canal to 32 feet, MSL was specified once the water level in the Intake Canal exceeds the maximum operating level of 30 feet, MSL. Simulations were done with various outflow rates from the Intake Canal to arrive at the minimum required flowrate of 440,000 gpm. The discharges from the Intake Canal were conveyed to the Discharge Canal using a channel inflow node and modeled by specifying a depth-discharge relationship given by:

$$Q = ah^b$$

Where: Q is the flow rate in cubic feet per second,

h is the flow depth in feet, and

a and b are dimensionless coefficients.

Two depth-discharge relationships were used in specifying the discharges out of the Intake Canal; one depth-discharge relationship for when the water surface elevation in the Intake Canal is below 30 feet, MSL and another depth-discharge relationship for when the water surface elevation in the Intake Canal is at or above 30 feet, MSL. For the situation when the water surface elevation in the Intake Canal was below 30 feet, MSL, the values for the coefficients "a" and "b" used in the depth discharge relationship were zero. For the situation when the water surface elevation in the Intake Canal was at or above 30 feet, MSL, the values for the coefficients "a" and "b" used in the depth discharge relationship were 3,921 and 0 respectively for Case 1 and 980 and 0 respectively for Case 2.

The flows discharged from the Intake Canal were routed through the Discharge Canal. It was conservatively assumed that the minimum allowed water level in the Discharge Canal, for both cases, was 6 feet, MSL, representing the level at which the Discharge Channel Control Structure holds water in the Discharge Canal (i.e., to maximize the water in the Discharge Canal).

Step 6: Define "Levees" including Vehicle Barrier Systems

The Vehicle Barrier System (VBS) at SPS was modeled in FLO-2D using the levee structures component of the model. The location of the VBS was based on the topographic survey (McKim & Creed, 2012a). The top elevations of the levees representing the VBS were based on information provided by the Plant (SPS, 2013c). The height of the levees was 2.67 feet, which is the height of the VBS (SPS, 2013c). Movable barriers, including gates, which could potentially keep water from exiting the site, were conservatively modeled as closed. Bollard openings at the locations of the gates were kept open. The levees in the FLO-2D model are shown on Figure 2.1-3.

Step 7: Define "Hydraulic Structures" within Vehicle Barrier Systems

The pedestrian openings within the VBS were modeled as weirs using the hydraulic structure component of FLO-2D due to the small width of these openings relative to the grid size. Small scuppers were not considered. The location and width of the openings within the VBS were based on the topographic survey of the site (McKim & Creed, 2012a), and as observed during site visits. The depth-discharge relationship for flow through the 2-foot openings within the VBS is presented in Table 2.1-2. The depth-discharge relationship was developed using the general weir equation, with a weir coefficient indicative of open channel flow. Hydraulic structures with the depth-discharge relationship were assigned to the locations within the VBS where the 2-foot openings exist. Tailwater conditions were continually computed during the FLO-2D simulations such that the depth for any given flow and time step was the difference in water surface elevations on either side of the levee (i.e., VBS).

Step 8: Define Settling Pond

The Settling Pond, located at the northern end of the site (Figure 2.1-2), was modeled as a Detention Basin. The grid elements within the area domain of the Settling Pond were assigned an elevation equal to the normal pool elevation of 32.5 ft, MSL (SPS, 2010) within the basin.

2.1.3.2 Rainfall Inputs

The HHA approach applied to rainfall inputs first conservatively calculated the 1-square-mile, 1-hour duration PMP and sub-one hour divisions using HMR-52 (NOAA, 1982). The LIP also includes the 10-square-mile, 6-hour duration PMP, which was also calculated based on the methodology of HMR-52 (NOAA, 1982). The results are shown in Table 2.1-3. Three potential temporal distributions were initially evaluated: 1) a front-loaded distribution with the most severe 5-minute and 1-hour duration PMP at the beginning of the 6-hour time series as per NUREG/CR-7046 (NRC, 2011); 2) a middle-loaded distribution with the most severe 1-hour duration PMP at the center of the 6-hour time series (e.g., between hours 3 and 4), and 3) an end-loaded distribution with the most severe 5-minute and 1-hour duration PMP at the end of the 6-hour time series (Figure 2.1-5). The results of the analysis of the three potential rainfall temporal distributions indicated that locating the most intense part of the PMP later resulted in more conservative flood depths.

A refined calculation was then performed using the results of a site-specific meteorology study. Section 5.2 of ANSI/ANS-2.8-1992 (ANS, 1992) indicates that parameters of the PMP should be determined by a meteorological study utilizing a storm based approach. This analysis followed the storm-based approach as followed in HMR 53 (NOAA, 1980) and HMR 51 (NOAA, 1978).

The World Meteorological Organization (WMO) Manual for PMP determination (WMO, 2009) recommends this same approach. Figure 2.1-6 displays the major steps used in the calculation of the 1- and 6-hour, 1-square mile PMP.

The initial step in the development of the PMP values was to identify a set of storms which represent rainfall events that are PMP-type local storm events. This included all storms used in HMR 51 (NOAA, 1978) and HMR 52 (NOAA, 1982), all storms included in the USACE Storm Studies analyses (USACE, 1973), as well as more recent storms through November, 2014. Storms were selected considering:

- the influence of the warm waters of the Gulf Stream that would provide moisture to a given storm event;
- the Appalachian Mountains to the west and how that topography provides an impediment to low-level moisture flow from the west and changes storm structures as storms cross the mountains.
- the topography of the Appalachians serves to initiate rising motions and anchor precipitation to preferred locations in a way that is not found over SPS.

This resulted in 12 events being evaluated for use in LIP calculations (Figure 2.1-7 and Table 2.1-4). Seven of the storms were not covered by the HMR or USACE analyses. For these newly identified extreme rainfall events without published Depth-Area-Duration (DAD) analyses, hourly rainfall grids and DADs were computed using the Storm Precipitation Analysis System (SPAS) computer program (Parzybok et al., 2014). There are two main steps in the SPAS DAD analysis: 1) The creation of high-resolution hourly precipitation grids and 2) the computation of Depth-Area (DA) rainfall amounts for various durations. Because this process has been the standard for many years (all DAD produced by the NWS in HMR 51 used this procedure) and holds merit, the SPAS DAD analysis process used in this study attempts to mimic the NWS procedure as much as possible. By adopting this approach, consistency between the newly analyzed storms and the hundreds of storms already analyzed by the NWS is achieved.

The storm-based approach uses actual data from historic rainfall events which have occurred over the site and in regions transpositionable to the site. These rainfall data are maximized in-place following standard maximization procedures (NOAA, 1978), then transpositioned to SPS.

Storm maximization is the process of increasing rainfall associated with an observed extreme storm under the potential condition that additional moisture could have been available to the storm for rainfall production. This is accomplished by increasing the surface dew points (or sea surface temperatures; SSTs) to some climatological maximum and calculating the enhanced rainfall amounts that could potentially have been produced if those enhanced amounts of moisture had been available when the storm occurred. In-place storm maximization is applied to each storm. This study utilized the 6-, 12-, and 24-hour average 100-year recurrence interval dew point climatology and SST +2 sigma monthly average climatology. The development and results of these updated dew point and SST climatologies were extensively peer reviewed and accepted for use in PMP calculation by Federal Energy Regulatory Commission (FERC) and state dam safety regulators (AWA, 2008 and AWA, 2013, respectively).

Once each storm is maximized in-place, it is then transpositioned from its original location to the site. Transfer of a storm from where it occurred to a location that is meteorologically and topographically similar is known as storm transpositioning. The transpositioning process accounts for differences in moisture and elevation between the original location and SPS. For a given storm event to be considered transpositionable, there must be similar meteorological/climatological and topographical characteristics at its original location versus the new location. The general guidelines described in HMR 51 Section 2.4.2 are followed in this analysis. For SPS in particular, two of the guidelines are most influential. These are to not allow storm to cross the Appalachians and to not move storms more than +/- 1,000 feet. In addition, guidelines regarding latitudinal extent were considered. This limited transposition of storms beyond 5° to 6° latitude. This follows the guidance in the HMRs, specifically HMR 57 Section 7.4 (NOAA, 1994). Further, differences in moisture between the original location and the site are accounted for and quantified. For the SPS location, this affects storms whose moisture source is the Gulf of Mexico versus the Atlantic Ocean, where only storms whose main moisture source was the Atlantic Ocean were considered.

The process produces a total adjustment factor that is applied to the original rainfall data for each storm. The result represents the maximum rainfall each storm could have produced at SPS had all factors leading to the rainfall been ideal. Table 2.1-4 provides each value used in this calculation, including the observed or derived 1-hour and 6-hour rainfall, the calculated total adjustment factor for each storm, and the resulting total adjusted 1-and 6-hour rainfall amounts.

After the maximization and transposition factors were calculated for each storm, the results were applied to the maximum 1- and 6-hour value for each storm to calculate the maximized 1- and 6-hour 1-mi^2 value. The largest of these values results in the site-specific LIP for the site (Table 2.1-5). After adjustments were applied, the Jewell, MD July 1897 storm had the highest 1-hour rainfall and the Ewan, NJ September 1940 storm had the highest 6-hour rainfall, with several other storms providing support with slightly smaller values. For final applications, the 1-hour value is then required to be split into sub-hourly increments of 5-, 15-, 30-minutes. Therefore, the ratios derived in HMR 52 (Figures 36-38 of HMR 52; NOAA, 1982) were applied specific to the site location. The PMP depths results from the site-specific meteorology study are shown in Table 2.1-5.

Recommendations for LIP temporal distribution included in the site-specific meteorology study, indicated that the maximum 1-hour precipitation could occur as a front loaded storm (hours 1-2) or a middle loaded storm (hours 3-4). Therefore, based on the results of the LIP simulations with the HMR-51/52 precipitation amounts and using the three potential temporal distributions, the 6-hour PMP hyetograph was conservatively constructed using the 1-hour PMP for the fourth hour and equal rainfall increments for the preceding 3 hours and the fifth and sixth hours. The resultant hyetograph is shown in Figure 2.1-8.

2.1.3.3 FLO-2D Model Simulations

Two sets of LIP simulations were performed based on the HHA approach applied to rainfall inputs. The first set of LIP simulations included the more conservative precipitation depths determined based on HMR-51 and HMR-52. The maximum flood elevations for the HMR-51/52 simulations exceeded the design basis LIP flood elevations presented in the SPS UFSAR (SPS, 2014). Therefore, a refined LIP simulation was performed using the precipitation depths based on a site-specific meteorology study. Two cases/scenarios were considered for each set of LIP simulations including:

Case 1: Plant fully operational. The 6-hour PMP was simulated for this scenario under the following conditions:

A. The initial water level in the Intake Canal was conservatively set to the upper end of the operational range of 30 feet, MSL (SPS, 2013b; SPS 2015a).

B. A flow of 3,921 cfs, representing the total cooling flow rate used by the two generating units at SPS (SPS, 2013c) was withdrawn from the Intake Canal when water levels in the canal at or above 30 feet, MSL. This flow rate was routed through the Discharge Canal.

C. A minimum water level of 6 feet, MSL was maintained in the Discharge Canal based on the top elevation of the Discharge Channel Control Structure.

Case 2: Plant shutdown but with the capability of maintaining water levels in the Intake Canal. The 6-hour PMP was simulated for this scenario under the following conditions:

A. The initial water level in the Intake Canal was conservatively set to the upper end of the operational range of 30 feet, MSL (SPS, 2013b; SPS 2015a).

B. A flow of 980 cfs, representing the minimum flow rate required to keep the water level in the Intake Canal from exceeding 32 ft, MSL was withdrawn from the Intake Canal when water levels in the canal are at or above 30 feet, MSL. This flow was routed through the Discharge Canal. Four water boxes in-service at the Intake Canal are sufficient to discharge the minimum flow of 980 cfs during the LIP (SPS, 2015b). The plant has a procedure in place to ensure that water box outlets are open to discharge flow from the Intake Canal when the water level in the Intake Canal is greater or equal to 30 ft, MSL (SPS, 2015a).

C. A minimum water level of 6 feet, MSL was maintained in the Discharge Canal based on the top elevation of the Discharge Channel Control Structure.

Results Based on HMR-51 and HMR-52 Rainfall Input

The results of the LIP simulations based on HMR-51 and HMR-52 rainfall inputs for Cases 1 and 2 are summarized in Tables 2.1-6 and 2.1-7 respectively and include maximum water surface elevations, maximum flow depths, and maximum flow velocities for representative grid elements at strategic locations identified by SPS personnel (Figure 2.1-9). The maximum water surface elevations for Cases 1 and 2 are displayed in Figures 2.1-11 and 2.1-12 respectively.

Under the Case 1 scenario, the LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 and 2 range from 27.1 feet, MSL at Door 1-BS-DR-74 (Door 43) to 30.2 feet, MSL at the Doors into the Maintenance Building at the south-eastern end of the site (Doors 17 and 18). The maximum water levels in the Intake Canal and Discharge Canal under the plant operating scenario are 31.4 ft, MSL and 11.1 ft, MSL respectively. Water level in the Intake Canal does not overtop the canal embankments of elevation 36 ft, MSL for the plant operating scenario. The flow direction in the Intake Canal under the plant operating scenario is towards the plant.

Under the Case 2 scenario, the LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 range from 27.2 feet, MSL at Door 1-BS-DR-74 (Door 43) to 30.2 feet, MSL at the Doors into the Maintenance Building at the south-eastern end of the site (Doors 17 and 18). The Intake Canal acts as a reservoir under this scenario: Water is neither flowing to nor away from the plant. The maximum water levels in the Intake Canal and Discharge Canal under the

Case 2 scenario are 36.4 ft, MSL and 11.1 ft, MSL respectively. Water level in the Intake Canal overtops the canal embankments which are at elevation 36 ft, MSL. Please note that, unlike the Case 2 scenario described for the analysis using the site specific precipitation, no discharges (i.e., withdrawals) were allowed from the Intake Canal except flows that overtop the Intake Canal embankments.

Results Based on Site-Specific Meteorology Study

The results of the LIP simulations based on the site-specific meteorology study rainfall inputs for both Cases 1 and 2 are nearly identical with the exception of the water levels in the Intake and Discharge Canals. The resulting water levels in the Intake Canal do not overtop the Intake Canal berm for both cases and do not contribute to flooding at the site. The results of the LIP simulations for Cases 1 and 2 are summarized in Table 2.1-8 and include maximum water surface elevations, maximum flow depths, and maximum flow velocities for representative grid elements at "strategic doors" identified by SPS personnel (Figure 2.1-9). The maximum water surface elevations for both Cases 1 and 2 are displayed in Figure 2.1-13. Flood stage hydrographs at representative locations are included as Figures 2.1-14 through 2.1-17. Doors 1, 14, 18, and 49 were selected as sample locations for timeplots due to their general locations within the power block area. Door 1 – west side of power block; Door 14 – south side of Turbine Building; Door 18 – southeast corner of Turbine Building; and Door 49 – north side of Reactor Containments.

The FLO-2D results are shown in plots included in Appendix B (Figures B1 through B6). Figure B1 shows the calculated ground elevation at each grid element. Figure B2 shows the calculated maximum flow depth. Figure B3 shows the calculated maximum water surface elevation. Figure B4 shows the calculated maximum velocity. Figure B5 shows the calculated maximum velocity vector. Figure B6 shows the grid element number.

The LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 and 2 range from a low of 26.9 feet, MSL at Door 1-BS-DR-74 (Door 43) to a high of 29.4 feet, MSL at the Doors into the Maintenance Building at the south-eastern end of the site (Doors 17 and 18). The calculated maximum water surface elevations are below the threshold elevations at Door numbers 11, 37, 47, 48, 51 and 53 but exceed the threshold elevations at all the other doors. A figure showing door numbers and locations is included as Figure 2.1-9. Calculated maximum flow depths in the immediate vicinity of SPS Units 1 and 2 range from 0.2 feet at the "ECST" (Door Number 11) to 5.0 feet at "1-BS-DR-D23-1 and -2" (Door Numbers 39 and 40). Calculated maximum flow velocities at SPS Units 1 and 2 range from 0.2 feet per second at Door 1-BS-DR-FP27-1 (Door Number 15) to 5.4 feet per second at "1-BS-DR-74" (Door Number 43).

The protected area is completely impervious and does not contain natural sources of vegetation and debris. The maximum velocities of up to 5.0 feet per second during the LIP are unlikely to result in debris loading issues at the site. Hydrodynamic and Hydrostatic loading against buildings at the site are also likely to be minimal due to the generally shallow depths and low velocities during the LIP.

The general direction of flow during the LIP is from south to north, towards the Discharge Canal. Water overtops the vehicle barriers at the parking lot on the south-western end of the plant and at the eastern end of the plant, near the Rad Waste facility (Figure 2.1-2) during the peak 5-minute period of the storm.

The maximum water levels in the Intake Canal and Discharge Canal for Case 1 are 31.0 ft, MSL and 10.5 ft, MSL respectively. The maximum water levels in the Intake Canal and Discharge Canal for Case 2 are 31.7 ft, MSL and 9.6 ft, MSL respectively. Water levels in the Intake Canal for both Cases 1 and 2 do not overtop the canal embankments which are at elevation 36 ft, MSL.

The FLO-2D reference manual (FLO-2D, 2014b) provide three keys to a successful project application. These include volume conservation, area of inundation and maximum velocities and numerical surging.

- Volume Conservation: Review of the FLO-2D output files indicates volume conservation errors up to 0.000003 percent for the FLO-2D run. This value is below the threshold of 0.001 percent specified in the FLO-2D Data Input manual (FLO-2D, 2014c) for a successful project application.
- Area of Inundation: Review of the FLO-2D output files indicates maximum inundated area of 427 acres. The FLO-2D model is made up of 82,435 grid elements, each 15 feet by 15 feet in dimension. The LIP was simulated within the entire computational domain of the model. The maximum inundation area should therefore be equal to the area of the computational domain of 426 acres (15 x 15 x 82,435) x (1 acre / 43,560 feet). The FLO-2D calculated maximum inundation area of 427 acres is reasonable and indicates a successful project application.
- Maximum Velocities and Numerical Surging: Numerical surging, if it exists, would be evident at unreasonably high velocities in the FLO-2D output files (FLO-2D, 2014b). A review of the velocity output file does not indicate unreasonably high velocities in the model runs and indicates a successful project application. The high velocities reported occur at locations of steep slopes mostly near the channels and are reasonable.

2.1.4 Conclusions

Two sets of LIP simulations were performed based on the HHA approach applied to rainfall inputs. The first set of LIP simulations included the more conservative precipitation depths determined based on HMR-51 and HMR-52 rainfall inputs. A set of refined LIP simulations was performed using the precipitation depths from a site-specific meteorology study. A summary of the results of the LIP simulations at SPS (Table 2.1-8) are as follows:

1. PMP Depths: The maximum flood elevation due to the HMR-51/52 LIP simulation at SPS results from a total rainfall depth of 18.6 inches within an hour and 28.8 inches within 6 hours. The maximum flood elevation due to the site-specific meteorology study LIP simulation at SPS results from a total rainfall depth of 12.4 inches within an hour and 28.8 inches within 6 hours.
2. LIP simulation based on HMR-51/52 rainfall inputs: Under the Case 1 scenario, the LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 and 2 range from 27.1 feet, MSL at Door 1-BS-DR-74 (Door 43) to 30.2 feet, MSL at the Doors into the Maintenance Building at the south-eastern end of the site (Doors 17 and 18). Water level in the Intake Canal does not overtop the canal embankments of elevation 36 ft, MSL for the plant operating scenario. Under the Case 2 scenario, the LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 range from 27.2 feet, MSL at Door 1-BS-DR-74 (Door 43) to 30.2 feet, MSL at the Doors into the Maintenance Building at the

south-eastern end of the site (Doors 17 and 18). The Intake Canal acts as a reservoir under this scenario: The maximum water levels in the Intake Canal and Discharge Canal under the Case 2 scenario are 36.4 ft, MSL and 11.1 ft, MSL respectively. Water level in the Intake Canal overtops the canal embankments which are at elevation 36 ft, MSL.

3. LIP simulation based on site-specific meteorology study rainfall inputs: The results of the LIP simulations based on the site-specific meteorology study rainfall inputs for both Cases 1 and 2 are identical with the exception of the water levels in the Intake and Discharge Canals. The resulting water levels in the Intake Canal do not overtop the Intake Canal berm for both cases and do not contribute to flooding at the site. The LIP maximum water surface elevations in the immediate vicinity of SPS Units 1 and 2 range from a low of 26.9 feet, MSL at Door 1-BS-DR-74 (Door 43) to a high of 29.4 feet, MSL at the Doors into the Maintenance Building at the south-eastern end of the site (Doors 17 and 18). The calculated maximum water surface elevations are below the threshold elevations at Door numbers 11, 37, 47, 48, 51 and 53 but exceed the threshold elevations at all the other doors.

The LIP results from the site-specific meteorology study are used in this report (see Section 3.1), as they are based on site-specific precipitation inputs that are more refined than the generic HMR-51/HMR-52 precipitation inputs. This process of refinement is consistent with the HHA approach described in NUREG/CR-7046 (NRC, 2011).

2.1.5 References

2.1.5-1 **ANS, 1992**. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8-1992).

2.1.5-2 **AWA, 2008**. Applied Weather Associates (AWA), Site-Specific Probable Maximum Precipitation (PMP) Study for the Blenheim-Gilboa basin, New York Power Authority, White Plains, NY, 2008.

2.1.5-3 **AWA, 2013**. Applied Weather Associates (AWA), Statewide Probable Maximum Precipitation (PMP) Study for the State of Ohio, Ohio Dam Safety, Columbus, OH, 2013.

2.1.5-4 **Chow, 1959**. Open-Channel Hydraulics, Ven Te Chow, Reprint of the 1959 Edition, McGraw Hill Book Company.

2.1.5-5 **ESRI, 2013**. ESRI ArcGIS Online World Imagery. http://goto.arcgisonline.com/maps/World_Imagery. Image downloaded from ArcMap on January 4, 2013.

2.1.5-6 **FLO-2D, 2014a**. FLO-2D Pro Model, Build No. 14.03.07 by FLO-2D Software, Inc., Nutrioso, Arizona.

2.1.5-7 **FLO-2D, 2014b**. FLO-2D® Pro Reference Manual, FLO-2D Software, Inc., Nutrioso, Arizona (www.flo-2d.com).

2.1.5-8 **FLO-2D, 2014c**. FLO-2D® Data Input Manual, FLO-2D Software, Inc., Nutrioso, Arizona (www.flo-2d.com).

2.1.5-9 **McKim & Creed, 2012a**. Topographic survey plan including: topographic, buildings, and structures information; Site Digital Terrain Model (DTM); and Site Aerial Photography. McKim & Creed, Inc., December 2012.

2.1.5-10 **McKim & Creed, 2012b**. "Aerial Mapping Validation Report-Surry Nuclear Generating Station," McKim & Creed, Inc., December 1, 2012.

2.1.5-11 **NOAA, 1978**. Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, Hydrometeorological Report No.51 (HMR-51) by U. S. Department of Commerce & USACE, National Oceanic and Atmospheric Administration, June 1978.

2.1.5-12 **NOAA, 1980**. Seasonal Variation of 10-square-mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No.53 (HMR-53) by US Department of Commerce and US Nuclear Regulatory Commission, National Oceanic and Atmospheric Administration, April 1980.

2.1.5-13 **NOAA, 1982.** "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", NOAA Hydrometeorological Report No. 52 (HMR-52) by US Department of Commerce & USACE, August 1982.

2.1.5-14 **NOAA, 1994.** "Probable Maximum Precipitation-Pacific Northwest States – Columbia River (including portions of Canada), Snake River and Pacific Coastal Drainages," NOAA Hydrometeorological Report No. 57 (HMR-57) by US Department of Commerce & USACE, October 1994.

2.1.5-15 **NRC, 2011**. Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046, United States Nuclear Regulatory Commission, November 2011.

2.1.5-16 **Parzybok et al., 2014**. Storm Precipitation Analysis System (v.9.5) Applied Weather Associates, 2014.

2.1.5-17 **SPS, 1986**. "Life Extension Evaluation of Intake / Discharge Structures and Canals", Project Topical Report for Unit No. 1 Surry Power Station, Stone and Webster Engineering Corporation, April 1986.

2.1.5-18 **SPS, 2010**. "Plot Plan Settling Pond Area – Surry Power Station – Unit 1 & 2," Drawing No. 5129-C-5001, Revision 11, November 16, 2010.

2.1.5-19 **SPS 2013a**. "DRS-BDB-FLD-SPS-13-008" SPS Units 1 and 2 Beyond Design Basis Project Vertical Datum Conversion Information, dated July 10, 2013.

2.1.5-20 **SPS, 2013b**. Surry Power Station Procedure 0-AP-37.01, Abnormal Environmental Conditions, Revision 59.

2.1.5-21 **SPS, 2013c**. "DRS-BDB-FLD-SPS-13-009" SPS Units 1 & 2 Beyond Design Basis Project. Requested Information for SPS LIP Calculation, dated August 20, 2013

2.1.5-22 **SPS, 2014**. Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR), Revision 46.02.

2.1.5-23 **SPS, 2015a**. SPS Annunciator Response Procedure "INTK CANAL HI-LO LVL" (1B-E1), Revision 2, Dominion Nuclear Engineering.

2.1.5-24 SPS, 2015b. Calculation Number ME-12135, Revision 0, Beyond Design Basis Flooding Hydraulic Analysis to Maintain Intake Canal Fluid Elevation at Surry Power Station, Dominion Nuclear Engineering, 2015.

2.1.5-25 **USACE, 1973**. US Army Corps of engineers (USACE), US Army 1936-1973, Storm Rainfall in the United States, Depth-Area-Duration Data, Office of Chief of Engineers, Washington, D.C.

2.1.5-26 USGS, 2013. United States Geological Survey (USGS) National Elevation Dataset (NED) 1/9 Arc Second Elevation Dataset. http://ned.usgs.gov. Data downloaded on December 2, 2013.

2.1.5-27 WMO, 2009. World Meteorological Organization (WMO), Manual for Estimation of Probable Maximum Precipitation, Operational Hydrology Report No 1045, WMO, Geneva, 2009.

Table 2.1-1: Manning's n Values for Selected Land Use Categories

Land Use Category	Manning's n
Paved / Concrete	0.02
Gravel	0.04
Open Ground	0.05
Short Grass	0.05
Sparse Vegetation	0.1
Shrubs/ Trees	0.3
Intake Canal / Concrete Lining	0.02
Discharge Canal / Fabriform	0.04

Table 2.1-2: Depth – Discharge Relationship for 2-feet Openings in the Vehicle Barrier System

Flow depth (feet)	Discharge (cfs)
0.0	0.0
0.4	1.6
0.8	4.4
1.2	8.2
1.6	12.5
2.0	17.5
2.4	23.1
2.8	29.0
3.2	35.5
3.6	42.3
4.0	49.6

Notes

1. Depth – Discharge relationship developed based on the Weir Equation $Q = CLH^{1.5}$ (Chow, 1959), where Q is discharge in cubic feet per second (cfs), C is the weir coefficient, L is the flow width in feet, and H is the flow depth;
2. Width (L) of the openings in the Vehicle Barrier System is 2 feet;
3. A weir coefficient of 3.1 was used.

Table 2.1-3: HMR-51/52 - Probable Maximum Precipitation Depths at SPS

Time (minutes)	PMP Depth (inches)
360	28.8
60	18.6
30	13.9
15	9.7
5	6.1

Table 2.1-4: Site-Specific Meteorology Study Storm List

Storm Name	State	Lat	Lon	Year	Month	Day	Maximum Total Storm Rainfall	Maximum 6-hour 10mi² Rainfall	Maximum 1-hour 1mi² Rainfall Using HMR 52 Ratio or SPAS Data	Maximum 6-hour 1mi² Rainfall Using SPAS Ratio or SPAS Data	Surry Total Adjustment Factor	Surry 1-hour 1mi² PMP	Surry 6-hour 1mi² PMP	Precipitation Source
JEWELL	MD	38.7550	-76.6184	1897	7	26	15.80	13.00	8.45	14.04	1.47	12.42	20.64	NA 1-7
MANNING	SC	33.6954	-80.2101	1893	8	26	13.20	8.40	5.46	9.07	1.11	6.06	10.07	SA 2-1
SPARTA	NJ	41.0300	-74.6400	2000	8	11	16.70	10.30	4.00	12.60	1.81	7.24	22.81	SPAS 1017
EWAN	NJ	39.7000	-75.1900	1940	9	1	24.00	19.20	7.30	21.00	1.37	10.00	28.77	NA 2-4
TABERNACLE	NJ	39.8805	-74.6900	2004	7	13	15.63	12.80	5.90	13.60	1.33	7.85	18.09	SPAS 1040
WESTFIELD	MA	42.1200	-72.7000	1955	8	17	19.80	8.40	5.46	7.85	1.22	6.66	9.58	SPAS 1001
NEWARK	NJ	40.7300	-74.2700	1999	9	15	14.45	6.09	3.96	6.53	1.24	4.91	8.10	SPAS 1002
WILLIAMSBURG	VA	37.2700	-76.7000	1999	9	14	16.98	6.60	4.29	6.70	1.16	4.98	7.77	SPAS 1012
ST GEORGE	GA	30.5212	-82.0372	1911	8	26	19.10	14.90	9.69	16.09	1.21	11.72	19.47	SA 3-11
DOUGLASVILLE	GA	33.8700	-84.7600	2009	9	19	25.37	14.95	5.84	17.03	1.12	6.54	19.07	SPAS 1218
MANAHAWKIN	NJ	39.6957	-74.2588	1939	8	19	17.80	9.70	6.31	10.48	1.19	7.50	12.47	NA 2-3
ISLIP	NY	40.8050	-73.0650	2014	8	13	14.23	13.29	6.94	13.95	1.34	9.30	18.69	SPAS 1415

Table 2.1-5: Site-Specific Meteorology Study - Probable Maximum Precipitation Depths at SPS

Time (minutes)	PMP Depth (inches)
360	28.8
60	12.4
30	9.3
15	6.3
5	4.1

Table 2.1-6: HMR-51/52 LIP Simulation Results for Case 1

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Velocity (fps)
1	Double Door into Old Admin Building	18,897	27.00	25.47	28.09	29.53	2.62	2.53	6.01	0.55
2	into Stairwell	17,883	27.17	25.17	27.66	29.10	2.49	1.93	6.00	3.94
3	into Stairwell and OSC	18,089	27.17	25.15	27.67	29.11	2.52	1.94	6.00	0.86
4	1-BS-DR-S27-15	18,294	27.17	25.06	27.69	29.13	2.63	1.96	6.00	0.95
5	into I&C Shop	18,498	27.17	25.08	27.72	29.16	2.64	1.99	6.00	0.79
6	1-BS-DR-S27-19A	19,109	27.00	24.85	27.90	29.34	3.05	2.34	6.00	1.42
7	1-BS-DR-S27-16A	19,312	27.00	24.37	27.97	29.41	3.60	2.41	6.00	1.69
8	0-SE-DR-CAS-100A	17,680	28.00	25.56	27.64	29.08	2.08	1.08	6.00	2.37
9	0-SE-DR-CAS-103A	17,478	28.00	25.57	27.65	29.09	2.08	1.09	6.00	0.63
10	0-SE-DR-CAS-101C	17,273	28.00	25.60	27.64	29.08	2.04	1.08	6.00	1.42
11	at ECST	17,066	30.50	27.28	27.63	29.07	0.35	-1.43	6.00	0.61
12	1-BS-DR-SEC	19,294	27.00	25.63	28.16	29.60	2.53	2.60	6.00	2.74
13	1-BS-DR-T27-1 and -2	20,292	27.00	25.15	28.30	29.74	3.15	2.74	6.01	1.69
14	1-BS-DR-T27-3 and -4	22,478	27.00	25.41	28.59	30.03	3.18	3.03	6.00	0.60
15	1-BS-DR-FP27-1	22,259	27.00	25.12	28.46	29.90	3.34	2.90	6.01	0.76
16	at SE Corner of Unit 2	23,869	27.00	25.27	28.66	30.10	3.39	3.10	6.00	0.69
17	into Maintenance Offices	24,267	27.00	25.24	28.75	30.19	3.51	3.19	6.01	1.04
18	into Maintenance Offices	24,267	27.00	25.24	28.75	30.19	3.51	3.19	6.01	1.04
19	into Maintenance Offices	25,053	27.00	25.57	28.54	29.98	2.97	2.98	6.01	0.70
20	into Maintenance Offices	25,448	27.00	25.36	28.50	29.94	3.14	2.94	6.02	1.37
21	into Maintenance Offices	25,845	27.00	25.16	28.45	29.89	3.29	2.89	6.00	1.65
22	into Maintenance Offices	25,650	27.00	25.22	28.41	29.85	3.19	2.85	6.01	2.24
23	into Maintenance Offices	25,257	27.00	25.16	28.37	29.81	3.21	2.81	6.00	1.55
24	into Maintenance Offices	24,864	27.00	25.23	28.27	29.71	3.04	2.71	6.00	1.82
25	into CP Building	22,703	27.00	25.16	27.74	29.18	2.58	2.18	6.00	1.87
26	into CP Building	22,504	28.00	24.96	27.70	29.14	2.74	1.14	6.00	0.83
27	into CP Building	22,306	27.00	25.08	27.69	29.13	2.61	2.13	6.00	1.32
28	1-BS-DR-DVP27-1	15,191	27.00	25.63	27.05	28.49	1.42	1.49	6.00	0.76
29 - 30	Not Used									
31	into Aux Boiler Room	21,520	27.00	24.95	27.45	28.89	2.50	1.89	6.01	1.50
32	into Storeroom	21,518	27.00	24.88	27.41	28.85	2.53	1.85	6.00	1.59
33	1-BS-DR-67	21,319	27.00	24.88	27.36	28.80	2.48	1.80	6.01	1.67
34	1-BS-DR-66	21,119	27.00	25.22	27.31	28.75	2.09	1.75	6.00	0.54
35	1-BS-DR-65	20,920	27.00	25.30	27.32	28.76	2.02	1.76	6.00	1.16
36	1-BS-DR-CS27-3	20,724	27.50	25.24	27.29	28.73	2.05	1.23	6.00	1.37
37	1-BS-DR-SG28-2	20,133	29.00	24.51	27.19	28.63	2.68	-0.37	6.00	1.96
38	1-BS-DR-27-3	18,525	27.00	22.75	26.99	28.43	4.24	1.43	5.99	1.74
39	1-BS-DR-D23-1	18,118	23.00	21.99	27.03	28.47	5.04	5.47	6.00	1.76

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Velocity (fps)
40	1-BS-DR-D23-2	17,914	23.00	25.84	27.00	28.44	1.16	5.44	6.00	0.40
41	2-BS-DR-DVP27-2	18,538	27.00	25.35	26.99	28.43	1.64	1.43	6.00	1.92
42	1-BS-DR-73	18,129	26.67	25.13	26.86	28.30	1.73	1.63	6.00	3.77
43	1-BS-DR-74	17,310	26.67	24.58	25.67	27.11	1.09	0.44	6.01	6.94
44	1-BS-DR-F27-5 and -7	17,711	27.00	25.34	26.96	28.40	1.62	1.40	6.00	0.50
45	1-BS-DR-F27-4	17,092	27.00	25.44	26.98	28.42	1.54	1.42	6.00	0.28
46	1-BS-DR-F27-3 and -6	17,087	27.00	25.41	27.01	28.45	1.60	1.45	6.00	0.42
47	Above Grade 5'	17,498	32.50	25.64	27.02	28.46	1.38	-4.04	6.00	0.80
48	Above Grade 5'	17,498	32.50	25.64	27.02	28.46	1.38	-4.04	6.00	0.80
49	1-BS-DR-WG27-1	17,495	27.50	24.40	27.03	28.47	2.63	0.97	6.00	0.30
50	1-BS-DR-F27-1 and -2	17,697	27.50	25.87	27.02	28.46	1.15	0.96	6.00	1.10
51	1-BS-DR-SG28-1	17,686	29.00	24.90	27.65	29.09	2.75	0.09	6.00	1.06
52	1-BS-DR-CS-27-1	18,704	27.50	24.80	27.82	29.26	3.02	1.76	6.00	2.03
53	at ECST	19,741	30.50	26.09	27.07	28.51	0.98	-1.99	6.00	1.26
54	at Auxiliary Building	19,725	27.50	25.29	27.51	28.95	2.22	1.45	6.00	0.61
55	1-BS-DR-A27-8	18,915	27.50	24.61	28.15	29.59	3.54	2.09	6.00	0.80
56	1-BS-DR-A27 1 and -2	19,317	27.50	24.38	28.13	29.57	3.75	2.07	6.00	0.92
57	at Auxiliary Building	19,923	27.50	25.15	27.51	28.95	2.36	1.45	6.00	0.88
58	into Clean Change	20,122	27.00	25.25	27.52	28.96	2.27	1.96	6.00	0.99
59	into Clean Change	20,720	27.00	24.95	27.39	28.83	2.44	1.83	6.00	1.93

See Figure 2.1-9 for Door Locations

Table 2.1-7: HMR-51/52 LIP Simulation Results for Case 2

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Velocity (fps)
1	Double Door into Old Admin Building	18,897	27.00	25.47	28.15	29.59	2.68	2.59	6.00	0.55
2	into Stairwell	17,883	27.17	25.17	27.69	29.13	2.52	1.96	6.00	3.98
3	into Stairweel and OSC	18,089	27.17	25.15	27.68	29.12	2.53	1.95	6.00	0.90
4	1-BS-DR-S27-15	18,294	27.17	25.06	27.71	29.15	2.65	1.98	6.00	0.87
5	into I&C Shop	18,498	27.17	25.08	27.72	29.16	2.64	1.99	6.00	0.73
6	1-BS-DR-S27-19A	19,109	27.00	24.85	27.91	29.35	3.06	2.35	6.00	1.50
7	1-BS-DR-S27-16A	19,312	27.00	24.37	28.01	29.45	3.64	2.45	6.00	1.81
8	0-SE-DR-CAS-100A	17,680	28.00	25.56	27.65	29.09	2.09	1.09	6.00	2.66
9	0-SE-DR-CAS-103A	17,478	28.00	25.57	27.66	29.10	2.09	1.10	6.00	0.53
10	0-SE-DR-CAS-101C	17,273	28.00	25.60	27.65	29.09	2.05	1.09	6.00	1.52
11	at ECST	17,066	30.50	27.28	27.64	29.08	0.36	-1.42	6.00	0.62
12	1-BS-DR-SEC	19,294	27.00	25.63	28.19	29.63	2.56	2.63	6.00	2.84
13	1-BS-DR-T27-1 and -2	20,292	27.00	25.15	28.33	29.77	3.18	2.77	6.00	1.53
14	1-BS-DR-T27-3 and -4	22,478	27.00	25.41	28.64	30.08	3.23	3.08	6.01	0.68
15	1-BS-DR-FP27-1	22,259	27.00	25.12	28.52	29.96	3.40	2.96	6.01	0.85
16	at SE Corner of Unit 2	23,869	27.00	25.27	28.67	30.11	3.40	3.11	6.01	0.93
17	into Maintenance Offices	24,267	27.00	25.24	28.77	30.21	3.53	3.21	6.01	1.12
18	into Maintenance Offices	24,267	27.00	25.24	28.77	30.21	3.53	3.21	6.01	1.12
19	into Maintenance Offices	25,053	27.00	25.57	28.57	30.01	3.00	3.01	6.01	0.60
20	into Maintenance Offices	25,448	27.00	25.36	28.54	29.98	3.18	2.98	6.01	1.56
21	into Maintenance Offices	25,845	27.00	25.16	28.48	29.92	3.32	2.92	6.00	1.78
22	into Maintenance Offices	25,650	27.00	25.22	28.44	29.88	3.22	2.88	6.00	2.32
23	into Maintenance Offices	25,257	27.00	25.16	28.37	29.81	3.21	2.81	6.02	1.84
24	into Maintenance Offices	24,864	27.00	25.23	28.29	29.73	3.06	2.73	6.01	1.79
25	into CP Building	22,703	27.00	25.16	27.75	29.19	2.59	2.19	6.01	1.73
26	into CP Building	22,504	28.00	24.96	27.69	29.13	2.73	1.13	6.01	0.74
27	into CP Building	22,306	27.00	25.08	27.69	29.13	2.61	2.13	6.01	1.21
28	1-BS-DR-DVP27-1	15,191	27.00	25.63	27.05	28.49	1.42	1.49	6.00	0.70
29 - 30	Not Used									
31	into Aux Boiler Room	21,520	27.00	24.95	27.47	28.91	2.52	1.91	6.00	1.49
32	into Storeroom	21,518	27.00	24.88	27.41	28.85	2.53	1.85	6.01	1.81
33	1-BS-DR-67	21,319	27.00	24.88	27.36	28.80	2.48	1.80	6.01	1.55
34	1-BS-DR-66	21,119	27.00	25.22	27.31	28.75	2.09	1.75	6.01	0.56
35	1-BS-DR-65	20,920	27.00	25.30	27.29	28.73	1.99	1.73	6.01	1.04
36	1-BS-DR-CS27-3	20,724	27.50	25.24	27.28	28.72	2.04	1.22	6.00	1.25
37	1-BS-DR-SG28-2	20,133	29.00	24.51	27.18	28.62	2.67	-0.38	6.00	1.95
38	1-BS-DR-27-3	18,525	27.00	22.75	27.13	28.57	4.38	1.57	6.01	2.22
39	1-BS-DR-D23-1	18,118	23.00	21.99	27.00	28.44	5.01	5.44	6.00	1.83

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Velocity (fps)
40	1-BS-DR-D23-2	17,914	23.00	25.84	27.00	28.44	1.16	5.44	6.00	0.48
41	2-BS-DR-DVP27-2	18,538	27.00	25.35	27.01	28.45	1.66	1.45	6.00	2.00
42	1-BS-DR-73	18,129	26.67	25.13	26.88	28.32	1.75	1.65	6.00	3.81
43	1-BS-DR-74	17,310	26.67	24.58	25.68	27.12	1.10	0.45	6.00	7.01
44	1-BS-DR-F27-5 and -7	17,711	27.00	25.34	26.98	28.42	1.64	1.42	6.00	0.47
45	1-BS-DR-F27-4	17,092	27.00	25.44	26.98	28.42	1.54	1.42	6.00	0.31
46	1-BS-DR-F27-3 and -6	17,087	27.00	25.41	27.02	28.46	1.61	1.46	6.00	0.36
47	Above Grade 5'	17,498	32.50	25.64	27.05	28.49	1.41	-4.01	6.00	0.80
48	Above Grade 5'	17,498	32.50	25.64	27.05	28.49	1.41	-4.01	6.00	0.80
49	1-BS-DR-WG27-1	17,495	27.50	24.40	27.02	28.46	2.62	0.96	6.00	0.37
50	1-BS-DR-F27-1 and -2	17,697	27.50	25.87	27.03	28.47	1.16	0.97	6.00	1.05
51	1-BS-DR-SG28-1	17,686	29.00	24.90	27.66	29.10	2.76	0.10	6.00	1.05
52	1-BS-DR-CS-27-1	18,704	27.50	24.80	27.81	29.25	3.01	1.75	6.00	1.82
53	at ECST	19,741	30.50	26.09	27.09	28.53	1.00	-1.97	6.01	1.32
54	at Auxiliary Building	19,725	27.50	25.29	27.49	28.93	2.20	1.43	6.00	0.61
55	1-BS-DR-A27-8	18,915	27.50	24.61	28.16	29.60	3.55	2.10	6.00	1.10
56	1-BS-DR-A27 1 and -2	19,317	27.50	24.38	28.20	29.64	3.82	2.14	6.00	1.19
57	at Auxiliary Building	19,923	27.50	25.15	27.48	28.92	2.33	1.42	6.00	1.14
58	into Clean Change	20,122	27.00	25.25	27.47	28.91	2.22	1.91	6.00	1.02
59	into Clean Change	20,720	27.00	24.95	27.38	28.82	2.43	1.82	6.00	1.87

See Figure 2.1-9 for Door Locations

Table 2.1-8: Site-Specific Meteorology Study - LIP Simulation Results for Cases 1 and 2

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Grid Elevation (ft, MSL/ Plant Datum)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Maximum Velocity (fps)
1	Double Door into Old Admin Building	18,897	27.00	25.47	26.91	27.52	28.96	2.05	1.96	3.60	0.44
2	into Stairwell	17,883	27.17	25.17	26.61	27.24	28.68	2.07	1.51	3.60	2.78
3	into Stairweel and OSC	18,089	27.17	25.15	26.59	27.25	28.69	2.10	1.52	3.59	0.94
4	1-BS-DR-S27-15	18,294	27.17	25.06	26.50	27.28	28.72	2.22	1.55	3.59	0.78
5	into I&C Shop	18,498	27.17	25.08	26.52	27.30	28.74	2.22	1.57	3.59	0.73
6	1-BS-DR-S27-19A	19,109	27.00	24.85	26.29	27.44	28.88	2.59	1.88	3.59	1.21
7	1-BS-DR-S27-16A	19,312	27.00	24.37	25.81	27.51	28.95	3.14	1.95	3.59	1.50
8	0-SE-DR-CAS-100A	17,680	28.00	25.56	27.00	27.23	28.67	1.67	0.67	3.58	2.32
9	0-SE-DR-CAS-103A	17,478	28.00	25.57	27.01	27.28	28.72	1.71	0.72	3.59	0.65
10	0-SE-DR-CAS-101C	17,273	28.00	25.60	27.04	27.28	28.72	1.68	0.72	3.59	0.33
11	at ECST	17,066	30.50	27.28	28.72	27.48	28.92	0.20	-1.58	3.58	0.50
12	1-BS-DR-SEC	19,294	27.00	25.56	27.00	27.54	28.98	1.98	1.98	3.62	2.24
13	1-BS-DR-T27-1 and -2	20,292	27.00	25.15	26.59	27.64	29.08	2.49	2.08	3.61	1.40
14	1-BS-DR-T27-3 and -4	22,478	27.00	25.41	26.85	27.88	29.32	2.47	2.32	3.65	0.34
15	1-BS-DR-FP27-1	22,259	27.00	25.12	26.56	27.75	29.19	2.63	2.19	3.61	0.24
16	at SE Corner of Unit 2	23,869	27.00	25.27	26.71	27.88	29.32	2.61	2.32	3.62	0.41
17	into Maintenance Offices	24,267	27.00	25.24	26.68	27.93	29.37	2.69	2.37	3.63	0.83
18	into Maintenance Offices	24,267	27.00	25.24	26.68	27.93	29.37	2.69	2.37	3.63	0.83
19	into Maintenance Offices	25,053	27.00	25.56	27.00	27.83	29.27	2.27	2.27	3.64	0.47
20	into Maintenance Offices	25,448	27.00	25.36	26.80	27.80	29.24	2.44	2.24	3.63	1.09
21	into Maintenance Offices	25,845	27.00	25.16	26.60	27.76	29.20	2.60	2.20	3.62	1.35
22	into Maintenance Offices	25,650	27.00	25.22	26.66	27.75	29.19	2.53	2.19	3.60	1.90
23	into Maintenance Offices	25,257	27.00	25.16	26.60	27.69	29.13	2.53	2.13	3.62	1.39
24	into Maintenance Offices	24,864	27.00	25.23	26.67	27.65	29.09	2.42	2.09	3.61	1.48
25	into CP Building	22,703	27.00	25.16	26.60	27.22	28.66	2.06	1.66	3.65	1.56
26	into CP Building	22,504	28.00	24.96	26.40	27.18	28.62	2.22	0.62	3.62	0.78
27	into CP Building	22,306	27.00	25.08	26.52	27.17	28.61	2.09	1.61	3.62	1.13
28	1-BS-DR-DVP27-1	15,191	27.00	25.56	27.00	26.73	28.17	1.17	1.17	3.58	0.70
29 - 30	Not Used										
31	into Aux Boiler Room	21,520	27.00	24.95	26.39	27.02	28.46	2.07	1.46	3.59	1.31
32	into Storeroom	21,518	27.00	24.88	26.32	26.97	28.41	2.09	1.41	3.60	1.50
33	1-BS-DR-67	21,319	27.00	24.88	26.32	26.93	28.37	2.05	1.37	3.59	1.55
34	1-BS-DR-66	21,119	27.00	25.22	26.66	26.88	28.32	1.66	1.32	3.59	0.55
35	1-BS-DR-65	20,920	27.00	25.30	26.74	26.87	28.31	1.57	1.31	3.59	1.04
36	1-BS-DR-CS27-3	20,724	27.50	25.24	26.68	26.85	28.29	1.61	0.79	3.58	1.35
37	1-BS-DR-SG28-2	20,133	29.00	24.51	25.95	26.78	28.22	2.27	-0.78	3.60	1.42
38	1-BS-DR-27-3	18,525	27.00	22.75	24.19	26.60	28.04	3.85	1.04	3.59	1.47

Door	Location	Representative Grid Element	Threshold Elevation (ft, MSL/ Plant Datum)	Grid Elevation (ft, NAVD88)	Grid Elevation (ft, MSL/ Plant Datum)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Maximum Flood Depth above Threshold (ft)	Time to Maximum Flood Elevation (hrs)	Maximum Velocity (fps)
39	1-BS-DR-D23-1	18,118	23.00	21.56	23.00	26.59	28.03	5.03	5.03	3.60	1.70
40	1-BS-DR-D23-2	18,118	23.00	21.56	23.00	26.59	28.03	5.03	5.03	3.60	1.70
41	2-BS-DR-DVP27-2	18,538	27.00	25.35	26.79	26.59	28.03	1.24	1.03	3.59	1.28
42	1-BS-DR-73	18,129	26.67	25.13	26.57	26.49	27.93	1.36	1.26	3.59	3.56
43	1-BS-DR-74	17,310	26.67	24.58	26.02	25.47	26.91	0.89	0.24	3.58	5.42
44	1-BS-DR-F27-5 and -7	17,711	27.00	25.34	26.78	26.57	28.01	1.23	1.01	3.59	0.53
45	1-BS-DR-F27-4	17,092	27.00	25.44	26.88	26.58	28.02	1.14	1.02	3.60	0.25
46	1-BS-DR-F27-3 and -6	17,087	27.00	25.41	26.85	26.71	28.15	1.30	1.15	3.58	0.27
47	Above Grade 5'	17,498	32.60	25.64	27.08	26.73	28.17	1.09	-4.43	3.58	0.63
48	Above Grade 5'	17,498	32.60	25.64	27.08	26.73	28.17	1.09	-4.43	3.58	0.63
49	1-BS-DR-WG27-1	17,495	27.50	24.40	25.84	26.73	28.17	2.33	0.67	3.58	0.25
50	1-BS-DR-F27-1 and -2	17,697	27.50	25.87	27.31	26.77	28.21	0.90	0.71	3.58	0.93
51	1-BS-DR-SG28-1	17,686	29.00	24.90	26.34	27.29	28.73	2.39	-0.27	3.59	0.30
52	1-BS-DR-CS-27-1	18,704	27.50	24.80	26.24	27.36	28.80	2.56	1.30	3.59	1.27
53	at ECST	19,741	30.50	26.09	27.53	26.67	28.11	0.58	-2.39	3.59	1.11
54	at Auxiliary Building	19,725	27.60	25.29	26.73	27.08	28.52	1.79	0.92	3.59	0.72
55	1-BS-DR-A27-8	18,915	27.50	24.61	26.05	27.61	29.05	3.00	1.55	3.58	0.45
56	1-BS-DR-A27 1 and -2	19,317	27.50	24.38	25.82	27.60	29.04	3.22	1.54	3.58	0.64
57	at Auxiliary Building	19,923	27.50	25.15	26.59	27.07	28.51	1.92	1.01	3.59	0.76
58	into Clean Change	20,122	27.00	25.25	26.69	27.07	28.51	1.82	1.51	3.59	0.70
59	into Clean Change	20,720	27.00	24.95	26.39	26.95	28.39	2.00	1.39	3.59	1.67

See Figure 2.1-9 for Door Locations

Figure 2.1-1: SPS General Site Location

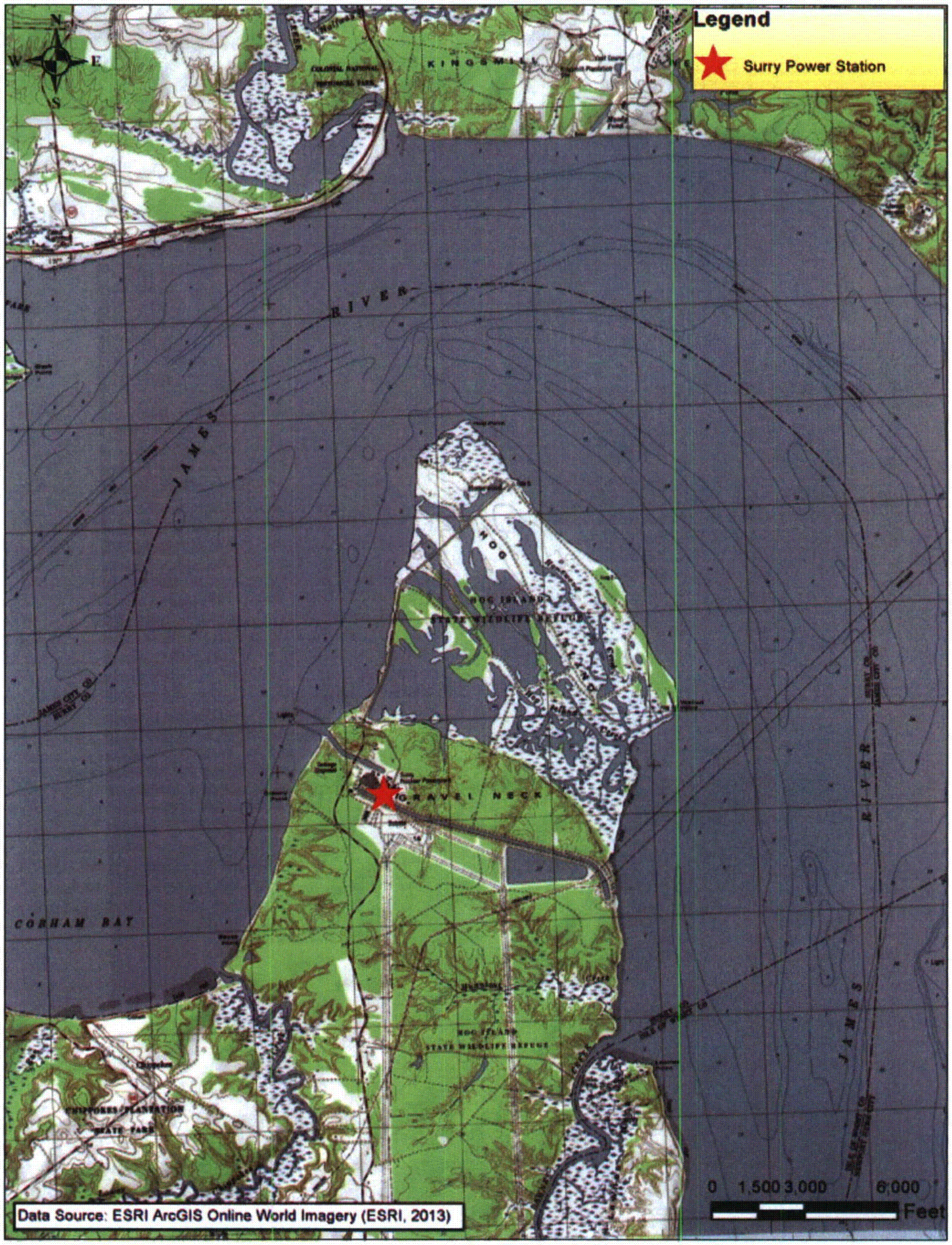
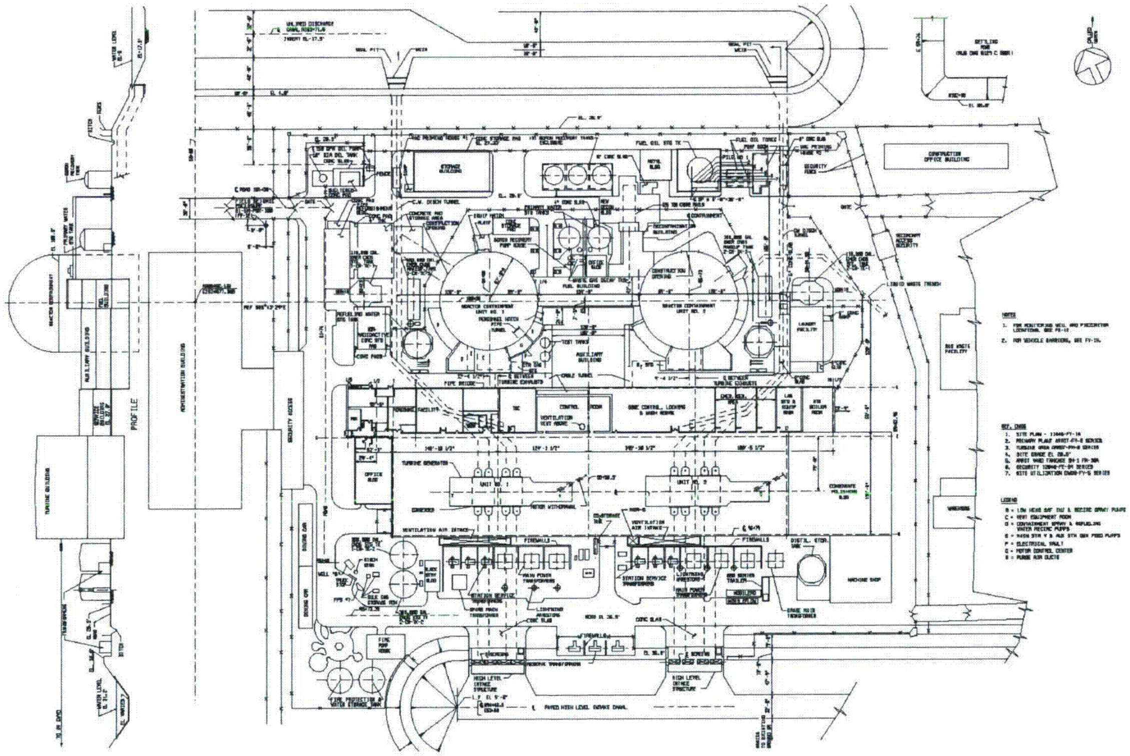


Figure 2.1-2: Surry Plot Plan

— NUCLEAR DESIGN INFORMATION PORTAL —



C:\usr\dgn\S1501002DGNmf1

Figure 2.1-3: FLO-2D Model Layout

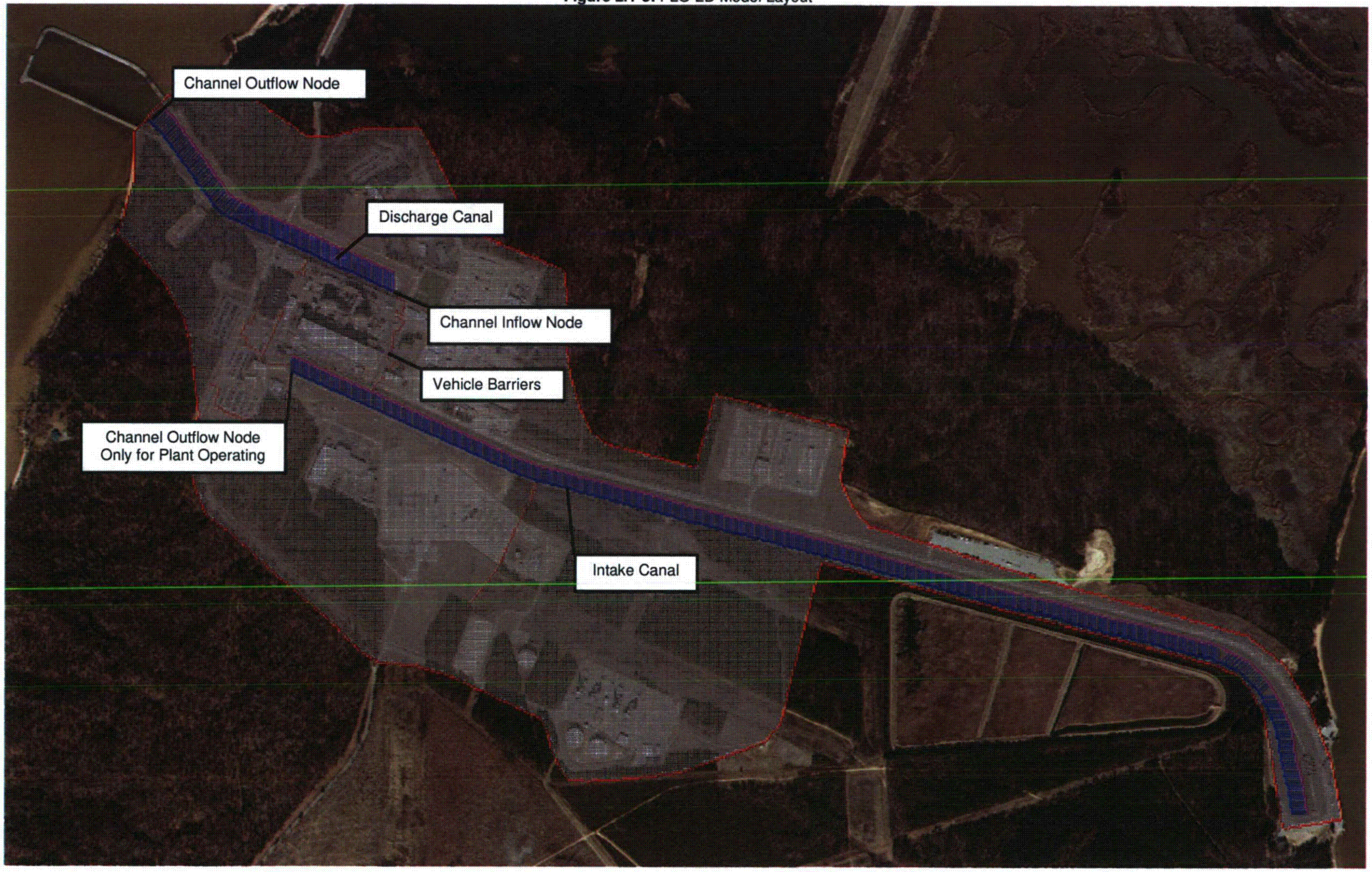


Figure 2.1-4: Grid Element Manning's Coefficient Rendering

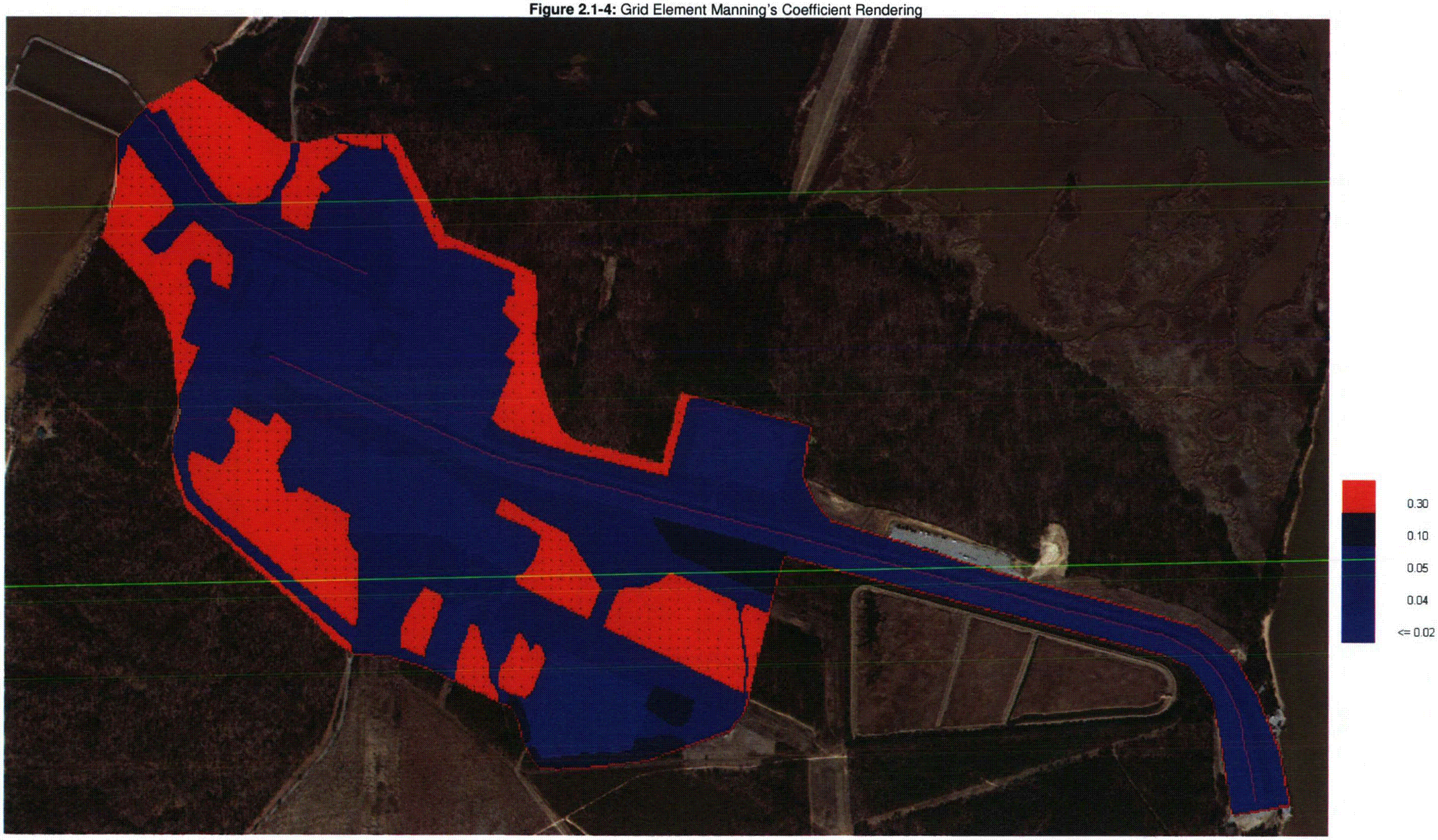


Figure 2.1-5: HMR-51/52 - Six-Hour Incremental Hyetograph – End Loading Temporal Distribution

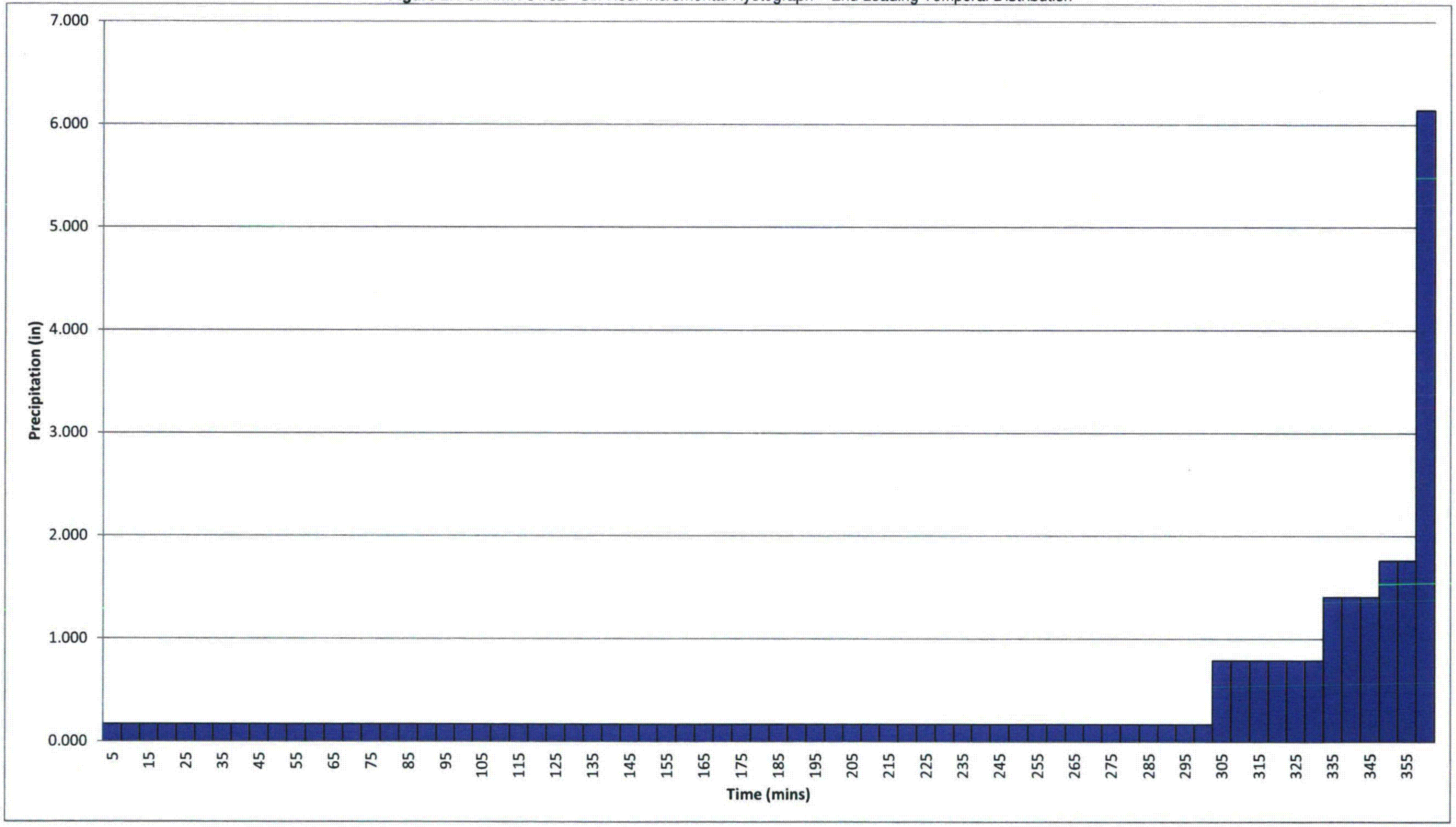


Figure 2.1-6: Flow Chart Showing Major Steps Involved in Calculating the Site-Specific LIP

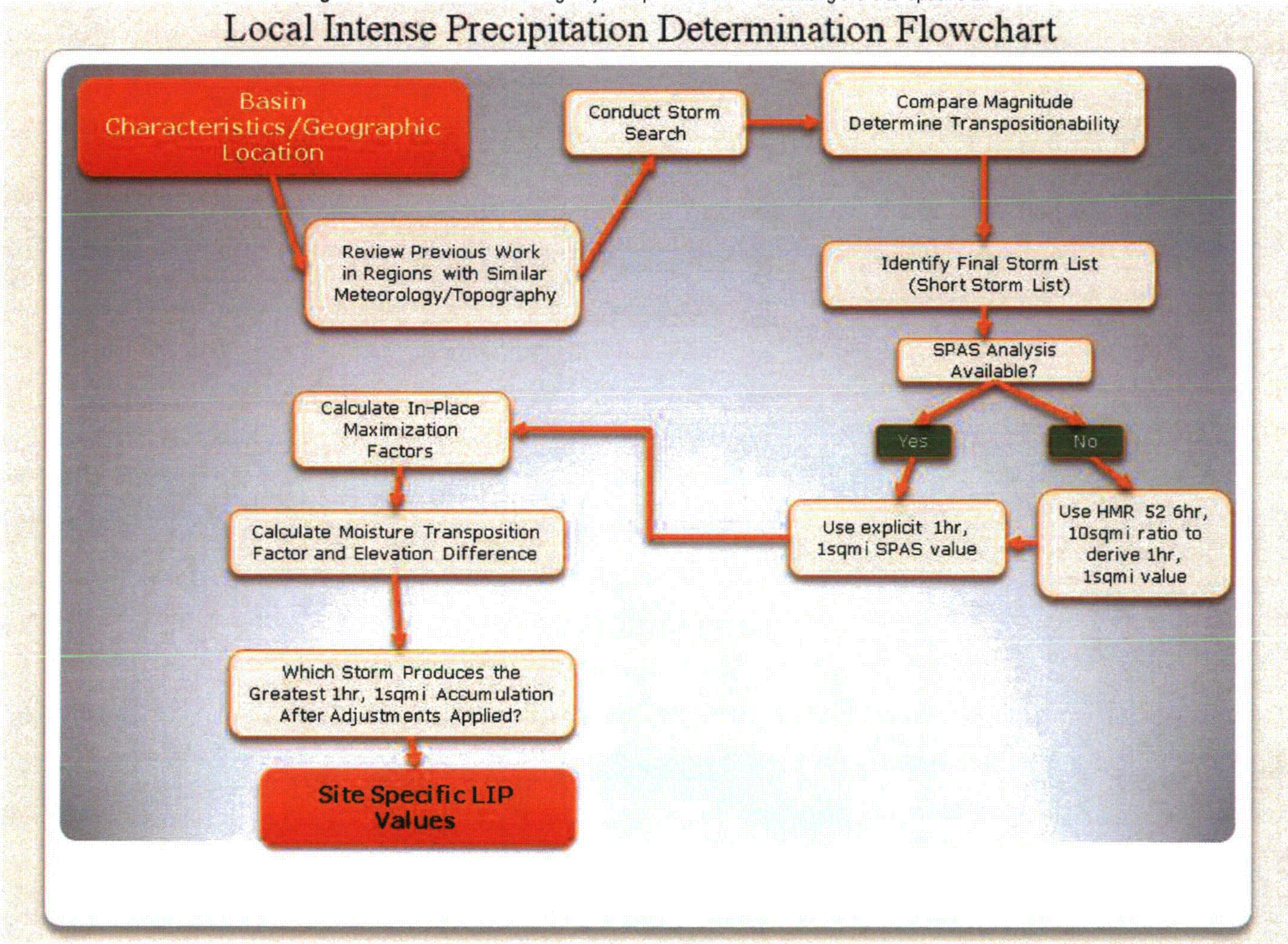


Figure 2.1-7: Storm Locations Used for Site-Specific LIP Development in Relation to SPS

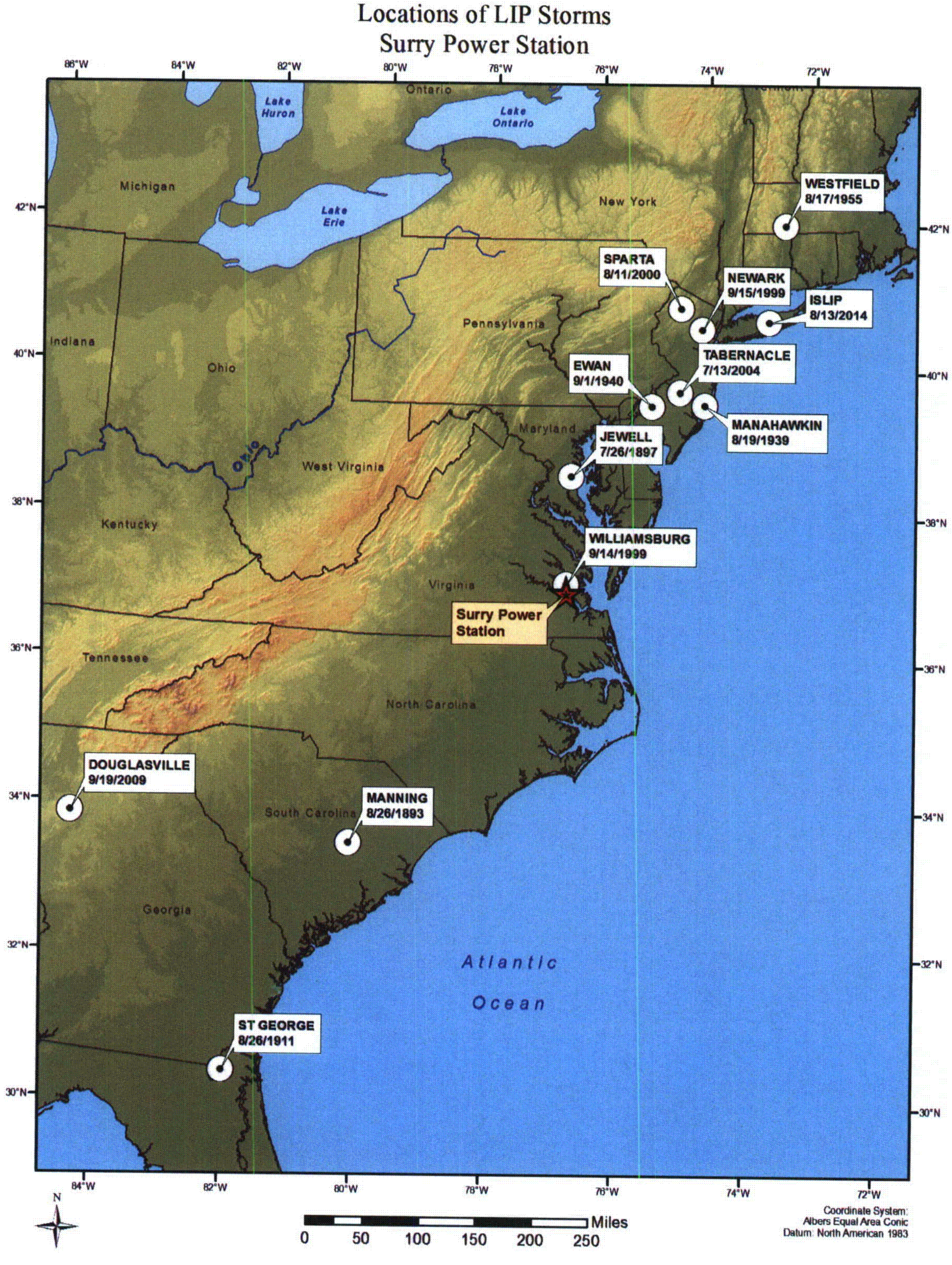


Figure 2.1-8: Site-Specific Meteorology Study-Six-Hour Incremental Hyetograph – Critical Loading Temporal Distribution

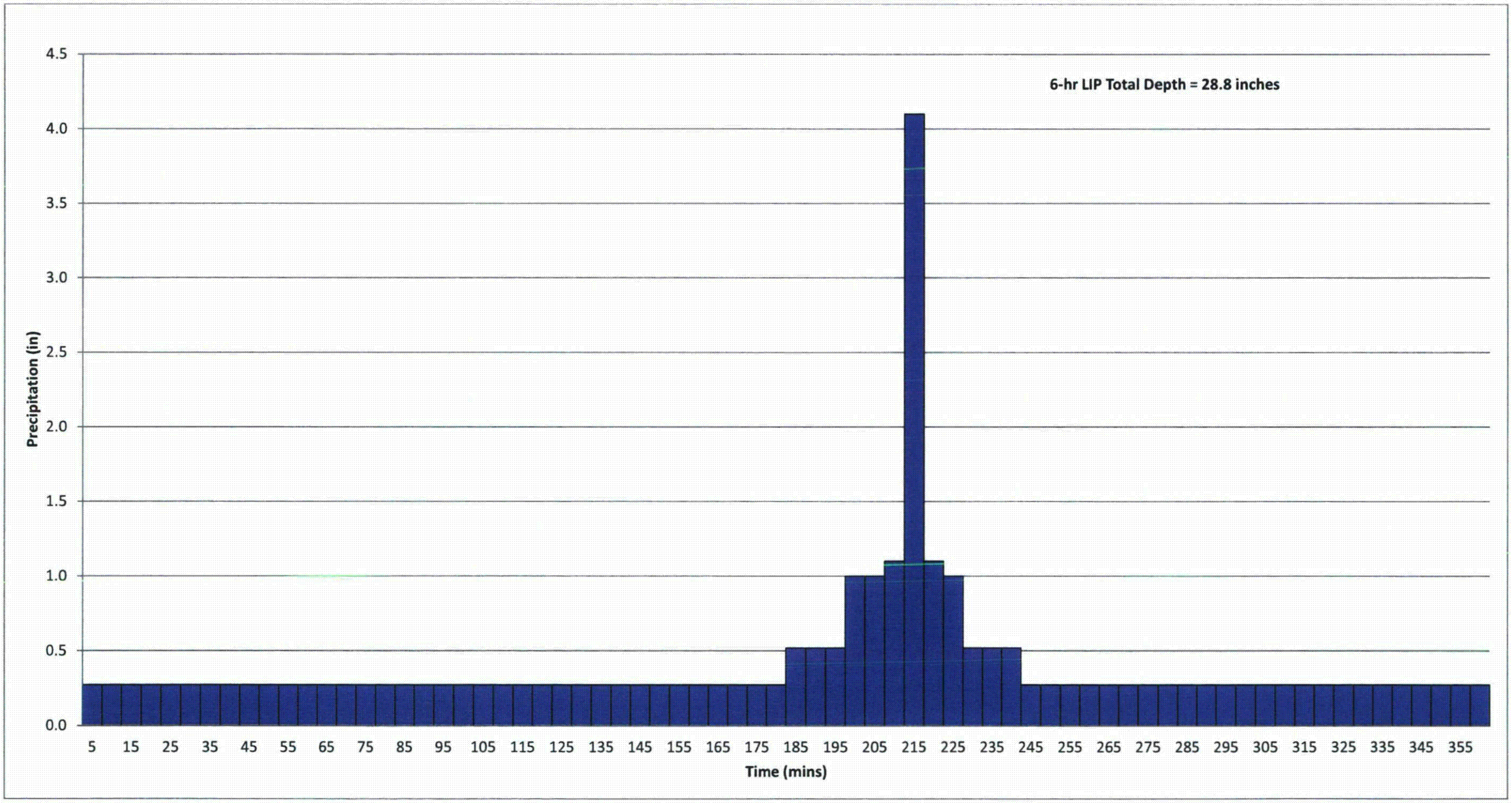


Figure 2.1-9: Numbered Door Locations

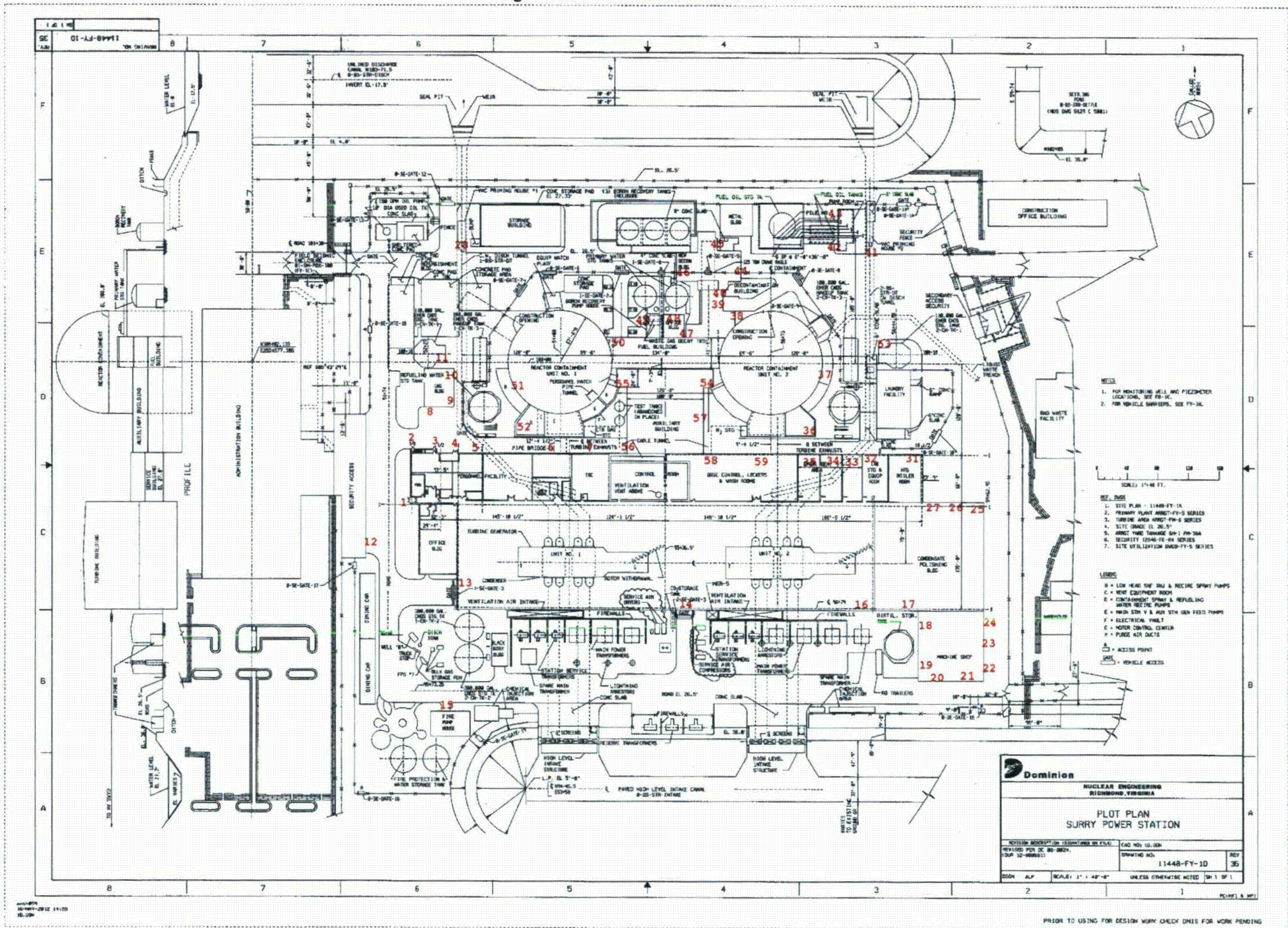


Figure 2.1-10: Ground Surface Elevations

Figure 2.1-11: HMR-51/52 Simulation-Grid Element Maximum Water Surface Elevations – Case 1 Scenario (feet, MSL)

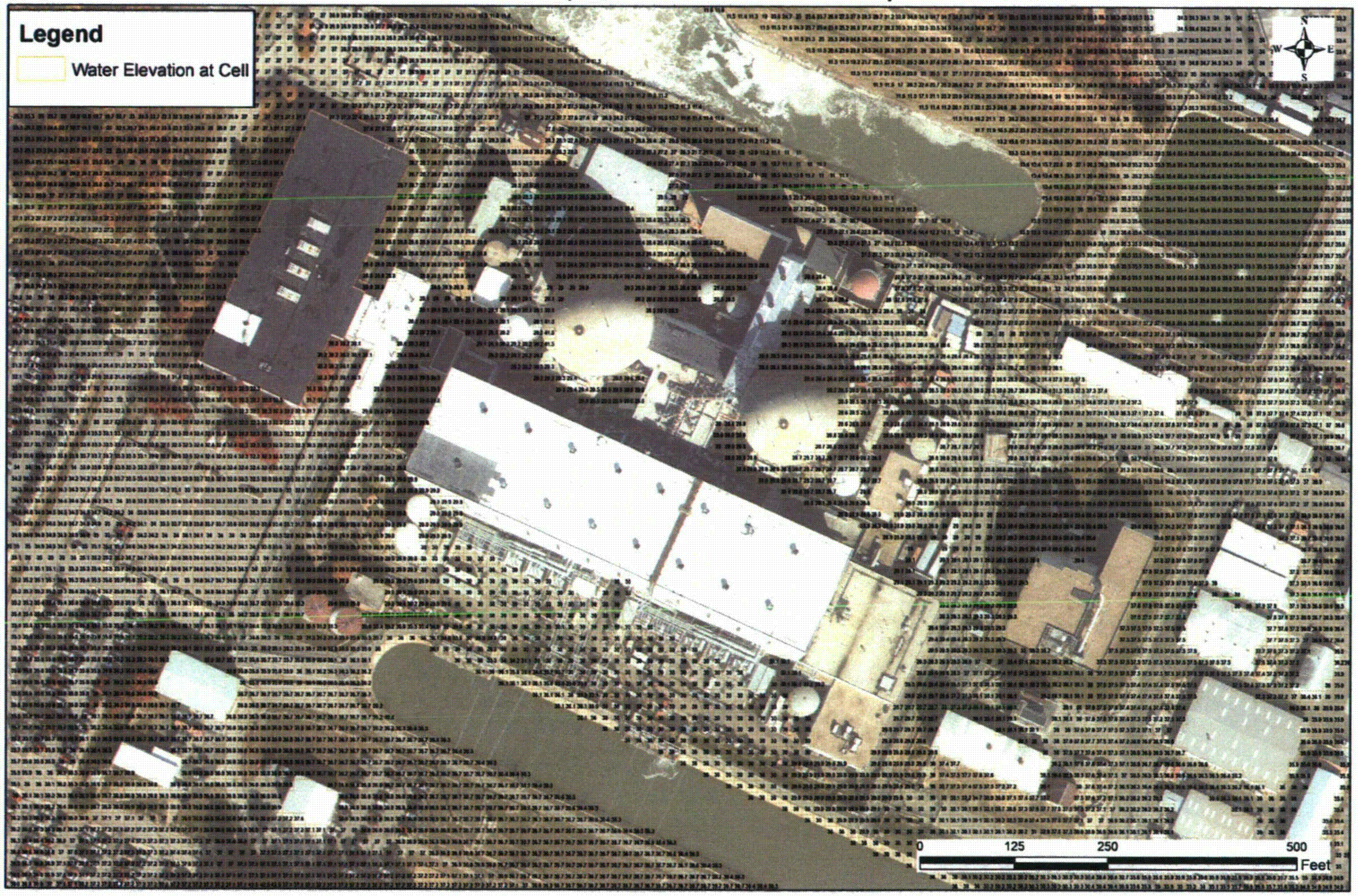


Figure 2.1-12: HMR-51/52 Simulation-Grid Element Maximum Water Surface Elevations – Case 2 Scenario (feet, MSL)

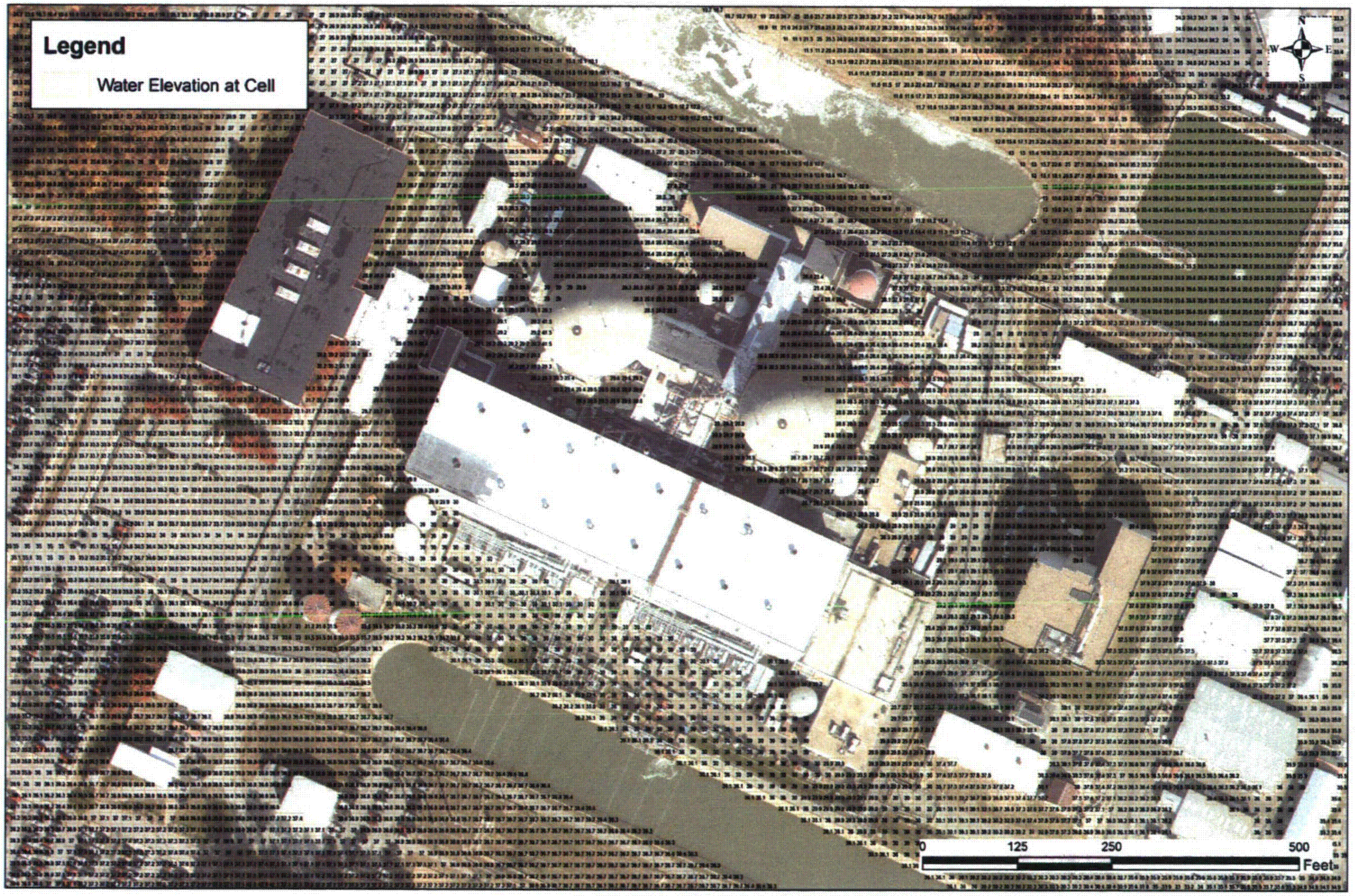


Figure 2.1-13: Site-Specific Meteorology Study Simulation-Grid Element Maximum Water Surface Elevations – Cases 1 and 2 (feet, MSL)

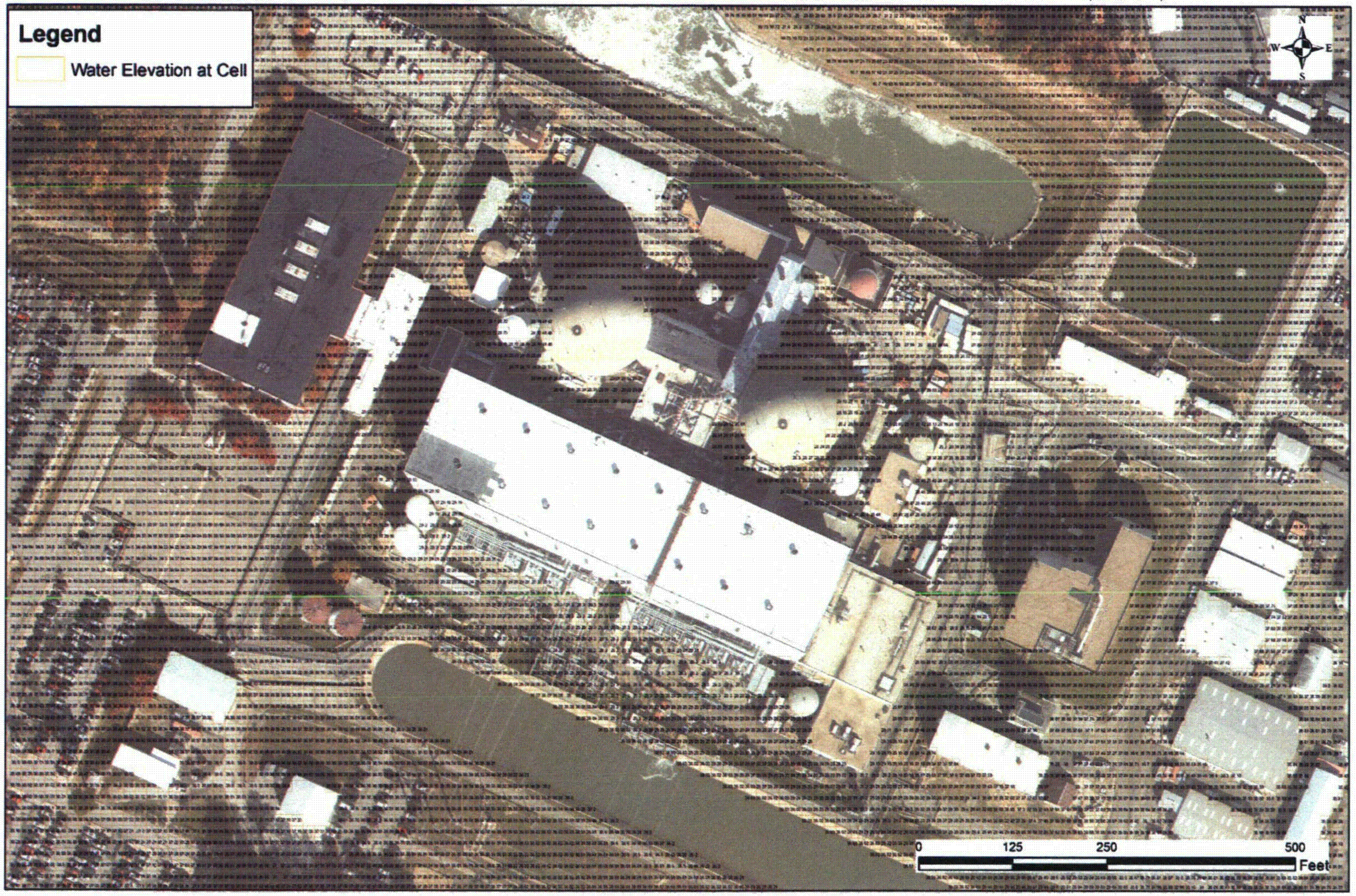
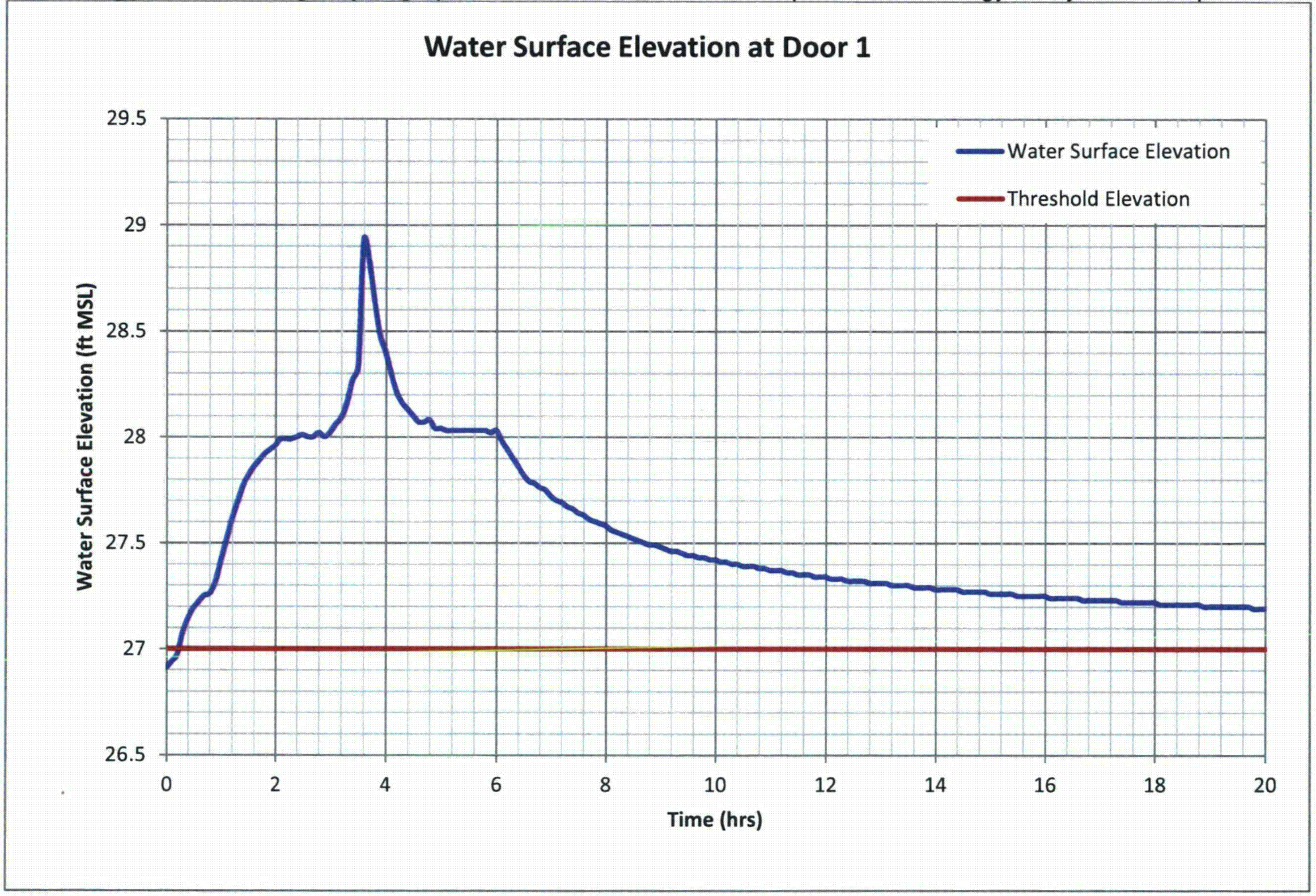
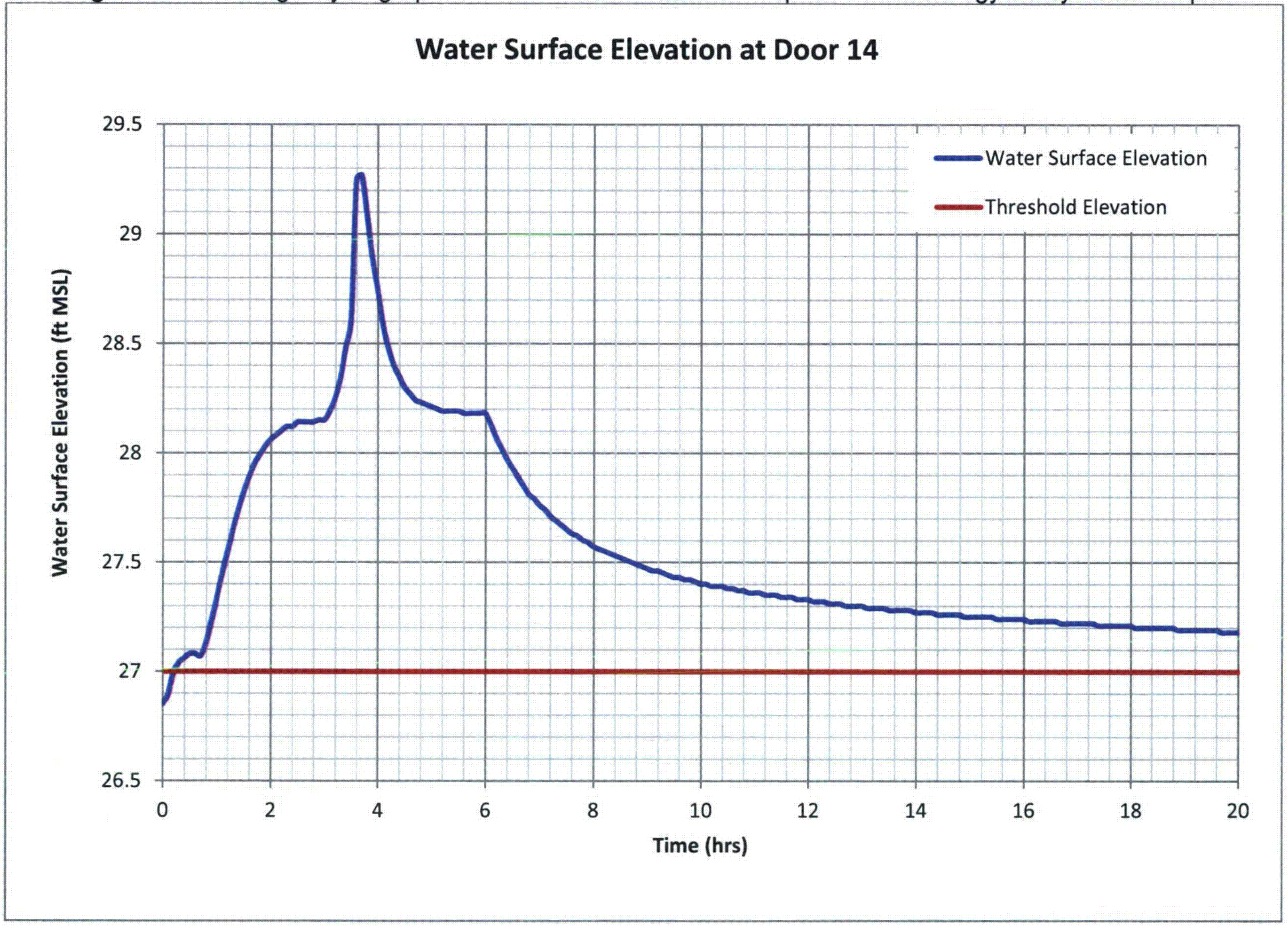


Figure 2.1-14: Stage-Hydrograph for Door # 1 Based on Site-Specific Meteorology Study Rainfall Input



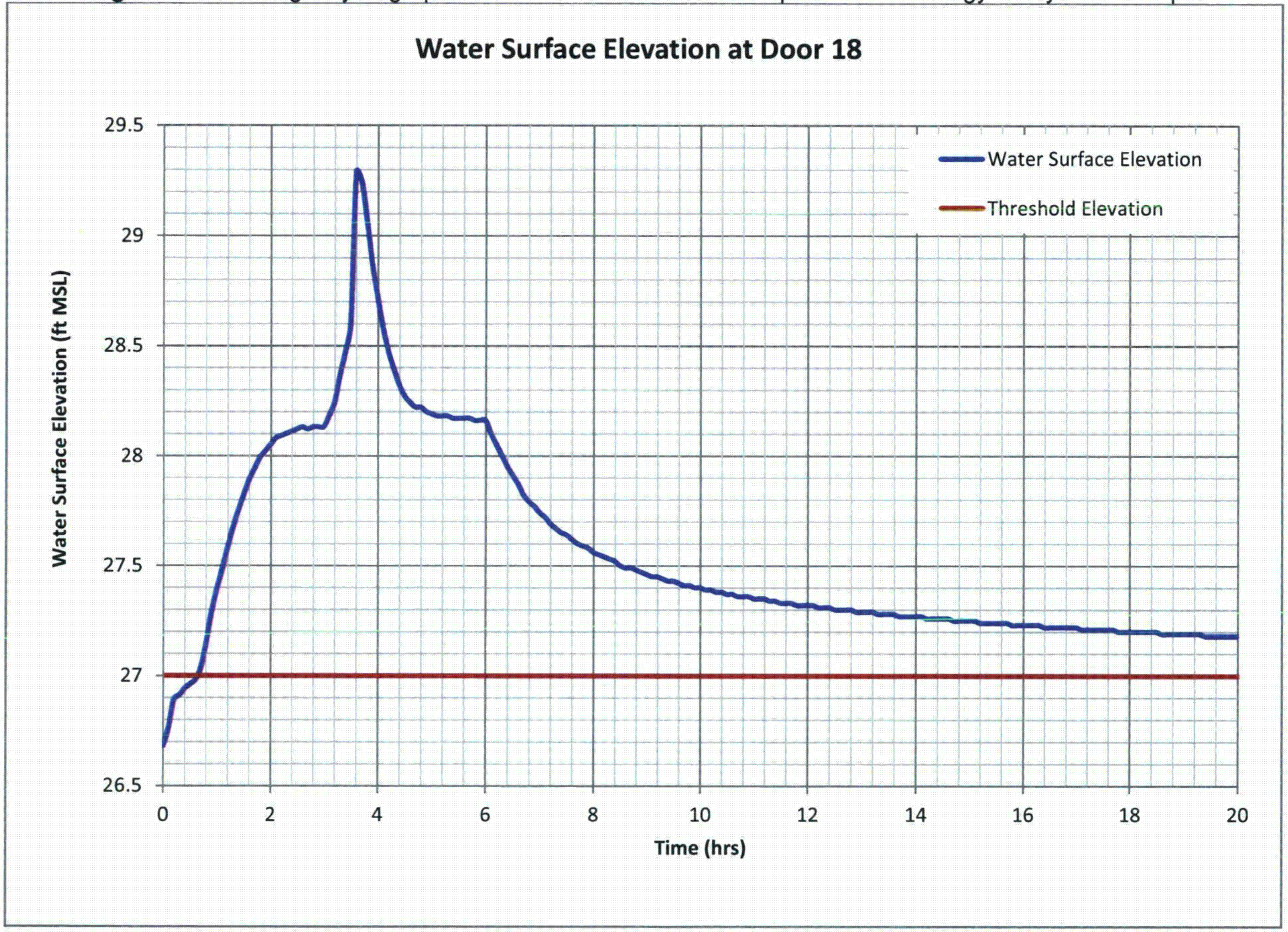
Note: Door locations are shown in Figure 2.1-9.

Figure 2.1-15: Stage-Hydrograph for Door # 14 Based on Site-Specific Meteorology Study Rainfall Input



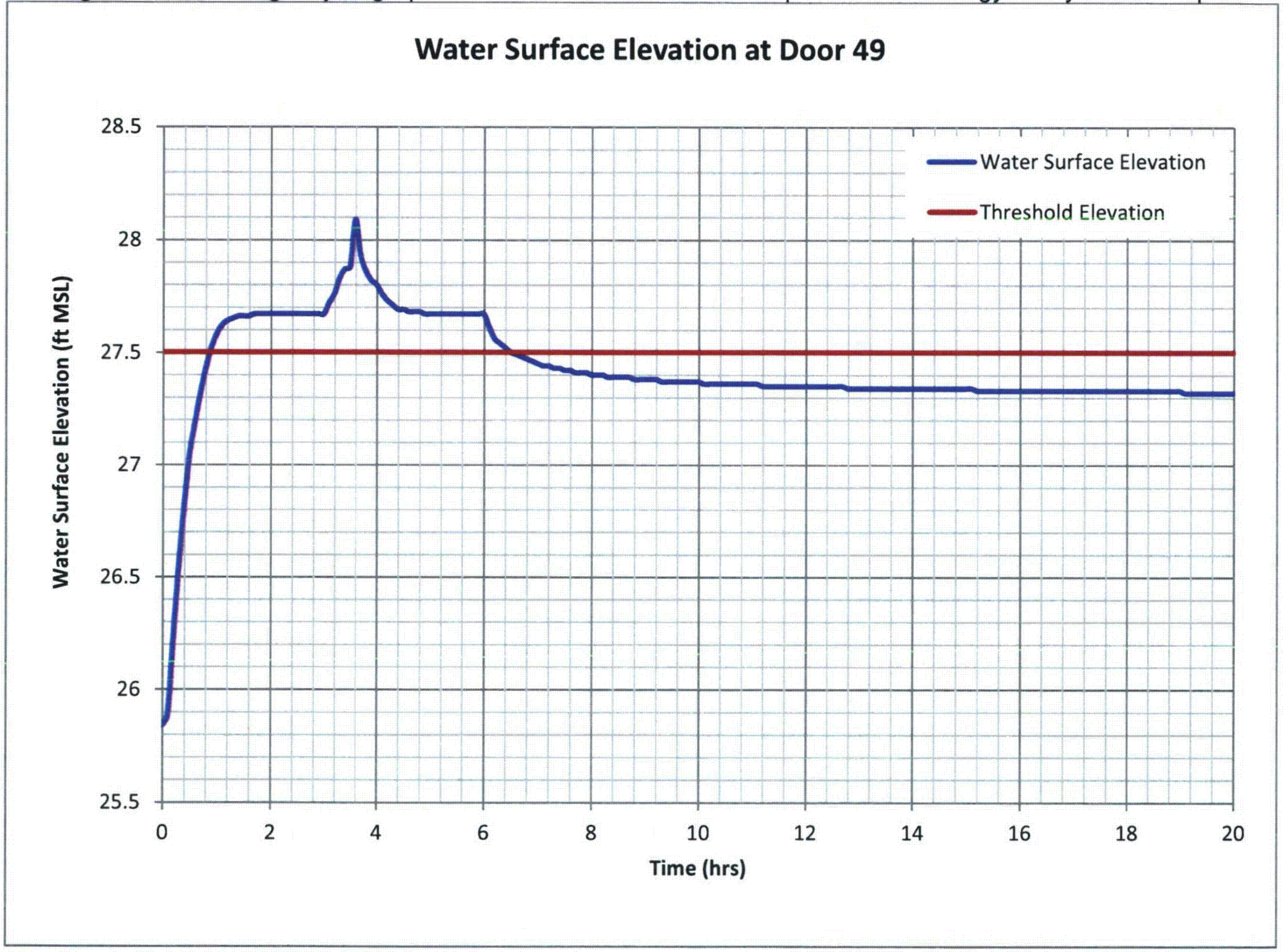
Note: Door locations are shown in Figure 2.1-9.

Figure 2.1-16: Stage-Hydrograph for Door # 18 Based on Site-Specific Meteorology Study Rainfall Input



Note: Door locations are shown in Figure 2.1-9.

Figure 2.1-17: Stage-Hydrograph for Door # 49 Based on Site-Specific Meteorology Study Rainfall Input



Note: Door locations are shown in Figure 2.1-9

2.2. Probable Maximum Flood on Rivers and Streams

This section addresses the potential for flooding at SPS due to the Probable Maximum Flood (PMF) on the James River. NUREG/CR-7046 defines the PMF as: "the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive Hydrometeorological application of the probable maximum precipitation (PMP) and other hydrologic factors favorable for the maximum flood runoff such as sequential storms and snowmelt." (NRC, 2011).

SPS is located on the Gravel Neck peninsula along the south bank of the James River. The James River and its watershed have the potential to contribute to flooding at SPS.

2.2.1. Method

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the PMF on the James River near SPS and resultant water surface elevations at SPS. The HHA assumptions adopted for the PMF evaluation are: (1) incorporation of an antecedent storm prior to the full PMF; and (2) application of nonlinearity adjustments to unit hydrographs. The evaluation used the following steps:

1. Delineate the watershed for the James River.
2. Calculate the all-season PMP based on Hydrometeorological Report (HMR) Numbers 51 and 52 (NOAA, 1978 and NOAA, 1982) using the BOSS HMR52 computer program.
3. Check the seasonal PMP using HMR-53 (NOAA, 1980). Calculate snow melt potential in the watershed using the energy budget method. Add the snow melt to the seasonal rainfall to calculate the largest combined seasonal PMP and snow melt value (cool-season PMP).
4. Develop rainfall-runoff model using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HMS) computer model. The Snyder Unit Hydrograph method and initial and constant losses were used to simulate the hydrology of the watershed.
5. Calibrate and verify HEC-HMS rainfall-runoff model using observed U.S. Geologic Survey (USGS) stream flow data by optimization of input parameters: Basin Lag Time (tp) and Peaking Coefficient (Cp), Initial Loss, and Constant Loss.
6. Perform PMF hydrologic simulations with HEC-HMS, including recommended non-linearity adjustments.
7. Calculate the PMF elevation on the James River near SPS using the USACE HEC Riverine Analysis System (RAS) computer model with the calculated HEC-HMS PMF hydrographs for the James River.

2.2.2. Results

2.2.2.1. Delineate Watershed

The James River watershed was delineated at SPS. The watershed was subdivided into three subwatersheds based on USGS stream gage locations (USGS, 2012a) and Digital Elevation Model (DEM) data (USGS, 1993). Hydrologic Unit (HU) GIS data included in the National Hydrography Dataset (NHD) provided by USGS (USGS, 2012b) was used as a supplemental reference to establish subwatershed boundaries. Two of the subwatersheds were gaged (e.g., a USGS stream flow gage was present at the subwatershed outlet) and the third subwatershed was ungaged. The two gaged subwatersheds, denoted as the Upper James River and the Appomattox River, are 6,753 square miles and 1,334 square miles in area, respectively. The third ungaged subwatershed, Lower James River subwatershed, is 1,434 square miles in area. Calculated watershed areas are shown in Table 2.2-1. The overall James River watershed is calculated to be 9,521 square miles. The subwatersheds are depicted in Figure 2.2-1.

2.2.2.2. Calculate the All-Season PMP

The all-season PMP was calculated for the contributory watershed using the methodology of HMR-51 and HMR-52. The BOSS HMR52 computer program was used for the calculations. Inputs included the basin boundary coordinates, initial storm orientation, depth-area-duration values, and storm temporal order. The final HMR-52 calculated storm orientation for the maximum all-season PMP was 285 degrees. The temporal distribution of 6-hour increments for the all-season 72-hour PMP was based on HMR-52 recommendations: 12, 10, 8, 6, 4, 2, 1, 3, 5, 7, 9 and 11, with 1 being the maximum rainfall depth increment and 12 being the minimum rainfall depth increment.

The maximum duration of 72-hours used in HMR-51 and HMR-52 was conservatively adopted for the evaluation. The controlling PMPs were determined to be the all-season 72-hour PMP with a total rainfall depth of 15.1 inches for the overall watershed, 15.5 inches for the James River subwatershed near Richmond, 15.9 inches for the Appomattox subwatershed near Dinwiddie and 12.6 inches for the lower James River subwatershed at SPS.

2.2.2.3. Calculate the Cool-Season PMP

The seasonal variation of the PMP was evaluated in combination with snowmelt to ensure cool-season PMP combined with snowmelt would not lead to a more severe PMF. Monthly PMPs were calculated using HMR-53 (NOAA, 1980) to develop the seasonal PMP depths for the 72-hour, 10 square mile storm by month of occurrence. Statistical monthly snow cover records at the nine stations (Millgap, Gathright, Hot Springs ,Covington Filter, Blacksburg, Lexington, Bremo Bluff, Farmville and Williamsburg) were obtained from the NCDC (NCDC, 2012a). The records indicate that there is snow cover possible between November and April.

The energy budget method was used to determine snow melt for the seasonal PMP, assuming an infinite snowpack, which is presented in USACE Engineering Manual 1110-2-1406 (USACE, 1998). For open or partly forested areas (less than 80 percent) the generalized equation for rain-on-snow scenario (USACE, 1998) was used. The overall watershed at SPS is largely forest-covered with a calculated forest cover of 72-percent. Each subwatershed was assigned the maximum wind speed and dewpoint derived from one selected climate station within its corresponding watershed area. The snowmelt calculation for the cool-season months is presented in Table 2.2-2.

Snowmelt was also calculated for each subwatershed area based on observed extreme snowpack data. The snowpack on the ground was converted to an equivalent depth of water based on a snow-water equivalent (SWE) ratio. Observed, historic SWE data is not available for the James River watershed. In lieu of site-specific data, snow densities of 0.25 for November to January, 0.30 for February and 0.35 for March and April were assumed based on typical snowpack densities (NRCS, 2012). The maximum snowpack from the eight stations located within the James River watershed (see Figure 2.2-2) were used. The snowpack melt calculation is presented in Table 2.2-3.

The snowmelt (M_0) calculated based on historical snowpack data are consistently less than the values (M_1) estimated by the Energy Budget method, comparing the results presented in Tables 2.2-2 and 2.2-3. Therefore, the snowpack melt data (M_0) were conservatively used for calculating combined seasonal PMPs with snowmelt. The combined seasonal PMP for each subwatershed was calculated for a particular cool-season month as the total of the calculated seasonal PMP for that month and the extreme snowpack melt for the same month, see Table 2.2-4.

2.2.2.4. Select the Controlling PMP

The controlling PMP for calculating the PMF on James River at SPS is the all-season 72-hour PMP (see Table 2.2-4). This is based on the all-season PMP total rainfall being larger than the sum of snowmelt and the seasonal PMP, and also upon the relative (higher) intensity of the rainfall-only all-season PMP in comparison to the snowmelt-influenced cool-season PMP. The snowmelt portion of the cool-season PMP is calculated in inches per day and distributed evenly over the 6-hr PMP increments used by HMR-52.

The 72-hour PMP calculation was performed in accordance with HMR-51 and HMR-52 (NOAA, 1978 and NOAA, 1982, respectively). The peak intensity is at the 7th 6-hour period of the total 72-hour duration. The subwatershed average PMP 72-hour values range from 12.6 to 15.9 inches. An antecedent storm, which is 40-percent of the 72-hour PMP, followed by 3 dry days, was simulated prior to the start of the full PMP. The final PMP input hyetograph for HEC-HMS simulations is shown in Figure 2.2-3.

2.2.2.5. Develop HEC-HMS Model

A HEC-HMS model was developed for the three delineated subwatersheds. The general approach to modeling the hydrology of the watershed was to use observed USGS stream gage data (USGS, 2012c) and observed precipitation data (NCDC, 2012b) to calibrate the HEC-HMS model through optimization of the following input parameters: (1) Snyder Peaking Coefficient, (2) Snyder Lag Time, (3) initial loss, and (4) constant loss rate (USACE, 2000). Model calibration is the process of selecting and refining HEC-HMS input parameters to produce a simulated hydrograph for a given flood that shows good agreement with an observed hydrograph for the same flood. Model verification is the process of testing the calibrated HEC-HMS input parameters, without further manipulation or refinement, to demonstrate that a simulated flood hydrograph, for a given flood, shows good agreement with the observed flood hydrograph. The parameters estimated or selected were considered initial inputs that were subsequently adjusted during HEC-HMS optimization to calibrate the model to observed data.

The Upper James River subwatershed was calibrated with USGS gage 02037500 and the Appomattox River subwatershed was calibrated with USGS gage 02041650 (see Figure 2.2-1). These two subwatershed make up 85-percent of the entire contributory watershed to SPS. For each gaged subwatershed, three storms were used for calibration and three storms were used for verification. The input parameters for the ungaged portion of the watershed were estimated based on the calibrated / verified parameters for the gaged portion of the watershed (USACE, 2000). The constant loss rate for the ungaged subwatershed was calculated using infiltration rates for each soil type (e.g., hydrologic soil group). The representative infiltration rate for each soil type was back-calculated using the verification results for the gaged subwatersheds. A summary of the calibration and verification results for each subwatershed are provided in Tables 2.2-5 and 2.2-6, respectively. Comparison of observed and modeled peak flow shows acceptable agreement with observed peak flows with the greatest differential in peaks for the calibration storms being 10-percent. Optimization of the parameters for the verification storms resulted in a maximum peak differential of about 30-percent. Model simulations over-predicted observed peak flows for calibration and verification floods to incorporate additional conservatism into the analysis. See Table 2.2-7 for finalized parameters, which are based on the results of the verification storms.

One dam (the Brasfield Dam) was included in the HEC-HMS model because it is located immediately upstream of the stream gage in the Appomattox River subwatershed and thus directly impacts the measurements at the stream gage. The dam was modeled using the outflow structure method in HEC-HMS. An elevation versus area curve was developed based on dam characteristics, a 2011 bathymetric survey of the reservoir, and an USACE Phase I Report and existing USGS topographic data (ARWA, 2011; USACE, 1978, and ESRI, 2011, respectively). The spillway crest elevation was used for the initial condition for the PMF. Weir coefficients were chosen based on the spillway type (USACE, 2010). Low level outlets and gates were not modeled (i.e., assumed to be closed or otherwise inoperable).

2.2.2.6. PMF Simulations

Nonlinearity adjustments were made to the calculated Snyder unit hydrographs (Figure 2.2-4) to include a 20-percent increase in peak discharge of the unit hydrograph, a 33-percent reduction in time to peak of the unit hydrograph, and adjustments to the falling limb of the unit hydrograph to conserve the volume under the unit hydrograph (NRC, 2011).

Base flow was calculated for each gaged subwatershed based on USGS Surface-Water Monthly Statistics data for USGS Stream Gages (USGS, 2012c). Base flow was calculated for the ungaged subwatershed using the base flow per square mile (cfsm) of the surrogate gaged subwatershed. The highest of the all season (June to October) monthly averaged flow for each stream gage for the period of record was used. Base flows of 900 cfs, 5,600 cfs, and 1,200 cfs were used at the Appomattox, Upper James, and ungaged subwatersheds, respectively.

Using the calculated parameters and the adjusted unit hydrograph, the PMF was simulated using the controlling all-season PMP. The PMF was calculated by using the 72-hour PMP calculated for the Upper James River subwatershed, the Appomattox River subwatershed, and the Lower James River subwatershed (see Table 2.2-8). The calculated PMF peak flow on the James River without non-linearity adjustment was 700,400 cfs (Figure 2.2-5). The calculated PMF peak flow on James River with non-linearity adjustments was 867,000 cfs (Figure 2.2-6).

2.2.2.7. Calculate PMF Elevation

A HEC-RAS hydraulic computer model was developed for an approximately 100-mile-long reach of the James River. The upstream limit is located approximately 50 miles upstream of SPS, and the downstream limit is near the confluence of James River and the Atlantic Ocean. A total of 28 cross sections were used and were based on bathymetric data (NOAA, 2012) and DEM data (USGS, 1993) . The locations of the cross-sections are shown in Figure 2.2-7. The flows from each of the three subwatersheds were input separately; the flow from the Upper James River subwatershed was input as the upstream boundary condition, and the other two subwatershed flows were input as lateral inflow at various stations. The downstream boundary condition was assigned with a constant stage hydrograph coincident with the Mean Higher High Water (MHHW) elevation based on NOAA's datum analysis at the Chesapeake Bay Bridge Tunnel (Station 8638863) tide gage location (NOAA, 2013): approximately elevation 3.1 feet MSL.

The HEC-RAS model was not calibrated because there are no observed stream gage data available on the main stem of the James River near or downstream of SPS. Manning's roughness coefficients were conservatively determined based on the guidelines provided by the HEC-RAS reference manual (USACE, 2010), land use data of the area, and the current orthoimagery. Manning's coefficient for the channel was conservatively selected to be 0.04. The Manning's coefficient for the overbank was conservatively selected to be 0.10.

The calculated hydraulic profile along the James River is shown in Figure 2.2-8. The peak PMF stage was calculated to be 12.1 feet MSL which is approximately 14.4 feet below the site grade of 26.5 feet MSL.

2.2.3. Conclusions

The PMF in the James River at SPS is estimated at 867,000 cfs. The peak PMF water surface elevation at SPS is 12.1 feet MSL, which is approximately 14.4 feet below the site grade of 26.5 feet MSL. The minimum difference between the protected level of 24.0 feet, MSL at the circulating water intake structure and the PMF elevation is 11.9 feet.

Based on the re-evaluated peak PMF elevation on James River at SPS, the peak PMF water surface elevation from the James River flood is below the plant grade elevation.

2.2.4. References

2.2.4-1 ARWA, 2011. Appomattox River Water Authority, Reservoir Bottom Elevations - Lake Chesdin, Appomattox River, Petersburg, Virginia, Prepared by Ocean Surveys, Inc., July 2011.

2.2.4-2 ESRI 2011. ESRI ArcGIS Online World Topographic Map service, Published February 2011 by ESRI ARCIMS Services.

2.2.4-3 NCDC, 2012a. Snow Climatology Data, National Climatology Data Center, National Oceanic and Atmospheric Administration (http://www.ncdc.noaa.gov/ussc/index.jsp, data accessed in December 2012).

2.2.4-4 NCDC, 2012b. National Climatology Data Center (NCDC) by National Oceanic and Atmospheric Administration (NOAA) (http://www.ncdc.noaa.gov/oa/ncdc.html – data downloaded in November and December 2012).

2.2.4-5 NRC, 2011. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants, NUREG/CR-7046, United States Nuclear Regulatory Commission (NRC), November 2011

2.2.4-6 NRCS, 2012. Natural Resources Conservation Service Oregon Snow Survey Program (http://www.or.nrcs.usda.gov/Snow/about/swe.html).

2.2.4-7 NOAA, 1978. Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, Hydrometeorological Report No.51 (HMR-51) by U. S. Department of Commerce & USACE, National Oceanic and Atmospheric Administration, June 1978.

2.2.4-8 NOAA, 1980. Seasonal Variation of 10-square-mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No.53 (HMR-53) by US Department of Commerce and US Nuclear Regulatory Commission, National Oceanic and Atmospheric Administration, April 1980.

2.2.4-9 NOAA, 1982. Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No.52 (HMR-52) by U. S. Department of Commerce & USACE, National Oceanic and Atmospheric Administration, August 1982.

2.2.4-10 NOAA, 2012. U.S. Estuarine Bathymetric Data Sets, National Ocean Service, National Oceanic and Atmospheric Administration (http://estuarinebathymetry.noaa.gov/ - data downloaded in October 2012).

2.2.4-11 NOAA, 2013. Tides and Currents, National Oceanic and Atmospheric Administration – data accessed on 11-19-2013 (http://tidesandcurrents.noaa.gov/map/)

2.2.4-12 USACE, 1978. U.S. Army Corps of Engineers, Norfolk District, Phase I Inspection Report, National Dam safety Program – George F. Brasfield Dam, March 1978.

2.2.4-13 USACE, 1998. Runoff from Snowmelt, EM-1110-2-1406 by U.S. Army Corps of Engineers, March 1998.

2.2.4-14 USACE, 2000. U.S. Army Corps of Engineers Hydrologic Engineering Center, Hydrologic Modeling System HEC-HMS, Technical Reference Manual, March 2000.

2.2.4-15 USACE, 2010. HEC-RAS River Analysis System, Hydraulic Reference Manual, v.4.1, January, 2010.

2.2.4-16 USGS, 1993. Data User Guide 5 – Digital Elevation Models, U.S. Department of Interior, U.S. Geological Survey, 1993 (DEM source data download from http://www.webgis.com/terr_us1deg.html), downloaded October 2012.

2.2.4-17 USGS, 2012a. U.S. Geological Survey (USGS) Instantaneous Data Archive (http://ida.water.usgs.gov/ida/ - data downloaded in October 2012).

2.2.4-18 USGS, 2012b. National Hydrography Dataset, U.S. Geological Survey (http://nhd.usgs.gov/, data downloaded on 10-30-2012

2.2.4-19 USGS, 2012c. U.S. Geological Survey (USGS) Surface-Water Monthly Statistics for the Nation (http://waterdata.usgs.gov/nwis/monthly/?referred_module=sw – data downloaded in December 2012).

Table 2.2-1: Summary of Watershed Delineation

No.	Watershed Name	Delineation Point / USGS Stream Gage Location			Basin Area (mi^2)	Percent of Total Area
		Lat. (deg)	Long. (deg)	Gage No.		
Sub 1	James River Subwatershed near Richmond	37.563N	77.547W	02037500	6,753	71%
Sub 2	Appomattox River Subwatershed near Dinwiddie	37.225N	77.475W	02041650	1,334	14%
Sub 3	Ungaged Lower James River Subwatershed at SPS	37.166N	76.699W	*Ungaged*	1,434	15%
Overall	James River Watershed at SPS			--	9,521	100%

Table 2.2-2: Snowmelt Calculation – Energy Budget Method

Basin Name	Month	Basin wind Coef.	Wind Velocity	Rate of Precip (Avg)	Temp of Sat. Air	Daily Snowmelt	72-hr Snowmelt
		k	v	P_r	T	$M = (0.029+0.0084kv+0.007P_r) \cdot (T-32)+0.09$	$M_1 = M \cdot 3$ days
		(-)	(miles / hour)	(in/day)	(°F)	(in/day)	(in)
Subwatershed @ Richmond	Jan	0.3	33	2.7	52	2.6	7.9
	Feb	0.3	31	2.7	48	2.1	6.3
	Mar	0.3	35	2.8	53	3.0	8.9
	Apr	0.3	33	3.0	56	3.3	9.8
	Nov	0.3	31	3.7	54	3.1	9.2
	Dec	0.3	31	2.9	50	2.4	7.1
Subwatershed @ Dinwiddie	Jan	0.3	22	2.7	57	2.6	7.9
	Feb	0.3	23	2.7	52	2.2	6.5
	Mar	0.3	24	2.8	60	3.1	9.4
	Nov	0.3	22	3.7	60	3.2	9.6
	Dec	0.3	22	3.0	60	3.1	9.2
Subwatershed @ SPS	Jan	0.4	26	2.2	58	3.5	10.6
	Feb	0.4	27	2.2	57	3.5	10.5
	Mar	0.4	30	2.3	62	4.4	13.3
	Nov	0.4	26	3.0	65	4.6	13.9
	Dec	0.4	29	2.3	61	4.2	12.6

Table 2.2-3: Snowmelt Calculation for Subwatersheds based on Snowpack Data

Basin	Month	Observed Extreme Snowpack	Snow-Water Equivalent Ratio	Snowpack Melt	Reference Stations
		D_s	SWE	$M_0 = SWE \cdot D_s$	
		(in)			
Subwatershed @ Richmond	Jan	24.0	0.25	6.0	8 selected stations (including Farmville 2N & Williamsburg 2N) (Figure 2.2-2)
	Feb	20.0	0.30	6.0	
	Mar	19.0	0.35	6.7	
	Apr	12.0	0.30	3.6	
	Nov	14.0	0.25	3.5	
	Dec	14.0	0.25	3.5	
Subwatershed @ Dinwiddie	Jan	24.0	0.25	6.0	Farmville 2N & Williamsburg 2N
	Feb	20.0	0.30	6.0	
	Mar	14.0	0.35	4.9	
	Nov	6.0	0.25	1.5	
	Dec	14.0	0.25	3.5	
Subwatershed @ SPS	Jan	24.0	0.25	6.0	
	Feb	20.0	0.30	6.0	
	Mar	14.0	0.35	4.9	
	Nov	6.0	0.25	1.5	
	Dec	14.0	0.25	3.5	

Table 2.2-4: Controlling PMP – Seasonal vs. All-Season

Basin Name	Month	72-hour Seasonal PMP (in)	Extreme Snowpack Melt (in)	Combined Seasonal PMP (in)	All-Season PMP (in)	Controlling PMP
Subwatershed @ Richmond	Jan	8.0	6.0	14.0	15.5	All-Season
	Feb	8.0	6.0	14.0		
	Mar	8.3	6.7	15.0		
	Apr	9.0	3.6	12.6		
	Nov	11.0	3.5	14.5		
	Dec	8.7	3.5	12.2		
Subwatershed @ Dinwiddie	Jan	8.2	6.0	14.2	15.9	All-Season
	Feb	8.2	6.0	14.2		
	Mar	8.5	4.9	13.4		
	Nov	11.2	1.5	12.7		
	Dec	8.9	3.5	12.4		
Subwatershed @ SPS	Jan	6.5	6.0	12.5	12.6	All-Season
	Feb	6.5	6.0	12.5		
	Mar	6.8	4.9	11.7		
	Nov	8.9	1.5	10.4		
	Dec	7.0	3.5	10.5		

Table 2.2-5: Calibration/Verification Summary for Appomattox River Subwatershed

Date	Observed Peak Flow (cfs)	Modeled Peak Flow (cfs)	Volume Residual (in)	Standard Lag (hr)	Peaking Coefficient Cp	Constant Loss Rate (in/hr)	Initial Loss Rate (in)
Calibration Floods							
September 1996	12,300	13,633	0.94	22	0.40	0.320	0.00
September 2003	19,300	20,283	-0.71	40	0.40	0.055	1.80
August 2004	21,800	21,901	-0.84	6	0.40	0.420	0.00
Verification Floods							
September 1999	13,100	15,638	-0.31	20	0.40	0.170	0.00
May 2003	25,900	37,850	-0.91	20	0.40	0.170	2.00
October 2006	12,900	10,323	-1.21	20	0.40	0.170	0.00

Table 2.2-6: Calibration/Verification Summary for Upper James River Subwatershed

Date	Observed Peak Flow (cfs)	Modeled Peak Flow (cfs)	Volume Residual (in)	Standard Lag (hr)	Peaking Coefficient, Cp	Constant Loss Rate (in/hr)	Initial Loss Rate (in)
Calibration Floods							
June 1995	80,000	80,078	0.39	85	0.73	0.095	0.00
September 1996	152,000	154,357	0.88	58	0.60	0.140	2.00
September 1999	73,200	73,097	0.55	49	0.73	0.045	0.00
Verification Floods							
September 2003	90,900	93,372	0.32	60	0.73	0.082	1.50
June 2006	75,500	76,361	0.32	60	0.73	0.082	1.80
October 2006	71,100	71,713	0.19	60	0.73	0.082	1.20

Table 2.2-7: Summary of Probable Maximum Flood HEC-HMS Input Parameters

Subwatershed	Standard Lag (hr)	Peaking Coefficient, Cp	Constant Loss Rate (in/hr)	Initial Loss Rate (in)
Appomattox	20	0.40	0.170	0.00
Upper James	60	0.73	0.082	0.00
Ungaged	32	0.73	0.063	0.00

Table 2.2-8: Summary of HEC-HMS Probable Maximum Flood Results

	Subwatershed	Peak Flow (cfs)
Without Nonlinearity Adjustment	Appomattox (Inflow into Brasfield Dam)	124,018
	Appomattox (Outflow from Brasfield Dam)	112,946
	Upper James	551,153
	Ungaged	160,185
	Combined	700,486
With Nonlinearity Adjustment	Appomattox (Inflow into Brasfield Dam)	142,890
	Appomattox (Outflow from Brasfield Dam)	124,436
	Upper James	645,662
	Ungaged	179,163
	Combined	867,254

Figure 2.2-1: Watershed Delineation Map

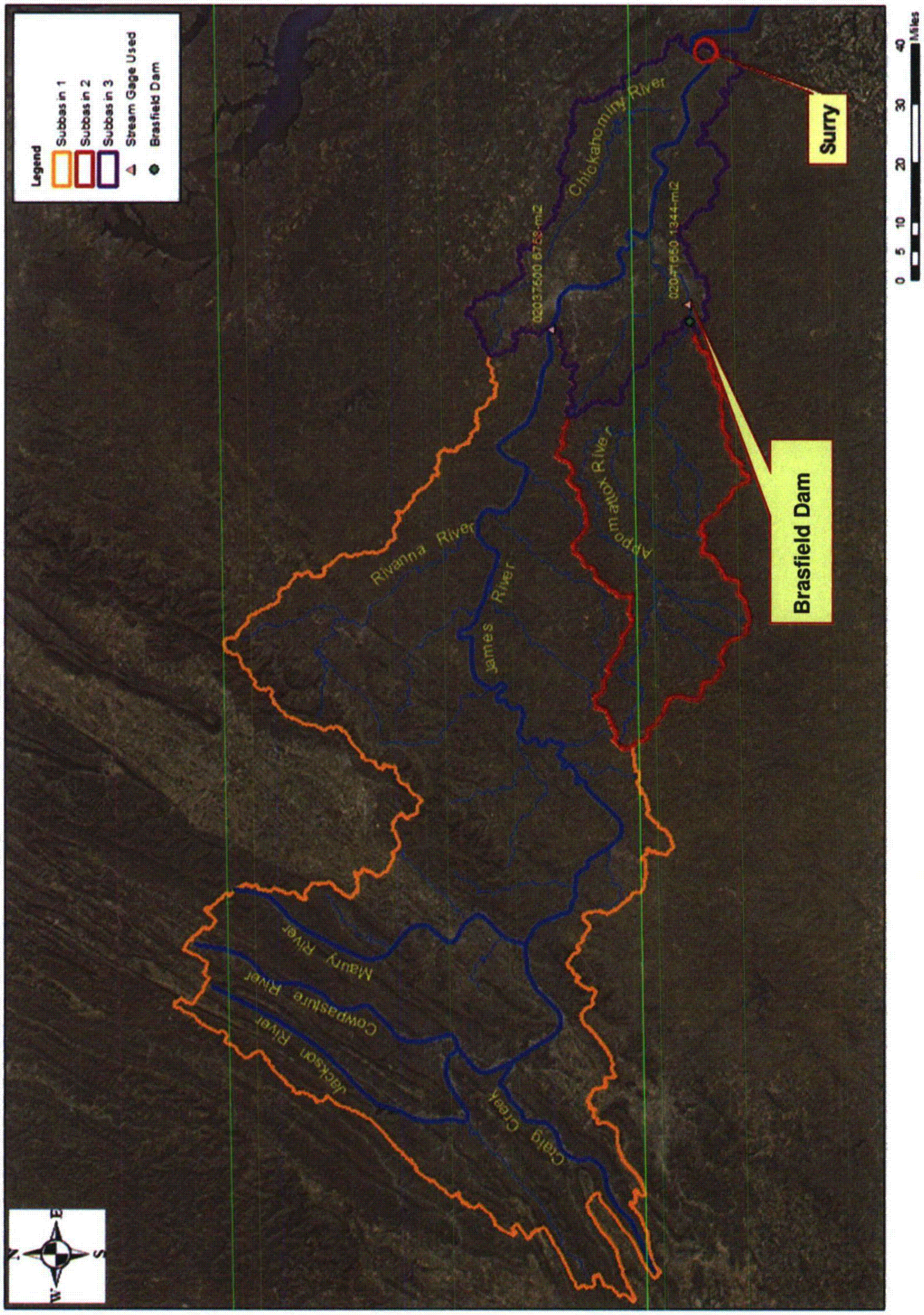


Figure 2.2-2: Precipitation Gages with Snow Climatology Data & Global Hourly Data

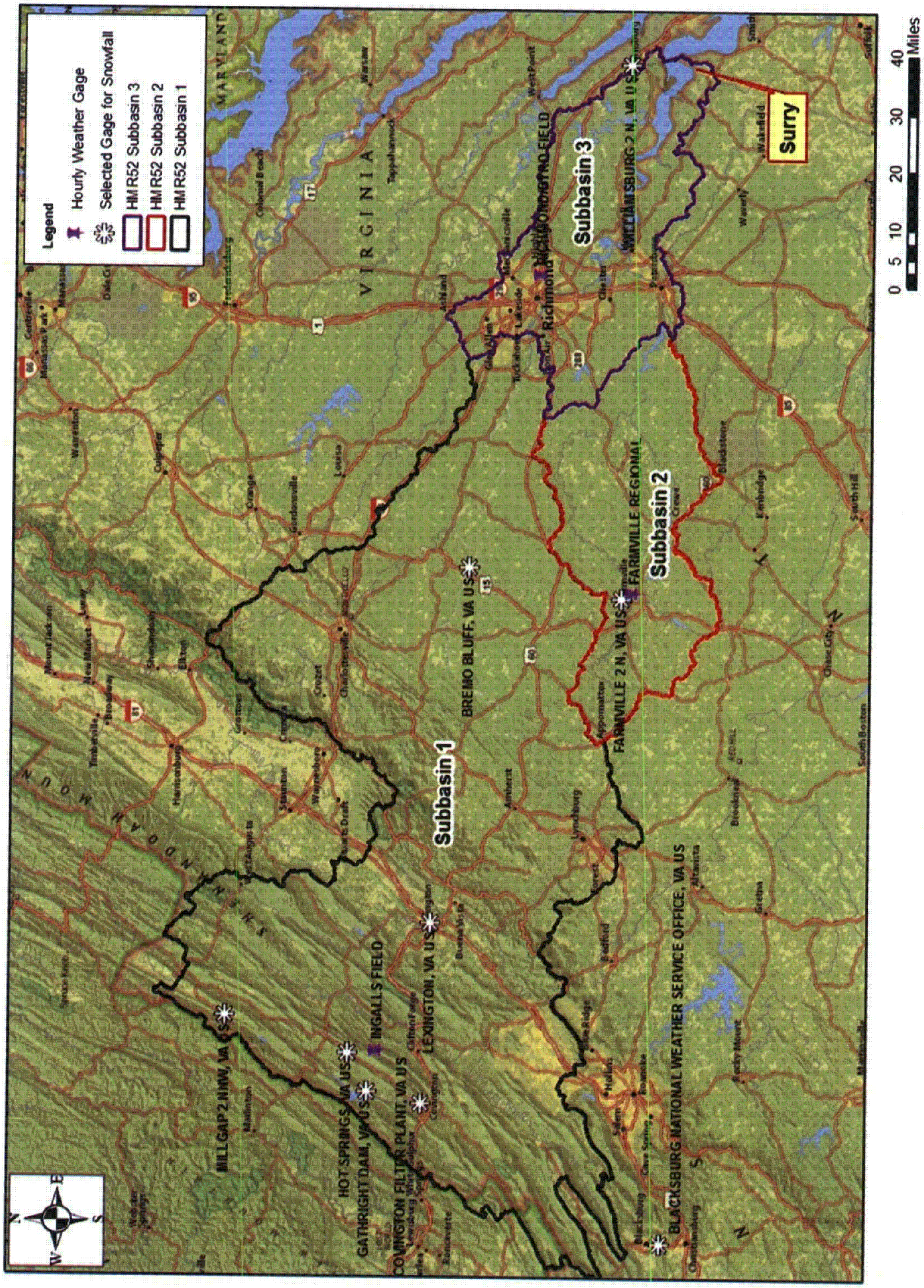


Figure 2.2-3: Input Hyetograph for 72-hour PMP

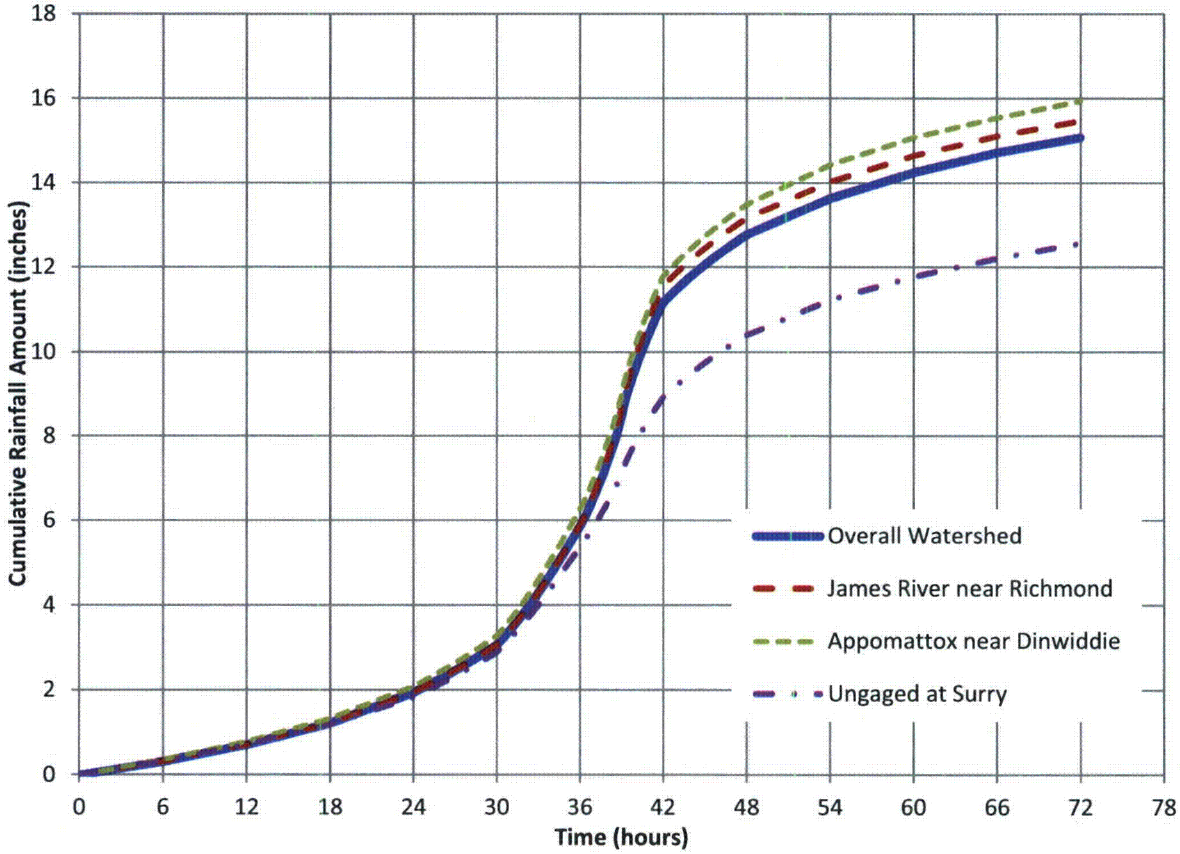


Figure 2.2-4: Unit hydrographs with and without Nonlinear Adjustment

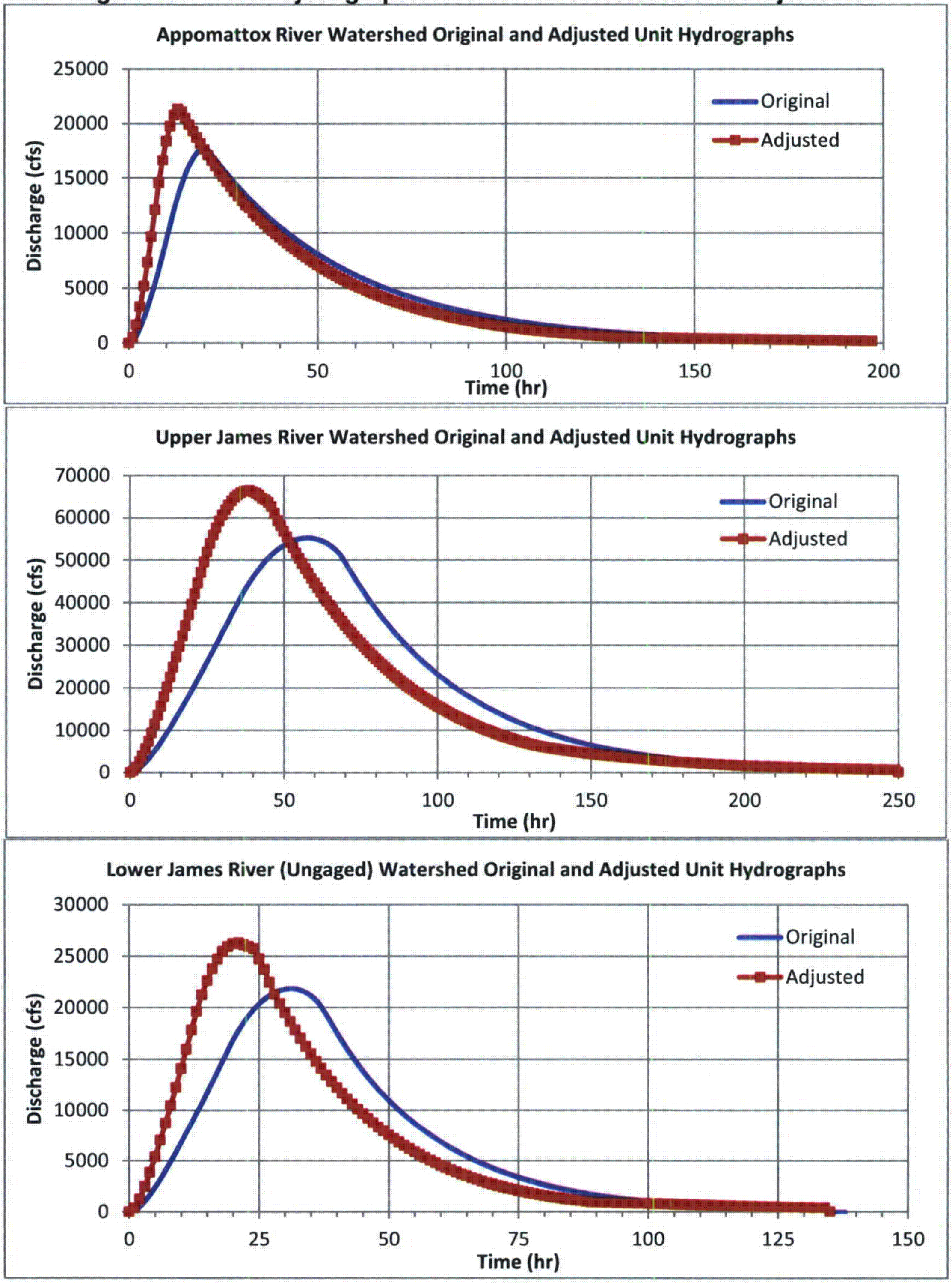


Figure 2.2-5: Results of 72 hr PMP Without Nonlinearity Adjustment

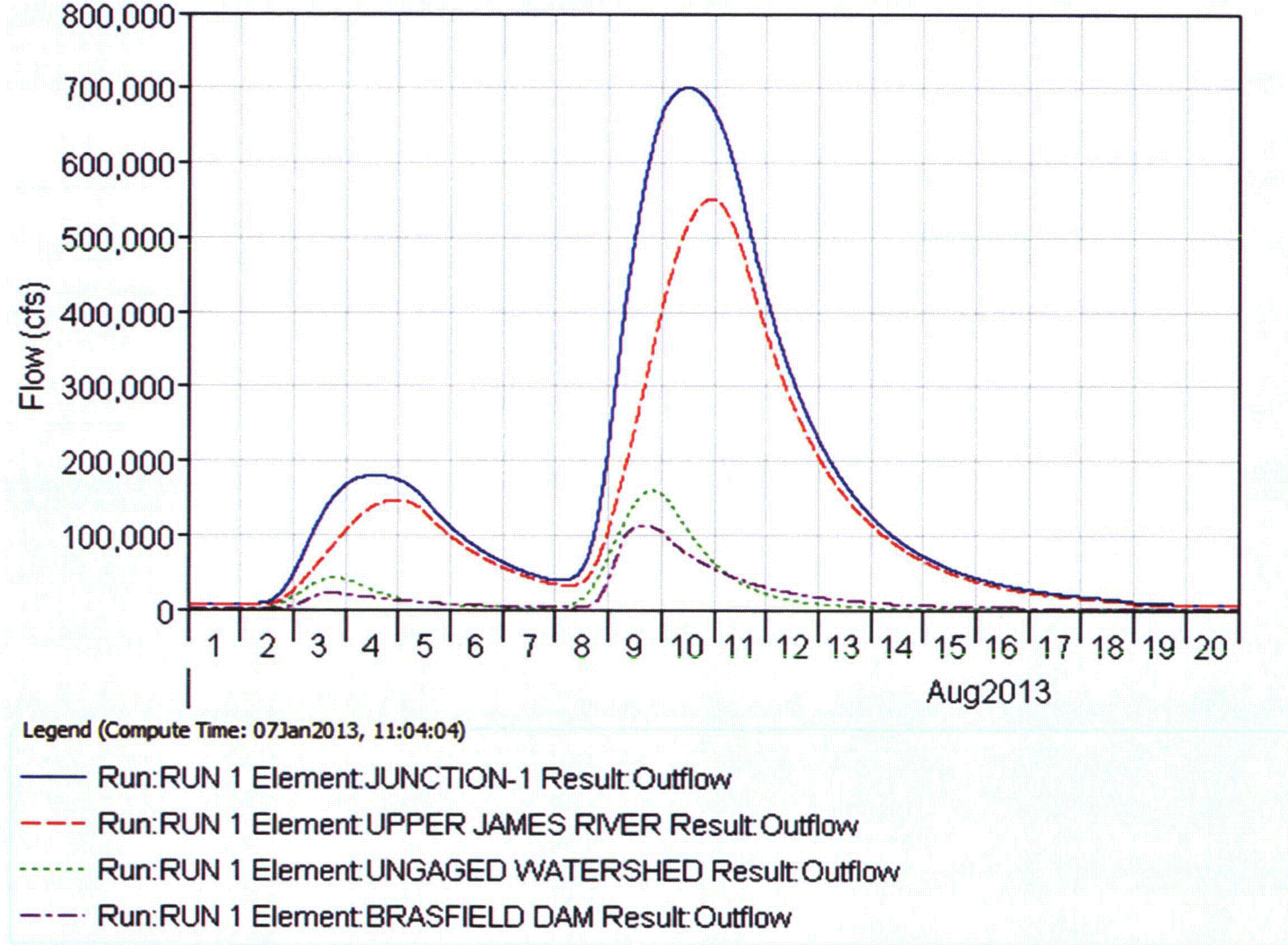


Figure 2.2-6: Results of 72 hr PMP With Nonlinearity Adjustment

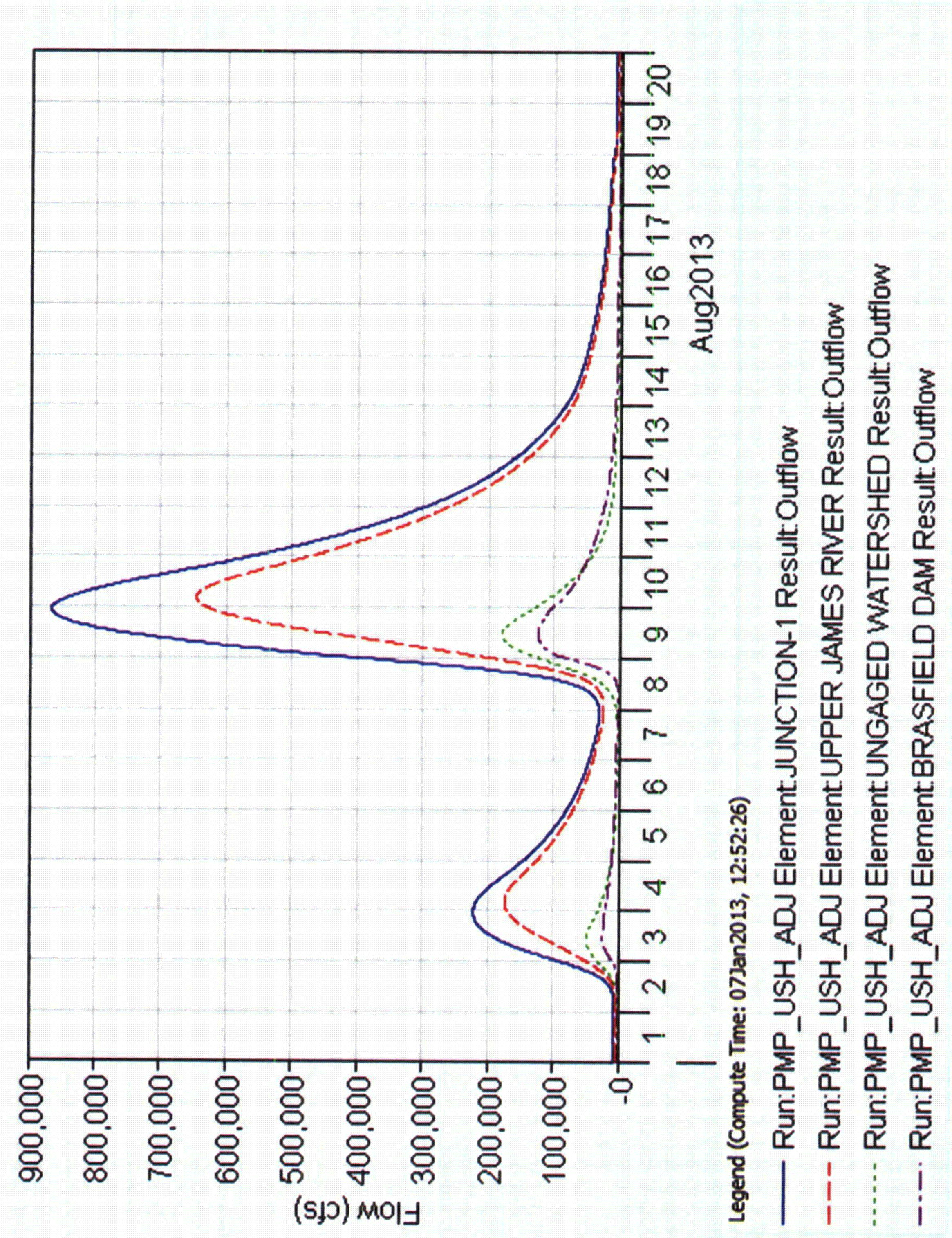
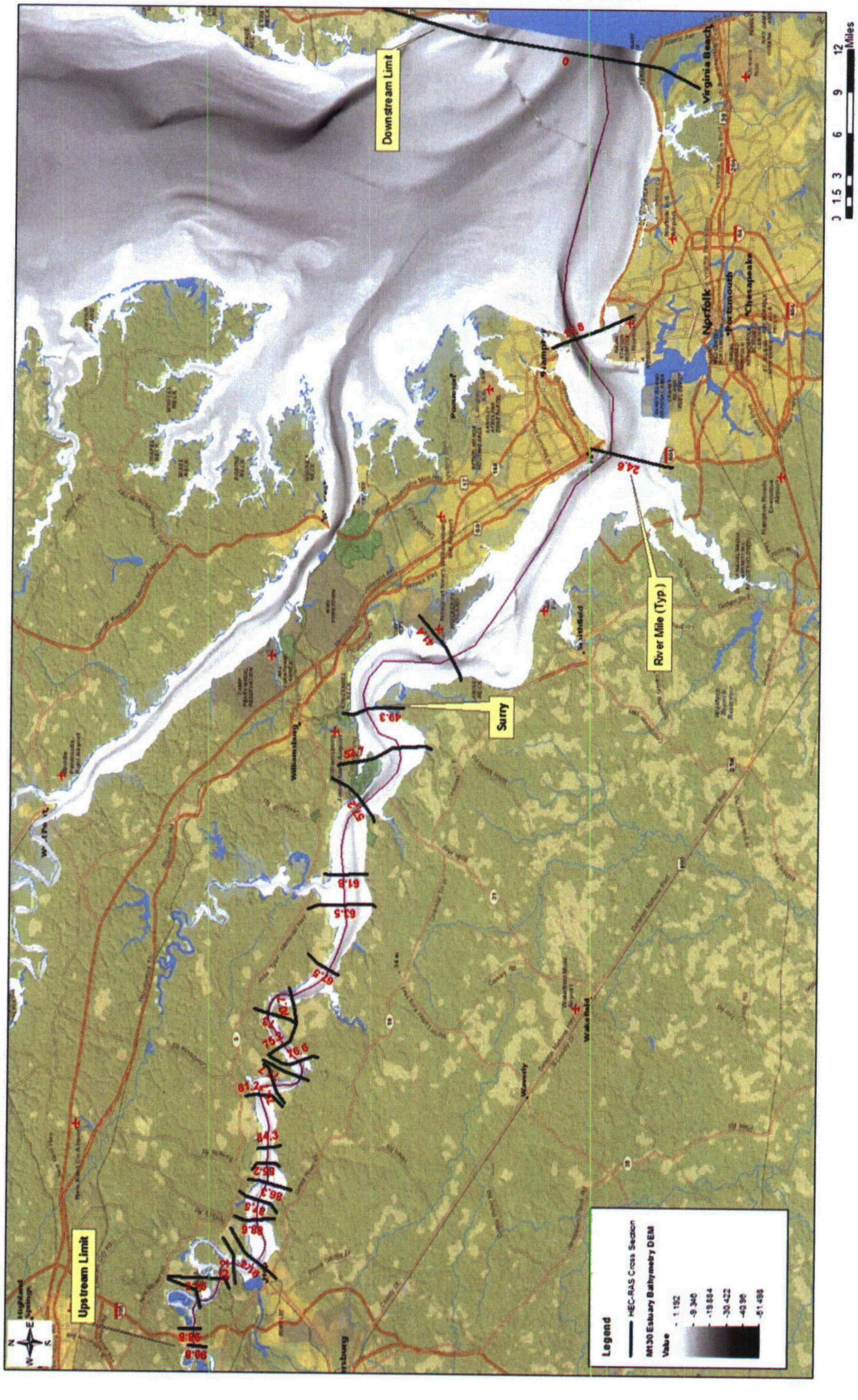


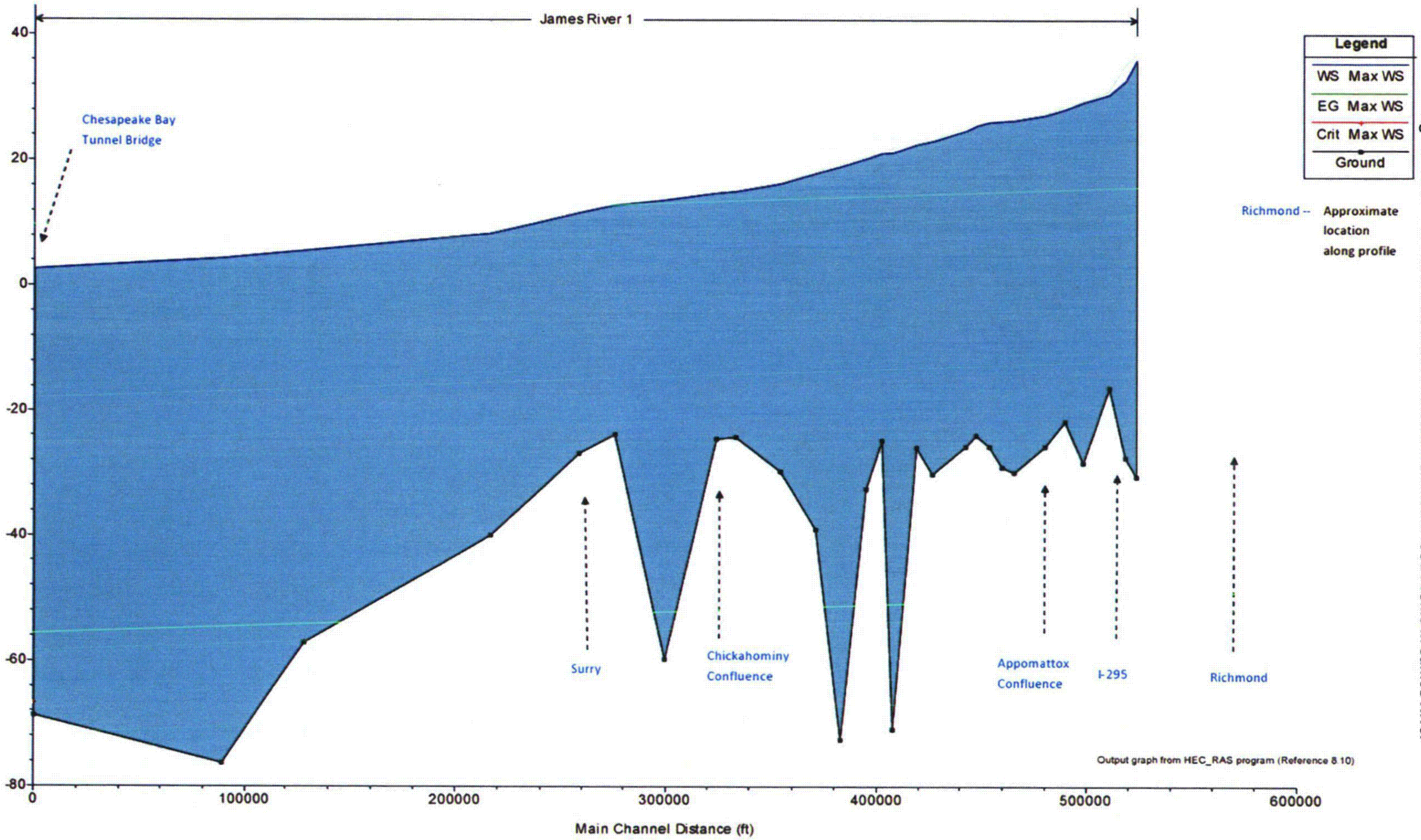
Figure 2.2-7: HEC-RAS Boundary Limits and Cross Sections



ZACHRY

Zachry Nuclear Engineering, Inc.

Figure 2.2-8: HEC-RAS Calculated Probable Maximum Flood Profile in James River



2.3. Dam Failures

The purpose of this calculation is to assess the effect of upstream dam failures on the water surface elevation of the James River near SPS and the potential for failure of the on-site Intake Canal and Settling Pond earthen embankments, which are partially located above the average SPS site grade elevation.

2.3.1. Methodology

2.3.1.1. Methodology for James River Watershed Upstream Dams

Upstream dam failures may result from a hydrologic event (i.e. PMF), a seismic event or embankment failure due to piping through the embankment (sunny-day failure). The seismic and sunny-day dam failure modes are usually characterized by the absence of a concurrent extreme flood. As per Appendix D of NUREG/CR-7046 (USNRC, 2011), the PMF scenario, discussed below, bounds Sunny Day and Seismic failure modes, because upstream reservoir levels used in the hydrologic event calculations are higher (i.e., coincident with top of dam) and coincident river flows are also larger (equal to the PMF).

The criteria for flooding from dam breaches and failures evaluation is provided in NUREG/CR-7046, Appendix D (USNRC, 2011). Two scenarios of dam failures are recommended and discussed in NUREG/CR-7046, Appendix D including:

1. Failure of individual dams (i.e., group of dams not domino-like failures) upstream of the site; and
2. Cascading or domino-like failures of dams upstream of the site.

The methodology used to calculate the water surface elevation at SPS resulting from upstream dam failures is consistent with the HHA approach described in Section 3.2 of the NRC's Interim Staff Guidance (ISG) for Assessment of Flooding Hazards Due to Dam Failure (USNRC, 2013) and involves the use of representative hypothetical dams for each subwatershed having the total storage volume of all individual dams represented by the hypothetical dam. The height of the hypothetical dam was conservatively assumed to be the height of the tallest represented dam, based on guidance contained in the dam failure ISG (USNRC, 2013). The hypothetical dams were located at the outlets of their respective subwatersheds.

The NRC's ISG lists four screening / simplified methods for calculating peak breach flow for watersheds with many dams. The methods are presented sequentially (1, 2, 3, and 4, with 1 being the most conservative and 4 being the most detailed). The third method (Peak Flow with Attenuation) was used in this calculation. This method is based on summing estimated discharges from upstream dam failures that simultaneously arrive at the site and have attenuated. The fourth method (Hydrologic Model Method) was also evaluated as a sensitivity check and to support the results of the Peak Flow with Attenuation Method. Potentially critical dams (as defined by the ISG and discussed below) were evaluated using more detailed Hydrologic Model Method.

The methods of analyses are summarized below:

1. Identify dams upstream of SPS within the James River contributory watershed.

2. Develop a conservative dam failure scenario, in which the multiple upstream dams within each subwatershed were grouped and modeled as one hypothetical dam per subwatershed representing the combined storage of the dams.
3. Calculate the peak breach flow at SPS from each hypothetical dam using regression equations (breach geometry and time to fail are not required input parameters), and attenuate the peak flows based on the U.S. Bureau of Reclamation (USBR) empirical attenuation regression equation (Colorado DWR, 2010). This methodology is consistent with the first and second steps of the Peak Outflow with Attenuation Method in Section 3.2 of the NRC's ISG (USNRC, 2013). The ISG states the USBR regression equation may be used but should be tested against available models and/or studies to justify the applicability to the river system to SPS. A literature review of historical examples of dam breach attenuation was performed in addition to the use of an alternative methodology following the Hydrologic Model Method of Section 3.2 of the NRC's ISG (USNRC, 2013) was used to assess the conservatism of the application of the USBR regression equation. The Hydrologic Model Method uses the Muskingum method for routing of dam breach hydrographs. Section 9.0 of the ISG lists the Muskingum method as one option for use in hydrologic modeling of dam breach flows.
4. Identify potentially critical dams and model them individually as reservoir elements in HEC-HMS.
5. Perform hydraulic simulations to calculate the peak water surface elevation resulting from the combined dam breach and PMF using the HEC-RAS model described in Section 2.2.

2.3.1.2 Methodology for On-site Intake Canal and Settling Pond

Failure of onsite water-storage (e.g. the Intake Canal and Settling Pond) or water-control structures that are located at or above the grade of structures, systems and components (SSCs) important to safety at the SPS site were also considered. The methodology included:

1. Assess Potential Failure Modes.
2. Develop FLO-2D model with site features including the Intake Canal and the Settling Pond.
3. Identify Potential Failure Locations.
4. Determine Breach Parameters including Geometry and Time to Failure.
5. Perform breach simulations of the Intake Canal and the Settling Pond to calculate the resulting maximum water surface elevations at the site.

2.3.2. Dam Breaches and Failures Results

2.3.2.1. Results for James River Watershed Upstream Dams

2.3.2.1.1 Identification of Upstream Dams

The National Inventory of Dams (NID) as maintained by the U.S. Army Corps of Engineers (USACE) was used to identify upstream dams (USACE, 2013). The dams within the James River Watershed at SPS are located in Virginia. The list of dams in the State of Virginia was extracted from the NID database with coordinates and basic dam characteristics (i.e., height, maximum pool volume and dam type). The three subwatersheds delineated as part of the PMF analysis (see Section 2.2) were used to obtain three corresponding groupings of dams. The Upper James River subwatershed was further divided into three smaller subwatersheds due to its large size (6,753 sq. mi.) and to better account for the attenuation of breach outflows from dams within the subwatershed.

The numbers of dams in each subwatershed (Figure 2.3-1) are summarized below:

Subwatershed	Total Number of Dams
Upper James	501
North Upper James	88
Northwest Upper James	8
Central Upper James	405
Appomattox	110
Lower James	132

Overall, 86 percent of the dams within the James River watershed at SPS are earthen dams. In the Lower James Subwatershed, 76 percent of the dams are earthen dams. Therefore, dam breach parameters for earthen dams were selected and used for modelling the Lower James hypothetical dam's breach in HEC-HMS. Locations of the hypothetical dams within the five subwatershed areas are shown in Figure 2.3-3.

2.3.2.1.2 Hypothetical Dams and Dam Breach Outflows

The dams within each subwatershed identified above were "combined" and modeled as one hypothetical dam located at the downstream outlet for each respective subwatershed. The hypothetical dams were modeled in parallel. The geometry and characteristics of the five hypothetical dams were calculated as follows (USNRC, 2013):

- Storage was calculated using the total of the NID storage (i.e., The NID storage represents each reservoir's maximum pool volume or, in the absence of maximum pool information, normal pool volume) of the dams in each subwatershed:
- Dam Height was calculated as the height of the tallest actual individual dam in each respective subwatershed.

Peak breach flow from each hypothetical dam was calculated using regression equations. The eight different regression equations referenced in the ISG (USNRC, 2013) and listed below were initially considered in calculating the peak dam breach outflow for the hypothetical dams.

- Kirkpatrick, 1977;
- Soil Conservation Service (SCS), 1981;
- United Stated Bureau of Reclamation (USBR), 1982;
- Singh and Snorranson, 1984a;
- Singh and Snorranson, 1984b;
- Froehlich, 1995;
- MacDonald and Langridge-Monopolis, 1984; and
- Costa, 1985.

The regression equations use only reservoir storage and/or dam height as input parameters. This makes the use of these equations ideal for modeling the hypothetical dam failure because hypothetical dams do not have physical characteristics to use in selection of breach parameters such as development time. The Singh and Snorranson regression equations were not used because they were derived from a small sample set (eight dams) all located within Illinois well beyond the limits of the Surry watershed. Additionally, the regression equations were derived using estimates for the peak breach outflow calculated using the National Weather Service's peak breach flow equations and not observed data. The Kirkpatrick, SCS and USBR dam breach peak flow regression equations were not used because they only use a single variable, dam height (Wahl, 2004). Since the hypothetical dams only use a representative dam height equal to the maximum dam height, varying the number of dams included in the hypothetical dam often results in no change in peak breach flow with these equations, and thus, the equations are not representative for this analysis. Therefore, peak breach flow from each hypothetical dam was calculated using three regression equations: Froelich, MacDonald and Langridge-Monopolis, and Costa. The largest calculated peak between the three methods was selected (Table 2.3-1).

Note that this approach is conservative because it includes the failure of all the NID dams within each subwatershed in a single flood. Realistically, it is highly improbable that over a large watershed of 9,500 square miles, that over 700 dams would experience failure during a single flood.

2.3.2.1.3 Attenuation of Hypothetical Dam Breach Outflows

The ISG (USNRC, 2013) discusses the use of conservative regression relations such as the USBR empirical attenuation regression equation (DWR, 2010) for analysis of a large number of dams. The USBR attenuation method used in this calculation is appropriate / conservative for the SPS hydrological setting for the following reasons:

1. Significant floodplain attenuation for large portions of the James River from the low lying areas with little topographic relief is expected (Figure 2.3-2).
2. There are 745 dams in the James River watershed as listed in the NID database (USACE, 2013) and accounting for attenuation from individual dams is not practical (USNRC, 2013). 68 of these dams are flood control dams.
3. The contributory watershed at SPS includes over three hundred and twenty miles of the James River, which provides a large distance to attenuate peak dam breach outflows.
4. There is significant flow attenuation on the lower James River reach based on the results of the PMF hydraulic analysis described in Section 2.2. Approximately 50 miles of the James River upstream of SPS was modelled as part of the PMF analysis. Within this reach of the river there was approximately a 15-percent reduction in peak flow from the upstream boundary to SPS. This demonstrates the potential of the James River floodplain to provide flow attenuation.

Peak breach outflow for each hypothetical dam (with the exception of the peak breach outflow from the hypothetical dam for the lower James watershed because the lower James hypothetical dam was located at the site) was attenuated based on the peak breach discharge at the dam and stream distance from the hypothetical dam to the site using the USBR regression equation (Colorado DWR, 2010). Figure 2.3-3 illustrates the stream distances used in the calculation. Results of the analysis are provided in Table 2.3-1.

The sum of the attenuated peak discharges for all of the hypothetical dams, with the exception of the lower James hypothetical dam, was input into HEC-RAS as a constant lateral inflow at a cross-section directly upstream of the site. This methodology is consistent with the fourth step of the Peak Outflow with Attenuation Method in Section 3.2 of the NRC's ISG (USNRC, 2013). Attenuation was not considered for the lower James hypothetical dam, since the lower James subwatershed discharges at SPS (i.e., stream distance upstream of SPS is zero).

A literature review of historical examples of dam breach attenuation (Costa, 1985, Fread, 1996, and Reed, 2011) was performed. The historical examples were compared to the attenuation observed with the USBR regression equation. The majority of the historical dam breaches presented in these studies demonstrate greater attenuation than the USBR regression equation. Note that the relationships for attenuation versus downstream distance presented in these studies do not exceed 190 miles. The tallest dams in the James River watershed are over 200 miles from SPS. Based on the studies, the use of the USBR regression equation for this calculation is appropriate.

In addition to the literature review of historical examples of dam breach attenuation, an alternative methodology involving routing of breach hydrographs using the Muskingum routing method to check the conservatism of the use of the USBR regression equation. The alternative methodology produces similar results to those presented in Table 2.3-1 and reinforces the appropriateness of the use of the USBR regression equation for this analysis.

2.3.2.1.4 Potentially Critical Dams

The ISG identifies potentially critical dams as dams whose failure, either alone or part of a multiple dam failure scenario, might cause inundation of the site. The ISG recommends that potentially critical dams be evaluated separately, using refined methods (USNRC, 2013). Little Creek Dam (NID No. VA09506) and Diascund Creek Dam (NID No. VA12703) were considered potentially critical based on their size and proximity to SPS. Failure of the Little Creek Dam and Diascund Creek Dam were modeled individually (i.e., not included in the combined hypothetical dam) using the refined Hydrologic Model Method in Section 3.2 of the NRC's ISG (USNRC, 2013).

Each potentially critical dam was modeled in HEC-HMS using a reservoir element parallel to each basin element. Prior to the dam breach, the reservoirs were assumed to have completely filled so that the pool elevation for each potentially critical dam is equivalent to the top of dam elevation. Storage attenuation of inflow to the reservoirs was not considered. Dam breach parameters were selected based on published guidance (such as Colorado DWR, 2010; FERC, 1993; Wahl, 2004). See Table 2.3-2 for the selected dam breach parameters.

2.3.2.1.5 Hydrologic and Hydraulic Simulations

The HEC-HMS model discussed in Section 2.2 was used. The hypothetical dams and the potentially critical dams developed were modeled as reservoir elements that were breached coincident with the watershed PMP in the HEC-HMS simulation. The HEC-HMS basin model flow diagram is shown in Figure 2.3-4. The results of the dam breach simulations are summarized in Table 2.3-3. The calculated total dam breach outflow during the PMF from the Upper James River watershed (including the attenuated steady-state flows from hypothetical dams) is 849,000 cfs, about 31-percent higher than the PMF without dam breach. The calculated total outflow from the Appomattox River watershed is 258,000 cfs, about 80-percent higher than the PMF without dam breach. The outflow from the ungaged lower James River watershed at SPS is 823,000 cfs, about 360-percent higher than the PMF without dam breach. The contribution to this flow from Little Creek Dam's breach and Diascund Creek Dam's breach are 265,000 cfs and 62,000 cfs, respectively.

The resulting dam breach flows were input into the HEC-RAS model developed as discussed in Section 2.2. The key inputs to HEC-RAS for dam breach during the PMF are:

a. Upstream boundary condition – HEC-RAS Station 99.8: PMF hydrograph from the Upper James River watershed;
b. Lateral Inflow 1 - Station 91.4: PMF hydrograph from the Appomattox River watershed;
c. Lateral Inflow 2 – Station 52.7 (approximately 3 miles upstream from SPS location):
   - combined PMF and dam breach hydrograph from the ungaged lower James River watershed;
   - constant flow equal to the sum of the attenuated breach outflows from all the hypothetical dams with the exception of the lower James watershed hypothetical dam;
d. Downstream boundary condition: The downstream boundary condition was assumed to be a constant stage equal to the Mean Higher High Water (MHHW) elevation of 3.1 feet at the river mouth of the James River (near the Chesapeake Bay Tunnel Bridge), based on National Oceanic and Atmospheric Administration (NOAA) datum analysis results at the tide gages near Virginia Beach, VA (NOAA, 2013).

The combined peak flow rate (dam breach flow rate and the full PMF flow in James River at SPS) is approximately 1,056,000 cfs after hydraulic routing in HEC-RAS. The peak stage resulting from the failure of the five combined hypothetical dams and two potentially critical dams under the PMF condition was calculated to be elevation 15.7 feet at SPS (HEC-RAS Station 49.3). The HEC-RAS calculated water surface elevation profile is shown in Figure 2.3-5. The calculated stage and flow hydrographs at SPS (HEC-RAS Station 49.3) are shown in Figure 2.3-6.

2.3.2.1.6 Discussion of Conservatism

A more detailed approach to calculating the water surface elevation at SPS from upstream dam failures would involve modeling the breach and routing processes in the same detailed hydraulic model. However, this approach is unnecessary because it is bounded by the methodology described in Section 3.1.1 through 3.1.5 for the following reasons:

i. All dams in the watershed are assumed to fail during a single event in this calculation. A hydraulic modeling approach would use the result of detailed PMF modeling to establish actual water surface elevations in dam impoundments. This added detail would reduce the number of dams that would fill to the maximum limit of their respective impoundment (reducing overall storage available for dam breach outflow), as well as reduce the number of dams that overtop and fail because the PMP that results in the PMF is specifically centered and oriented to produce the maximum flow at SPS at the watershed outlet. Dams along the fringes of the PMP and the overall James River watershed at SPS would likely not be experiencing a PMF-level flood since their individual watersheds are much smaller and the PMP is not centered over them;

ii. Peak breach flows from hypothetical dams are assumed to reach the site simultaneously in this calculation;

iii. Dams are clustered/combined into hypothetical dams in this calculation. A hydraulic modeling approach could model more dams as potentially critical which would reduce the peak flows from hypothetical dams due to reduced storage volumes and, in general, dam height (since the highest dam is used for the representative hypothetical dam).

Additionally, the results of the analysis performed using the HHA approach indicated that considerable vertical margin is available at SPS. The screening level approach, combined with hydraulic routing calculations for potentially critical dams, is more conservative than a detailed modeling approach for the reasons discussed above. Therefore, additional detailed dam failure modeling is not warranted.

2.3.2.2. <u>Results for On-site Intake Canal and Settling Pond</u>

2.3.2.2.1 Failure Modes for the On-site Intake Canal and Settling Pond

The Intake Canal at SPS is an earthen embankment about 1.7 miles long, trapezoidal in section, and with top and bottom widths of 125 feet and 32 feet, respectively (SPS, 1986). The Intake Canal runs nearly west to east from the plant to the low level intake near the James River. The invert elevation of the Intake Canal at the plant side is 5 feet, MSL and the top of embankment elevation is 36 feet, MSL. The Intake Canal has side slopes of 1.5 feet horizontal to 1 foot vertical. Two nearly identical High Level Intake Structures (one for each unit) are located near the plant side of the Intake Canal embankments. Each High Level Intake Structure is a one story reinforced concrete structure approximately 40 feet wide by 78 feet long by 36 feet high and supported on a 3 feet thick mat (SPS, 2014b).

The Settling Pond is an earthen embankment located on the western end of the pond just upstream of the Discharge Canal. The embankment slopes steeply towards the Discharge Canal. The main power block is located south of the embankment. The Settling Pond embankment is trapezoidal in shape with a side slope of 2 horizontal to 1 vertical. The bottom of the pond is at elevation 21 feet, MSL and the minimum top elevation is 35 feet, MSL. The Settling Pond is 285 feet long and 226 feet wide, with a 15 feet wide (top width) dike (New Dike) protruding into the pond from the western end of the Settling Pond embankment (SPS, 2010). With the exception of the western end of the Settling Pond, all the other sides of the pond are at or below the surrounding grade elevation (i.e. cut section).

An assessment of the potential failure modes under the three failure types is presented below:

- Hydrologic Failure

Hydrologic dam failure is the failure of a dam during its PMF/LIP. An analysis of the Intake Canal and Settling Pond was performed to evaluate the potential for these structures to fail during their respective PMF/LIP.

  - Intake Canal

Hydrologic failure of the Intake Canal embankment and the High Level Intake Structures has been eliminated from further evaluation (SPS, 1994, SPS, 2014a, and SPS, 2014b) for the following reasons:

1. SPS Calculation CE-2000 (SPS, 2014b) indicates that both the Unit 1 and Unit 2 High Level Intake Structures can withstand a maximum Intake Canal water elevation of 36 feet, MSL (top of embankment elevation) due to the PMP.
2. SPS Calculation CE-1128 (SPS, 1994) indicates that elevating the water level in the Intake Canal to Elevation 32 feet will not significantly increase the phreatic surface in the embankment, and this will not affect the stability of the canal dike (embankment).
3. As per Section 2.1, SPS LIP results indicates that the maximum water surface elevation in the Intake Canal during extreme floods (e.g. LIP) are at or below 32 feet, MSL with flooding durations on the order of hours.
4. SPS Calculation CE-1997 (SPS, 2014a) indicates that an acceptable factor of safety (greater than 1.4) against the Intake Canal dike / embankment instability exists under maximum flooding conditions (water elevation in the Intake Canal at elevation 36 feet, MSL).

  - Settling Pond

Hydrologic failure of the Settling Pond embankment was not screened out. The SPS LIP calculation using site specific probable maximum precipitation indicates that the Settling Pond embankments are overtopped during the LIP, which could lead to failure of the embankment. Therefore, Hydrologic failure of the Settling Pond embankment was analyzed.

- Seismic Failure

  - Intake Canal

A seismic screening analysis was performed for SPS in response to the NRC's 50.54 letter (USNRC, 2012). The purpose of the request was to gather information concerning, in part, seismic hazards at operating reactor sites (including SPS) and to enable the NRC staff to determine whether licenses should be modified, suspended, or revoked. A site is screened out of further evaluation when the Safe Shutdown Earthquake (SSE) bounds the Ground Motion Response Spectrum (GMRS). The NRC correspondence (USNRC, 2014) concludes that the SSE (Current Licensing Basis) bounds the GMRS and therefore SPS screens out of any further evaluation. In addition, SPS Calculations CE-1128 and CE-2000 (SPS, 1994; SPS, 2014b) indicates that the Intake Canal is capable of withstanding the SSE. Seismic failure of the Intake Canal was therefore not considered at SPS.

  - Settling Pond

Seismic dam failure is bounded by hydrologic failure since the starting water elevation in the Settling Pond would be less for the seismic failure scenario than during the LIP. Therefore, seismic failure of the Settling Pond was not analyzed in this calculation

- Sunny Day Failure

Sunny day failures cannot be screened out as per the ISG for Assessment of Flooding Hazards Due to Dam Failure and should be the default failure scenario if no other failure mechanisms exist (USNRC, 2013).

- Intake Canal

Sunny day failure of the Intake Canal is the default failure scenario because hydrologic and seismic failures of the Intake Canal have been screened-out (SPS, 1994, SPS, 2014a, and USNRC, 2014). In addition, failures of the High Level Intake Structures have also been screened out (SPS, 2014b). Therefore, the failure of the earthen embankment sections of the Intake Canal was analyzed in this calculation.

- Settling Pond

Sunny day failure of the Settling Pond is bounded by hydrologic failure since the starting water elevation in the Settling Pond would be less for the sunny day scenario than during the LIP.

2.3.2.2.2 Dam Failure Analyses for On-site Intake Canal and Settling Pond

Dam Breach Locations and Parameters

Failure of the Intake Canal impoundment structure and the Settling Pond embankment were simulated using the two-dimensional model, FLO-2D. A simple uniform rate of breach expansion failure mode ("Prescribed Failure" mode in FLO-2D) was used in the analysis. FLO-2D computes the discharge through the breach, the change in upstream storage, the tailwater and backwater effects (i.e. weir submergence), and the downstream flood routing (FLO-2D, 2014a). The FLO-2D model described in Section 2.1 was used for this analysis. The FLO-2D model used in the simulation of the Intake Canal breach and the Settling Pond Breach differ from the SPS LIP FLO-2D model as follows:

- The Intake Canal and Settling Pond were modeled as reservoirs. The bottom elevation of the reservoir was set to the site grade elevation to represent the portion of the respective impoundment available for dam failure outflow and the initial pool elevation was set to the maximum operating pool elevation (SPS, 2013);
- The breach location sections (Figure 2.3-7) were modeled as a levee. All other portions of the Intake Canal embankments were modeled as elevated grid cells with elevations equal to the top of embankment elevation.
- The south-eastern gate was conservatively modeled as blocked to keep the breach outflow from escaping from the PA.

The most conservative breach location (area of maximum dam height where breach will result in a flow direction nearest SPS) was established to be along the south western end of the Settling Pond embankment. The Settling Pond embankment failure section was based on visual inspection of site topography (SPS, 2012).

Three failure locations along the plant side (e.g., north side) of the Intake Canal embankment were selected based on the sections of the embankment within the PA where failure is possible. The failure location section also considered locations along the embankment where failure is likely to result in the most conservative flood depths at the site.

Breach parameters for the Intake Canal and Settling Pond were selected based on a review of published guidance for dam failure evaluation, including publications from the USBR, USACE, and FERC. As stated in the ISG on Dam Failure (USNRC, 2013), the state of practice in dam modeling shows a preference for regression-based approaches to predict the final parameters for breach opening (e.g. size, shape, and time to fully develop). Breach parameters for the sunny day failure of the Intake Canal are summarized in Table 2.3-4. Table 2.3-4 also shows a comparison of the selected Intake Canal breach parameters with the range of parameters from published guidance and from the regression equations (USNRC, 2013, FERC, 1993 and Froehlich, 2008). The key Intake Canal breach parameters are as follows:

a. Prescribed failure elevation = 30 feet, MSL;
b. Base elevation of breach = 26.5 feet, MSL;
c. Maximum Breach Width = 5 x height of embankment, or 48 feet;
d. Time to failure of 30 minutes (0.5 hours).

The selected key breach parameters for the Settling Pond were developed in a similar fashion, as listed below:

a. Water elevation at time of failure = 35 feet, MSL;
b. Base elevation of breach = 21 feet, MSL;
c. Maximum Breach Width = 5 x height of embankment, or 70 feet;
d. Time to failure of 30 minutes (0.5 hours).

<u>FLO-2D Results for the Intake Canal embankment breach</u>

The results of the simulation of the sunny day failure of the Intake Canal (for the three breach locations) are summarized in Table 2.3-6. Results include maximum water surface elevations, maximum flow depths, and maximum flow velocities for representative grid elements at "strategic doors" identified by SPS (see Figure 2.1-9). Maximum water surface elevations at the site range from 26.4 feet, MSL at "1 BS DR-74" (Door 43) to 28.1 feet, MSL at Doors 16 (at South East Corner of Unit 2) and Doors 17, 18, 19, and 20, into the Maintenance Offices. Maximum flow depths range from 0.3 feet to 4.2 feet (in localized low areas). The average flood depth at affected door locations at SPS is 1.3 feet. Maximum velocities at door locations are up to 4.4 feet per second. The resulting flood depths at Doors 8, 9,10,11, 26, 37, 43, 45, 46, 47, 48, 49, 50, 51, 53 and 54 are at or below the door threshold elevations and are unaffected by the sunny day breach of the Intake Canal. The maximum water surface elevations for the three breach locations are shown in Figures 2.3-9a, 9b, and 9c, respectively. Maximum flow depths for the three breach locations are shown in Figures 2.3-10a, 10b, and 10c, respectively.

The breach occurs at the specified time of 0.5 hours and Intake Canal water levels decrease sharply for the first 5 minutes and more gradually over the following 30 hours. The reduction of the rate in the rate of flow out of the Intake Canal after the first 5 minutes is a result of submergence of the breach due to flood depths on the north (plant) side of the Intake Canal, thereby reducing the available hydraulic head for driving flow out of the Intake Canal. Similarly, the long duration of the Intake Canal drainage is attributed to the relatively small breach section (48 feet wide) and the small hydraulic head (less than 3.5 feet at its maximum) which limit outflow.

<u>FLO-2D Results for the Settling Pond embankment breach</u>

The hydrologic failure of the Settling Pond during the LIP does not increase LIP flood depths, as shown in Table 2.3-5. LIP flood depths at SPS, with and without breach of the Settling Pond, are identical. The Settling Pond has a limited storage capacity (about 3 acre-feet under normal pool conditions) and the site topography directs breach outflow into the Discharge Canal. The breach outflow from the Settling Pond results in a minor increase (less than 0.2 feet) in the water levels in the Discharge Canal.

The FLO-2D reference manual (FLO-2D, 2014a) provide three keys to a successful project application. These include volume conservation, area of inundation and maximum velocities, and numerical surging.

- Volume Conservation: Reviews of the FLO-2D "SUMMARY.OUT" files indicate volume conservation errors less than 0.0005 percent for the FLO-2D runs (both the Intake Canal and Settling Pond). This value is well below the threshold of 0.001 percent specified in the FLO-2D Data Input manual (FLO-2D, 2014b) for a successful project application.
- Area of Inundation: Reviews of the FLO-2D "SUMMARY.OUT" files indicate a maximum inundated area of 427 acres for the simulation of the hydrologic failure of the Settling Pond. The FLO-2D model is made up of 82,435 grid elements, each 15 feet by 15 feet in dimension. The LIP was simulated within the entire computational domain of the model. The maximum inundation area should therefore be equal to the area of the computational domain of 426 acres (15 x 15 x 82,435) x (1 acre / 43,560 feet). The FLO-2D calculated maximum inundation area of 427 acres for the simulation of the hydrologic failure of the Settling Pond is reasonable.

  The "SUMMARY.OUT" output for the simulation of the sunny day failure of the Intake Canal indicates a maximum inundated area of 40 acres and is reasonable based on the areas inundated (Figures 2.3-9a, 9b, and 9c). These results indicate a successful project application.
- Maximum Velocities and Numerical Surging: Numerical surging, if it exists, would be evident at unreasonably high velocities in the FLO-2D output files. A review of the FLO-2D "VELTIMEC.OUT" and "VELTIMEFP.OUT" files do not indicate unreasonably high velocities in the model runs and indicates a successful project application. The high velocities reported in the "VELTIMEFP.OUT" file occur at locations where the embankments were failed and are reasonable.

2.3.3. Conclusions

The following summarizes the results and conclusions:

1. The PMF peak flow in the James River at SPS with upstream dam failures after hydraulic routing is 1,056,000 cubic feet per second (cfs), and the resultant PMF peak water surface elevation at SPS with upstream dam failures is 15.7 feet MSL. This corresponds to a 3.6 foot increase in depth above the PMF elevation without dam failures of 12.1 feet MSL. The flood elevation resulting from upstream dam failures is well below the existing general site grade of 26.5 feet MSL (Dominion, 2014).

2. Sunny Day Failure of the Intake Canal results in maximum water surface elevations at the site ranging from 26.4 feet, MSL at "1 BS DR-74" (Door 43) to 28.1 feet, MSL at Doors 16 (at South East Corner of Unit 2) and Doors 17, 18, 19, and 20, into the Maintenance Offices. Maximum flow depths range from 0.3 feet at "1-BS-DR-DVP27-1" (Door 28) to 4.2 feet at "1 BS-DR-D23 -1 and -2" (Doors 39 and 40). Maximum velocities at door locations are up to 4.4 feet per second at "1-BS-DR-67" (Door 35). The resulting flood

depths at Doors 8, 9,10,11, 26, 37, 43, 45, 46, 47, 48, 49, 50, 51, 53 and 54 are at or below the door threshold elevations and are unaffected by the sunny day breach of the Intake Canal

3. Failure of the Settling Pond has no effect on flood levels at SPS. The breach outflow from the Settling Pond flows directly into the Discharge Canal.

2.3.4. References

2.3.4-1 **Costa, 1985**. "Floods from Dam Failures", Costa, J.E., U.S. Geological Survey, Open-File Report 85-560, 1985.

2.3.4-2 **Dominion, 2014.** Surry Power Station Updated Final Safety Analysis Report, Revision 46.02.

2.3.4-3 **DWR, 2010.** Guidelines for Dam Breach Analysis, State of Colorado Department of Natural Resources Division of Water Resources, Office of the State Engineer Dam Safety Branch, February 2010.

2.3.4-4 **FERC, 1993.** Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 2 – Selecting and Accommodating Inflow Design Floods for Dams, Federal Energy Regulatory Commission (FERC), October 1993.

2.3.4-5 **FLO-2D, 2014a**. FLO-2D® Pro Reference Manual, FLO-2D Software, Inc., Nutrioso, Arizona.

2.3.4-6 **FLO-2D, 2014b**. FLO-2D® Pro Data Input Manual, FLO-2D Software, Inc., Nutrioso, Arizona.

2.3.4-7 **Fread, 1996**. "Dam-Breach Floods," Fread, D.L., 1996, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service.

2.3.4-8 **Froehlich, 2008**, "Embankment Breach Parameters and Their Uncertainties," Froehlich, David C. 2008. Journal of Hydraulic Engineering, 134(12), pp. 1708-1721.

2.3.4-9 **NOAA, 2013.** Tides and Currents, National Oceanic and Atmospheric Administration (http://tidesandcurrents.noaa.gov/map/ accessed on 11-19-2013).

2.3.4-10 **Reed, 2011**. "Validation of a new GIS Tool to Rapidly Develop Simplified Dam Break Models," Reed, Seann, and Halgren, James, September 2011, NOAA National Weather Service, Office of Hydrologic Development.

2.3.4-11 **SPS, 1986**. "Life Extension Evaluation of Intake / Discharge Structures and Canals", Project Topical Report for Unit No. 1 Surry Power Station, Stone and Webster Engineering Corporation, April 1986.

2.3.4-12 **SPS, 1994**. SPS Calculation CE-1128, "Intake Canal stability with an increase in water level to elevation 32 ft, MSL", Dominion Nuclear Engineering, January, 1994.

2.3.4-13 **SPS, 2010**. SPS Drawing No.5129-C-5001, Rev.11. "Plot Plan, Settling Pond Area, Surry Power Station – Unit 1 & 2", November, 2010.

2.3.4-14 **SPS, 2012**. Aerial Mapping for Surry Power Station, McKim & Creed, Inc., December 2012

2.3.4-15 **SPS, 2013**. Surry Power Station Procedure 0-AP-37.01, Abnormal Environmental Conditions, Revision 59

2.3.4-16 **SPS, 2014a**. SPS Calculation CE-1997, "Surry Power Station Intake Canal Stability Under Flood Conditions with High Water at Elevation 36.0 feet", Dominion Nuclear Engineering, October, 2014.

2.3.4-17 **SPS, 2014b**. SPS Calculation CE-2000,"High Level Intake Structure BDD/PMP Evaluation", Dominion Nuclear Engineering, December, 2014.

2.3.4-18 USACE, 2013. National Inventory of Dams, U. S. Army Corps of Engineers (http://geo.usace.army.mil/ – accessed on 1-31-2013).

2.3.4-19 USNRC, 2011. Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046, United States Nuclear Regulatory Commission, November 2011.

2.3.4-20 USNRC, 2012. Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3 and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident, U.S. Nuclear Regulatory Commission, March 2012.

2.3.4-21 USNRC, 2013. JLD-ISG-2013-01: Interim Staff Guidance Japan Lessons-Learned Project Directorate - Guidance For Assessment of Flooding Hazards Due to Dam Failure, Revision 0, U.S. Nuclear Regulatory Commission, July 2013.

2.3.4-22 USNRC, 2014. Screening and Prioritization Results Regarding Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident., Nuclear Regulatory Commission, October 3, 2014.

2.3.4-23 Wahl, 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters, T. L. Wahl, Journal of Hydraulic Engineering, ASCE, May 2004.

Table 2.3-1: Dam Breach Peak Flows for Hypothetical Dams

Hypothetical Dam	Dam Storage (acre-ft)	Dam Height (ft)	Peak Dam Breach Calculation				Distance of dam Upstream of SPS (miles)	Attenuated Peak Flow (cfs)
			Froelich (cfs)	MacDonald and Langridge-Monopolis (cfs)	Costa (cfs)	Maximum (cfs)		
Northwest Upper James	522,000	381	3,093,000	1,231,000	1,279,000	3,093,000	292	4,000
North Upper James	112,000	77	270,000	337,000	342,000	342,000	151	11,000
Central Upper James	198,000	184	942,000	612,000	627,000	942,000	70	188,000
Appomattox River	229,000	80	350,000	461,000	469,000	469,000	61	115,000
Lower James River	98,380	73	244,000	313,000	317,000	317,000	0	317,000

Table 2.3-2: Dam Breach Parameters for Dams Modeled in HEC-HMS

Dam Name	Hypothetical or Individual	Height of Breach (ft)	Top of Dam / Pool Elevation (ft)	Bottom of Breach Elevation (ft)	Average Breach Width (ft)	Side Slope (--)	Top Width (ft)	Bottom Width (ft)	Develop-ment Time (hours)	Storage (acre-feet)
Little Creek	Individual (VA09506)	67	67	0	201	0.5	235	168	0.5	32,143
Diascund Creek	Individual (VA12703)	35	35	0	105	0.5	123	88		29,093
Lower James	Hypothetical	73	73	0	185	0.5	222	149		98,380

Notes:

1. Assumed reservoir bottom elevation at zero. Height of breach equals dams height.
2. Used development time of 0.5 hour for earthen dams (FERC, 1993).
3. References:
   a. Breach parameters based on guidance from: Colorado DWR, 2010; FERC, 1993; Wahl, 2004
   b. Dam storage and height from USACE, 2013.

Table 2.3-3: HEC-HMS Calculated Peak Discharges with Combined Hypothetical Dams

Watershed	Subbasin / Dam Name	Peak Discharge from Element with dam break	Peak Discharge with dam break[1]	Peak Discharge without dam break[2]	Increase in Discharge due to Dam Failure
		(cfs)	(cfs)	(cfs)	(%)
Upper James River	Upper James River Subbasin	646,000	849,000	646,000	31%
	Northwest Upper James Hypothetical Dam	4,000			
	North Upper James Hypothetical Dam	11,000			
	Central Upper James Hypothetical Dam	188,000			
Appomattox River	Appomattox River Subbasin	143,000	258,000	143,000	80%
	Appomattox Hypothetical Dam	115,000			
Lower James River	Lower James River Subbasin	179,000	823,000	179,000	360%
	Lower James Hypothetical Dam	317,000			
	Little Creek Dam (VA09506)	265,000			
	Diascund Creek Dam (VA12703)	62,000			

Notes:
1. Equals the sum of the peak discharge from each element in the watershed.
2. SPS PMF without dam failure

Table 2.3-4: Selected Breach Geometry Parameters

Parameter	Range (USNRC, 2013; FERC, 1993)	Froehlich, 2008 (see estimates below)	Selected Value
Breach Width	19 feet to 47.5 feet (2 x Dam Height to 5 x Dam Height)	35 feet	47.5 feet (5 x Dam Height)
Ratio of Top and Bottom Breach Widths	1.0 to 1.7	-	1.0
Breach Formation Time	6 minutes to 1 hour	35 minutes	30 minutes

Notes:

The Froehlich, 2008 average breach width in meters $\bar{B}$ is:

$$\bar{B} = 0.27 k_o V_w^{0.32} H_b^{0.04}.$$

Where

$$k_o = \begin{cases} 1.3 & \text{for overtopping failures} \\ 1.0 & \text{for other failure modes} \end{cases}$$

V_w = volume of impoundment above minimum breach elevation in meters, and
H_b = height of embankment in meters (2.9 m)

The Froehlich, 2008 breach formation time (in seconds) t_f is:

$$t_f = 63.2 \sqrt{\frac{V_w}{g H_b^2}}$$

Where g is the standard gravitational acceleration of 9.81 m/s^2

For the purposes of this calculation, the Intake Canal embankment height is calculated to be 9.5 feet (top elevation of 36 ft, MSL –general site grade of 26.5 ft, MSL).

The impoundment volume of the Intake Canal from Elevation 30 feet, MSL to Elevation 26.5, MSL is estimated as 88,600 m^3. This is based on a canal length of 2810 meters, a top width at elevation 30 feet and 26.5 feet, MSL of 102 feet and 92 feet, respectively and side slopes of 1.5H to 1V.

Table 2.3-5: LIP Results with Breach of Settlement Pond

Door	Location	Representative Grid Element	Grid Elevation (ft, MSL/ Plant Datum)	Maximum Flood Depth without Settling Pond Breach (ft)	Maximum Flood Depth with Settling Pond Breach (ft)	Difference in Depths (ft)
1	Double Door into Old Admin Building	18,897	26.91	2.05	2.05	0.0
2	into Stairwell	17,883	26.61	2.07	2.05	0.0
3	into Stairweel and OSC	18,089	26.59	2.10	2.10	0.0
4	1-BS-DR-S27-15	18,294	26.50	2.22	2.20	0.0
5	into I&C Shop	18,498	26.52	2.22	2.20	0.0
6	1-BS-DR-S27-19A	19,109	26.29	2.59	2.56	0.0
7	1-BS-DR-S27-16A	19,312	25.81	3.14	3.11	0.0
8	0-SE-DR-CAS-100A	17,680	27.00	1.67	1.66	0.0
9	0-SE-DR-CAS-103A	17,478	27.01	1.71	1.69	0.0
10	0-SE-DR-CAS-101C	17,273	27.04	1.68	1.67	0.0
11	at ECST	17,066	28.72	0.20	0.20	0.0
12	1-BS-DR-SEC	19,294	27.00	1.98	2.01	0.0
13	1-BS-DR-T27-1 and -2	20,292	26.59	2.49	2.49	0.0
14	1-BS-DR-T27-3 and -4	22,478	26.85	2.47	2.47	0.0
15	1-BS-DR-FP27-1	22,259	26.56	2.63	2.63	0.0
16	at SE Corner of Unit 2	23,869	26.71	2.61	2.61	0.0
17	into Maintenance Offices	24,267	26.68	2.69	2.68	0.0
18	into Maintenance Offices	24,267	26.68	2.69	2.68	0.0
19	into Maintenance Offices	25,053	27.00	2.27	2.27	0.0
20	into Maintenance Offices	25,448	26.80	2.44	2.45	0.0
21	into Maintenance Offices	25,845	26.60	2.60	2.62	0.0
22	into Maintenance Offices	25,650	26.66	2.53	2.52	0.0
23	into Maintenance Offices	25,257	26.60	2.53	2.55	0.0
24	into Maintenance Offices	24,864	26.67	2.42	2.41	0.0
25	into CP Building	22,703	26.60	2.06	2.07	0.0
26	into CP Building	22,504	26.40	2.22	2.24	0.0
27	into CP Building	22,306	26.52	2.09	2.10	0.0
28	1-BS-DR-DVP27-1	15,191	27.00	1.17	1.16	0.0
29 - 30	Not Used					
31	into Aux Boiler Room	21,520	26.39	2.07	2.08	0.0
32	into Storeroom	21,518	26.32	2.09	2.10	0.0
33	1-BS-DR-67	21,319	26.32	2.05	2.05	0.0
34	1-BS-DR-66	21,119	26.66	1.66	1.65	0.0
35	1-BS-DR-65	20,920	26.74	1.57	1.60	0.0
36	1-BS-DR-CS27-3	20,724	26.68	1.61	1.63	0.0
37	1-BS-DR-SG28-2	20,133	25.95	2.27	2.28	0.0
38	1-BS-DR-27-3	18,525	24.19	3.85	3.84	0.0
39	1-BS-DR-D23-1	18,118	23.00	5.03	5.03	0.0
40	1-BS-DR-D23-2	18,118	23.00	5.03	5.03	0.0
41	2-BS-DR-DVP27-2	18,538	26.79	1.24	1.25	0.0
42	1-BS-DR-73	18,129	26.57	1.36	1.36	0.0
43	1-BS-DR-74	17,310	26.02	0.89	0.89	0.0
44	1-BS-DR-F27-5 and -7	17,711	26.78	1.23	1.23	0.0
45	1-BS-DR-F27-4	17,092	26.88	1.14	1.13	0.0
46	1-BS-DR-F27-3 and -6	17,087	26.85	1.30	1.31	0.0
47	Above Grade 5'	17,498	27.08	1.09	1.09	0.0
48	Above Grade 5'	17,498	27.08	1.09	1.09	0.0
49	1-BS-DR-WG27-1	17,495	25.84	2.33	2.32	0.0
50	1-BS-DR-F27-1 and -2	17,697	27.31	0.90	0.92	0.0
51	1-BS-DR-SG28-1	17,686	26.34	2.39	2.38	0.0
52	1-BS-DR-CS-27-1	18,704	26.24	2.56	2.55	0.0
53	at ECST	19,741	27.53	0.58	0.58	0.0
54	at Auxiliary Building	19,725	26.73	1.79	1.72	-0.1
55	1-BS-DR-A27-8	18,915	26.05	3.00	2.95	0.0
56	1-BS-DR-A27 1 and -2	19,317	25.82	3.22	3.18	0.0
57	at Auxiliary Building	19,923	26.59	1.92	1.87	0.0
58	into Clean Change	20,122	26.69	1.82	1.77	-0.1
59	into Clean Change	20,720	26.39	2.00	2.00	0.0

Note: Results for Maximum flood depths without Settling Pond breach are from the SPS LIP calculation using site specific probable maximum precipitation

Table 2.3-6: Intake Canal Sunny Day Breach Results

General Information					Failure Location 1					Failure Location 2					Failure Location 3				
Door #	Location	Representative Grid Element	Grid Elevation (ft, MSL/Plant Datum)	Threshold Elevation (ft, MSL/ Plant Datum)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Time to Maximum Flood Elevation (hrs)	Maximum Velocity (fps)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Time to Maximum Flood Elevation (hrs)	Maximum Velocity (fps)	Maximum Flood Elevation (ft, NAVD88)	Maximum Flood Elevation (ft, MSL / Plant Datum)	Maximum Flood Depth (ft)	Time to Maximum Flood Elevation (hrs)	Maximum Velocity (fps)
1	Double Door into Old Admin Building	18,897	26.9	27.0	26.5	27.9	1.0	2.0	0.3	26.5	27.9	1.0	2.0	0.3	26.4	27.9	0.9	2.1	0.3
2	into Stairwell	17,883	26.6	27.2	26.4	27.9	1.2	2.0	3.0	26.4	27.8	1.2	1.7	2.8	26.4	27.8	1.2	2.0	2.7
3	into Stairweel and OSC	18,089	26.6	27.2	26.4	27.8	1.3	1.9	1.5	26.4	27.8	1.2	1.7	1.5	26.3	27.8	1.2	2.0	1.4
4	1-BS-DR-S27-15	18,294	26.5	27.2	26.4	27.8	1.3	2.0	0.8	26.4	27.8	1.3	1.7	0.9	26.3	27.8	1.3	2.0	0.8
5	into I&C Shop	18,498	26.5	27.2	26.4	27.8	1.3	1.9	0.6	26.4	27.8	1.3	1.8	0.6	26.3	27.8	1.3	2.0	0.6
6	1-BS-DR-S27-19A	19,109	26.3	27.0	26.4	27.8	1.5	2.0	2.0	26.4	27.8	1.5	1.8	2.0	26.3	27.8	1.5	2.1	1.8
7	1-BS-DR-S27-16A	19,312	25.8	27.0	26.4	27.8	2.0	2.1	1.0	26.4	27.8	2.0	1.8	1.1	26.3	27.8	2.0	2.1	0.8
8	0-SE-DR-CAS-100A	17,680	27.0	28.0	26.4	27.8	0.8	1.9	0.4	26.4	27.8	0.8	1.7	0.5	26.3	27.8	0.8	2.0	0.4
9	0-SE-DR-CAS-103A	17,478	27.0	28.0	26.4	27.8	0.8	2.0	0.3	26.4	27.8	0.8	1.8	0.3	26.3	27.8	0.8	2.1	0.3
10	0-SE-DR-CAS-101C	17,273	27.0	28.0	26.4	27.8	0.8	2.0	0.2	26.4	27.8	0.8	1.8	0.2	26.3	27.8	0.7	2.1	0.2
11	at ECST	17,066	28.7	30.5	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
12	1-BS-DR-SEC	19,294	27.0	27.0	26.5	27.9	0.9	1.9	1.4	26.5	27.9	0.9	1.8	1.1	26.4	27.9	0.9	2.1	0.8
13	1-BS-DR-T27-1 and -2	20,292	26.6	27.0	26.5	28.0	1.4	1.9	1.2	26.5	27.9	1.4	1.8	1.1	26.5	27.9	1.3	2.0	0.8
14	1-BS-DR-T27-3 and -4	22,478	26.9	27.0	26.6	28.1	1.2	1.9	0.7	26.7	28.1	1.3	1.8	0.7	26.6	28.0	1.2	1.9	0.7
15	1-BS-DR-FP27-1	22,259	26.6	27.0	26.6	28.0	1.5	2.0	1.0	26.5	28.0	1.4	1.8	1.1	26.5	27.9	1.4	2.0	1.3
16	at SE Corner of Unit 2	23,869	26.7	27.0	26.5	28.0	1.3	1.9	0.1	26.6	28.1	1.3	1.8	0.1	26.7	28.1	1.3	1.0	0.1
17	into Maintenance Offices	24,267	26.7	27.0	26.5	28.0	1.3	1.9	0.2	26.6	28.1	1.4	1.8	0.2	26.7	28.1	1.4	1.0	0.2
18	into Maintenance Offices	24,267	26.7	27.0	26.5	28.0	1.3	1.9	0.2	26.6	28.1	1.4	1.8	0.2	26.7	28.1	1.4	1.0	0.2
19	into Maintenance Offices	25,053	27.0	27.0	26.5	28.0	1.0	1.9	1.1	26.6	28.0	1.0	1.7	1.2	26.7	28.1	1.1	1.0	0.6
20	into Maintenance Offices	25,448	26.8	27.0	26.5	27.9	1.1	2.0	1.4	26.6	28.0	1.2	1.9	1.6	26.7	28.1	1.3	1.0	1.3
21	into Maintenance Offices	25,845	26.6	27.0	26.5	27.9	1.3	1.9	3.0	26.5	28.0	1.4	1.8	3.2	26.6	28.0	1.4	1.0	2.2
22	into Maintenance Offices	25,650	26.7	27.0	26.5	27.9	1.2	2.1	2.5	26.5	28.0	1.3	1.8	2.6	26.6	28.0	1.4	1.7	2.2
23	into Maintenance Offices	25,257	26.6	27.0	26.4	27.9	1.3	2.1	1.9	26.5	27.9	1.3	1.9	2.3	26.5	28.0	1.4	1.7	1.6
24	into Maintenance Offices	24,864	26.7	27.0	26.4	27.8	1.2	2.0	1.6	26.5	27.9	1.2	1.7	1.7	26.5	28.0	1.3	1.7	1.4
25	into CP Building	22,703	26.6	27.0	26.3	27.7	1.1	2.0	1.0	26.3	27.7	1.1	2.1	1.6	26.3	27.8	1.2	1.7	1.1
26	into CP Building	22,504	26.4	28.0	26.2	27.7	1.3	2.1	1.9	26.3	27.7	1.3	1.8	3.0	26.3	27.8	1.4	1.7	1.2
27	into CP Building	22,306	26.5	27.0	26.2	27.7	1.2	2.0	0.8	26.3	27.7	1.2	1.8	0.9	26.3	27.8	1.2	1.5	0.7
28	1-BS-DR-DVP27-1	15,191	27.0	27.0	25.9	27.3	0.3	4.0	0.1	25.8	27.3	0.3	6.4	0.0	25.6	27.1	0.1	15.1	0.0
29 - 30	Not Used																		
31	into Aux Boiler Room	21,520	26.4	27.0	26.2	27.6	1.2	2.0	1.2	26.2	27.7	1.3	1.9	2.0	26.3	27.7	1.3	1.7	2.0
32	into Storeroom	21,518	26.3	27.0	26.2	27.6	1.3	2.2	1.4	26.2	27.6	1.3	1.8	2.8	26.2	27.7	1.3	1.6	3.2
33	1-BS-DR-67	21,319	26.3	27.0	26.1	27.6	1.3	2.1	3.2	26.2	27.6	1.3	1.9	4.1	26.2	27.6	1.3	1.7	4.4
34	1-BS-DR-66	21,119	26.7	27.0	26.1	27.5	0.9	2.0	0.6	26.1	27.6	0.9	1.8	0.8	26.2	27.6	0.9	1.6	1.0
35	1-BS-DR-65	20,920	26.7	27.0	26.1	27.5	0.8	2.0	0.6	26.1	27.6	0.8	2.0	1.1	26.1	27.6	0.8	1.6	1.2
36	1-BS-DR-CS27-3	20,724	26.7	27.5	26.1	27.5	0.8	2.0	0.7	26.1	27.6	0.9	2.0	1.0	26.1	27.6	0.9	1.9	1.1
37	1-BS-DR-SG28-2	20,133	26.0	29.0	26.1	27.5	1.6	2.1	0.9	26.1	27.5	1.6	1.9	1.1	26.1	27.6	1.6	1.7	1.6
38	1-BS-DR-27-3	18,525	24.2	27.0	25.7	27.2	3.0	4.9	0.1	25.7	27.2	3.0	3.7	0.2	25.8	27.2	3.1	3.1	0.2
39	1-BS-DR-D23-1	18,118	23.0	23.0	25.7	27.2	4.2	4.8	0.3	25.7	27.2	4.2	3.7	0.4	25.8	27.2	4.2	3.2	0.4
40	1-BS-DR-D23-2	18,118	23.0	23.0	25.7	27.2	4.2	4.8	0.3	25.7	27.2	4.2	3.7	0.4	26.8	28.2	4.2	3.2	0.4
41	2-BS-DR-DVP27-2	18,538	26.8	27.0	25.8	27.2	0.4	2.2	0.3	25.8	27.2	0.4	2.0	0.3	25.8	27.3	0.5	1.7	0.3
42	1-BS-DR-73	18,129	26.6	26.7	25.7	27.2	0.6	2.2	1.7	25.8	27.2	0.6	1.9	1.8	25.8	27.2	0.7	1.7	2.2
43	1-BS-DR-74	17,310	26.0	26.7	24.9	26.3	0.3	2.3	0.9	24.9	26.3	0.3	2.0	1.0	25.0	26.4	0.4	1.8	1.1
44	1-BS-DR-F27-5 and -7	17,711	26.8	27.0	25.5	26.9	0.2	6.7	0.0	25.6	27.1	0.3	5.5	0.0	25.8	27.2	0.4	4.1	0.1
45	1-BS-DR-F27-4	17,092	26.9	27.0	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
46	1-BS-DR-F27-3 and -6	17,087	26.9	27.0	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
47	Above Grade 5'	17,498	27.1	32.6	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
48	Above Grade 5'	17,498	27.1	32.6	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
49	1-BS-DR-WG27-1	17,495	25.8	27.5	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
50	1-BS-DR-F27-1 and -2	17,697	27.3	27.5	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
51	1-BS-DR-SG28-1	17,686	26.3	29.0	26.4	27.8	1.5	2.0	0.8	26.4	27.8	1.5	1.8	0.8	26.3	27.8	1.4	2.1	0.7
52	1-BS-DR-CS-27-1	18,704	26.2	27.5	26.4	27.8	1.6	2.0	1.3	26.4	27.8	1.6	1.8	1.4	26.3	27.8	1.5	2.1	1.3
53	at ECST	19,741	27.5	30.5	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry	dry
54	at Auxiliary Building	19,725	26.7	27.6	26.1	27.5	0.8	2.0	0.1	26.1	27.5	0.8	2.0	0.3	26.1	27.6	0.8	1.6	0.3
55	1-BS-DR-A27-8	18,915	26.1	27.5	26.4	27.8	1.8	2.1	0.1	26.4	27.8	1.8	1.9	0.1	26.3	27.8	1.7	2.1	0.1
56	1-BS-DR-A27 1 and -2	19,317	25.8	27.5	26.4	27.8	2.0	2.1	0.6	26.4	27.8	2.0	2.0	0.6	26.3	27.8	2.0	2.1	0.6
57	at Auxiliary Building	19,923	26.6	27.5	26.1	27.5	0.9	2.0	0.6	26.1	27.5	1.0	2.0	0.7	26.1	27.6	1.0	1.6	0.8
58	into Clean Change	20,122	26.7	27.0	26.1	27.5	0.8	2.0	0.3	26.1	27.5	0.9	2.0	0.4	26.1	27.6	0.9	1.6	0.4
59	into Clean Change	20,720	26.4	27.0	26.1	27.5	1.1	2.0	1.1	26.1	27.5	1.2	2.0	1.2	26.1	27.6	1.2	1.6	1.9

Note: Grey highlights indicate door locations that are not affected by the sunny day failure of the Intake Canal (completely dry or flood depth is at or below door threshold elevation); Yellow highlights indicate door locations where the highest flood depths occur based on all three breach/failure locations; "Dry" refers to door locations that are not in the path of the dam breach flood wave and therefore remain completely dry.

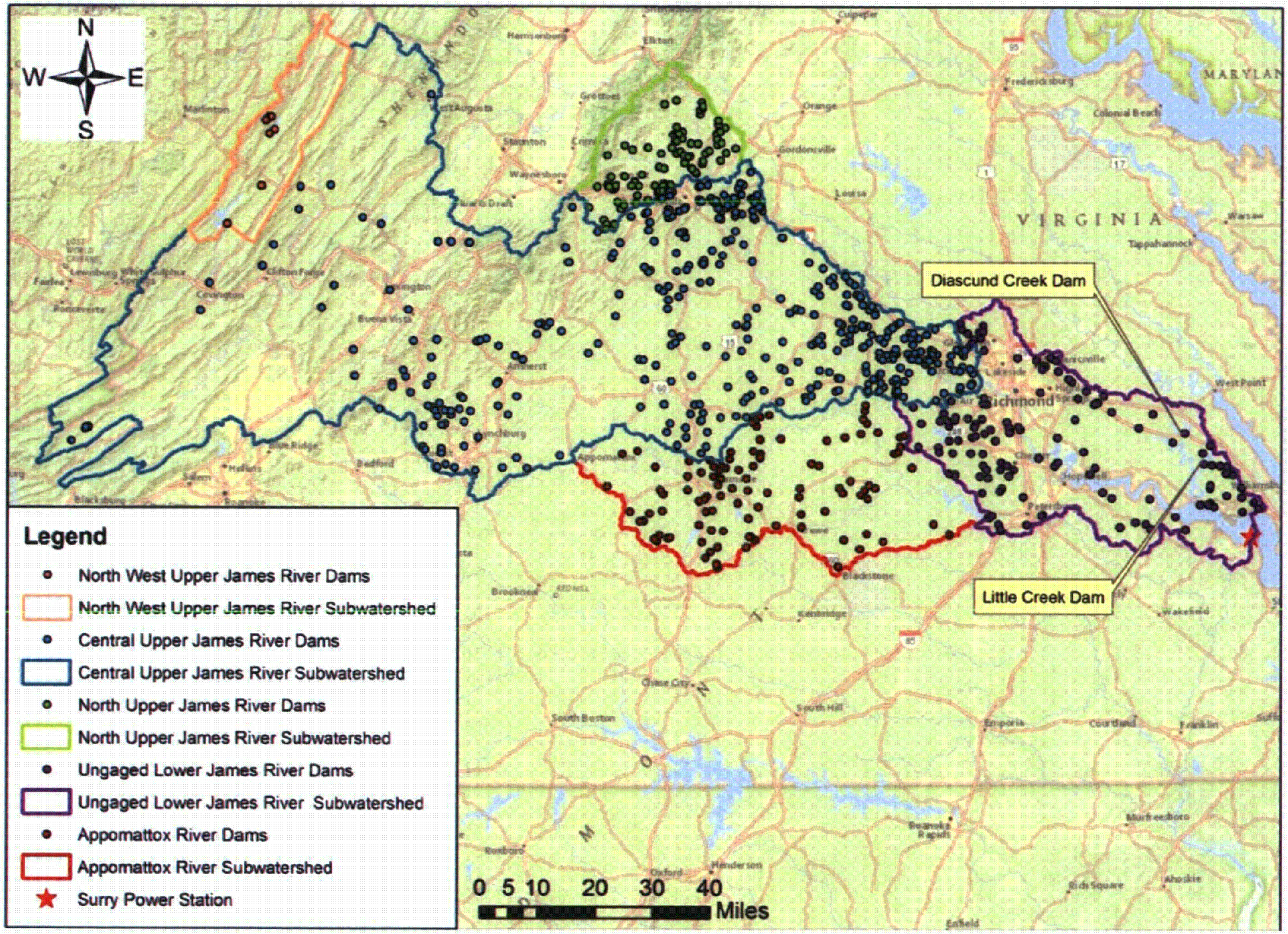


Figure 2.3-1: NID Dams within Delineated Subwatershed Areas

Figure 2.3-2: SPS Watershed Topography with Major Stream Channels

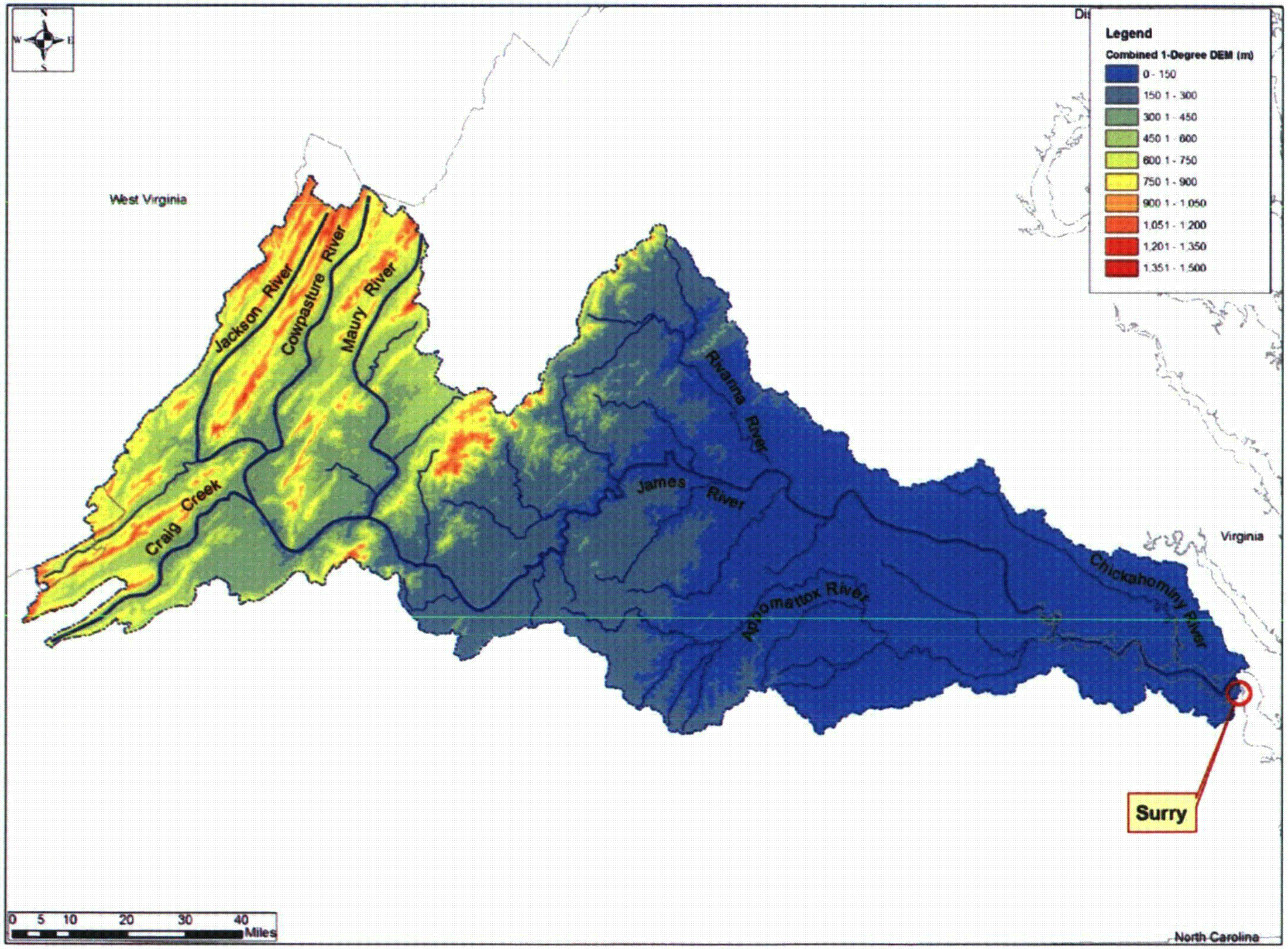


Figure 2.3-3: Hypothetical Dam Locations with Stream Channels

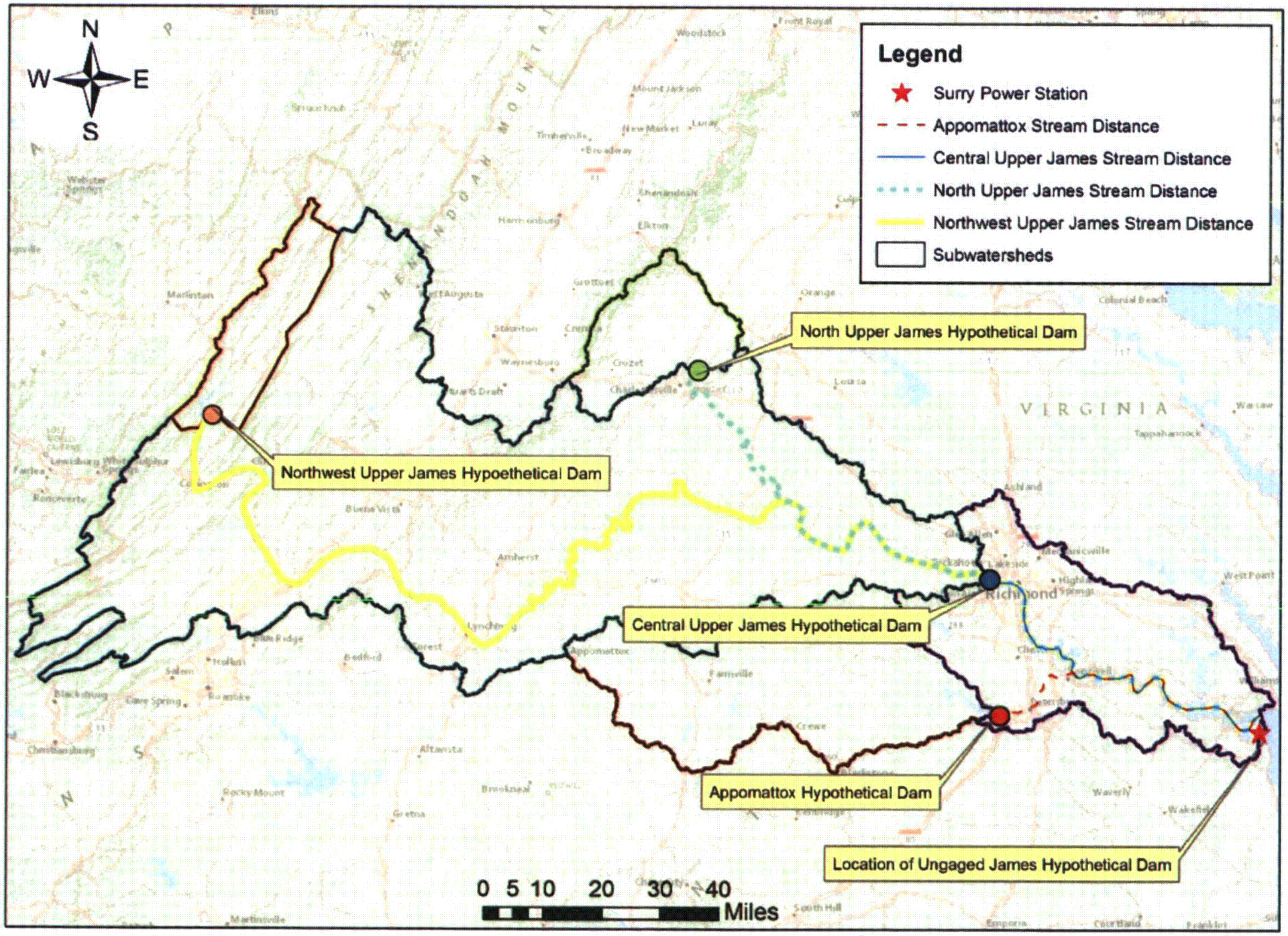
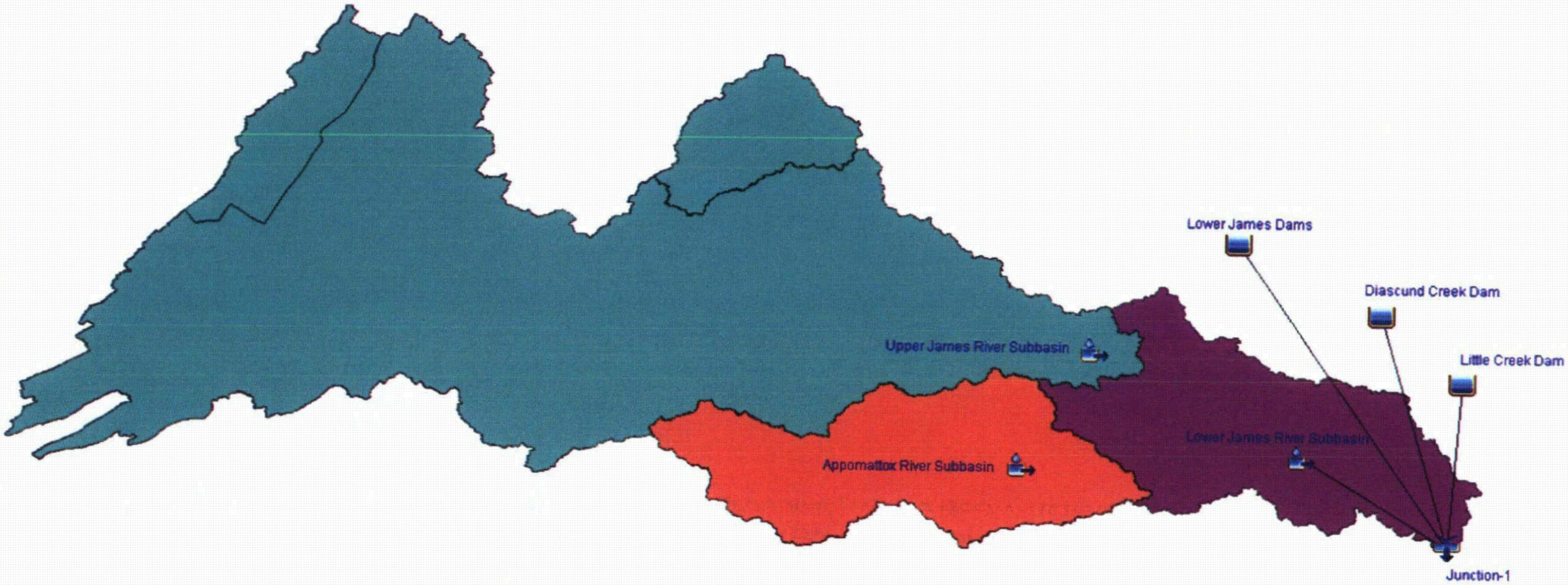


Figure 2.3-4: HEC-HMS Dam Breach Model Schematic



Note: This figure was developed from HEC-HMSv3.5. The breach flows from the three hypothetical dams in the Upper James River subwatershed and the one hypothetical dam in the Appomattox River subwatershed were calculated using regression equations were not included in the HEC-HMS model.

Figure 2.3-5: HEC-RAS James River Water Surface Profile

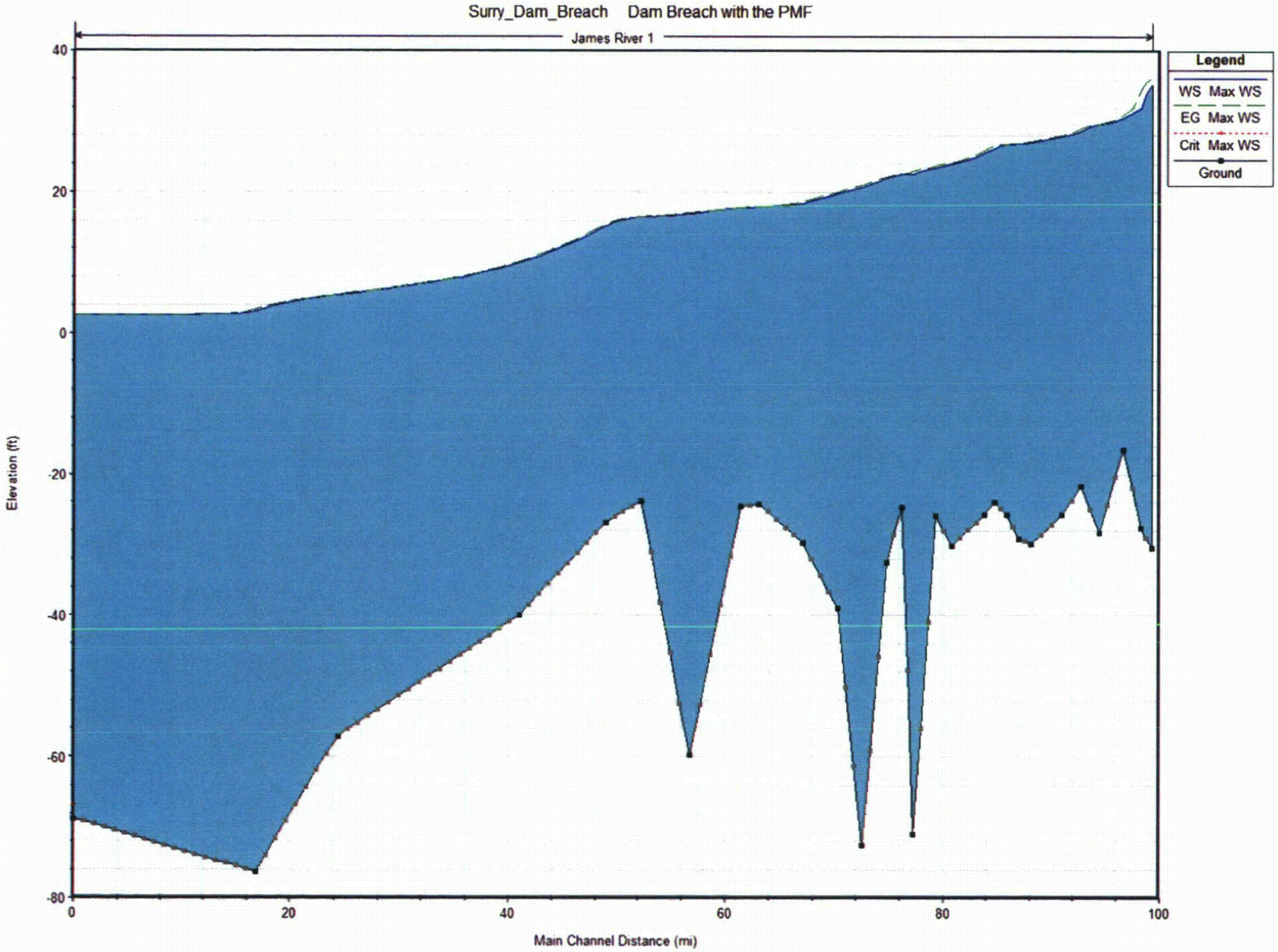
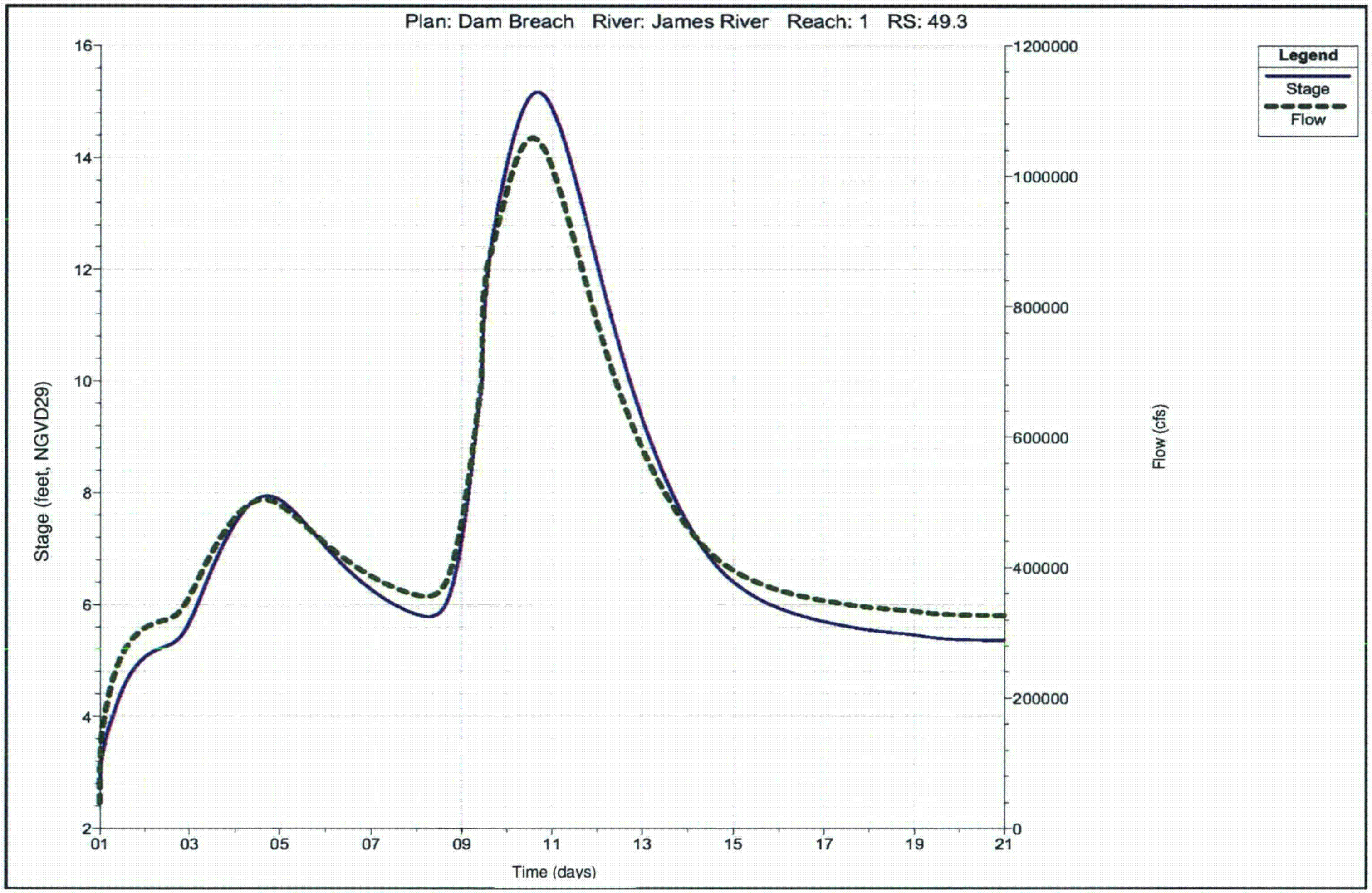


Figure 2.3-6: SPS Dam Breach Flow and Stage Hydrographs



Note: This figure was generated with HEC-RAS v.4.1.

Figure 2.3-7: On-site Impoundment Breach Locations

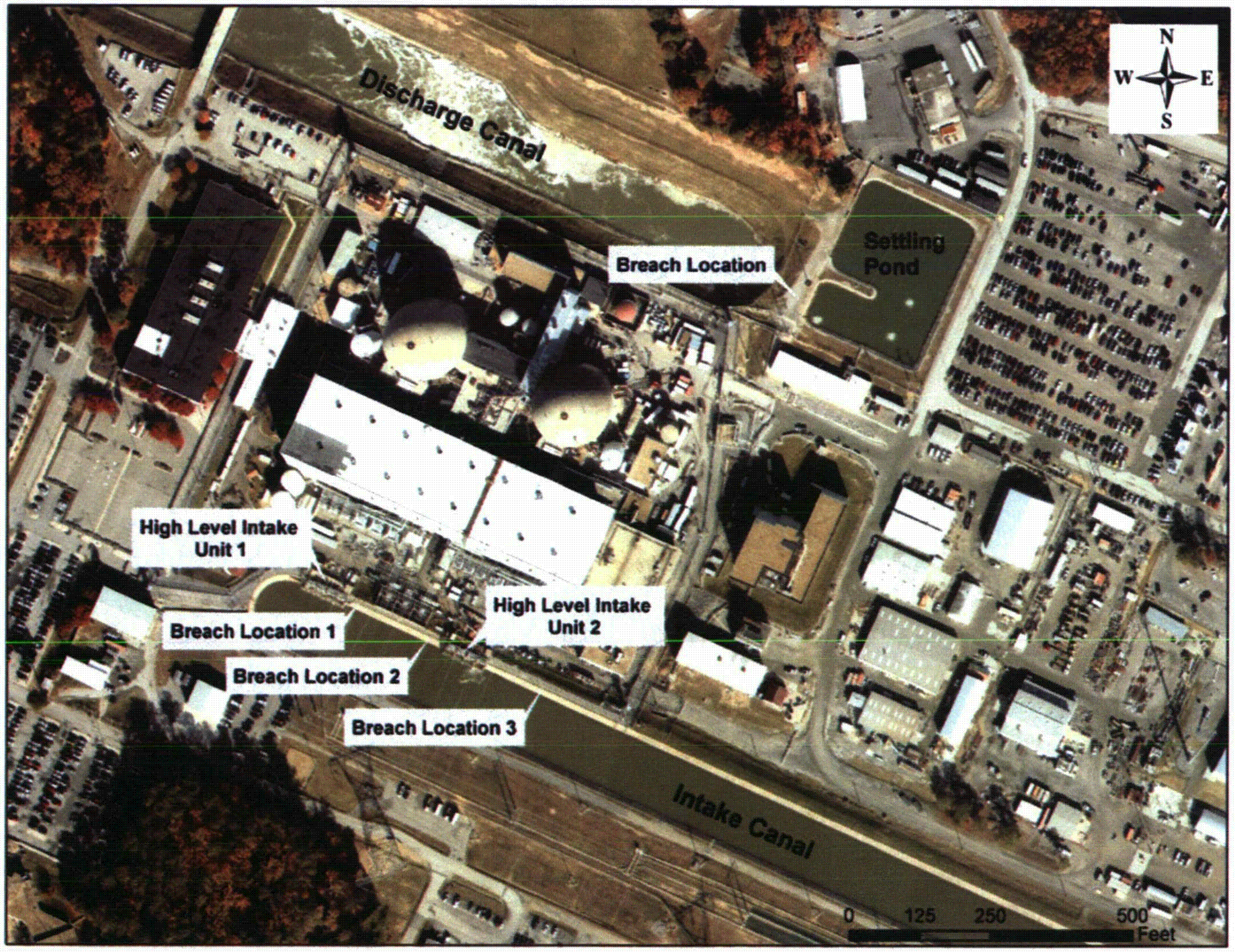


Figure 2.3-8a: Maximum Water Surface Elevation for Intake Canal Breach Location 1

Figure 2.3-8b: Maximum Water Surface Elevation for Intake Canal Breach Location 2

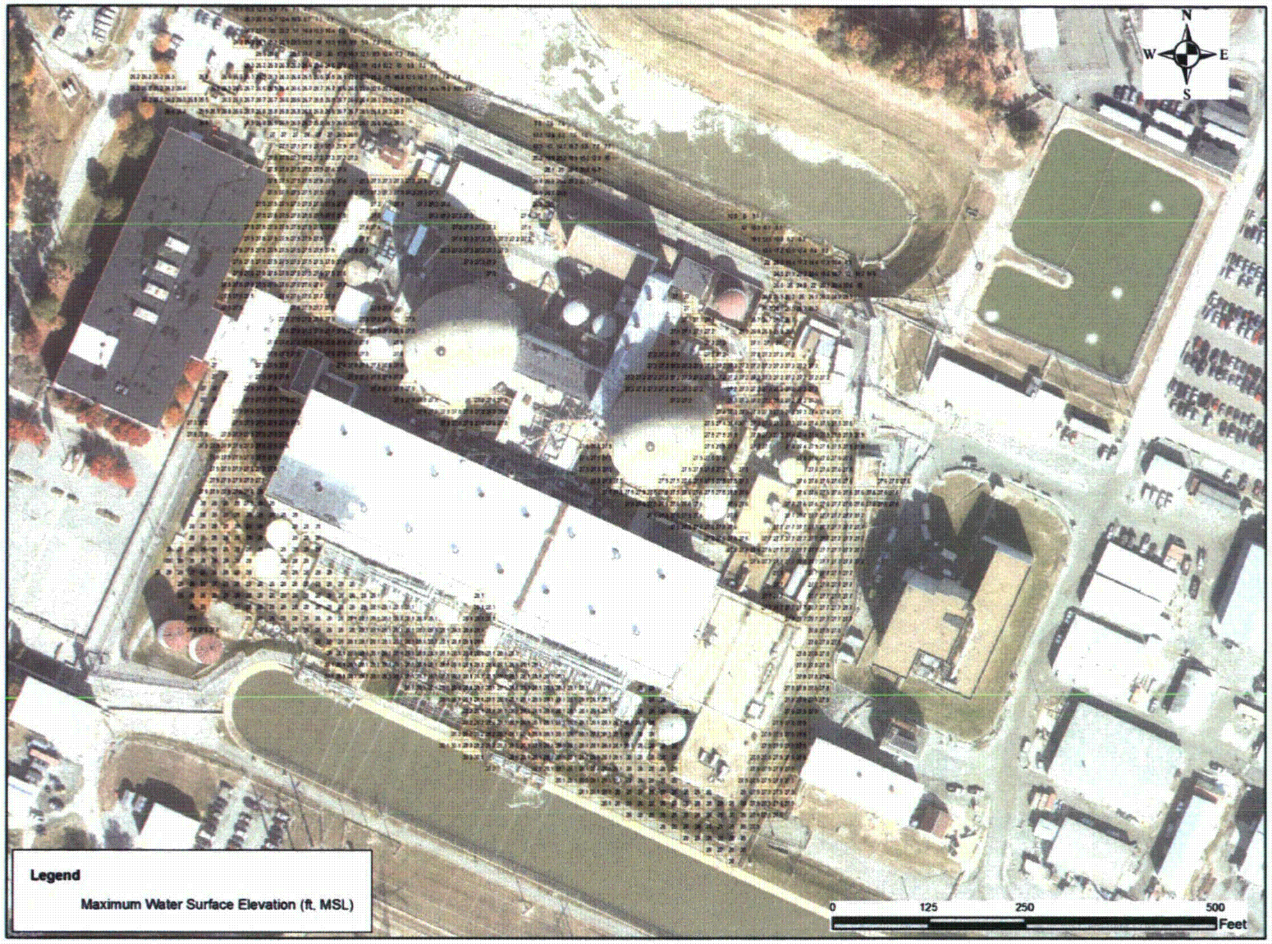


Figure 2.3-8c: Maximum Water Surface Elevation for Intake Canal Breach Location 3

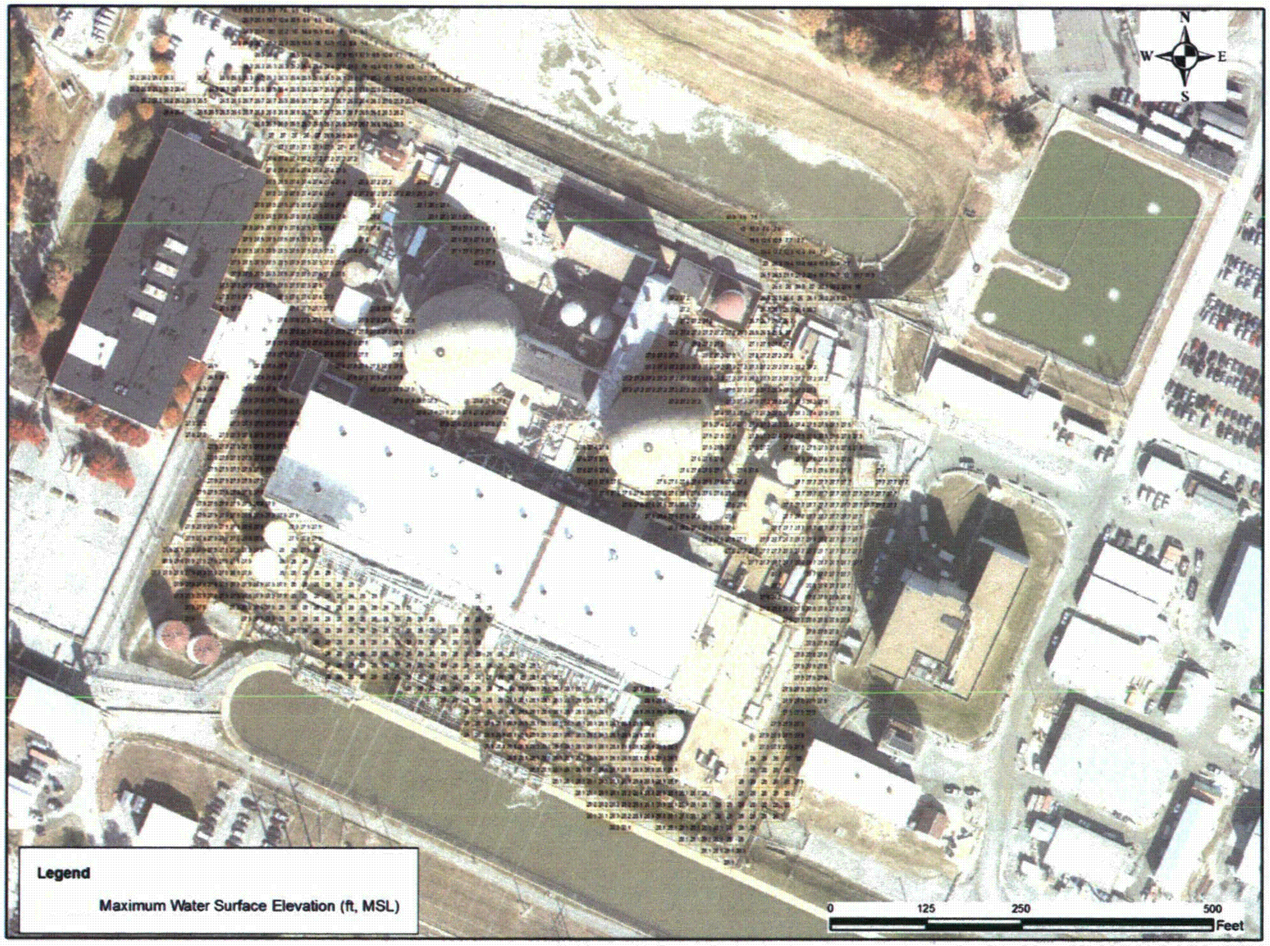


Figure 2.3-9a: Maximum Flood Depths for Intake Canal Breach Location 1

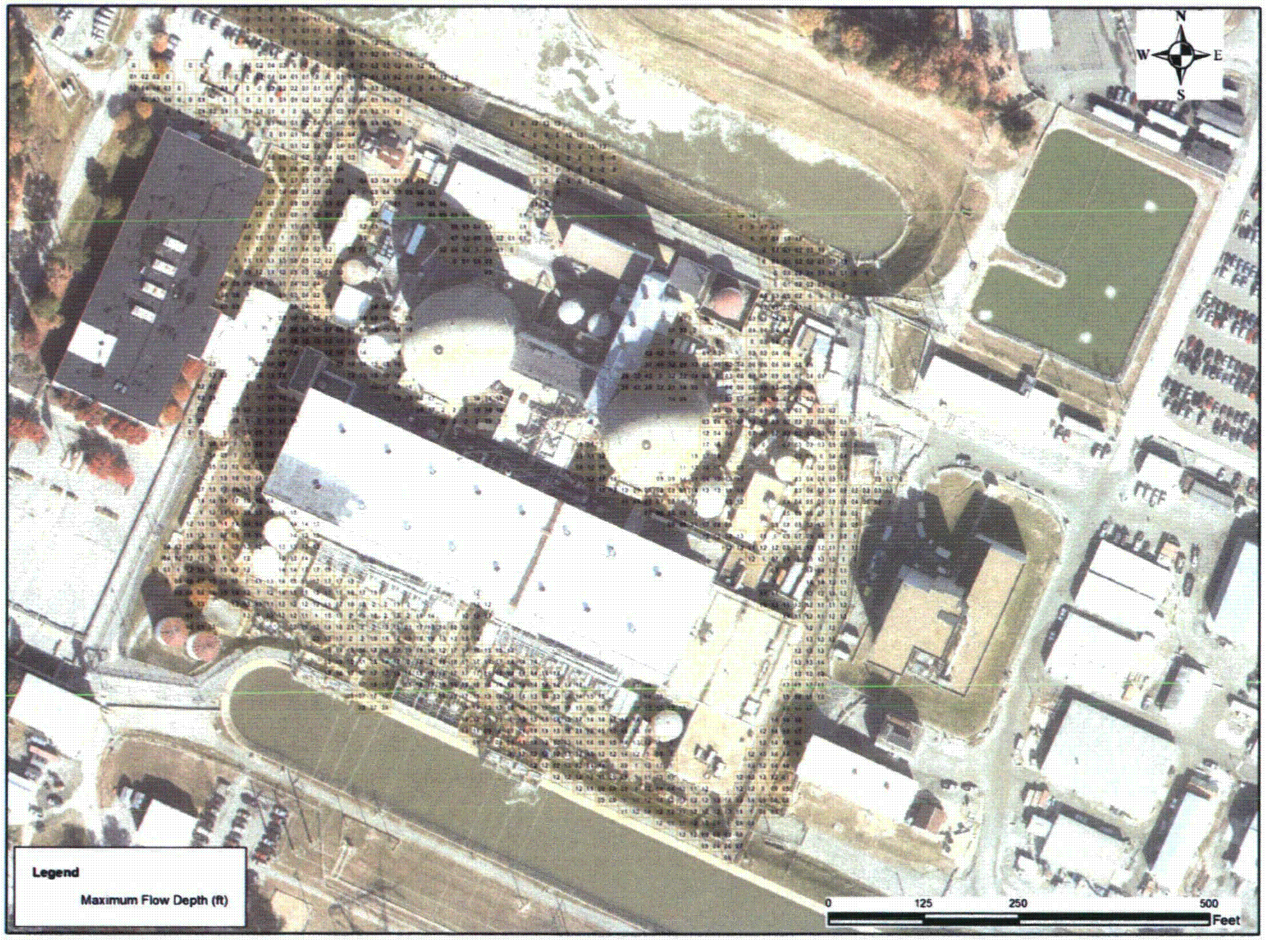


Figure 2.3-9b: Maximum Flood Depths for Intake Canal Breach Location 2

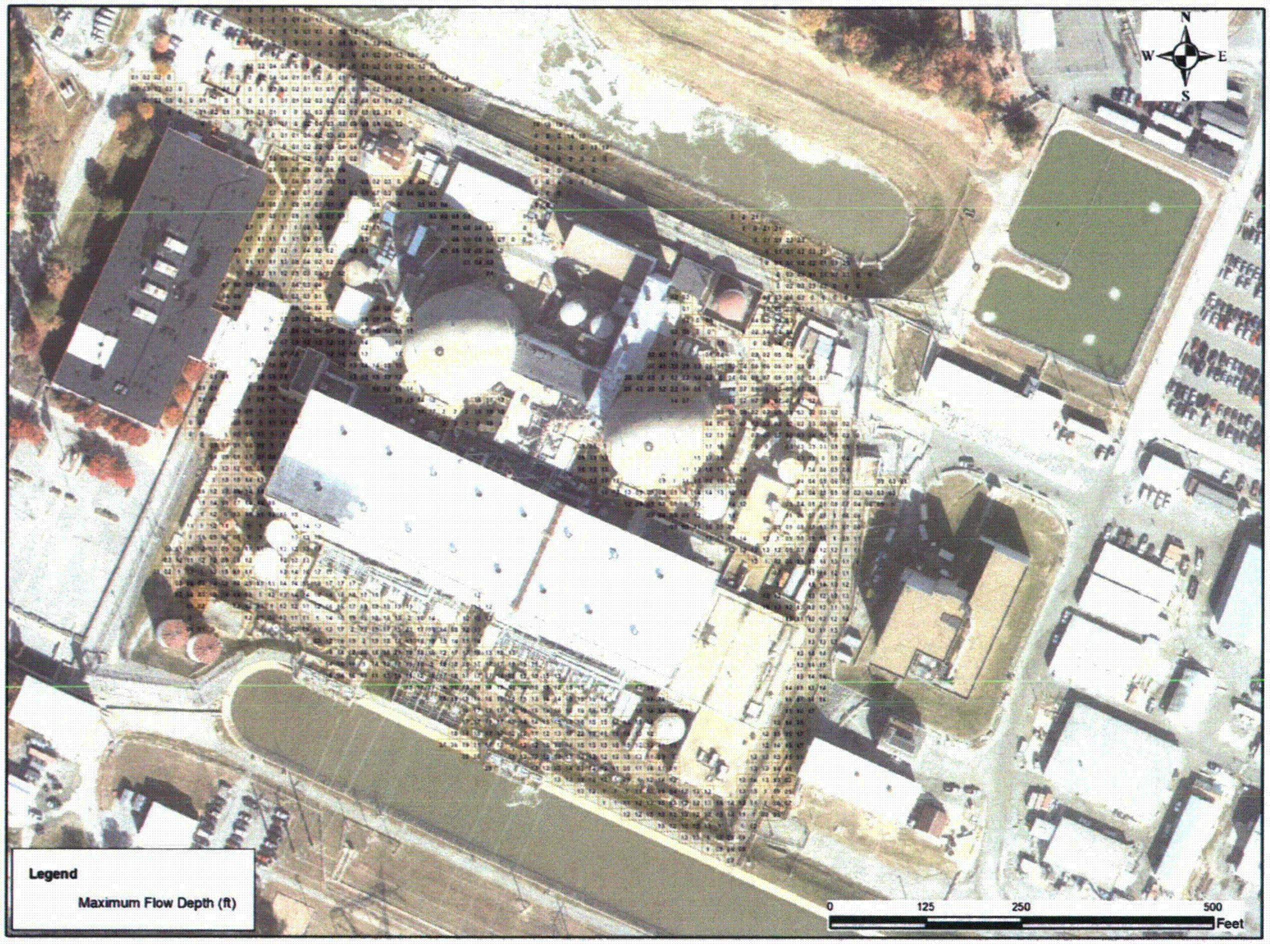
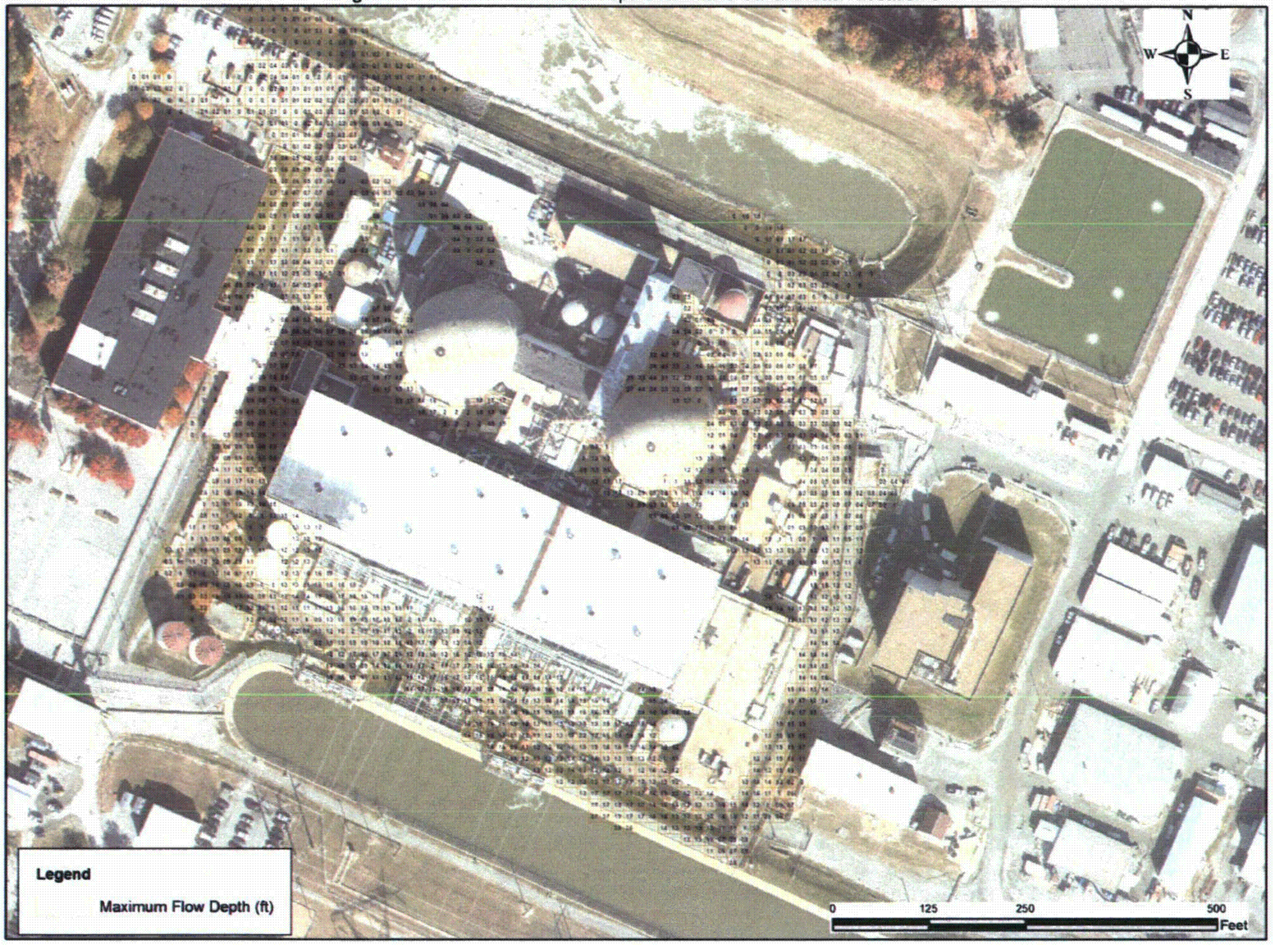


Figure 2.3-9c: Maximum Flood Depths for Intake Canal Breach Location 3



2.4. Probable Maximum Storm Surge

An evaluation of the Probable Maximum Storm Surge (PMSS) flood hazard at SPS was performed in a manner consistent with the HHA approach (NRC, 2011). The evaluation included detailed analyses of the Probable Maximum Hurricane (PMH), the resulting deterministic stillwater elevation (i.e., the water surface elevation in the absence of waves, wave set-up and river flood PMSS) and the probabilistically-derived 1E-6 Annual Exceedance Probability (AEP) stillwater elevation.

To support the deterministic PMSS analysis, parameters defining the PMH were developed through a review of National Weather Service (NWS) guidance and a Site and Region Specific Hurricane Climatology Study, which included an analysis of a large set of synthetic hurricane data. Hydrodynamic modeling was then used to simulate the parameterized PMH and identify the resulting PMSS elevation at SPS. The Site and Region Specific Hurricane Climatology Study also supported the evaluation of the 1E-6 AEP stillwater elevation by providing probabilistic definitions of key hurricane parameters. These definitions were used as input to a method of recovering combined event probabilities to evaluate the relationship between maximum stillwater elevation and AEP at SPS.

The methodology and results associated with each of the analyses described in the following sections were the subjects of an independent review performed by Dr. Donald T. Resio, Professor of Ocean Engineering at the University of North Florida (refer to Attachment C). The favorable findings of this review act to support the conclusions presented with respect to the PMSS flooding hazard at SPS.

2.4.1 Methodology

The following sections summarize the methodology used to evaluate the PMH, the deterministic stillwater PMSS elevation (i.e., deterministic storm surge) and the probabilistically-derived 1E-6 AEP stillwater PMSS elevation (i.e., probabilistic storm surge) at SPS.

2.4.1.1 <u>Probable Maximum Hurricane</u>

A step-wise approach consistent with HHA methodology, as described by NUREG/CR-7046 (NRC, 2011), was used to deterministically evaluate the PMH at SPS. The evaluation included analyses of National Hurricane Center (NHC) historical, "Best-Track" hurricane data (i.e., HURDAT2) and, to supplement the limited historical data, synthetic hurricane data representative of a large set of synthetic tropical cyclone tracks. The synthetic data were developed for the SPS region by Dr. Kerry Emanuel of WindRiskTech, LLC (WRT) using coupled intensity and atmospheric models (WRT, 2013). Details of the development methodology, comparisons with historical hurricane data and the application of extant methods are described in two key peer reviewed references (Emanuel et al, 2004; Emanuel et al 2006).

The methodology used to develop the synthetic tropical cyclone tracks and storm parameters includes: 1) storm generation; 2) storm track generation; and 3) deterministic modeling of hurricane intensity. Using this methodology, a large number (10,013) of synthetic storm tracks (i.e., WRT storm set) was generated and filtered within a radius of 200 kilometers (km) of Westhampton, New York to support the evaluation of the PMH at SPS. The WRT storm set was verified by evaluating the statistical consistency with the HURDAT2 data set and, where present, variance from the historical hurricane data was identified. Comparisons were performed for parameters reflecting storm intensity, direction, size and speed. Based on these comparisons, the WRT data set was ultimately validated as a conservative reflection of storm characteristics within the SPS vicinity. Following validation, the WRT storm set was spatially filtered to eliminate data not reflective of conditions within the potential storm surge production region (i.e., filtered to include only over-water points within the 200 km radius zone)

The following steps were used to evaluate the historical and synthetic data and characterize the PMH at SPS:

1. Identification of controlling event type: hurricanes and extra-tropical storms are of primary concern in Atlantic coastal areas located in the regional area of SPS. The first calculation step is to confirm which type of storm event (i.e. hurricane or extra-tropical storm) controls the PMSS. This evaluation is performed by: 1) reviewing recorded water level data from the NOAA CO-OPS Stations at the Sewells Point, Virginia (Station 8638610, "Sewells Point"), Chesapeake Bay Bridge Tunnel (Station 8638863, "CBBT"), and Kiptopeke, Virginia (Station 863220, "Kiptopeke") stations to identify the events that resulted in historical extreme water levels (Figure 2.4-1); 2) examining the extreme water levels predicted for Category 1 through Category 4 Hurricanes storm surge elevations by NOAA using the Sea, Lakes, and Overland Surges from Hurricanes (SLOSH) software model and presented in the NOAA SLOSH Display Program at the three NOAA CO-OPS stations located near SPS relative to recorded water level data (NOAA 2012a); and 3) reviewing the historical hurricanes in the vicinity of SPS using historical storm data from the years 1851-2010 (Blake et al 2011 and NOAA, 2013).

2. Determine NWS-23 PMH parameters: consistent with guidance presented in NUREG/CR-7046 (NRC, 2011), ranges of permissible PMH meteorological parameters are initially determined using NWS 23 (NOAA 1979). These parameters include the following: 1) peripheral pressure, 2) central pressure, 3) permissible range for radius of maximum winds, 4) permissible range of forward speeds, 5) permissible range of track direction, and 6) estimated maximum 10-meter, 10-minute over-water wind speed. Figure 2.4-2 illustrates several of the key PMH parameters.

3. Site and Region Specific Hurricane Climatology Study: the Site and Region Specific Hurricane Climatology Study included a statistical analysis of the HURDAT2 database and verification and statistical analysis of the synthetically-developed hurricane parameter data set. The analysis focused on data reflecting storm intensity, direction and physical dimensions in the region of SPS. Parameter selection was based on data availability within the HURDAT2 database and relevance with respect to comparison to parameter estimates derived from NWS 23. Probability Density Functions (PDFs) were constructed for these parameters from Probability Density Histograms (PDHs) using a non-parametric kernel method. To further refine the analysis of the low probability portion of the 1-min, 10-m average wind speed (mxw) distribution, Extreme Value Analysis (EVA) was used based on the Peak Over Threshold (POT) method and the Generalized Pareto Distribution (GPD). A detailed statistical analysis of the filtered WRT storm data was then performed including a univariate storm parameter probability analysis, an analysis of storm parameter covariance, development of a large synthetic storm set extension (i.e., the 3,000,000 or 3M data set), and an evaluation of error, uncertainty, and conservatism. Based on this analysis, a

dimensionless scaling function was developed to conservatively reflect the deterministic upper-limit of storm intensity (i.e., maximum wind speed) in the SPS vicinity in consideration of co-variability with storm direction (i.e., storm bearing). Additional relationships were also developed to characterize additional storm parameters (e.g., forward speed and radius of maximum winds) in consideration of parameter co-variability.

The above-described methodology provided input to the deterministic and probabilistic storm surge analyses described below.

2.4.1.2 Deterministic Storm Surge

A step-wise approach consistent with HHA methodology, as described by NUREG/CR-7046 (NRC, 2011), was used to deterministically evaluate the PMSS stillwater elevation at SPS. As discussed below, two different hydrodynamic models were applied in a phased approach.

A screening-level assessment was performed using the two-dimensional Sea, Lakes and Overland Surges from Hurricanes (SLOSH) computer model (NOAA, 2012a and NOAA, 2012b). SLOSH is computationally efficient, allowing many simulations to be performed over a relatively short period of time; however, the SLOSH model has limitations, including its relatively coarse, structured model grid and the inability to represent dynamic tides and external boundary fluxes (e.g., river flow). Therefore, in a second phase of modeling (i.e., refinement-level assessment), additional simulations were performed using the ADvanced CIRCulation (ADCIRC) model (USACE, 1994). While ADCIRC is not hindered by many of the limitations associated with SLOSH, the high-resolution, finite-element mesh and related high computational demand prevent broad applications (i.e., only a limited number of storm simulations is practicable in the context of a given analysis). Therefore, ADCIRC was applied in a targeted fashion (i.e., refinement-level assessment) to further evaluate the storms identified during the screening-level assessment as potentially causing large surges at SPS and develop the final PMSS stillwater elevations.

Inclusive of the phased modeling approach, the calculation methodology included the following steps:

1. Generation of the Initial Storm Set: An Initial Storm Set was generated using the SPS PMH results as input. Hypothetical storm tracks were first created by combining 11 potential storm bearings (i.e., -120° to -20° in 10° intervals) with five potential landfall locations between NWS 23 Mile Posts 2200 and 2300 (NOAA, 1979). The storm tracks were assigned maximum wind speeds based on the bearing-specific values derived as during the SPS PMH evaluation. Each potential storm track was then expanded into a set of storms using the bearing-specific ranges of forward speed and radius of maximum wind. Each range was divided into finite units, and a unique hypothetical (i.e., synthetic) storm was created for each combination.

2. Calculation of the Antecedent Water Level: An Antecedent Water Level (AWL) was calculated using data obtained from the Sewells Point, Virginia NOAA tidal gaging station per applicable regulatory guidelines (NRC, 2011 and ANS, 1992). In accordance with these guidelines, observed monthly maximum tide data obtained over a continuous 21-year period (i.e., January 1, 1993 through December 31, 2013) were

used to calculate the 10% exceedance high tide. Cumulative Sea Level Rise (SLR) was then added to the 10% exceedance high tide to obtain the AWL.

3. Screening-Level Assessment (SLOSH): Screening-level storm surge simulations were performed using the SLOSH model, the Initial Storm Set and the AWL. These simulations were performed to identify: 1) the sensitivity of storm surge at SPS to different storm parameters (i.e., storm track, radius of maximum winds, etc.) as constrained by the PMH calculation; and 2) the specific combinations of storm parameters and storm tracks that result in the largest predicted storm surges at SPS, also constrained by the PMH results. The screening-level simulations performed using SLOSH assumed steady-state conditions (i.e., storm parameters were not varied from the initial specifications).

4. Selection of the Refinement Storm Set: A Refinement Storm Set was selected for ADCIRC simulations after processing the results of the screening-level assessment (i.e., SLOSH model predictions). Maximum simulated stillwater elevations were recovered from the model output at the SPS intake and discharge locations. These simulated elevations were ranked and sorted, and storms within the Initial Storm Set resulting in SLOSH-simulated stillwater elevations exceeding an applied threshold of 23 feet relative to the North American Vertical Datum of 1988 or NAVD88 were identified and used to form the Refinement Storm Set.

5. Refinement-Level Assessment (ADCIRC): Refined storm surge simulations were performed using ADCIRC and a representation of dynamic tidal fluctuations for storms indicated by the screening-level assessment (i.e., Step 3, above) to be representative of the PMH. A preliminary sensitivity analysis was performed to establish storm arrival timing such that maximum stillwater elevations were achieved at SPS within the model (i.e., storm surge combined with variations in tide). Simulations were performed assuming steady conditions similar to the screening-level assessment for the purpose of comparing ADCIRC to SLOSH.

The above-described methodology produced simulated stillwater elevations at a location within the ADCIRC model domain representative of the SPS discharge and intake structures. Because dynamic tides were simulated based on astronomical/predicted tides (i.e., excluding meteorological effects and SLR), a final adjustment was made to linearly add the difference between the peak predicted tide and the calculated AWL. The results represent maximum predicted stillwater (i.e., without wind-wave action and/or river flood flow) PMSS elevations at SPS.

2.4.1.3 Probabilistic Storm Surge

A step-wise approach consistent with HHA methodology, as described by NUREG/CR-7046 (NRC, 2011), was used to probabilistically evaluate the PMSS stillwater storm surge elevations at SPS. Similar to the deterministic analysis described in the previous section, two different hydrodynamic models (i.e., SLOSH and ADCIRC) were used in a phased approach that involved applications of the Joint Probability Method (JPM) and Joint Probability Methodology with Optimal Sampling (JPM-OS).

The JPM is a statistical method developed in the 1970s that is commonly used to probabilistically evaluate the coastal surge risk due to tropical cyclones (ESSA, 1970). The JPM utilizes a set of synthetic storms representing local climatology by combining individual storm meteorological parameters (e.g., intensity, bearing, forward speed, etc.), with each storm having a joint probability of occurrence calculated from the combined probability of each of the individual parameter probabilities. A frequency response relationship may then be derived from these parameter combinations and associated probabilities via a hydrodynamic model.

The most significant limitation associated with applying the JPM is the significant computational requirement; many simulations representing a large number of unique parameter combinations are required in order to characterize the JPM integral. This limitation is exacerbated when using more robust, higher-precision numerical hydrodynamic models that require more computational effort (i.e., longer simulation times). To address this limitation and facilitate the use of these more robust models, modified forms of the JPM that rely on characterizing storm parameter space with fewer simulations through Optimal Sampling (OS) techniques have been developed (FEMA, 2012).

The Joint Probability Method – Optimal Sampling, or JPM-OS, was developed to statistically characterize surge-frequency relationships in the same manner as the traditional JPM but using fewer storm surge simulations. By limiting the number of storm surge simulations required, a robust, computationally intensive storm surge model, such as ADCIRC, can be applied. In this calculation, the basic JPM OS concept is to: 1) define complete surge-frequency relationships using a computationally efficient surge model (i.e., the NOAA SLOSH model); 2) statistically represent complete surge-frequency relationships using relatively few storms, based on the appropriate selection of storm parameters and an understanding of surge response to varying storm parameters; 3) perform storm surge simulations for the storm subset using a robust storm surge model (i.e., ADCIRC); and 4) extend the results of these simulations to define surge-frequency relationships at SPS.

In this calculation, the JPM-OS Response Surface technique developed by the United States Army Corps of Engineers (USACE) and applied by the Federal Emergency Management Agency (FEMA) was used (FEMA, 2012). In this approach, parameter space is characterized primarily via interpolation and extrapolation based on a carefully-selected set of reference parameter combinations or synthetic storms, where parameter perturbations and sensitivity testing based on this reference set are used to evaluate response functions. In concept, the JPM-OS calculation involves two steps: 1) searching for a reference storm based on the proximity of the given subject storm parameter combination to reference storm parameters; and 2) applying best-estimated surge responses along the multi-dimensional space.

The steps used to evaluate response functions and apply the JPM and JPM-OS are described as follows:

1. Creation of the JPM Storm Set: Storm parameters, including storm bearing (i.e., translational direction, fdir), forward speed (fspd), radius of maximum winds (RMW) and maximum (i.e., 1-minute average at an altitude of 10 meters) wind speed (Vm) were combined to generate hypothetical storms. Each storm track (i.e., storm bearing and landfall location) was expanded into a set of storms by considering ranges of forward speed, radius of maximum winds and maximum wind speed. Each range was discretized into units of 5 knots (kt), 5 nautical miles (nm) and 10 kt, respectively. This discretization process resulted in 8 potential values of forward speed ranging from 15 kt to 50 kt, 9 potential values of radius of maximum winds ranging from 15 nm to 55 nm, and 9 potential values of maximum wind speed ranging from 70 kt to 150 kt. A

unique hypothetical storm was created for each combination of values, storm track direction and landfall location (i.e., 8 x 9 x 9 x 11 x 5 = 35,640 hypothetical storms). Joint probabilities based on simultaneous occurrences of the Fdir, Fspd, RMW and Vm parameters were calculated for each hypothetical storm by querying the 3M data set to recover parameter co-variability. Hypothetical storms with joint probabilities of zero (i.e., no events within the 3,000,000, or 3M, data set matching or exceeding the parameter combination; a joint probability of less than 3.33E-7 or less than 1 in 3,000,000) were eliminated; the remaining 6,970 parameter combinations represented the JPM Storm Set. This method of establishing limits on parameter combinations differs from use of the 3M data set in the PMH calculation, where a dimensionless scaling relationship was derived to adjust the NWS 23 maximum wind speed in recognition of co-variability of intensity and storm bearing.

2. Addition of tidal condition: An initial condition (i.e., static starting water level) was required by SLOSH for each simulation in performing the JPM analysis. Mean High Water (MHW) and Mean Low Water (MLW) at the Sewells Point, Virginia NOAA CO-OPS station were selected as being representative of high and low tide conditions at the site, respectively. Both conditions were conservatively represented as having equal occurrence probabilities of 0.5 (i.e., equal probabilities of a hypothetical storm occurring at conditions representative of high and low tides).

3. Calculation of Annual Exceedance Probabilities: Annual Exceedance Probability (AEP) values were calculated for each hypothetical storm based on the joint probabilities calculated during Step 2. To develop surge-frequency relationships reflecting annualized probabilities, two additional factors were considered. First, the joint probabilities were multiplied by a normalized (i.e., normalized per unit length of coastline within the study area) annual storm occurrence frequency (i.e., 0.412 storms per year divided by 400 kilometers, or the diameter of the capture zone from the PMH calculation). As a final adjustment, the annualized probabilities were multiplied by a factor of 0.5 to represent coincidence with a high or low tide condition. Thus, for each simulated JPM Storm Set event, the maximum stillwater surge event was assigned an AEP value representing the storm parameter combination, the annual storm frequency and the tidal condition.

4. Performance of SLOSH simulations: A total of 13,940 simulations (i.e., 6,970 JPM Storm Set events, each at a high and a low tide condition) were performed using the SLOSH model. These simulations were performed to: 1) develop preliminary surge-frequency relationships at the SPS intake and discharge locations based on potential hurricane parameter combinations constrained by the PMH evaluation; and 2) identify the set of storms to be simulated with ADCIRC for the purpose of refining these surge-frequency relationships. Results of the SLOSH simulations were extracted in the form of maximum stillwater surge elevations at several SLOSH model cells including the cells representing the SPS intake and discharge locations.

5. Generation of an initial stillwater surge-frequency relationship: As a first step in developing initial stillwater surge-frequency relationships at SPS, histograms were created for each location using maximum stillwater surge elevations and the associated AEP values produced during Step 3. Bin sizes were defined as increments (i.e., 0.5 feet [ft]) of stillwater surge elevation based on the results of a sensitivity

analysis, and AEP values associated with the storms producing stillwater surge elevations falling into each bin were summed. The summed AEP values were then summed again from highest elevation bin to the lowest elevation bin to produce bin-specific cumulative AEP values. The stillwater surge elevations representing the center of each bin were then plotted versus their respective cumulative AEP values to generate initial stillwater surge-frequency relationships at the SPS intake and discharge locations.

6. Identification of the OS Storm Set: As previously noted, perhaps the most significant challenge associated with the JPM-OS (i.e., where OS refers to optimal sampling) technique used in this analysis is selecting the storm parameter combinations used to formulate the basis for evaluating surge response. To guide this selection process in the case of this analysis, experiments were performed using the initial surge-frequency relationships developed using SLOSH. The goal of the experiments was to identify the minimum number of storm parameter combinations required to reproduce (i.e., using JPM-OS) the surge-response relationships at SPS with reasonable accuracy. Ultimately, the experiments suggested that the very-low probability ranges of the surge-frequency relationships were well-defined by analyzing only the surges produced by storms with bearings between -70° and -30° making landfall south of latitude 37.16° (i.e., Mile Post 2300); this finding is consistent with previous findings from the deterministic storm surge evaluation, which indicate the largest simulated stillwater surge elevations at SPS are likely to be produced by storms with bearings within this sector. Furthermore, the experiments suggested that surge-frequency relationships at both locations could be accurately reproduced using 20 production runs spanning the -70° to -30° bearing range (i.e., the reference set) and 16 additional simulations to establish sensitivities to various parameter perturbations (i.e., the sensitivity set).

7. Performance of ADCIRC simulations: As a first step in refining the surge-frequency relationships at SPS, reference and sensitivity set parameter combinations identified during the previous step were simulated using ADCIRC (i.e., a total of 36 simulations). Whereas initial conditions representative of high and low tide levels were specified as input to the SLOSH simulations, a mean tide initial condition was used for the ADCIRC simulations; coincidence between surge and fluctuating tides was addressed during final adjustments (i.e., refer to Step 9). Results of the ADCIRC simulations were extracted in the form of maximum stillwater surge elevations at locations within the ADCIRC mesh representative of the SPS intake and discharge locations.

8. Refinement of stillwater surge-frequency relationships: Refined stillwater surge-frequency relationships were developed for SPS based on the results of the ADCIRC simulations using the JPM-OS technique. Maximum stillwater elevations were estimated via JPM-OS for an expanded set of storms (i.e., including two additional potential forward speeds: 5 and 10 kt) with bearings between -70° and -30° making landfall south of latitude 37.16° (i.e., Mile Post 2300) to support assessment of aleatory variability in maximum wind speeds (i.e., Step 9). AEP values were adjusted to correct for the use of a single initial condition (i.e., mean tide) prior to histogram development.

9. Adjustments to the surge-frequency relationship to reflect uncertainty, error and sea level rise: Adjustments to account for uncertainty (i.e., epistemic uncertainty and

aleatory variability) and projected sea level rise (SLR) were required in order to probabilistically characterize storm surge at SPS. The uncertainty adjustments considered the following factors: variability in tide occurring coincident with maximum storm surge; model skill associated with ADCIRC and the applied wind/vortex formulation; and aleatory variability associated with maximum wind speed specifications. A uniform adjustment for SLR was added linearly as a final step.

The above-described methodology produced an estimated stillwater surge-frequency relationships for the SPS intake and discharge locations. Stillwater elevations associated with the 1E-6 AEP level then extracted from these relationships.

2.4.2 Results

2.4.2.1 Probable Maximum Hurricane

The following sections describe the results of the evaluation of the PMH at SPS.

2.4.2.1.1 *Determination of the Controlling Storm Event*

As Table 2.4-1 indicates, both extra-tropical storms and hurricanes have resulted in significant coastal storm surges at the three stations (i.e., Sewells Point, CBBT and Kiptopeke). The data indicate that four of the top five extreme water levels at each of these three stations were caused by tropical storms or hurricanes. Figure 2.4-3 shows the historical storm tracks intersecting the area of interest, including those storms responsible for many of the recorded high water levels at the stations identified above.

Storm surge elevations predicted by NOAA (NOAA, 2012a) using the SLOSH model for hurricanes ranging from Category 1 to Category 4 were compared to the historical water level data. The purpose of this comparison was to determine whether large hurricanes are expected to result in storm surges greater than those measured in the historical record resulting from extra-tropical storms. A comparison of the Maximum of MEOW (MOM, where MEOW represents Maximum Envelope Of Water) values presented in Table 2.4-2 to the recorded water levels in Table 2.4-1 clearly indicates that historic extra tropical storms have caused storm surges roughly equivalent to those predicted for a simulated Category 1 hurricane. Table 2.4-2 also confirms that the recorded water levels resulting from historic hurricanes have caused storm surges roughly equivalent to those predicted for simulated Category 1 and 2 hurricanes. By definition, the PMH is a "hypothetical steady state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speeds that can probably occur at a specified coastal location" (NOAA, 1979). By inference, the PMH is expected to be reflective of a higher category hurricane (i.e., greater than those hurricanes represented by the relatively limited historic hurricane database). At higher hurricane category levels (i.e., Category 3 and above), the potential surge elevations predicted by NOAA significantly exceed historical water levels recorded at the CO-OPS stations.

Finally, the frequency of hurricane strikes on the U.S. East Coast was analyzed in the NWS NHC-6 using the data from 1851 to 2010. Of all hurricanes making landfall in the U.S., 18 percent struck North Carolina, 3.5 percent struck Virginia and 0.7 percent struck Maryland/Delaware. According to NHC-6, the Mid-Atlantic coastline, as represented by Virginia, North Carolina, Maryland and Delaware, was impacted by 12 major hurricanes (Category 3 or higher) between 1851 and 2010 (Blake et al 2011).

Some of these storms occurred prior to 1900, and as a result, many accounts do not include storm surge, tide values, central pressure, or other specific storm details. With respect to the storms that are well characterized by historical records, many weakened significantly or dramatically changed direction prior to reaching the SPS vicinity; therefore, only some of these storms produced significant storm surges. For storms that did impact the subject area after 1900, available track information is shown in Figure 2.4-4.

Based on: 1) a review of historical extreme water level data from the NOAA CO-OPS stations; 2) an examination of the extreme water level events associated with predicted hurricane storm surge elevations produced by NOAA using the SLOSH model at three NOAA CO-OPS stations located near SPS; and 3) a review of available historical storm information, it is concluded that a major hurricane (i.e., the PMH) will be the controlling storm resulting in the PMSS at SPS.

2.4.2.1.2 *Determination of Hurricane Parameters from NWS-23*

The location of SPS is shown in Figure 2.4-5 in relation to coastal distance intervals (i.e., mile posts) presented in NWS 23 (NOAA 1979). Storm surge at SPS is caused by the coastal storm surge in the vicinity of the mouth of the James River. As indicated on Figure 2.4-5, the mouth of the river is located between approximately NWS 23 mile posts 2200 and 2300 (more specifically, mile posts 2280 and 2300 define the confluence of Chesapeake Bay and the Atlantic Ocean), where coastal distance is measured in nautical miles from the Gulf of Mexico. Based on the location of SPS the following range of PMH parameters are as follows:

Parameter	Lower Limit	Upper Limit
Peripheral Pressure (millibar)	1020	1020
Central Pressure (millibar)	895.3	897.3
Radius of Max Winds (nautical miles)	9	26
Forward Speed (knots)	10	38
Track Direction (degrees)	73	188
1-min, 10-meter over water wind speed (miles per hour)	176.3	176.3

The methods of parameter development presented in NWS 23 are generally not consistent with the current state of knowledge for characterizing the PMH affecting the SPS vicinity. In specific reference to PMH intensity reflected by maximum wind speed, NWS 23 values are recognized as lacking a reflection of the relationship between storm direction and storm magnitude (i.e., co-variability). Thus, a detailed Site and Region Specific Hurricane Climatology study was performed to develop the hurricane meteorological parameters for use in subsequent storm surge analyses.

2.4.2.1.3 *Site and Region Specific Hurricane Climatology Study*

2.4.2.1.3.1 Statistical Analysis of Historical Data

Univariate Parameter Probability Analysis

Best track positions of tropical storms and hurricanes are maintained by the NHC in the annually-updated HURDAT2 database (NOAA, 2013). The official HURDAT2 database contains data representing cyclones occurring between 1851 and 2012. The analysis of the HURDAT2 database for the SPS study area used all applicable 1851-2012 storm data, including the 1979-2012 subset containing central pressure data.

As the focus of this calculation is on land-falling storms in the vicinity of the mouth of the Chesapeake Bay (i.e., since these storms result in large storm surges at SPS), the HURDAT2 database was filtered to extract storm parameters associated with three zones of increasing spatial coverage, as shown in Figure 2.4-6, with the Inner Region (IR) representing storms occurring in the SPS site vicinity:

- The IR – 200 kilometer radius centered at the Chesapeake Bay entrance between Cape Charles and Virginia Beach;
- The Outer Region (OR) – 500 kilometer radius encompassing the IR and extending to the southeast; and
- The Remote Region (RR) – 800 kilometer radius encompassing the IR and OR and extending to the southeast.

Probability Density Functions (i.e., PDFs) were developed for key hurricane parameters within the IR using a non-parametric kernel method. The resulting IR distributions were compared to similarly sized sample distributions developed by randomly drawing from the OR and RR. The difference between any IR sample distribution and the OR and RR distributions was evaluated to determine statistical consistency for the purpose of maximizing the sample population (i.e., determine if an expanded spatial filter could improve sample size without bias). The non-parametric estimate of the population PDF for maximum wind speed, mxw, for the IR is shown in Figure 2.4-7 as a line. The 90% confidence interval, indicated by gray shading, was estimated by randomly drawing sample sizes from the OR and RR equivalent to the IR population (i.e., N=48).

Comparisons between the IR PDFs and the corresponding OR and RR sample-based PDFs for the fdir, fspd and cpd parameters are shown in Figure 2.4-8 through Figure 2.4-10. Much like the mxw comparison, substantial differences are evident between the IR PDF and the OR and RR populations.

With central pressures routinely recorded in the HURDAT2 database only after 1979, the sample sizes for analyzing central pressure and pressure tendencies are considerably reduced relative to the full dataset. For each mid-6-hour position in the three regions, the central pressure deficit, cpd, was conservatively calculated using a peripheral pressure of 1020 mb (30.12 in Hg). The resulting PDFs for the IR are compared to the OR and RR samples in Figure 2.4-10. Figure 2.4-11 presents the PDF and Probability Density Histogram (PDH) for maximum sustained winds inside the IR.

Because the HURDAT2 data contain both tropical storm and hurricane data, all distributions peak at cpd values slightly greater than 20 mb (i.e., central pressure equal to 1000 mb). For pressure deficits greater than 20 mb, the PDFs generally indicate similarity between the IR, OR and RR distributions; however, the probabilities associated with more significant pressure deficits within the IR (i.e., more intense storms) are substantially lower than those within the OR and RR. In consideration of this result, the empirical distribution of the mxw parameter represents a preferable metric of storm intensity, as it has a much larger sample size.

In summary, these results suggest that the IR cannot be reliably expanded to increase the historical data sample size.

Hurricane Parameter Co-variability

The annual probabilities from the parameter-specific, univariate PDFs can be directly combined as a product to obtain joint probability estimates of various parameter combinations as long as the distributions can be demonstrated to be independent. If significant co-variability exists among the hurricane parameters, the probabilities of certain combinations may be different from the product of their independent probabilities.

A cross-correlation matrix of four hurricane parameters (mxw, fdir, fspd, dmxw) from data within the IR, OR and RR is shown in Figure 2.4-12. The statistical significance of each cross-correlation was determined in a manner similar to determining the significance of each parameter's distribution.

Although paired parameter correlations are quite low in general, many are statistically significant (i.e., highlighted in yellow). Scatter plots of the paired parameters in the off-diagonal elements of the cross-correlation matrix shown in Figure 2.4-12 are presented in Figure 2.4-13 through Figure 2.4-18 along with a least-squares estimate of each respective regression line. The figures are presented in order from least viable linear co-variability to most-viable co-variability, as assessed by visually inspecting the scatter of the parameter plotted on the ordinate axis along the range of the parameter plotted on the abscissa axis.

Figure 2.4-13 shows a complex co-variability of forward storm speed (fspd) and forward direction (fdir). The fastest moving storms are those moving toward the north and east (i.e., bearings between 0° and 90°); whereas, storms with strong eastward or westward motions typically move at slower speeds. This characteristic is consistent among the three sampled regions. Similarly, Figure 2.4-14 and Figure 2.4-15 indicate a linear relationship exists between the storm intensity, as measured by mxw, and forward storm direction (fdir) and forward speed (fspd), respectively. Storms moving with strong eastward and westward motions are less intense compared to storms moving with northward or northeastward components, as indicated by Figure 2.4-14. The less-intense storms also move more slowly than the stronger storms, as indicated by Figure 2.4-15.

Figure 2.4-16, through Figure 2.4-18 show scatter plots of storm intensity change, as indicated by dmxw, versus forward direction (fdir), storm intensity (mxw) and forward speed (fspd), respectively. As indicated by Figure 2.4-16, the probability distributions of intensity changes are nonlinearly related to storm forward direction, at least in terms of the width of the distributions. Storms moving in north to northeastward directions exhibit broader intensity change distributions compared to the more westward and eastward moving storms. Within the IR, the majority of storms are weakening when moving west of north.

Figure 2.4-17 indicates that distributions of intensity changes as functions of intensity bulge in width for moderately-strong storms and exhibit narrower spreads at lower and higher intensities. Figure 2.4-18 shows little variation in the breadth of the intensity change distribution with forward speed, and the small slope associated with the regression line suggests that this paired parameter set exhibits nearly independent co-variability.

The nonlinearity exhibited by the variation of the spread in one parameter in relation to the value of the second parameter is evident within all sampled regions. This finding argues against characterizing the co-variability by linear means. Also, in reference to Figure 2.4-13, which shows the relationship between fdir and fspd, treating these parameters as independent would produce a high probability that fast moving storms approach the IR with westward bearings; a condition that is not consistent with the analysis of the HURDAT2 data set.

In summary, the statistical analysis of historical data suggests that the IR cannot be reliably expanded to increase the historical data sample size. Furthermore, the data within the IR represent a small sample size for determining the joint probability of storm parameters, especially considering that many of the storms within the IR have made landfall south of the region and are passing inland prior to approaching the study area. Fortunately, validation and analysis of the synthetic WRT storm set, as presented below, indicates that parameter distributions are similar to those developed from historical data. Thus, the WRT storm set represents an acceptable basis for characterizing PMH parameters in the vicinity of SPS.

2.4.2.1.3.2 Statistical Analysis and Verification of Synthetic Hurricane Data

The synthetic storm set contains over 10,000 storms (i.e., tropical storms and hurricanes) characterized by various angles, translational speeds, intensities, and maximum wind radii. The storm parameters are available at 2-hr intervals and represent 10,013 storms pre-screened to impact the vicinity of the Chesapeake Bay opening (i.e., the source of surge impacts to SPS). The pre-screening (i.e., limiting the synthetic storm set to storms with tracks that approach SPS within 200 km of the Chesapeake Bay opening) has an effect on the probability density distribution of some parameters; a fact that is considered in the validation of the data set.

Figure 2.4-19 through Figure 2.4-22 present the validation results for the hurricane parameters fdir, fspd, dmxw and mxw, respectively. Each figure contains three panels, showing distribution comparisons for the IR (top), OR (middle), and RR (lower) domains, respectively. The gray shaded region with the central gray line within each panel provides estimates of the WRT population distribution made from HURDAT2-sized sampling (i.e., sampling from the WRT storm set); whereas, the ten superimposed lines show PDFs calculated for the 10 HURDAT2 samples.

As noted through inspection of the figures referenced above, the results of the validation indicate that use of the WRT representation of the empirical storm data for estimating independent and joint variability of hurricane parameters will contain a conservative bias. While storm intensity is generally well-represented by the WRT data for major storms, the WRT's bias toward faster forward speeds and more westerly storms is expected, given the sensitivity to storm surge within the vicinity of the Chesapeake Bay opening, to conservatively predict more frequent and larger storm surges near SPS.

2.4.2.1.3.3 PMH Parameter Calculations

Following validation, the WRT storm set was used to statistically characterize storm parameters within the storm surge production region. The analysis was focused on data located primarily within the over-water area of the IR, referred to as the Offshore IR (OIR), as spatially filtering the data to include only over-water points will produce a conservative set of storm parameters to support storm surge modeling. Figure 2.4-23 shows the IR, the OIR and the reduced number of storms (i.e., 6,874) and 2-hr storm segments (i.e., 24,185) resulting from spatial filtering to the dimensions of the OIR versus the IR.

With a reduction in the number of storms relative to the IR, the annual frequency of storms within the OIR is also reduced, as are the numbers of hurricanes of various intensities. Table 2.4-3 lists the numbers of storms and 2-hr storm segments for all storms and for the three major storm categories. Figure 2.4-24 shows the annual frequency of synthetic storms within the IR and OIR by year and as a 31-year average (i.e., the 1980 to 2010 period corresponding to the synthetic storm simulations).

Univariate PDFs of relevant hurricane parameters from the WRT storm set filtered to the storm surge production region are presented in Figure 2.4-25 through Figure 2.4-28. The vertical lines represent central points for each interval. The tabulated probability densities apply to all data values within the stated intervals centered on the central point values. The middle rows of the tabulated probabilities represent interval-integrated values, and the bottom rows of values represent the middle row values multiplied by the adjusted annual frequency of occurrence for the WRT storm set (i.e., approximately 0.412). This adjusted annual frequency of occurrence represents the average year-by-year frequency of occurrence of all synthetic storms (i.e., intersecting the OIR) during the period between 1980 and 2010 within the storm surge production region.

Using an extension of the WRT data set based on sampling from each univariate PDF developed for synthetic data within the OIR (i.e., the 3M data set), parameters and parameter ranges for the PMH at SPS were developed in recognition of parameter co-variability. PMH parameters were determined by identifying a dimensionless scaling function that recovered variability of the NWS 23 PMH maximum wind speed, as described below. While the process of identifying this scaling function involved probability calculations for parameter combinations (i.e., storm bearing and maximum wind speed), the resulting parameter combinations represent conservative deterministic PMH upper limits, as the NWS 23 PMH maximum wind speed is used as the basis of function development.

In developing the dimensionless scaling function, Annual Exceedance Probability (AEP) values were first assigned to maximum intensities for 10° storm bearing increments spanning the potential PMSS-causing sector (i.e., 10° sectors centered at -120°, -110°, -100°,...-20°) by querying the 3M data set. Criteria based on upper and lower limits were assigned to reflect a considered bearing sector (e.g., greater than or equal to -65° and less than -55° for the 10° bearing sector centered at -60°). Then, the number of events within the 3M data set meeting these criteria (i.e., all parameters falling within the pre-defined bounds) was counted. Finally, the resulting count for each considered bearing sector was divided by the size of the data set (i.e., 3,000,000) to produce the joint probability associated with the parameter combination; the reciprocal of this probability multiplied by the OIR's annual storm frequency (i.e., 0.412) was calculated to reflect the return period (i.e., reciprocal of the AEP) for each parameter combination.

Finally, bearing-specific PMH intensities were calculated using the following steps:

1. Storm intensity variation with bearing was evaluated over the potential PMSS-causing sector (i.e., -120° to -20°) and the sector calculated from NWS 23 (i.e., -107° to 8°) by arbitrarily specifying data set ranks.

2. As indicated by Figure 2.4-29, the 10° bearing interval associated with the maximum intensity was identified as -10° (i.e., storms from the 3M data set with bearings greater than or equal to -15° and less than -5°).

3. Using the NWS 23 PMH maximum wind speed (i.e., 153.2 kt), the data set rank associated with the 3M data set intensity most closely matching this value for the 10° bearing interval was identified (i.e., the 31^{st} highest intensity for this bearing sector, 153.3 kt). This rank recognizes NWS 23 as being a conservative representation of PMH intensity within the -107° to 8° bearing sector.

4. Using the PMH intensity and bearing rank, intensities were extracted from the 3M data set for the 10° bearing intervals spanning the potential PMSS-causing bearing sector (i.e. -120° to -20°).

A least-squares regression line (i.e., second-order polynomial anchored to 153.2 kt at the 10° bearing interval) was then applied, and bearing-specific PMH intensities (i.e., **vm** as a function of **fdir**) were extracted from the regression function (Figure 2.4-29). The considered bearing range includes storms with bearings between -120° and -20° to provide a bounding parameter set (i.e., relative to the anticipated PMH) inclusive of more intense, northerly-bound storms.

Ranges of the rmw and fspd parameters were developed on a bearing-specific basis using the 3M data set and the results of the intensity analysis presented above (Figure 2.4-30). In the case of rmw, upper and lower parameter bounds were assigned based on the relationship to intensity, which shows a trend toward smaller radius and tighter range as intensity increases. For fspd, upper and lower parameter bounds were assigned based on relationships to intensity and bearing, which indicate a general trend toward faster forward speed as bearing shifts from -120° to -20° and intensity increases.

Recommended PMH-level parameters and parameter ranges are presented in Table 2.4-4 over a range of bearings (+/- 5°, centered on 10° increments) to capture the storm that causes the PMSS on a bearing-specific basis. The parameter ranges for fspd and rmw reflect bounding conditions relative to the likely controlling (i.e., maximum surge-producing) event within each storm bearing range. The maximum wind speeds represent conservative PMH intensities in consideration of applicable uncertainty and error, as supported by the following:

- Conservative intensity bias of WRT storm set versus HURDAT2 data;
- Assessment of potential uncertainty associated with the WRT storm set; and
- Assessment of error introduced by non-parametric, kernel-based fit.

Thus, the revised parameters and parameter ranges presented in this calculation represent a conservative assessment of the PMH and provide input to the deterministic PMSS evaluation at SPS. Similarly, the PDFs developed from the WRT data within the OIR represent conservative reflections of parameter occurrence likelihoods and provide input to the probabilistic storm surge evaluation at SPS.

2.4.2.2 Deterministic Storm Surge

The following sections describe the results of the deterministic evaluation of the PMSS at SPS.

2.4.2.2.1 *Generation of the Initial Storm Set*

Fifty-five storm tracks were created based on eleven potential bearings and five potential landfall locations (Figure 2.4-31) to span the potential surge generation region in the SPS vicinity. Each track was then assigned a maximum intensity based on the bearing-specific maximum wind speed determined as part of the PMH analysis. Finally, the tracks were assigned to unique pairings of the remaining, discretized parameters (i.e., radius of maximum winds and forward speed) to create the Initial Storm Set of 1,620 hypothetical events, each with a unique storm identification (STORMID) number.

Whereas a single maximum wind speed was determined for each bearing, the radius of maximum winds and forward speed parameters were presented as ranges (i.e., with upper and lower bounds varying by bearing). Thus, in generating the Initial Storm Set, these ranges were finely discretized by multiples of 5 nm and 5 kt, respectively. The upper and lower bounds of the ranges presented in the PMH calculation do not necessarily correspond to multiples of 5 nm or 5 kt; therefore, in generating the Initial Storm Set, the ranges were initially rounded to the nearest multiple (i.e., to span each range)

2.4.2.2.2 *Calculation of the Antecedent Water Level*

In accordance with NUREG/CR-7046 (NRC, 2011), the PMSS is required to be evaluated coincidentally with an AWL equal to the 10% exceedance high tide plus long term changes in sea level. The 10% exceedance high tide is defined as the high tide level that is equaled or exceeded by 10% of the maximum monthly tides over a continuous 21 year period. In accordance with ANSI/ANS-2.8-1992 (ANS, 1992), this tide can be determined from recorded tide data or from predicted astronomical tide tables.

In this calculation, the 10% exceedance high tide was calculated using recorded monthly maximum tide elevations from the Sewells Point tidal gaging station. Using this approach, a value of 3.524 ft NAVD88 was obtained. In consideration of Sea Level Rise (SLR), which was projected over 50 years using the annual rate at the Sewells Point station, the AWL was determined to be 4.3 ft NAVD88.

2.4.2.2.3 *Screening-Level Assessment (SLOSH)*

As part of performing the screening-level assessment performed using the NOAA SLOSH model, results from the 1,620 simulations, in the form of simulated surge elevation time series for each simulation, were extracted at four locations within the hor3 basin, including the model cells representing the SPS intake and discharge locations (Figure 2.4-32 and Figure 2.4-33). The time series were reduced to peak surge elevations at these locations for each simulated storm. The storms responsible for the largest simulated surges at SPS were identified, as discussed in the following section.

Figure 2.4-34 and Figure 2.4-35 summarize the results at the SPS discharge in the form of three-dimensional surfaces depicting maximum stillwater elevations as functions of storm bearing and forward speed or radius of maximum winds, respectively. Figure 2.4-34 suggests significant sensitivity to storm bearing and forward speed, and Figure 2.4-35 indicates a strong positive correlation between surge and radius of maximum winds but generally consistent behavior across the storm bearing range.

Based on these responses, storm surge at SPS appears to be maximized by large-radius, slow-moving, northwesterly storms capable of moving significant coastal surge into the lower Chesapeake Bay region. Due to the location of the maximum winds within the simulated, idealized cyclones, simulated storm surge is also maximized by storms making landfall to the south of the Chesapeake Bay opening. These slow-moving storms are able to maintain momentum and route surge from the interior of the bay up through the lower stretch of the James River to SPS (i.e. due, in part, to the storm bearings, which generally align the storm tracks with the bay opening and the lower stretch of the river). While large coastal surges can also be generated by faster-moving storms, the results of the screening-level assessment suggest that momentum behind the surge is lost as surge is routed from the coast to the river (i.e., faster-moving storms are able to overtake a significant portion of the surge volume, thus sacrificing momentum responsible for carrying the surge up through the river). Furthermore, while storms with more northerly bearings have been associated with higher storm intensities for the SPS region, these storms are not efficient with respect to routing surge from coast to SPS.

2.4.2.2.4 *Selection of the Refinement Storm Set*

The results of the screening-level assessment were used to target specific storm parameter combinations resulting in the largest stillwater surge elevations at SPS for refinement using the ADCIRC model (i.e., refinement-level assessment, which was performed using the Refinement Storm Set). The parameter combinations associated with the Refinement Storm Set events are summarized in Table 2.4-5. As Table 2.4-5 indicates, the 15 storms making up the Refinement Storm Set represent 5 different potential storm bearings, 3 potential landfall locations, 3 potential forward speeds and 3 potential radii of maximum winds.

2.4.2.2.5 *Refinement-Level Assessment (ADCIRC)*

Prior to performing dedicated refinement simulations, a sensitivity analysis was performed to determine the appropriate storm arrival time relative to the tide phase at Newport and SPS that would produce the most conservative results (i.e., the highest stillwater elevations at SPS). Time of landfall was used as an indicator for storm arrival time in this analysis.

Five simulations were performed, each with a different time of landfall relative to high tide at Sewells Point and SPS. The results indicated that when a storm made landfall one hour earlier (13:00) than the time for peak tide at Sewells Point (14:00), maximum water levels were obtained at both SPS and Sewells Point. Therefore, the storm tide elevation was considered to be maximized when a storm made landfall one hour prior to the peak tide at Sewells Point.

Guided by the results of the tidal phasing sensitivity analysis, ADCIRC simulations were performed using input defining the 15 storms within the Refinement Storm Set. Based on these simulations, the following combination of storm parameters was identified as being responsible for the deterministic PMSS at SPS:

STORMID = 1097

- Track Direction (Θ) = -60°;
- Landfall Mile Post = 2225 (Latitude 35.913°, Longitude -75.596°);

- Radius of Maximum Winds (Rmax) = 35 nm;
- Forward Speed (Vf) = 15 kt
- Maximum 1-min, 10-m Overwater Wind Speed (Vm) = 119.9 kt; and
- Central Pressure Deficit (CPD) = 98 mb.

The ADCIRC simulation representing this combination of parameters (i.e., STORMID 1097) resulted in maximum stillwater elevations of 21.3 and 20.9 ft NAVD88 (i.e., reflecting linear adjustment to the AWL) at the SPS intake and discharge locations, respectively. These deterministically-derived maximum stillwater elevations represents conservative results in consideration of the following:

- Conservatisms associated with the deterministic PMH inputs, as previously discussed;
- Consideration of the sensitivity to tidal phasing and coincidence between peak simulated tide and maximum storm surge;
- The range of simulated Holland B values; and
- The conservative value of the AWL applied in this analysis.

2.4.2.3 Probabilistic Storm Surge

The following sections describe the results of the probabilistic evaluation of the PMSS at SPS.

2.4.2.3.1 *Creation of the JPM Storm Set*

As a first step in generating the JPM Storm Set, potential storm bearings were assessed to identify combinations likely to contribute to the low-probability range of the surge-frequency relationships at SPS. As low-probability surge responses (i.e., relatively high maximum stillwater surge elevations) were anticipated for storms traveling west-of-north based on storm surge sensitivities observed as part of the deterministic evaluation, the range of considered storm bearings was limited to -120° to -20°.

Storm bearings between -120° and -20° (i.e., in 10° intervals) were combined with landfall locations to create a set of 55 storm tracks (Figure 2.4-36). Each potential storm track was expanded into a set of storms by considering ranges of forward speed, radius of maximum winds and maximum wind speed. This discretization process resulted in 8 potential values of forward speed ranging from 15 kt to 50 kt, 9 potential values of radius of maximum winds ranging from 15 nm to 55 nm, and 9 potential values of maximum wind speed ranging from 70 kt to 150 kt. A unique synthetic storm, each identified with a unique STORMID number, was created for each combination of parameters including storm track direction and landfall location (i.e., 8 x 9 x 9 x 11 x 5 = 35,640 synthetic storms).

Joint probabilities were calculated for each synthetic storm in a manner that recovered parameter co-variability, as reflected within the 3M data set. Based on these calculations, 6,970 parameter combinations were identified as having non-zero joint probabilities (i.e., at least one record within the 3M data set falling within the upper and lower parameter bounds on each parameter). These parameter combinations were isolated to create the JPM Storm Set.

2.4.2.3.2 *Addition of Tidal Condition*

Given the computational efficiency of the NOAA SLOSH model (i.e., as compared to the more robust but computationally cumbersome ADCIRC model), two bounding tidal conditions could be practically simulated. Mean High Water (MHW) and Mean Low Water (MLW) at the Sewells Point NOAA CO-OPS station were selected as being representative of bounding tidal conditions at SPS.

Each storm in the JPM Storm Set was split into two conditions: one version of the storm occurring coincidentally with the high tide (i.e., MHW) condition, and another version of the same storm occurring coincidentally with the low tide (i.e., MLW) condition. This process doubled the size of the JPM Storm Set (i.e., 6,970 to 13,940 unique parameter combinations.

2.4.2.3.3 *Calculation of Annual Exceedance Probabilities*

In order to convert joint probabilities to annual exceedance probabilities (AEPs), two additional factors were considered: the probability associated with the simulated tidal condition and the omni-directional annual storm occurrence rate. Each joint probability was first multiplied by 0.5 to represent the probability of occurrence for the associated simulation's tidal condition (i.e., high tide or low tide). Each modified value was then multiplied by an omni-directional annual storm occurrence rate, which considered annual storm frequency (i.e., determined from the analysis of the WRT storm set), the approximate length of evaluated coastline and storm track spacing.

2.4.2.3.4 *SLOSH Simulations*

A total of 13,940 SLOSH simulations were performed using input representing the reduced JPM Storm Set. In accordance with applicable guidance (e.g., NRC, 2013), storms were simulated as steady-state events (i.e., input parameters, including storm bearing, were not varied from the initial values prior to landfall). Time series extracted for each storm in the JPM Storm Set were reduced to peak simulated surge elevations at several locations within the hor3 basin, including the locations representative of the SPS intake and discharge. These maximum simulated stillwater elevations were used to develop preliminary stillwater surge-frequency relationships at SPS, as described below.

2.4.2.3.5 *Generation of an Initial Stillwater Surge-Frequency Relationship*

Using the SLOSH model results and the calculated AEP values, preliminary stillwater surge frequency relationships were calculated for the SPS intake and discharge locations using the standard JPM (i.e., non-OS). The calculations aligned with FEMA methodology for surge frequency determination (FEMA, 2012). This step established the basis for reducing the JPM Storm Set to the OS Storm Set (i.e., required for implementation of JPM-OS with ADCIRC), as described below.

2.4.2.3.6 *Identification of the OS Storm Set*

As indicated by Figure 2.4-37, experiments performed using the SLOSH model suggested that the very-low probability ranges of the surge-frequency relationships were well-defined (i.e., lower than an AEP value of approximately 1E-6) by an OS Storm Set comprised of model-simulated surges produced by storms with bearings between -70° and -30° making landfall south of latitude 37.26°. This finding is consistent with previous findings indicating the largest simulated stillwater surge elevations at SPS are likely to be produced by storms generally following these west-of-north tracks. Furthermore, the experiments suggested that stillwater surge-frequency relationships at both locations represented by the remaining parameter combinations could be accurately reproduced using 20 production runs spanning the -70° to -30° bearing range (i.e., the reference set) and 16 additional simulations to establish sensitivities (i.e., calculate derivative terms) to various parameter perturbations (i.e., the sensitivity set).

2.4.2.3.7 *Performance of ADCIRC Simulations*

Results of the ADCIRC simulations (i.e., OS Storm Set) were evaluated at several locations within the model mesh, including the nodes representative of the SPS intake and discharge locations. ADCIRC simulations were performed using a static initial condition (i.e., 0 ft NAVD88) approximately representative of a mean tide condition at SPS; whereas, SLOSH simulations for the entire JPM Storm Set were performed using two static initial conditions representative of high and low tide conditions. This difference was addressed during final adjustments to account for uncertainty. Simulated maximum stillwater elevations at the SPS intake and discharge locations for the OS Storm Set – reference set are shown in Table 2.4-6; the corresponding OS Storm Set – sensitivity set results are shown in Table 2.4-7.

For the storms simulated using ADCIRC, the ADCIRC wind field profiles generally compared favorably to the SLOSH predictions. Where slight differences were evident (e.g., large distances from the storm centers), the comparisons indicated conservatism in the ADCIRC representation (i.e., ADCIRC is predicting higher wind speeds compared to SLOSH). These favorable comparisons support the utility of applying SLOSH as screening-level assessment tool.

2.4.2.3.8 *Refinement of Stillwater Surge-Frequency Relationships*

Based on the ADCIRC results described above, refined stillwater surge-frequency relationships were developed for SPS. AEP values were revised to exclude the tidal adjustment factor (i.e., 0.5) in recognition of the representation of a mean tide condition versus high and low tide conditions. In addition to deriving the stillwater surge-frequency relationship from ADCIRC results as opposed to SLOSH results, two other methodological modifications were made at this stage:

1. Potential forward speeds of 5 and 10 kt were considered by extrapolating the fspd surge response at 15 kt. This was done to fully span the low range of the forward speed parameter given the relatively high independent probability of a slow-moving storm and the previously-identified inverse correlation between forward speed and peak surge for some storm bearings at SPS.

2. Stillwater surge elevations were estimated for every storm parameter combination without consideration of non-zero joint probability (i.e., expanded to include forward speeds of 5 and 10 kt). These elevation estimates were necessary in order to characterize aleatory variability, as discussed in the following section.

The surge-frequency relationships at SPS were finalized by assessing uncertainty and linearly adding projected SLR, as described in the following section

2.4.2.3.9 *Adjustments to the Surge-Frequency Relationship to Reflect Uncertainty, Error and Sea Level Rise*

Two forms of uncertainty were considered in this analysis: epistemic uncertainty and aleatory variability. The former form of uncertainty generally represents a "lack of understanding" of the physics within the system (i.e., measurement uncertainty, model skill, etc.); whereas, the latter form of uncertainty is attributed to sample size limitations associated with empirical data and/or the existence of unresolved or unpredictable variations in system behavior (NRC, 2012). Error associated with characterizing maximum wind speeds was considered to be negligible based on a comparison of the EVA fits to the WRT and 3M data sets performed during the PMH evaluation.

The sources of significant uncertainty considered in this analysis included (note that the first two potential sources are examples of epistemic uncertainty, the third potential source is an example of aleatory variability):

1. uncertainty in representing tide occurring coincidentally with surge:

   The effect of this source of uncertainty was quantified based on datum analysis results at the Sewells Point NOAA CO-OPS station. This uncertainty accounts for tide variation from the simulated initial condition (i.e., 0 ft NAVD88). This difference was calculated as the attenuated difference between the Mean High Water (MHW) elevation and the simulated initial condition (i.e., NAVD88 datum).

2. bias or uncertainty in numerical surge and wind field models (i.e., ADCIRC):

   The effect of this source of uncertainty was quantified based on the results of ADCIRC verifications performed by GZA. To estimate the 95% confidence interval, the maximum absolute error calculated based on observed and simulated peak water levels was conservatively multiplied by a factor of 2.

3. uncertainty due to sampling (i.e., aleatory variability associated with maximum wind speed):

   This source of uncertainty is variable as a function of maximum wind speed. The effect was quantified based on random sampling (i.e., "bootstrapping") performed using maximum wind speed values from the 3M dataset.

Consistent with FEMA methodology (FEMA, 2012), this analysis considered only uncertainty which resulted in higher or more-probable surge results (i.e., added conservatism); therefore, all uncertainty terms were considered to be positive such that they increased calculated surge elevations.

The final uncertainty-adjusted stillwater relationships at the SPS intake and discharge locations are shown in Figure 2.4-38. In addition to the uncertainty adjustments described above, these relationships also reflect additional linear adjustments to account for the 50-year SLR projection at SPS. The same relationships, converted to the MSL vertical datum, are shown in Figure 2.4-39.

Similar to the deterministic PMSS evaluation, the probabilistic storm surge results described