2.7. Ice-Induced Flooding

This section evaluates the potential of ice effects to contribute to flooding at SPS.

2.7.1. Method

The criteria for ice-induced flooding are provided in NUREG/CR-7046, Appendix G (NRC, 2011). Two ice-induced events that may lead to flooding at the site are discussed in NUREG/CR-7046, Appendix G including:

1. Ice jams or dams that form upstream of a site that collapse, causing a flood wave; and
2. Ice jams or dams that form downstream of a site that result in backwater flooding.

With respect to ice effects at SPS, the HHA used the following steps:

1. Review historical ice events and information on backwater effects due to ice jams in the James River near SPS.
2. Evaluate historical water temperatures and minimum daily air temperatures to assess the feasibility of the formation of ice jams in the James River near SPS.
3. Calculate flood elevations which could result at SPS from potential ice jams upstream or downstream in the James River near SPS.

2.7.2. Results

2.7.2.1. Review of Historical Ice Events

The USACE maintains records of historical ice jams and dams on the Ice Jam Database (USACE, 2012), which can be queried (using state/city/river name) to obtain information regarding historical ice events. Four records of historical ice jams along the James River exist in the USACE database (Table 2.7-1). Three of these events occurred in Buchanen and Lickrun, VA, which are both about 200 miles upstream of SPS (USGS 2012). The fourth and nearest ice jam to SPS occurred in Richmond, which is about 50 miles upstream of SPS. SPS is located on the southern bank of the James River approximately 45 river miles upstream of the Atlantic Ocean. The James River close to SPS is tidal and brackish (CBP, 2013). Therefore, the occurrence of ice jams near SPS is less likely, when compared to Richmond.

2.7.2.2. Evaluate water and air temperature near SPS

Water Temperature

Water temperature data for the James River at selected USGS stream gages were used to derive monthly minimum water temperatures (Figure 2.7-1). December, January and February are consistently the months with lowest observed water temperatures at the selected gages. It is noted that the monthly minimum temperature of 0 ºC measured at Buchanan in December and January 1981 appeared to agree with the timing of the ice jam event in the USACE's Database at this location (USACE, 2012). However, the average monthly minimums are above

0ºC at these selected gages (USGS, 2013). Table 2.7-2 summarizes the monthly minimum water temperatures.

Cartersville (at USGS stream gage no. 02035000) is the nearest location to SPS with water temperature data. Cartersville is approximately 90 river miles (northwest) upstream of SPS. SPS is located on the southern bank of the James River approximately 45 river miles upstream of the Atlantic Ocean.

The occurrence of ice jams near SPS is less likely, when compared to Cartersville due in part to the salinity of the water near SPS. Mean salinity of surface water in James River near SPS ranges from 0.6 to 2.5 parts per thousand (CBP, 2013). The salinity concentration gradually increases to around 20 parts per thousand near the mouth of the James RIver. The salinity in the water reduces the freezing point to be lower than 32 °F (0 °C) (NOAA, 2013). For example, the freezing point is 30 °F when the salinity is 17 ppt (NOAA, 2013). This is likely one of the reasons that there has been no ice jam event recorded downstream of Richmond.

Air Temperature

The normal daily minimum air temperature, lowest temperature of record, the mean number of days with temperatures at or below freezing and normal daily means for Richmond and Norfolk as reported by NCDC (NCDC, 2012) are summarized in Table 2.7-3. Richmond is approximately 45 miles northwest of SPS and Norfolk is located near the river mouth of James River, approximately 35 miles southeast of SPS. The mean number of days annually with minimum temperatures at or below 32 °F for Richmond is 83 days. The mean number of days with a minimum temperature at or below 32 °F indicates the frequency of occurrence of days with freezing temperatures. The same comparative data indicates that normal daily minimum temperatures in Norfolk are above 32 °F most of the time. The lowest temperature of record is -3 °F in January. The mean number of days annually with minimum temperatures at or below 32 °F is 50 days for Norfolk.

Because SPS is located geographically midway between Richmond and Norfolk over a relatively open flood plain, the minimum temperatures and mean number of days annually with sub-freezing temperatures at SPS are expected to be in the range defined by the data collected from Richmond and Norfolk.

2.7.2.3. Calculate possible flood elevation

Flood elevation due to backwater effects caused by ice jams downstream or ice dam failure upstream of SPS was estimated based on the information provided by the 1936 event in Richmond (USACE 2012). The news article reads:

"*Richmond, Va. – Battered by an 18-foot wall of ice and water from an icejam break in the James River here near midnight, ...*"

This evaluation used a conservative approach which translates the height of the ice jam above normal water level as a backwater flood at a downstream bridge or constriction or a hypothetical ice dam flood wave from an upstream source. It is conservative in that it does not consider the attenuation of the hypothetical upstream ice dam failure flood wave as it travels the 50 miles between Richmond and SPS. The larger width of the waterway near SPS would also make the height of ice jam lower than 18 feet, if an ice jam event with the same total volume of ice occurs near or downstream of Surry.

In the event that the historically largest 18-foot of ice dam were to occur near SPS, on top of a normal water level in the James River (mean tidal range of 1.0 ft at SPS, FEMA 2009), the freeboard of the site from being flooded was calculated as follows:

Maximum Water Level = Elevation 1.0 ft (MSL) + 18 ft = Elevation 19 ft (MSL)

The mean tide elevation was selected as the reference water surface elevation because under high tide conditions the James River flows east to west and could convey ice upstream (limiting propagation of ice jam flood effects). Mean tide elevation was conservatively selected over mean low tide elevations. See Section 3.7 for more details regarding freeboard at SPS.

2.7.3. Conclusions

Although temperature data indicates infrequent but possible temperatures below freezing, the historic record indicates ice jams are infrequent in the James River and have not been recorded in the vicinity of SPS. The site is not expected to be flooded due to the occurrence of ice jams, in the vicinity, upstream, or downstream of the site, based on the results of a conservative analysis of ice jam potential.

2.7.4. References

2.7.4-1 **CBP 2013.** Chesapeake Bay Mean Surface Salinity, Chesapeake Bay Program – A Watershed Partnership (PDF file downloaded on 1-17-2013).

2.7.4-2 **FEMA 2009.** Flood Insurance Study, Surry County, Virginia and Incorporated Areas, Federal Emergency Management Agency, Flood Insurance Study Number 51181CV000A, April 2009.

2.7.4-3 **NCDC 2012.** Comparative Climate Data for the United States through 2011, National Climatic Data Center (NCDC) (http://www1.ncdc.noaa.gov/pub/data/ccd-data/CCD 2011.pdf, file downloaded on 11-29-2012).

2.7.4-4 **NOAA 2013.** JetStream – Online School for Weather, National Climatic and Oceanic Administration (http://www.srh.noaa.gov/jetstream/ocean/seawater.htm – web page printed on 1-19-2013).

2.7.4-5 **NRC 2011**. Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046, United States Nuclear Regulatory Commission, November 2011.

2.7.4-6 **USACE 2012.** Ice Jam Database, U.S. Army Corps of Engineers, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, 2012 (https://rsgisias.crrel.usace.army.mil/icejam – data retrieved on 1-23-2013).

2.7.4-7 **USGS 2012.** National Hydrography Dataset, U.S. Geological Survey (http://nhd.usgs.gov/ - data downloaded on 10-31-2012).

2.7.4-8 **USGS 2013.** National Water Information System – USGS Water Data for the Nation, U. S. Geological Survey (http://waterdata.usgs.gov/nwis – data downloaded on 1-16 2013).

Table 2.7-1: Summary of Historical Ice Jams in the James River, Virginia

Event No.	Database Record ID	Gage ID	Location	Date	Jam Type
1	517	02037500	Richmond, VA	2/11/1936	Break-up
2	20090126121110	02016500	Lick Run, VA	1/6/1981	Unknown
3	20090126135213	02019500	Buchanan, VA	1/11/1981	Freeze-up
4	20090504084001	02019500	Buchanan, VA	1/12/1981	Unknown

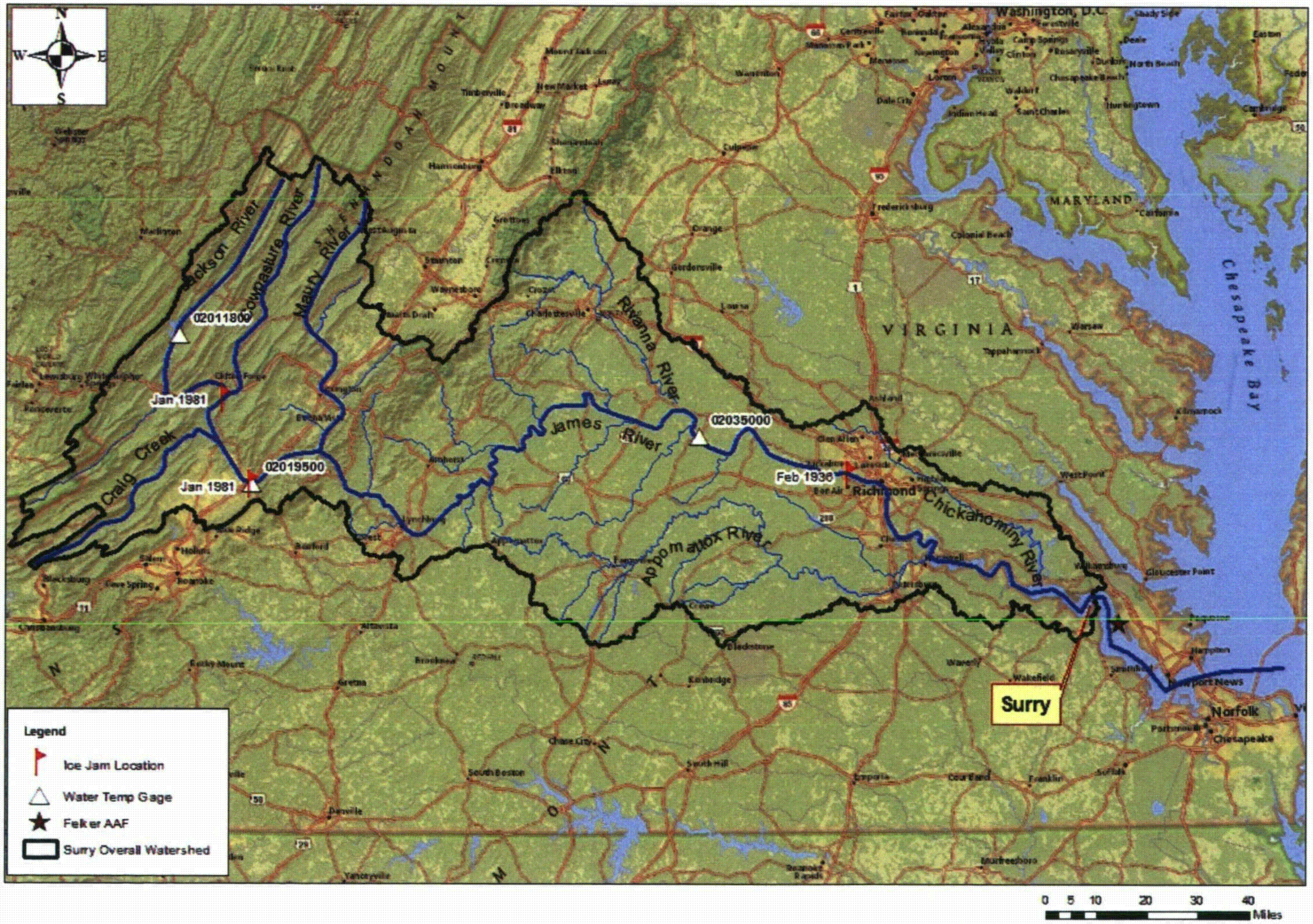
Table 2.7-2: Summary of Monthly Minimum Water Temperatures (degrees Celsius) in the James River, Virginia

		Jan		Feb		Dec	
USGS Gage No.	Location	Lowest Monthly Minimum	Average Monthly Minimum	Lowest Monthly Minimum	Average Monthly Minimum	Lowest Monthly Minimum	Average Monthly Minimum
02011800	Gathright Dam, VA	1	4.3	0.2	3.8	1.3	6.3
02019500	Buchanan, VA	0	1.1	0	1.9	0	1.2
02035000	Cartersville, VA	0	1.7	0	2.5	0	2.4

Table 2.7-3: Comparative Climate Data Summary for Richmond and Norfolk, Virginia through 2011

	Richmond				Norfolk			
Month	Normal Daily Minimum Temp. (°F)	Lowest Temp. of Record (°F)	Mean No. of Days with Temp. 32 °F or Lower	Normal Daily Mean Temp. (°F)	Normal Daily Minimum Temp. (°F)	Lowest Temp. of Record (°F)	Mean No. of Days with Temp. 32 °F or Lower	Normal Daily Mean Temp. (°F)
Jan	27.6	-12.0	21.0	36.4	32.3	-3.0	15.0	40.1
Feb	29.7	-10.0	18.0	39.5	33.6	8.0	13.0	42.0
Mar	37.0	10.0	10.0	47.7	40.1	18.0	5.0	49.0
Apr	45.3	23.0	1.0	57.1	47.8	28.0	6.0	57.4
May	54.6	31.0	2.0	65.4	57.6	36.0	0.0	66.3
Jun	63.3	40.0	0.0	73.5	66.2	45.0	0.0	74.5
Jul	68.3	51.0	0.0	77.9	71.4	54.0	0.0	79.1
Aug	66.8	46.0	0.0	76.3	70.1	49.0	0.0	77.4
Sep	59.9	35.0	0.0	69.8	64.8	45.0	0.0	72.1
Oct	47.2	21.0	2.0	58.3	52.8	27.0	0.0	61.1
Nov	38.4	10.0	10.0	49.0	43.7	20.0	3.0	52.3
Dec	31.1	-1.0	20.0	40.4	36.1	7.0	13.0	44.2
No. of Years	30	82	82	30	30	63	63	30

Figure 2.7-1: Climate Station Locations and SPS Site Location



2.8. Channel Migration or Diversion

This section of the report evaluates the potential for natural channels to meander or otherwise change alignment in a manner that could flood or otherwise affect Structures, Systems, and Components (SSCs) important to safety at SPS. NUREG/CR-7046 (NRC, 2011) includes the following statement in Section 3.8-Flooding Resulting from Channel Migration or Diversion:

> *Natural channels may migrate or divert either away from or toward the site. The relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site.*

2.8.1. Method

The channel migration and diversion flooding evaluation followed the HHA approach described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC, 2011). With respect to channel migration and diversion at SPS, the HHA used the following two steps:

1. Review historical records and hydrogeomorphological data to assess whether the James River has exhibited the tendency to meander towards the site.
2. Evaluate present-day channel maintenance measures in place to mitigate channel migration of the James River.

2.8.2. Results

Historic Channel Diversion

A study on the evolution of the near shore zone of Surry County since 1937 was performed by the Virginia Institute of Marine Science (VIMS, 2011). The study involved the analysis of aerial photographs of the shore zone of Surry County from 1937, 1954, 1963, 1978, 1994, 2002, 2007, and 2009. Results of the study indicate minimal changes in the James River coastline at the site. The average rate of change at the James River coastline segment west of the site range from 0 to -0.4 feet per year and -1.1 feet per year at the shoreline east of the site. The change in the shoreline was attributed to wave action along the coast.

A literature review did not yield evidence suggesting there have been significant historical diversions of the James River near SPS over the last century. A comparison of a 1965 and 1999 USGS Topographic maps (USGS, 2013) illustrates continuity of the river course over the last 34 years, see Figure 2.8-1.

Site Geology

The following information is summarized from the SPS UFSAR (Dominion, 2014) : The site area is generally devoid of any structural features indicative of folding or faulting. Surface inspection and subsurface investigations at the site show no evidence of structural deformation. Borings indicate no offsets or folding of strata. There is no surface or subsurface evidence of prior landslides, cratering, or fissures that may be indicative of prior intense earthquake effects. The upper 65 feet of soil at the site consists of a series of alternating strata of clay and sands of Pleistocene age which lie on Miocene clays that have in their upper portion a series of thin sand lenses. These thin Miocene sand lenses were found intermittently between about Elevations -55 and -62, and were individually only a few inches to a foot or so in thickness. The site grade is approximately 26.5 feet.

Channel Maintenance

The U.S. Army Corps of Engineers, Norfolk District performs maintenance dredging of the James River federal navigation project, as necessary to maintain the authorized project dimensions, thereby assuring safe and economical use of the James River by shipping interests. The navigational channel near SPS is 300 feet wide and 25 feet deep (USACE, 2013).

2.8.3. Conclusions

A review of historical data indicates that the James River has not exhibited a tendency to meander towards the site in a manner that could flood or otherwise affect SSCs at the site. The James River is a maintained, navigable waterway near SPS. Much of the critical shoreline at SPS is composed of soils that are strong and stable, with moderate to high shearing strengths. Given these conditions, channel migration is not considered to be a potential contributor to flooding at SPS.

2.8.4. References

2.8.4-1 NRC 2011. NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. Nuclear Regulatory Commission, November 2011.

2.8.4-2 USACE 2013. The U.S. Army Corps of Engineers, Norfolk District, James River Navigational Project http://www.nao.usace.army.mil/Default.aspx.

2.8.4-3 Dominion, 2014. Surry Power Station Updated Final Safety Analysis Report (SPS UFSAR), Revision 46.02.

2.8.4-4 USGS 2013. USGS Historical Topographic Maps. Downloaded on January 20, 2013 from http://nationalmap.gov/historical/.

2.8.4-5 VIMS 2011. Shoreline Evolution: Surry County, Virginia. James River Shorelines, Virginia Institute of Marine Science , College of William & Mary, Gloucester Point, Virginia, September 2011.

Figure 2.8-1: Historic Topographic Maps (USGS, 2013)

Imagery taken in 1963

Imagery taken in 1999

2.9 Combined Effects Flooding

An evaluation of the combined external flood effects associated with riverine and coastal flooding at SPS was performed. River flood processes include flooding by precipitation events and flooding due to dam failures. Coastal processes include coastal storm surge, wind set-up within the river fetch, tide including sea level rise and wind-generated waves.

2.9.1 Method

The HHA approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of the combined external flood effects on the James River at SPS. Deterministic combined effect flooding was evaluated first, followed by a refined probabilistic combined effect flooding which is judged to represent the most accurate estimate of flooding potential at SPS.

Due to its coastal and riverine setting, combined effect flooding at SPS requires evaluation of the following alternatives according to guidance presented in NUREG/CR-7046:

- Flooding caused by precipitation events
- Flooding caused by seismic dam failures
- Floods along the shores of open and semi-enclosed bodies of water
- Floods caused by tsunamis

As part of the HHA approach, the deterministic storm surge was also evaluated probabilistically, where floods along the shores of open bodies of water were evaluated using the probabilistically-determined storm surge corresponding to the annual exceedance probability (AEP) of 1E-6.

<u>2.9.1.1 Deterministic Combined Effect Flood</u>

There are two combined effect mechanisms presented in NUREG/CR-7046 that are relevant to riverine flooding and three combined flood effect mechanisms relevant to coastal flooding evaluated in this section. Combined effect flooding is evaluated at the west side of the peninsula (near and/or within the Discharge Canal), the east side of the peninsula (at the Circulating Water Intake Structure and the Intake Canal earthen embankment), and within the Intake Canal, as needed for each alternative.

<u>H.1 – Floods Caused by Precipitation Events</u>

The following criteria for floods caused by precipitation events (NUREG/CR-7046, Appendix H, Section H.1) were evaluated.

- <u>Alternative 1</u> - A combination of mean monthly base flow, median soil moisture, antecedent or subsequent rain, the PMP, and waves induced by 2-year wind speed applied along the critical direction;
- <u>Alternative 2</u> - A combination of mean monthly base flow, probable maximum snowpack, a 100-year snow-season rainfall, and waves induced by 2-year wind speed applied along the critical direction; and
- <u>Alternative 3</u> - A combination of mean monthly base flow, a 100-year snowpack, snow-season PMP, and waves induced by 2-year wind speed applied along the critical direction.

The results of the PMF analysis (Section 2.2; Figure 2.9-1) indicate that Alternative 1 is the controlling precipitation event scenario. Therefore, Alternatives 2 and 3 are not further considered.

The James River at SPS would be susceptible to the formation of wind-generated waves. The H.1 combined effect flood evaluation for SPS used the following steps:

1. Calculate the straight line fetch in the critical direction;
2. Calculate sustained wind speed:

   Calculate the 2-year return period wind speed using the fastest 2-minute wind speed data from National Climatic Data Center (NCDC) Station Global Historical Climatology Network-Daily (GHCND) USW00093741: Newport News International Airport, VA gage (NCDC, 2014), by applying the Gumbel Distribution to the observed data;
3. Calculate wave height and period using empirical equations governing wave growth with fetch (USACE, 2008);
4. Calculate wave runup near the SPS Discharge Canal, Intake Structure, and intake embankment using the Technical Advisory Committee for Water Retaining Structures (TAW) method (van der Meer, 2002; also referenced in USACE, 2006).

H.2 – Floods Caused by Seismic Dam Failures

The following criteria for floods caused by seismic dam failures (NUREG/CR-7046, Appendix H, Section H.2) were evaluated:

- Alternative 1 – This alternative consists of a combination of: (a) a 25-year flood; (b) a flood caused by dam failure resulting from a safe shutdown earthquake (SSE) and coincident with the peak of the 25-year flood; and (c) waves induced by 2-year wind speed applied along the critical direction;
- Alternative 2 – This alternative involves: (a) a combination of the lesser of one-half of Probable Maximum Flood (PMF) or the 500-year flood; (b) a flood caused by dam failure resulting from an operating basis earthquake (OBE), and coincident with the peak of one-half of PMF or the 500-year flood; and (c) waves induced by 2-year wind speed applied along the critical direction.

Flood elevations resulting from seismic dam failures (NUREG/CR-7046, Appendix H.2) are bounded by those resulting from the PMF with coincident hydrologic dam failure on the James River at SPS (Section 2.3). Therefore, further calculations to address NUREG/CR-7046 Appendix H.2 are not necessary. The elevation resulting from the PMF with coincident hydrologic dam failure also bound the H.1 alternatives (floods caused by precipitation events). Wave effects were therefore calculated based on this bounding alternative.

H.3 - Floods along the Shores of Open and Semi-Enclosed Bodies of Water

The following criteria for floods along the shore of open or semi-enclosed bodies of water (NUREG/CR-7046, Appendix H, Section H.3) were evaluated:

- Alternative 1 – The combination of the lesser of one-half of the PMF or the 500-year flood, the surge and seiche from the worst regional hurricane or windstorm with wind-wave activity, and the antecedent 10 percent exceedance high tide.

  The one-half PMF was selected because it is conservative (i.e., the one-half PMF is greater than the 500-year flood). The one-half PMF is calculated by multiplying the PMF hydrograph by a constant ratio of 0.5. The worst regional hurricane in the vicinity of SPS (i.e., Chesapeake Bay area), on the basis of the maximum historic, documented storm surge was a result of Hurricane Isabel. The hurricane reached Category 5 status on the Saffir-Simpson Hurricane Scale. It made landfall near Drum Inlet on the Outer Banks of North Carolina as a Category 2 hurricane on September 18th, 2003. Waves generated by Hurricane Isabel were calculated by the coupled ADCIRC+SWAN computer model.

- Alternative 2 – The combination of the PMF, the 25-year storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide.

  The 25-year storm surge was calculated based on a statistical analysis performed using the NOAA tide gage at Sewells Point, Virginia (NOAA Station 8638610; NOAA, 2014), which is located in Norfolk, VA, approximately 26 river miles southeast of SPS (Figure 2.9-2). The maximum annual water level from each year was obtained for the 35-year period of record. A frequency analysis was performed by applying the log-Pearson Type III distribution as per U.S. Geologic Survey (USGS) Bulletin 17b (USDOI, 1982) to the data. Wind wave activity was calculated similarly as Alternative H.1.

- Alternative 3 – The combination of the 25-year flood in the stream, the probable maximum storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide.

  - Observed USGS annual peak stream flow from USGS Gage No. 02037500 on James River, near Richmond, VA (located on the main stem of the James River, Figure 2.9-3) was used as the input data series for the flow frequency analysis. Data is obtained for the entire period of record for the gage (USGS, 2014). This is the closest James River USGS gage to SPS that records stream flow. A statistical analysis to calculate the 25-year flood flow was performed. A frequency analysis was performed by applying the log-Pearson Type III distribution (USDOI, 1982) to the data.

Storm parameter combinations resulting in maximum stillwater elevations at the SPS discharge and intake locations, as described in the Deterministic Probable Maximum Storm Surge Calculation, were used as input to ADCIRC+SWAN. Three parameter combinations (i.e., STORMIDs 948, 1097 and 1098) that resulted in stillwater elevations within 0.5 feet of the maxima at the SPS intake and discharge locations were considered. During the Deterministic PMSS, the maximum stillwater elevation does begin to encroach upon the western portion of SPS (see Figure 2.9-4). However, due to the direction of waves throughout the PMSS, most wave action will be headed away from the site (i.e. predominantly in the south west and north east directions), as shown in the time series of wave direction for the Deterministic PMSS near the discharge (see Figure 2.9-5, see Figure 2.9-17 for output discharge node). Wave direction is measure in Cartesian convention, measured counterclockwise from the positive x-axis indicating the direction waves are going toward. While there is a small portion of inundation encroaching upon the site, due to heavy vegetation and the groins present at the area of the Discharge Canal, it is extremely unlikely for waves that would effect SPS to form within this area. Therefore, wave runup effects are considered negligible on the western portion of the site and are not included as part of this calculation. Figure 2.9-6 shows where wave transects were placed for the evaluation of wind wave effects during the Deterministic PMSS. Wave characteristics were extracted directly from the ADCIRC+SWAN model at the same nodes used to extract surge results.

- Alternative 4 – For drainage areas less than 300 square miles in hurricane-prone areas, a combination of the PMF, the PMH in the open or semi-enclosed water body, and the antecedent 10 percent exceedance high tide.

Alternative 4 does not apply because the drainage area of the James River at SPS is 9,521 square miles. Also, seiche was demonstrated to be inconsequential to flooding potential at SPS (Section 2.5).

The antecedent 10 percent exceedance high tide elevation includes the calculated sea level anomaly and the expected SLR in accordance with ANSI/ANS 2.8 (ANS, 1992).

H.4 - Floods along the Shores of Enclosed Bodies of Water

The criteria for floods along the shore of enclosed bodies of water (NUREG/CR-7046, Appendix H, Section H.4) do not apply to SPS since the site is not located on an enclosed body of water.

H.5 - Floods Caused by Tsunamis

Evaluation of the potential for tsunamis at the SPS site concluded that tsunamis are not a significant flood-causing mechanism (Section 2.6). Therefore, no further analysis of tsunami-induced flooding combined with other mechanisms has been performed.

2.9.1.2 Combined Effect Flood with Probabilistic Storm Surge

In addition to applying the combined flood effects presented above to the deterministic flood analyses, the combined flood effects were also evaluated for the probabilistic storm surge corresponding to the AEP of 1E-6 (refer to Section 2.4). The combined effects for the probabilistic analyses were assumed to be consistent with NUREG/CR-7046 (NRC, 2011) and ANSI/ANS 2.8 (ANS, 1992), including:

- Storm surge corresponding to the to the AEP of 1E-6;
- 25-year flood in the James River; and
- Coincident wind-wave activity.

The stillwater elevation corresponding to the 1E-6 AEP was combined deterministically with the 25-year river flood flow. Two parameter combinations resulted in elevations approximately consistent with the 1E-6 AEP levels at the SPS intake and discharge locations; therefore, two ADCIRC+SWAN simulations were performed. The initial conditions and approach to representing coincidental tidal conditions used for probabilistically calculating the surge-frequency relationships at the SPS intake and discharge locations were described in Section 2.4.

The resulting uncertainty-adjusted maximum WSELs from the combination of the 25-year flood and the storm surge resulting from STORMID PR_1 were calculated to be 19.9 feet, MSL and 20.3 feet, MSL at the SPS intake and discharge locations, respectively. Figure 2.9-17 presents the ADCIRC+SWAN mesh and Intake and Discharge nodes used to extract hydrograph results. The resulting maximum WSELs from the combination of the 25-year flood and the storm surge resulting from STORMID PR_2 were calculated to be 20.8 feet, MSL and 20.8 feet, MSL at the SPS Intake and Discharge locations, respectively.

The higher overall stillwater elevation of 20.8 feet, MSL was used in the determination of maximum water level resulting from the Probabilistic Combined Effect Flood. The resulting stage hydrograph (i.e., without waves) is shown in Figure 2.9-15. Estimated wind speed and duration based on ADCIRC+SWAN results is shown on Figure 2.9-16.

During the probabilistic storm surge, the maximum stillwater elevation encroaches upon the western portion of SPS (see Figure 2.9-7). However, due to the direction of waves throughout the storm surge, most wave action will be headed away from the site, as shown in the time series of wave direction for the probabilistic storm surge near the discharge (see Figure 2.9-8, see Figure 2.9-17 for discharge output node location). While there is a small portion of inundation encroaching upon the site, due to heavy vegetation and the groins present at the Discharge Canal, it is extremely unlikely for waves to form within this area. Therefore, wave runup effects are considered negligible on the western portion of the site and are not included as part of this calculation.

Wind wave effects were calculated at both the Intake Canal earthen embankment and the Circulating Water Intake Structure, see Figure 2.9-6.

2.9.1.3 Hydrostatic Force and Hydrodynamic Loading and Debris

Resulting flood depths were used to develop hydrostatic force and hydrodynamic loads. The flood depths used for the calculation of hydrodynamic, hydrostatic and impact loads should include increases in depth that may occur as a result of erosion and scour.

<u>Hydrostatic Loads</u>

Hydrostatic loads are those caused by water above or below the ground surface, free or confined which is either stagnant or moves at velocities less than 5 feet per second (fps) (ASCE, 2010). These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. The hydrostatic lateral forces (per linear foot of surface) were calculated using ASCE guidance.

<u>Flow Velocity</u>

Floodwater flow velocities include velocity components due to the river flow and wind-generated waves. Estimating design flood velocities in coastal flood hazard areas is subject to considerable uncertainty. Flood velocities were estimated conservatively by assuming floodwaters can approach from the most critical direction relative to the site and by assuming that flow velocities can be high (FEMA, 2011). The upper bound flood velocity was used to calculate hydrodynamic and impact loads.

<u>Hydrodynamic Loads</u>

Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides and suction at the downstream side. Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads calculated use steady-state flow velocities (FEMA, 2011; FEMA, 2012).

The hydrodynamic force calculated above is then divided by the width of the structure it is acted upon to get the pounds per foot acting on the structure.

The drag coefficient is a function of the shape of the object around which flow is directed. When an object is something other than a round, square or rectangular pile, the coefficient was determined by one of the following ratios (FEMA, 2012):

1. The ratio of the width of the object (w) to the height or the object (h) if the object is completely submerged.
2. The ratio of the width of the object (w) to the stillwater depth of the water (d_s) if the object is not fully submerged.

Note that the hydrodynamic loads applied above are for rigid structures. Dividing the horizontal drag force by the building width yields a force per length (pounds per linear foot).

Debris Impact Loads

Debris impact loads are imposed on a building (or structure) by objects carried by moving water. The loads are influenced by where the building is located in the potential debris stream, specifically if it is:

- immediately adjacent to or downstream from another building;
- downstream from large floatable objects; or
- among closely spaced buildings.

General ASCE guidance is used in the absence of site-specific information for debris weight or availability. Per ASCE 7-10 (ASCE, 2010), a debris object weight of 1,000 pounds is a reasonable average for flood-borne debris (representing trees, logs and other large woody debris). This weight corresponds to a log approximately 30 feet long and about 1 foot in diameter. This weight also represents a reasonable weight for other types of debris ranging from small ice floes to large sediment to man-made objects. Also per ASCE 7-10 (ASCE, 2010), in riverine floodplains, large woody debris (trees and logs) predominates, with weights typically ranging from 1,000 pounds to 2,000 pounds.

Wave Loads

Loads due to broken waves are similar to hydrodynamic loads from flowing or surging water. The forces from breaking waves are the largest and most severe; therefore this load condition was used as the design wave load (FEMA, 2011). The three breaking wave load conditions (FEMA, 2011) include: a) waves breaking against submerged small diameter piles; b) waves breaking against submerged walls; and c) wave slam, where the top of the wave strikes against a vertical wall. The term "wave slam" refers to the action of wave crest striking the elevated portion of a structure (FEMA, 2011). Wave slam is only calculated for elevated structures.

This calculation applies to the condition where the space behind the wall is dry (e.g., the interior of a building). The loads are applied as shown in Figure 2.9-9 (FEMA, 2011).

2.9.2 Results

2.9.2.1 Combined Effect Flood: Deterministic

2.9.2.1.1 Flooding Caused by Precipitation Events

Determine Wind Wave Transects

The greatest straight line fetch for head-on, fully incident waves during the PMF with dam failures at the SPS intake was determined using the maximum inundation from the PMF stillwater elevation and the 7.5 minute USGS topographic map (ESRI, 2014). Wind wave transects were placed at both the Circulating Water Intake Structure and intake canal embankment on the east side of the embankment to determine the effects of waves on both the structure itself, as well as the canal embankment. As shown in Figure 2.9-1, flooding is limited to the James River shoreline on the western portion of the site, with minor inundation near the Discharge Canal. The greatest straight line fetch within the Intake Canal was determined along the longitudinal axis using the maximum controlled water level of 30 feet MSL (SPS, 2012d). Each transect is shown in Figure 2.9-10, where the Discharge Canal transect length is equal to 0.33 miles, the Intake Canal transect length is equal to 0.45 miles, and the Intake Structure transect length is equal to 3.9 miles respectively.

Transect profiles at the Low Level Intake Structure and Intake Canal earthen embankment, and intake canal slope were developed to calculate wind wave effects in these locations during combined effect flooding. A plan view of these profiles is shown in Figure 2.9-6. Profiles at the Low Level Intake Structure and Intake Canal earthen emebankment were developed using plant drawings (SPS, 2012a; SPS, 2012b) and the NED 1/9 arc second elevation data (USGS, 2013). Transect profiles are presented in Figure 2.9-11 and Figure 2.9-12.

Calculate Sustained Wind Speed

The best data with the longest duration that is available from the National Climatic Data Center (NCDC) is the 2-minute duration wind speed. Recorded wind speed information was obtained from the nearby Newport News International Airport, VA gage (GHCND USW00093741). The fastest 10-meter altitude, 2-minute duration wind speed was selected because it has the most complete available data in the vicinity of SPS (Figure 2.9-2). The 2-year return period wind speed was determined using the Gumbel Distribution (Maidment, 1993).

The 2-year return period, 2-minute duration wind speed was calculated using the Gumbel Distribution to be 42.0 miles per hour (mph).

Development of Wave Height and Period

Wave heights and periods during the PMF were calculated using methodology from the Coastal Engineering Manual (CEM) for wave growth with fetch (USACE, 2008). Fetch limited conditions are conservative for calculating wave runup effects at SPS. Wave characteristics near the SPS Low Level Intake Structure were calculated to be a significant wave height of 2.7 feet with a peak period of 2.8 seconds, and in the discharge canal a wave height of 0.8 feet with a peak period of 1.2 seconds. Within the Intake Canal, wave characteristics were determined to be a significant wave height of 0.9 feet and 1.4 seconds. Within the Discharge Canal, wave characteristics include a significant wave height of 0.8 feet with a peak period of 1.2 seconds. Table 2.9-1 provides a summary of wave characteristics.

Development of Wave Runup

NRC guidance Sections 5.1 and 5.3 of JLD-ISG-2012-06 (NRC, 2013) state that runup should be calculated using USACE CEM methodology. Wave runup along sloping barriers was computed using the recommended approach from the FEMA Atlantic Coast Guidelines (FEMA, 2007) based on the surf similarity parameter (ξ) and the reduction factors listed below. Referred to as the Technical Advisory Committee for Water Retaining Structures (TAW) method, this computational method uses reduction factors for influence of berms, structure porosity, surface roughness, and influence of wave angle. The TAW method also utilizes an average slope method, useful for conditions where a berm is present. The average slope definition ignores the influence of a berm (van der Meer, 2002). For the characteristics at SPS, reduction factors for berms and wave angle of attack were utilized. The angle of incidence of wave attack, β, is defined as the angle between the direction of propagation of the waves and the perpendicular to the structure. The wave field during storm conditions can be considered short crested (van der Meer, 2002).Wave setup is inherently included in the runup heights because some wave setup influence is present in laboratory tests that led to the development of the empirical equations of the TAW method (van der Meer, 2002). This method is also included in the USACE CEM guidance for calculating wave runup on embankments (USACE, 2006). The formulas used within this method specifically use the significant wave height as the statistical wave height for runup calculations in lieu of the maximum wave height. The calculated runup using the significant wave input is the 2% runup, which inherently includes consideration to other statistical wave heights. Therefore, the significant wave height is considered appropriate for use in calculating runup at SPS.

Runup within the discharge canal was based on the PMF + dam failures still water elevation of 15.7 feet MSL, and calculated to have a maximum runup elevation of 17.6 feet MSL. Runup within the discharge canal will not overtop the channel embankments. Runup due to waves at the intake embankment will result in a maximum runup elevation of 16.7 feet, MSL at the Intake Canal earthen embankment and 32.6 feet, MSL within the Intake Canal at the high level intake structure. Waves will not overtop the intake embankment of 36 feet MSL. Runup heights and wave crest elevations are presented in Table 2.9-2.

Vertical walls cause a reflected or standing wave against the seaward (i.e., river-facing) side of the wall (ASCE, 2010). The reflected standing wave height on the Low Level Intake Structure was calculated. ASCE guidance states that this standing wave crest reaches a height above stillwater of 1.2 times the depth at the wall (ASCE, 2010). This relationship assumes depth-limited conditions (ASCE, 2010). A conservative approach was taken as the depth-limited wave, which is the maximum wave height possible for a given depth. As previously stated, the use of depth-limited wave height (i.e., maximum breaker height) is also consistent with NRC guidance JLD-ISG-2012-06 (NRC, 2013). Maximum wave crest elevations at the Low Level Intake Structure are 20.1 feet MSL, and were calculated using ASCE 7-10 methodology for standing wave crest heights on a vertical structure (ASCE, 2010). Wave crest elevations at the Low Level Intake Structure are considered conservative as these are the maximum depth limited waves that could physically occur on the deck of the structure itself.

2.9.2.1.2 Floods Along the Shores of Open Bodies of Water

Floods along the Shores of Open Bodies of Water - Alternative 1

The maximum flow rate for the one-half PMF in the James River at SPS was calculated at 430,500 cfs. The tide corresponding to the static antecedent 10 percent high tide and the wind field for extra-tropical storm Isabel (worst regional hurricane) were simulated together in ADCIRC.

The resulting maximum water surface elevation from the combination of the One-Half PMF, the antecedent 10 percent high tide and storm surge resulting from Hurricane Isabel was calculated to be 11.9 feet NAVD88 (13.3 feet MSL) and 12.4 feet NAVD88 (13.8 feet MSL) at the Intake and Discharge Canals at SPS, respectively, with consideration of applicable SLR and potential bias or uncertainty in the applied numerical hydrodynamic model.

The calculated stillwater elevations at SPS from this alternative are well below site grade (Elevation 26.5 feet MSL; Dominion, 2014) and are bounded by the calculated stillwater elevations from Alternative 3. Therefore, coincident wave effects were not further evaluated for this alternative.

Floods along the Shores of Open Bodies of Water - Alternative 2

The PMF peak flow rate at SPS was calculated as 867,300 cfs. The 25-year surge height was calculated to be 5.2 feet. The 10-percent exceedance high tide used for this alternative was 3.5 feet NAVD88 (4.9 feet MSL) at SPS.

HEC-RAS model results from the combination of the PMF with the 25-year surge and the antecedent 10 percent high tide result in a maximum stillwater surface elevation of 14.1 feet NAVD88 (15.5 feet MSL) at SPS.

The calculated stillwater elevations at SPS from this alternative is bounded by the calculated stillwater elevations from Alternative 3. Coincident wave effects were calculated for the bounding alternative and not for this alternative.

Floods along the Shores of Open Bodies of Water - Alternative 3

The 25-year flood flow was calculated as 267,300 cfs. The deterministic PMSS is the maximum result from the following events:

a) STORMID 1097 - a slow-moving (i.e., 15 knots), intense (i.e., maximum wind speed of 119.9 knots) hurricane bearing in a west-of-north direction (i.e., -60° bearing) and making landfall along the Outer Banks of North Carolina.
b) STORMID 948 - a slow-moving (i.e., 15 knots) intense (i.e., maximum wind speed of 115.1 knots) hurricane bearing in a west-of-north direction (i.e., -70° bearing) and making landfall near the Virginia/North Carolina border.
c) STORMID 1098 - a slow-moving (i.e., 15 knots) intense (i.e., maximum wind speed of 119.9 knots) hurricane bearing in a west-of-north direction (i.e., i.e., -60° bearing) and making landfall near the Virginia/North Carolina border.

The 25-year flood and the storm parameter combinations described above were simulated together using ADCIRC+SWAN.

For the SPS intake and discharge locations, consideration of epistemic uncertainty/ADCIRC+SWAN model skill resulted in increases to the maximum ADCIRC+SWAN predicted water levels of 0.8 feet (i.e., representative of bias or uncertainty in the ADCIRC+SWAN).

The resulting maximum SWELs from the combination of the 25-year flood and the storm surge resulted from STORMID 948 and were calculated to be 24.2 feet MSL and 24.1 feet MSL at the SPS intake and discharge locations, respectively. The resulting stage hydrograph (i.e., representative of storm-induced surge and wave setup) for Alternative 3 is shown in Figure 2.9-13. Simulated wind speed and duration based on ADCIRC+SWAN results is shown on Figure 2.9-14.

Table 2.9-3 provides the wave characteristics from the ADCIRC+SWAN model used to develop the coincident wave runup.

Runup occurring as a result of the deterministic PMSS was determined using the TAW method at the intake embankments. Because the Intake Structure will be flooded under the stillwater elevation, coincident wave activity will be against a vertical structure, as shown in Figure 2.9-11. Standing wave crest heights due to depth limited waves were calculated using guidance from ASCE 7-10 (ASCE, 2010).

Wave crest elevations and wave runup under the PMSS scenario are presented in Table 2.9-4. Runup along the intake canal embankment would potentially reach 43.6 feet MSL but it actually would overtop the embankment elevation of 36 feet MSL. Wave overtopping occurs when wave runup exceeds the freeboard of a particular structure. Overtopping is generally defined as a mean discharge, i.e., Volume per unit of time (cubic feet per second – cfs). FEMA defines overtopping into various categories depending on volume as spray, splash, runup wedge, and wave transmission (FEMA, 2007). Overtopping of the intake canal embankment was calculated using FEMA methodology from the Atlantic Coast Guidelines (FEMA, 2007). Overtopping was calculated to be approximately 1400 cfs. Erosion of the grassed Intake Canal embankment due to wave action is likely. Overtopping depth of the intake canal embankment was calculated using the weir coefficient (Brater, 1976). The flow depth over the top of the embankment is approximately 0.9 feet.

Wave crest elevations of 38.8 feet MSL at the Low Level Intake Structure overtop the rooftop housing the Emergency Service Water Pump Room. While significant wave heights calculated by SWAN near the intake reach 11.2 feet, these wave heights will break at the deck of the Low Level Intake Structure, as the depth limited wave on the deck is approximately 10 feet. Therefore, the results of reflected standing wave crest heights due to depth limited waves are conservative at the Low Level Intake Structure.

2.9.2.2 Combined Effect Flood: Probabilistic Storm Surge

The 1E-6 AEP surge is the maximum result from the following events:

a) STORMID PR_1 - a very slow-moving (i.e., 5 knots), intense (i.e., maximum wind speed of 120 knots) hurricane bearing in a west-of-north direction (i.e., -50° bearing) and making landfall near the Virginia/North Carolina border.

b) STORMID PR_2 - a very slow-moving (i.e., 10 knots) intense (i.e., maximum wind speed of 130 knots) hurricane bearing in a west-of-north direction (i.e., -50° bearing) and making landfall near the Virginia/North Carolina border.

The 25-year flood and the storm parameter combinations described above were simulated together using ADCIRC+SWAN.

The resulting maximum water surface elevations at the SPS intake and discharge locations were then adjusted to include measures of uncertainty and SLR (i.e., additional feet of surge) applicable at the 1E-6 AEP level based on the assessment of uncertainty presented in Section 2.4. For the SPS intake and discharge locations, consideration of uncertainty and SLR resulted in increases to the maximum ADCIRC+SWAN predicted water levels of 2.95 feet and 2.85 feet, respectively.

The higher overall stillwater elevation of 20.8 feet MSL resulting from STORMID PR_2 was used in the determination of maximum water level resulting from the Probabilistic Combined Effect Flood. The resulting stage hydrograph (i.e., without waves) is shown in Figure 2.9-15. Estimated wind speed and duration based on ADCIRC+SWAN results is shown on Figure 2.9-16.

Table 2.9-5 provides the wave characteristics extracted from the ADCIRC+SWAN model used to develop the coincident wave runup. Runup occurring as a result of the probabilistic storm surge was determined using the TAW method at the intake embankments. Because the Intake Structure will be flooded from the stillwater elevation, coincident wave activity will be against a vertical structure, as shown in Figure 2.9-11. For this condition, standing wave crest heights due to depth limited waves were calculated using guidance from ASCE 7-10 (ASCE, 2010).

Wave crest elevations and wave runup results for the PMSS scenario are present in Table 2.9-6. Runup along the intake embankment of 36.3 feet MSL overtops the embankment elevation of 36 feet MSL. An overtopping rate of approximately 106 cfs was calculated along the length of the intake canal exposed to wave action during the probabilistic PMSS. Erosion of the grassed Intake Canal embankment due to wave effects along the earthen embankment is likely. Overtopping depth of the intake canal embankment was calculated using the weir coefficient (Brater, 1976). The flow depth over the top of the embankment is approximately 0.2 feet.

Reflected wave crest elevations of 31.4 feet MSL do not overtop the roof elevation of 33.5 feet MSL at the Low Level Intake Structure (SPS, 2012a). While significant wave heights calculated by SWAN near the intake reach 11.2 feet, these wave heights will break at the deck of the Low Level Intake Structure, as the depth limited wave on the deck is approximately 7.3 feet. Therefore, the results of reflected wave crest heights due to depth limited waves are conservative at the Low Level Intake Structure.

2.9.2.3 Hydrostatic Force and Hydrodynamic Loading and Debris

Typical hydrostatic and hydrodynamic forces were calculated at the SPS intake under three scenarios:

(A) the controlling deterministic combined flood effects due to precipitation;
(B) the controlling deterministic combined flood effects along the shores of open bodies of water (Alternative 3); and
(C) the probabilistic combined flood effects due to storm surges.

The deck elevation of the Emergency Service Water Pump and Oil Storage Room is 10.56 feet NAVD88 (SPS, 2012a) was used to compute the appropriate depth of flood water.

Hydrostatic Loads

The following summarizes the calculated hydrostatic forces for each scenario:

Scenario	Hydrostatic Force (lb/ft)	Acting at Elevation (feet NAVD88)
A	427	11.8
B	4644	14.6
C	2422	13.5

Hydrostatic loads are based on generalized extreme approximations (extremely conservative) and are subject to individual assessment during the integrated assessment.

Upper Bound Flow Velocity

Flow velocities were calculated as:

Scenario	Flow Velocity (feet/second)
A	10.9
B	19.8
C	16.8

Hydrodynamic Loads

The hydrodynamic loading analysis (calculated at the Emergency Service Water Pump and Oil Storage Room) resulted in the following loads for each scenario:

Scenario	Hydrodynamic Load (lb/ft)	Acting at Elevation (feet NAVD88)
A	554	12.4
B	5798	16.7
C	3015	15.0

Hydrodynamic loads are based on generalized extreme approximations (extremely conservative) and are subject to individual assessment during the integrated assessment.

Debris Impact Loads

Typical debris impact loads on exterior portions of structures (for debris weight of 2,000 lbs) were calculated as the following for each scenario. Due to the relatively shallow flooding at the Low Level Intake Structure and the location of the structure on the fringe of the floodplain, most large debris would remain in the main channel of the James River:

Scenario	Debris Impact Load (lbs)
A	13,080
B	31,680
C	26,880

Debris impact loads are based on generalized extreme approximations (extremely conservative) and are subject to individual assessment during the integrated assessment.

Wave Loads

Loads due to non-breaking waves were calculated as the hydrostatic and hydrodynamic loads described above. The typical breaking wave load on vertical walls at the SPS intake were calculated as:

Scenario	Wave Loads (lb/ft)
A	5,339
B	58,048
C	30,270

Wave loads are based on generalized extreme approximations (extremely conservative) and are subject to individual assessment during the integrated assessment.

2.9.3 Conclusions

A summary of combined event scenario maximum water elevations are presented in Table 2.9-7.

- Floods caused by precipitation events (H.1): A maximum stillwater elevation at SPS of 15.7 feet, MSL was calculated. A standing wave crest elevation of 20.1 feet, MSL at the Low Level Intake Structure is predicted. Impacts due to wave effects at SPS are not expected.
- Floods along the shores of open bodies of water (H.3): Both deterministic and probabilistic combined effect flood analyses were performed for this combination:
  - The resulting stillwater elevation for the deterministic analysis is 24.2 feet, MSL. This elevation is the combination of the maximum modeled stillwater (i.e., inclusive of wave setup and the 25-year river flood flow) elevation of 21.0 feet MSL, uncertainty effects of 0.8 feet, and the difference between the peak simulated tide elevation at Sewells Point, VA and the antecedent water level of 2.374 feet, which includes applicable sea level rise. The direction of the coincident waves near the SPS power block (near the Discharge Canal) for the PMSS are away from the site and do not result in wave runup. Coincident waves during the PMSS are limited to the area near the Low Level Intake Structure, where a maximum standing wave crest elevation of 38.8 feet, MSL was calculated. As shown in Figure 2.9-13, the peak of the deterministic surge occurs after approximately 2 days of storm surge simulation (i.e., where day 14 marks the start of the storm simulation with representation of dynamic tide conditions).
  - The resulting stillwater elevation for the probabilistic analysis is 20.8 feet, MSL. This elevation is the combination of the modeled stillwater (i.e., including wave setup and the 25-year river flood flow) elevation of 17.9 feet MSL, and uncertainty effects of 2.95 feet (i.e., based on Intake Canal assessment), which includes applicable sea level rise. The direction of the coincident waves near the power block (near the Discharge Canal) for the 1E-6 AEP storm surge are also away from the site and do not result in wave runup near the power block area of the site. Coincident waves during the 1E-6 AEP storm surge results in a maximum wave crest elevation of 31.4 feet, MSL near the Low Level Intake Structure. As shown in Figure 2.9-15, the peak of the probabilistic surge occurs between 2.0 and 2.1 days of storm surge simulation (i.e., where day 0 marks the start of the storm simulation under static tide conditions).

2.9.4 References

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Table 2.9-1: Wave Characteristics for Flooding Caused by Precipitation Events

Location	Fetch Length (miles)	Significant Wave Height (feet)	Peak Period (seconds)
Discharge Canal	0.33	0.8	1.2
Intake Canal[2]	0.45	0.9	1.4
Intake Embankment	3.9	2.7	2.8
Low Level Intake Structure	3.9	2.7	2.8

Table 2.9-2: Wave Runup and Wave Crest Elevations for Flooding Caused by Precipitation Events

Location	Runup Height (feet)	Wave Crest Height (feet)	Runup Elevation (feet, MSL)
Discharge Canal	1.9	--	17.6
Intake Canal[2]	2.6	--	32.6
Intake Embankment	1.0	--	16.7
Low Level Intake Structure	--	4.4	20.1

Table 2.9-3: Wave Characteristics During the Deterministic PMSS

Location	Significant Wave Height (feet)	Peak Period (seconds)	Wave Direction (degrees)[3]
Intake	11.2	6.4	170

[2] Wave characteristics at the intake canal are based on the 2-year wind speed calculated as part of the precipitation events combined event scenario.

[3] Cartesian convention, measured counter clockwise from the positive x-axis indicated direction waves are going to.

Table 2.9-4: Wave Runup and Wave Crest Elevations Caused by the Deterministic PMSS

Location	Stillwater Elevation (feet, MSL)	Runup Height (feet)	Wave Crest Height (feet)	Resultant Elevation (feet, MSL)
Intake Embankment near Low Level Intake Structure	24.2	15.7	Not calculated	39.9
Low Level Intake Structure	24.2	Not calculated	14.6	38.8

Table 2.9-5: Wave Characteristics during the Probabilistic Storm Surge

Location	Significant Wave Height (feet)	Peak Period (seconds)	Wave Direction (degrees)[4]
Intake	9.9	6.4	168

Table 2.9-6: Wave Runup and Wave Crest Elevation Caused by the Probabilistic Storm Surge

Location	Stillwater Elevation (feet, MSL)	Runup Height (feet)	Wave Crest Height (feet)	Resultant Elevation (feet, MSL)
Intake Embankment near Low Level Intake Structure	20.8	15.5	Not calculated	36.3
Low Level Intake Structure	20.8	Not calculated	10.6	31.4

[4] Cartesian convention, measured counter clockwise from the positive x-axis indicated direction waves are going to.

Table 2.9-7: Summary of Combined Event Maximum Water Elevations

Combined Event Alternative	Stillwater Elevation (feet, MSL)	Maximum Wave Crest Elevation at Low Level Intake Structure (feet, MSL)	Maximum Runup at Intake Canal Earthen Embankment (feet, MSL)
H.1 – Probable Maximum Flood with Dam Failure	15.7	20.1	16.7
H.3 – Deterministic PMSS	24.2	38.8	39.9
H.3 – Probabilistic Storm Surge	20.8	31.4	36.3

Figure 2.9-1: PMF with Dam Failures Stillwater Elevation Inundation Map

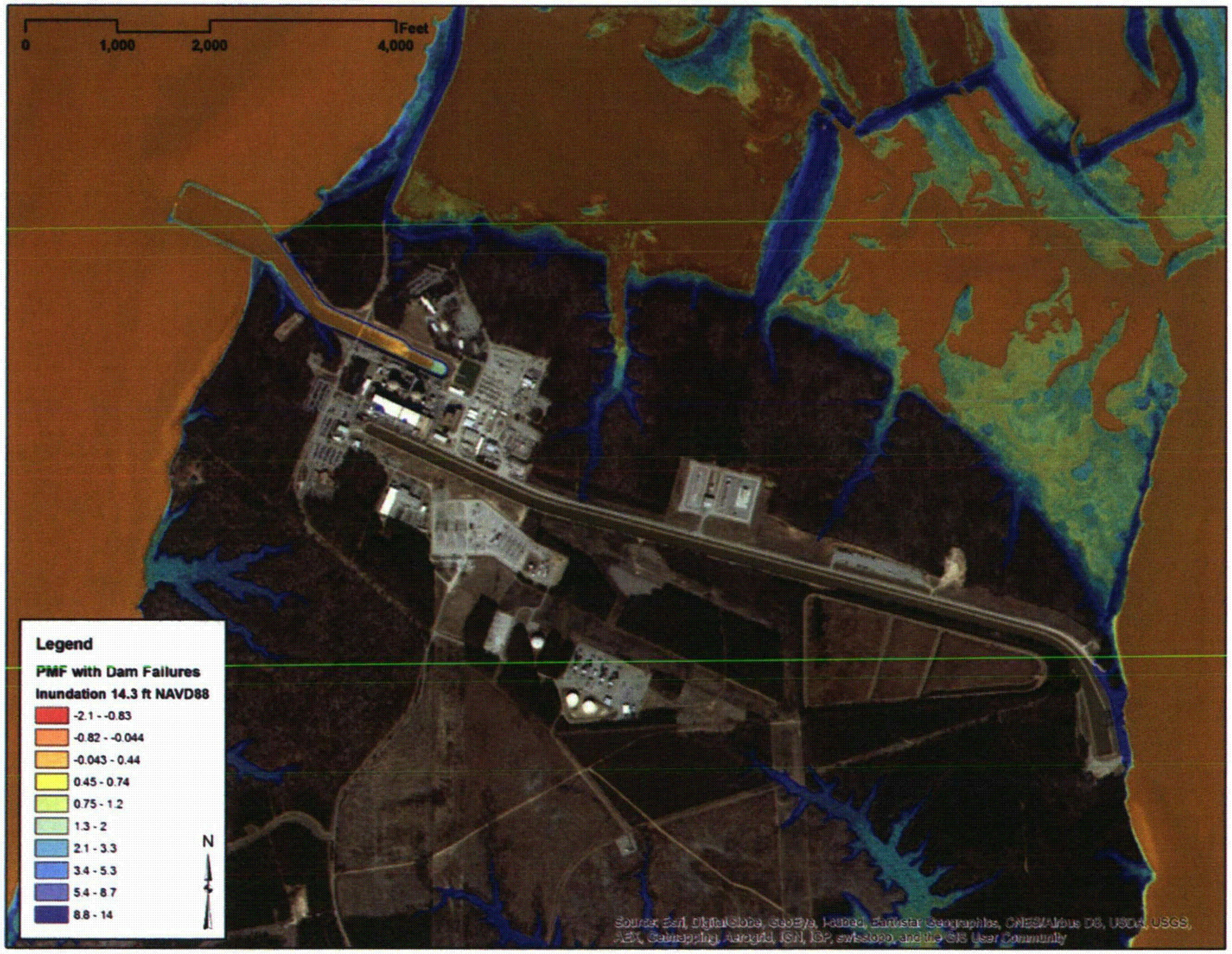


Figure 2.9-2: NCDC Wind Gage and NOAA Tide Gage Locations

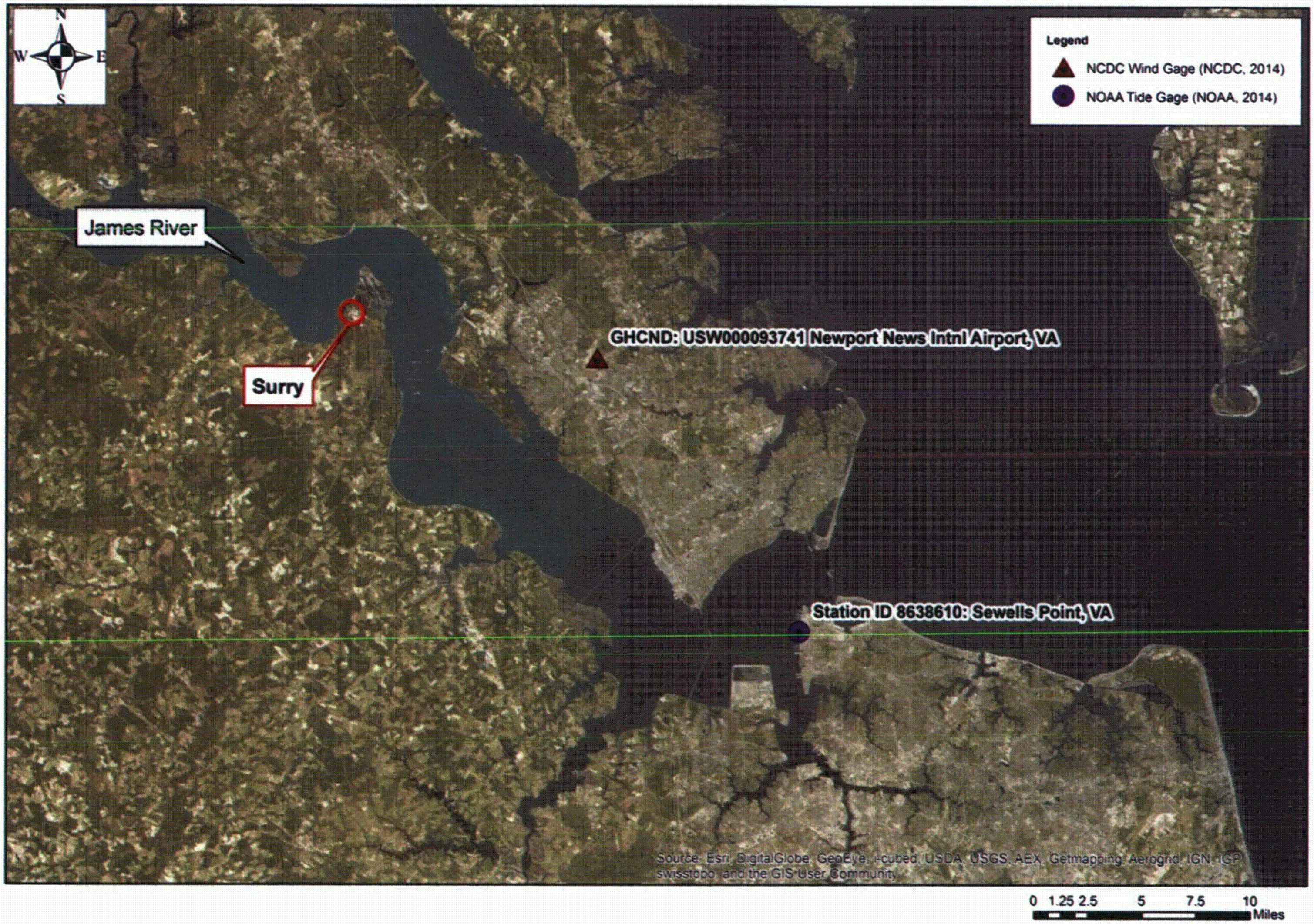


Figure 2.9-3: USGS Stream Gage Locations

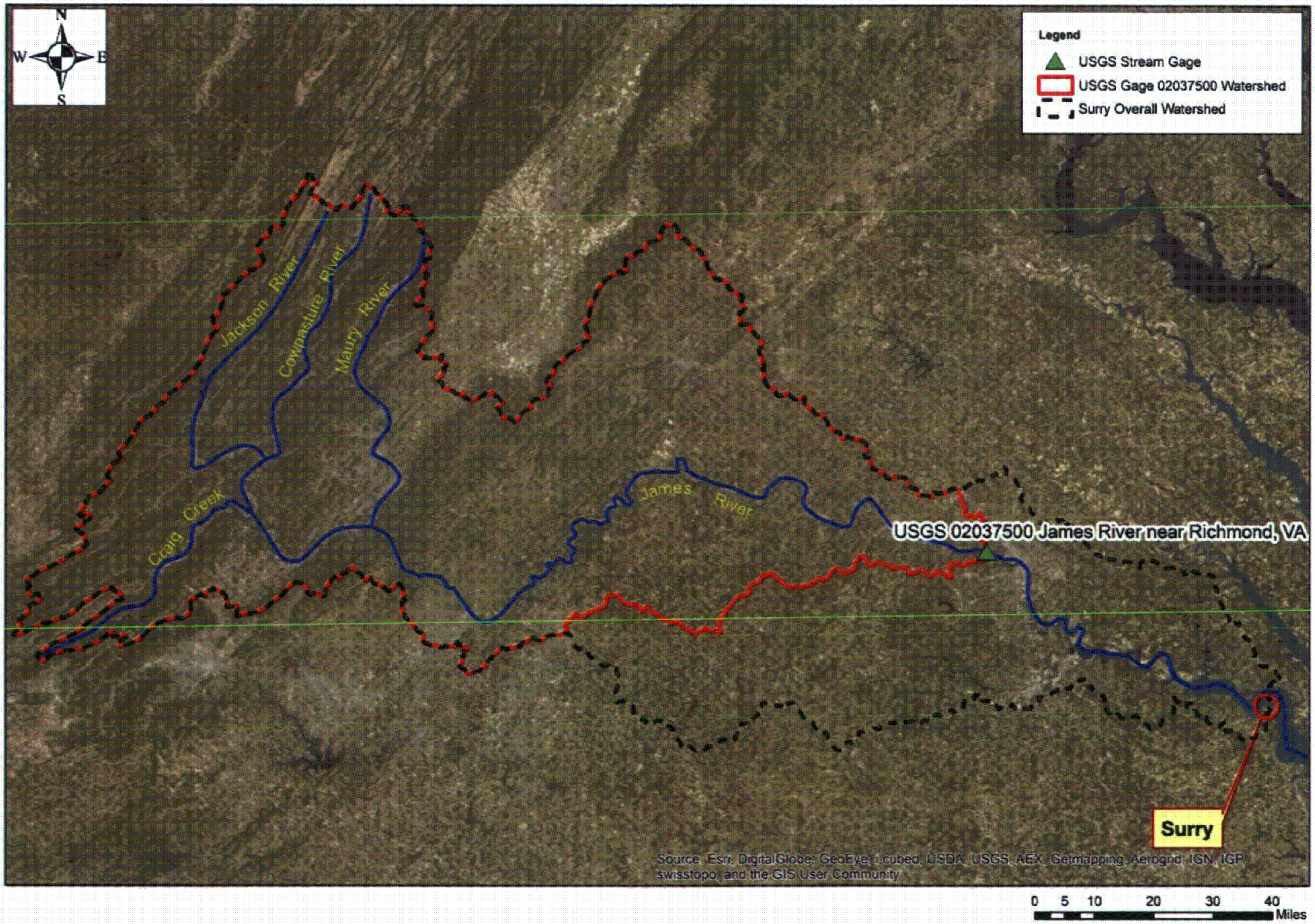


Figure 2.9-4: Deterministic PMSS Stillwater Elevation Inundation Map

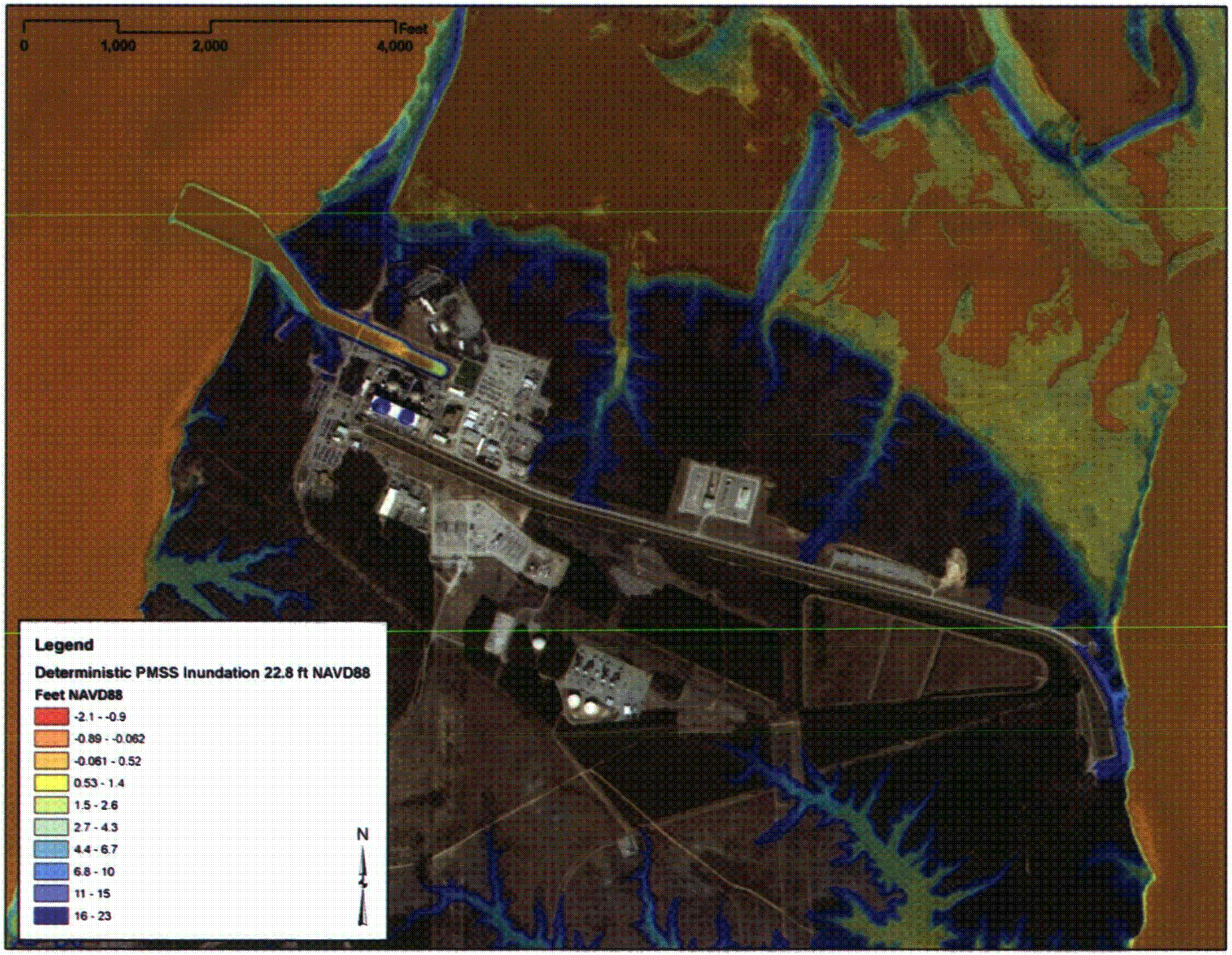
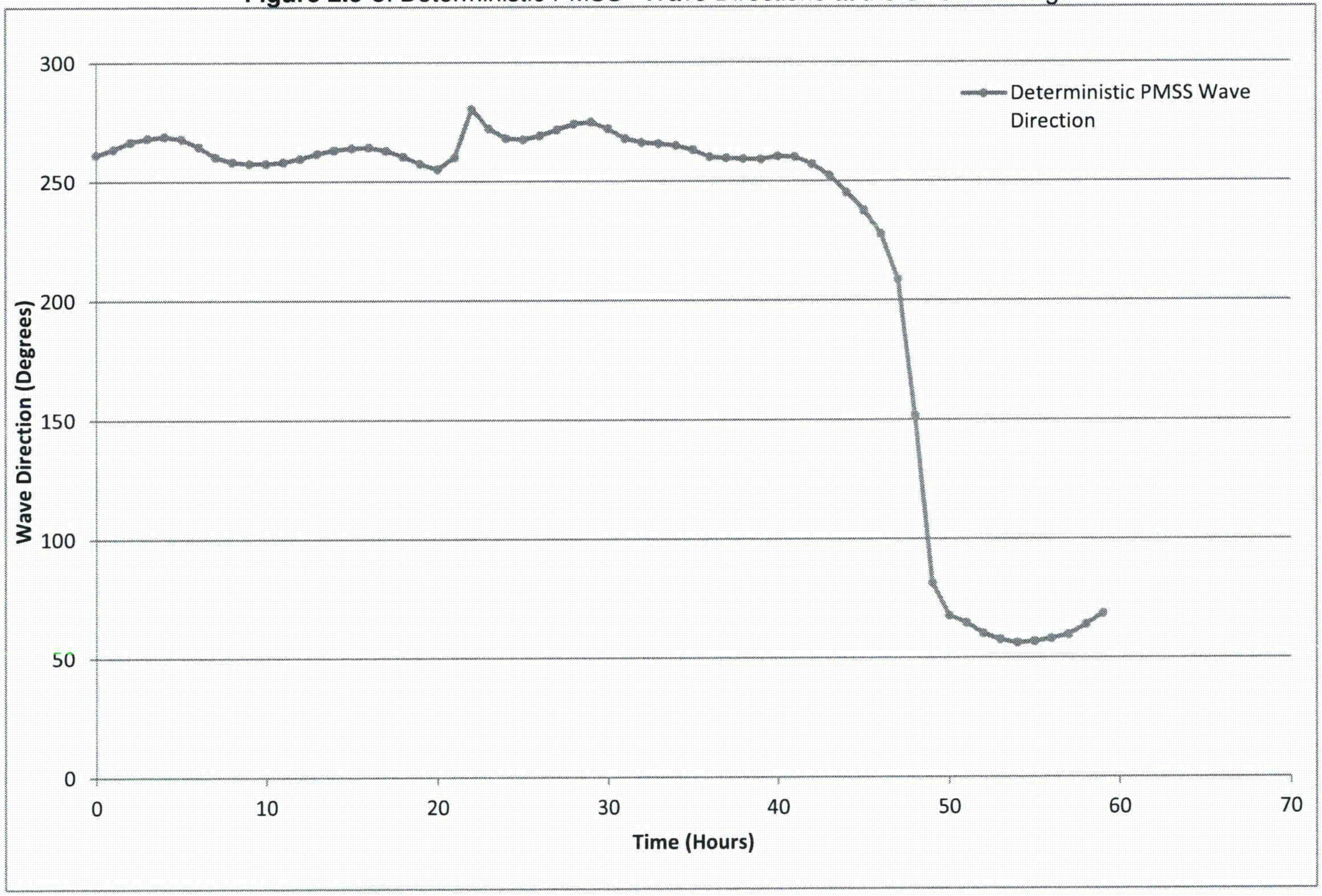


Figure 2.9-5: Deterministic PMSS –Wave Directions at the SPS Discharge*



* Wave direction is measured in Cartesian convention, counter clockwise from the positive x-axis indicating the direction waves are going to.

Figure 2.9-6: Transect Slope Locations for Wind Wave Effects

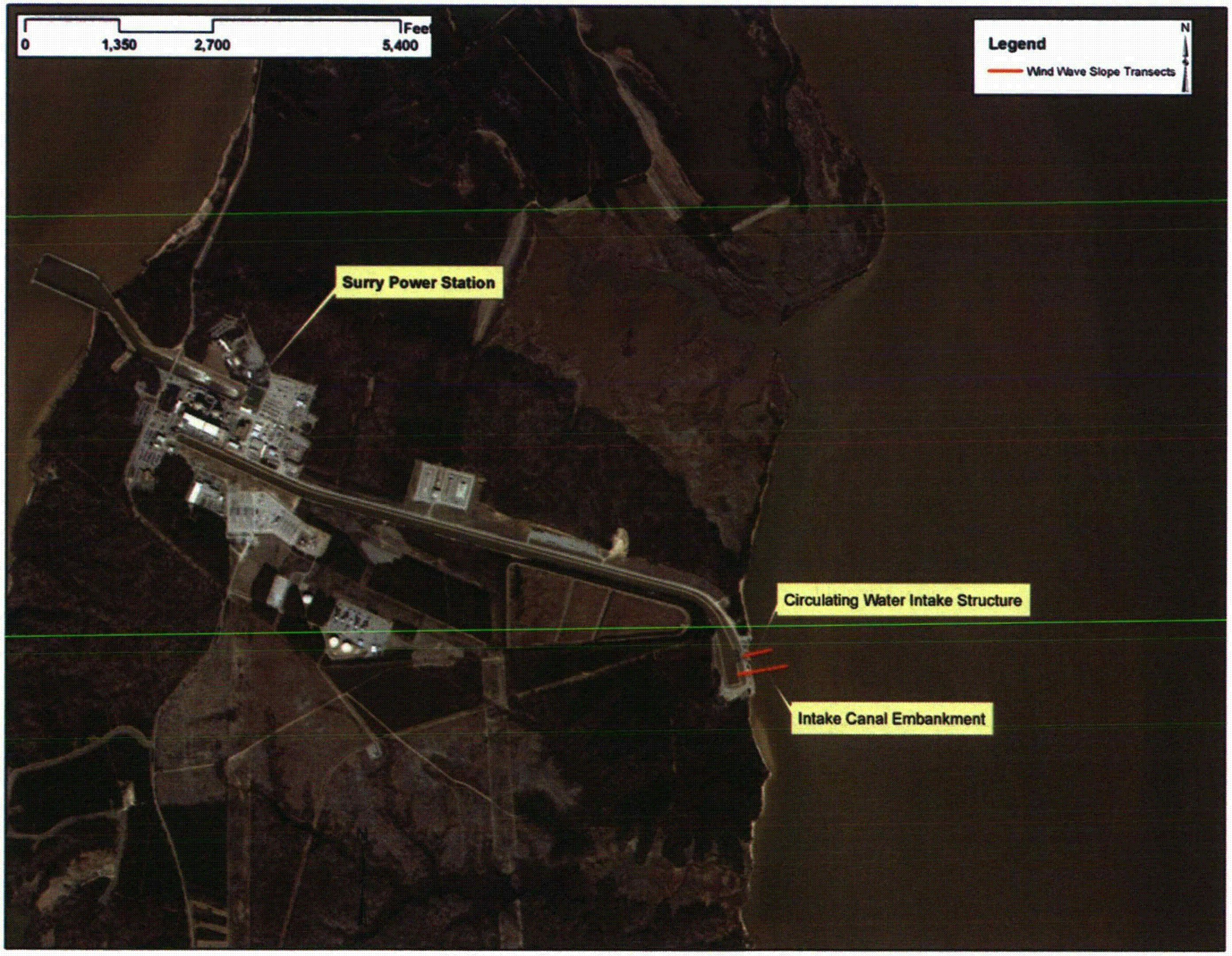


Figure 2.9-7: Probabilistic Storm Surge Stillwater Elevation Inundation Map

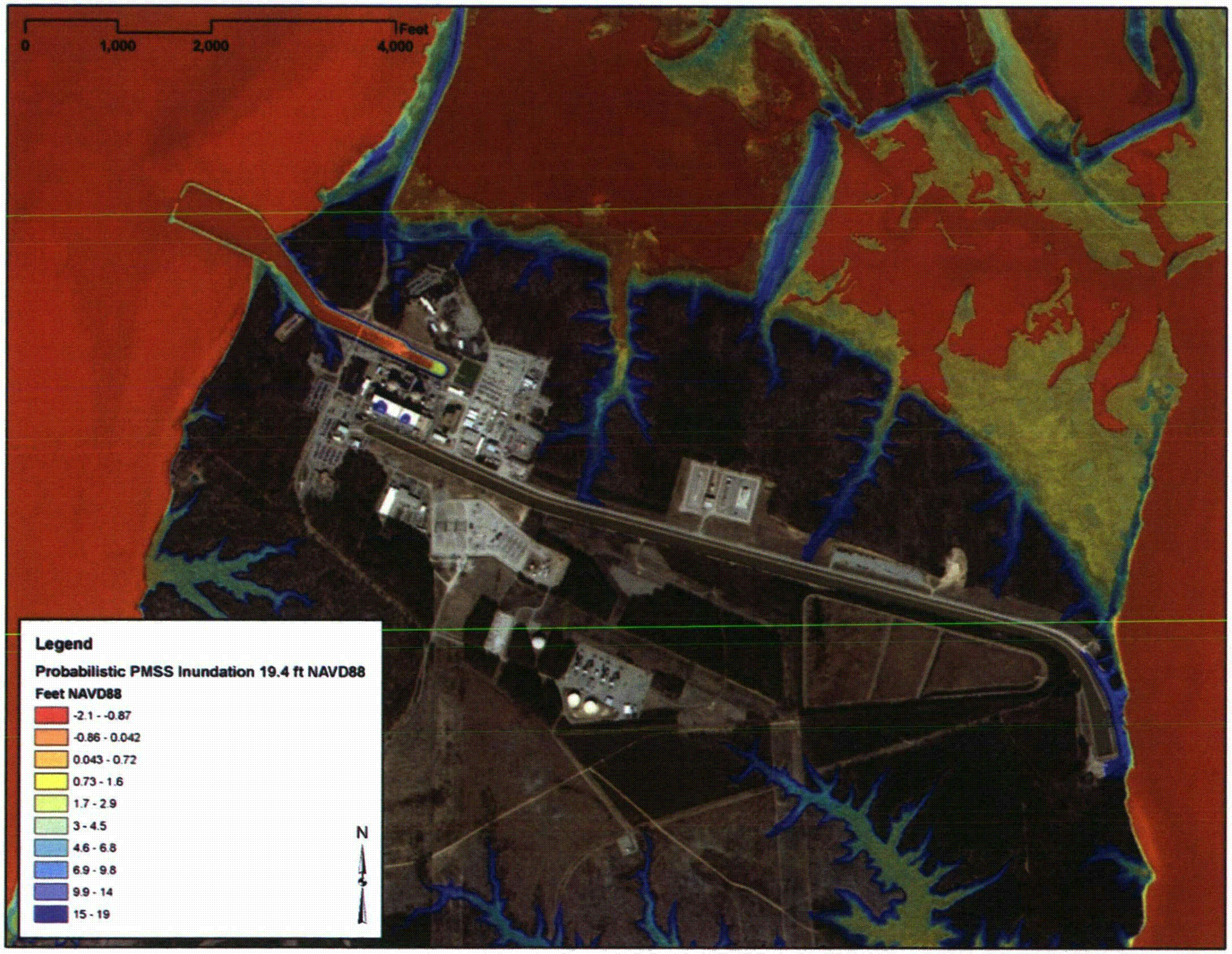
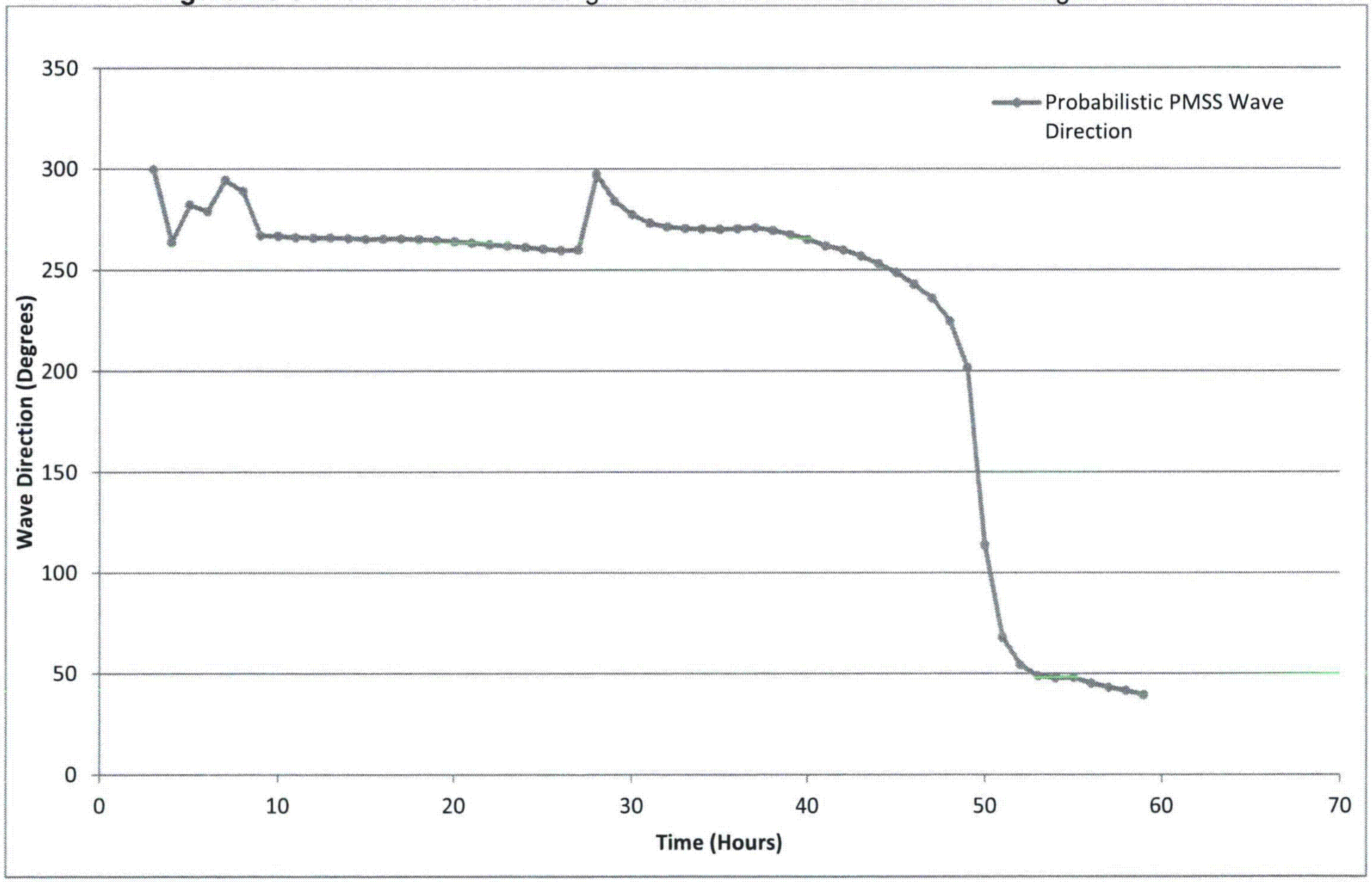


Figure 2.9-8: Probabilistic Storm Surge –Wave Directions at the SPS Discharge†



† Wave direction is measured in Cartesian convention, counter clockwise from the positive x-axis indicating the direction waves are going to.

Figure 2.9-9: Schematic of Wave Loading

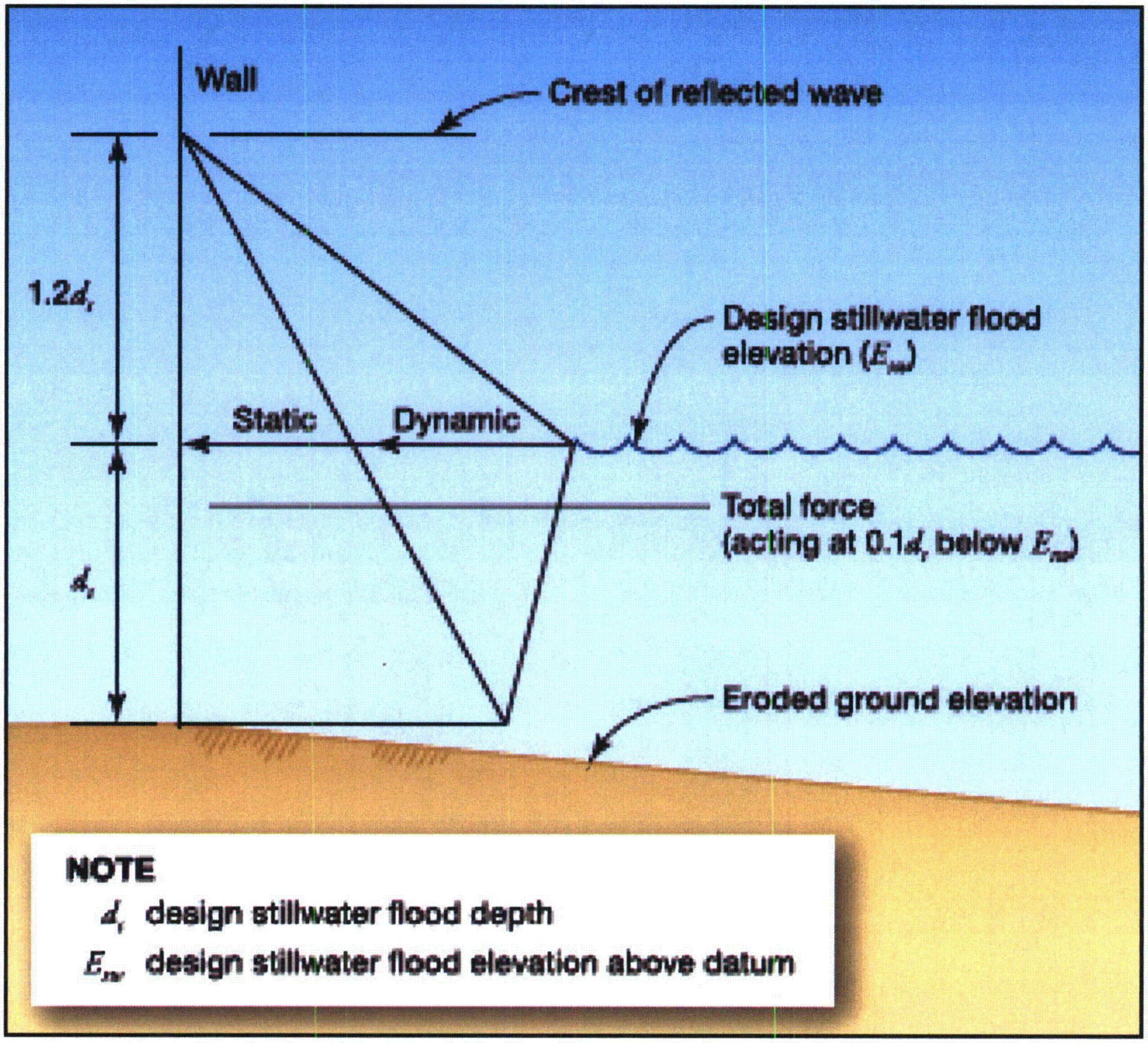


Figure 2.9-10: Transects for Fetch Limited Wave Growth

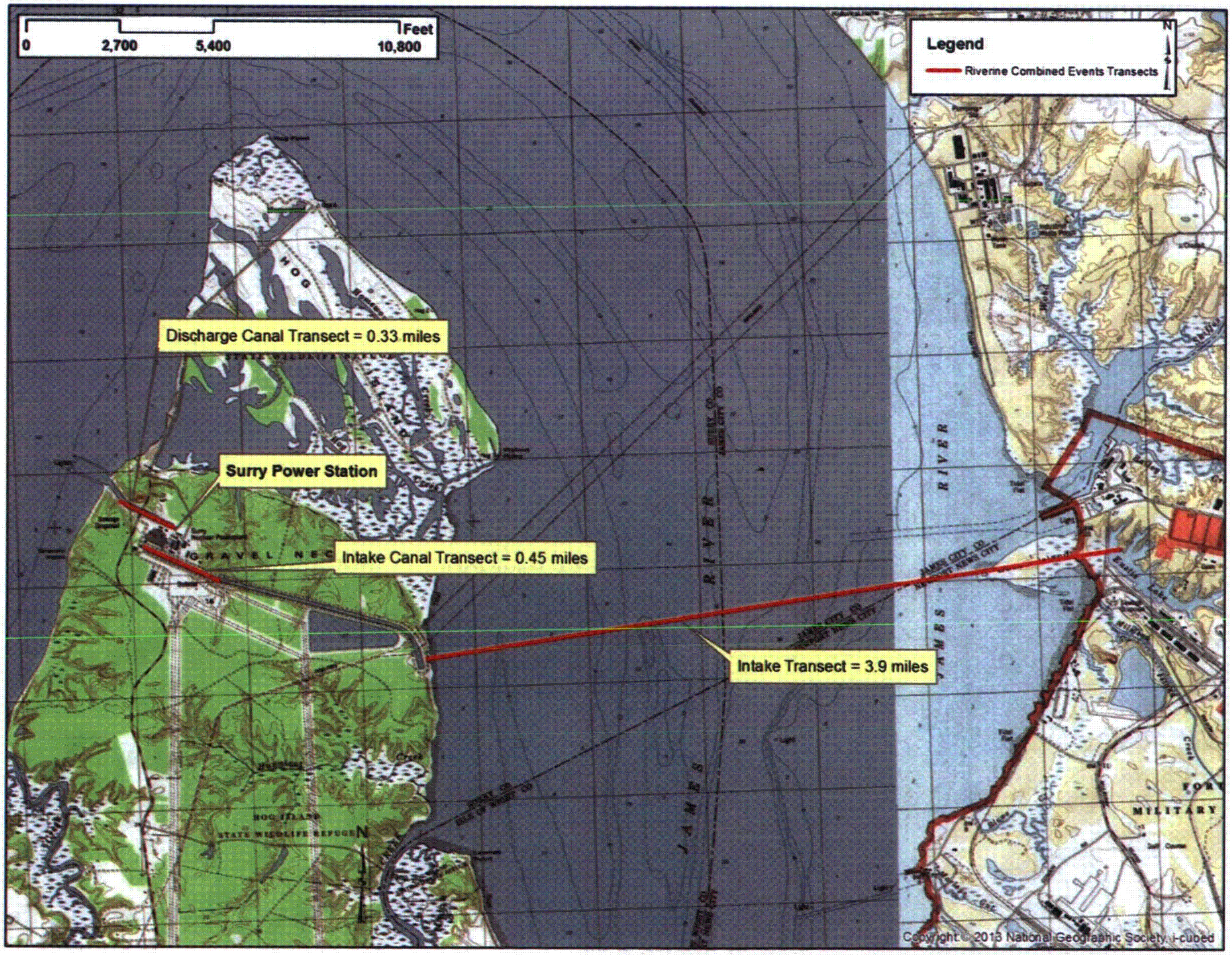


Figure 2.9-11: Low Level Intake Structure Profile

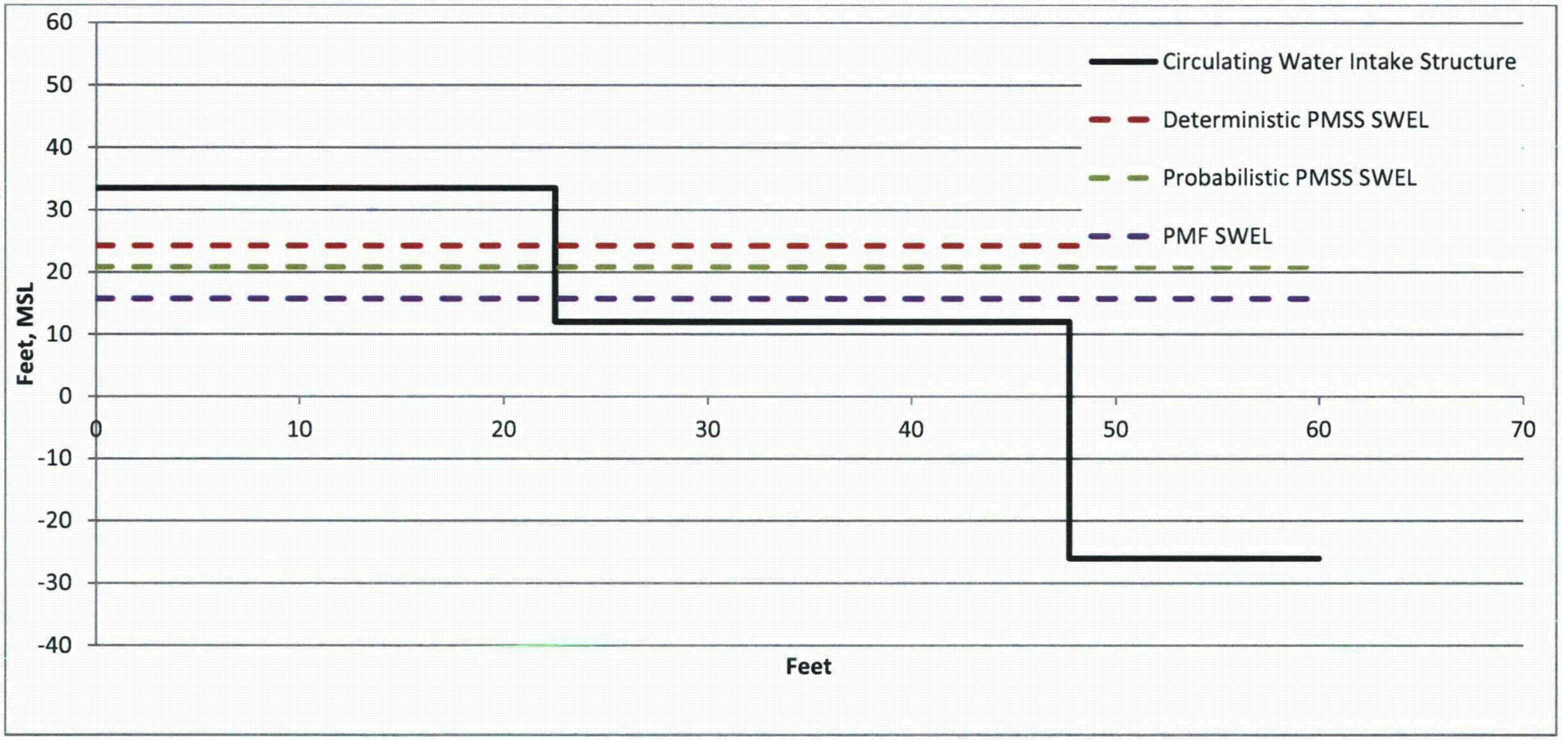
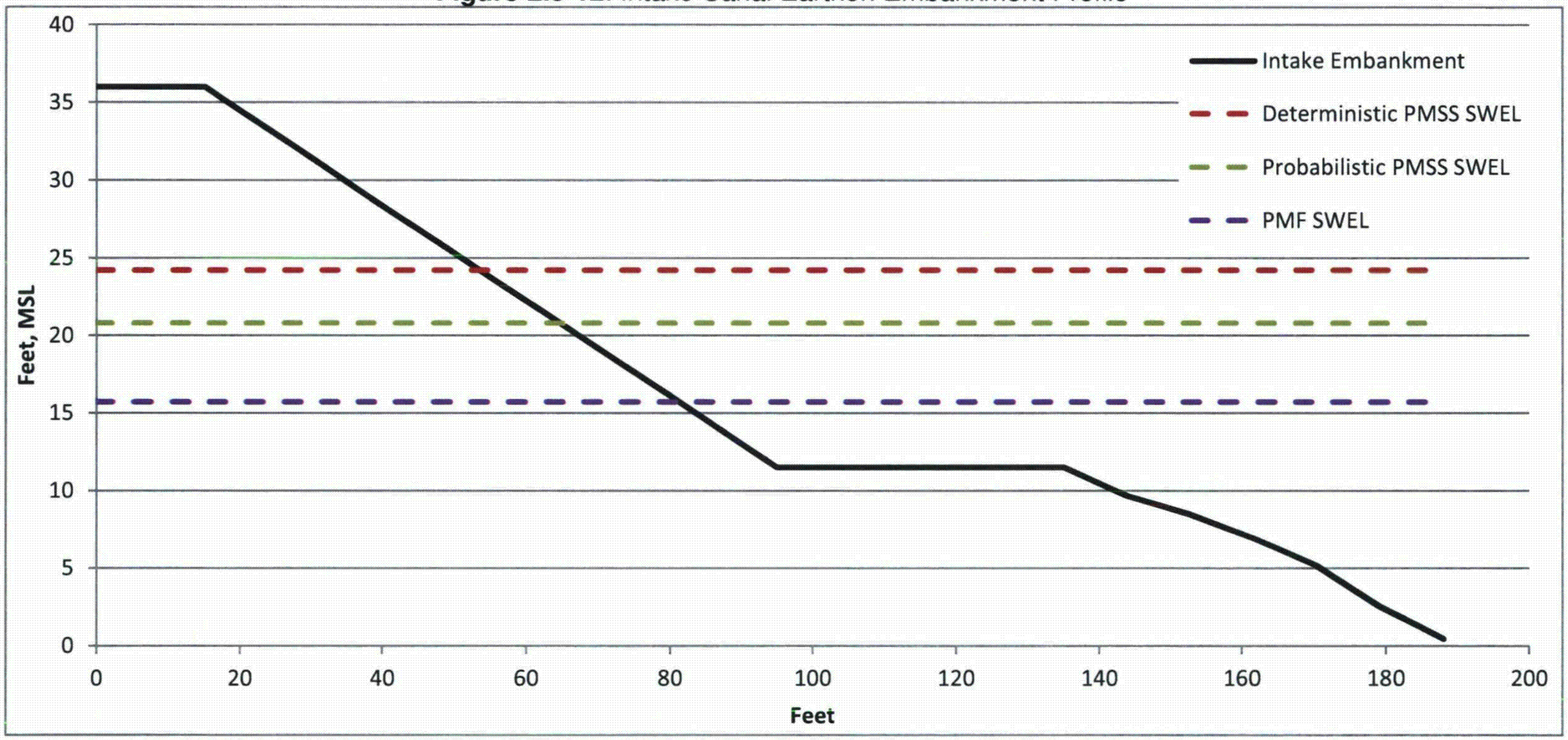


Figure 2.9-12: Intake Canal Earthen Embankment Profile*



* Profile is based off of the Intake Earthen Embankment in Drawing No. 11448-FY-1E SH-001 (Dominion, 2014a). Seaward slope based on the NED 1/9 arc-second DEM (USGS, 2013).

Figure 2.9-13: Stage Hydrographs (surge+wave setup) for Alternative 3 - deterministic

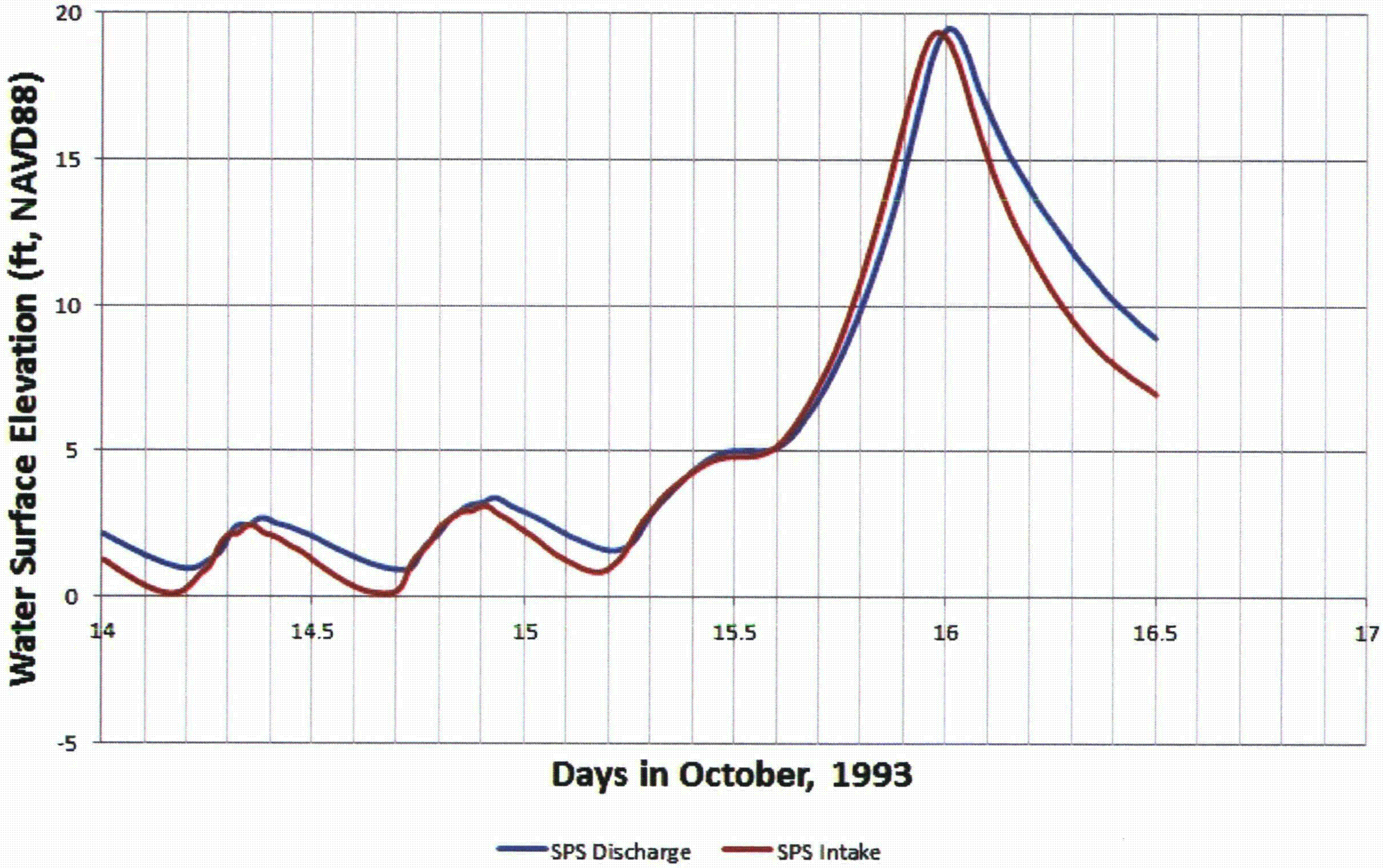


Figure 2.9-14: Wind Speed vs time for Alternative 3 –deterministic

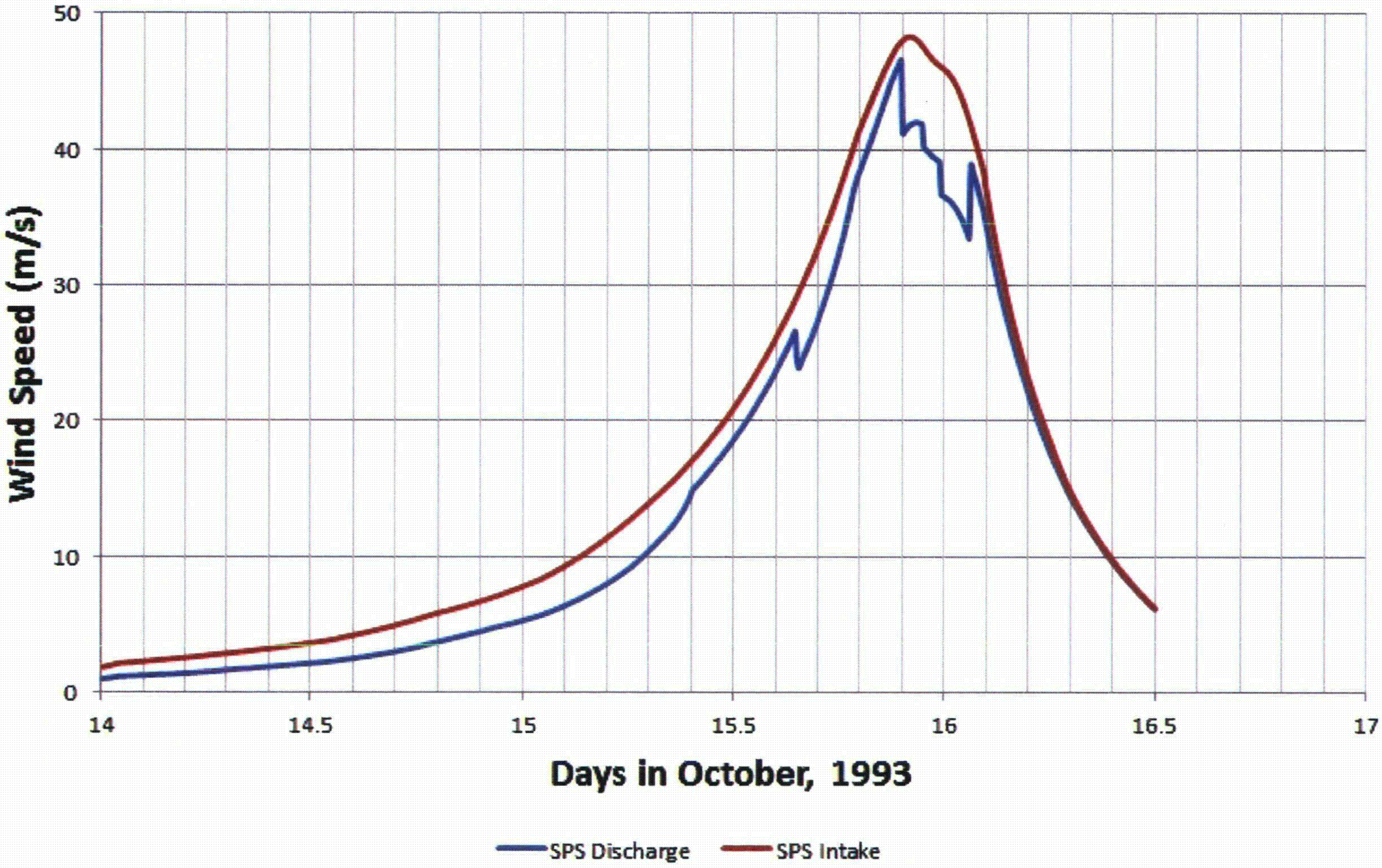


Figure 2.9-15: Stage Hydrographs (surge+wave setup) for Probabilistic Storm Surge

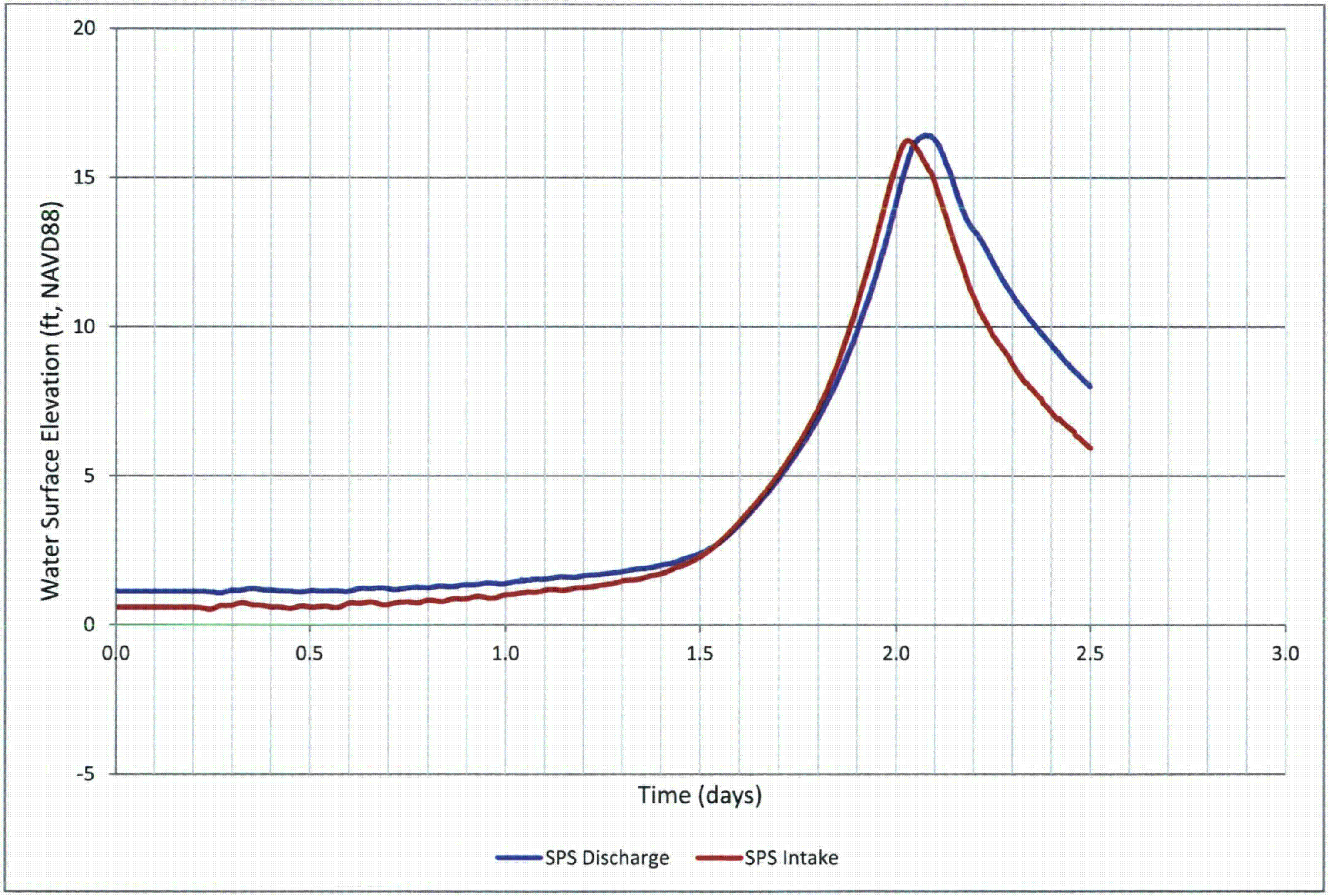


Figure 2.9-16: Wind Speed vs Time for Probabilistic Storm Surge

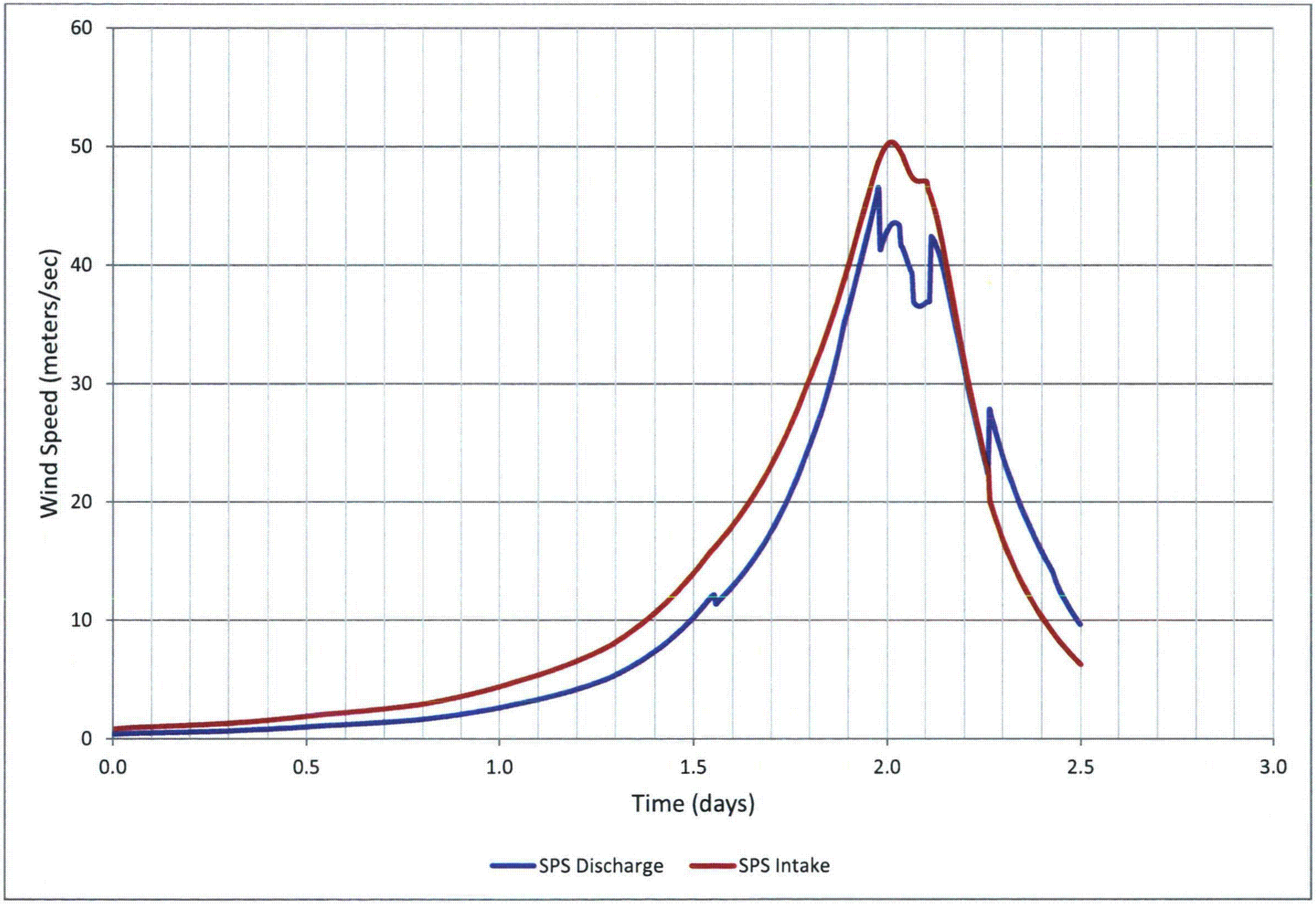


Figure 2.9-17: ADCIRC+SWAN Mesh and Intake/Discharge Nodes

3.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

This section provides a comparison of current and reevaluated flood causing mechanisms at SPS identified in Enclosure 2 of the NRC RFI letter pursuant to Title 10 CFR 50.54(f) dated March 12, 2012.

An assessment of the current design basis flood elevation is provided relative to the beyond design basis, reevaluated flood elevation. A conclusion of whether or not the current design basis flood bounds the reevaluated flood hazard is provided for each flood mechanism at SPS. The UFSAR for SPS (Dominion 2014) is used as a source of current design basis information for flooding. The SPS Flooding Walkdown report, which was reviewed and approved by the NRC, also contains information describing the current design basis (Virginia Electric and Power Company, 2012).A summary table is provided in Table 3.0-1. Flood protection elevations for various SSCs at SPS are provided in Table 3.0-2.

As discussed below, the following reevaluated external flood mechanisms exceed the current design basis flood elevation at one or more areas of SPS:

- Local Intense Precipitation (LIP) (see Section 3.1);
- Dam Failures (see Section 3.3);
- Storm Surge (see Section 3.4);
- Combined Effect Flooding (see Section 3.9).

Interim flood protection measures for the safety-related and important-to-safety SSCs are described in Section 4 of this report.

3.1. Local Intense Precipitation

Current Design Basis

The SPS UFSAR discusses general extreme 24-hour precipitation amounts (historic values of 11.4 inches) but does not contain analyses or provide flood elevations due to the LIP (Dominion 2014). There are no design features credited for mitigating the LIP. Flooding walkdowns performed at SPS at each site location around the power block structures assumed the flood level due to LIP to be the ground level with no accumulation or significant ponding (Virginia Electric and Power Company, 2012).

Reevaluation Results

The reevaluation used a two-dimensional hydrodynamic computer program to develop flood levels due to the LIP. A site-specific meteorology study was performed to develop the local Probable Maximum Precipitation (PMP) as an input to the LIP analysis. The site-specific PMP values are more refined than generic Hydrometeorological Report Nos. 51 and 52 and are used in this Section, consistent with the Hierarchical Hazard Assessment (HHA) approach. Resulting maximum flood depths and maximum LIP flood elevations vary by location. The LIP results in maximum water surface elevations at the site ranging from 26.9 feet, MSL at Door 1-BS-DR-74 to 29.4 feet, MSL at the Doors into the Maintenance Building at the south eastern end of the site. The resulting maximum flood depths at the site range from 0.2 ft at the "ECST" to 5.0 ft at Doors 1-BS-DR-D23-1 and -2. Table 2.1-8 presents the maximum LIP flood depths and elevations at many door locations throughout SPS.

Please refer to Section 4.0 for a discussion of interim actions that have been developed to respond to LIP flooding.

3.2. Probable Maximum Flood in Streams and Rivers

Current Design Basis

The SPS UFSAR (Dominion, 2014) summarizes historic flood information on the James River, such as the 1972 flood resulting from Hurricane Agnes. The UFSAR concludes that, due to the wide floodplain at SPS, the rise above normal water levels was relatively minor during the 1972 flood. The UFSAR also notes an analysis for the 50-year return period flood which resulted in a rise of no more than 1 foot above normal mean river level, if not accompanied by unusual meteorological tides. Probable Maximum Flood analyses are not discussed in the UFSAR.

Reevaluation Results

The reevaluation addresses the potential for flooding at SPS due to the Probable Maximum Flood (PMF) on the James River (Section 2.2). The PMF peak flow in the James River at SPS was estimated as 867,000 cfs. The maximum 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 sill of the pump room door and the air intake louver openings for the Emergency Service Water Pump House located on the Circulating Water Intake Structure (Low Level Intake) are located at elevation 21.2 feet (MSL). The air intake louver openings are protected against flooding up to elevation 24.0 feet (MSL). Therefore, the freeboard is 11.9 feet. No adverse impact is expected on the intake canal and structure, due to the rise of water level in the James River caused by the PMF.

Therefore, the current design basis flood evaluation is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

3.3. Dam Failures

Current Design Basis

The SPS UFSAR (Dominion 2014) notes that there are no known or planned river control structures on the James River. It also notes that small impoundments on tributaries in the upper reaches of the James River exist, but their size and location preclude any effect or danger to the safety-related structures at SPS. Detailed dam failure analyses are not discussed in the UFSAR.

Reevaluation Results

The reevaluation considers upstream dam failure in combination with the PMF on the James River and the hypothetical failure of the Intake Canal embankment and the Settling Pond embankment (Section 2.3). Guidance applicable to dam failure including NRC's Interim Staff Guidance (ISG) for Assessment of Flooding Hazards Due to Dam Failure dated July, 2013 (NRC, 2013) was not required when SPS was design and constructed.

The PMF combined with hydrologic upstream dam failures results in a calculated peak water surface elevation at SPS of 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. The PMF and hydrologic dam failure elevation bounds the 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). While Sunny Day and Seismic failure modes may, in general, present warning time challenges beyond controlling water surface elevations, the available vertical margin for the hydrologic dam failure demonstrates that no such challenges are present at SPS. Therefore, the current design basis flood evaluation for the failure of upstream dams is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

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 (i.e., there is no postulated breach location that would direct flooding water toward the plant). Further action is not necessary.

Failure of the Intake Canal embankment is not plausible under hydrologic and seismic loading conditions (see Section 2.3). The Sunny Day embankment failure scenario cannot be screened out as per NRC's ISG (USNRC, 2013). The Sunny Day failure analysis uses conservative assumptions consistent with an LIP analysis (i.e., no infiltration, storm drains non-functional). The results indicated Sunny Day failure flood depths generally on the order of approximately one foot, with specific depths ranging from zero (dry) to locally as high as about 4 feet (corresponding to elevations ranging from 26.4 feet MSL to 28.1 feet MSL). See Section 4.0 for additional information.

3.4. Probable Maximum Storm Surge

Current Design Basis

The SPS UFSAR describes potential flooding due to the PMH (Dominion 2014). The PMH was developed using NOAA Technical Report HUR 7-97 which has been superseded by NWS-23. The PMH parameters are as follows:

- Central pressure index = 26.97 inches
- Radius of maximum winds = 35 nautical miles
- Forward speed of translation = 22 knots
- Maximum wind speed = 135.4 miles per hour (117.7 knots)

Calculation of open coast surge was performed using two computer programs (unidentified). The UFSAR states: "The first program utilizes functions of wind speed, wind vector, and radial distance along the design axis and the traverse to compute the onshore and alongshore wind stress components, the rise in water level due to atmospheric pressure reduction for each time period at the beginning and end of each reach, and the average wind stress coefficient for each reach. The second program utilizes the output from the first program and the offshore bottom profile to compute the onshore and alongshore components of the open coast surge."

Storm surge routing in the James River was performed using the methods of a 1973 USACE Technical Bulletin (USACE, 1973). An additional wind setup component resulting from an average wind speed of 91 miles per hour along the PMH maximum wind axis is added along the river reach.

The maximum PMH stillwater level at SPS near the Low Level Intake Structure was calculated to be 22.7 feet MSL. Wave effects were also calculated and are described in Section 3.9.

The maximum PMH stillwater level of 22.7 feet MSL is 3.8 feet below the SPS site grade elevation of 26.5 feet MSL. SSCs important to safety are protected to at least elevation 24 feet MSL. The Low Level Intake Structure is located on the eastern shore of the Hog Island peninsula, about 1.7 miles east of SPS. Emergency service water pumping equipment (pumps, diesel-driven pump motors, fuel oil tanks, etc.) is housed in a reinforced-concrete structure, the emergency service water pump house (ESPH), above the deck of the Circulating Water Low Level Intake Structure. The floor and walls of the ESPH are watertight. The sill of the ESPH door and the air intake louver openings are located at El. 21.17 feet (21 feet, 2 inches). 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 up to elevation 24 feet MSL.

The corresponding seal plates and exterior covers for both ESPH doors and the intake louver openings are required to be installed whenever hurricane conditions exist, or are forecast to exist. The door seal plates and louver opening covers are procedurally installed. A procedure requires the ESPH doors to have a temporary steel plate (i.e., flood gate) installed to limit water ingress into the ESPH before the arrival of the PMH. The door seal plates, in the installed position, provide protection up to elevation 24 feet MSL.

With the normal air intake louvers covered, air for operation of the diesel-driven emergency service water pumps (ESWP) would be provided through the motor-operated dampers located in the top of the ESPH structure. The location of these dampers under the exhaust hood on the ESPH roof 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 feet. The roof elevation of the structure is 33.5 feet MSL. The roof is watertight and the exhaust outlet is configured to prevent rainwater flow into the exhaust.

Reevaluation Results

The reevaluation performed detailed analyses of the PMH and storm surge consistent with the HHA approach. First, the PMH was developed deterministically and the resulting PMSS was calculated using a two-dimensional hydrodynamic program, ADCIRC. As a second step, refinement of the analysis was performed by completing a probabilistic storm surge calculation, supported by a site-specific hurricane meteorology and climatology study. Several additional ADCIRC simulations were performed to support a Joint Probability Method-Optimum Sampling calculation of very low probability storm surge. At an annual exceedance probability of approximately 1E-6 (i.e., return period of 1,000,000 years), storm surge stillwater elevations at the SPS intake and discharge locations are calculated to be 18.9 ft MSL and 18.4 ft MSL, respectively. These stillwater elevations are used as an input to the combined effect analysis to develop final maximum flood levels at SPS—see Section 3.9.

3.5. Seiche

Current Design Basis

The UFSAR for SPS does not include a discussion of flood elevations due to seiche (Dominion 2014).

Reevaluation Results

Seiche was analyzed as part of the re-evaluation. Significant seiches on the James River, intake canal and discharge canal at SPS are not expected based on the screening analysis performed using Merian's formula, a statistical analysis of historical water level data and literature review. No further analysis or detailed modeling was necessary.

Therefore, the current design basis flood evaluation is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

3.6. Tsunami

Current Design Basis

The SPS UFSAR does not discuss tsunami potential (Dominion 2014).

Reevaluation Results

The tsunami flooding reevaluation analysis concluded that there is a regional tsunami hazard potential at SPS. Numerical modeling was then performed to account for the complex geography in and around the James River and Chesapeake Bay (see Section 2.6).

Several tsunamigenic sources were assessed. Based on simulations, the worst-case scenario (i.e. the scenario causing maximum inundation at SPS) is a tsunami resulting from an extreme flank failure of the Cumbre Vieja Volcano, when the arrival of the leading wave crest is synchronized with the maximum tide in the vicinity of SPS. For this case, the inundation elevations at SPS reach Elevation 7.0 feet MSL. Since SPS site grade is 26.5 feet MSL and SSCs important to site safety are protected to at least 24.0 feet MSL (Table 3.0-2), tsunami is not expected to result in flooding at SPS.

Drawdown due to tsunami was also calculated as part of the re-evaluation. The most severe drawdown was calculated to be -3.6 feet MSL, as a result of a near-field submarine mass failure. This value is less severe than existing site low water thresholds (-4.8 feet MSL).

Therefore, the current design basis flood evaluation is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

3.7. Ice-Induced Flooding

The SPS UFSAR (Dominion, 2014) states that: "*it is highly unlikely that the formation of ice on the James River would obstruct the flow and cause flooding, due to the salinity of the river below the site. Thus, ice flooding is precluded as a source of flooding the site.*" Detailed analyses are not discussed in the UFSAR.

Reevaluation Results

The re-evaluation concluded that temperature data indicates infrequent but possible temperatures below freezing. The historic record indicates ice jams are infrequent in the James River and have not been recorded in the vicinity of SPS. The largest historic ice jam of 18 feet occurred in 1936 near Richmond, Virginia (Section 2.7). The HHA approach was used and the largest historic 18-foot ice jam was applied at SPS on top of a normal water level in the James River (mean tidal range of 1.0 ft at SPS). The resulting freeboard was calculated as follows:

Maximum Water Level = Elevation 1.0 ft (MSL) + 18 ft = Elevation 19 ft (MSL)

Minimum Freeboard = Station Ground Grade – Maximum Water Surface Elevation = Elevation 26.5 ft (MSL) – Elevation 19 ft (MSL) = 7.5 ft

The sill of the pump room door and the air intake louver openings for the Emergency Service Water Pump House located on the Low Level Intake Structure are located at elevation 21.2 feet (MSL). Therefore, the freeboard is 2.2 feet. The air intake louver openings are protected against flooding up to elevation 24.0 feet (MSL). No adverse impact is expected on the intake canal and structure, due to the rise of water level in the James River caused by ice jams. The remaining freeboard is also available to convey elevated riverine base flows or contain elevated tide levels that may occur coincident with ice-induced flooding. Since no impacts were noted, the analysis ended as per the HHA approach. Note that the ice-induced flood elevation would be lower if more detailed analyses were performed accounting for the James River width near SPS and attenuation of the ice jam flood.

Therefore, the current design basis flood evaluation is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

3.8. Channel Migration or Diversion

The SPS UFSAR (Dominion, 2014) does not discuss channel migration or diversion.

Reevaluation Results

The reevaluation concluded that the James River has not exhibited a tendency to meander towards SPS in a manner that could flood or otherwise affect SSCs. The James River is a maintained, navigable waterway near SPS. Much of the critical shoreline at SPS is composed of soils that are strong and stable, with moderate to high shearing strengths. Given these conditions, channel migration is not considered to be a potential contributor to flooding at SPS. Therefore, the current design basis flood evaluation is considered to be consistent with the conclusions of the reevaluated flood hazard evaluation and further action is not necessary.

3.9. Combined Effect Flooding

Current Design Basis

The SPS UFSAR (Dominion, 2014) summarizes the wave calculations performed coincident with the PMH at the east end of the site, at slopes, and the ESPH. The waves at the east end of the site near the ESPH were calculated to be 9.7 feet in height with a wave length and period of 159 feet and 5.6 seconds, respectively. An average slope of bank of 1 vertical to 5 horizontal was used to calculate runup of 8.24 feet for smooth slopes and 3.60 feet for rubble slopes. These values were averaged to 5.9 feet because the slopes consist of material between the roughness of smooth and rubble slopes. The maximum runup elevation was therefore considered to be 28.6 feet MSL (22.7 feet MSL stillwater plus 5.9 feet runup).

The UFSAR notes that the maximum wind speed at the site was assumed to be 120.5 miles per hour from the east. With the wind blowing from the east, waves would travel away from the west side of SPS and no wave runup would be generated. Wave reflection at the west side of SPS was considered, however. The reflected wave height was calculated to be 1.3 feet. Runup of waves on the west side of the site based on slopes of 1 vertical to 2 horizontal was 1.3 feet. The maximum runup elevation on the west side of the site was therefore 24.0 feet MSL (22.7 feet stillwater plus 1.3 feet runup).

Wave runup at the Low Level Intake structure and the ESPH was not specifically described in the UFSAR beyond the waves summarized above. However, the UFSAR does acknowledge that occasional waves may cause splash and spray up the walls of the structure up to Elevation 36.2 feet MSL. It is noted these would not affect the integrity of the screen wells in the ESPH because the roof is watertight and the exhaust outlet is above all wave generated splash.

Maximum wave runup at the west end of the high-level intake canal was calculated based on a wind speed of 120.5 miles per hour associated with the PMH and an effective fetch of 1,500 feet. The wave height was calculated to be 1.7 feet and the wave runup for the smooth canal inner slope of 1 vertical to 1.5 horizontal was calculated to be 4.0 feet. The normal water level at the power station end of the canal will vary between Elevation 26 feet MSL and Elevation 30 feet MSL (Dominion, 2014). A minimum freeboard of greater than 4 feet between the canal water surface elevation and the top of the earthen embankment berm at Elevation 36 feet MSL is procedurally maintained in the intake canal during a hurricane. No overtopping was anticipated.

Please refer to Section 3.4 for a description of SPS structure elevations, including the ESPH and Low Level Intake Structure.

The SPS UFSAR reports a resultant wave thrust on the wall of the ESPH (on the deck of the Low Level Intake Structure) of 29,300 pounds per linear foot, acting at elevation 22.7 feet (e.g., the maximum stillwater elevation). The highly reinforced wall of the ESPH can withstand the loading. Other flood-related loading calculations are not provided.

Reevaluation Results

The reevaluation of combined effect flooding was based on the combination of floods provided in NUREG/CR-7046, Appendix H. These combined effect floods were considered to be appropriate for SPS. The results of the controlling combined effect flood scenarios for riverine and coastal flood hazards are provided below:

Floods caused by precipitation events (H.1): A maximum stillwater elevation at SPS of 15.7 feet, MSL was calculated. A standing wave crest elevation of 20.1 feet, MSL at the Low Level Intake Structure is predicted. Impacts due to the riverine stillwater and wave effects at SPS are not expected.

Floods along the shores of open bodies of water (H.3): Both deterministic and probabilistic combined effect flood analyses were performed for this combination. The probabilistic results are provided herein, since they represent the most refined analyses performed as part of the re-evaluation. The resulting stillwater elevation for the probabilistic analysis is 20.8 feet, MSL (including adjustments for uncertainty and wind setup). This elevation is the combination of the modeled stillwater (i.e., including wave setup and the 25-year river flood flow) elevation of 17.9 feet MSL, and uncertainty effects of 2.95 feet (i.e., based on Intake Canal assessment), which includes applicable sea level rise.

SPS site grade elevation is 26.5 feet, MSL, which is at least 5.7 feet above the maximum probabilistic storm surge combined effect flood stillwater elevation. Structures, systems, and components important to safety are protected to at least elevation 24 feet, MSL (Dominion, 2014). The direction of the coincident waves near the power block (near the Discharge Canal at the west side of SPS) for the 1E-6 AEP storm surge are away from the site. Therefore, there are no combined effect flood or wave impacts near the power block area of SPS.

Wave-related impacts are limited to the area near the Low Level Intake Structure and ESPH, including the Intake Canal embankment, east of SPS. The maximum standing wave crest elevation is 31.4 feet, MSL during the 1E-6 AEP storm surge near the Low Level Intake Structure. The standing wave condition would create a periodic flooding hazard, above the stillwater elevation of 20.8 feet MSL. The floor and walls of the ESPH are watertight. The sill of the ESPH door and the air intake louver openings are located at 21.17 feet (21 feet, 2 inches) MSL. 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 up to elevation 24 feet MSL. This is below the standing wave crest calculated in the reevaluation of 31.4 feet MSL.

With the normal air intake louvers covered, air for operation of the diesel-driven emergency service water pumps (ESWP) would be provided through the motor-operated dampers located in the top of the ESPH structure. The location of these dampers under the exhaust hood on the ESPH roof 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 feet. The roof elevation of the structure is 33.5 feet, MSL. The roof is watertight and the exhaust outlet is configured to prevent rainwater flow into the exhaust. The Probabilistic storm surge combined with the 25-year flood and wave action (31.4 feet, MSL) is not predicted to exceed the roof elevation of the ESPH.

Additionally, the east-facing slope of the Intake Canal embankment near the Low Level Intake is subject to wave action. The embankment in this area is grassed and may experience scour due to wave action during the combined effect flood.

Hydrostatic, hydrodynamic, and debris loading forces were developed for the Probabilistic combined effect flooding scenario. These forces are anticipated to be localized to the area around the Low Level Intake Structure. Hydrostatic forces total 2,422 pounds per foot acting at an elevation of 13.5 feet. Hydrodynamic forces total 3,105 pounds per foot acting at elevation 15.0 feet. Typical breaking wave loads are conservatively estimated to be up to 30,270 pounds per foot, which is similar to the 29,300 pound per foot value calculated as part of the current licensing basis. Debris impact loads act at the water surface elevation (stillwater elevation of 20.8 feet MSL) and are estimated to be up to 26,880 pounds per foot based on a debris weight of 2,000 pounds. Impact forces for flood loading conditions are not discussed in detail for the current licensing basis. Please refer to Section 4 for more information.

3.10. References

3.10-1 Dominion, 2014. Surry Power Station Updated Final Safety Analysis Report, Revision 46.02.

3.10-2 NRC, 2013. 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.

3.10-3 USACE, 1973. U.S. Army Corps of Engineers, "*The Computation of Tides and Currents in Estuaries and Canals*," Technical Bulletin 16, June, 1973.

3.10-4 Virginia Electric and Power Company, 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

Table 3.0-1: Current Design Basis Flood Elevations for Safety-Related and Important-to-Safety SSCs for SPS

Flooding Mechanism	Flood Critical Structure (Per UFSAR)	Current Design Basis Flood Level (MSL)	Current Flood Protection Elevation (MSL)	Reevaluated Flood Level (MSL)
Local Intense Precipitation	No flooding Expected	No Flooding Expected	26.5 ft	26.9 ft to 29.4 ft [1]
Combined Effects (East Side of Plant)	Refer to Table 3.0-2 [1]	28.6 ft (East Side)	Refer to Table 3.0-2 [1]	31.4 ft (Low Level Intake Structure)
Combined Effects (West Side of Plant)	Refer to Table 3.0-2 [1]	24.0 ft (West Side)	26.5 ft	20.8 ft (Protected Area)
Storm Surge (Stillwater Elevation)	Refer to Table 3.0-2 [1]	22.7 ft	Refer to Table 3.0-2 [1]	18.9 ft (Low Level Intake Structure)
Upstream Dam Failures	No Flooding Expected	No Flooding Expected	No Flooding Expected	15.7 ft (James River, No Flooding Expected)
On-site Dam Failures	No Flooding Expected	No Flooding Expected	No Flooding Expected	Settling Pond – No Flooding Expected Intake Canal – 26.4 ft to 28.1 ft [2]
Ice Induced Flooding	No Flooding Expected	No Flooding Expected	No Flooding Expected	Up to 19 ft (No Flooding Expected)
Probable Maximum Flood induced by Probable Maximum Precipitation	No Flooding Expected	No Flooding Expected	No Flooding Expected	12.1 ft (No Flooding Expected)

Notes are located on the next page

Table 3.0-1 (Continued): Current Design Basis Flood Elevations for Safety-Related and Important-to-Safety SSCs for SPS

Flooding Mechanism	Flood Critical Structure (Per UFSAR)	Current Design Basis Flood Level (MSL)	Current Flood Protection Elevation (MSL)	Reevaluated Flood Level (MSL)
Tsunami (including wave runup)	No Flooding Expected	No Flooding Expected	No Flooding Expected	7.2 ft (No Flooding Expected - Below Site Grade)
Seiche	No Flooding Expected	No Flooding Expected	No Flooding Expected	No Flooding Expected
Channel Migration or Diversion	No Flooding Expected	No Flooding Expected	No Flooding Expected	No Flooding Expected

Notes:
[1] Flood level is location dependent
[2] Intake Canal embankment failure was screened out, except for the sunny day failure which cannot be screened out according to NRC guidance (NRC, 2013)

Table 3.0-2. Current Design Basis Maximum-Probable-Flood Protection Levels for SPS Class I Structures

Class I Structure	Flood Protection Level (feet, 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 Low Level Intake Structure (emergency service water pump house)	24.0
High-level intake structure	36.0
Seal pit	Not Applicable

4.0 INTERIM EVALUATIONS AND ACTIONS

This section identifies the interim evaluation and actions taken or planned prior to completion of the integrated assessment to address any greater flooding hazards relative to the CLB. Identification of interim actions was requested in Enclosure 2 of the NRC RFI letter pursuant to Title 10 CFR 50.54(f) dated March 12, 2012.

Local Intense Precipitation resulting from the Site Specific Probable Maximum Precipitation is the bounding event that exceeds the Current Licensing Basis Flood Level. The proposed interim evaluations and actions to address this flooding concern are discussed in Section 4.1. Additionally, unique flooding concerns associated with Combined Effects Flooding (due to storm surge) and Dam Failure will be discussed in Sections 4.2 and 4.3, respectively.

4.1. Local Intense Precipitation

The LIP calculation, following Hydrometeorological Report (HMR) No. 52 methodology endorsed by the 10 CFR 50.54(f) letter, produced results which were above the current flood protection levels on site for an extended period of time. In response to this, two actions were performed. The first action, a site specific LIP calculation, was performed relying on modern day technology and methodology (instead of the HMR No. 52 methodology). The second action performed was the development of the following strategy for responding to potential flooding due to the LIP event.

A review of structures housing equipment important to maintaining safe shutdown was performed. Findings indicate two specific locations where preventive actions may be required to minimize flood water ingress.

- The fuel Oil Pump House doors are outward swinging and in good condition. However, the room is relatively small and water storage capacity available prior to potential flooding impacts is minimal for the 1'- 3" predicted peak water height.
- The Decontamination Building east side roll up door and 36" personnel door (outward swinging) are 4'-0" below average site grade, thereby potentially exposing these openings to 5'-0" peak flood water height. The rooms inside this area do not contain installed equipment relied upon for safe shutdown. However, they include pipe chases that run directly to the Auxiliary and Fuel Building lower elevations.

Other doors are in good condition and swing out to provide the most support for water pressure against the door jamb. The peak flood heights reduce significantly in one hour with a considerably lower height for an 8 hour or longer period. With the exception of those two areas discussed above, the buildings with doorways exposed to flooding conditions have large storage capacity. Therefore, damage to installed equipment relied upon for safe shutdown is not expected for the limited amount of water that could potentially leak through doors provided they are verified to be properly closed and secured. The analysis performed during the integrated assessment will evaluate plant equipment impact and the strategy for responding to the flooding hazard.

Additionally, a preliminary evaluation, as discussed below, has been conducted to assess additional strategies available to respond to the potential of external flood waters that may bypass existing protection features.

The Turbine Building (TB) basement has installed sump pumps with capacities of approximately 8000 gpm. Should flood water ingress exceed the capacity of the sump pumps, installed level switches will trigger alarms to direct operations to enter abnormal procedures for TB Flooding and direct actions to place both units in a shutdown condition. If a BDB event causes flood water ingress which exceeds the capacity of the pumps and compromises the installed Emergency Switchgear Room flood protection dike, vital power supporting control room instrumentation may be disabled. Additional response procedures direct operations to monitor key plant parameters from the Remote Monitoring Panel (RMP) located on the 35' elevation of the plant in the Cable Spreading Room. The RMP panel can be powered from a UPS (to be installed during the 2015 spring refueling outage) or a portable generator in the event station emergency power is lost. Key parameter indications can be communicated to support execution of ECA 0.0 and FLEX Support Guidelines (FSGs) to maintain core cooling and spent fuel pool cooling for an extended period of time.

Additionally, the Intake Canal level is monitored and procedurally controlled to maintain the elevation of water within operational limits during all modes of operation and shut down conditions. Based on the LIP evaluation, the intake canal level can rise above the normal operating upper limit of 30 ft MSL without operator action. In order to respond to the high canal level alarm received in the control room, the operators will initiate procedures to begin reducing the level back to within the normal operational limits. However, to have the capability to relieve the precipitation and watershed from the LIP event, the proper plant equipment will need to be available. Therefore, further enhancements to station administrative and work control procedures will be required to ensure the minimum quantity of circulating water, service water, and bearing cooling system components are in service for plant operational or shut down conditions to discharge the appropriate volumetric flowrate out of the Intake Canal during a LIP event.

Based on these findings, the interim action will be to review, revise, and include necessary steps to enhance the applicable station abnormal weather, operational, and work management procedures for mitigation of a BDB potential flooding event due to a local intense precipitation (LIP) event. The procedure updates will consider actions such as verification of critical exterior doors being closed, installation of temporary flood barriers at specific locations as required, maintaining necessary equipment in service, and making use of additional supplemental equipment and procedures for flood water mitigation and removal. Further, procedures will be updated with a clear entry condition (trigger event) to initiate required actions.

Additional actions due to the LIP event will be addressed in the Integrated Assessment.

4.2. Combined Effects Flooding

The Combined Effects Flooding considered two different approaches to storm surge (probabilistic and deterministic analysis). The basis for this section of the Flood Hazard Reevaluation Report will rely on the probabilistic analysis approach only. The results of this analysis created stillwater elevations, which are below the stillwater elevations in the CLB, however they produce wave run-up on the intake embankment, and the Intake Structure, which are above the CLB. This run-up is calculated to exceed the height of the intake canal embankment, but does not overtop the Emergency Service Water Pump (ESWP) intake structure roof, where the alternate air intake motor-operated dampers and the exhaust outlet for the diesel engine are located.

The Intake embankment and ESPH structure will be evaluated in the Integrated Assessment to address the wave run-up and new loadings due to recalculated hydrostatic loading, hydrodynamic loading, debris impact, and wave impact. The new loading is qualitatively considered acceptable but will need to be further addressed in the Integrated Assessment. Beyond the identified Integrated Assessment items and the current protection features considered, there are no additional concerns with the flooding levels due to combined effects.

4.3. Dam/Site Impoundment Failures

An evaluation was performed for the Surry site in response to upstream dam failures which could contribute to a flooding condition along the James River. The analysis concluded the maximum elevation of the flood water would be well below the average site grade. Site impoundments were also evaluated based on Regulatory requirements, for potential plant flooding hazards. The evaluation performed for the Intake Canal concluded a Sunny Day Failure is the only potential failure mechanism that could cause flooding above site grade.

The flood levels created by this event are bounded by LIP flood levels. The difference with the Sunny Day Failure of the intake canal is that there is little warning time between initiation of a typical dam failure and a resultant flood. The Sunny-Day breach scenario was performed assuming normal operation conditions, and a conservative water level at the upper limit of the control band (30 ft MSL). The level is procedurally controlled and monitored by operators. A control room annunciator is activated when the level is outside the procedural band. With the canal at 30 ft MSL, a head differential of 3.5 ft between the canal and the site grade would exist.

If a breach were to occur through the concrete lining of the Earthen Embankment, given the seepage path distance, a relatively low gradient, and the low permeability of the Earthen Embankment, seepage would take a significant amount of time to occur and the seepage velocities would be low. This breach would be relatively slow to develop and quickly identified through the use of various resources, such as: Operator rounds, manned security towers, security cameras, etc. Immediate Operator action would be taken to lower the canal level to below site grade (normal canal levels vary between 26 and 30 ft MSL, according to the UFSAR). Therefore flooding due to a sudden failure of the Intake Canal under normal conditions is not likely.

The Interim Action will be to review, revise, and include necessary steps to enhance Intake Canal Impoundment Surveillance and Operations Procedures for a Sunny Day Breach scenario (including identification of a breach, and communicating this information to the control room in order to initiate operator actions to mitigate the flood hazard).

4.4. All Other Flood Causing Mechanisms

Probable Maximum Flood in Rivers/Streams induced by Probable Maximum Precipitation, Ice Induced Flooding, Tsunami, Seiche, and Channel Migration/Diversion evaluations all produced results that are either below current design basis, do not challenge existing flood protection features, or are not a threat to generate a new flooding condition for Surry Power Station.

Therefore, no further evaluation or interim actions are required for these flood-causing mechanisms.

4.5. Conclusion

Based on the scenarios discussed in Sections 4.1 and 4.2, an Integrated Assessment will be performed that addresses any concerns from the LIP and Combined Effects events. Section 4.3 will be resolved as an interim action with improvements to station procedures currently in place. This and other identified interim actions will provide flood protection until the Integrated Assessment can be performed. All interim actions will be entered into the Dominion corrective action program.

5.0 ADDITIONAL ACTIONS

There are no additional actions beyond those discussed in Section 4.0. During the development of the Integrated Assessment, additional actions may be required, which will be developed and addressed in the Integrated Assessment, or will be identified in a Condition Report and addressed appropriately.

Appendix A

FLO-2D Technical Description

FLO-2D TECHNICAL DESCRIPTION

1. Model Description

The FLO-2D Pro Model, Build No. 14.03.07 (FLO-2D) computer program was developed by FLO-2D Software, Inc., Nutrioso, Arizona. FLO-2D is a combined two-dimensional hydrologic and hydraulic model that is designed to simulate river and overbank flows as well as unconfined flows over complex topography and variable roughness, split channel flows, mud/debris flows and urban flooding. FLO-2D is a physical process model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces using the dynamic wave approximation to the momentum equation. FLO-2D moves flood volume on a series of tiles (grid) for overland flow or through stream segments for channel routing.

Application of the model requires knowledge of the site, the watershed (and coastal, as appropriate) setting, goals of the study, and engineering judgment.

2. Model Components

FLO-2D has components to simulate overland flow, channel/riverine flow including flow through culverts, flow exchange between a channel and the floodplain, buildings and obstructions, rainfall-runoff and levees. The model also has components to simulate street flow, spatially variable rainfall and infiltration, evaporation, sediment transport, and levee and dam breach failures.

Overland Flow Simulation

This FLO-2D component simulates overland flow and computes flow depth, velocities, impact forces, static pressure and specific energy for each grid. Predicted flow depth and velocity between grid elements represent average hydraulic flow conditions computed for a small time step. For unconfined overland flow, FLO-2D applies the equations of motion to compute the average flow velocity across a grid element (cell) boundary. Each cell is defined by 8 sides representing the eight potential flow directions (the four compass directions and the four diagonal directions). The discharge sharing between cells is based on sides or boundaries in the eight directions. At runtime, the model sets up an array of side connections that are only accessed once during a time step. The surface storage area or flow path can be modified for obstructions including buildings and levees. Rainfall and infiltration losses can add or subtract from the flow volume on the floodplain surface.

Channel Flow Simulation

This component simulates channel flow in one-dimension. The channel is represented by natural, rectangular or trapezoidal cross sections. Discharge between channel grid elements are defined by average flow hydraulics of velocity and depth. Flow transition between subcritical and supercritical flow is based on the average conditions between two channel elements. River channel flow is routed with the dynamic wave approximation to the momentum equation. Channel connections can be simulated by assigning channel confluence elements.

Channel - floodplain Interface

This FLO-2D component exchanges channel flow with the floodplain grid elements in a separate routine after the channel, street and floodplain flow subroutines have been completed. An overbank discharge is computed when the channel conveyance capacity is exceeded. The channel-floodplain flow exchange is limited by the available exchange volume in the channel or by the available storage volume on the floodplain. Flow exchange between streets and floodplain are also computed during this subroutine. The diffusive wave equation is used to compute the velocity of either the outflow from the channel or the return flow to the channel.

Floodplain Surface Storage Area Modification and Flow Obstruction

This FLO-2D component enhances detail by enabling the simulation of flow problems associated with flow obstructions or loss of flood storage. This is achieved by the application of coefficients (Area reduction factors (ARFs) and width reduction factors (WRFs)) that modify the individual grid element surface area storage and flow width. ARFs can be used to reduce the flood volume storage on grid elements due to buildings or topography and WRFs can be assigned to any of the eight flow directions in a grid element to partially or completely obstruct flow paths in all eight directions simulating floodwalls, buildings or berms. Floodplain modifications due to buildings and/or storage basins can also be achieved by manually modifying grid element elevations.

Rainfall – Runoff Simulation

Rainfall can be simulated in FLO-2D. The storm rainfall is discretized as a cumulative percent of the total. This discretization of the storm hyetograph is established through local rainfall data or through regional drainage criteria that defines storm duration, intensity and distribution. Rain is added in the model using an S-curve to define the percent depth over time. The rainfall is uniformly distributed over the grid system and once a certain depth requirement (0.01-0.05 ft) is met, the model begins to route flow.

Hydraulic Structures and Storm Drains

Hydraulic structures including bridges and culverts and storm drains may be simulated in FLO-2D using the hydraulic structures component. Discharge through round and rectangular culverts with potential for inlet and outlet control can be computed using equations based on experimental and theoretical results from the U.S. Department of Transportation procedures (Hydraulic Design of Highway Culverts; Publication Number FHWA-NHI-01-020 revised May, 2005). The equations include options for box and pipe culverts and take into account different entrance types for box culverts (wingwall flare 30 to 70 degrees, wingwall flare 90 or 15 degrees and wingwall flare 0 degrees) and three entrance types for pipe culverts (square headwall, socket end with headwall and socket end projecting).

Storm drains are modeled using the EPA SWMM Model. FLO-2D is linked to the EPA SWMM Model at runtime to exchange surface water and storm drain conveyance. FLO-2D computes the surface water depth at grid elements prescribed with storm drains and computes the discharge inflow to the storm drain based on input storm drain geometry. The EPA SWMM model then computes the pipe network flow distribution and potential return flow to the surface water.

Levees

This FLO-2D component confines flow on the floodplain surface by blocking one of the eight flow directions. A levee crest elevation can be assigned for each of the eight flow directions in a given grid element. The model predicts levee overtopping. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad-crested weir flow equation with a 3.1 coefficient. Weir flow occurs until the tailwater depth is 85% if the headwater depth. At higher flows, the water is exchanged across the levees using the difference in water surface elevations.

3. Governing Equations

The general constitutive fluid equations include the continuity equation, and the equation of motion (dynamic wave momentum equation):

$$\frac{\partial h}{\partial t} + \frac{\partial h V}{\partial x} = i$$

$$S_f = S_o - \frac{\partial h}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t}$$

where h is the flow depth and V is the depth-averaged velocity in one of the eight flow directions x. The excess rainfall intensity (i) may be nonzero on the flow surface. The friction slope component S_f is based on Manning's equation. The other terms include the bed slope (S_o), pressure gradient and convective and local acceleration terms.

The equations of motion in FLO-2D are applied by computing the average flow velocity across a grid element boundary one direction at time. There are eight potential flow directions, the four compass directions (north, east, south and west) and the four diagonal directions (northeast, southeast, southwest and northwest). Each velocity computation is essentially one-dimensional in nature and is solved independently of the other seven directions. The stability of this explicit numerical scheme is based on strict criteria to control the magnitude of the variable computational timestep.

4. Model Implementation

4.1 Assumptions

The inherent assumptions in a FLO-2D simulation are as follows:

- Grid element is represented by a single elevation, n-value, flow depth
- Steady flow for the duration of the timestep
- Hydrostatic pressure distribution
- 1-dimensional channel flow (no secondary currents, no vertical velocity distributions)
- Rapidly varying flow such as hydraulic jumps or shock waves are smoothed out in model calculations. Subcritical and supercritical flow transitions are assimilated into the average hydraulic conditions between two grid elements.

4.2 Spatial and Temporal Discretization Schemes

The solution domain in the FLO-2D model is discretized into uniform, square grid elements. The differential form of the continuity and momentum equations in the FLO-

2D model is solved with a central, finite difference numerical scheme. This explicit algorithm solves the momentum equation for the flow velocity across the grid element boundary one element at a time.

4.3 Interpolation Methods

Grid element elevation data is based on imported digital terrain (DTM) points or elevation points that are added to the working region. Interpolation methods available in FLO-2D include:

- Using a user specified minimum number of closest DTM points within the vicinity of a grid element to compute the grid elevation;
- Using a user specified radius of interpolation which defines a circle around each grid element node to select DTM points for use in computing the grid element elevation; and
- Using an inverse distance weighting formula exponent to assign elevations to the grid element from the DTM points

4.4 Solution Procedures and Convergence Criteria

The solution algorithm incorporates the following steps:

1. The average flow geometry, roughness and slope between two grid elements are computed.

2. The flow depth dx for computing the velocity across a grid boundary for the next timestep (i+1) is estimated from the previous timestep i using a linear estimate (the average depth between two elements).

$$d_x^{i+1} = d_x^{i} + d_{x+1}^{i}$$

3. The first estimate of the velocity is computed using the diffusive wave equation. The only unknown variable in the diffusive wave equation is the velocity for overland, channel or street flow.

4. The predicted diffusive wave velocity for the current timestep is used as a seed in the Newton-Raphson solution to solve the full dynamic wave equation for the solution velocity. It should be noted that for hyperconcentrated sediment flows such as mud and debris flows, the velocity calculations include the additional viscous and yield stress terms.

5. The discharge Q across the boundary is computed by multiplying the velocity by the cross sectional flow area. For overland flow, the flow width is adjusted by the width reduction factors (WRFs).

6. The incremental discharge for the timestep across the eight boundaries (or upstream and downstream channel elements) are summed,

$$\Delta Q_x^{i+1} = Q_n + Q_e + Q_s + Q_w + Q_{ne} + Q_{se} + Q_{sw} + Q_{nw}$$

and the change in volume (net discharge x timestep) is distributed over the available storage area within the grid or channel element to determine an incremental increase in the flow depth.

$$\Delta d_x^{i+1} = \Delta Q_x^{i+1} \Delta t / A_{surf}$$

where ΔQ_x is the net change in discharge in the eight floodplain directions for the grid element for the timestep Δt between time i and i + 1.

7. The numerical stability criteria are then checked for the new grid element flow depth. If any of the stability criteria are exceeded, the simulation time is reset to the previous simulation time, the timestep increment is reduced, all the previous timestep computations are discarded and the velocity computations begin again.

8. The simulation progresses with increasing timesteps until the stability criteria are exceeded.

The convergence criteria for the solution in FLO-2D are ± 0.01 ft/s for velocity and ± 0.01 ft for depth.

4.5 Timestep Selection

FLO-2D has a variable timestep that varies depending on whether the numerical stability criteria are not exceeded or not. Timesteps generally range from 0.1 second to 30 seconds. The model starts with the a minimum timestep equal to 1 second and increases it until the numerical stability criteria exceeded, then the timestep is decreased. If the stability criteria continue to be exceeded, the timestep is decreased until a minimum timestep is reached. If the minimum timestep is not small enough to conserve volume or maintain numerical stability, then the minimum timestep can be reduced, the numerical stability coefficients can be adjusted or the input data can be modified. The timesteps are a function of the discharge flux for a given grid element and its size. Small grid elements with a steep rising hydrograph and large peak discharge require small timesteps. Accuracy is not compromised if small timesteps are used, but the computational time can be long if the grid system is large.

5 Input Data Requirements

The major design inputs to the FLO-2D computer model are:

- Digital terrain model of the land surface,

- inflow hydrograph and/or rainfall data,
- Manning's roughness coefficient and
- Soil hydrologic properties such as the SCS curve number.

The digital terrain model of the land surface is used in creating the elevation grid system over which flow is routed. The specific design inputs depend on the modeling purpose and the level of detail desired.

6 Output Details

FLO-2D model outputs include:

- Maximum flow depths at each grid element;
- Maximum velocity at each grid element;
- Maximum water surface elevation at each grid element;
- Time the peak water surface elevations and velocities occur;
- The discharge hydrograph overtopping a levee within a grid element;
- The discharge hydrograph through a hydraulic structure; and
- Maximum flow depth and water surface elevation in channel segments.

References

1. FLO-2D Software, Inc, 2014. FLO-2D® Pro Reference Manual, Nutrioso, Arizona, www.flo-2d.com
2. FLO-2D Software, Inc, 2011. FLO-2D Model Validation for Version 2009 and up prepared for FEMA, June 2011.

Appendix B

FLO-2D Results Plots

Note: Pages B2 through B8 are not numbered. In lieu of numbering, page numbers are identified above.

FIGURE B1 - CELL ELEVATIONS (FT, MSL/ PLANT DATUM)


FIGURE B2 - MAXIMUM FLOW DEPTH (FT) - CASES 1 AND 2
N
W
E
S
Legend
Flow Depth at Cell
0
1,000
2,000
4,000
Feet


N
S
E
W
FIGURE B3 - MAXIMUM WATER ELEVATIONS (FT, MSL/ PLANT DATUM) - CASES 1 AND 2
Legend
Water Elevation at Cell
0
1,000
2,000
4,000
Feet

FIGURE B4 - MAXIMUM CELL VELOCITIES (FT/S) - CASES 1 AND 2

FIGURE B5 - MAXIMUM CELL VELOCITY VECTORS - CASES 1 AND 2

FIGURE B6-SHEET 1 of 2 - GRID CELL NUMBERS

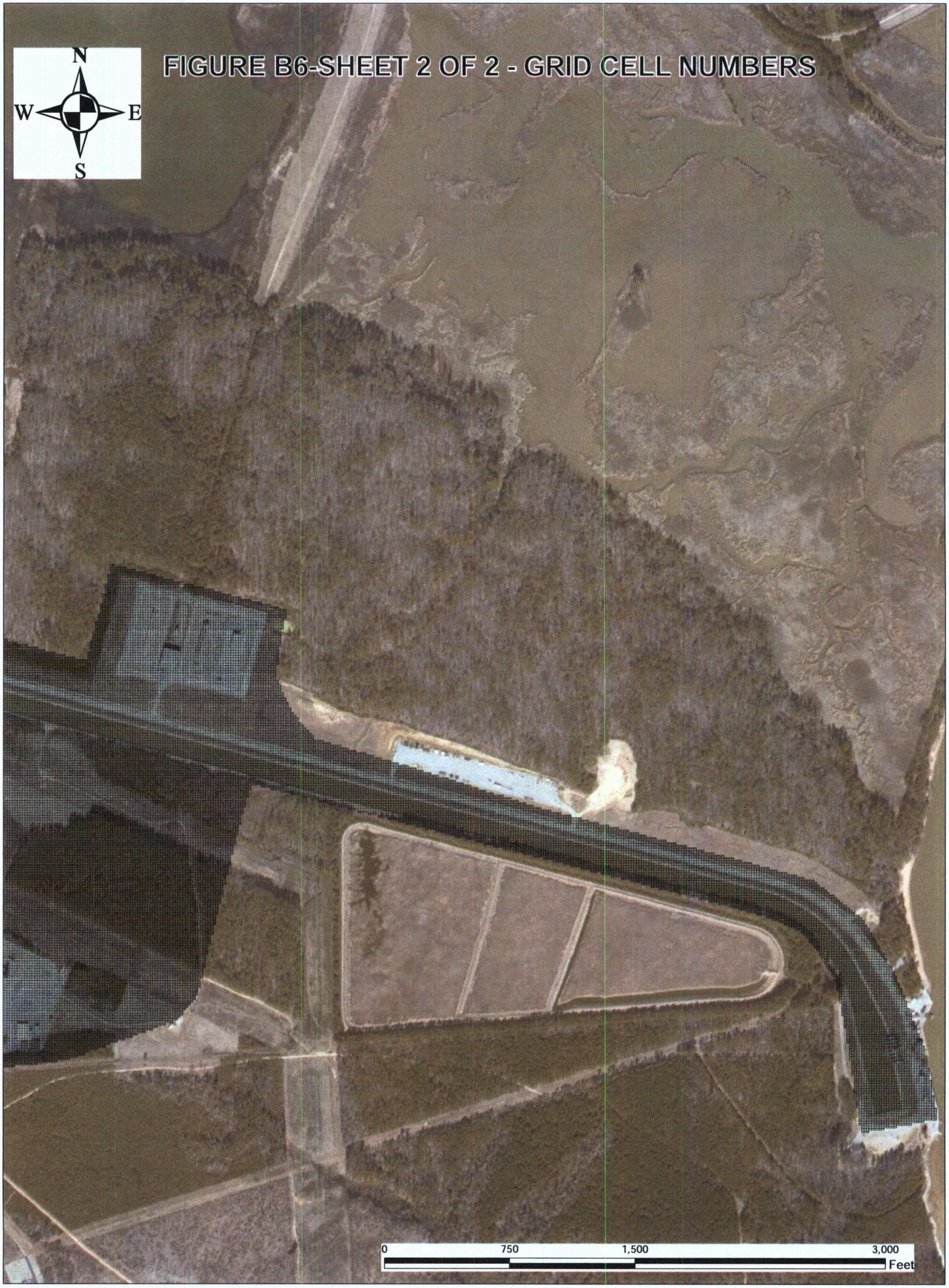


FIGURE B6-SHEET 2 OF 2 - GRID CELL NUMBERS
N
W
E
S
0
750
1,500
3,000
Feet

Appendix C

Third-Party Review

GZA
GeoEnvironmental, Inc.
Engineers and Scientists

December 8, 2014
File No. 01.0171382.22

Zachry Nuclear Engineering, Inc.
14 Lord's Hill Rd
Stonington, CT 06378

Attention: Mr. Michael Kerst
Project Manager

Re: Transmittal and Response to Third Party Review Comments
Dominion Nuclear Flood Hazard Re-Evaluation Project

249 Vanderbilt Avenue
Norwood, Massachusetts
02062
781-278-3700
FAX 781-278-5701
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Dear Mr. Kerst,

The purpose of this letter is to transmit and provide responses to the independent peer review of the External Flood Hazard Re-Evaluation hurricane and surge calculations for Surry Power Station (SPS) by Dr. Donald T. Resio (Attachment 2). A summary of Dr. Resio's experience and qualifications are provided in Attachment 1.

Dr. Resio performed a focused review of the following calculations, which represent elements of a step-wise assessment of the coastal flooding hazard at SPS:

- Calculation No. 14-028, Rev. 0 – Probable Maximum Hurricane for Surry Power Station
- Calculation No. 14-116, Rev. 0 – Deterministic Probable Maximum Storm Surge for Surry Power Station
- Calculation No. 14-117, Rev. 0 – Probabilistic Storm Surge for Surry Power Station

In addition the calculation documentation, Dr. Resio's review was informed by discussions with GZA during a series of teleconferences between May of 2014 and December of 2014. This review culminated in the opinion summary provided as Attachment 2. In general, Dr. Resio's comments and recommendations were considered by GZA prior to finalizing each calculation above. A summary of Dr. Resio's comments for each calculation and GZA discussion follows:

Calculation No. 14-028, Rev. 0 – "Probable Maximum Hurricane for Surry Power Station"

Overall, Dr. Resio concurred with the employed methodology and results associated with this calculation. Items highlighted by Dr. Resio's review judged by GZA to require additional discussion are as follows.

- On Page 2 of Attachment 2, Dr. Resio notes that it is difficult to validate the WRT synthetic data as being representative of extreme conditions. GZA agrees with this position, and points to the fact that available historical data do not characterize these extremes due to a paucity of data relative to the range of annual exceedance probabilities being considered. Expert meteorologists and climatologists were retained to support this calculation, and their review of these data highlighted general consistency with available historical data and a slight conservative bias with respect to storm intensity and general surge generation potential. Therefore, the synthetic WRT data are considered to be an effective tool for characterizing extreme hurricanes affecting the SPS vicinity.

- On Page 3 of Attachment 2, Dr. Resio comments on sensitivity of the GPD function to threshold selection. While GZA agrees that probability estimates derived from GPD fits can be sensitive to the selected threshold, it is important to note that the GPD function was not used to develop the 3M data set; therefore, sensitivity of the GPD fits to selected thresholds would not affect the scaling function used to calculate PMH intensities, nor would it affect maximum wind speed probabilities derived from the 3M data set itself. GPD functions were only used to evaluate error in the development of the data set extension (i.e., the 3M data set) through direct comparison to the synthetic WRT data.

Calculation No. 14-116, Rev. 0 – "Deterministic Probable Maximum Storm Surge for Surry Power Station"

Overall, Dr. Resio concurred with the employed methodology and results associated with this calculation. One item highlighted by Dr. Resio's review judged by GZA to require additional discussion follows.

- On Page 3 of Attachment 2, Dr. Resio comments on comparing SLOSH and ADCIRC to demonstrate consistency between the models. While absolute results may differ between the models due to model resolution and/or other contributing factors, similar parameter sensitivities are expected. This expectation is confirmed by the results of the Probabilistic Storm Surge calculation, which shows similar parameter-specific sensitivities between SLOSH and ADCIRC despite different absolute maximum stillwater elevation estimates.

Calculation No. 14-117, Rev. 0 – "Probabilistic Storm Surge for Surry Power Station"

Overall, Dr. Resio concurred with the employed methodology and results associated with this calculation. It is noted that Dr. Resio adjusted his comments related to utilizing Bayesian Quadrature to recognize the use of Response Surface methodology during a December 4, 2014 telephone conversation. Items highlighted by Dr. Resio's review judged by GZA to require additional discussion are as follows:

- On Page 4 of Attachment 2, Dr. Resio comments on demonstrating consistency in probability mass as parameter-specific probabilities transition to the surge-frequency response. GZA recognizes the desire to verify the recovery of all probability mass reflective of the probability level considered in this analysis (i.e., 1E-6 annual exceedance probability, or AEP, level). A comparison of the storm parameter definitions associated with this calculation and the univariate probability density functions presented in the PMH calculation shows that, while not all probability mass is directly recovered, mass associated with storm parameter responsible for extreme surge elevations has been completely represented. Probability mass that has not been considered is limited to more frequent, lower-risk level characteristics (e.g., maximum wind speeds below 70 knots and storms traveling east-of-north). Exclusion of this probability mass is analogous to excluding contributions to the surge-frequency relationship from extra-tropical events. With respect to storm parameter combinations with probabilities smaller than 1-in-

3,000,000, it is important to note that maximum wind speeds equal to or above bearing-specific PMH levels have been included in certain cases (i.e., to promote conservatism). As such, the 1-in-3,000,000 lower probability threshold is shown to be adequately conservative such that lower-probability storms would not contribute to the 1E-6 AEP level.

- On page 4 of Attachment 2, Dr. Resio comments on evaluating aleatory variability (i.e., note: Figure 59, which is specifically referenced in Dr. Resio's review, has changed to Figure 60 in the final version of the calculation): This method of characterization (i.e., via a linear functional fit, as opposed to a more complex functional fit) was necessary, as the FEMA tool employed to distribute uncertainty requires this simplification. As demonstrated by Figure 60, the linear fit, which is necessitated by the uncertainty adjustment formulations, is conservative for the majority of the wind speed range (i.e., over-estimates the maximum wind speed difference at the 95% confidence limit between 90 and approximately 138 knots).

In consideration of the attached review summary and the additional discussion presented above, GZA considers the peer review of Calculation No. 14-028, 14-116 and 14-117 to be complete.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Michael A. Mobile, Ph.D.
Originator

Daniel C. Stapleton, P.E.
Verifier

MAM/DCS:kr

Attachments
1. Summary of Experience and Qualifications, Donald T. Resio
2. Peer Review of Storm Surge Analysis at Surry Power Station in Virginia

Attachment 1. Summary of Experience and Qualifications, Donald T. Resio

Dr. Resio's credentials as a subject matter expert are summarized as follows:

Dr. Resio is currently a Professor of Ocean Engineering at the University of North Florida (UNF) and the Director of the Taylor Engineering Research Institute (TERI). A biographical sketch available on the NRC's website[1] states the following with respect to Dr. Resio's background as of 2010 (i.e., prior to taking his position at UNF): "Dr. Resio was appointed to the position of Senior Technologist (ST) in May 1994. This position represents the highest technical rank in the DoD civil service, with less than forty such positions authorized within the Army. Dr. Resio has been involved in performing and directing engineering and oceanographic research for over 30 years. He serves as the technical leader for the Coastal Military Engineering program and is the Technical Manager (TM) for a recent successfully completed Advanced Technology Concept Demonstration (ACTD) for military logistics. He also conducts/directs research that spans a wide range of environmental and engineering areas within the Corps Civil Works Program. In this capacity he directs the MORPHOS project aimed at improving the predictive state of the art for winds, waves, currents, surges, and coastal evolution due to storms. Most recently, Dr. Resio has been selected as the co-leader (with Professor Emeritus Robert Dean of the University of Florida) for the IPET Task 5a (analysis of wave and surge effects, overtopping and related forces on levees during Katrina) and as the leader of the Risk Analysis team for the South Louisiana Hurricane Protection Project, including consideration of the effects of climatic variability on hurricane characteristics in the Gulf of Mexico. Dr. Resio led the team that developed the new technical approach for hurricane risk assessment along US coastlines and is now leading an effort sponsored by the Nuclear Regulatory Agency to extend this approach to the estimation of hazards for Nuclear Power Plants in coastal areas. Recently, under the sponsorship of the Department of Homeland Security, Dr. Resio led a team of researchers in the development of innovative methods for the rapid repair of levee breaches. This work appears to offer new options for improved flood mitigation in many areas of the US."

[1] from information associated with the Regulatory Information Conference, 2010: *http://www.nrc.gov/public-involve/conference-symposia/ric/past/2010/bio/resiod.pdf*

UNF UNIVERSITY of NORTH FLORIDA

Attachment 2: Peer Review of Storm Surge Analysis at Surry Power Station in Virginia

Research Agreement #1309-001

October 30, 2014

Prepared for:
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Review of Zachry Nuclear, Inc.

Professor Donald T. Resio

University of North Florida

1. Introduction

This report presents a review of three documents pertaining to the estimation of water levels produced by the "controlling storm" at the Dominion/Surry Power Station in Virginia. The first report contains material which describes the theoretical and empirical basis for the definition of the controlling storm and its deterministic and probabilistic attributes. The second report provides a deterministic analysis of the Probable Maximum Storm Surge (PMSS) resulting from the combination of meteorological parameters generating the PMSS at the SPS. The third report provides a probabilistic analysis of storm surge for Surry Power Station (SPS) using state of the art numerical models combined with the probabilities of meteorological parameters developed in the first report. This analysis focuses on the very-low probability range of Annual Exceedance Probability (AEP) for still water at the SPS site.

2. Review of Report Entitled "Probable Maximum Hurricane for Surry Power Station "

This report documents the approach used in developing Probable Maximum Hurricane (PMH) parameters for Dominion/Surry Power Station (SPS) and the approach used to develop probabilistic representations of parameters to be used in Probable Maximum Storm Surge calculations and for probabilistic (JPM) calculations at this site.

2.1 Review of PMH Parameter Development

Step 1: Develop A Rationale for Selection of the Controlling Event for the PMH. Identify the controlling meteorological event. This involved a relatively straightforward analysis of tropical and extratropical storms in this areas and it was determined that, for the extreme range of low probability considered, hurricanes would be the dominant contributor to the maximum surge at this site. This is an easy case to make and should be readily accepted.

Step 2: Develop parameters Based on NWS 23 Report. Utilize NWS 23 (1979) to develop a set of meteorological parameters for the PMH in the area of the SPS. An initial review of parameters developed in the 1979 report (NWS 23) suggested that the storm characteristics for the PMH in this area as estimated in that study were quite intense and might not be representative of local conditions at the SPS, primarily due to the inclusion in NWS 23 of headings that do not produce maximum surges at the SPS.

One factor that could use some additional discussion in this section is the treatment of maximum wind speed as the defining factor for storm intensity instead of the more conventional (at least in terms of storm surge generation) pressure differential. A table or graph showing the relationship between the two (which might be a family of curves depending on latitude, storm size and forward speed) would be extremely helpful in understanding the transition from one parameter space to the other.

Step 3: Part 1 Development of Deterministic PMH Parameters. Most of the storm parameters were analyzed in a fashion that produced values very consistent with the NWS 23 valued. The one exception is the treatment of storm intensity. Motivated by the existence of a strong co-variation between storm heading directions and storm intensities a site-specific study was undertaken to examine storm behavior in this area in more detail. A set of synthetic storms was created by WindRisk Tech (WRT) using a well validated model developed by Emanuel et al. (2004). This set of storms was used to create a scaling function for storm intensity as a function of storm heading. The maximum of this directional function was set to be equal to the NSW 23 value for this area. Unfortunately, the manner in which this is written makes it sound like a probabilistic development of a maximum wind speed rather than a dimensionless scaling function which is used to allow natural variability of the NWS maximum wind speed with respect to storm heading direction. I recommend that this section be recast in terms of using the results from the WRT simulations to scale the maximum wind speeds for hurricanes approaching from different directions, rather than introducing any probabilistic terms into this analysis which might be misunderstood. Such a misunderstanding might then necessitate a discussion of probability levels, sources of uncertainty and other related non-deterministic aspects of this analysis. The WRT methodology is robust; however, it is difficult to argue that this method for generating synthetic storms is correct in an absolute sense for prediction of extremes, since the data for local comparison of such extremes is very sparse.

Step 3: Part 1 Development of Probabilistic PMH Parameter Framework. This section is straightforward in its development but the joint probability information could be displayed in a clearer fashion. An equation for p(x1,x2,x3,x4...) should be written with any jointly varying terms written as such and graphical diagrams or equations should be presented to demonstrate clearly the final probability distributions, cumulative distributions, and complementary distributions. Such information would really help reviewers if it were placed in the final summary section.

Two small points that might be considered for changing are as follows:

a. On Page 24, it is implied that information on central pressures is limited to the 1979-2012 time frame due to lack of data. Most hurricanes that passed close to the US east coast have central pressure data back into the 1950s or so. Perhaps the intent here is to make the analysis somewhat consistent in a climatological sense, due to changes in weather patterns, but this is not how the comment is posed.

b. The FEMA report for this area (from the USACE-Vickery study) does contain some information on storm sizes and should probably be referenced as a relevant source of data. The data there seem fairly consistent with the results presented in the WRT analyses.

Step 4: Development of Joint Probabilities for Hurricane. Once the synthetic storm set is developed and included within the methodology for estimating joint probabilities for the JPM approach, a careful analysis of univariate and multivariate probabilities is performed as part of this report. This section is very thorough in its treatment of these different terms. One question which might be asked relative to this work is the application of the GPD in estimating hurricane wind speeds. The GPD can be quite sensitive to the choice of the chosen threshold value. Many studies perform analyses using at least 3 different thresholds to investigate this potential source of variation. Since NRC reviewers are well aware of this potential issue, it would probably be a good idea to be proactive on this issue and perform these analyses before their review. Looking at the shape of the curve, I do not think that there will be a large sensitivity, but it should be quantified.

Summary of Review of Probable Maximum Hurricane for Surry Power Station

Overall, this is a very high-level analysis and is carefully performed. A few minor points as noted should be addressed, but I do not think any of the issues raised in this review will significantly affect the PMH parameter or probabilistic results. Some relevant points include the following:

1. The upper ranges of the rmax reach relatively large sizes for all heading angles, 28.4 – 41.7 nm.

2. The vmax values are developed to include a storm-heading dependence which is used to deterministically scale the NWS 23 values of windspeed, which seems reasonable.

3. Upper and lower bounds on forward speeds seems reasonable.

4. The range of storm bearings for surge simulation seems sufficiently broad to cover the entire ranged needed.

3. Review of Report 2 Entitled "Deterministic Probable Maximum Storm Surge for Surry Power Station"

This report presents the deterministic analysis of the Probable Maximum Storm Surge (PMSS) for Surry Power Station, including the combined effects of storm surge, antecedent water level, waves and river flood. It relies on report 1 for all estimates of all meteorological parameters associated with a set of hurricane parameters shown to be capable of producing the highest storm surges reasonably expected at this site.

The modeling approach seems straightforward and uses state of the art methods and models to perform all estimates. The SLOSH model was used as a screening tool to select a small set of storms for detailed simulation with the ADCIRC model. There is always the possibility of mismatched physics producing storms which are not ordered in the same sequence when using results from different models. The ADCIRC model is forced by a slightly different wind field formulation than that used in the SLOSH model, however, for low values of the Holland B parameter, the net differences in winds should be relatively small. Since the values used here (characteristic of this region) range from 1.08 to 1.37, this should be the case here. Thus the differences in the ordering seem to relatively small. It is recommended that the ADCIRC results be plotted against the SLOSH results at the sites of interest (SPS Discharge site and SPS Intake site) to make this point graphically.

Fifteen ADCIRC simulations were utilized to cover the range of parameter combinations found to produce the largest combined water levels at the Surry Power Station. Given that the maximum wind speeds are reasonably defined as a function of storm heading, this set of combinations appears to cover the range needed for this purpose. A plot of the parameters in Table 4 as a function of the heading along with the maximum conditions defined as a function of heading in Report 1 would help make the point that the simulated storms constitute a set that should provide a good estimate of the maximum surges.

4. Review of Report 3 Entitled "Probabilistic Storm Surges for Surry Power Station"

As in Report 2, the hydrodynamic models are state of the art and are executed in a straightforward manner, so there should be no problems with the results from these models.

This report describes the effort to produce a probabilistic analysis of storm surge (JPM study) for Surry Power Station, using a Bayesian Quadrature method typical of many FEMA applications today. In this approach, a joint probability of storm parameters is taken from Report 1; however, documentation of the joint probability density functions is lacking. Since the Bayesian Quadrature is used to define the probabilities of the 20 individual ADCIRC simulations, the individual probability masses defined for each of the storms needs to be shown somewhere in a table in order to enable a reviewer to validate the probability estimates. These masses are determined by a Monte Carlo method and some assumptions pertaining to the correlation lengths of different parameters. These correlation lengths should be clearly specified and information on all the probability masses should be included somewhere in the report, particularly since the description suggests that there might be some constraints on the event combinations. It is essential to be able to check that the complementary probabilities sum to one where appropriate. I tend to agree with the motivation to discretize the event count in defining the probabilities such that less than 1/3,000,000 is equal to zero, but it is more defensible in a probabilistic method to let these small values (even when a number of them are summed) actually shown to be negligible. In Section 6.2.6 (Identification of the OS Storm Set), paragraph 2 is not very clear. More information on the selection process and the application of the Surge-Stat program would be very helpful to reviewers.

The treatment of epistemic uncertainty is consistent with previous studies in this area. The treatment of aleatory uncertainty seems adequate and provides the magnitude of increase that seems typical for inclusion of this type of uncertainty. The variation of surge level with vmax is clear, as is the equation to parameterize it. However, the curve for the aleatory variation of surge elevation looks like it is not well fit with a linear equation. Since the curve extends beyond the region of primary contribution to the probabilities, it is recommended that Figure 59 be redone to focus on the region of primary contribution to the probabilities. It is very likely that this difference in aleatory fitting is not a problem due to the range of probabilities that are affected here, but this should be checked.

Donald T. Resio

Donald T. Resio, Ph.D.

ATTACHMENT 2

SURRY NTTF 2.1: FLOODING HAZARD RE-EVALUATION INTERIM ACTIONS PLAN

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

Item Number	Report Reference	Action Item List	Estimated Implementation Date
1	Section 4.1	Revise and implement applicable SPS abnormal weather procedures to address mitigation of a potential Beyond Design Basis flooding event of structures, systems and components within the power block due to a local intense precipitation (LIP) event.	August 31, 2015
2	Section 4.1	Evaluate the need for temporary or permanent barriers to prevent potential flood water infiltration into the Fuel Oil Pump House and the Decon Building Roll-up door and personnel door.	September 30, 2015
3	Section 4.1	Revise and implement applicable SPS procedures to address mitigation of a Beyond Design Basis potential flooding event associated with the intake canal due to a local intense precipitation (LIP) event.	August 31, 2015
4	Section 4.3	Revise and implement applicable SPS procedures to prevent a Beyond Design Basis potential flooding event due to a breach of the intake canal.	August 31, 2015
5	Section 4.5	Perform an integrated assessment of the flood hazards for SPS Units 1 and 2.	February 28, 2017 (May change based on guidance from the NRC)