## 2.7. Ice-Induced Flooding

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

### 2.7.1. Method

The criteria for ice-induced flooding is provided in NUREG/CR-7046, Appendix G (NRC 2011). Two ice-induced events may lead to flooding at the site and are recommended and 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-induced flooding at MPS, the HHA used the following steps:

1. Review historical ice events and information on backwater effects due to ice jams in the Niantic River near MPS.
2. Evaluate historical salinity levels of the Niantic River and Long Island Sound to assess the feasibility of the formation of ice jams in the Niantic River near MPS.
3. Calculate flood elevations which could result at MPS from potential ice jams upstream or downstream in the Niantic River.

### 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. There are no records of historical ice jams on the Niantic River or on the Long Island Sound in the USACE Ice Jam Database.

2.7.2.2. Salinity of Water in Niantic River at Millstone

The mean salinity of surface water in the Niantic River near MPS ranges from approximately 27.4 to 30.3 psu (practical salinity unit), based on the data retrieved from the Long Island Sound Resource Center (University of Connecticut and the Connecticut DEP, 2004). According to the National Snow and Ice Data Center (NSIDC, 2013), a psu is nearly equivalent to a ppt (parts per thousand). Salinity in water has the potential to reduce 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).

Although the salinity in water reduces the freezing point to be lower than 32 °F (0 °C), and reduces the likelihood of ice jams near Millstone, the potential for ice jam formation on the Niantic River was conservatively not disregarded based on possible extended period of time of low air temperature in the region.

2.7.2.3. Flood Elevation due to Ice Jam

MPS is located along the shore of the Niantic Bay in the Long Island Sound, which does not contain structures downstream of the site where an ice jam can occur. Therefore, the potential for flooding to occur at MPS as a result of a downstream ice jam is not significant.

The closest structure upstream of MPS is the Amtrak Niantic River Bridge, which is located approximately 0.8 miles upstream (see Figure 2.7-1). The maximum water depth above normal river elevation resulting from an ice jam at the Amtrak Niantic River Bridge was conservatively calculated to be equal to the bridge clearance of 16 feet (Amtrak, 2013). It is assumed that the ice dam instantaneously fails and the resulting flood wave was conservatively translated directly from the Amtrak Niantic River Bridge to the vicinity of MPS without consideration of flood wave attenuation within the Niantic Bay. It is assumed that the ice dam fails and the peak flow resulting from the flood wave was conservatively translated directly from the Amtrak Niantic River Bridge to the vicinity of MPS without consideration of flood wave attenuation (i.e. decrease of discharge) within the Niantic Bay. The resultant peak flow from the ice dam failure was calculated using two empirical dam breach peak flow equations that use metric units (i.e., m, $m^3/s$) as follows:

Bureau of Reclamation: $Q_p = 19.1\ (h_w)^{1.85}$ Eq-1 (Wahl, 2004)

Kirkpatrick: $Q_p = 1.268\ (h_w + 0.3)^{2.5}$ Eq-2 (Wahl, 2004)

Where:

$Q_p$ = Dam breach peak flow;
$h_w$ = Head water.

The dam breach peak flow ($Q_p$) was calculated using the bridge clearance of 16 feet. The results showed that the peak dam breach flow using the Bureau of Reclamation resulted in a greater peak flow than the Kirkpatrick equation. Therefore, the peak flow of 12,645 cfs was selected as the ice dam breach peak flow calculated for the ice dam failure at the Amtrak Niantic River Bridge.

The resultant rise in water level at Millstone was calculated using the Manning Formula (Chow, 1959) for channel depth calculation:

$$Q = A\frac{1.49}{n}R^{2/3}S^{1/2}$$

Where:

A = cross section area (square-feet);
n = Manning's n roughness;
R = hydraulic radius (the cross sectional area of flow divided by the wetted perimeter);
S = Slope of energy line

A rectangle channel with a bottom width equal to the Niantic Bay width at water surface elevation zero NAVD88 in the vicinity of Millstone and vertical walls at both sides of the channel (Figure 2.7-1) was assumed to calculate the normal depth using the Manning Formula. Therefore, the floodplain slopes and the Niantic Bay bathymetry (i.e., flow area below water surface elevation zero NAVD88) was conservatively ignored in the normal depth calculation. The resulting rise in water level at MPS was conservatively estimated to be 2.9 feet, which is well below site grade at MPS (see Section 3).

This estimate is conservative because attenuation of the flood wave is not considered. Niantic Bay, located immediately downstream of the Amtrak Niantic River Bridge, significantly increases in area near MPS, which would result in significant attenuation of the flood wave. This would be anticipated to greatly reduce the size of the flood wave due to the failure of an upstream ice jam before it reached MPS.

Note that the Amtrak Niantic River Bridge is a movable structure, which can be raised in the event of an ice jam formation. As a result, ice jams can be released by raising the bridge structure.

### 2.7.3. Conclusions

The USACE Ice Jam Database (USACE, 2012) does not include records of ice jams occurring on the Niantic River. MPS's location at the downstream-most end of the Niantic Bay creates conditions which are unlikely to sustain a downstream ice dam due to both water salinity and channel morphology. Therefore, the potential for flooding to occur at MPS as a result of a downstream ice jam is not significant.

The failure of a conservatively-estimated hypothetical upstream ice jam would not exceed the protected elevation at MPS (see Section 3).

### 2.7.4. References

**2.7.4-1 Amtrak, 2013**. Amtrak - Niantic River Bridge, ARRA Project Overview (http://www.amtrak.com/ccurl/377/521/Niantic-River-Bridge-Replacement-Fact-Sheet.pdf – web page printed on 3/6/13).

**2.7.4-2 Chow, 1959.** Ven Te Chow, "Open-Channel Hydraulics," reprint of the 1959 Edition, McGraw Book Company, Inc, 1959.

**2.7.4-3 NOAA, 2013**. "JetStream – Online School for Weather – Sea Water," National Climatic and Oceanic Administration (http://www.srh.noaa.gov/jetstream/ocean/seawater.htm – web page printed on 1-19-2013).

**2.7.4-4 NRC 2011**. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011.

**2.7.4-5 NSIDC, 2013**. National Snow and Ice Data Center (NSIDC). Salinity and Brine (http://nsidc.org/cryosphere/seaice/characteristics/brine_salinity.html - web page printed 3/7/13).

**2.7.4-6 University of Connecticut and the Connecticut DEP, 2004**. Long Island Sound Resource Center, prepared by the University of Connecticut and the Connecticut Department of Environmental and Protection (DEP) with the support of researchers and organizations throughout the Long Island Sound watershed. Niantic River 2004 (http://www.lisrc.uconn.edu/eelgrass/LocationData.html - web page printed on 3/6/13).

**2.7.4-7 USACE, 2012**. Ice Jam Database, U.S. Army Corps of Engineers, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, 2012.

**2.7.4-8 Wahl, 2004.** Tony L. Wahl, "Uncertainty of Predictions of Embankment Dam Breach Parameters," Journal of Hydraulic Engineering ASCE, May 2004.

**Figure 2.7-1: Location of First Structure Upstream of MPS: Amtrak Niantic River Bridge**

## 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 MPS. 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, the HHA used the following two steps:

1. Review historical records and hydrogeomorphological data to assess whether the Niantic River has exhibited the tendency to meander towards the site.
2. Evaluate the foundation type at critical structures and shoreline protection features to assess potential susceptibility to erosion caused by possible channel migration.

### 2.8.2. Results

#### 2.8.2.1. Review of Historical Records

A literature review did not yield evidence suggesting there have been significant historical diversions of the Niantic River near Millstone or the small unnamed coastal stream east of Millstone for more than 50 years. A comparison of a 1958 USGS Topographic map (USGS, 1958) and a 2012 USGS Topographic map (USGS, 2012) illustrates continuity of the river course for more than 50 years, see Figures 2.8-1 and 2.8-2. Note that a former quarry located south of Millstone shown in the 1958 USGS Topographic map (Figure 2.8-1) has been decommissioned (Figure 2.8-2). The area of the former quarry has been flooded to be the plant cooling water discharge area.

Millstone is located at the mouth of the Niantic Bay where the bay opening is approximately 2.1 miles wide. NUREG/CR-7046 (NRC, 2011) includes the following statement in Section 3.8-Flooding Resulting from Channel Migration or Diversion:

> *Because most channel diversion occurs during high flows when the stream or river overflow its banks, flood data, particularly stage, may also prove useful in the determination.*

The Niantic River watershed is approximately 31 square-miles. High flows in the Niantic River dissipate quickly in the Long Island Sound; therefore, high velocity overflows of the banks of the river near Millstone that could result in channel diversion or severe erosion are not anticipated. As a result, channel diversion is not expected to occur near Millstone due to high riverine flows in the Niantic River. The small coastal stream near Millstone is not expected to produce high flows that

could result in channel diversion toward Millstone due to its limited drainage area of 87 acres (see Section 2.2).

2.8.2.2. Review of foundation Types and Susceptibility to Erosion

The foundations for a majority of the critical structures including the reactor containment are on bedrock. However, the emergency generator, waste disposal enclosure, and turbine building, are founded on dense basal till which overlies rock. The control building is founded on structural backfill overlying till and bedrock. Generally, bedrock is highest on the eastern portion of the site and dips to the west towards Long Island Sound. MPS is located on till that contains silt, sand, and stony fill; artificial fill, and bedrock. Subsurface explorations included in the MPS-3 FSAR (Dominion, 2014) generally show that till, when present, ranges from depths of 0 to 20 feet and rests on top of bedrock.

The soils and rock underlying the site are strong, stable materials that are not susceptible to loss of strength, subsidence, or other instabilities during earthquake motion. The soil and bedrock at Millstone are of very low permeability (Dominion, 2014).

### 2.8.3. Conclusions

A review of historical data indicates that the Niantic River has not exhibited a tendency to meander towards the site. High flows in the Niantic River dissipate quickly in the Long Island Sound, therefore, high velocity overflows of the banks of the river near Millstone that could result in channel diversion or severe erosion are not anticipated. In addition, most of the site's critical structures are founded on bedrock or structural backfill overlain on bedrock, and the shoreline is protected by a robust riprap revetment. Given these conditions, channel migration as a result of riverine flooding is not considered to be a potential contributor to flooding at Millstone.

### 2.8.4. References

**2.8.4-1 Dominion, 2014**. Millstone Power Station Final Safety Analysis Report (MPS-3 FSAR), Rev. 25.2.

**2.8.4-2 USGS, 1958.** U.S. Geological Survey (USGS), Niantic Quadrangle 7.5-Minute Series Historical Topographic Map, revised on 1958 and topography surveyed in 1934.

**2.8.4-3 USGS, 2012.** USGS, Niantic Quadrangle 7.5-Minute Series Topographic Map, contours based on National Elevation Dataset, 2012.

**2.8.4-4 NRC, 2011.** "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011.

## Figure 2.8-1: 1958 Historical Topographic Map

## Figure 2.8-2: 2012 Current Topographic Map

## 2.9. Combined Effect Floods

An evaluation of the combined external flood effects associated with coastal flooding at MPS was performed. The combined flood effects were evaluated for both deterministic (Probable Maximum Storm Surge) and probabilistic flood analyses (associated with a flood annual exceedance probability of 1E-6).

### 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 at MPS. 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 MPS.

#### 2.9.1.1. Deterministic Combined Effect Flood

MPS is subject to coastal flood hazards including storm surge, wind-generated waves and tsunamis. The coupled ADCIRC + SWAN model was used to simulate storm surge and waves due to the deterministic PMSS and the probabilistic storm surge. The following approach to deterministically combining external flood hazards was used consistent with NUREG/CR-7046:

H.1 – Floods Caused by Precipitation Events

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

- Alternative 1 - 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;
- Alternative 2 - 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
- Alternative 3 - 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 PMF was calculated for the small intermittent stream located approximately 200 feet east of the Independent Spent Fuel Storage Installation (ISFSI) (Section 2.2). The small intermittent stream's watershed is 87 acres (about 0.14 square miles) and terminates at the Millstone Road embankment without a visible outlet to Long Island Sound. The resulting PMF elevation on the small coastal stream (due in large part to the access road embankment obstruction) near Millstone is 11.2 feet (Section 2.2), which is approximately 12.8 feet below the MPS3 site grade of 24.0 feet (Dominion, 2014a) and approximately 2.8 feet below the MPS2 site grade of 14.0 feet (Dominion, 2014b). Significant wave propagation toward MPS is not expected due to the distance and the land use and land cover (e.g., woods and security barriers) between MPS and the small coastal stream and the available vertical margin or freeboard between MPS site grade and the PMF flood elevation. Additionally, flood velocities are anticipated to be very low because flooding is impounded by the Millstone Road embankment, limiting the potential for scour and erosion. Therefore, combined effect floods caused by precipitation events (NUREG/CR-7046, Appendix H, Section H.1) are not considered further.

The results of the dam failure section (Section 2.3) indicate that the potential for flooding at MPS resulting from upstream dam failure is not applicable based on the lack of dams within the Millstone local drainage basin and the watershed contributing to the small coastal stream.

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

Due to the shore-side location of MPS, criteria for shore-side location combined events were evaluated (Appendix H, Section H.3.1):

- Probable maximum surge and seiche with wind-wave activity;
- Antecedent 10 percent exceedance high tide.

The antecedent 10 percent exceedance high tide elevation includes the calculated sea level anomaly and the expected sea level rise in accordance with ANS 2.8 (ANS, 1992). Sea level anomaly and the expected sea level rise were calculated in the Deterministic PMSS section (Section 2.4).

Wave action at MPS was calculated using SWAN, which is a component of the coupled ADCIRC+SWAN model. Figure 2.9-6 provides the model bathymetric and topographic contours used to model the combined storm surge and wind-wave activity. Both the MPS2 and MPS3 intakes were identified as safety-related structures pertinent to the calculation of wave effects during combined event scenarios, as well as the MPS2 and MPS3 turbine buildings. Due to the structure geometry of the intakes, as shown in Figure 2.9-1 and Figure 2.9-2, respectively, wave effects will be applied against vertical walls.

Wave effects include inundation (resulting in hydrostatic and hydrodynamic loads) and wave breaking (resulting in slash and spray and wave loads). Unbroken waves in front of a vertical structure located in relatively deep water result in an elevated, reflected standing wave (Goda, 2010). This is due to reflection and transformation of non-breaking waves at a vertical face, where there can be an upward flow (vertical shift, referred to here as runup) that can be higher than the height of the unbroken wave. Reflected wave crest elevations due to non-breaking waves were calculated using the Sainflou Formula[1] as presented in the USACE Coastal Engineering Manual (CEM) (USACE, 2006) for predicting wave forces on a vertical structure.

The calculated "standing" (also referred to as "reflected") wave crest height is added to the PMSS stillwater elevation, to calculate the elevation of the standing wave crests at both intake structures and turbine buildings.

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 MPS since the site is not located on an enclosed body of water.

[1] In the case of irregular waves, wave height, H, should be taken as the characteristic wave height (USACE, 2006). For the purposes of this evaluation, Hs, the significant wave height, is used.

H.5 - Floods Caused by Tsunamis

Combined flood effects associated with tsunamis are included as part of the analyses required by NUREG/CR-7046 (Appendix H, Section H.5). Considering MPS's hydrologic setting (i.e., the small stream has a small watershed, no outlet to Long Island Sound and no hydraulic route for upstream tsunami propagation exists) and based on the results of the PMF and Dam Failure, combinations of tsunami and small stream flooding are insignificant. Therefore, the single combined effect flood alternative for a shore location was used:

- Alternative H.5.1 – Combination of Probable Maximum Tsunami (PMT) run-up and antecedent 10 percent exceedance high tide.

Evaluation of the potential for tsunamis at the MPS site concluded that the PMT results in lower flood elevations than the PMSS. PMT maximum flood elevations are locally as high as 14.7 feet near the MPS2 and MPS3 intake structures, including the 10 percent exceedance high tide. The PMT maximum flood elevation is a result of a far-field source (i.e., Cumbre Vieja subaerial landslide).

Although the PMT run-up elevation is bounded (i.e., less than) by the storm surge stillwater elevation, tsunamis may be associated with high velocity flow. Therefore, hydrodynamic and hydrostatic loading due to the PMT is evaluated in this section.

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 probabilistically-determined storm surge corresponding to the annual exceedance probability (AEP) of 1E-6. The combined effects for the probabilistic analyses were assumed to be consistent with NUREG/CR-7046 (NRC, 2011) and ANS 2.8 (ANS, 1992) for a shore location, including:

- Storm surge corresponding to the to the AEP of 1E-6 (Section 2.4);
- Coincident wind-wave activity.

While NUREG/CR-7046 (NRC, 2011) does not contain specific guidance for probabilistic inputs to combined effect scenarios, ANSI/ANS 2.8 infers that the acceptable average exceedance probability for combined effect flooding should be on the order of 1E-6 for design basis floods (ANSI, 1992). While the tidal component of the probabilistic surge does not include the 10-percent exceedance high tide (as used as input for the deterministic surge), the combination of the exceedance probability of the tidal condition used with the probabilistic storm surge parameters, equals an exceedance probability of 1E-6. Using a probabilistic input to the combined effect scenario with an exceedance probability equal to the exceedance probability of the combined effect flooding, is therefore considered conservative.

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 include increases in depth that may occur as a result of erosion and scour.

Hydrostatic Loads

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.

Flow Velocity

Floodwater flow velocities include velocity components due to flooding 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 that 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 (FEMA, 2011) was used to calculate hydrodynamic and impact loads.

Hydrodynamic Loads

Water flowing around a building (or structure) imposes loads on the building. 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. Hydrodynamic loads calculated here used steady-state flow velocities consistent with FEMA guidance (FEMA, 2011; FEMA, 2012). 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). The maximum forces at the bottom of the intakes were calculated using the area of the uniform pressure distribution.

Hydrodynamic forces for low velocity flow (less than 10 feet per second) were analyzed as an equivalent hydrostatic force. Resultant force acts at a distance of H/2 above the ground.

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 impacted structure 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.

Debris impact loads at the water surface were calculated using the guidelines described in FEMA P-259 (FEMA, 2012) and by considering debris weight recommended in ASCE-7-10 (ASCE, 2010).

Per ASCE 7-10 (ASCE, 2010), in coastal areas debris weights may range from 1,000 to 2,000 pounds. A debris object weight of 1,000 pounds is a reasonable average for flood-borne debris (representing trees, logs and other large woody debris (ASCE, 2010). A debris weight of 2,000 pounds was conservatively used.

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.

The maximum breaking wave pressures and loads on vertical walls were calculated for structures at MPS (ASCE, 2010). The calculations apply 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 2.9-3 (FEMA, 2011).

2.9.1.1.1 Tsunami Loading

The results of the tsunami simulations indicate that the highest predicted runup elevations in the vicinity of the MPS site result from the subaerial landslide (extreme flank failure) of the CVV. The CVV results in maximum water levels of approximately 14.7 feet, MSL at MPS2 and MPS3 (see Section 2.6). Fluid density of tsunami flow is assumed to be 1.2 times the density of freshwater, 2.33 slugs per cubic foot, based on a sediment volume concentration of 10% in seawater (FEMA, 2008).

Hydrostatic Loads

The hydrostatic lateral forces (per linear foot of surface) were calculated at both the MPS2 and MPS3 intakes (FEMA, 2008). These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts.

Hydrodynamic Forces

Hydrodynamic forces were calculated at both the MPS2 and MPS3 intakes based on FEMA guidance (FEMA, 2008). Resultant hydrodynamic forces are applied at approximately the centroid of the wetted surface of the structure.

Impulsive Forces

Impulsive forces are caused by the leading edge of the surge water (i.e. tsunami wave) impacting a structure. The impulsive forces are conservatively estimated as 1.5 times the hydrodynamic force (FEMA, 2008).

Debris Impact Forces

The debris impact forces were calculated at both the MPS2 and MPS3 intakes based on FEMA guidance (FEMA, 2008).

Two types of waterborne debris were analyzed, a log and a 20-foot heavy shipping container. The effective stiffness and mass of a log is $2.4 \times 10^6$ newton per meter and 450 kilograms respectively (FEMA, 2008). The effective stiffness and mass of a 20-foot long heavy shipping container is $1.7 \times 10^9$ Newtons per meter and 2,400 kilograms respectively (FEMA, 2008).

### 2.9.2. Results

2.9.2.1. Combined Effect Flood: Deterministic

The shore location combined effects flood alternative consists of the combination of 1) the PMSS, 2) the antecedent 10% exceedance high tide, and 3) coincident wind-wave activity.

The PMSS and Antecedent 10 Percent Exceedance High Tide:

The maximum stillwater elevation of 25.8 feet[2], MSL (Section 2.4) was used in the determination of maximum water level resulting from combined effects of storm surge and wind-driven wave activity. The resulting stage hydrograph (with waves) for the deterministic PMSS is shown in Figure 2.9-4. Estimated wind speed and duration based on ADCIRC results are shown on Figure 2.9-5.

Coincident wind-wave activity

Wind wave effects at the MPS2 and MPS3 buildings were determined based on the maximum significant wave heights presented in Figure 2.9-7. While there will be wave effects at the MPS1 reactor building and turbine building, these are no longer safety-related structures. These structures will also significantly dissipate wave action that could otherwise affect both MPS2 and MPS3. As shown in Figure 2.9-7, waves on the eastern portion of the site are not of substantial height (i.e. less than 0.5 meters) and are travelling in a northeast direction. The presence of non-safety related buildings such as the MPS2 maintenance shop and other warehouse buildings would significantly dissipate any wave generation and wave energy that could impact the eastern portion of the site. The combination of wave direction and dissipation of wave energy due to the MPS1 buildings and other non-safety related structures indicate that wave effects are negligible on the southern and eastern portions of MPS. Therefore, wave effects were calculated on the western portion of the site where waves will impact the MPS2 and MPS3 intakes and turbine buildings. Significant wave heights and peak periods at these locations were extracted from the deterministic PMSS ADCIRC+SWAN model, as shown in Table 2.9-1. Figure 2.9-9 presents the locations of each node for the SWAN output locations.

Reflected wave crest heights and elevations are presented in Table 2.9-2, and correspond to 17.9 feet at the MPS2 intake, and 16.2 feet at the MPS3 intake, respectively. Maximum elevations associated with reflected wave crests are 43.7 feet, MSL at the MPS2 intake, and 42.0 feet, MSL at the MPS3 intake. The MPS2 intake would be overtopped by approximately 4.7 feet, for a period of approximately 3 hours. Non-breaking wave overtopping due to the significant wave height will have a non-impulsive, (i.e.,"green water") overtopping effect. Smaller waves and broken waves against the intake structure will result in splash and spray on the structure, but will not result in significant overtopping effects.

As shown in Figure 2.9-7, there is a significant decrease in wave height once waves propagate onto the site grade on the western portion of MPS. The dissipation in wave height is largely due to friction of land and uneven topography. Non-breaking wave heights near the MPS2 turbine building are

[2] 25.8 feet MSL is inclusive of model uncertainty (i.e. 0.78 feet), wave setup, and the difference between the peak simulated tide elevation at Watch Hill, RI and the antecedent water level of 1.026 feet: 23,3' MSL (Section 2.4) + 0.7' setup + 0.78' model uncertainty + 1.026' tidal difference = 25.8' MSL."

approximately 5.4 feet. The maximum wave height at the MPS3 Turbine building was calculated using the relationship of depth-limited wave heights (0.78 x depth; ANS, 1992), resulting in a wave height of 1.4 feet. The reflected wave crest height at the MPS2 turbine building is 6.7 feet and 1.9 feet at the MPS3 turbine building, as shown in Table 2.9-2. The maximum standing wave crest elevations are 32.5 feet and 27.7 feet, MSL, respectively.

2.9.2.2. Combined Effect Flood: Probabilistic Storm Surge

The combined effects for the 1E-6 Annual Exceedance Probability Probabilistic Storm Surge results from a combination of 1) the storm surge corresponding to the 1,000,000-year recurrence interval, 2) the mean high tide with sea level rise, and 3) coincident wind-wave activity.

Stillwater Elevation

The stillwater level resulting from the combination of the storm surge corresponding to the 1,000,000-year recurrence interval and mean high tide with sea level rise was calculated to be 21.0 feet, MSL including aleatory variability and epistemic uncertainty at the 1E-6 AEP level and 50-year sea level rise projections.

Since wave heights were initially developed using the ADCIRC+SWAN models based on the modeled stillwater elevation of 16.8 feet, MSL, wave heights in this model are biased lower and are not inclusive of uncertainty. Therefore, a second ADCIRC+SWAN model simulation was performed to include the uncertainty effects by adding 4.249 feet to the initial static water level. The stage hydrograph (with waves) from a representative storm that results in an elevation corresponding to the 1,000,000-year recurrence interval inclusive of uncertainty factors is shown in Figure 2.9-10. A comparison between the two model simulations shows that wave heights are slightly higher using the model inclusive of error uncertainty.

Coincident Wind-Wave Activity

Wave heights and periods from the probabilistic storm surge SWAN model are included in Table 2.9-3. The SWAN results indicate that at the MPS2 intake, wave heights are approximately 6.3 feet with a peak period of 4.3 seconds. At the MPS3 intake wave heights are approximately 7.0 feet with a corresponding peak period of 7.3 seconds. Reflected wave crest heights and elevations are presented in Table 2.9-4, and correspond to 7.8 feet at the MPS2 intake, and 7.7 feet at the MPS3 intake, respectively. Maximum elevations associated with reflected wave crests are 28.7 feet, MSL at the MPS2 intake, and 28.7 feet, MSL at the MPS3 intake which will not result in overtopping of the intake structures. While there may be a portion of waves breaking against the intakes, this would result in splash and spray on the structures, and not result in any significant overtopping.

Under the probabilistic storm surge stillwater elevation, MPS3 is not exposed to flooding as the site grade of 24 feet, MSL is above the stillwater elevation of 21 feet, MSL. While there will be wave effects at MPS1, these are no longer safety-related structures, and the presence of MPS1 will significantly decrease wave heights affecting MPS2. As shown in Figure 10, wave heights on the eastern portion of the site are not of substantial height (i.e. less than 0.5 meters). The SWAN model does not currently include detail inclusive of all MPS buildings, however, the presence of non-safety related buildings (such as the MPS2 maintenance shop, MPS2 Maintenance Snubber Shop, Health Facility, the Fire Water Tanks, Security Operations Center, and Refuel Outage Building) would significantly dissipate any wave generation and wave energy that could impact the eastern portion of the site (i.e. the MPS2 reactor building). Due to the dissipation of wave energy by the various non-

safety related buildings, the wave effects are considered negligible on the eastern side of the MPS2 reactor building. Therefore, wave effects were evaluated on the western portion of the site where waves will impact the MPS2 turbine building. Wave heights at these locations were extracted from the probabilistic ADCIRC+SWAN model, as shown in Table 3. As shown in Figure 9, there is a significant reduction in wave height once the waves propagate onto the site grade. This is due to bottom friction effects and steep changes in topography. Wave heights are dissipated to approximately 2.8 feet with a 9.1 second peak period. Reflected wave crest elevations were determined using the Sainflou formulas for fully head on reflected wave crests. The results are presented in Table 4. The reflected wave crest at the MPS2 Turbine building is 3.4 feet, with a maximum elevation of 24.4 feet, MSL. While this elevation is about 2.4 feet above the flood wall elevation of 22 feet, MSL, the siding of the flood wall will prevent water resulting from splash effects from entering the building (Dominion, 2014a). Splash effects are due to the reflected wave crests overtopping the flood wall at the turbine building.

2.9.2.3. Hydrostatic Force and Hydrodynamic Loading and Debris

Typical hydrostatic and hydrodynamic forces were calculated for the controlling deterministic combined flood effects and the probabilistic combined flood effects. Calculation equations, constants, and corresponding units were described in Section 2.4. Typical hydrostatic and hydrodynamic forces were calculated at the area near MPS2, the area near MPS3, the MPS2 intake structure and the MPS3 intake structure. The foot of the MPS2 and MPS3 intake structures is at elevation -30.0 feet, MSL (Dominion, 2014b and Dominion, 2014a, respectively).

Hydrostatic Loads

The maximum stillwater water elevation of 25.8 feet[3], MSL for the deterministic combined effect flood was used, and results in a depth of flood water of 11.8 feet at MPS2 turbine building, 1.8 feet at the MPS3 turbine building and 55.8 feet at the intake structures. The typical hydrostatic forces were calculated as:

| Location | Hydrostatic Load (lb/ft) | Elevation (feet MSL) |
|---|---|---|
| MPS2 Turbine Building | 4,456 | 17.9 |
| MPS3 Turbine Building | 104 | 24.6 |
| Intake Structures | 99,636 | -11.4 |

The pressure at the bottom of the intakes was determined to be 3,571 psf.

The maximum stillwater elevation of 21.0 feet, MSL for the probabilistic combined effect flood was used and results in a depth of flood water of 7 feet at the MPS2 turbine building and 51 feet at the

[3] 25.8 feet MSL is inclusive of model uncertainty (i.e. 0.78 feet), wave setup, and the difference between the peak simulated tide elevation at Watch Hill, RI and the antecedent water level of 1.026 feet: 23,3' MSL (Section 2.4) + 0.7' setup + 0.78' model uncertainty + 1.026' tidal difference = 25.8' MSL.

intake structures. The area near MPS3 is not flooded for the probabilistic combined effect flood. The typical hydrostatic forces were calculated as:

| Location | Hydrostatic Load (lb/ft) | Elevation (feet MSL) |
|---|---|---|
| MPS2 Turbine Building | 1,568 | 16.3 |
| MPS3 Turbine Building | N/A | N/A |
| Intake Structures | 83,232 | -13 |

The pressure at the bottom of the intakes was determined to be 3,264 psf.

Flow Velocity

For the intake structures, the flood depth used to calculate the flow velocity, the hydrodynamic force and debris loads on the MPS2 and MPS3 intakes was selected based on the elevations at the confluence of the sloped intake channel and the bay (MPS, 1989 and Dominion, 2014b). These elevations are -16 feet, MSL for MPS3 intake and -15 feet, MSL for MPS2 intake. An upper bound flow velocity was calculated to be 36.2 feet per second at the MPS2 intake and 36.7 feet per second at the MPS3 intake for the deterministic combined effects flood.

An upper bound flow velocity was calculated to be 19.5 feet per second at MPS2 turbine building and 7.6 feet per second at the MPS3 turbine building for the controlling deterministic combined effects flood.

An upper bound flow velocity was calculated to be 15.0 feet per second at the MPS2 turbine building, 34.0 feet per second at the MPS2 intake structure and 34.5 feet per second at the MPS3 intake structure for the probabilistic combined effects flood.

Hydrodynamic Loads

The hydrodynamic loading analysis was calculated along various buildings throughout the site (see Table 2.9-5) for the controlling deterministic combined effect flood. The hydrodynamic loading varies from 5,436 pounds per linear foot to 6,089 pounds per linear foot near MPS2 for the controlling deterministic combined effect flood. The hydrodynamic loading varies from 135 pounds per linear foot to 207 pounds per linear foot near MPS3 for the controlling deterministic combined effect flood. The hydrodynamic forces at MPS3 were analyzed as an equivalent hydrostatic force because the flood flow velocity was less than 10 feet per second. The hydrodynamic loading was calculated to be 66,498 pounds per linear foot at the MPS2 intake structure and 70,023 pounds per linear foot at the MPS3 intake structures. The hydrodynamic loading near MPS2 acts at elevation 19.9 feet, MSL. The hydrodynamic loading near MPS3 acts at elevation 24.9 feet MSL. The hydrodynamic loading at the intake structures acts at elevation -2.1 feet, MSL.

The hydrodynamic loading analysis was calculated along various buildings throughout the site (see Table 2.9-6) for the probabilistic storm surge. The hydrodynamic loading varies from 1,962 pounds per linear foot to 2,747 pounds per linear foot near MPS2 for the controlling probabilistic combined effect flood. The hydrodynamic loading near MPS2 act at elevation 17.5 feet, MSL. The

hydrodynamic loading was calculated to be 51,760 pounds per linear foot at the MPS2 intake structure and 54,774 pounds per linear foot at the MPS3 intake structure. The hydrodynamic loading at the intake structures acts at elevation -4.5 feet, MSL.

Debris Impact Loads

Typical debris impact loads on exterior portions of structures (for debris weight of 2,000 pounds) were calculated for the deterministic PMSS as 40,560 pounds near MPS2, 3,952 pounds near MPS3 and 75,296 pounds for the MPS2 intake structure and 76,336 pounds for the MPS3 intake structure for the controlling deterministic combined effects flood.

Debris impact loads on exterior portions of structures were calculated for the probabilistic storm surge as 31,200 pounds near MPS2, 70,720 pounds at the MPS2 intake structure and 71,760 pounds at the MPS3 intake structure for the probabilistic combined effects flood.

Wave Loads

Loads due to non-breaking waves were calculated as the hydrostatic and hydrodynamic loads described above.

For the deterministic PMSS, the typical breaking wave load on vertical walls was calculated as 55,696 pounds per foot near MPS2, 1,296 pounds per foot near MPS3 and 1,245,456 pounds per foot for the intake structures for the controlling deterministic combined effects flood.

For the probabilistic storm surge, the maximum breaking wave load on vertical walls was calculated as 19,600 pounds per foot near MPS2 and 1,040,400 pounds per foot for the intake structures for the probabilistic combined effects flood.

2.9.2.3.1 Tsunami Loading

Typical hydrostatic and hydrodynamic forces were calculated at the area near MPS2, the MPS2 intake structure and the MPS3 intake structure. The inundation extent along the MPS2 Turbine building was approximately 630 feet (see Section 2.6). The MPS2 intake structure is approximately 80 feet wide. The MPS3 intake structure is approximately 135 feet wide.

Hydrostatic Loads

The maximum water surface elevation of 14.7 feet MSL for the tsunami at MPS2 and MPS3 (Section 2.6) results in a depth of flood water of 0.7 feet at MPS2 and 44.7 feet at the intake structures. The area near MPS3 is not flooded due to the tsunami. The typical hydrostatic forces were calculated as:

| Location | Hydrostatic Load (lb/ft) | Elevation (feet MSL) |
|---|---|---|
| MPS2 Turbine Building | 18.4 | 14.2 |
| MPS3 Turbine Building | N/A | N/A |
| MPS2 Intake Structure | 74,954 | -15.1 |
| MPS3 Intake Structure | 74,954 | -15.1 |

Hydrodynamic Forces

The hydrodynamic loading was calculated to be 326 pounds per linear foot near MPS2 and 20,961 pounds per linear foot at the intake structures. The hydrodynamic loading near MPS2 acts at elevation 14.5 feet, MSL. The hydrodynamic loading at the intake structures acts at elevation -7.7 feet, MSL.

Impulsive Forces

The impulsive force was calculated to be 489 pounds per linear foot near MPS2 and 31,441 pounds per linear foot at the intake structures.

Debris Impact Forces

The maximum flow velocity of 1.05 meters per second (3.4 feet per second) was calculated in Section 2.6. The debris loads was calculated to be 15,515 pounds for a log and 953,593 pounds for a heavy shipping container at the MPS2 and MPS3 intake structures.

### 2.9.3. Conclusions

A summary of combined event scenario maximum water elevations are presented in Table 2.9-7. MPS is considered to be a shore location because riverine and dam failure-induced flooding has been demonstrated to be negligible. Both deterministic and a refined probabilistic combined effect flood analyses were performed for this combination:

- The resulting stillwater elevation for the deterministic analysis is 25.8 feet, MSL. This elevation is the combination of the modeled stillwater elevation of 23.3 feet MSL, wave setup of 0.7 feet, uncertainty (i.e. 0.78 feet) and the difference between the peak simulated tide elevation at Watch Hill, RI and the antecedent water level of 1.026 feet, which includes applicable sea level rise. The results of the reflected wave crest elevations at the MPS2 and MPS3 intakes are 43.7 feet and 42.0 feet, MSL, respectively. Reflected wave crest elevations on the western sides of the MPS2 and MPS3 turbine buildings are 32.5 feet and 27.7 feet, MSL, respectively. As shown in Figure 2.9-4, the peak of the deterministic surge occurs between 2.1 and 2.15 days of storm simulation (i.e., where day 15 marks the start of the storm simulation with representation of dynamic tide conditions).
- The resulting stillwater elevation for the probabilistic analysis is 21.0 feet, MSL. This elevation is the combination of the modeled stillwater (i.e., including wave setup) elevation of 16.8 feet MSL and the uncertainty effects of 4.249 feet., which include consideration of applicable sea level rise An additional ADCIRC+SWAN model was run for the probabilistic storm surge to account for model uncertainty. This model was run with a condition of adding 4.2 feet to the starting static water level. The results of the reflected wave crest elevations at the MPS2 and MPS3 intakes are 28.8 feet and 28.7 feet, MSL, respectively. Reflected wave crest elevations at the western side of the MPS2 turbine building is 24.4 feet, MSL. The probabilistic storm surge stillwater elevation does not inundate MPS3. As shown in Figure 2.9-10, the peak of the probabilistic surge occurs slightly after 2.1 days of storms 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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**2.9.3-2 ASCE, 2010**. "Minimum Design Loads for Buildings and Other Structures," ASCE/SEI 7-10, American Society of Civil Engineers (ASCE), 2010.

**2.9.3-3 Dominion, 2014a**. Millstone Power Station Unit 3, Final Safety Analysis Report (FSAR), Revision 25.2.

**2.9.3-4 Dominion, 2014b**. Millstone Power Station Unit 2, Final Safety Analysis Report (FSAR), Revision 30.2.

**2.9.3-5 FEMA, 2008**. "Guidelines for Design of Structures for Vertical Evacuation from Tsunamis," FEMA P646, June 2008.

**2.9.3-6 FEMA, 2011**. "Coastal Construction manual: Principles and Practices of Planning, Siting, designing, Constructing and Maintaining Residential Buildings in Coastal Areas," FEMA 55, 2011.

**2.9.3-7 FEMA, 2012**. "Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures," FEMA-P-259, 2012.

**2.9.3-8 Goda, 2010**. "Random Seas and Design of Maritime Structures," Advanced Series on Ocean Engineering – Volume 33, 3rd Edition, Y. Goda, 2010.

**2.9.3-9 MPS, 1989**. "Shorefront & Dredging Plan & Details," Drawing Number 12179-BCY-11A-3 SH 1, July 11, 1989.

**2.9.3-10 NRC, 2011**. Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants – NUREG/CR-7046, United States Nuclear Regulatory Commission, November 2011.

**2.9.3-11 USACE, 2006**. Coastal Engineering Manual – Part VI, Chapter 5, "Fundamentals of Design," EM 1110-2-1100, U.S. Army Corps of Engineers, June 2006.

**Table 2.9-1: Deterministic SWAN Results**

| Location | Significant Wave Height (feet) | Peak Wave Period (seconds) |
|---|---|---|
| MPS2 Intake | 15.1 | 9.4 |
| MPS2 Turbine Building | 5.4 | 8.9 |
| MPS3 Intake | 13.9 | 9.4 |
| MPS3 Turbine Building | 2.8 | 9.4 |

**Table 2.9-2: Deterministic Sainflou Reflected Wave Crest Results**

| Location | Reflected Wave Crest Height (feet) | Reflected Wave Crest Elevation (feet, MSL) |
|---|---|---|
| MPS2 Intake | 17.9 | 43.7 |
| MPS2 Turbine Building | 6.7 | 32.5 |
| MPS3 Intake | 16.2 | 42.0 |
| MPS3 Turbine Building | 1.9 | 27.7 |

**Table 2.9-3: Probabilistic SWAN Results**

| Location | Significant Wave Height (feet) | Peak Wave Period (seconds) |
|---|---|---|
| MPS2 Intake | 6.3 | 4.3 |
| MPS2 Turbine Building | 2.8 | 9.1 |
| MPS3 Intake | 7.0 | 7.3 |
| MPS3 Turbine Building | N/A | N/A |

**Table 2.9-4: Probabilistic Sainflou Reflected Wave Crest Results**

| Location | Reflected Wave Crest Height (feet) | Reflected Wave Crest Elevation (feet, MSL) |
|---|---|---|
| MPS2 Intake | 7.8 | 28.8 |
| MPS2 Turbine Building | 3.4 | 24.4 |
| MPS3 Intake | 7.7 | 28.7 |
| MPS3 Turbine Building | N/A | N/A |

**Table 2.9-5: Hydrodynamic loading for the controlling deterministic combined effect flood**

| | Building Name | Width in North - South Direction (feet) | Depth (feet) | Width to Height Ratio | Cd | Velocity (ft/sec) | Fdyn (lb/ft) | dh | Fdh (lb/ft) |
|---|---|---|---|---|---|---|---|---|---|
| MPS2 | Auxiliary | 165 | 11.8 | 14.0 | 1.3 | 19.5 | 5,654 | - | - |
| | Bldg 118 (Control Bldg East, Unit 1) | 30 | 11.8 | 2.5 | 1.25 | 19.5 | 5,436 | - | - |
| | Enclosure | 80 | 11.8 | 6.8 | 1.25 | 19.5 | 5,436 | - | - |
| | Fire Pump House | 65 | 11.8 | 5.5 | 1.25 | 19.5 | 5,436 | - | - |
| | Intake | 80 | 11.8 | 6.8 | 1.25 | 19.5 | 5,436 | - | - |
| | Turbine | 315 | 11.8 | 26.7 | 1.4 | 19.5 | 6,089 | - | - |
| MPS3 | Aux Building | - | 1.8 | - | - | 7.6 | - | - | - |
| | Control Building | 91 | 1.8 | 50.6 | 1.75 | 7.6 | - | 1.575 | 181 |
| | DWST | 40 | 1.8 | 22.2 | 1.4 | 7.6 | - | 1.26 | 145 |
| | EDG Building | 60 | 1.8 | 33.3 | 1.5 | 7.6 | - | 1.35 | 156 |
| | ESF Building | 150 | 1.8 | 83.3 | 1.8 | 7.6 | - | 1.62 | 187 |
| | Fuel Building | 75 | 1.8 | 41.7 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Hydrogen Recombiner Building | 60 | 1.8 | 33.3 | 1.5 | 7.6 | - | 1.35 | 156 |
| | Intake | 95 | 1.8 | 52.8 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Maint Shop | 150 | 1.8 | 83.3 | 1.8 | 7.6 | - | 1.62 | 187 |
| | RWST | 60 | 1.8 | 33.3 | 1.5 | 7.6 | - | 1.35 | 156 |
| | Service Bldg. | - | 1.8 | - | - | 7.6 | - | - | - |
| | Steam Valve Building | 85 | 1.8 | 47.2 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Turbine Building | 355 | 1.8 | 197.2 | 2 | 7.6 | - | 1.8 | 207 |
| | Waste Disposal | 80 | 1.8 | 44.4 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Building Name | Width in East - West Direction (feet) | Depth (feet) | Width to Height Ratio | Cd | Velocity (ft/sec) | Fdyn (lb/ft) | dh | Fdh (lb/ft) |
| MPS2 | Auxiliary | 85 | 11.8 | 7.2 | 1.25 | 19.5 | 5,436 | - | - |
| | Bldg 118 (Control Bldg East, Unit 1) | - | 11.8 | - | - | 19.5 | - | - | - |
| | Enclosure | 175 | 11.8 | 14.8 | 1.3 | 19.5 | 5,654 | - | - |
| | Fire Pump House | 20 | 11.8 | 1.7 | 1.25 | 19.5 | 5,436 | - | - |
| | Intake | 70 | 11.8 | 5.9 | 1.25 | 19.5 | 5,436 | - | - |
| | Turbine | - | 11.8 | - | - | 19.5 | - | - | - |
| MPS3 | Aux Building | 110 | 1.8 | 61.1 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Control Building | 120 | 1.8 | 66.7 | 1.75 | 7.6 | - | 1.575 | 181 |
| | DWST | 40 | 1.8 | 22.2 | 1.4 | 7.6 | - | 1.26 | 145 |
| | EDG Building | 80 | 1.8 | 44.4 | 1.75 | 7.6 | - | 1.575 | 181 |
| | ESF Building | 50 | 1.8 | 27.8 | 1.4 | 7.6 | - | 1.26 | 145 |
| | Fuel Building | - | 1.8 | - | - | 7.6 | - | - | - |
| | Hydrogen Recombiner Building | - | 1.8 | - | - | 7.6 | - | - | - |
| | Intake | 120 | 1.8 | 66.7 | 1.75 | 7.6 | - | 1.575 | 181 |
| | Maint Shop | 110 | 1.8 | 61.1 | 1.75 | 7.6 | - | 1.575 | 181 |
| | RWST | 60 | 1.8 | 33.3 | 1.5 | 7.6 | - | 1.35 | 156 |
| | Service Bldg. | 30 | 1.8 | 16.7 | 1.3 | 7.6 | - | 1.17 | 135 |
| | Steam Valve Building | - | 1.8 | - | - | 7.6 | - | - | - |
| | Turbine Building | - | 1.8 | - | - | 7.6 | - | - | - |
| | Waste Disposal | 115 | 1.8 | 63.9 | 1.75 | 7.6 | - | 1.575 | 181 |

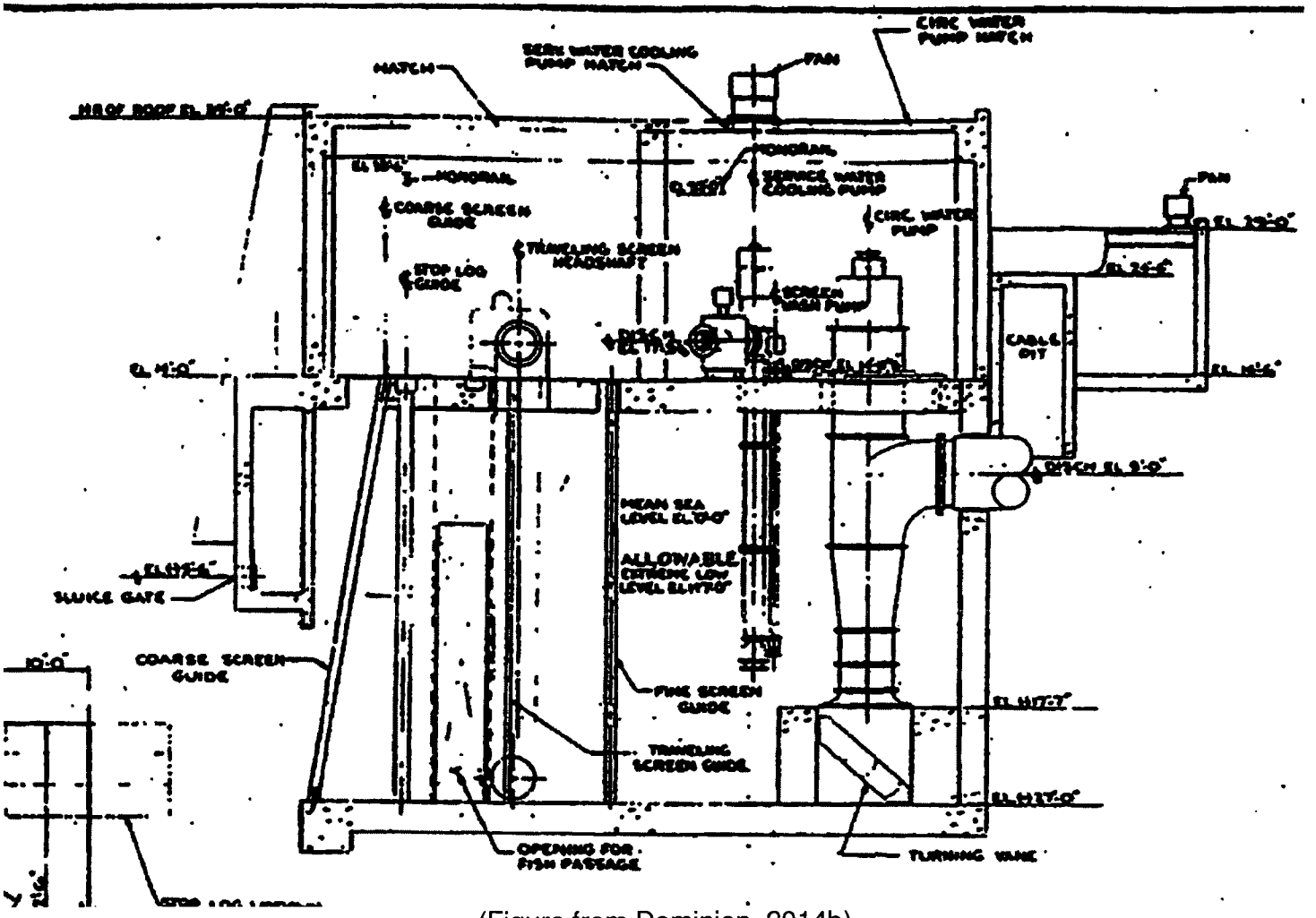
**Table 2.9-6: Hydrodynamic loading for the controlling probabilistic combined effect flood**

| | Building Name | Width in North - South Direction (feet) | Depth (feet) | Width to Height Ratio | Cd | Velocity (ft/sec) | Fdyn (lb/ft) |
|---|---|---|---|---|---|---|---|
| MPS2 | Auxiliary | 165 | 7 | 23.6 | 1.4 | 15.0 | 2,198 |
| | Bldg 118 (Control Bldg East, Unit 1) | 30 | 7 | 4.3 | 1.25 | 15.0 | 1,962 |
| | Enclosure | 80 | 7 | 11.4 | 1.25 | 15.0 | 1,962 |
| | Fire Pump House | 65 | 7 | 9.3 | 1.25 | 15.0 | 1,962 |
| | Intake | 80 | 7 | 11.4 | 1.25 | 15.0 | 1,962 |
| | Turbine | 315 | 7 | 45.0 | 1.75 | 15.0 | 2,747 |
| MPS3 | Aux Building | - | 0 | - | - | - | - |
| | Control Building | 91 | 0 | - | - | - | - |
| | DWST | 40 | 0 | - | - | - | - |
| | EDG Building | 60 | 0 | - | - | - | - |
| | ESF Building | 150 | 0 | - | - | - | - |
| | Fuel Building | 75 | 0 | - | - | - | - |
| | Hydrogen Recombiner Building | 60 | 0 | - | - | - | - |
| | Intake | 95 | 0 | - | - | - | - |
| | Maint Shop | 150 | 0 | - | - | - | - |
| | RWST | 60 | 0 | - | - | - | - |
| | Service Bldg. | - | 0 | - | - | - | - |
| | Steam Valve Building | 85 | 0 | - | - | - | - |
| | Turbine Building | 355 | 0 | - | - | - | - |
| | Waste Disposal | 80 | 0 | - | - | - | - |
| | Building Name | Width in East - West Direction (feet) | Depth (feet) | Width to Height Ratio | Cd | Velocity (ft/sec) | Fdyn (lb/ft) |
| MPS2 | Auxiliary | 85 | 7 | 12.1 | 1.3 | 15.0 | 2,041 |
| | Bldg 118 (Control Bldg East, Unit 1) | - | 7 | - | - | 15.0 | - |
| | Enclosure | 175 | 7 | 25.0 | 1.4 | 15.0 | 2,198 |
| | Fire Pump House | 20 | 7 | 2.9 | 1.25 | 15.0 | 1,962 |
| | Intake | 70 | 7 | 10.0 | 1.25 | 15.0 | 1,962 |
| | Turbine | - | 7 | - | - | - | - |
| MPS3 | Aux Building | 110 | 0 | - | - | - | - |
| | Control Building | 120 | 0 | - | - | - | - |
| | DWST | 40 | 0 | - | - | - | - |
| | EDG Building | 80 | 0 | - | - | - | - |
| | ESF Building | 50 | 0 | - | - | - | - |
| | Fuel Building | - | 0 | - | - | - | - |
| | Hydrogen Recombiner Building | - | 0 | - | - | - | - |
| | Intake | 120 | 0 | - | - | - | - |
| | Maint Shop | 110 | 0 | - | - | - | - |
| | RWST | 60 | 0 | - | - | - | - |
| | Service Bldg. | 30 | 0 | - | - | - | - |
| | Steam Valve Building | - | 0 | - | - | - | - |
| | Turbine Building | - | 0 | - | - | - | - |
| | Waste Disposal | 115 | 0 | - | - | - | - |

**Table 2.9-7: Summary of Reflected Wave Crest Elevations at MPS**

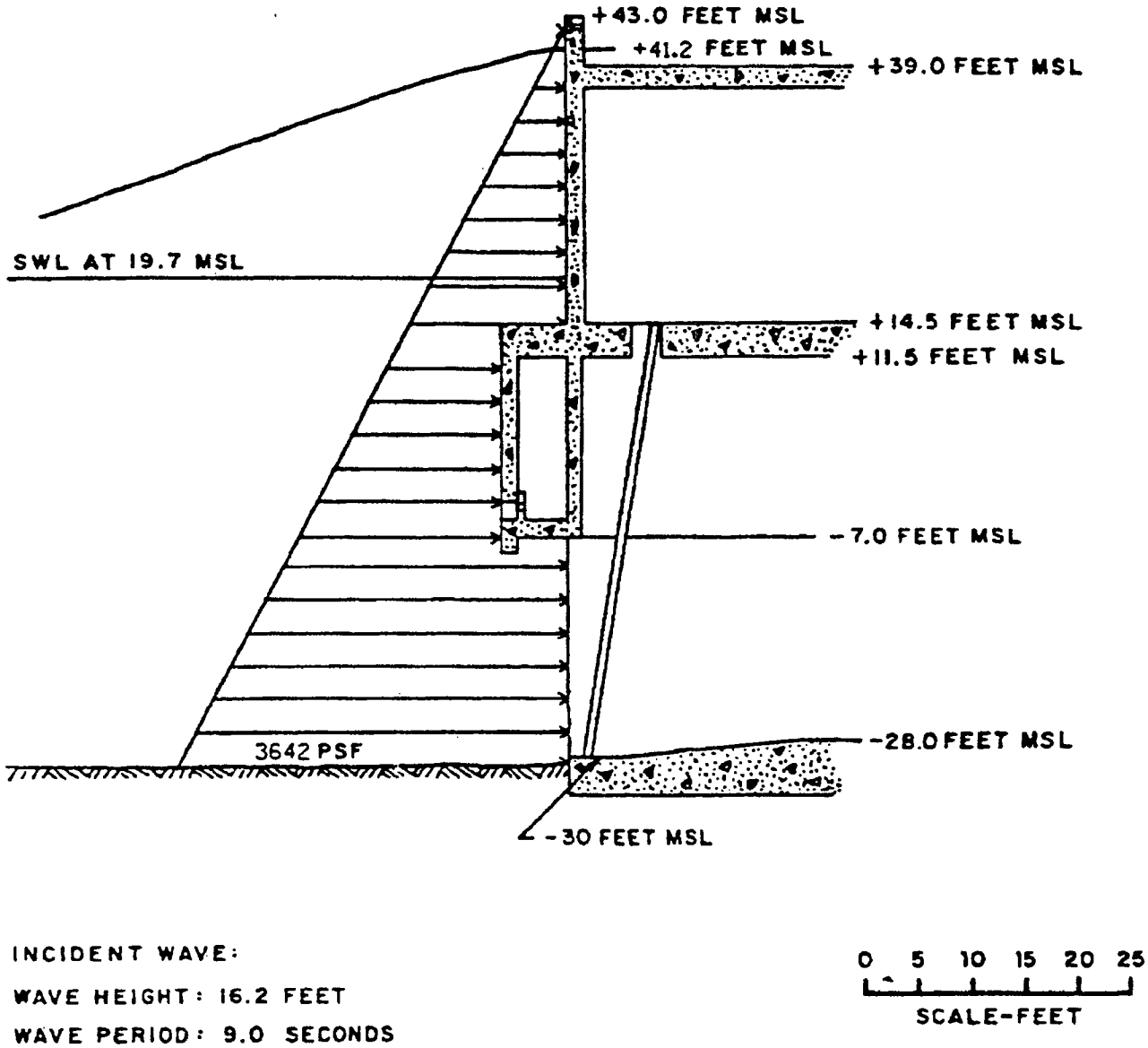
| Location | Deterministic PMSS | | Probabilistic Storm Surge | |
|---|---|---|---|---|
| | Reflected Wave Crest Height (feet) | Reflected Wave Crest Elevation (feet, MSL) | Reflected Wave Crest Height (feet) | Reflected Wave Crest Elevation (feet, MSL) |
| MPS2 Intake | 17.9 | 43.7 | 7.8 | 28.8 |
| MPS2 Turbine Building | 6.7 | 32.5 | 3.4 | 24.4 |
| MPS3 Intake | 16.2 | 42.0 | 7.7 | 28.7 |
| MPS3 Turbine Building | 1.9 | 27.7 | N/A | N/A |

**Figure 2.9-1: MPS2 Intake Structure Layout**



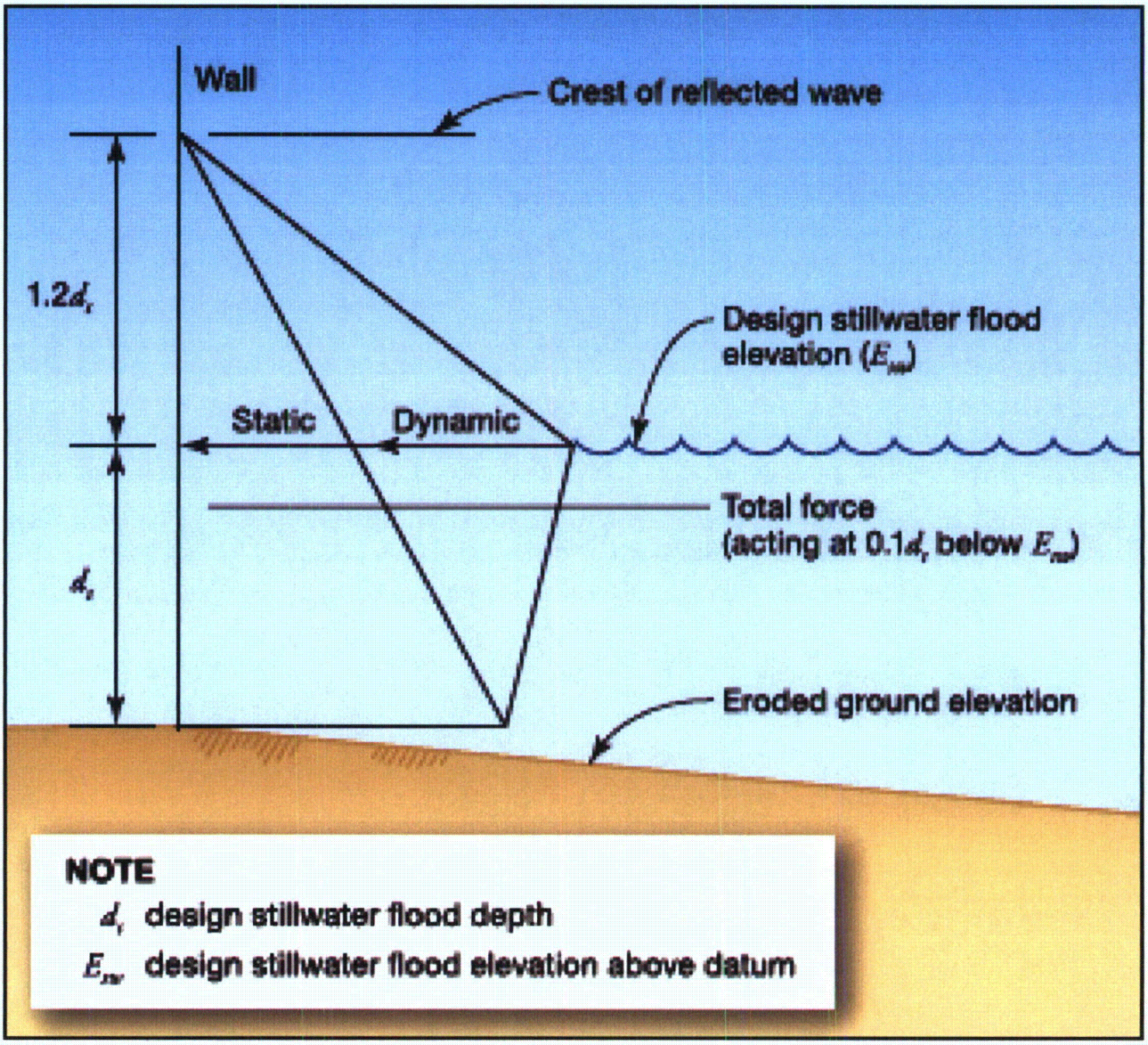

(Figure from Dominion, 2014b)

**Figure 2.9-2: MPS3 Intake Structure Layout**




(Figure from Dominion, 2014a)

**Figure 2.9-3: Wave load schematic**




(Figure from FEMA, 2011)

**Figure 2.9-4: Stage Hydrograph (Surge +Wave Setup) for Deterministic PMSS**

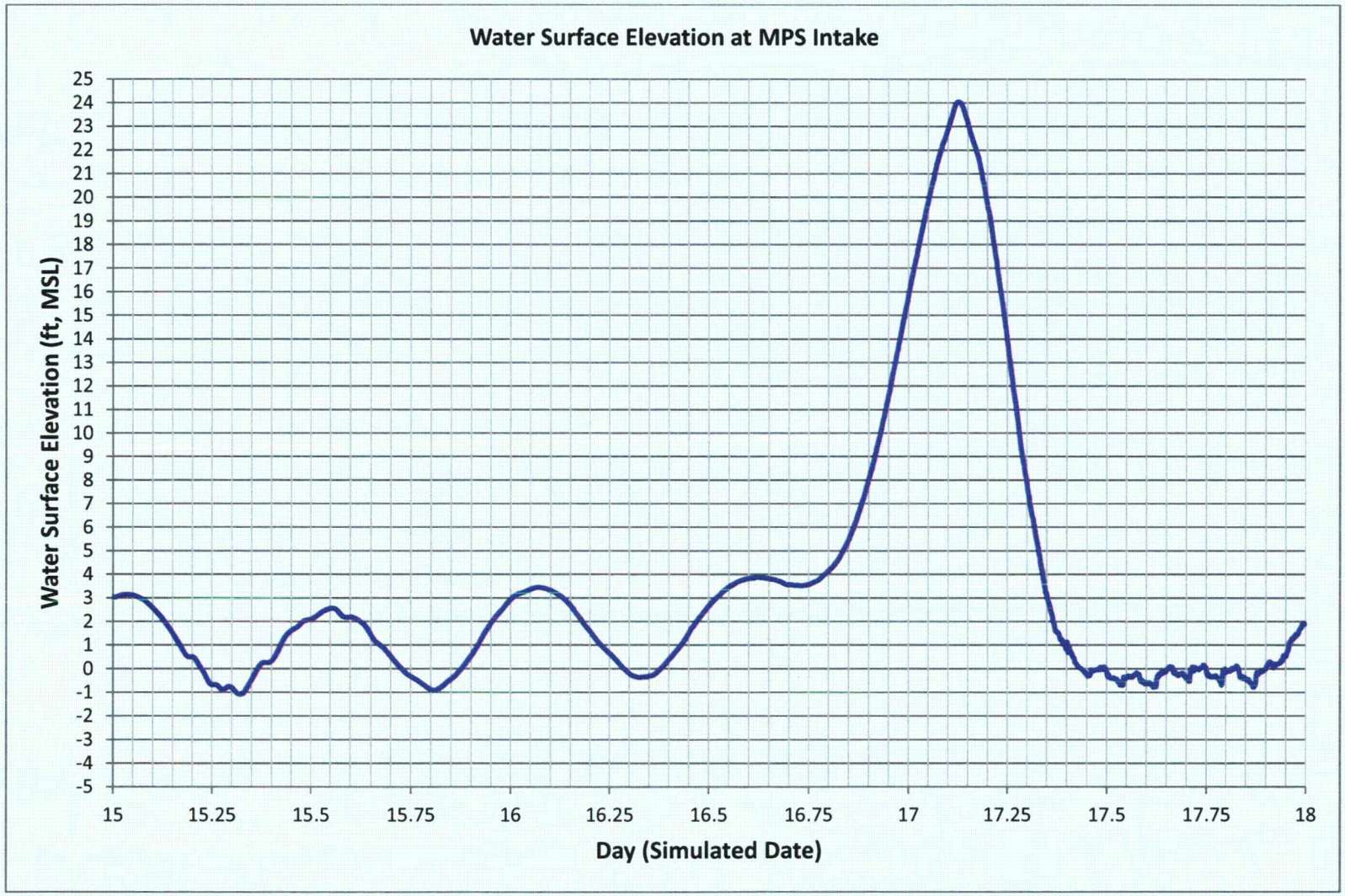


**Figure 2.9-5: Wind Speed vs Time for the Deterministic PMSS**

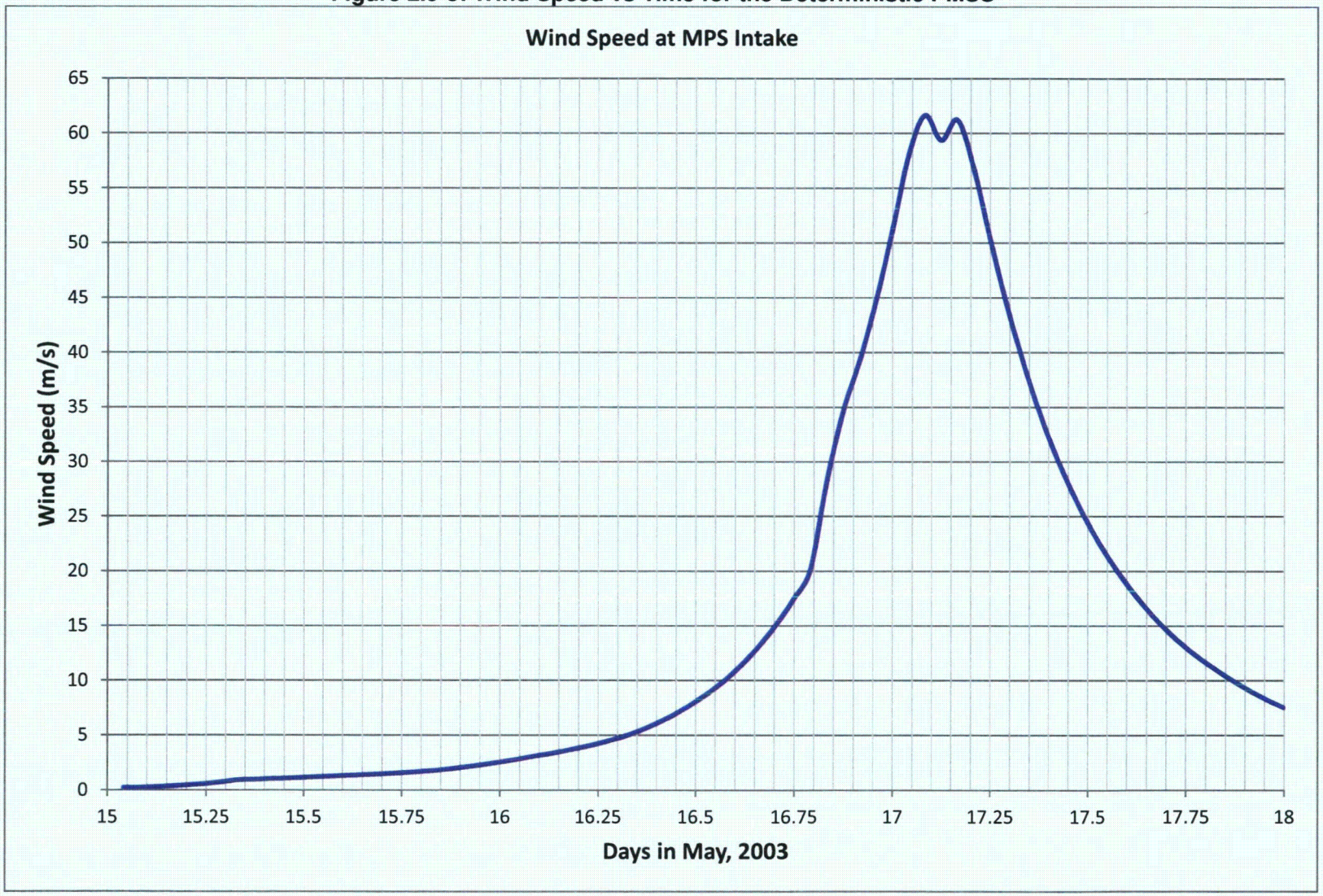
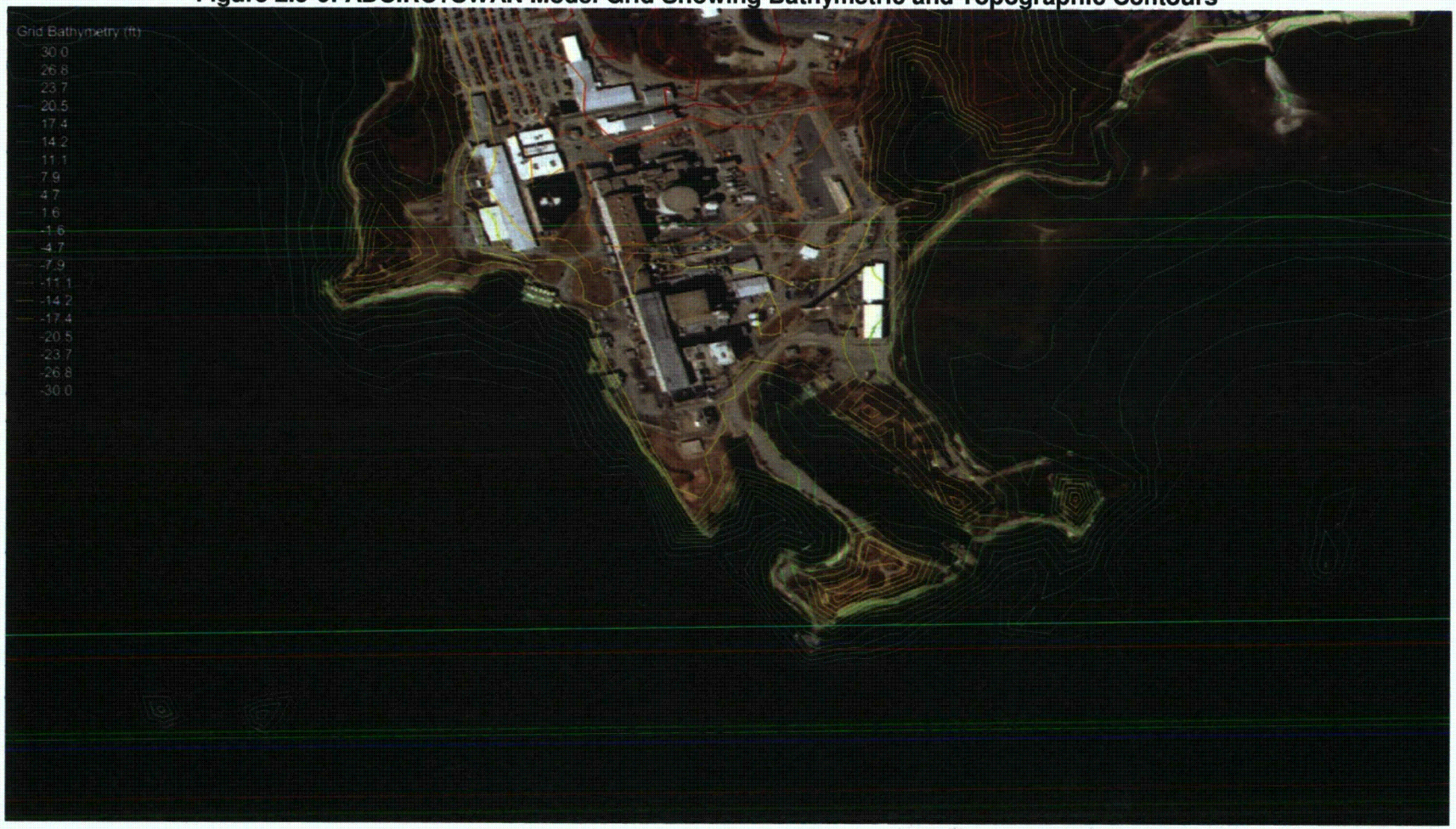


**Figure 2.9-6: ADCIRC+SWAN Model Grid Showing Bathymetric and Topographic Contours***




* Negative values indicate topographic contours.

**Figure 2.9-7: Maximum Significant Wave Height and Corresponding Wave Direction– Deterministic PMSS**

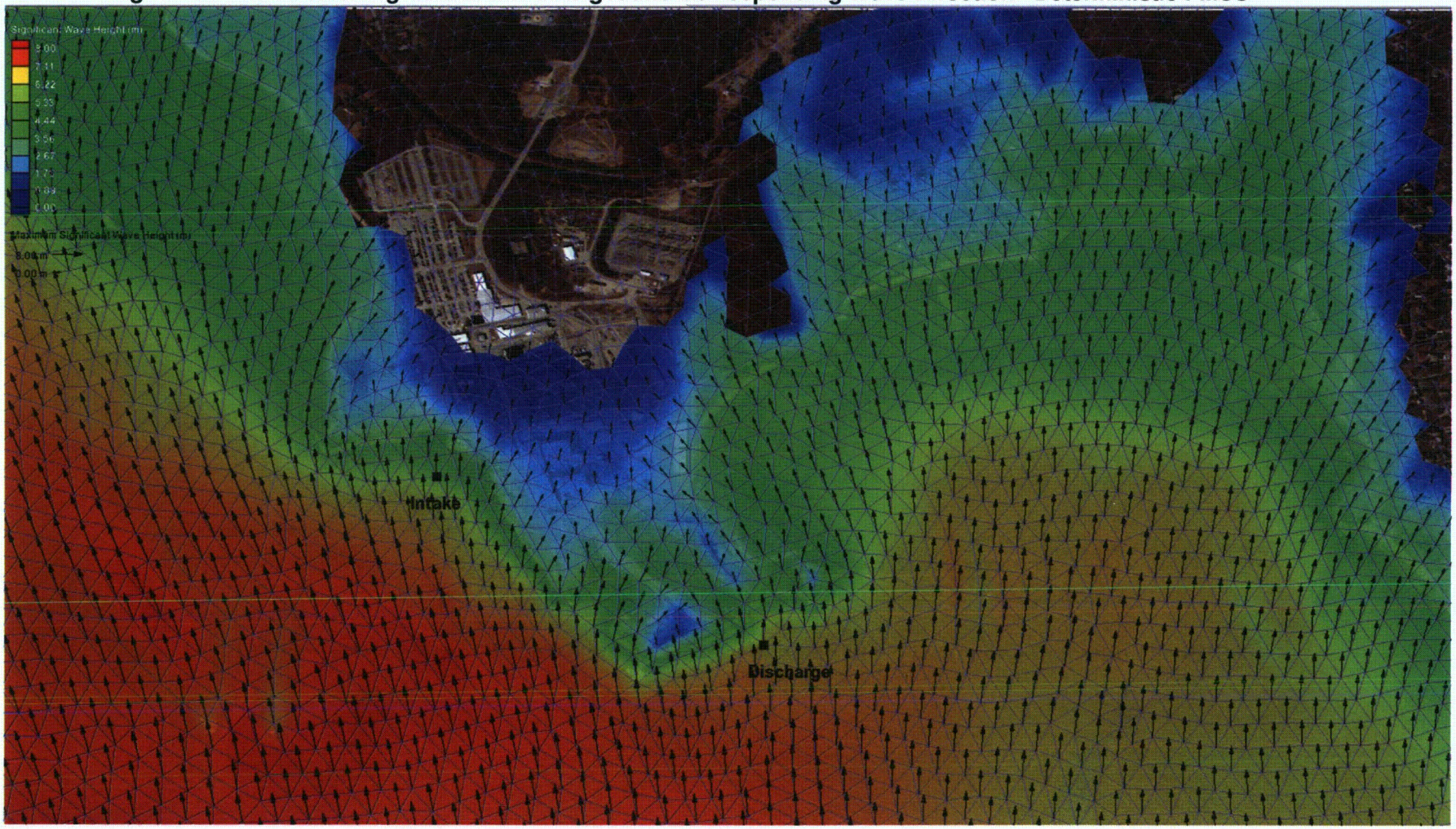


**Figure 2.9-8: Maximum Significant Wave Height and Corresponding Wave Direction– Probabilistic Storm Surge + 4.2 Feet Initial Level**

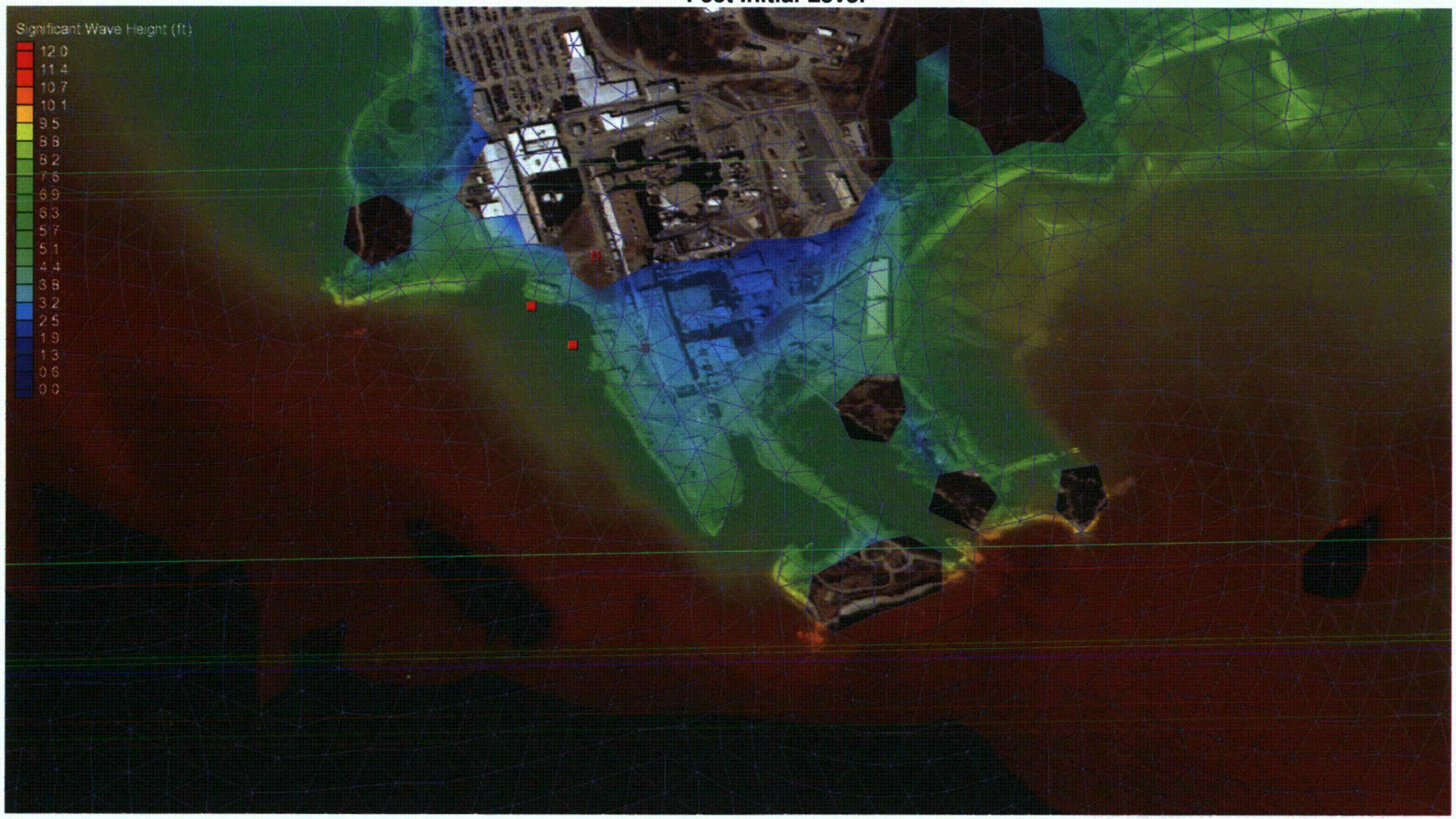
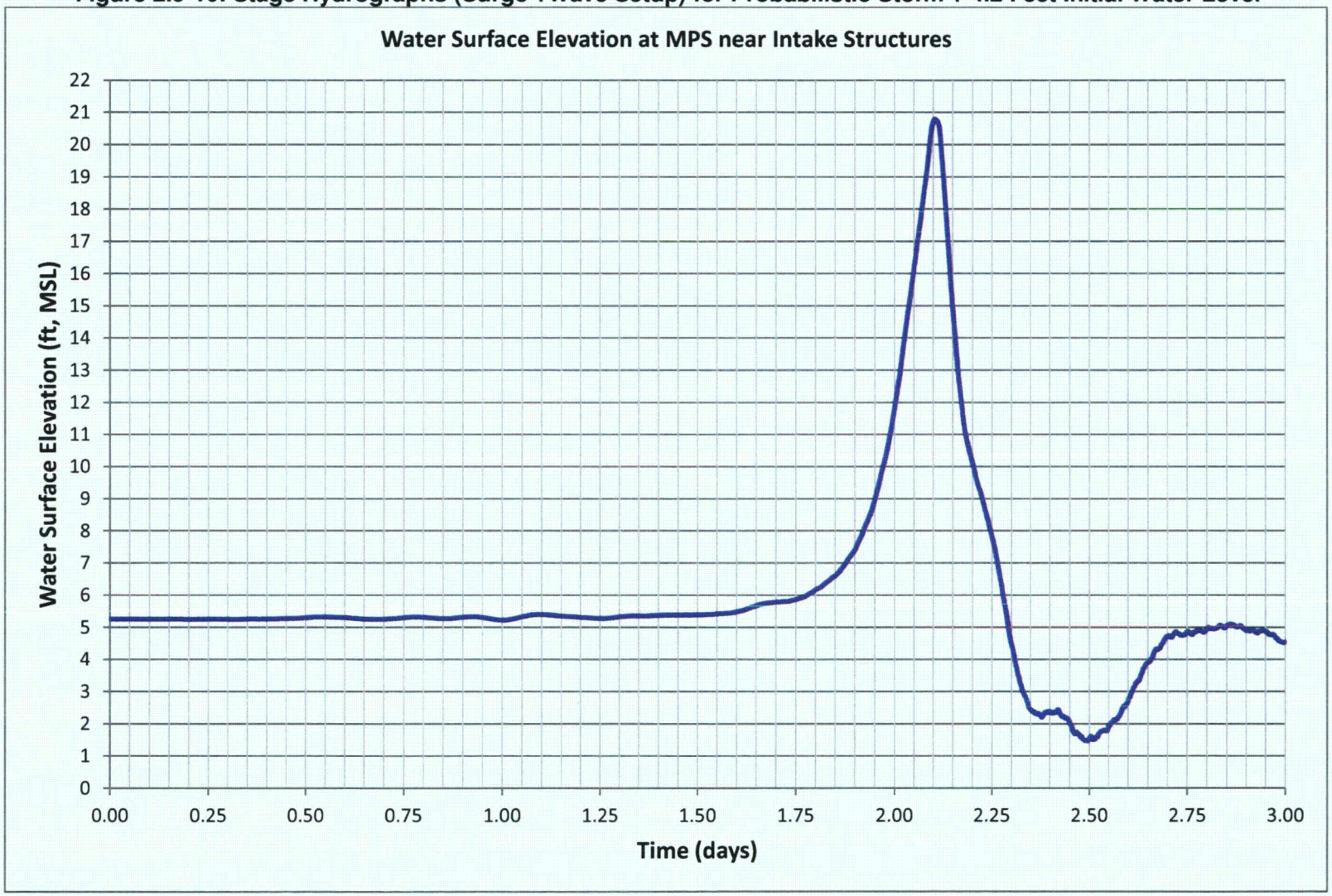


**Figure 2.9-9: Wave Height and Period Output Locations**

**Figure 2.9-10: Stage Hydrographs (Surge +Wave Setup) for Probabilistic Storm + 4.2 Feet Initial Water Level**



# 3.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

This section provides a comparison of current and reevaluated flood causing mechanisms at MPS 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 each of MPS2 and MPS3. The FSAR for MPS3 (Dominion 2014a) is used as a source of current design basis information for flooding. MPS2, constructed before MPS3, also provides information on current design basis information for flooding (Dominion 2014b). The MPS Flooding Walkdown report, which was reviewed and approved by the NRC, also contains information describing the current design basis (Dominion Nuclear Connecticut, 2012). Summary tables are provided in Table 3.0-1 and Table 3.0-2 for MPS2 and MPS3, respectively. A detailed LIP comparison at MPS3 is provided in Table 3.0-3.

As discussed below, the following reevaluated external flood mechanisms exceed the current design basis flood elevation at one or more areas of MPS2 and/or MPS3:

- Local Intense Precipitation (see Section 3.1);
- Storm Surge (see Section 3.4);
- Tsunami (see Section 3.5);
- 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 FSAR for MPS2 summarizes flooding due to LIP. Runoff was calculated using the Rational Method, based on a rainfall intensity of 9.4 inches per hour (Dominion, 2014b). A total runoff flow of 60 cubic feet per second (cfs) was calculated and compared to the capacity of the storm drain (i.e., catch basin number 9 outfall to Niantic Bay) of 8.8 cfs. Excess runoff was to accumulate in the yard area until it reaches Elevation 14.5 feet MSL and overtop a site access roadway, into Jordan Cove and Niantic Bay. Site grade at MPS2 is 14.0 feet MSL; therefore, the flood depth is 0.5 feet. The FSAR also notes that the MPS2 rainfall event would not produce a more significant flood than the flood associated with the storm surge (see Section 3.4).

MPS3 uses HMR-51 and 52 to calculate the flooding due to LIP (Dominion, 2014a). The one-hour PMP was calculated as 17.4 inches and the six-hour PMP was calculated to be 26.0 inches. The LIP analysis was performed using one-dimensional methods: a rainfall-runoff analysis was performed using the USACE HEC-1 computer program (predecessor to HEC-HMS) and water surface elevations were calculated using HEC-2 (predecessor to HEC-RAS). The calculation assumed no credit for the storm drain system and zero infiltration. The plant area was divided into individual drainage basins and the resulting computed runoff values were routed through "channels" based on site topography and project features such as buildings, roadways, and railroad tracks. Computed maximum water surface elevations (in feet, MSL) for each structure are summarized below (reprinted from Table 2.4-11 of Dominion 2014a):

| | |
|---|---|
| Auxiliary Building | 24.85 |
| Control Building | 24.27 |
| Emergency Generator Enclosure | 24.27 |
| Main Steam Valve Building | 24.85 |
| Hydrogen Recombiner Building | 24.85 |
| Auxiliary Building | 24.85 |
| Engineered Safety Features Building | 24.85 |
| Fuel Building | 24.85 |
| RWST/SIL Valve Enclosure | 24.85 |
| Demineralized Water Storage Tank Block House | 24.85 |

Although some of the above water surface elevations exceed the typical door sill elevation at MPS3 of 24.50 feet MSL, no affects upon safety-related equipment were anticipated due to insignificant leakage rates through doors.

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 consistent with the Hierarchical Hazard Assessment (HHA) approach. Resulting maximum flood depths and maximum LIP flood elevations vary by location.

Maximum LIP elevations at MPS3 are below the current licensing basis values. Maximum LIP elevations at the MPS3 Control Building and the Emergency Generator Enclosure are 24.2 feet MSL and 24.0 feet MSL, respectively (compared to the current design basis value of 24.27 feet MSL at both locations). The maximum LIP elevations for the remainder of MPS3 locations identified above are 24.8 feet MSL or less, which is at least 0.05 feet below the current design basis maximum LIP elevation.

Maximum LIP elevations at MPS2 locally exceed the current licensing basis values. Maximum LIP elevations at MPS2 range from El. 14.3 feet MSL at Flood Gate No. 20 (Item 218) situated at the intake structure to El. 17.5 feet MSL at Flood Gate No. 13 (Item 211) at the northern perimeter of the Containment Enclosure building. The LIP maximum flood elevations in the immediate vicinity of MPS3 range from El.14.0 feet MSL at Door WP-14-7A (Item. 302) to locally as high as El. 24.8 feet MSL at Door A-24-6 (Item 357) in the alleyway south of the Service Building (Building No. 317). Table 2.1-7 presents the maximum LIP flood depths and elevations at many door locations throughout MPS.

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 MPS3 FSAR (Dominion, 2014a) states that: "There are no major rivers or streams in the vicinity of Millstone Point, nor are there any watercourses on the site." The MPS3 FSAR acknowledges the number of small brooks which flow into Jordan Cove, east of MPS, but concludes that: "In each area, local topography precludes flooding of any portion of the site from the landward side." Detailed analyses or calculations were not performed. The MPS2 FSAR similarly concludes that, due to the limited drainage area of the Niantic River, riverine flooding would not result in flooding of MPS. (Dominion, 2014b).

Reevaluation Results

The reevaluation addresses the potential for flooding at MPS due to the Probable Maximum Flood (PMF) on the small unnamed coastal stream near MPS. Riverine flooding in the Niantic River was not analyzed because flooding from the Niantic River is expected to dissipate into Niantic Bay and have a negligible effect on MPS.

As described in Section 2.2, the PMF peak flow rate in the small coastal stream near MPS was calculated to be 1,100 cfs. The peak PMF water surface elevation at MPS is 11.2 feet MSL, which is below MPS site grade at MPS2 of 14 feet MSL (Dominion, 2014b) and MPS3 site

grade of 24 feet MSL (Dominion, 2014a). 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 FSARs for MPS2 and MPS3 do not include calculation of flood elevations due to dam failure because there are no dams on the Niantic River and no major rivers or streams in the vicinity of MPS (Dominion 2014b, Dominion 2014a).

Reevaluation Results

The local drainage area of MPS and the 87-acre watershed contributing to a small coastal stream located approximately 200 feet east of the ISFSI were evaluated for potential dam failures as part of the flood hazard re-evaluation. The review of the databases did not identify any dams within the local drainage basins near MPS. Additionally, any upstream dam failure flows that reach Niantic Bay will dissipate quickly in Niantic Bay (i.e., Long Island Sound) and no significant increase in water surface elevation in Niantic Bay is expected.

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.

### 3.4. Probable Maximum Storm Surge

Current Design Basis

MPS2:

The MPS2 FSAR describes potential flooding due to the PMH (Dominion 2014b). 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 = 27.26 inches (Peripheral pressure 30.56 inches)
- Radius of maximum winds = 48 nautical miles
- Forward speed of translation = 15 knots
- Maximum gradient wind = 123 miles per hour
- Maximum (overwater) wind speed = 124 miles per hour (108 knots)

The track of the PMH was generally northwestward across Long Island and Long Island Sound, with landfall occurring east of New Haven, Connecticut. Other combinations of storm size and forward speed were evaluated but did not result in higher surges than the PMH presented above. The calculated surge components were:

- Wind setup = 12.41 feet;

- Water level rise due to pressure drop = 2.20 feet;
- Astronomical tide = 2.50 feet;
- Initial rise (forerunner) = 1.00 feet;
- Total surge stillwater level increase = 18.11 feet.

The initial rise value was based on several discussions with the Atomic Energy Commission (AEC), predecessor to the NRC. The FSAR notes that the AEC accepted a PMH total stillwater surge elevation of 18.2 feet MSL. Wave action was also calculated and combined with storm surge, as described in Section 3.9.

MPS2 is generally protected by gates and walls to an elevation of 22.0 feet MSL. The MPS2 intake structure has service water pump motors and associated equipment that are also protected to an elevation of 22.0 feet MSL. The MPS2 walkdown report notes that one service water pump motor is protected to Elevation 26.5 feet MSL (Dominion Nuclear Connecticut, 2012).

MPS3:

The MPS3 FSAR also uses NOAA technical report HUR 7-97 to develop the PMH (Dominion 2014a). Nine different PMH candidate combinations were evaluated. The surge analysis used a computerized bathystrophic storm surge model. The highest surge resulted from the following PMH parameters:

- Central pressure index = 27.26 inches (Peripheral pressure 30.56 inches)
- Radius of maximum winds = 48 nautical miles
- Forward speed of translation = 15 knots
- Maximum gradient wind = 124 to 131 miles per hour (108 to 114 knots)

In addition, the surge was combined with an astronomical tide (10 percent exceedance high tide) of 2.4 feet above MSL and an initial rise of 1.0 foot. The hurricane track followed a similar path as the MPS2 PMH. The resulting maximum surge stillwater elevation was calculated to be 19.7 feet MSL. Wave action was also evaluated, as described in Section 3.9.

The safety-related structures and equipment at MPS3 are protected from flooding by the site grade elevation of 24 feet MSL, with the exception of the circulating and service water pump house (i.e., intake structure). The seaward wall of the intake structure is constructed to withstand the forces of a standing wave, or clapotis, with a maximum crest elevation of 41.2 feet MSL.

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), the storm surge stillwater elevation at MPS was calculated to be 21.0 ft MSL. The stillwater elevation was then used as an input to the combined effect analysis to develop final maximum flood levels at MPS—see Section 3.9.

## 3.5. Seiche

Current Design Basis

The FSARs for MPS2 and MPS3 do not include calculation of flood elevations due to seiche (Dominion 2014b, Dominion 2014a). The MPS2 FSAR does not discuss seiche. The MPS3 FSAR sections on surge and seiche focus on storm surge.

Reevaluation Results

Seiche within two surface water bodies at MPS were analyzed for reevaluation, including: 1) the Long Island Sound and 2) the discharge basin (former quarry). Seiche was found to pose no flood risk to MPS based on the screening analysis performed using Merian's formula and literature review. Indications of resonance that could lead to significant seiche development were not found. 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 MPS3 FSAR notes that the North Atlantic coastline has an extremely low probability of tsunamis (Dominion 2014a). Thus, analyses of flooding and drawdown were not discussed in the MPS3 FSAR. The MPS2 FSAR does not discuss tsunami potential (Dominion 2014b).

Reevaluation Results

The tsunami flooding reevaluation analysis concluded that there is a regional tsunami hazard potential at MPS. Numerical modeling was then performed to account for the complex geography in and around Long Island Sound (see Section 2.6).

Several tsunamigenic sources were assessed. The analysis indicated the highest predicted runup elevations in the vicinity of MPS result from the subaerial landslide (extreme flank failure) of the CVV. Other tsunamigenic sources, such as the near-field submarine mass failure, do not result in flooding at MPS.

Propagation of the initial CVV surface waves across the Atlantic Ocean and into Long Island Sound results in maximum water levels of approximately 14.7 feet MSL near MPS as shown below:

| Maximum Water Surface Elevation at MPS MPS2 and MPS3 (feet MSL) | Maximum Water Surface Elevation on the Eastern Side of the Site (feet MSL) | Maximum Depth of Water Above MPS2 Average Site Grade (14 ft MSL) | Depth of Water Above MPS3 Average Site Grade (24 ft MSL) | Time from Tsunamigenic Source Event to Tsunami Reaching MPS (hr) |
|---|---|---|---|---|
| 14.7 | 12.0 | 0.7 | 0.0 | 8.7 |

As shown on Figure 2.6-19, inundation areas are highest in areas west of MPS2 and MPS3, in the vicinity of the parking areas, storage buildings, and wooded areas north and west of the intake structures. MPS is protected from flooding due to high water in these areas primarily by topography, but also by buildings not important to safety and security barriers.

However, maximum flood elevations of 14.7 feet MSL are predicted at the intake structures and at MPS2. In these areas, shallow flooding above average MPS2 site grade of 14 feet is possible (up to 0.7 feet).

A warning time of at least 8 hours from the initiating tsunamigenic event is predicted. Additionally, the NOAA National Tsunami Warning Center (NTWC) provides tsunami detection, forecasts, and warnings for the U.S. including the Atlantic coast. NTWC operates 24 hours per day, with a goal of issuing tsunami warnings within five minutes of an earthquake (NTWC, 2014).

MPS3 is not impacted by flooding due to tsunami, owing to the higher average site grade at MPS3 of 24 ft MSL.

See Section 4.0 for a discussion of interim actions planned or taken in response to tsunami hazard.

### 3.7. Ice-Induced Flooding

The criteria for ice-induced flooding is provided in NUREG/CR-7046, Appendix D (NRC 2011). Two ice-induced events may lead to flooding at MPS and are recommended and discussed in NUREG/CR-7046, Appendix D 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.

The MPS3 FSAR (Dominion, 2014a) does not specifically discuss the potential for flooding due to upstream or downstream ice jams. It does note that there is no history of ice in Niantic Bay or in the area of the circulating and service water pumphouse. The MPS3 FSAR describes preventive measures to recirculate water to prevent icing near the circulating and service water pumphouse, as well as features of the pumphouse that prohibit ice from entering the pumphouse. The MPS2 FSAR (Dominion, 2014b) notes that the formation of ice in front of the intake structure is highly unlikely and also discusses a recirculation procedure that can be used to limit icing.

Reevaluation Results

The re-evaluation concluded that the MPS's location at the downstream-most end of the Niantic Bay creates conditions which are unlikely to sustain a downstream ice dam due to both water salinity and channel morphology. Therefore, the potential for flooding to occur at MPS as a result of a downstream ice jam is not significant.

The failure of a conservatively-estimated hypothetical upstream ice jam would not exceed the protected elevation at MPS. The resulting rise in water level at Millstone was conservatively estimated to be 2.9 feet (see Section 2.7).

Safety-related structures at MPS3 are flood-protected up to a minimum elevation 24.0 feet except for the service water pumps and pump motors located in the intake structure, which are flood protected to elevation 25.5 feet (Dominion, 2014a). MPS2 is passively (i.e., does not require manual actions) flood protected up to the average site grade elevation of 14 feet NGVD29, except for the service water pump motors and associated electrical and control equipment located in the intake structure, which are flood protected to elevation 22 feet NGVD29 (Dominion 2014b). The estimated freeboard to the protected elevation in the MPS2 intake structure is:

MPS2 Intake Structure El. 22 feet – 2.9 feet = 19.1 feet

and the estimated freeboard to the protected elevation in the MPS3 intake structure is:

MPS3 Intake Structure El. 25.5 feet – 2.9 feet = 22.6 feet

The lowest protected elevation at MPS2 is:

MPS2 average site grade El. 14 feet – 2.9 feet = 11.1 feet

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 MPS3 FSAR (Dominion, 2014a) states that: "There are no channel diversions to the cooling water supply which would have any effect on safety related equipment." Detailed analyses or calculations were not performed. The MPS2 FSAR does not discuss channel migration or diversion (Dominion, 2014b).

Reevaluation Results

The reevaluation concluded that the Niantic River has not exhibited a tendency to meander. Long Island Sound also serves to dissipate high flows in the river. The geology and foundation materials at the site are resistant to erosion. The shoreline near MPS is protected with riprap. Given these conditions, channel migration or diversion is not considered to be a potential contributor to flooding at MPS. 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

MPS2:

The MPS2 FSAR describes coincident wave action combined with the PMSS elevation of 18.1 feet (Dominion, 2014b). The FSAR reports the wind would be from the southeast during the peak of the surge and MPS1 would "shield" MPS2 (other than the intake structure) from direct wave attack. The maximum PMSS stillwater depth of 4.1 feet was calculated from the MPS2 average ground elevation near plant buildings of 14.0 feet MSL. A maximum, depth-limited wave of 3.2 feet could be generated anywhere around MPS2 buildings, producing a maximum runup elevation of 25.1 feet MSL. While this is 3.1 feet above the top of the flood gates and flood walls protecting MPS2, the minimum elevation of the exterior concrete walls of the containment building, auxiliary building, and warehouse building is up to elevation 54.5 feet MSL. The turbine building and the enclosure building are protected by metal siding which is continuous over the exterior flood walls and sealed at the interface between the flood wall and siding with waterproof caulked connections. Therefore, the FSAR concluded that the wave runup elevation of 25.1 feet MSL does not result in adverse effects on any safety-related equipment.

A maximum wave level of 42.5 feet MSL was calculated at the vertical wall of the intake structure, which is open to the coast. The maximum water level inside the intake structure caused by the standing wave condition was calculated to be 26.5 feet MSL. The analysis considered the profile of the incident wave, in-leakage through the louvers and system head loss. The service water system is the only safety-related system in the intake structure. The service water pump motors and electrical equipment are protected to elevation 22 feet MSL, with one exception: The MPS2 walkdown report notes that one service water pump is protected through installation of protection for the service water motor to elevation 26.5 feet MSL (Dominion Nuclear Connecticut, 2012).

The FSAR notes the intake structure and vicinity is designed to be stable against all forces from wave action, including buoyancy and scour. The shores are protected by post-tensioned reinforced concrete walls founded upon bedrock. Areas immediately back of the walls are protected by riprap designed for the PMH condition. The maximum pressure at the foot of the intake structure was calculated to be 3,960 pounds per square foot and the stability of the structure was found to be stable under such conditions. The louvers in the front of the intake structure are capable of withstanding a maximum pressure of 1,120 pounds per square foot due to pressure from a nonbreaking wave.

MPS3:

The MPS3 FSAR (Dominion, 2014a) discusses the calculation of deep water waves, shallow water waves, wave shoaling, refraction, and resulting runup. The FSAR notes that the topography and configuration of Millstone Point protects the MPS3 area from open ocean waves and breaking waves during the period of peak tidal flooding when the winds are from the southeast. The FSAR indicates very large deepwater maximum waves approaching or exceeding 100 feet are reduced to 10 to 16 feet in maximum height by the time waves near the Millstone Point shoreline.

Three transects were used to calculate runup: two at the west side of MPS3, including the intake structure, and one at the east side of the structure. The calculation used Saville's method of composite slopes using wave steepness, structure type, and depth at the structure toe as input values. The maximum calculated runup value was 23.8 feet MSL. The maximum water level on the intake structure was calculated to be 41.2 feet MSL, based on a maximum wave height of 16.2 feet.

MPS3 safety-related structures are protected by the site grade elevation of 24 feet MSL. Wave action only effects the intake structure, which is designed to withstand the PMH, including resultant loading. Service water pumps and pump motors inside the intake structure pumphouse are housed in individual watertight cubicles. The cubicles are watertight up to elevation 25.5 feet MSL. Access openings below 23.8 feet MSL are fitted with watertight doors capable of withstanding the maximum hydrostatic loading. The seaward wall of the intake structure is reinforced concrete designed to withstand the standing wave or clapotis up to 41.2 feet MSL. Maximum wave loading was calculated to be 3,642 pounds per square foot. Maximum uplift pressure on the pumphouse floor was calculated to be 863 pounds per square foot.

Reevaluation Results

The reevaluation evaluated combined effect flooding based on the combination of floods provided in NUREG/CR-7046, Appendix H. These combined effect floods were considered to be appropriate for MPS. Riverine hazards were screened-out.

The stillwater level resulting from the combination of the storm surge corresponding to the 1E-6 annual exceedance probability (i.e., 1,000,000-year-return period) and mean high tide with sea level rise was calculated to be 21.0 feet, MSL. This elevation is the combination of the modeled stillwater (i.e., including wave setup) elevation of 16.8 feet MSL and the uncertainty effects of 4.249 feet., which include consideration of applicable sea level rise. Thus, the stillwater level includes wind setup, aleatory variability and epistemic uncertainty and 50-year sea level rise projections. MPS3 is not exposed to flooding as the site grade of 24 feet, MSL is above the stillwater elevation of 21 feet, MSL. Due to the dissipation of wave energy by the MPS1 buildings and lack of inundation on the eastern portion of the site, the wave effects are considered negligible on the eastern side of the MPS2. The reflected wave crest at the west side of MPS2 is 3.4 feet, with a maximum elevation of 24.4 feet, MSL. This elevation is about 2.4 feet above the flood wall elevation of 22 feet, MSL at MPS2.

MPS2 and MPS3 each have intake structures west of the main building complex that are ocean-front structures. Wave heights are approximately 6.3 feet at the MPS2 intake. At the MPS3 intake, wave heights are approximately 7.0 feet. Reflected wave crest heights are 7.8 feet at MPS2 intake and 7.7 feet at MPS3 intake, respectively. Maximum elevations associated with reflected wave crests are 28.8 feet, MSL outside the MPS2 intake, and 28.7 feet, MSL outside the MPS3 intake which will not result in overtopping of the intake structures. While there may be a portion of waves breaking against the intakes, this would result in splash and spray on the structures, and not result in any significant overtopping. The effect of wave action on outside of the intake structures on components inside the Intake Structures will be further evaluated - see Section 4.0 for more information.

Hydrostatic, hydrodynamic, and debris loading forces were conservatively developed for the Probabilistic combined effect flooding scenario, which bounds the tsunami scenario. These

forces are anticipated to be localized to the area around the MPS2 and MPS3 Intake Structures, as well as the west side of MPS2. Hydrostatic forces at the Intakes are estimated to be approximately 83,232 pounds per foot, acting at an elevation of -13 feet MSL. The pressure at the bottom of the intakes was estimated to be 3,264 pounds per square foot.

The hydrodynamic loading varies from 1,962 pounds per linear foot to 2,747 pounds per linear foot near MPS2 for the controlling probabilistic combined effect flood. The hydrodynamic loading near MPS2 act at elevation 17.5 feet, MSL. The hydrodynamic loading was calculated to be 51,760 pounds per linear foot at the MPS2 intake structure and 54,774 pounds per linear foot at the MPS3 intake structure. The hydrodynamic loading at the intake structures acts at elevation -4.5 feet, MSL. The maximum breaking wave load on vertical walls was calculated as 19,600 pounds per foot near MPS2 and 1,040,400 pounds per foot for the intake structures, based on a conservative upper bound water velocity up to 34.5 feet per second. Debris impact loads on exterior portions of structures were calculated as 31,200 pounds for the west side of MPS2, 70,720 pounds at the MPS2 intake structure, and 71,760 pounds at the MPS3 intake structure. Debris impact loads act at the water surface elevation.

Impact forces for flood loading conditions are not discussed in detail for the current licensing basis and differing methodologies used for the reevaluation make it difficult to provide specific comparisons to the current design basis for loading. Please refer to Section 4 for more information.

## 3.10. References

3.10-1 **Dominion, 2014a**. Millstone Power Station Final Safety Analysis Report (MPS-3 FSAR), Rev. 25.2.

3.10-2 **Dominion, 2014b**. Millstone Power Station Final Safety Analysis Report (MPS-2 FSAR), Rev. 30.2.

3.10-3 **Dominion Nuclear Connecticut, 2012**. Millstone Power Station Units 2 and 3, Flooding Walkdowns Results Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding, November, 2012

3.10-4 **NRC, 2011**. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011.

3.10-5 **NTWC, 2014**. National Oceanic and Atmospheric Administration, National Weather Service, National Tsunami Warning Center, "User's Guide for the Tsunami Warning System in the U.S. National Tsunami Warning Center Area-of-Responsibility," Updated July, 2014.

**Table 3.0-1: Summary of the Comparison of Current and Reevaluated Flood Causing Mechanisms for MPS2**

| Flooding Mechanism | Flood Critical Structure (Per FSAR) | Current Design Basis Flood Level (MSL) | Current Flood Protection Elevation (MSL) [2] | Reevaluated Flood Level (MSL) |
|---|---|---|---|---|
| Combined Effects | MPS2, except Intake Structure | 21.3 ft (Stillwater plus wave crest)<br>25.1 ft<br>(Wave runup) | 22 ft | 21.0 ft at east side of MPS2;<br>24.4 ft at west side of MPS2 |
| | MPS2 Intake Structure | 26.5 ft (standing wave inside Intake Structure) | 22 ft<br>except 26.5 ft (at one service water pump motor) | Wave runup up to 28.8 ft at the Intake structure |
| Storm Surge (Stillwater Elevation) | Diesel Generator & Intake Structure | 18.2 ft<br>[3] | 22 ft | 21.0 ft |
| Local Intense Precipitation | Containment & Enclosure Building, Aux Building, EDG Buildings, Control Building, Turbine Building, Intake Structure, Fire Pump House, and RSST | 14.5 ft | 14.5 ft<br>(22 ft if the Flood Gates are closed) | 14.3 ft to 17.5 ft<br>[1] |
| Tsunami (including wave runup) | Intake Structures | No Flooding Expected | 14.5 ft<br>(22 ft if the Flood Gates are closed) | 14.7 ft |
| Flooding in Streams and Rivers | No Flooding Expected | No Flooding Expected | No Flooding Expected | 11.2 ft<br>(No Flooding Expected – Below Site Grade) |
| Upstream Dam Failures | No Flooding Expected | No Flooding Expected | No Flooding Expected | No Flooding Expected |

Notes are located on the next page

**Table 3.0-1 (Continued): Summary of the Comparison of Current and Reevaluated Flood Causing Mechanisms for MPS2**

| Flooding Mechanism | Flood Critical Structure (Per FSAR) | Current Design Basis Flood Level (MSL) | Current Flood Protection Elevation (MSL) [2] | Reevaluated Flood Level (MSL) |
|---|---|---|---|---|
| Seiche | No Flooding Expected | No Flooding Expected | No Flooding Expected | No Flooding Expected |
| Ice Induced Flooding | 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] Flood Protection Elevation 22 ft. assumes that there is sufficient warning time to close all MPS2 flood gates;
[3] Current Design Basis Flood Level considers stillwater level plus wave runup. Wave action in conjunction with wave runup is projected to cause higher levels in some locations and was independently calculated.

**Table 3.0-2: Summary of the Comparison of Current and Reevaluated Flood Causing Mechanisms for MPS3**

| Flooding Mechanism | Flood Critical Structure (Per FSAR) | Current Design Basis Flood Level (MSL) | Current Flood Protection Elevation (MSL) | Reevaluated Flood Level (MSL) |
|---|---|---|---|---|
| Combined Effects | Intake Structure | 23.8 ft (near MPS3 except at front of Intake Structure)<br>41.2 ft (at seaward wall of Intake Structure)[2] | 24 ft (25.5 ft for SW Pumps) | 21.0 ft (stillwater elevation – site grade protects against wave runup except at Intake)<br>28.7 ft at Intake |
| Storm Surge (Stillwater Elevation) | Intake Structure | 19.7 ft<br>[2] | 24 ft (25.5 ft for SW Pumps) | 21.0 ft |
| Local Intense Precipitation | Aux Building, Control Building, DWST Block House, Emergency Generator Enclosure, ESF Building, Fuel Building, Hydrogen Recombiner Building, MSV Building, and RWST/SIL Valve Enclosure | 24.85 ft except<br>24.27 ft at Control Building and Emergency Generator Enclosure.<br>(See Table 3.0-3) | Typical door sill elevation is 24.5 ft – No affects upon safety-related equipment anticipated for water levels up to El. 24.85 ft. | Up to 24.8 ft;<br>24.2 ft at Control Building; 24.0 ft at Emergency Generator Enclosure<br>(See Table 3.0-3)<br>[1] |
| Tsunami (including wave runup) | Intake Structure | No flooding Expected | 24 ft (25.5 ft for SW Pumps) | 14.7 ft<br>(No flooding expected) |
| Flooding in Streams and Rivers | No Flooding Expected | No Flooding Expected | No Flooding Expected | 11.2 ft<br>(No Flooding Expected – Below Site Grade) |
| Upstream Dam Failures | No Flooding Expected | No Flooding Expected | No Flooding Expected | No Flooding Expected |
| Seiche | No Flooding Expected | No Flooding Expected | No Flooding Expected | No Flooding Expected |

Notes are located on next page

**Table 3.0-2 (Continued): Summary of the Comparison of Current and Reevaluated Flood Causing Mechanisms for MPS3**

| Flooding Mechanism | Flood Critical Structure (Per FSAR) | Current Design Basis Flood Level (MSL) | Current Flood Protection Elevation (MSL) | Reevaluated Flood Level (MSL) |
|---|---|---|---|---|
| Ice Induced Flooding | 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] Current Design Basis Flood Level considers stillwater level plus wave runup. Wave action in conjunction with wave runup is projected to cause higher levels in some locations and was independently calculated.

**Table 3.0-3: Summary of the Comparison of Current and Reevaluated LIP for MPS3**

| Building | Current Design Basis Maximum Flood Elevation (feet, MSL) | Reevaluated Maximum Flood Elevation (feet, MSL) | Representative FLO-2D Grid Element | Representative Location |
|---|---|---|---|---|
| Auxiliary Building | 24.85 | 24.57 | 43655 | Aux Building Door A-24-1 |
| Control Building | 24.27 | 24.24 | 45449 | Control Building Door - C-24-1 |
| Emergency Generator Enclosure | 24.27 | 24.08 | 43336 | EDG Building Door - EG-24-1 |
| Main Steam Valve Building | 24.85 | 24.50 | 50744 | Steam Valve Building Door - SV-24-3 |
| Hydrogen Recombiner Building | 24.85 | 24.19 | 51892 | Hydrogen Recombiner Door HR-24-5 |
| Auxiliary Building | 24.85 | 24.78 | 48433 | Aux Building Door A-24-6 |
| Engineered Safety Features Building | 24.85 | 24.20 | 49907 | ESF Building Door - SF-24-2 |
| Fuel Building | 24.85 | 24.50 | 44888 | Fuel Building Door - F-24-4 |
| RWST/SIL Valve Enclosure | 24.85 | 24.26 | 49042 | North Side of Structure |
| Demineralized Water Storage Tank Block House | 24.85 | 24.23 | 48170 | South Side of Structure |

# 4.0 INTERIM EVALUATION AND ACTIONS

This section identifies the interim evaluation and actions taken or planned prior to the 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.

Combined Effects Flooding due to storm surge 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 Local Intense Precipitation resulting from the Site Specific Probable Maximum Precipitation, and a Tsunami resulting from the subaerial landslide (extreme flank failure) of the Cumbre Vieja Volcano will be discussed in Sections 4.2 and 4.3, respectively.

## 4.1. Combined Effects Flooding

The Combined Effects Flooding considers two different approaches to storm surge (probabilistic and deterministic analysis), and investigates structural loading due to flooding. The basis for this section of the Flood Hazard Reevaluation Report will rely on the probabilistic analysis approach only. The Combined Effects Flooding analysis produced stillwater elevations of 21.0 ft MSL, which are above the CLB stillwater elevations for both MPS2 and MPS3, however, this level is below current flood protection levels.

The combined effects flooding results due to wave action vary across the site. Due to the dissipation of wave energy acting on MPS1 and the lack of inundation on the east side of the site, no significant wave activity is expected in these areas. Therefore, MPS2 is bounded under current flood protection levels on the east side of the plant. MPS3 (with the exception of the Intake Structure) is unaffected by wave activity based on the general site grade of 24.0 ft MSL.

The west side of the MPS2 Turbine Building and both MPS2 and MPS3 Intake structures may be subjected to flooding levels higher than the current license basis. The reflected wave crest, imposed on top of the stillwater level, creates a periodic wave that reaches an elevation of 24.4 ft MSL on the west side of the MPS2 Turbine Building and causes a BDB flooding elevation of 28.6 ft MSL and 28.7 ft MSL on the MPS2 and MPS3 Intake Structures (external flooding elevations), respectively. Additionally there are loads on the west side of the MPS2 Turbine Building from hydrostatic loading, hydrodynamic loading, debris impact, and wave impact that need further evaluation. The same types of loadings are seen on the Intake structures, except the loadings at the Intake Structures are of a larger magnitude and there is an additional combined effect loading introduced from the tsunami. Based on these results, in the event that either one or both Intake Structures become inoperable due to Combined Effects Flooding, the planned interim action is to invoke Millstone's FLEX strategies to respond to a loss of ultimate heat sink (UHS) event.

In addition to flood gates, the west side of the MPS2 Turbine Building has a concrete wall up to 22 ft MSL. On the exterior of the building, metal siding overlaps this wall and extends up to the parapet of the Turbine Building. The probabilistic storm surge is created by the Probable Maximum Hurricane (PMH), which has wind characteristics of a Category 4/5 hurricane on the Saffir-Simpson Hurricane Wind Scale. Based on the different hurricane parameters (such as

wind, wind generated debris, etc.), in addition to flooding, relying on the siding to keep flood waters out of the Turbine Building will be further evaluated in the Integrated Assessment.

The Combined Effects hazard (caused by the PMH) would be identified in advance by meteorological forecasting. There are current measures (procedure driven) that would be invoked by the site to prepare for these events. Some of the existing features include adjustments to staffing, power levels, various tank levels, and the installation of flood protection barriers in preparation for hurricane surge and loss of power on site.

Potential flooding inside the MPS2 Turbine Building has been considered and abnormal operations procedures have been updated to prepare for such an event. To mitigate flooding in the Turbine Building (due to flood water bypassing floodgates) the following preparatory actions will be taken in advance of an approaching storm in accordance with existing station procedures.

The following equipment will be staged in the Turbine Building condenser pit:

- Self-powered pumps
- Electric pumps with generators
- Air-driven diaphragm pumps
- Hoses to direct water outside

Additionally a BDB AFW pump will be staged at the Turbine Building Railway Access.

Operations will request that sandbag walls (at least 2 feet high) be established at the following locations:

- Outside the 125VDC Swithgear room door
- Inside the Machine Shop to West Service Corridor door
- Inside Service Building Hallway door between the men's locker room and the Service Building elevator area
- Inside the Control Building northwest door
- Outside East entrance to TDAFW Pump Room door
- Outside TDAFW Pump Room to Outside double doors

Therefore, an integrated assessment will be performed in response to the results of the Combined Effects flood hazards for MPS2 and MPS3. The assessment will validate existing and/or develop new mitigating strategies in response to combined effects flooding which may compromise existing flood protection and challenge SSCs in the MPS2 Turbine Building. Additionally, the MPS2 and MPS3 Intake Structures will be evaluated based on increased flood levels and new/increased structural loading.

### 4.2. 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. As an immediate action, a site specific LIP calculation was performed, relying on modern day technology and methodology (instead of the HMR No. 52 methodology). The results indicated that the flood levels at MPS3 are below those for the current LIP model considered in the MPS3 FSAR. Based on this no further action is required for MPS3. The results for MPS2 are above the CLB, but are bounded by flood levels for the CLB storm surge.

The interim action will be to review, revise, and include necessary steps to enhance the applicable station abnormal weather procedures for mitigation of a BDB potential flooding event due to a local intense precipitation (LIP) event. The procedure update will implement existing station flood protection features (for example closing flood gates) based on a notification of an imminent LIP event and include an entry condition (trigger event) to initiate required actions.

## 4.3. Tsunami

The controlling tsunami wave (generated from the subaerial landslide (extreme flank failure) of the Cumbre Vieja Volcano) impacts the south side of the plant at an elevation of 14.7 ft MSL, which is above the current site grade of MPS2 (14.0 ft MSL). Other potential tsunami sources investigated produced results which are below the current site grade. The flood levels produced from the tsunami are bounded by storm surge; however the warning time on the tsunami is less than that of a storm surge. The tsunami is predicted to take an estimated 8.7 hours to reach MPS from the initiation of the event.

The interim action will be to review, revise, and include necessary steps to enhance the applicable station abnormal weather procedures for prevention and mitigation of a potential flooding event due to a tsunami. The procedure updates will implement existing station flood protection features (for example closing flood gates) based on a notification of an imminent tsunami and include an entry condition (based on a tsunami warning from NOAA's/NWS National Tsunami Warning Center) to initiate required actions.

## 4.4. All other Flood Causing Mechanisms

Probable Maximum Flood in Streams and Rivers, Dam Failure, Seiche, Ice Induced Flooding, and Channel Migration/Diversion evaluations 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 Millstone 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 Section 4.1, an Integrated Assessment will be performed that addresses any concerns from the Combined Effects event. Sections 4.2 and 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 $\Delta Q_x$ is the net change in discharge in the eight floodplain directions for the grid element for the timestep $\Delta t$ 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 Grid Elements Results



Note: Pages B2 through B7 are not numbered. In lieu of numbering, page numbers are identified above.

# B1 - GRID ELEMENT NUMBER


Notes:

1) Aerial photo (Mckim & Creed, 2012a) for reference only as it shows buildings at an oblique angle. Use building outlines from survey (Mckim & Creed, 2012a) for reference to actual door locations.

2) Strategic location (Item) # 344 uses same FLO-2D grid element as Item # 343 and Item # 393 uses same FLO-2D grid element as Item # 389. Item # 344 and Item # 393 are not labeled in this map for display clarity.

Building No. Legend

100 - Unit 1 Gas Turbine
102 - Unit 1 Discharge
103 - Unit 1 Maintenance Office
105 - Unit 1 Turbine Building
107 - Unit 1 Intake Structure
109 - Unit 1 Gas Recombiner Room
110 - Unit 1/2 Engineering Office
111 - Unit 1 Reactor Building
112 - Distilled Water Storage Tank
113 - Unit 1 Condensate Pump House
114 - Condensate Storage Tank
115 - Waste Surge Tank
116 - Xenon-Krypton Building
117 - Railroad Access - Reactor
118 - Unit 1 Control Room
119 - Unit 1 Radwaste Building
120 - Unit 1 Radwaste Truck Bay
121 - Fire Water Tank 1
122 - Fire Water Tank 2
123 - Fire Pump House - Unit 1
124 - Fire Pump House - Unit 2
125 - Stack

200 - Unit 2 Discharge
201 - SAS
202 - Unit 2 Intake Structure
203 - Unit 2 Turbine Building
205 - Unit 2 Auxiliary Building
206 - Unit 2 Health Physics
207 - Unit 2 Enclosure Building/Containment
208 - Unit 2 Fuel Building
209 - Unit 2 Pure Water Storage Tank
210 - Unit 2 Refueling Water Storage Tank
211 - Unit 2 Maintenance Shop
212 - Condensate Polishing Building
215 - Make-up Water Facility
216 - Millstone Radwaste Reduction Facility (MRRF)
217 - Unit 2 Condensate Surge Tank
219 - Unit 2 Condensate Storage Tank

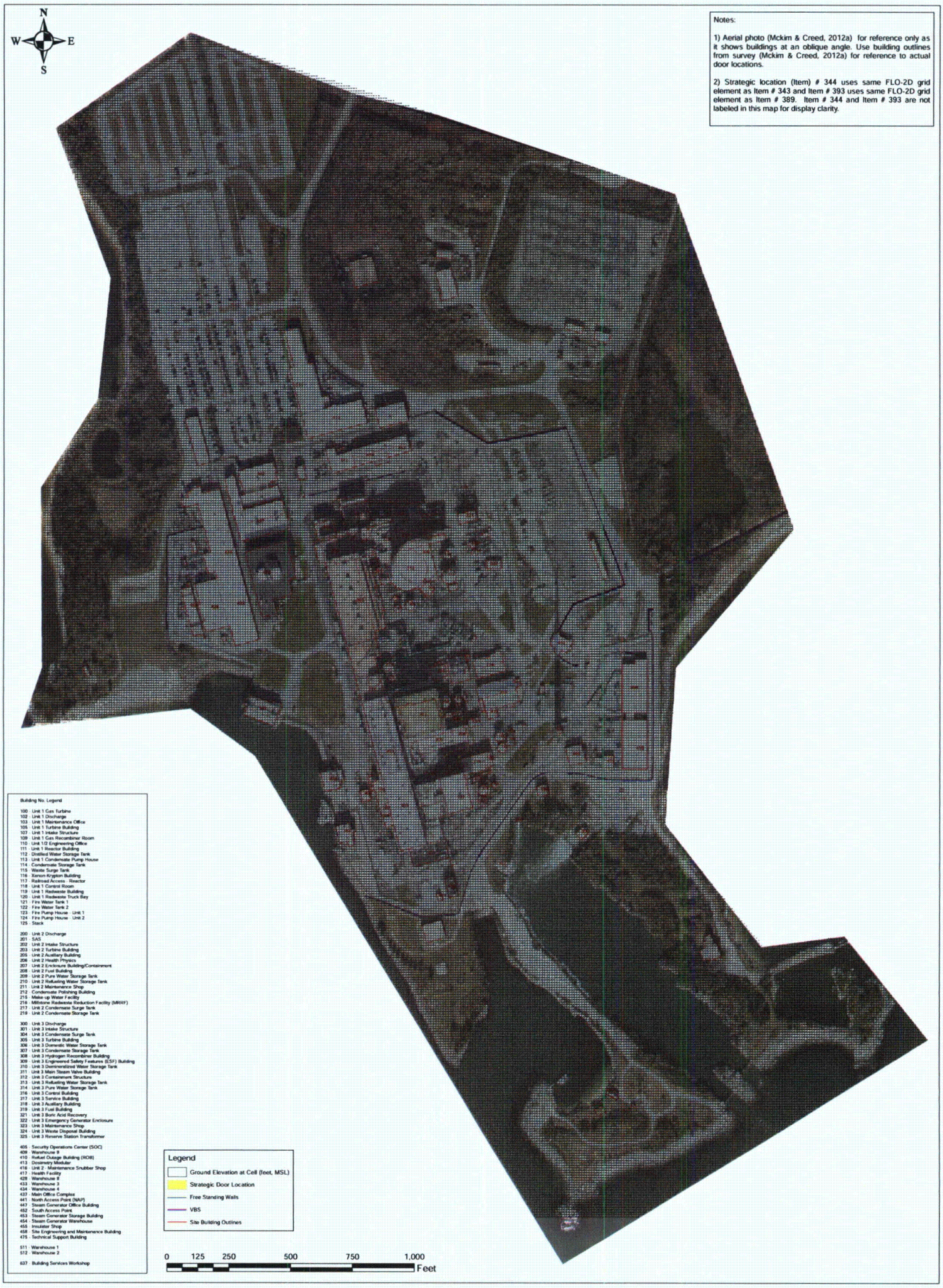
300 - Unit 3 Discharge
301 - Unit 3 Intake Structure
304 - Unit 3 Condensate Surge Tank
305 - Unit 3 Turbine Building
306 - Unit 3 Domestic Water Storage Tank
307 - Unit 3 Condensate Storage Tank
308 - Unit 3 Hydrogen Recombiner Building
309 - Unit 3 Engineered Safety Features (ESF) Building
310 - Unit 3 Demineralized Water Storage Tank
311 - Unit 3 Main Steam Valve Building
312 - Unit 3 Containment Structure
313 - Unit 3 Refueling Water Storage Tank
314 - Unit 3 Pure Water Storage Tank
316 - Unit 3 Control Building
317 - Unit 3 Service Building
318 - Unit 3 Auxiliary Building
319 - Unit 3 Fuel Building
321 - Unit 3 Boric Acid Recovery
322 - Unit 3 Emergency Generator Enclosure
323 - Unit 3 Maintenance Shop
324 - Unit 3 Waste Disposal Building
325 - Unit 3 Reserve Station Transformer

405 - Security Operations Center (SOC)
409 - Warehouse 9
410 - Refuel Outage Building (ROB)
413 - Dosimetry Modular
416 - Unit 2 - Maintenance Snubber Shop
417 - Health Facility
428 - Warehouse 8
433 - Warehouse 3
434 - Warehouse 4
437 - Main Office Complex
441 - North Access Point (NAP)
447 - Steam Generator Office Building
452 - South Access Point
453 - Steam Generator Storage Building
454 - Steam Generator Warehouse
455 - Insulator Shop
458 - Site Engineering and Maintenance Building
475 - Technical Support Building

511 - Warehouse 1
512 - Warehouse 2

637 - Building Services Workshop

# B2 - GROUND ELEVATIONS (FEET, MSL)

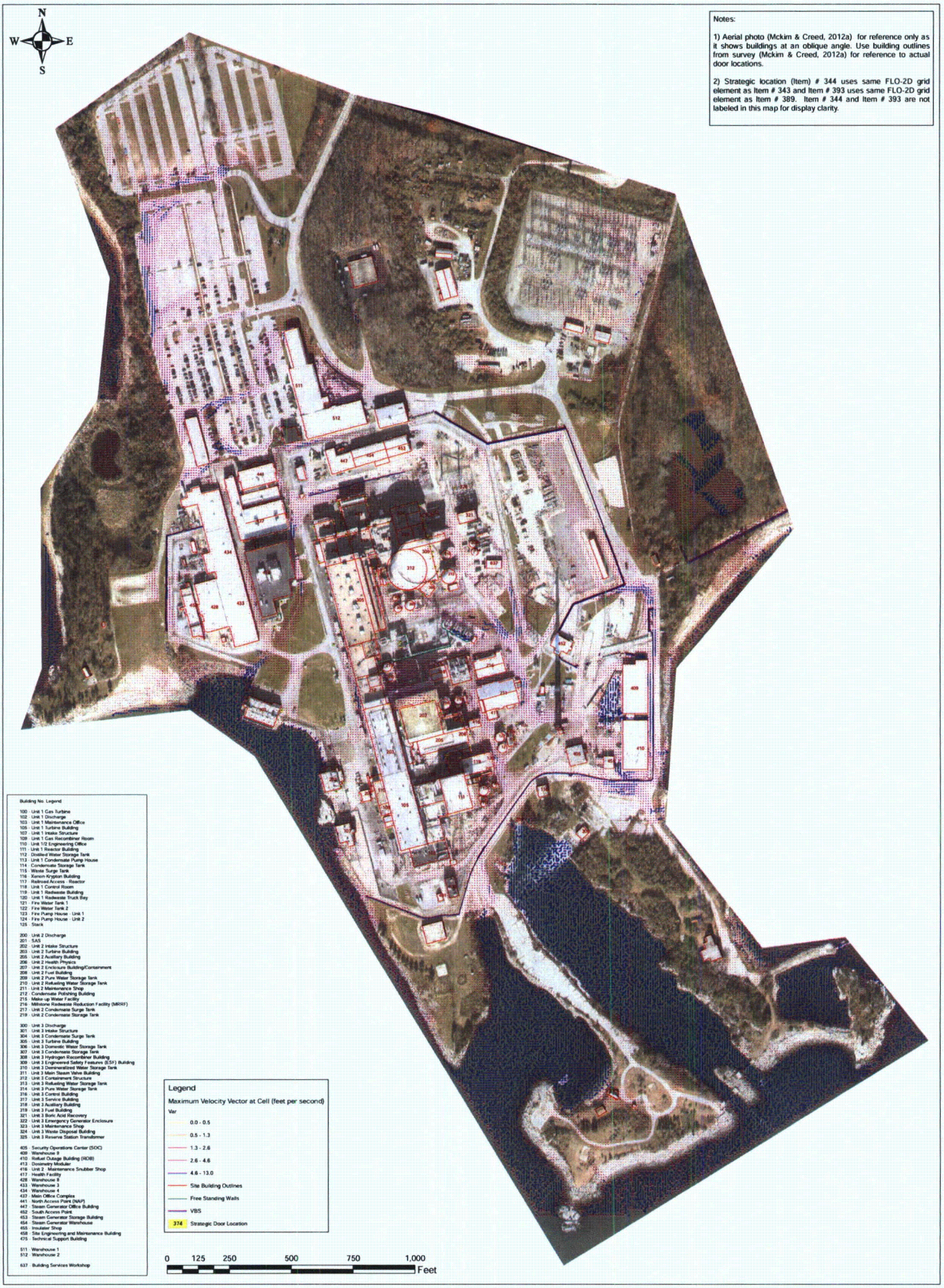


# B3 - MAXIMUM FLOW DEPTH (FEET)

# B4 - MAXIMUM WATER SURFACE ELEVATION (FEET, MSL)

# B5 - MAXIMUM FLOW VELOCITY (FEET PER SECOND)

# B6 - MAXIMUM FLOW VELOCITY VECTOR



# Appendix C
# Building Locations

Note: Pages C2 to C10 are not numbered due to their size.


## MILLSTONE STATION BUILDING NUMBERS

AS OF 4/7/92

| | | | |
|---|---|---|---|
| 100 | Gas Turbine | 200 | Unit 2 discharge |
| 101 | Unit 1 Maintenance Office | 201 | ESS |
| 102 | Unit 1 Discharge | | |
| 103 | Unit 1 Maintenance Shop | | |
| 104 | Unit 1 Switchyard | 300 | Unit 3 Discharge |
| 105 | Unit 1 Turbine Building | | |
| 106 | Unit 1 Strainer Pit | | |
| 107 | Unit 1 Fire Protectors | | |
| 108 | Unit 1 Hydro-Chloride System Room | 400 | Hydrogen Cylinder Storage Area |
| 109 | Unit 1 Air Accumulator Room | 401 | Fuel Farm |
| 110 | Unit 1/2 Engineering Offices | 402 | Central Guard Hookup Building |
| 111 | Unit 1 Reactor Building | 404 | Reactor Building PCM Enclosure |
| 112 | Unit 1 Condensate Storage Tank | | |
| 113 | Unit 1 Condensate Pump House | | |
| 114 | Condensate Storage Tank | 500 | Red Barn |
| 115 | Waste Surge Tank | 510 | Steam Generator Markup Building |
| 116 | Reach-Krypton Building | 515 | SRT Tower Equipment Enclosure |
| | | 516 | Red Barn Rain Shelter |

502 Environmental Lab

| | | | |
|---|---|---|---|
| 117 | Unit 1 Railroad Access - Reactor | 303 | Unit 3 Condensate Polishing Enclosure |
| 118 | Unit 1 Control Room | 303 | Unit 3 Auxiliary Boiler Enclosure |
| 119 | Unit 1 Radwaste Building | 304 | Unit 3 Condensate Surge Tank |
| 120 | Unit 1 Radwaste Treat Bldg | 305 | Unit 3 Turbine Building |
| 121 | Fire Water Tank 1 | 306 | Unit 3 Demineralized Water Storage Tank |
| 122 | Fire Water Tank 2 | 307 | Unit 3 Condensate Storage Tank |
| 123 | Fire Pump House - Unit 1 | 308 | Unit 3 Hydrogen Accumulator Building |
| 124 | Fire Pump House - Unit 2 | 309 | Unit 3 SBF Building |
| 125 | Stack | 310 | Unit 3 Demineralized Water Storage Tank |
| 126 | Stack Monitor Equipment | 311 | Unit 3 Main Steam Valve Building |
| 127 | Security DPS - to be removed | 312 | Unit 3 Refueling Water Storage Tank |
| | | 313 | Unit 3 Refueling Water Storage Tank |
| | | 314 | Unit 3 Pure Water Storage Tank |
| | | 315 | Technical Support Center |
| 301 | Unit 3 Intake Structure | 316 | Unit 3 Control Building |
| 302 | Unit 3 Turbine Building | 317 | Unit 3 Control Room |
| 304 | Unit 3 Control Room | 318 | Unit 3 Auxiliary Building |
| 305 | Unit 3 Auxiliary Building | 319 | Unit 3 Fuel Building |
| 306 | Unit 3 Fuel Building | 320 | Unit 3 Fuel Pool Access |
| 307 | Unit 3 Enclosure Building/Containment | 321 | Unit 3 Boric Acid Recovery |
| 308 | Unit 3 Fuel Building | 322 | Unit 3 Emergency Diesel Generator Enclosure |
| 309 | Unit 3 Pure Water Storage Tank | 323 | Unit 3 Maintenance Shop |
| 310 | Unit 3 Refueling Water Storage Tank | 324 | Unit 3 Diesel Fuel Oil Building |
| 311 | Unit 3 Maintenance Shop | 325 | Reserve Station Transformer |
| 312 | Condensate Polishing Facility | 326 | Unit 3 Maintenance Flammable Gas/Liq Storage |
| 313 | Annex Station Service Transformer | | Building and East Side of Build g 323 |
| 314 | Fire Protection | 327 | Unit 3 Maintenance Bottled Gas/Storage Building |
| 315 | Make-up Water Polishing Addition | | |
| 316 | Millstone Radwaste Reduction Facility (MRRF) | | |
| 317 | Unit 3 Condensate Surge Tank | | |
| 318 | Reserve Service Transformer | | |
| 319 | Unit 3 Condensate Storage Tank | | |

| | | | |
|---|---|---|---|
| 403 | Security Operations Center (SOC) | 554 | Schoolhouse |
| 409 | Warehouse 3 | 581 | Low Level Radwaste Storage |
| 410 | Safeguard Storage Building (SOB) | 586 | Switchyard Control Building |
| 411 | [illegible] Shelter | 611 | Warehouse 1 |
| 412 | Unit 1 Maintenance | 612 | Warehouse 2 |
| 413 | Unit 1 Maintenance Machine Shop | 614 | Sub Station 2 |
| 414 | Health Facility | 616 | Sub Station 1 |
| 415 | Unit 1 Maintenance Annex | 630 | Switchyard Storage Building |
| 420 | Security Guard Enclosure | 631 | East Parking Security Enclosure (Check Point 1 |
| 421 | Steam Generator Storage Building | 632 | Steam Generator Staging and Training Building |
| 422 | Clock House (Historic) | 633 | SGF Security Enclosure |
| 423 | Crafts Assembly Facility | | |
| 424 | Tree Ring Building | | |
| 425 | | | |
| 426 | Steam Generator Project Office | 637 | Building Services Workshop |
| 451 | Fuel Service Modular | 651 | Hydrogen Accumulator Portable PCM Enclosure |
| 452 | South Access Point | | |
| 453 | Steam Generator Storage Building | | |
| 454 | Steam Generator Warehouse | | |
| 455 | Fire Station | 700 | SGSF A/E Office Trailer Number 1 |
| 456 | Health Facility Annex | 701 | SGSF A/E Office Trailer Number 2 |
| 461 | NRAM PCM Enclosure Building | 702 | SGSF A/E Office Trailer Number 3 |
| 462 | SSF Building PCM Enclosure | | |
| 463 | Auxiliary Building PCM Enclosure | | |
| 464 | Reactor Building | | |
| 465 | VAP Boiler Fuel Bunk | | |
| 470 | Guard House (Near Building 467) | | |
| 471 | Modular Office Building, East of 443 | | |
| 472 | Aboveground Gasoline Tank | | |
| 474 | Diesel Tank | | |

301 Unit 3 Intake Structure

| | |
|---|---|
| 405 | |
| 429 | Warehouse 6 |
| 431 | Instrument Pub Shop |
| 433 | Rigging Loft |
| 434 | Warehouse 1 |
| 435 | Warehouse 2 |
| 436 | |
| 437 | Main Office Complex |
| 441 | North Access Point (NAP) |
| 450 | Gas Bottle Storage Building |
| 457 | Chemistry Safety Storage Building |
| 458 | Site Engineering and Maintenance Building |

| | |
|---|---|
| 503 | Airlock |
| 514 | Warehouse 4 |
| 527 | Check Point 2 |
| 528 | Day Point South Restroom |

NUC-153
ATTACHMENT MAP

4/7/92

INDICATES PHOTO CONTROL POINTS
BK. F-21 / PGS. 73-77
See Dwg. - No. 21582-C (For Ties)

SCALE 1" = 200'

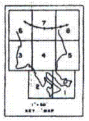
200 0 200 400 600 800

DATE OF PHOTOGRAPHY 12/3/91 CONTOUR INTERVAL ONE FOOT

200 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC.
PENNSAUKEN, NJ




SHEET 1 OF 1

THE CONNECTICUT LIGHT & POWER
BERLIN, CONN.

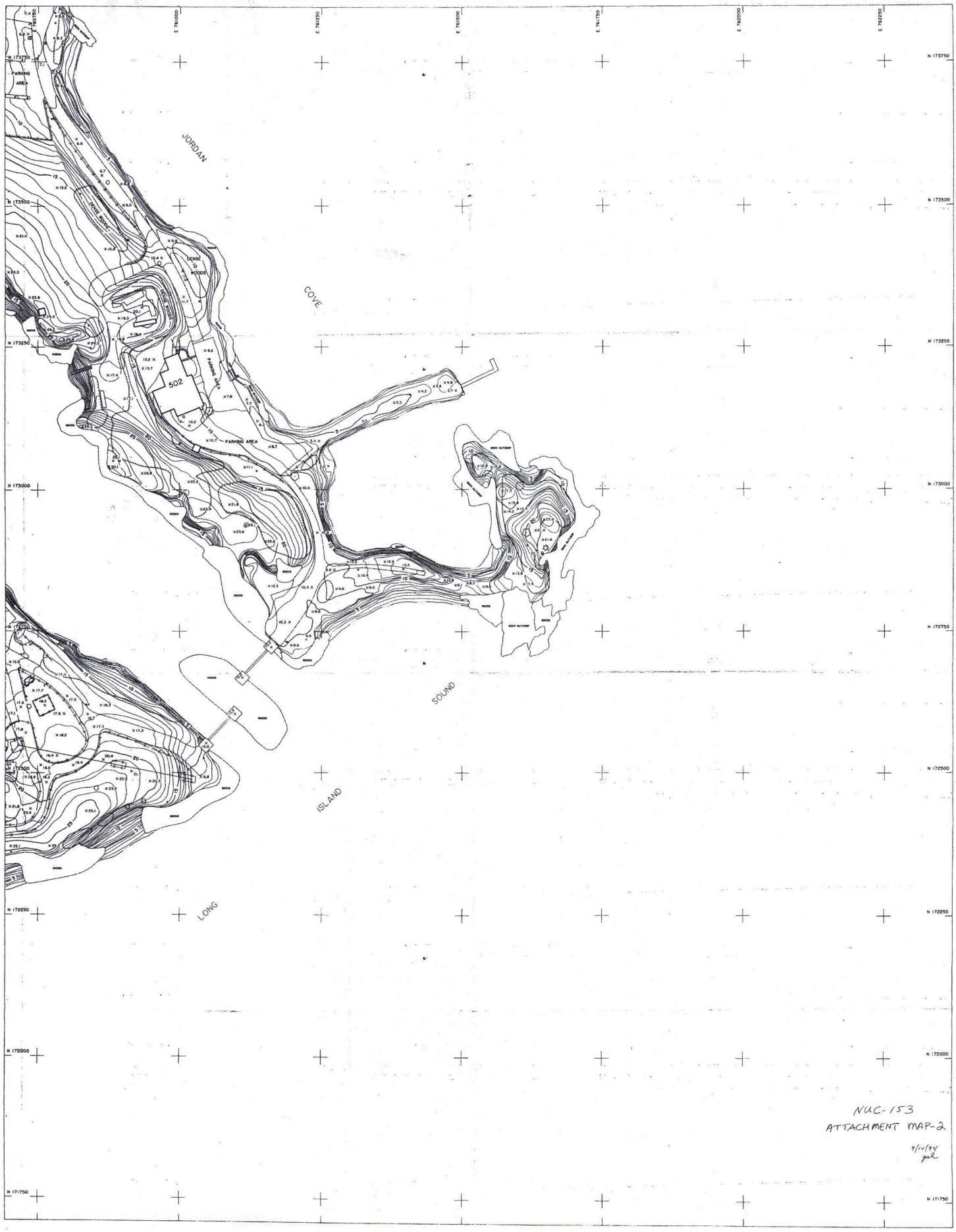
1991 PHOTOGRAMMETRY

CONTRACT REVISIONS

PROJECT No. 152-18 SCAL

PROJECT MILLSTONE POINT

DWG.No. 21582 - A V.S. R - 21




NUC-153
ATTACHMENT MAP-2
9/14/94
gal

**MILLSTONE STATION BUILDING NUMBERS**
AS OF 4/7/92

502 Environmental Lab

SCALE 1" = 50'

DATE OF PHOTOGRAPHY 12/8/91 CONTOUR INTERVAL ONE FOOT

250 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC.
PENNSAUKEN, NJ

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 20982A

SHEET 1 OF 8 SHEETS

THE CONNECTICUT LIGHT & POWER
BERLIN, CONN.

1991 PHOTOGRAMMETRY

PROJECT No. 152-18 SCALE: 1:

PROJECT MILLSTONE POINT

DWG. No. 20982 V.S. R-21-4


NUC-153
ATTACHMENT MAP-3

BUILDING STATION BUILDING NUMBERS
AS OF 1/31/92

| | | | |
|---|---|---|---|
| 100 | Gas Turbine | 203 | Unit 3 Discharge |
| 101 | Unit 1 Maintenance Office | 232 | 848 |
| 102 | Unit 1 Discharge | | |
| 103 | Unit 1 Maintenance Shop | | |
| 104 | Unit 1 Switchyard | 300 | Unit 3 Discharge |
| 105 | Unit 1 Turbine Building | | |
| 106 | Unit 1 Circulator Pit | | |
| 107 | Unit 1 Intake Structure | | |
| 108 | Unit 1 Emergency Power System Room | 400 | Hydrogen Cylinder Storage Area |
| 109 | Unit 1 Air Secondary Room | 401 | Fuel Farm |
| 110 | Unit 1/2 Engineering Offices | 404 | Control Record Holding Building |
| 111 | Unit 1 Reactor Building | 480 | Reactor Building PCN Bunkers |
| 112 | Distilled Water Storage Tank | | |
| 113 | Unit 1 Condensate Storage Tanks | | |
| 114 | Condensate Storage Tank | | |
| 115 | Waste Surge Tank | 500 | Sand Dam |
| 116 | Nitrogen-Oxygen Building | 516 | Steam Generator Mockup Building |
| | | 520 | 480 Power Equipment Enclosure |
| | | 536 | 848 Auto Gate Shelter |

SCALE 1" = 50'

50 0 50 100 150 200

DATE OF PHOTOGRAPHY 12/8/91 CONTOUR INTERVAL ONE FOOT

250 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC
PENNSAUKEN, NJ

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 2582A

SHEET 2 OF 8 SHEETS

THE CONNECTICUT LIGHT & POWER
BERLIN, CONN.

1991 PHOTOGRAMMETRY

PROJECT No. 152-18 SCALE: 1"

PROJECT MILLSTONE POINT

DWG. No. 2582 V.S. R-2-4


NUC-153
ATTACHMENT MAP-4
9/14/94
gah

**MILLSTONE STATION BUILDING NUMBERS**
AS OF 4/7/92

| | |
|---|---|
| 301 | Unit 3 Intake Structure |
| 408 | Watchhouse 6 |
| 410 | Ironworker Pub Shop |
| 412 | Rigging Loft |
| 423 | Warehouse 3 |
| 424 | Warehouse 4 |
| 427 | Main Office Complex |
| 431 | North Access Point (NAP) |
| 432 | Gas Bottle Storage Building |
| 433 | Chemistry Safety Storage Building |
| 458 | Site Engineering and Maintenance Building |
| 503 | A-Frame |
| 516 | Warehouse 6 |
| 535 | Chart Point 2 |
| 536 | Bay Point Beach Restrooms |

KEY MAP

SCALE 1" = 50'

DATE OF PHOTOGRAPHY 12/8/91 CONTOUR INTERVAL ONE FOOT

250 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC.
PENNSAUKEN, NJ

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 2582A

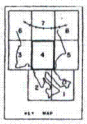
SHEET 3 OF 8 SHEETS

THE CONNECTICUT LIGHT & POWER C
BERLIN, CONN.

1991 PHOTOGRAMMETRY

| | |
|---|---|
| PROJECT No. 152-18 | SCALE: 1" |
| PROJECT MILLSTONE POINT | |
| DWG. No. 2582 | V.S. R-21-4 |


ATTACHMENT MAP-5




SCALE 1" = 50'

DATE OF PHOTOGRAPHY 12/6/90 CONTOUR INTERVAL ONE FOOT

2500 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA MEDIATION ASSOCIATES, INC.
PLYMOUTH MEETING, PA

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SHEET 4 OF 8 SHEETS

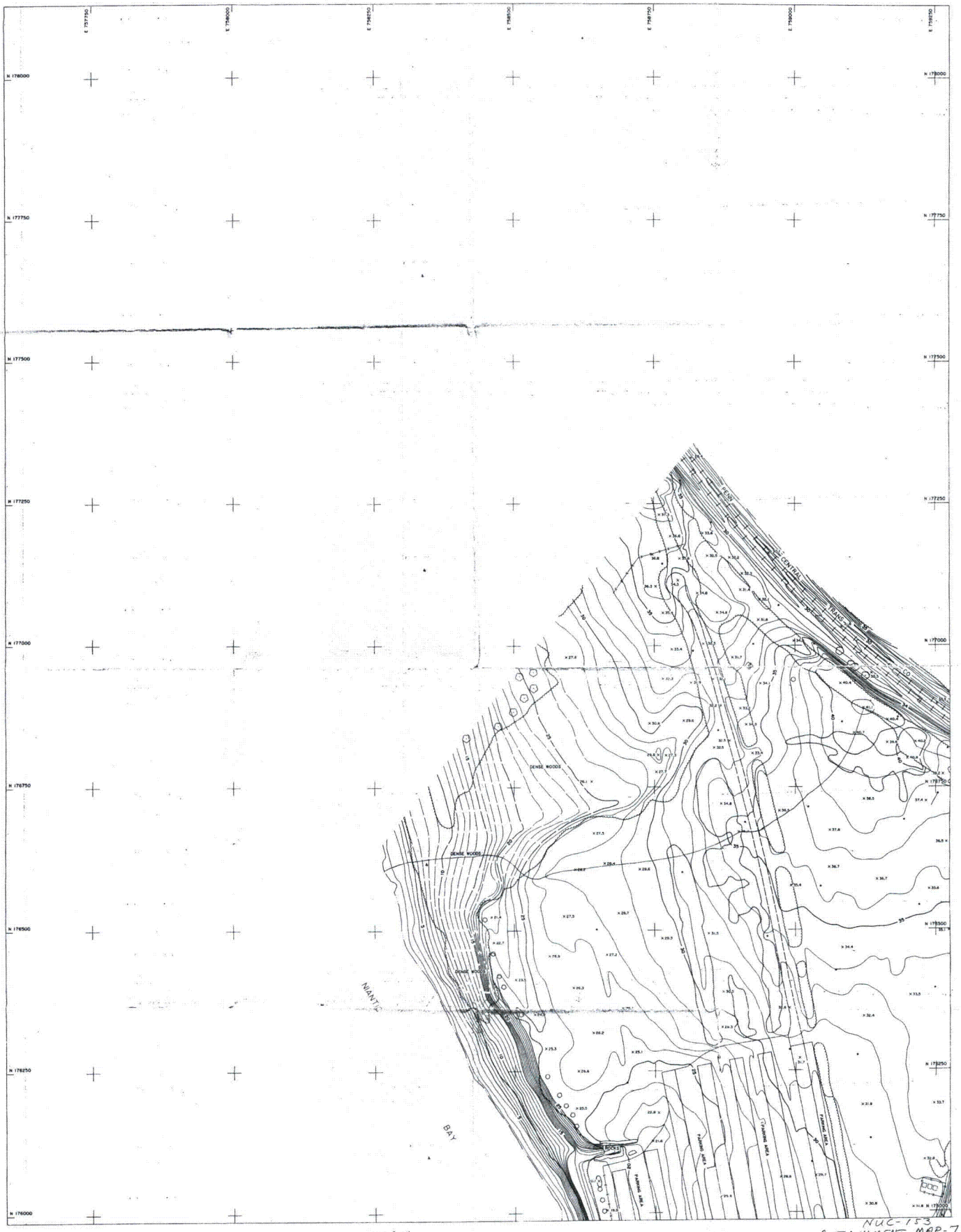
THE CONNECTICUT LIGHT & POWER
BERLIN, CONN.

1991 PHOTOGRAMMETRY

PROJECT No. 192-18 SCALE

PROJECT MILLSTONE POINT

DWG.No. 2582 V.S. R-25-4




NUC-153
ATTACHMENT MAP-7,

SCALE 1" = 50'

50 0 50 100 150 200

DATE OF PHOTOGRAPHY 12/8/81 CONTOUR INTERVAL ONE FOOT

250 FOOT BMS BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC
PENNSAUKEN, NJ

KEY MAP

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 2582A

SHEET 6 OF 8 SHEETS

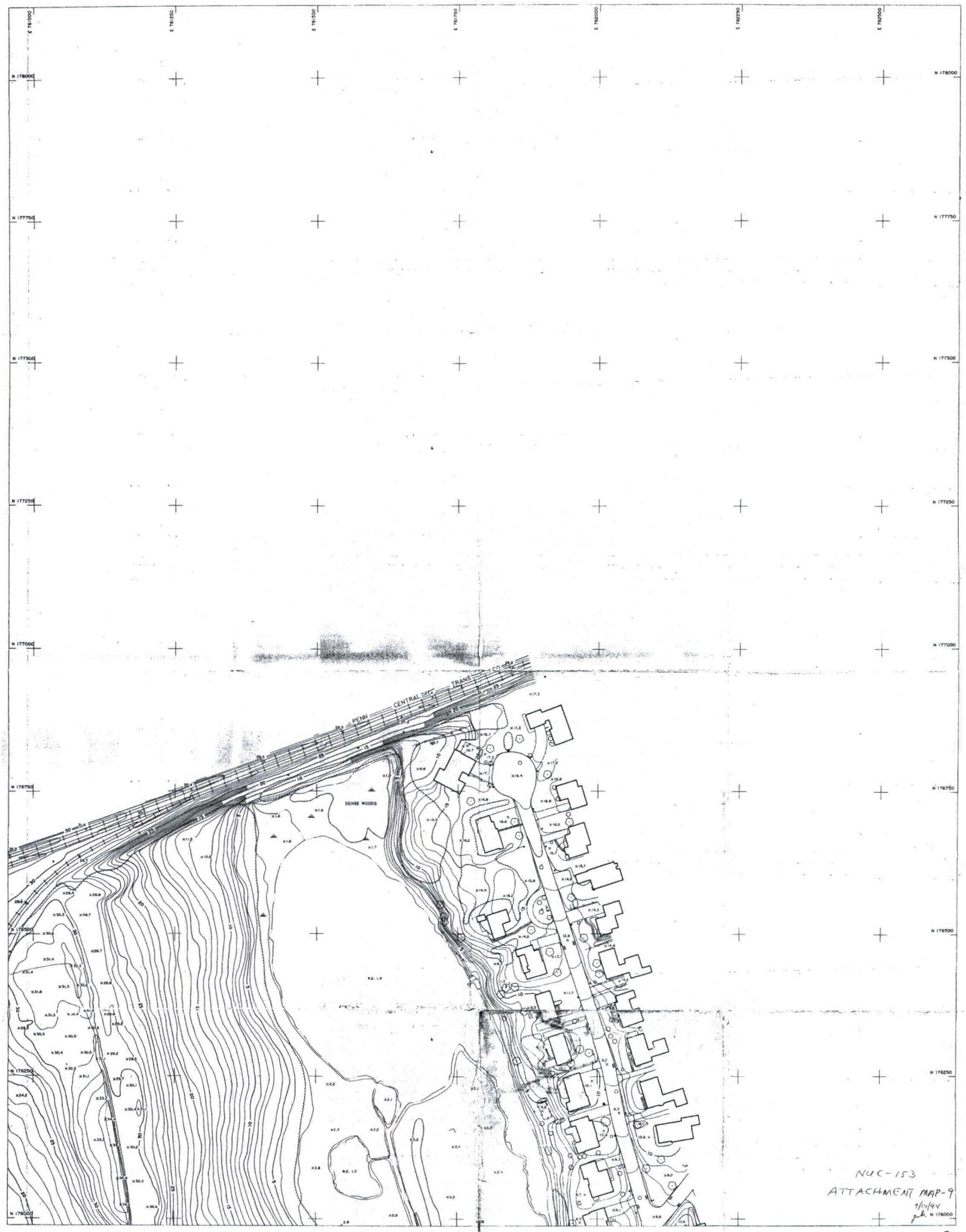
THE CONNECTICUT LIGHT & POWER C
BERLIN, CONN.

1991 PHOTOGRAMMETRY

PROJECT No. 182-18 SCALE 1"

PROJECT MILLSTONE POINT

DWG. No. 2582 V.S. R-25-4




NUC-153
ATTACHMENT MAP-9
9/14/94

SCALE 1" = 50'

DATE OF PHOTOGRAPHY 12/6/91 CONTOUR INTERVAL ONE FOOT

200 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC. PENNSAUKEN, N.J.

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 2582A

SHEET 6 OF 8 SHEETS

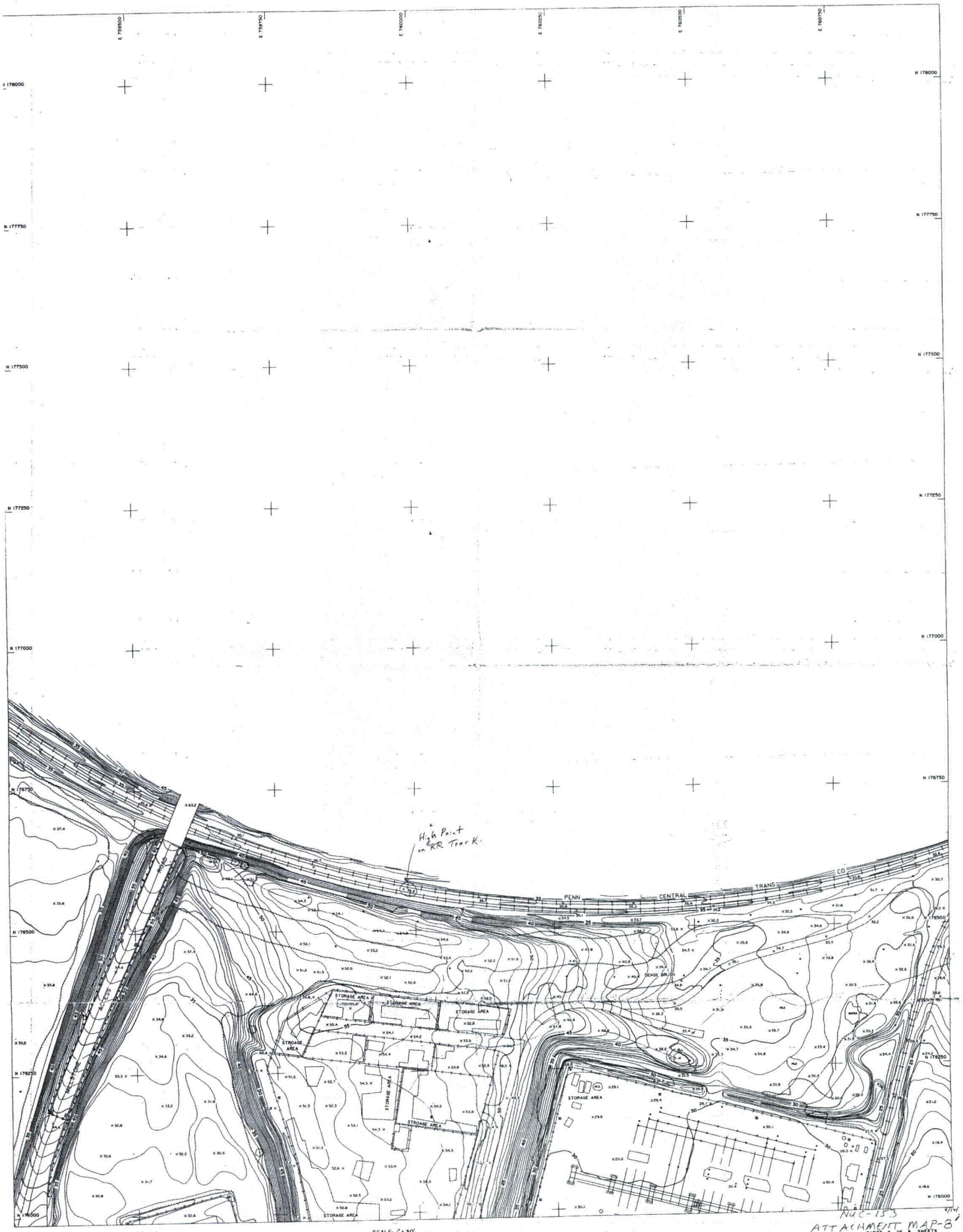
THE CONNECTICUT LIGHT & POWER C
BERLIN, CONN.

1991 PHOTOGRAMMETRY

PROJECT No. 152-1.0 SCALE-1

PROJECT - MILLSTONE POINT

DWG. No. 2582 V.S. R-2-4




ATTACHMENT MAP-8

SHEET 7 OF 8 SHEETS

SCALE: 1" = 50'

30 0 50 100 150 200

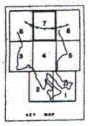
DATE OF PHOTOGRAPHY 12/8/91 CONTOUR INTERVAL ONE FOOT

250 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

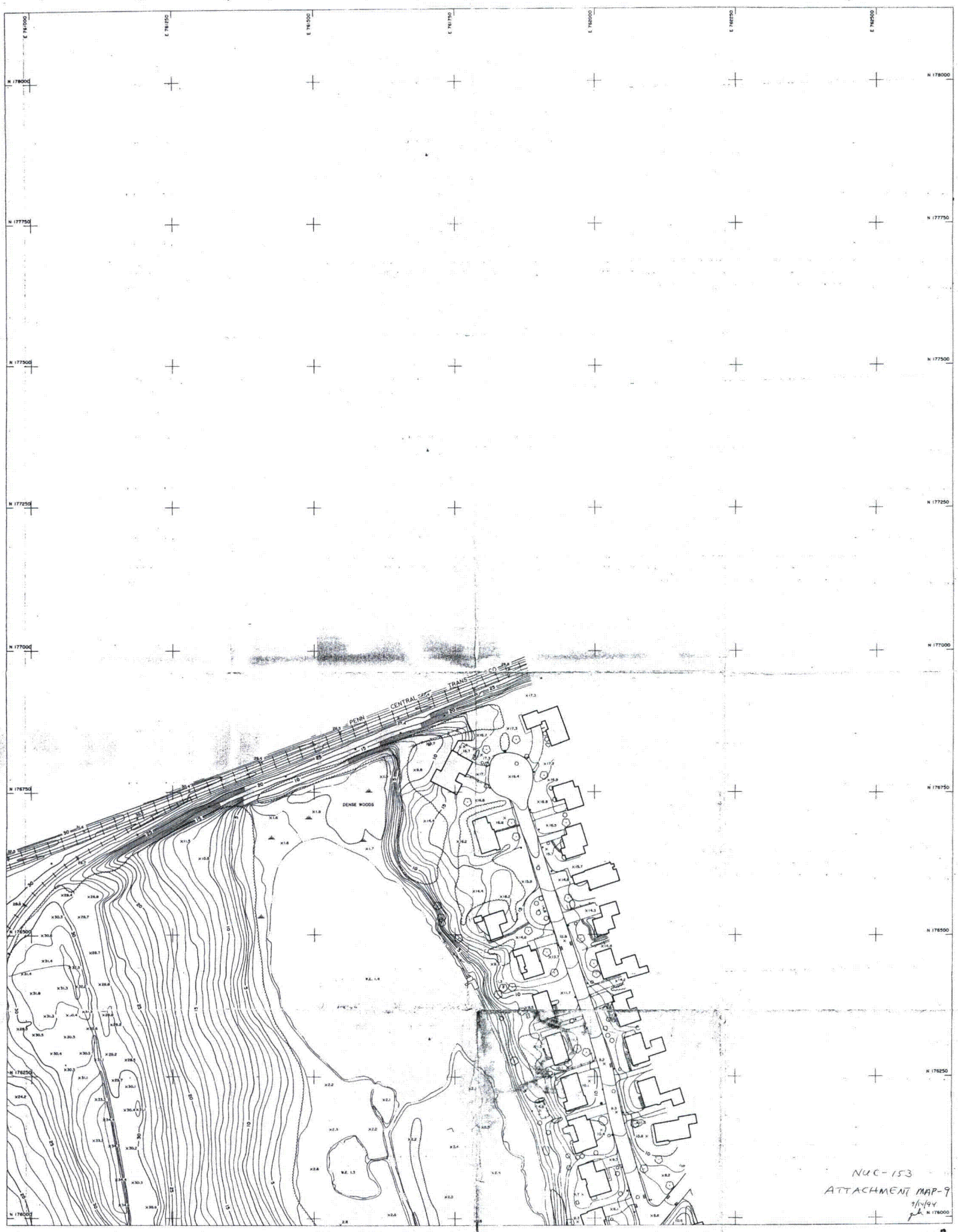
VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC. PENNSAUKEN, NJ

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 2082A.




| | |
|---|---|
| THE CONNECTICUT LIGHT & POWER BERLIN, CONN. | |
| 1991 PHOTOGRAMMETRY | |
| PROJECT No. 152-18 | SCALE: |
| PROJECT MILLSTONE POINT | |
| DWG. No. 2082 | V.S. R-2-4 |




SCALE 1" = 50'

20 0 50 100 150 200

DATE OF PHOTOGRAPHY 12/8/91 CONTOUR INTERVAL ONE FOOT

250 FOOT GRID BASED ON THE CONNECTICUT STATE PLANE COORDINATE SYSTEM.

VERTICAL DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

PREPARED BY AERIAL DATA REDUCTION ASSOCIATES, INC. PENNSAUKEN, NJ

For 200 Scale, 1991 Photogrammetry, See Dwg. No. 25824

KEY MAP

SHEET 8 OF 8 SHEETS

THE CONNECTICUT LIGHT & POWER C
BERLIN, CONN.

1991 PHOTOGRAMMETRY

DATE | REVISIONS

PROJECT No. 152-18 | SCALE 1"

PROJECT MILLSTONE POINT

DWG. No. 25582 | V.S. R-21-4

# Appendix D

# Extended Third-Party Review

GZA
GeoEnvironmental, Inc.

*Engineers and Scientists*

December 15, 2014
File No. 01.0171382.13

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
www.gza.com

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 calculation methodology by Dr. Donald T. Resio (Attachment 2). It is GZA's opinion that Dr. Resio's review of External Flood Hazard Re-Evaluation hurricane and surge calculation methodology used at Surry Power Station (SPS) is valid for hurricane and surge calculations for Millstone Power Station (MPS) because the analyses were performed by GZA in parallel using nearly identical methodological approaches. It is important to note that extension of this review to the MPS calculations is limited to methodology only, as Dr. Resio has not specifically reviewed the MPS calculations or the associated results. A summary of Dr. Resio's experience and qualifications is 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

GZA's calculations for MPS follow the identical process:

- Calculation No. 14-034, Rev. 0 – Probable Maximum Hurricane for Millstone Power Station
- Calculation No. 14-162, Rev. 0 – Deterministic Probable Maximum Storm Surge for Millstone Power Station
- Calculation No. 14-161, Rev. 0 – Probabilistic Storm Surge for Millstone 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. As previously indicated, the methodologies used to develop these calculations are consistent with the methodologies used to develop MPS Calculation No. 14-034, 14-162 and 14-161. In consideration of these consistencies, GZA also considers the peer reviews of the methodologies used to develop MPS Calculation No. 14-034, 14-162 and 14-161 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:**
**GZA GeoEnvironmental, Inc.**
**249 Vanderbilt Avenue**
**Norwood, MA 02062**
**POC: Michael Mobile**

*1 UNF Drive, Science & Engineering Building 50, Suite 3200, Jacksonville, Florida 32224*

# 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

# MILLSTONE NTTF 2.1: FLOODING HAZARD RE-EVALUATION INTERIM ACTIONS PLAN

DOMINION NUCLEAR CONNECTICUT, INC.
MILLSTONE POWER STATION UNITS 2 AND 3

| Item Number | Report Reference | Action Item List | Estimated Implementation Date |
|---|---|---|---|
| 1 | Section 4.1 | Verify procedures are in place to initiate FLEX strategies in response to a loss of ultimate heat sink if either one or both Millstone Power Station (MPS) Intake Structures becomes inoperable due to combined effects flooding. | June 30, 2015 |
| 2 | Section 4.2 | Review/revise, the applicable station abnormal weather and operational procedures for mitigation of a Beyond Design Basis (BDB) potential flooding event due to a local intense precipitation (LIP) event for MPS Unit 2 (MPS2). | June 30, 2015 |
| 3 | Section 4.3 | Revise applicable abnormal weather and operational procedures for mitigation of a BDB potential flooding event due to a tsunami for MPS2. | June 30, 2015 |
| 4 | Section 4.5 | Perform Integrated Assessment of the flood hazards for MPS2 and MPS3. | March 12, 2017 (may change based on guidance from the NRC) |