ATTACHMENT 1

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

March 2015

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

ZACHRY

Zachry Nuclear Engineering, Inc.

ENGINEERING EVALUATION
14-E16

DOMINION FLOODING HAZARD REEVALUATION REPORT FOR MILLSTONE POWER STATION UNITS 2 AND 3

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

REVISION 1

QA CLASSIFICATION: SAFETY RELATED

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REVISION HISTORY

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

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LIST OF ACRONYMS AND ABBREVIATIONS

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

Acronym/Abbreviation	Description
ft	feet
GPD	Generalized Pareto Distribution
gpm	Gallons per minute
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
HUC	Hydrologic Unit Code
HURDAT2	Hurricane Database (NOAA)
ISFSI	Independent Spent Fuel Storage Installation
ISG	Interim Staff Guidance (NRC)
JPM	Joint Probability Method
JPM-OS	Joint Probability Method – Optimum Sampling
kt	Knots
LIDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
MHW	Mean High Water
MHHW	Mean High High Water
MLW	Mean Low Water
MLLW	Mean Low Low Water
MPS	Millstone Power Station
MPS2	Millstone Power Station Unit 2
MPS3	Millstone Power Station Unit 3

Acronym/Abbreviation	Description
MSL	Mean Sea Level (Local Site Datum)
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NED	National Elevation Dataset
NGDC	National Geophysical Data Center
NGVD29	National Vertical Datum of 1929
NID	National Inventory of Dams
nm	Nautical miles
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTHMP	National Tsunami Hazard Mitigation Program
NTTF	Near-Term Task Force
NWS	National Weather Service
OBE	Operating Basis Earthquake
PA	Protected Area
PDF	Probability Density Function
PDH	Probability Density Histogram
PMF	Probable Maximum Flood
PMH	Probable Maximum Hurricane
PMP	Probable Maximum Precipitation
PMS	Probable Maximum Seiche
PMSS	Probable Maximum Storm Surge

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

INTRODUCTION

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

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

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

- To gather information with respect to NTTF Recommendation 2.1, as amended by staff requirements memoranda (SRM) associated with SECY -11-0124 and SECY -11-0137, and the Consolidated Appropriations Act, for 2012 *(Pub Law* 112-74), Section 402, to reevaluate seismic and flooding hazards at operating reactor sites

- To collect information to facilitate NRC's determination if there is a need to update the design basis and systems, structures, and components (SSCs) important to safety to protect against the updated hazards at operating reactor sites

- To collect information to address Generic Issue (GI) 204 regarding flooding of nuclear power plant sites following upstream dam failures

Millstone Power Station (MPS), located in Waterford, Connecticut, is one of the sites being required to submit information compliant with this RFI. MPS consists of two operational units (Units 2 and 3) and one unit that is decommissioned (Unit 1). In response to the RFI this Report is being generated to address NTTF Recommendation 2.1 with respect to Units 2 and 3 only.

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

MPS has adopted the U.S. Coast & Geologic Survey Mean Sea Level (MSL) datum as its reference for elevations. The MSL datum may also be referred to as the National Geodetic Vertical Datum of 1929 (NGVD 29). Parts of this report may mention the North American Vertical Datum of 1988 (NAVD 88). The difference in Elevation between these datums at the site is 0.99 ft (NAVD88 0.00 ft = 0.99 ft MSL), or 1.0 ft (conservatively). Unless specified, this report will refer to all elevations in MSL.

Section 1 of this report provides detailed site information related to flooding hazards. Section 2 includes the reevaluation of flood hazards for each reevaluated flood causing mechanism. A

comparison of the reevaluated flood hazards to those in the current licensing basis is provided in Section 3. Interim flood protection measures and actions for higher flood hazards identified are summarized in Section 4. Additional Actions required are concluded in Section 5. There are four Appendices attached, three of which are discussed in Section 2. The fourth Appendix, Appendix D, is an extended third party review of the hurricane/storm surge calculations performed in support of this Evaluation.

Introduction References

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

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

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

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

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

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

7. **Dominion 2013**. Dominion Engineering Technical Evaluation ETE-MP-2013-1178, Rev. 0 "Documentation of Vertical Datum Point for Millstone Power Station", June 2013

1.0 SITE INFORMATION RELATED TO FLOODING HAZARDS

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

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

The requested information is presented in Sections 1.1 through 1.6.

1.1. Detailed Site Information

MPS is located in the town of Waterford, New London County, Connecticut, on the north shore of Long Island Sound, as shown in Figure 1.1-1. The 524-acre site occupies the tip of Millstone Point between Niantic Bay on the west and Jordan Cove on the east and is situated 3.2 miles west-southwest of New London, Connecticut and 40 miles southeast of Hartford, Connecticut. The surrounding area is primarily residential with some commercial and industrial uses.

In 2001, Millstone Units 1, 2 and 3 operating licenses were transferred from Northeast Nuclear Energy Company to Dominion Nuclear Connecticut, Inc. (DNC). The transmission and distribution assets on the site are owned by Connecticut Light and Power (CL&P) and are operated under an Interconnection Agreement between CL&P and DNC. The exclusion area boundary and property line for the site are shown on Figure 1.1-2. DNC has complete control of all activities within the exclusion area, except for the passage of trains along the Providence & Worcester (P&W)/Amtrak Railroad track which runs east-west through the site. Immediately adjacent to the site is the Millstone Point Colony development, a residential area. Additionally a portion of the exclusion area is leased to the Town of Waterford for public recreation and is used primarily for soccer and baseball games.

The topography around Millstone is marked by low rolling hills rising inland from the shoreline. The maximum height of the surrounding terrain within 5 miles of the site is about 250 feet above mean sea level (MSL) at 3.2 miles to the north-northwest. To the south of the site, from east through west, is open water. Both MPS Unit 2 and Unit 3 (MPS2 & MPS3) have different site elevations and different finish grade elevations. MPS2 has a site grade elevation of 14 feet, and the associated SSCs and road elevations near MPS2 have a finish grade elevation of 14.5 feet. MPS3 has a site grade elevation of 24 feet, and the associated SSCs have a finish grade elevation of 24.5 feet; with the exception of the circulating and service water pump house. These SSCs are located at an operating grade elevation of 14 feet. The site plan is shown in Figure 1.1-3 and the site topography, in 2 foot elevation contours, is provided in Figure 1.1-4.

The groundwater environment at the MPS site is characterized by generally impermeable bedrock acquicludes overlain by soil masses of varying permeabilities. There is an abandoned granite quarry located on the southeast side of Millstone Point. The bedrock is mostly Monson gneiss, with a Westerly granite dike intruding the gneiss in the quarry area. Neither rock is permeable, and there appears to be little movement of water through fissures in either formation since the quarry did not fill with either fresh or salt water after its abandonment in 1960. Locally perched water table conditions occur in some areas of soil stratification and shallow ponded water is frequently visible in localized bedrock troughs. Additionally some surface water collects in depressions in the marshy areas north of the site. In Section 2, Figure 2.2-1 provides a plot of the small coastal stream watershed on the site.

Normal tides at Millstone Point are semidiurnal with a mean range of 2.7 feet and a spring range of 3.2 feet. Tides in excess of the mean high water occur on an average as follows: in excess of 3 feet about once a year, in excess of 2 feet about 5 times a year, and in excess of 1 foot about 98 times a year. Mean high water (MHW) at Millstone Point is 1.3 feet MSL. Mean low water (MLW) is -1.4 feet MSL.

1.1.1. References

1.1.1-1 **Dominion 2014a**. Millstone Power Station Final Safety Analysis Report (MPS2 FSAR), Rev. 30.2

1.1.1-2 **Dominion 2014b**. Millstone Power Station Final Safety Analysis Report (MPS3 FSAR), Rev. 25.2

1.1.1-3 **ESRI, 2013**. "National Geographic World Map" online map service, last revised December 7, 2013

1.1.1-4 **ESRI, 2014**. "Ocean Basemap" online map service, last revised March 4, 2014

1.1.1-5 **Zachry 2014a**. Zachry Calculation 13-113, Rev. 0, "Probable Maximum Precipitation (PMP) at Millstone Power Station"

1.1.1-6 **Zachry 2014b**. Zachry Calculation 13-114, Rev. 0, "Evaluation of Local Intense Precipitation-Induced Flood at Millstone Power Station"

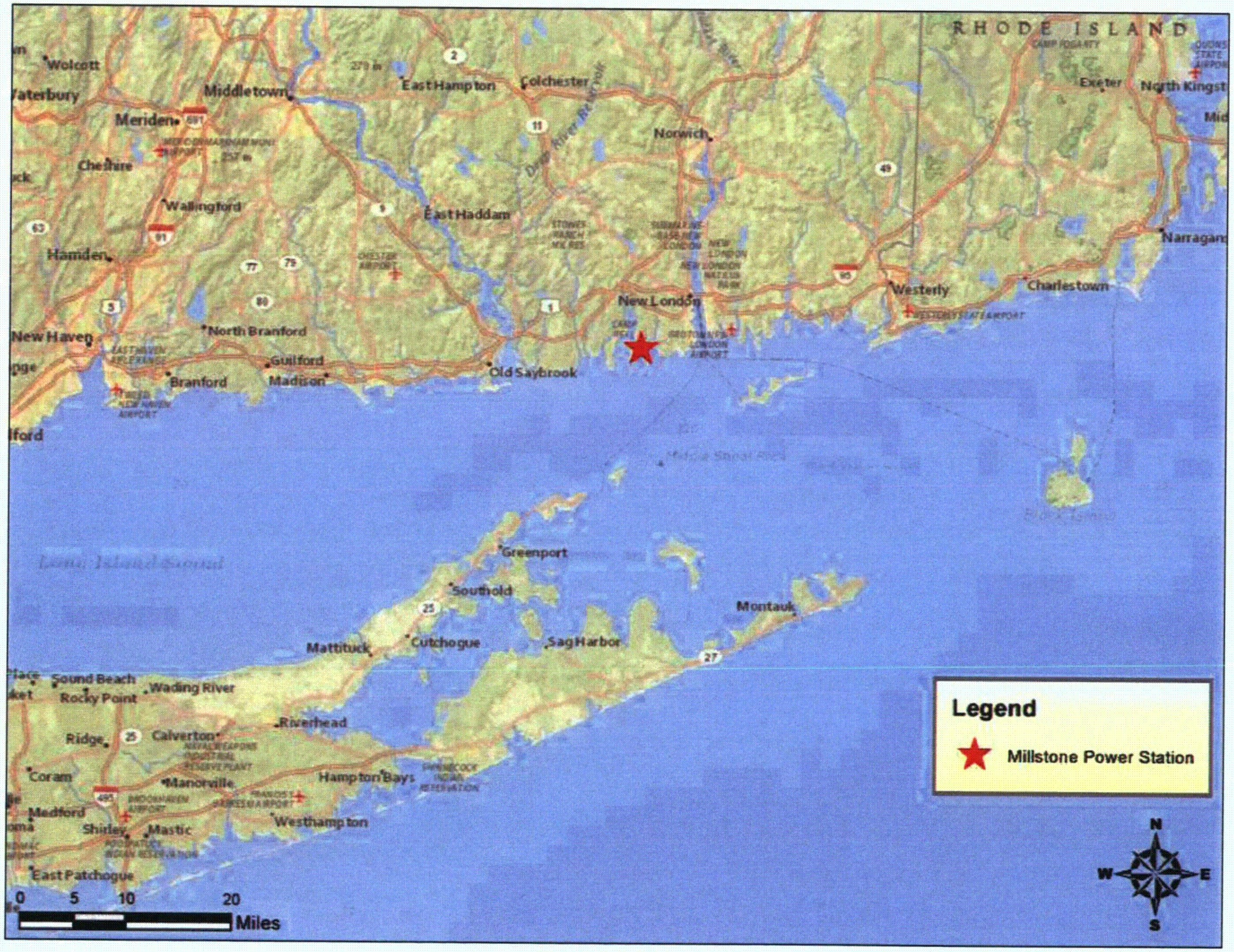

Figure 1.1-1: Site Locus Map (ESRI 2013 and ESRI 2014)

MPS-3 FSAR

FIGURE 2.1–3 SITE LAYOUT

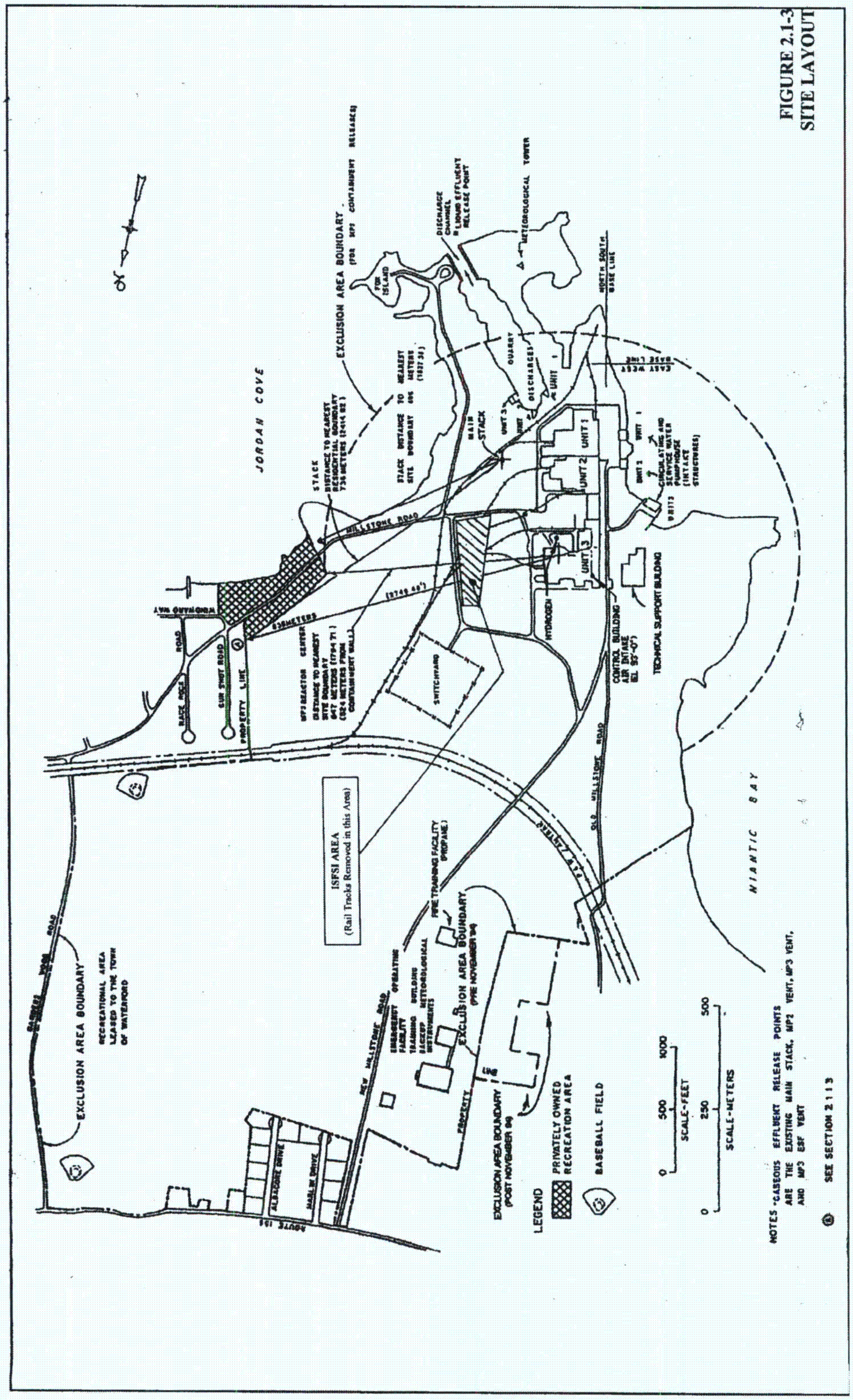

Figure 1.1-2: Site Layout (Dominion 2014b)

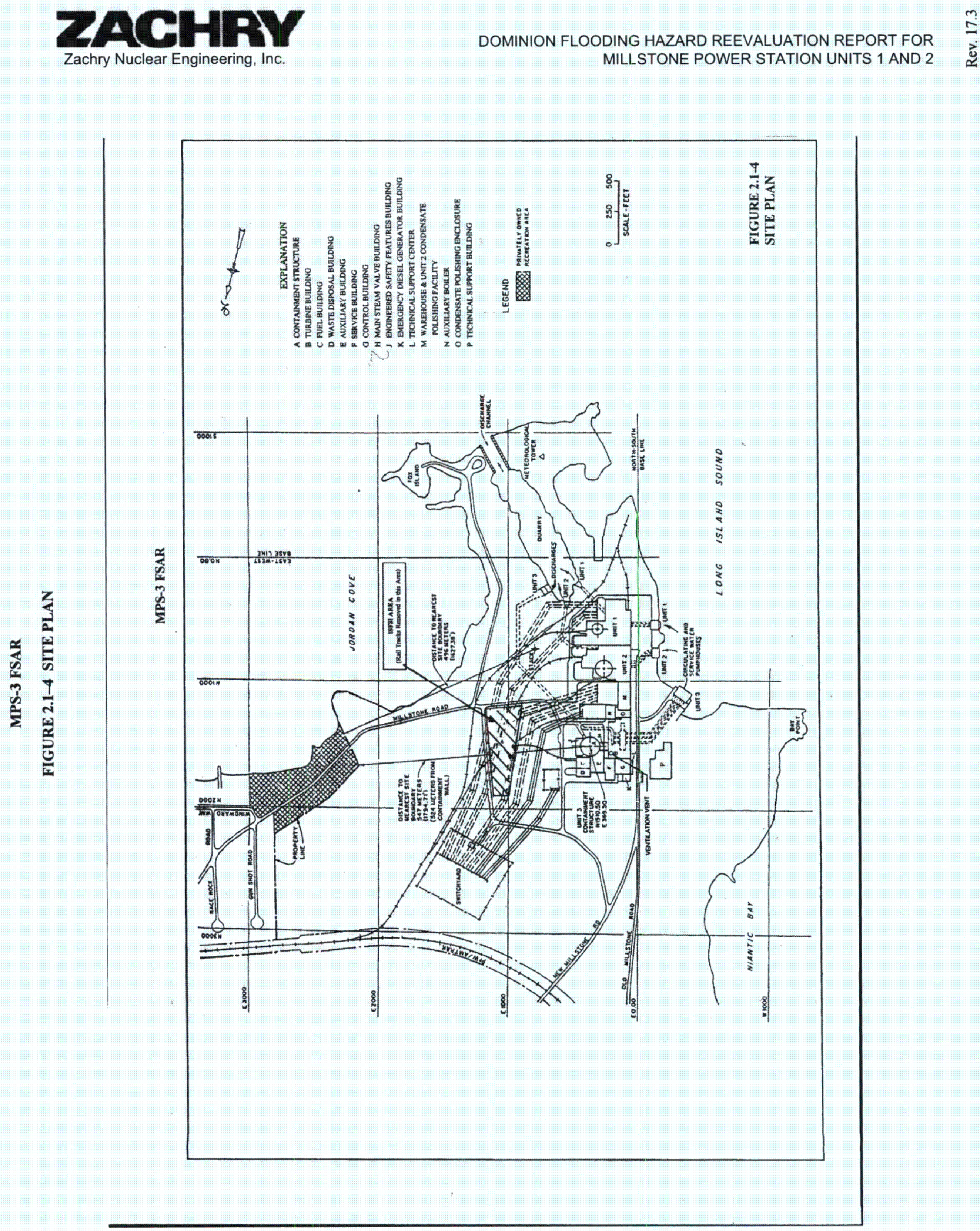

Figure 1.1-3: Site Plan (Dominion 2014b)

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

1.2. Current Design Basis Flood Elevations

As discussed in the Flooding Walkdowns Results Report, related to NTTF Recommendation 2.3; MPS2 was designed and constructed to the draft General Design Criterion (GDC) 2 dated July 11, 1967, which stated:

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

Prior to issuance of the MPS2 operating license, the design and construction of the nuclear power facility was evaluated against the newer GDC 2. MPS2 FSAR, Appendix 1A, states that, "All structures, systems, and components important to safety have been designed to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena," and that, "Appropriate natural phenomena are considered in the designs of structures, systems, and components." Based on this evaluation, MPS2 identified no exceptions to the current GDC 2 with respect to flooding. MPS2 was confirmed to comply with the current GDC 2 for natural phenomena related to flooding.

The MPS2 Current Design/Licensing Basis flood elevations for the different flood causing mechanisms are addressed in the MPS2 FSAR, Chapter 2, Section 2.5. The MPS2 FSAR refers to the MPS3 FSAR for all flooding events, with the exception of storm surge and a general discussion of a maximum rainfall event. The current flood elevations for MPS2 are summarized in Table 1.2-1.

A review and reevaluation of external flooding for MPS2 was performed in response to NRC Generic Letter GL 88-20 "Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities, and resolution of Generic Issue GI-103, Design for Probable Maximum Precipitation (PMP)." This IPEEE evaluation was beyond design basis and is not included in the CLB for MPS2.

MPS3 was designed, constructed, and licensed in accordance with the current GDC 2 which states: "Systems, structures, and components important to safety shall be designed to withstand the effects of natural phenomena, such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without the loss of capability to perform their safety functions. The design basis for these structures, systems, and components shall reflect:

> 1) Appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the

limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2) Appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena.

3) The importance of the safety functions to be performed.

The MPS3 Current Design/Licensing Basis flood elevations for the different flood causing mechanisms are addressed in the MPS3 FSAR, Chapter 2, Section 2.4. The MPS3 FSAR discusses flooding potential due to storm surge (controlling event), local intense precipitation, probable maximum flood on streams and rivers, potential dam failures, probable maximum seiche, wave action, clapotis on the intake structure, tsunami flooding, ice effects, and channel diversion. The current flood elevations for MPS3 are summarized in Table 1.2-2.

1.2.1. References

1.2.1-1 **Dominion 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

1.2.1-2 **Dominion 2014a**. Millstone Power Station Final Safety Analysis Report (MPS2 FSAR), Rev. 30.2

1.2.1-3 **Dominion 2014b**. Millstone Power Station Final Safety Analysis Report (MPS3 FSAR), Rev. 25.2

1.2.1-4 **Dominion 1994a**. Millstone Calculation No. NUC-152, Rev. 0, "Millstone IPEEE External Flooding (Site Design Basis)", for Millstone Units 1 and 2

1.2.1-5 **Dominion 1994b**. Millstone Calculation No. NUC-153, Rev. 0, "Millstone IPEEE External Flooding (Local Intense PMP)", for Millstone Units 1 and 2

Table 1.2-1: Current Design Basis Flood Elevations for MPS2

Flooding Mechanism	Design Basis Flood Level (MSL)	FSAR Section
Storm Surge (including wave-runup)	21.8 ft	2.5.4.2 (MPS2)
Local Intense Precipitation	14.5 ft	2.5.4.2.2 (MPS2)
Tsunami (including wave runup)	No flooding expected	2.4.6 (MPS3)[1]
Flooding in Streams and Rivers	No flooding expected	2.5.3 (MPS2)
Dam Failures	No flooding expected	2.5.3 (MPS2)
Seiche	No flooding expected	2.4.5 (MPS3)[1]
Ice Induced Flooding	No flooding expected	2.4.7 (MPS3)[1]
Channel Migration or Diversion	No flooding expected	2.4.9 (MPS3)[1]

Note 1: The MPS2 FSAR refers the reader to the MPS3 FSAR

Table 1.2-2: Current Design Basis Flood Elevations for MPS3

Flooding Mechanism	Design Basis Flood Level (MSL)	FSAR Section (MPS3)
Storm Surge (including wave-runup)	23.8 ft	2.4.2.2
Local Intense Precipitation	Refer to Table 1.2-3	2.4.2.3
Tsunami (including wave runup)	No flooding expected	2.4.6
Flooding in Streams and Rivers	No flooding expected	2.4.3
Dam Failures	No flooding expected	2.4.4
Seiche	No flooding expected	2.4.5
Ice Induced Flooding	No flooding expected	2.4.7
Channel Migration or Diversion	No flooding expected	2.4.9

Table 1.2-3: Computed Water Surface Elevations at Safety-Related Structures due to LIP Effects at MPS3

Structure	Maximum Water Surface Elevations (ft)	Associated Drainage Basin[1]
Auxiliary Building	24.85	C
Control Building	24.27	C
Emergency Generator Enclosure	24.27	C
Main Steam Valve Building	24.85	D
Hydrogen Recombiner Building	24.85	D
Auxiliary Building	24.85	D
Engineered Safety Features Building	24.85	D
Fuel Building	24.85	D
RWST/SIL Valve Enclosure	24.85	D
Demineralized Water Storage Tank Block House	24.85	D

Note 1: The drainage Basins are defined as shown in Figure 2.4-7 located in the MPS3 FSAR. They are included here for information only.

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

There has been no change to the CLB flooding elevations beyond what is described in the MPS FSARs. The Flooding Walkdowns Results Report identified some deficiencies with the current flood protection at the site. These deficiencies included material condition, such as component rusting, degraded gaskets, seals, and weather stripping (majority) and some due to configuration management, such as below grade unsealed penetrations. These deficiencies were addressed by Condition Reports (CRs) and were entered into the Corrective Action Program. Other flood protection features were found available, functional, and fairly well maintained. Any new flood protection measures planned will be addressed by the licensee after review of this Flooding Hazard Reevaluation Report.

1.3.1. References

1.3.1-1 **Dominion 2014a**. Millstone Power Station Final Safety Analysis Report (MPS2 FSAR), Rev. 30.2

1.3.1-2 **Dominion 2014b**. Millstone Power Station Final Safety Analysis Report (MPS3 FSAR), Rev. 25.2

1.3.1-3 **Dominion 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

1.4. Changes to the Watershed and Local Area

The local area is discussed in Section 1.1. While there have been some inevitable small local area changes due to population growth, there have been no site changes pertinent to flood hazards. Additionally, there have been no major changes to the small coastal watershed near Millstone that negatively affect flooding hazards.

1.4.1. References

1.4.1-1 **Dominion 2014a**. Millstone Power Station Final Safety Analysis Report (MPS2 FSAR), Rev. 30.2

1.4.1-2 **Dominion 2014b**. Millstone Power Station Final Safety Analysis Report (MPS3 FSAR), Rev. 25.2

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

As discussed in Section 1.2 there are multiple flooding events considered that could cause external flooding to the site.

For MPS2, two scenarios are considered to produce external flooding on site. The first is storm surge due to PMH; which creates a maximum stillwater elevation of 18.1 feet and a maximum wave runup of 21.8 feet (Discharge Structures); and a standing wave inside the intake structure of elevation 26.5 feet. The second is flooding due to a maximum rainfall event; which accumulates in the yard, until it reaches 14.5 feet and overtops the road.

Similar to MPS2, MPS3 has two scenarios that are considered to cause external flooding on site. The first, storm surge, is expected to create a maximum stillwater elevation of 19.7 feet, and a maximum wave runup of 23.8 feet. The second, Probable Maximum Precipitation, is computed and expected to cause a water elevation of 24.27 feet for the Control Building and Emergency Generator Enclosure and 24.85 feet for other safety-related structures.

1.5.1. MPS2 Flood Protection and Mitigation Features (Summarized from the Flooding Walkdowns Results Report)

Probable Maximum Hurricane Event Protection

The containment building, auxiliary building and the warehouse building have exterior concrete walls up to elevation 54.5 ft minimum, which is above projected flooding levels and are therefore protected. The turbine building and enclosure building have metal siding above elevation 22 ft. The metal siding is continuous over the exterior flood wall and is connected to the flood wall with waterproof caulked connections. This siding prevents water, resulting from splashing effects, from entering the building.

Openings into the auxiliary and turbine buildings are provided with hinged flood gates or stop logs to elevation 22 ft to insure tightness from water and debris. Flood gates, with sealing compressive membranes, are used wherever possible. If the gates at an opening pose an operational encumbrance, stop logs with equivalent sealing capability are used in those locations.

At the interface between MPS2 and Millstone Power Station Unit 1 (MPS1), the flood protection capability of MPS1 was originally credited in the evaluation of MPS2 flood protection capability. The entire periphery of MPS1 was flood protected. However, with the decommissioning of MPS1, the flood boundary was revised to support the decommissioning effort. A flood wall is provided to a minimum elevation of 22 feet along the common area between MPS1 and the MPS2 Turbine Building. Protection for the MPS2 Auxiliary Building is provided by the adjacent MPS1 Control Building which is provided with flood protection on the south and east walls to a minimum elevation of 22 feet.

Intake Structure Flood Protection

The intake structure is constructed of reinforced concrete with an invert elevation of -27 feet, operating deck at elevation 14 feet, and a cutoff wall to elevation -10 feet. The only safety-related system in the intake structure is the service water system. The service water pump motors and associated electrical and control equipment are protected to elevation 22 ft.

Provisions for protecting one service water pump motor would be implemented by plant personnel for flood levels above elevation 22 ft. This would be accomplished by de-termination of electrical connections from one of the MPS2 service water pump motors and installation of a cover to protect the motor from potential flood waters in the intake structure.

Rainfall Event Protection

The drainage system throughout the MPS2 site, including the roof and yard drainage, is designed based on an intensity of three inches per hour rainfall. Once this is exceeded, excess runoff will be accumulated in the yard area until it reaches elevation 14.5 ft and will overtop the road. Water will then flow out into Jordan Cove on the east and Niantic Bay on the west. Drain connections from the buildings to the MPS2 storm drain system are provided with backwater valves, preventing backflow of water into the buildings; also, stop logs and flood gates are provided at the entrances to prevent water from flowing in.

Flood Protection of Electrical Equipment

Although the site is protected to 22 ft, in the case of water ingress into the plant, there are some essential electrical MCCs, switchgear and panels located below elevation 22 ft. Essential equipment is located in close proximity to stairways or other openings to lower floors to reduce the risk of accumulation of water at these locations. Where this is not the case, the equipment is mounted on a four inch raised concrete (housekeeping) pad.

Power and control cables, cable terminations, and electrical devices required for the trouble-free operation of the service water pumps in the intake structure are located above elevation 22 ft, where possible. Any of these located below this point are of watertight construction.

Entry of cables connecting outdoor equipment to equipment within the flood protected areas is so designed to preclude leakage or overflow of flood water into these areas by provision of proper seals and/or by carrying cable raceways to elevations at or above 22 ft.

1.5.2. MPS3 Flood Protection and Mitigation Features (Summarized from the Flooding Walkdowns Results Report)

Flood Protection Measures for Seismic Category I Structures

MPS3 safety-related structures and equipment, except the circulating and service water pumphouse, are protected from flooding by the site grade of elevation 24 ft. Within the Service Water pumphouse, each pair of service water pumps and pump motors are located at elevation 14.5 ft inside individual watertight cubicles, which are protected up to elevation 25.5 ft.

Accesses to safety-related structures and facilities are at an elevation of 24.5 feet above the nominal site grade elevation of 24 feet and are consequently protected from flooding due to groundwater, storm surge, and direct rainfall, except for the doors that are discussed in MPS3 Service Water FSAR Section 2.4.2.3. The two access openings to the service water cubicles inside the pumphouse are fitted with watertight steel doors capable of withstanding the maximum hydrostatic load occurring at their respective locations. Equipment access openings on the pumphouse roof over the service water cubicles are fitted with watertight covers. During normal plant operation, the service water cubicles have open drain lines installed in the cubicle sump to enable the service water pump seal water leak off to drain directly into the intake

structure pump bay. During severe weather or flooding conditions, the drain lines are isolated and the service water cubicle sumps are drained using portable sump pumps.

Some additional features are summarized below:

- Foundations of safety-related structures are constructed of reinforced concrete and subgrade joints between walls and slabs are sealed with waterstops cast in the concrete.
- The storm drain system uses catch basins and underground piping and/or drainage ditches to convey runoff to Niantic Bay.
- The seaward wall of the intake structure is constructed of reinforced concrete, designed to withstand the forces of a standing wave, or clapotis, with a maximum crest elevation of 41.2 feet.
- The pumphouse floor is designed to withstand uplift pressure due to wave action and storm surge.
- The discharge outfall structure is also designed to withstand maximum wave forces induced by the most critical combination of wave action and storm surge during PMH conditions.

Flood Protection from PMP Event

Site ground elevation surrounding buildings is elevation 24.0 ft with the safety related building entrances and ground level floors set at elevation 24.5 ft except the Demineralized Water Storage Tank (DWST) Block House and Refueling Water Storage Tank (RWST) Valve Enclosure. The entrance elevation for the DWST Block House is elevation 24.33 ft with ground level floor set at elevation 24.0 ft and the entrance and ground level floor for the RWST Valve Enclosure is set at elevation 24.33 ft. The yard area north of the control building and the waste disposal building is depressed below elevation 24.0 ft to create a swale to drain the PMP flood flow.

The computed water surface elevations at the safety related structures are elevation 24.27 ft for the Control Building and Emergency Generator Enclosure and elevation 24.85 ft for the other safety-related structures. Per MPS3 FSAR Section 2.4.2.3, accumulating precipitation runoff is considered to slow down and backup causing water to enter the safety related structures with the exception of the Control Building and the Emergency Generator Enclosure. The quantity and location of water calculated to enter these buildings has been evaluated and determined to not interfere with safety related equipment inside these structures.

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

MPS Common Operating Procedure

Millstone Procedure C OP 200.6, *Storms and Other Hazardous Phenomena (Preparation and Recovery)*, provides guidance to shift managers, department managers, station officers, and emergency response personnel for preparation, response, and recovery from significant storms

and other hazardous phenomena. This procedure may be implemented by a unit shift manager, station director or officer when a natural phenomenon or external condition poses or is predicted to pose a situation that threatens the safety of the plant and/or a situation that significantly hampers site personnel in performance of duties necessary for safe operation of the plant.

MPS2 Site-Specific Abnormal Operating Procedures

Weather service forecasts provide adequate hurricane status such that there will be sufficient time to secure the plant against flooding. Securing the plant will be done at the discretion of the shift supervisor and can be accomplished by one man who would close and lock the hinged gates and install the flood stop logs. The flood stop logs are specifically designated as pieces of flood protection equipment and stored in the vicinity where they will be used.

During a hurricane, the plant operations will be in accordance with normal, abnormal and/or emergency operating procedures. Procedure AOP 2560, *Storms, High Winds and High Tides*, provides actions to place MPS2 in a safe condition during a severe storm, high winds, or high tides. This procedure includes direction to de-terminate the electrical connections from one of the MPS2 service water pump motors and install a cover to protect the motor from potential flood waters in the intake structure.

The emergency shutdown equipment, including the auxiliary power sources are flood protected.

MPS3 Site-Specific Abnormal Operating Procedures

For the Circulating and Service Water pumphouse, Procedure AOP 3569, *Severe Weather Conditions*, addresses safety measures to be taken in the case of severe weather conditions. These measures ensure that watertight doors are in place and the pump cubicle sump drain lines are isolated and thus safety-related structures and components are protected from flooding.

Corporate Hurricane Response Plan

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

1.5.4. References

1.5.4-1 **Dominion 2014a**. Millstone Power Station Final Safety Analysis Report (MPS2 FSAR), Rev. 30.2

1.5.4-2 **Dominion 2014b**. Millstone Power Station Final Safety Analysis Report (MPS3 FSAR), Rev. 25.2

1.5.4-3 **Dominion 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

1.5.4-4 **Dominion 2014c**. Millstone Procedure C OP 200.6, "Storms and Other Hazardous Phenomena (Preparation and Recovery)", Revision 003-01

1.5.4-5 **Dominion 2014d**. Millstone Unit 2 Procedure AOP 2560, "Storms, High Winds and High Tides", Rev. 010-17

1.5.4-6 **Dominion 2014e**. Millstone Unit 3 Procedure AOP 3569, "Severe Weather Conditions", Rev. 20

1.5.4-7 **Dominion 2014f**. Dominion Procedure, CO-PROC-OOO-HRP-NUCLEAR, "Hurricane Response Plan (Nuclear)", Rev. 12

1.6. Additional Site Details

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

2.0 FLOODING HAZARD REEVALUATION

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

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

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

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

- Zachry Calculation 13-113, Rev. 0, “Probable Maximum Precipitation (PMP) at Millstone Power Station”
- Zachry Calculation 13-114, Rev. 0, “Evaluation of Local Intense Precipitation-Induced Flood at Millstone Power Station”
- Zachry Calculation 13-115, Rev. 0, “Probable Maximum Flood (PMF) at Millstone Power Station”
- Zachry Calculation 13-116, Rev. 0, “Dam Failure Analysis”
- Zachry Calculation 13-132, Rev. 0, “Probable Maximum Tsunami (PMT) at Millstone Power Station (Regional and Site Screening Analysis)”
- Zachry Calculation 13-145, Rev. 0, “Probable Maximum Seiche (PMS) at Millstone Power Station (Regional and Site Screening Analysis)”
- Zachry Calculation 13-154, Rev. 0, “Evaluation of Channel Migration/Diversion at Millstone Power Station”
- Zachry Calculation 13-156, Rev. 0, “Evaluation of Ice Effects at Millstone Power Station”

- Zachry Calculation 14-027, Rev. 0, "Detailed Tsunami Modeling for Millstone Power Station (MPS)"
- Zachry Calculation 14-034, Rev. 0, "Probable Maximum Hurricane for Millstone Power Station"
- Zachry Calculation 14-161, Rev. 0, "Probabilistic Storm Surge for Millstone Power Station"
- Zachry Calculation 14-162, Rev. 0, "Deterministic Probable Maximum Storm Surge for Millstone Power Station"
- Zachry Calculation 14-208, Rev. 0, "Site Specific Probable Maximum Precipitation (PMP) at Millstone Power Station"
- Zachry Calculation 14-222, Rev. 0, "Local Intense Precipitation Flooding Using Site Specific Precipitation Information – Millstone Power Station"
- Zachry Calculation 14-223, Rev. 0, "Beyond Design Basis Combined Effect Flood at Millstone Power Station"

2.1. Local Intense Precipitation

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

The elevations in this section refer to the Millstone mean sea level (MSL) vertical datum unless otherwise noted. The elevations in the topographic survey (McKim & Creed, 2012a), which was an input to the calculation described below, are referenced to the North American Vertical Datum of 1988 (NAVD88). To convert NAVD88 to MSL at Millstone, add 1.0 foot to the NAVD88 elevation (Dominion, 2013).

2.1.1 Site Description

Millstone is situated on a point of land located on the east side (end) of the Niantic Bay and on the north shore of the Long Island Sound in Waterford, New London County, Connecticut. A general locus map is presented in Figure 2.1-1. Millstone is a two-unit facility. MPS3 is on the north side of the main complex building and MPS2 is adjacent to MPS3 on the south side.

Surface runoff generally discharges to the shoreline towards the west, south and east due to the general topography of the site and the peninsula-like site layout (Figure 2.1-2). Overall the site grades slope down from north to south, and towards the shoreline. The main building complex at Millstone is constructed at approximately elevation 24 feet MSL at MPS3 to elevation 14 feet MSL at MPS2 (Dominion, 2014b and Dominion, 2014a, respectively).

Site drainage is normally accomplished through a system of catch basins and underground storm drains. Surface drainage is constricted by the perimeter Vehicle Barrier System (VBS) to the east and south of MPS2 and MPS3, which are generally about 2.7 feet high. Gaps up to approximately 3 feet are present between the blocks of the concrete security barriers.

2.1.2 Method

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

The HHA for LIP used the following steps:

1. Define FLO-2D model limits for LIP analysis.
2. Develop the FLO-2D computer model with site features.
3. Develop LIP/PMP inputs:
    a. Using the National Oceanic and Atmospheric Administration (NOAA) and U.S. Army Corps of Engineers (USACE) Hydrometeorological Report Nos. 51 (HMR-51) and 52 (HMR-52);
    b. Refining generic HMR-51 and HMR-52 rainfall values through a site-specific meteorological study at Millstone: The PMP values provided in HMR-51 for Millstone provide values starting at 6-hours and 10-square-miles. There are no

explicit values provided at the 1- and 6-hour durations for 1-square-mile. HMR-52 provides information to derive the 1-hour 1- and 10- square-mile values based on the 6-hour 10-square-mile PMP in HMR-51. Unfortunately, the most recent storm evaluated in HMR 51 occurred in 1972. In addition, because HMR-51 and 52 cover large domains, generalization and conservatism were employed in the development of their respective PMP values that do not necessarily reflect the site-specific characteristics of Millstone.

4. Perform flood simulations in FLO-2D and estimate maximum water surface elevations at Millstone.
   a. Perform Sensitivity Analyses (i.e., presence/absence of VBS);
   b. Perform final LIP simulations.

NUREG/CR-7046 recommends that runoff losses be ignored during the LIP event to maximize runoff (NRC, 2011). Therefore, infiltration (i.e., constant loss) is conservatively not simulated in FLO-2D (FLO-2D, 2014a), even in relatively undeveloped areas such as woods. Initial abstraction was also set to be zero. Time of concentration or lag time is not a direct input in FLO-2D (i.e. the time component of flow routing is computed internally in FLO-2D) because FLO-2D computes overland flow (i.e., routing) based on ground surface conditions, such as elevations and roughness coefficients, within FLO-2D (FLO-2D, 2014b).

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

2.1.3 Results

2.1.3.1 FLO-2D Model Development

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

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

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

The FLO-2D model developed for the LIP analysis was based on Millstone site features including: topography, site location, concrete barrier layout, and structures (Figure 2.1-3). The FLO-2D computational area is approximately 220 acres. The selected grid element size for the model was 10 feet by 10 feet, which was selected based on the level of detail judged appropriate for the project.

Step 2: Assign Elevation Data to Grid Elements

The elevation data used to develop the FLO-2D model consist of Digital Terrain Model (DTM) data (McKim & Creed, 2012a) prepared by photogrammetric methods using aerial photography and field survey. The grid spacing of the Millstone DTM is 10 feet by 10 feet.

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

A minimum of two closest DTM points within the vicinity of a grid element was used in computing grid elevations. FLO-2D interpolated elevations for grid elements were spot checked for accuracy and modified as necessary based on site survey. Some FLO-2D ground interpolated elevations near the retaining wall between MPS2 and MPS3 (north of Building No. 215) were manually adjusted to conform with the site survey plan (McKim & Creed, 2012a) as some of the grid cells near the retaining wall had their elevations interpolated based on the two distinct MPS2 and MPS3 grades (MPS2 grades are several feet lower than MPS3 grades).

Step 3: Define Surface Roughness Parameters

Manning's n-values used in FLO-2D are composite values that represent flow resistance. Grid element Manning's n-values were assigned based on land cover types at the site, and recommended Manning's roughness coefficients (i.e., Table 1 of FLO-2D, 2014b and Table 5-6 of Chow, 1959). Table 2.1-1 shows the relationship between Manning's n values and selected land cover categories. The Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete or paved areas to 0.3 for wooded areas to 0.8 for the line of boulders between Buildings 441 and 703 (Fitness Center). Figure 2.1-4 shows the Manning's coefficients selection for each land cover. The numbering of the buildings is displayed in plots (Dominion, 1991) included in Appendix C. The land cover type upon which the Manning's roughness coefficients assignments were made was based on the high resolution aerial imagery of the site, obtained as part of the Millstone topographical survey (McKim & Creed, 2012a) and confirmed during a site visit on March 18, 2014.

Step 4: Represent Building Rooftops and Other Flow Obstructions

Buildings at Millstone were incorporated into the FLO-2D model based on the surveyed topographic site plan (McKim & Creed, 2012a) by manually adjusting (increasing) grid element elevations. Buildings were represented by grid elements with a ground elevation at least five feet higher than surrounding areas to ensure that runoff from the roofs freely flows to adjacent ground grid elements and flows around the building footprint (i.e. not through the building). Buildings that appeared to have flat roofs based on the survey plan were assigned uniform elevations to the grid elements representing that single building so as to ensure that runoff from rooftops are uniformly distributed to the surrounding areas. For buildings with different rooftop elevations adjacent to each other based on the survey plan, the relative changes in rooftop elevations were represented as a minimum 2-feet relative difference in building grid element elevations. The peak 1-hour duration LIP depth of 17.4 inches based on HMR-51 and HMR-52

is less than the relative change in elevation of at least 2 feet. Therefore, water is not expected to build-up high enough to drive flow from rooftops with lower elevations to adjacent rooftops with higher elevations. This ensures that general flow directions of runoff from rooftops are considered.

Step 5: Define Water Surface Elevation at Long Island Sound

As a conservative approach, the ten percent exceedance high tide elevation plus long term changes in sea level of Long Island Sound of 3.2 feet NAVD88 or 4.2 feet MSL were used along the model extent of Long Island Sound as a boundary condition in the FLO-2D model. Therefore, Long Island Sound is modeled in the LIP analysis as a constant (i.e., in time) conservative (high) elevation along the boundary of the FLO-2D model.

Step 6: Define "Levees" including Vehicle Barrier Systems

The VBS and walls that were not associated with buildings (free standing walls) and had the potential to divert or backwater flow were modeled in FLO-2D using the levee structures component of the model. The location of these structures was based on the site topographic survey plan (McKim & Creed, 2012a). The height of the VBS modeled as levees was 2.7 feet, which is the approximate height of the VBS at Millstone based on the site topographic survey (McKim & Creed, 2012a). The height of the walls was arbitrarily selected as 20 feet based on assessment of oblique aerial imagery (Microsoft, 2014). The selected height of 20 feet for the walls also had the intent to prevent overtopping of these structures.

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

The gaps between individual sections of the VBS were modeled using the hydraulic structure component of FLO-2D due to the small width of these openings relative to the 10-foot grid size. These gaps are therefore treated as small weirs. The location and width of the gaps within the VBS were based on the topographic survey of the site (McKim & Creed, 2012a). Three depth-discharge relationships for flow through three general sized gaps including: 0.5-foot, 1.4-foot, and 2.5-foot gaps within the VBS (Table 2.1-2) were developed using the general weir equation, with a weir coefficient indicative of open channel flow. Figure 2.1-5 includes the gap dimensions, locations, and the selected depth-discharge relationship for each gap modeled. The depth-discharge relationship was conservatively selected for each hydraulic structure (i.e., VBS gap) based on whether it would divert or backup flow.

a. The VBS located on the north of the site diverts flow as shown in the FLO-2D resultant velocity vectors figure included as Appendix B6. Therefore, the depth-discharge relationship selected to model the gaps within the VBS in the north area were conservatively set to 2.5 or 1.4 feet to be equal or larger than the actual gap dimension, which ranged from 0.6 to 2.4 feet.

b. The VBS located on the south and east of the site back up flow as shown in the FLO-2D resultant velocity vectors figure included as Appendix B6. The depth-discharge relationship selected to model the gaps within the VBS in these areas were conservatively set to 0.5, 1.4, or 2.5 feet to be equal or smaller than the gap dimension, which ranged from 0.5 to 3.2 feet..

Tailwater conditions were analyzed during the FLO-2D simulations such that the depth for any given flow was the difference in water surface elevations on either side of the levee. Note that weir submergence due to tailwater effects (i.e., decrease of weir discharge coefficient due to

backwater flooding from downstream structures, boundary conditions, or constrictions) was not accounted in the VBS gaps modeling based on the results shown in Appendix B3 that shows no significant tailwater in places where the VBS backs up flow such as southeast of Building No. 410.

Step 8: Perform Sensitivity Analyses

A sensitivity analysis was done on the presence or absence of the VBS. The general topography of Millstone is such that runoff will flow from the north towards the south and east. Therefore, the VBS located south of MPS2 and east of the Independent Spent Fuel Storage Installation (ISFSI) area have the potential to back up water, while a portion of the VBS located north of MPS3 diverts flow away from MPS2 and MPS3 (Figure 2.1-3). The FLO-2D model was run with the VBS in place and separately without the VBS.

The results of the sensitivity analysis of the presence/absence of the VBS indicate that the presence of the VBS results in the most conservative LIP depths in the MPS2 area. The results generally indicate an increase of approximately 0.2 feet with the presence the VBS in comparison to ignoring the VBS. For the MPS3 area, the results indicate that the presence of the VBS does not significantly impact (i.e., generally less than 0.1 feet difference) the maximum flow depths in this area. The VBS was therefore included in the final LIP analysis.

2.1.3.2 Rainfall Inputs

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

A refined calculation was then performed using the results of a site-specific meteorology study. Section 5.2 of ANSI/ANS-2.8-1992 (ANS, 1992) indicates that parameters of the PMP should be determined by a meteorological study utilizing a storm based approach. This analysis followed the storm-based approach as followed in HMR 53 (NOAA, 1980) and HMR 51 (NOAA, 1978). The World Meteorological Organization (WMO) Manual for PMP determination (WMO, 2009) recommends this same approach. Figure 2.1-7 displays the major steps used in the calculation of the 1- and 6-hour, 1-square mile PMP.

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

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

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

This resulted in 11 events being evaluated for use in LIP calculations (Figure 2.1-8 and Table 2.1-4). Eight of the storms were not covered by the HMR or USACE analyses. For these newly identified extreme rainfall events without published Depth-Area-Duration (DAD) analyses, hourly rainfall grids and DADs were computed using the SPAS computer program (Parzybok et al., 2014). There are two main steps in the SPAS DAD analysis: 1) the creation of high-resolution hourly precipitation grids and 2) the computation of Depth-Area (DA) rainfall amounts for various durations. Because this process has been the standard for many years (all DAD produced by the NWS in HMR 51 used this procedure) and holds merit, the SPAS DAD analysis process used in this study attempts to mimic the NWS procedure as much as possible. By adopting this approach, consistency between the newly analyzed storms and the hundreds of storms already analyzed by the NWS is achieved.

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

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

Section 7.4 (NOAA, 1994). Further, differences in moisture between the original location and the site are accounted for and quantified. For the Millstone location, this affects storms whose moisture source is the Gulf of Mexico versus the Atlantic Ocean, where only storms whose main moisture source was the Atlantic Ocean were considered.

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

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

The results of a rainfall temporal distribution sensitivity analysis indicated that a 6-hour rainfall temporal distribution with the peak rainfall at the end of the 6-hour period results in the most conservative LIP depths at the site (i.e. produce the largest flood depths). Out of the three temporal distributions analyzed, the front loading distribution resulted in the least conservative depth (i.e. produce the least flood depths). Recommendations for LIP temporal distribution included in the site-specific meteorology study, indicated that the maximum 1-hour precipitation could occur as a front loaded storm (hours 1-2) or a middle loaded storm (hours 3-4). Therefore, based on the results of the temporal distribution sensitivity analysis and the recommendations for temporal distributions included in the site-specific meteorology study, the 6-hour PMP hyetograph was therefore constructed using the 1-hour PMP for the fourth hour and equal rainfall increments for the preceding three hours and the fifth and sixth hours. The resultant hyetograph is shown in Figure 2.1-9.

2.1.3.3 FLO-2D Model Simulations

Two LIP simulations were performed based on the HHA approach applied to rainfall inputs. The first LIP simulation included the more conservative precipitation depths determined based on HMR-51 and HMR-52. The maximum flood elevations for the HMR-51/52 simulation exceeded the design basis LIP flood elevations presented in the Millstone FSAR (Dominion, 2014a and Dominion, 2014b). Therefore, a refined LIP simulation was performed using the precipitation depths based on a site-specific meteorology study.

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

The results of the LIP simulation based on HMR-51 and HMR-52 are summarized in Table 2.1-6 and include maximum water surface elevations, maximum flow depths, time to maximum water surface elevations, and maximum flow velocities for representative grid elements at strategic

locations identified by Millstone personnel. The maximum water surface elevations are displayed in Figures 2.1-10 and 2.1-11.

The LIP maximum water surface elevations in the immediate vicinity of MPS2 range from El. 14.4 feet MSL at Flood Gate No. 20 (Item 218) at the intake structure to El. 18.5 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 Millstone MPS3 range from El.14.2 feet MSL at Door WP-14-7A (Item. 302) to locally as high as El. 25.3 feet MSL at Door S-24-20 (Item 378) at the north corner of the Service Building (Building No. 317) with the Auxiliary Building (Building No. 318). Note that the calculated maximum flood elevations at MPS3 are higher in some locations (e.g., Control Building) than the design basis presented in the Millstone FSAR (Dominion, 2014b). The numbering of the buildings is displayed in plots (Dominion, 1991) included in Appendix C.

Results Based on Site-Specific Meteorology Study

The results of the LIP simulation based on the site-specific meteorology study are summarized in Table 2.1-7 and include maximum water surface elevations, maximum flow depths, time to maximum water surface elevations, and maximum flow velocities for representative grid elements at strategic locations identified by Millstone personnel. The maximum water surface elevations are displayed in Figures 2.1-12 and 2.1-13. Flood stage hydrographs at representative locations are included as Figures 2.1-14 through 2.1-17.

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

MPS2

The LIP maximum water surface elevations in the immediate vicinity of MPS2 range from El. 14.3 feet MSL at Flood Gate No. 20 (Item 218) 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 (see Figure 13).

Calculated maximum depths range from 0.2 feet at Flood Gate No. 20 (Item 218) at the intake structure to 4.2 feet at Flood Gate No. 13 (Item 211) at the northern perimeter of the Containment Enclosure building where grades are generally lower than surrounding areas (see Appendix B2).

Calculated flow velocity is up to 4.5 feet per second (fps) north of the Containment Enclosure building (Building 207) and 4.3 fps between the Maintenance Snubber Shop (Building 416) and Health Facility (Building 417) (see Appendix B5).

MPS3

The LIP maximum water surface 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) (see Appendix B4). The results indicate that the FLO-2D maximum water surface elevations are lower than the design basis presented in the Millstone FSAR (Dominion, 2014b).

Calculated maximum depths range from 0.1 (Item 346) to 3.1 feet (Item 342 - Intake Cable interface with Control Building) (see Appendix B3).

Calculated flow velocity is up to 6.4 fps at the intake structure (Item 302, see Appendix B5).

The protected area is impervious and does not contain natural sources of vegetation and debris. The maximum velocities of up to 4.5 fps north of the MPS2 Containment Enclosure building (Building 207) and up to 6.4 fps at the MPS3 intake structure during the LIP are unlikely to result in debris loading issues at the site. Hydrodynamic and Hydrostatic loading against buildings at the site are also likely to be minimal due to the generally shallow depths and low velocities during the LIP.

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

- Volume Conservation: Review of the FLO-2D output files indicates volume conservation errors of 0.000007 percent for the FLO-2D run. This value is below the threshold of 0.001 percent specified in the FLO-2D Data Input manual (FLO-2D, 2014c) for a successful project application.
- Area of Inundation: Review of the FLO-2D output files indicates maximum inundated area of 220 acres. The FLO-2D model is made up of 96,049 grid elements, each 10 feet by 10 feet in dimension. The LIP was simulated within the entire computational domain of the model. The maximum inundation area should therefore be equal to the area of the computational domain of 220 acres (10 x 10 x 96,049) x (1 acre / 43,560 feet). The FLO-2D calculated maximum inundation area of 220 acres is reasonable and indicates a successful project application.
- Maximum Velocities and Numerical Surging: Numerical surging, if it exists, would be evident at unreasonably high velocities in the FLO-2D output files (FLO-2D, 2014b). A review of the velocity output file does not indicate unreasonably high velocities in the model runs and indicates a successful project application. The high velocities reported occur at the relatively steep grade approaching the alleyway between Buildings 409 and 410, south of the ISFSI, locations near the Niantic Bay shoreline where the slopes are relatively steep, and at the pond east of the ISFSI that impounds the small coastal stream near Millstone. These high velocities appear to be reasonable given the local conditions in this area.

2.1.4 Conclusions

Two LIP simulations were performed based on the HHA approach applied to rainfall inputs. The first LIP simulation included the more conservative precipitation depths determined based on HMR-51 and HMR-52. A refined LIP simulation was performed using the precipitation depths from a site-specific meteorology study. A summary of the results of the LIP simulations at Millstone (Tables 2.1-6 and 2.1-7) are as follows:

1. PMP Depths: The maximum flood elevation due to the HMR-51/52 LIP simulation at Millstone results from a total rainfall depth of 17.4 inches within an hour and 26.0 inches within 6 hours. The maximum flood elevation due to the site-specific meteorology study LIP simulation at Millstone results from a total rainfall depth of 11.0 inches within an hour and 23.3 inches within 6 hours.
2. LIP simulation based on HMR-51/52 rainfall inputs: The LIP maximum water surface elevations in the immediate vicinity of MPS2 range from El. 14.4 feet MSL at Flood Gate No. 20 (Item 218) at the intake structure to El. 18.5 feet MSL at Flood Gate No. 13 (Item 211) at the northern perimeter of the Containment Enclosure building (Building No. 207). The LIP maximum flood elevations in the immediate vicinity of MPS3 range from El.14.2 feet MSL at Door WP-14-7A (Item. 302) to locally as high as El. 25.3 feet MSL at Door S-24-20 (Item 378) at the north corner of the Service Building (Building No. 317) with the Auxiliary Building (Building No. 318).
3. LIP simulation based on site-specific meteorology study rainfall inputs: The LIP maximum flood elevations in the immediate vicinity of 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).

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

2.1.5 References

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

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

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Table 2.1-1: Manning's n Values for Land Cover Type (FLO-2D, 2014 and Chow, 1959)

Definition	Manning's n
Water	0.025
Paved/Concrete	0.02
Trees/Shrubs	0.30
Short Grass	0.05
Building Roof	0.02

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

	Discharge (cubic feet per second)		
Flow Depth (feet)	0.5 Foot Width of Gap	1.4 Foot Width of Gap	2.5 Feet Width of Gap
0.0	0.0	0.0	0.0
0.4	0.4	1.1	2.0
0.8	1.1	3.1	5.5
1.2	2.0	5.7	10.2
1.6	3.1	8.8	15.7
2.0	4.4	12.3	21.9
2.4	5.8	16.1	28.8
2.8	7.3	20.3	36.3
3.2	8.9	24.8	44.4
3.6	10.6	29.6	52.9
4.0	12.4	34.7	62.0
4.4	14.3	40.1	71.5
4.8	16.3	45.6	81.5

Note:

1. Depth-Discharge relationship developed based on the Weir Equation $Q=CLH^{1.5}$ (Chow, 1959), where Q is discharge in cubic feet per second (cfs), C is the weir coefficient, L is the flow width in feet, and H is the flow depth in feet;;
2. A weir coefficient of 3.1 was used (USBR, 1987).

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

Time (minutes)	PMP Depth (inches)
360	26.0
60	17.4
30	13.2
15	9.2
5	5.9

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

Storm Name	State	Lat	Lon	Year	Month	Day	Maximum Total Storm Rainfall	Maximum 6-hour $10mi^2$ Rainfall	Maximum 1-hour $1mi^2$ Rainfall Using HMR 52 Ratio or SPAS Data	Maximum 6-hour $1mi^2$ Rainfall Using SPAS Ratio or SPAS Data	Millstone Total Adjustment Factor	Millstone 1-hour $1mi^2$ PMP	Millstone 6-hour $1mi^2$ PMP	Precipitation Source
JEWELL	MD	38.7550	-76.6184	1897	7	26	15.80	13.00	8.83	14.04	1.25	11.03	17.55	NA 1-7
SPARTA	NJ	41.0300	-74.6400	2000	8	11	16.70	10.30	4.00	12.60	1.57	6.28	19.78	SPAS 1017
EWAN	NJ	39.7000	-75.1900	1940	9	1	24.00	19.20	7.30	21.00	1.11	8.10	23.31	NA 2-4
TABERNACLE	NJ	39.8805	-74.7100	2004	7	13	15.63	12.80	5.90	13.60	1.10	6.49	14.96	SPAS 1040
DELAWARE COUNTY	NY	42.0100	-74.9000	2007	6	19	11.69	10.30	4.30	11.40	1.71	7.35	19.49	SPAS 1049
MAPLECREST (IRENE)	NY	42.2000	-74.1600	2011	8	27	22.91	13.20	2.80	13.76	1.23	3.44	16.92	SPAS 1224
WESTFIELD	MA	42.1200	-72.7000	1955	8	17	19.80	8.40	5.70	7.85	1.14	6.50	8.95	SPAS 1243
NEWARK	NJ	40.7300	-74.2700	1999	9	15	14.45	6.09	3.82	6.53	1.20	4.58	7.84	SPAS 1002
WILLIAMSBURG	VA	37.1500	-76.4900	1999	9	14	16.98	6.60	4.14	6.70	0.97	4.01	6.50	SPAS 1012
MANAHAWKIN	NJ	39.6957	-74.2588	1939	8	19	17.80	9.70	6.59	10.48	1.05	6.92	11.00	NA 2-3
ISLIP	NY	40.8050	-73.0650	2014	8	13	14.23	13.29	6.94	13.95	1.08	7.50	15.07	SPAS 1415

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

Time (minutes)	PMP Depth (inches)
360	23.3
60	11.0
30	8.4
15	5.8
5	3.7

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
MPS2									
201	Flood Gate #1 / Door (Aux Bldg, East, DC Swgr)	Auxiliary	14.5	67915	14.20	17.88	3.68	6.0	3.8
202	Flood Gate #2 / Door (Aux Bldg, South)	Auxiliary	14.5	67749	13.99	17.73	3.74	6.0	4.0
203	Flood Gate #5 / Door (Aux Bldg, East, HP Area)	Auxiliary	14.5	67247	14.14	16.80	2.66	6.0	1.6
204	Flood Gate #6 / Door (Aux Bldg, East, Railway Access)	Auxiliary	14.5	66367	14.11	16.88	2.77	6.0	2.3
205	Flood Gate #7 / Door (Aux Bldg, East, Radwaste Drumming Area)	Auxiliary	14.5	66004	14.11	17.02	2.91	6.0	2.8
206	Flood Gate #8 / Door (Aux Bldg, East, "B" EDG)	Auxiliary	14.5	65244	14.11	17.28	3.17	6.0	2.4
207	Flood Gate #9 / Door (Aux Bldg, East, "A" EDG)	Auxiliary	14.5	65033	14.13	17.32	3.19	6.0	3.4
208	Flood Gate #10 / Door (Enclosure Bldg. North)	Enclosure	14.5	63288	14.92	18.01	3.09	6.0	3.1
209	Flood Gate #11 / Door (Enclosure Building, Equipment Hatch)	Enclosure	14.5	63285	13.58	18.28	4.70	6.0	4.9
210	Flood Gate #12 / Door (Turbine/Enclosure Bldg., North)	Turbine	14.5	64051	13.63	18.26	4.63	6.0	5.0
211	Flood Gate #13 / Door (Turbine Bldg RW Access, East)	Turbine	14.5	63530	13.30	18.53	5.23	6.0	6.1
212*	Flood Gate #14 / Door (Turbine Bldg, North, CPF Area)	Turbine	14.5	63529	13.28	18.24	4.96	6.0	4.9
213	Flood Gate #15 / Door (Turbine	Bldg 110	19.0	69038	13.96	14.82	0.86	6.0	1.0

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
	Bldg, Southwest)								
214	Flood Gate #21 / Door (Control Bldg East,, Unit 1)	Bldg 118	14.5	68577	14.17	17.73	3.56	6.0	2.9
215	Flood Gate #16 / Door (Intake, Hypochlorite Room)	Intake	14.5	69506	13.68	14.63	0.95	6.0	0.5
216*	Flood Gate #17 / Door (Intake, MCC Room Access Door, North)	Intake	14.5	68697	13.71	14.59	0.88	6.0	1.5
217	Flood Gate #18 / Door (Intake, MCC Room to Pump Room, West)	Intake	14.5	68697	13.71	14.59	0.88	6.0	1.5
218	Flood Gate #20 / Door (Intake, South)	Intake	14.0	70138	14.09	14.39	0.30	6.0	0.8
220	Flood Gate #3 (MP2 Electric Fire Pump Enclosure, South))	Fire Pump House	14.5	66564	14.36	16.53	2.17	6.0	1.1
221	Flood Gate #4 / Door (MP2 Electric Fire Pump Enclosure, North))	Fire Pump House	14.5	66021	14.41	16.68	2.27	6.0	1.5
222	Flood Gate #22 / Door (MP3 Fire Pump House, East)	Fire Pump House	14.5	66741	14.47	16.51	2.04	6.0	1.3
266	RWST Pipe Chase	Yard	14.5	64568	13.58	17.83	4.25	6.0	2.7
267	CST Pipe Chase	Yard	14.5	60182	13.85	18.25	4.40	6.0	3.6
268	U2 Manhole-01A, SE Corner	Intake	24.3	69824	13.98	14.61	0.63	6.0	0.5
269	U2 Manhole-01B, SE Corner	Intake	24.3	69824	13.98	14.61	0.63	6.0	0.5
270	U2 Manhole-02A, NE Corner	Intake	24.3	68531	13.71	14.54	0.83	6.0	1.9
271	U2 Manhole-02B, NE Corner	Intake	24.3	68531	13.71	14.54	0.83	6.0	1.9
272	U2 Manhole-03A, South of Unit 2	Yard	14.5	67921	14.00	17.65	3.65	6.0	3.4

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
	Aux iliary Bldg								
273	U2 Manhole-04, North of Aux Building, RWST Area	Yard	14.5	69250	14.02	15.93	1.91	6.0	3.5
274	U2 Manhole-05, NE Corner of Unit 2 Fire Pump House	Yard	14.5	65467	14.30	16.64	2.34	6.0	1.3
275	U2 Manhole-03B, North of Unit 2 Fire Pump House	Yard	14.5	65652	14.52	16.73	2.21	6.0	1.6
MPS3									
301*	C-11-1A Control Building to Tech Support Center Door	Control Building	11.0	48117	23.29	23.91	0.62	6.0	0.9
302*	WP-14-7A, "A" Service Water Pump Cubicle to CWS Door	Intake	14.5	62697	13.68	14.18	0.50	6.0	9.5
303*	WP-14-9A, "B" Service Water Pump Cubicle to CWS Door	Intake	14.5	64260	13.95	14.69	0.74	6.0	1.8
307	3PBS-LS26A, Main Transformer "A" Pit	Yard	21.3	56568	17.18	23.63	6.45	6.0	9.3
308	3PBS-LS26B, Main Transformer "B" Pit	Yard	21.3	56030	17.37	24.02	6.65	6.0	9.3
309	3PBS-LS26C, NSST "A" Pit	Yard	21.4	57073	22.67	24.47	1.80	6.0	0.4
310	3PBS-LS26D, NSST "B" Pit	Yard	21.4	57601	23.02	24.45	1.43	6.0	1.4
311	3PBS-LS27A, RSST "A" Pit	Yard	21.3	39190	23.72	25.15	1.43	6.0	1.3
312	3PBS-LS27B, RSST "B" Pit	Yard	21.3	39190	23.72	25.15	1.43	6.0	1.3
342*	Handhole 3EHH-03, Intake Cable interface with Control Building	Yard	23.5	48107	19.03	22.82	3.79	6.0	4.8

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
343	Manhole 3EMH-01A, Intake cable interface with Control Building.	Yard	18.2	55417	17.88	18.53	0.65	6.0	1.9
344	Manhole 3EMH-01B, Intake cable interface with Control Building.	Yard	18.2	55417	17.88	18.53	0.65	6.0	1.9
345	Manhole 3EMH-04, Unit 3 Fire Pump House Interface with ESF Building	Yard	24.3	51336	23.48	24.12	0.64	6.0	1.5
346	Manhole 3EMH-10, Intake cable interface with Turbine Building.	Yard	23.4	57310	22.88	22.96	0.08	6.0	0.2
347	Manhole 3EMH-19, Intake cable interface with Control Building.	Yard	20.0	52675	22.49	22.72	0.23	5.9	0.8
356	Aux Building Door A-24-1	Aux Building	24.5	43655	23.11	25.14	2.03	6.0	0.3
357	Aux Building Door A-24-6	Aux Building	24.5	48433	23.62	25.10	1.48	6.0	0.9
358	Aux Building Door A-24-9	Aux Building	24.5	43358	22.90	25.12	2.22	6.0	0.6
359	Hydrogen Recombiner Building Door HR-24-5	Hydrogen Recombiner Building	24.5	51892	23.77	24.32	0.55	6.0	0.9
360	Control Building Door - C-24-1	Control Building	24.5	45449	23.71	24.72	1.01	6.0	0.8
361	EDG Building Door - EG-24-1	EDG Building	24.5	43336	23.61	24.46	0.85	6.0	1.7
362	EDG Building Door - EG-24-2	EDG Building	24.5	44549	23.66	24.55	0.89	6.0	1.3
363	EDG Building Door - EG-24-3	EDG Building	24.5	43933	23.63	24.22	0.59	6.0	2.1
364	EDG Building Door - EG-24-4	EDG Building	24.5	45145	23.74	24.14	0.40	6.0	0.4
365	Fuel Building Door - F-24-2	Fuel Building	24.5	43978	23.38	24.96	1.58	6.0	0.6
366	Fuel Building Door - F-24-4	Fuel Building	24.5	44888	23.35	24.95	1.60	6.0	0.5

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
367	Fuel Building Door - F-24-5	Fuel Building	24.5	45191	23.49	24.94	1.45	6.0	0.5
368	Maint Shop Door - M-24-3	Maint Shop	24.5	44855	23.13	24.57	1.44	6.0	0.5
369	Maint Shop Door - M-24-9	Maint Shop	24.5	43652	23.29	25.23	1.93	6.0	1.5
370	Maint Shop Door - M-24-10	Maint Shop	24.5	40046	22.69	24.93	2.24	6.0	2.4
371	Maint Shop Door - M-24-11	Maint Shop	24.5	41243	23.25	25.02	1.77	6.0	0.9
372	Maint Shop Door - M-24-18	Maint Shop	24.5	40932	23.78	24.49	0.71	6.0	0.8
373	Maint Shop Door - M-24-19	Maint Shop	24.5	42133	23.61	24.50	0.89	6.0	0.9
374	Maint Shop Door - M-24-20	Maint Shop	24.5	43340	23.36	24.50	1.14	6.0	1.0
375	Maint Shop Door - M-24-22	Maint Shop	24.5	40345	23.28	24.97	1.69	6.0	2.4
376	Maint Shop Door - M-24-23	Maint Shop	24.5	41231	23.79	24.48	0.69	6.0	0.8
377	Service Bldg. Door - S-24-8	Service Bldg.	24.5	47553	23.90	25.18	1.28	6.0	0.8
378	Service Bldg. Door - S-24-20	Service Bldg.	24.5	43955	23.19	25.29	2.10	6.0	1.7
379	ESF Building Door - SF-24-1	ESF Building	24.5	50477	23.45	24.21	0.76	6.0	0.9
380	ESF Building Door - SF-24-2	ESF Building	24.5	49907	23.22	24.43	1.21	6.0	4.0
381	ESF Building Door - SF-24-4	ESF Building	24.5	49329	23.63	24.52	0.89	6.0	1.7
382	ESF Building Door - SF-24-7	ESF Building	24.5	48747	23.58	24.55	0.97	6.0	1.2
383	ESF Building Door - SF-24-12	ESF Building	24.5	47576	23.68	24.59	0.91	6.0	1.4
384	ESF Building Door - SF-24-13	ESF Building	24.5	46985	23.63	24.61	0.98	6.0	0.9
385	ESF Building Door - SF-24-14	ESF Building	24.5	46389	23.54	24.68	1.14	6.0	1.0
386	Steam Valve Building Door - SV-24-3	Steam Valve Building	24.5	50744	23.43	24.71	1.28	6.0	1.2

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

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
387	Steam Valve Building Door - SV-24-4	Steam Valve Building	24.5	49593	23.60	24.98	1.38	6.0	2.6
388	Steam Valve Building Door - SV-24-5	Steam Valve Building	24.5	50454	23.60	24.89	1.29	6.0	2.6
389	Turbine Building Door T-24-1	Turbine Building	24.5	56265	24.05	24.54	0.49	6.0	0.3
390	Turbine Building Door T-24-3	Turbine Building	24.5	48722	23.62	25.10	1.48	6.0	1.3
391	Turbine Building Door T-24-4	Turbine Building	24.5	49573	23.50	23.84	0.34	6.0	0.3
392	Turbine Building Door T-24-5	Turbine Building	24.5	55,714	22.73	23.19	0.46	6.0	1.0
393	Turbine Building Door T-25-1	Turbine Building	24.5	56265	24.05	24.54	0.49	6.0	0.3
394	Waste Disposal Bldg Door - WD-24-1	Waste Disposal	24.5	43070	23.73	24.94	1.21	6.0	0.2
395	Waste Disposal Bldg Door - WD-24-6	Waste Disposal	24.5	41264	23.72	25.08	1.36	6.0	0.8
396	Waste Disposal Bldg Door - WD-24-7	Waste Disposal	24.5	40962	23.55	25.13	1.58	6.0	0.4
397	Waste Disposal Bldg Door - WD-24-8	Waste Disposal	24.5	41257	23.55	25.16	1.61	6.0	0.8
398	Refueling Water Storage Tank Door RWST-24-1	RWST	24.5	49042	23.87	24.52	0.65	6.0	1.0
399	Demineralized Water Storage Tank Block House	DWST	24.5	48170	23.81	24.50	0.69	6.0	1.0

Notes:

1) Results based on precipitation end-loading temporal distribution.
2) * The critical item is located inside the specified building. Therefore, a representative grid outside of the building was selected.

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
MPS2									
201	Flood Gate #1 / Door (Aux Bldg, East, DC Swgr)	Auxiliary	14.5	67915	14.20	16.95	2.75	3.6	2.4
202	Flood Gate #2 / Door (Aux Bldg, South)	Auxiliary	14.5	67749	13.99	16.79	2.80	3.6	2.0
203	Flood Gate #5 / Door (Aux Bldg, East, HP Area)	Auxiliary	14.5	67247	14.14	16.24	2.10	3.6	0.3
204	Flood Gate #6 / Door (Aux Bldg, East, Railway Access)	Auxiliary	14.5	66367	14.11	16.31	2.20	3.6	1.6
205	Flood Gate #7 / Door (Aux Bldg, East, Radwaste Drumming Area)	Auxiliary	14.5	66004	14.11	16.42	2.31	3.6	2.7
206	Flood Gate #8 / Door (Aux Bldg, East, "B" EDG)	Auxiliary	14.5	65244	14.11	16.65	2.54	3.6	1.7
207	Flood Gate #9 / Door (Aux Bldg, East, "A" EDG)	Auxiliary	14.5	65033	14.13	16.69	2.56	3.6	1.5
208	Flood Gate #10 / Door (Enclosure Bldg. North)	Enclosure	14.5	63288	14.92	17.15	2.23	3.6	1.0
209	Flood Gate #11 / Door (Enclosure Building, Equipment Hatch)	Enclosure	14.5	63285	13.58	17.35	3.77	3.6	3.1
210	Flood Gate #12 / Door (Turbine/Enclosure Bldg., North)	Turbine	14.5	64051	13.63	17.30	3.67	3.6	2.6
211	Flood Gate #13 / Door (Turbine Bldg RW Access, East)	Turbine	14.5	63530	13.30	17.45	4.15	3.6	3.6
212*	Flood Gate #14 / Door (Turbine Bldg, North, CPF Area)	Turbine	14.5	63529	13.28	17.30	4.02	3.6	3.1
213	Flood Gate #15 / Door (Turbine	Bldg 110	19.0	69038	13.96	14.76	0.80	3.6	0.8

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
	Bldg, Southwest)								
214	Flood Gate #21 / Door (Control Bldg East,, Unit 1)	Bldg 118	14.5	68577	14.17	16.79	2.62	3.6	1.5
215	Flood Gate #16 / Door (Intake, Hypochlorite Room)	Intake	14.5	69506	13.68	14.49	0.81	3.6	0.6
216*	Flood Gate #17 / Door (Intake, MCC Room Access Door, North)	Intake	14.5	68697	13.71	14.43	0.72	3.6	1.3
217	Flood Gate #18 / Door (Intake, MCC Room to Pump Room, West)	Intake	14.5	68697	13.71	14.43	0.72	3.6	1.3
218	Flood Gate #20 / Door (Intake, South)	Intake	14.0	70138	14.09	14.30	0.21	3.6	0.7
220	Flood Gate #3 (MP2 Electric Fire Pump Enclosure, South))	Fire Pump House	14.5	66564	14.36	16.07	1.71	3.6	0.9
221	Flood Gate #4 / Door (MP2 Electric Fire Pump Enclosure, North))	Fire Pump House	14.5	66021	14.41	16.16	1.75	3.6	1.6
222	Flood Gate #22 / Door (MP3 Fire Pump House, East)	Fire Pump House	14.5	66741	14.47	16.05	1.58	3.6	1.1
266	RWST Pipe Chase	Yard	14.5	64568	13.58	16.99	3.41	3.6	2.0
267	CST Pipe Chase	Yard	14.5	60182	13.85	17.37	3.52	3.6	2.5
268	U2 Manhole-01A, SE Corner	Intake	24.3	69824	13.98	14.48	0.50	3.6	0.3
269	U2 Manhole-01B, SE Corner	Intake	24.3	69824	13.98	14.48	0.50	3.6	0.3
270	U2 Manhole-02A, NE Corner	Intake	24.3	68531	13.71	14.40	0.69	3.6	1.3
271	U2 Manhole-02B, NE Corner	Intake	24.3	68531	13.71	14.40	0.69	3.6	1.3
272	U2 Manhole-03A, South of Unit 2	Yard	14.5	67921	14.00	16.61	2.61	3.6	1.7

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
	Aux iliary Bldg								
273	U2 Manhole-04, North of Aux Building, RWST Area	Yard	14.5	69250	14.02	15.59	1.57	3.6	2.8
274	U2 Manhole-05, NE Corner of Unit 2 Fire Pump House	Yard	14.5	65467	14.30	16.13	1.83	3.6	1.1
275	U2 Manhole-03B, North of Unit 2 Fire Pump House	Yard	14.5	65652	14.52	16.20	1.68	3.6	1.6
MPS3									
301*	C-11-1A Control Building to Tech Support Center Door	Control Building	11.0	48117	23.29	23.76	0.47	3.6	0.2
302*	WP-14-7A, "A" Service Water Pump Cubicle to CWS Door	Intake	14.5	62697	13.68	14.04	0.36	3.6	6.4
303*	WP-14-9A, "B" Service Water Pump Cubicle to CWS Door	Intake	14.5	64260	13.95	14.46	0.51	3.6	0.9
307	3PBS-LS26A, Main Transformer "A" Pit	Yard	21.3	56568	17.18	23.45	6.27	3.6	9.6
308	3PBS-LS26B, Main Transformer "B" Pit	Yard	21.3	56030	17.37	24.00	6.63	3.6	10.0
309	3PBS-LS26C, NSST "A" Pit	Yard	21.4	57073	22.67	24.36	1.69	3.6	0.2
310	3PBS-LS26D, NSST "B" Pit	Yard	21.4	57601	23.02	24.34	1.32	3.6	1.1
311	3PBS-LS27A, RSST "A" Pit	Yard	21.3	39190	23.72	24.62	0.90	3.6	0.6
312	3PBS-LS27B, RSST "B" Pit	Yard	21.3	39190	23.72	24.62	0.90	3.6	0.6
342*	Handhole 3EHH-03, Intake Cable interface with Control Building	Yard	23.5	48107	19.03	22.11	3.08	3.6	3.8

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
343	Manhole 3EMH-01A, Intake cable interface with Control Building.	Yard	18.2	55417	17.88	18.25	0.37	3.6	1.1
344	Manhole 3EMH-01B, Intake cable interface with Control Building.	Yard	18.2	55417	17.88	18.25	0.37	3.6	1.1
345	Manhole 3EMH-04, Unit 3 Fire Pump House Interface with ESF Building	Yard	24.3	51336	23.48	24.01	0.53	3.6	1.3
346	Manhole 3EMH-10, Intake cable interface with Turbine Building.	Yard	23.4	57310	22.88	22.94	0.06	3.6	0.2
347	Manhole 3EMH-19, Intake cable interface with Control Building.	Yard	20.0	52675	22.49	22.72	0.23	3.5	0.7
356	Aux Building Door A-24-1	Aux Building	24.5	43655	23.11	24.57	1.46	3.6	0.3
357	Aux Building Door A-24-6	Aux Building	24.5	48433	23.62	24.78	1.16	3.6	0.8
358	Aux Building Door A-24-9	Aux Building	24.5	43358	22.90	24.56	1.66	3.6	0.2
359	Hydrogen Recombiner Building Door HR-24-5	Hydrogen Recombiner Building	24.5	51892	23.77	24.19	0.42	3.6	0.4
360	Control Building Door - C-24-1	Control Building	24.5	45449	23.71	24.24	0.53	3.6	0.7
361	EDG Building Door - EG-24-1	EDG Building	24.5	43336	23.61	24.08	0.47	3.6	0.5
362	EDG Building Door - EG-24-2	EDG Building	24.5	44549	23.66	24.13	0.47	3.6	0.3
363	EDG Building Door - EG-24-3	EDG Building	24.5	43933	23.63	23.93	0.30	3.6	0.3
364	EDG Building Door - EG-24-4	EDG Building	24.5	45145	23.74	23.90	0.16	3.6	0.2
365	Fuel Building Door - F-24-2	Fuel Building	24.5	43978	23.38	24.51	1.13	3.6	0.4
366	Fuel Building Door - F-24-4	Fuel Building	24.5	44888	23.35	24.50	1.15	3.6	0.4

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
367	Fuel Building Door - F-24-5	Fuel Building	24.5	45191	23.49	24.51	1.02	3.6	0.5
368	Maint Shop Door - M-24-3	Maint Shop	24.5	44855	23.13	24.15	1.02	3.6	0.3
369	Maint Shop Door - M-24-9	Maint Shop	24.5	43652	23.29	24.61	1.32	3.6	1.1
370	Maint Shop Door - M-24-10	Maint Shop	24.5	40046	22.69	24.42	1.73	3.6	2.0
371	Maint Shop Door - M-24-11	Maint Shop	24.5	41243	23.25	24.50	1.25	3.6	1.0
372	Maint Shop Door - M-24-18	Maint Shop	24.5	40932	23.78	24.09	0.31	3.6	0.2
373	Maint Shop Door - M-24-19	Maint Shop	24.5	42133	23.61	24.09	0.48	3.6	0.3
374	Maint Shop Door - M-24-20	Maint Shop	24.5	43340	23.36	24.11	0.75	3.6	1.4
375	Maint Shop Door - M-24-22	Maint Shop	24.5	40345	23.28	24.45	1.17	3.6	2.1
376	Maint Shop Door - M-24-23	Maint Shop	24.5	41231	23.79	24.08	0.29	3.6	0.3
377	Service Bldg. Door - S-24-8	Service Bldg.	24.5	47553	23.90	24.82	0.92	3.6	0.7
378	Service Bldg. Door - S-24-20	Service Bldg.	24.5	43955	23.19	24.66	1.47	3.6	1.5
379	ESF Building Door - SF-24-1	ESF Building	24.5	50477	23.45	24.06	0.61	3.6	0.7
380	ESF Building Door - SF-24-2	ESF Building	24.5	49907	23.22	24.20	0.98	3.6	3.0
381	ESF Building Door - SF-24-4	ESF Building	24.5	49329	23.63	24.26	0.63	3.6	1.5
382	ESF Building Door - SF-24-7	ESF Building	24.5	48747	23.58	24.29	0.71	3.6	1.0
383	ESF Building Door - SF-24-12	ESF Building	24.5	47576	23.68	24.35	0.67	3.6	1.1
384	ESF Building Door - SF-24-13	ESF Building	24.5	46985	23.63	24.34	0.71	3.6	0.6
385	ESF Building Door - SF-24-14	ESF Building	24.5	46389	23.54	24.38	0.84	3.6	0.9
386	Steam Valve Building Door - SV-24-3	Steam Valve Building	24.5	50744	23.43	24.50	1.07	3.6	0.8

Table 2.1-7: Site-Specific Meteorology Study - LIP Simulation Results

Physical Plant Location						FLO-2D Model Results			
Item #	Location	Building	Threshold Elevation (feet, MSL)	Representative Grid Element	Ground Surface Elevation at Grid Element (feet, MSL)	Maximum Flood Elevation (feet, MSL)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
387	Steam Valve Building Door - SV-24-4	Steam Valve Building	24.5	49593	23.60	24.67	1.07	3.6	2.0
388	Steam Valve Building Door - SV-24-5	Steam Valve Building	24.5	50454	23.60	24.61	1.01	3.6	1.9
389	Turbine Building Door T-24-1	Turbine Building	24.5	56265	24.05	24.41	0.36	3.6	0.2
390	Turbine Building Door T-24-3	Turbine Building	24.5	48722	23.62	24.75	1.13	3.6	1.0
391	Turbine Building Door T-24-4	Turbine Building	24.5	49573	23.50	23.75	0.25	3.6	0.2
392	Turbine Building Door T-24-5	Turbine Building	24.5	55,714	22.73	23.13	0.40	3.6	0.9
393	Turbine Building Door T-25-1	Turbine Building	24.5	56265	24.05	24.41	0.36	3.6	0.2
394	Waste Disposal Bldg Door - WD-24-1	Waste Disposal	24.5	43070	23.73	24.50	0.77	3.6	0.2
395	Waste Disposal Bldg Door - WD-24-6	Waste Disposal	24.5	41264	23.72	24.57	0.85	3.6	0.5
396	Waste Disposal Bldg Door - WD-24-7	Waste Disposal	24.5	40962	23.55	24.59	1.04	3.6	0.4
397	Waste Disposal Bldg Door - WD-24-8	Waste Disposal	24.5	41257	23.55	24.59	1.04	3.6	0.8
398	Refueling Water Storage Tank Door RWST-24-1	RWST	24.5	49042	23.87	24.26	0.39	3.6	0.3
399	Demineralized Water Storage Tank Block House	DWST	24.5	48170	23.81	24.23	0.42	3.6	0.3

Figure 2.1-1: Millstone General Site Location

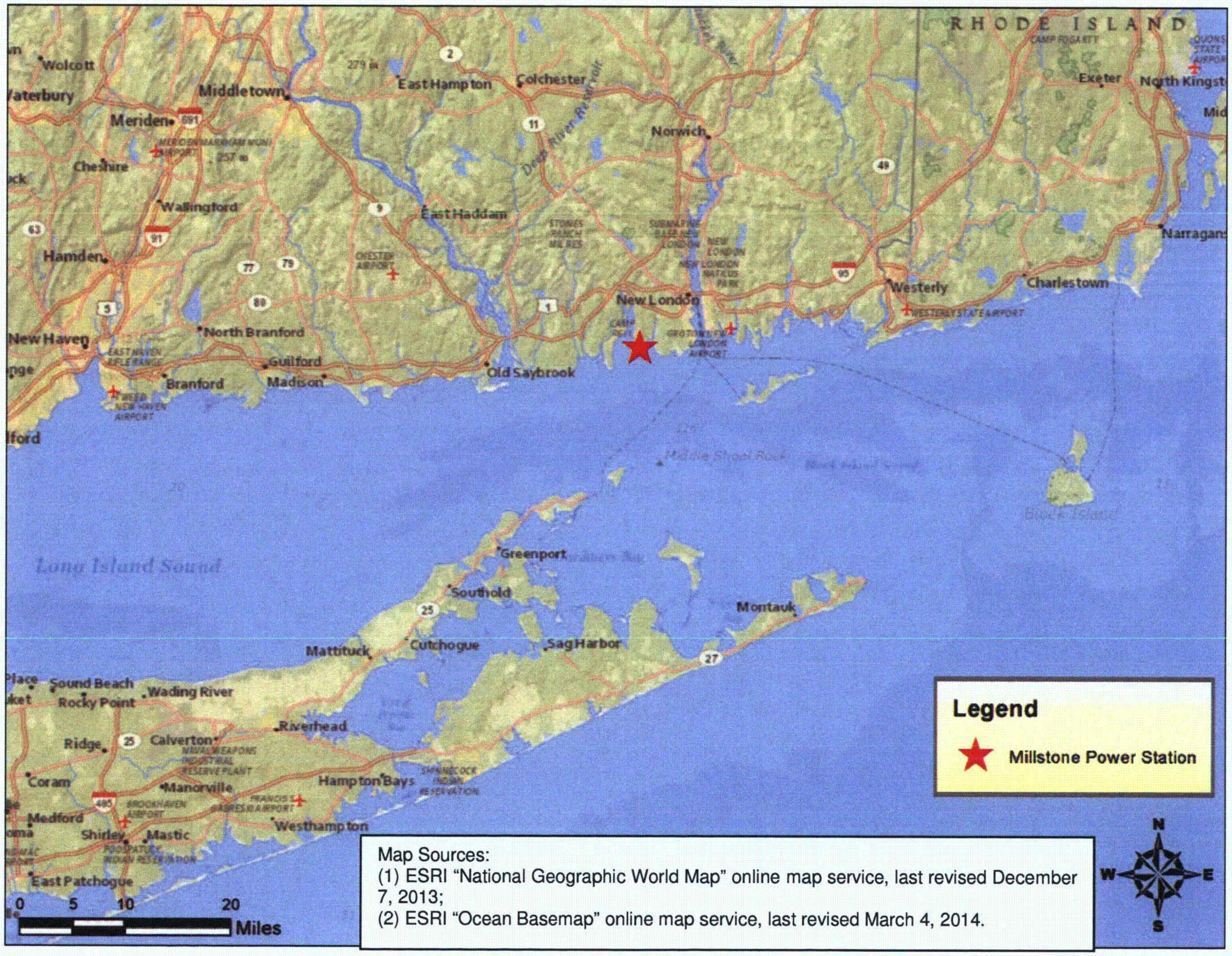
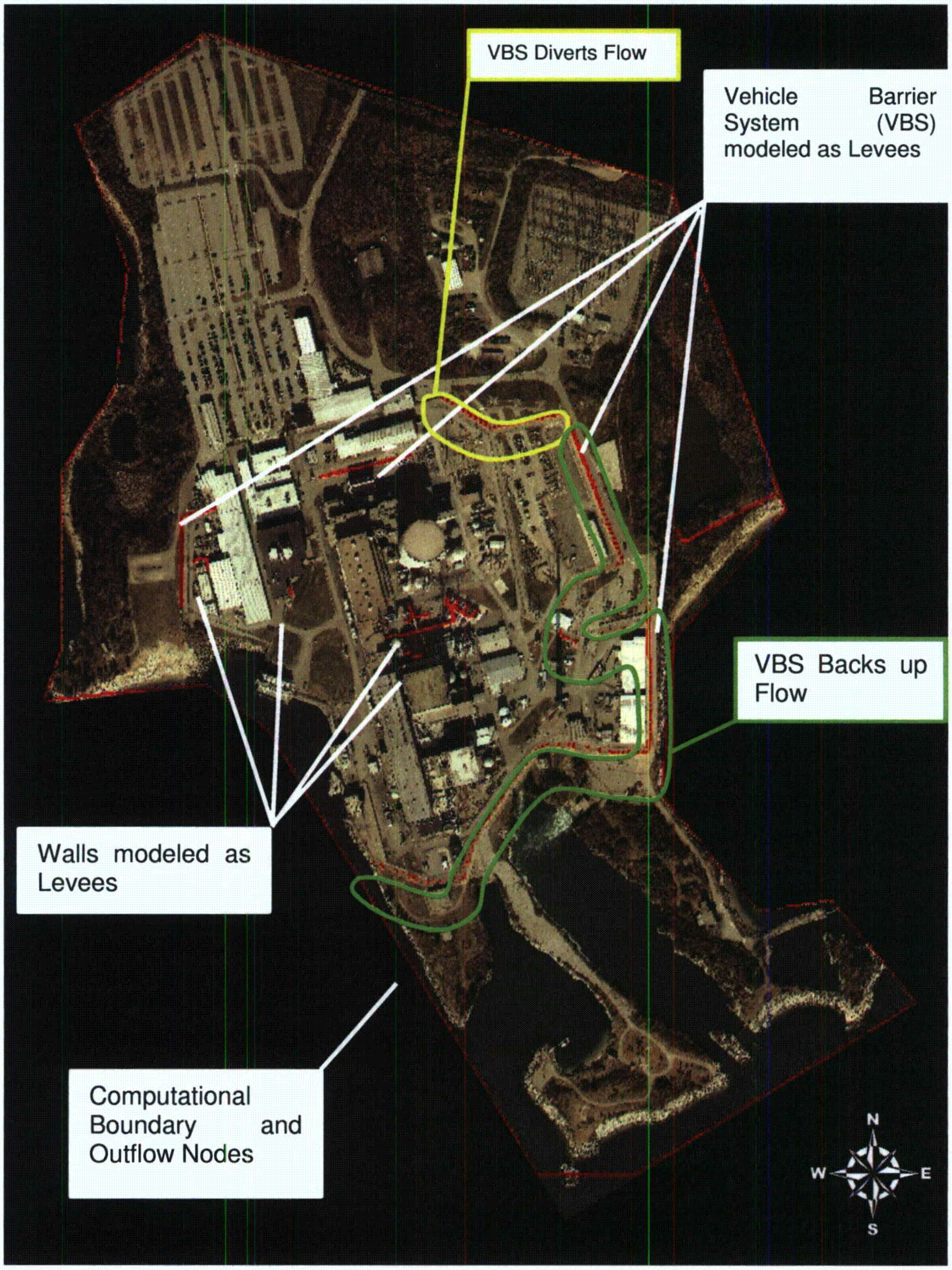


Figure 2.1-2: MPS2 and MPS3 Local Drainage Area Boundary

Figure 2.1-3: FLO-2D Model Layout



Notes: Aerial imagery from McKim & Creed, 2012a. This figure was generated with FLO-2D Pro Model (FLO 2D, 2014).

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

Notes: Manning's n values based on FLO-2D, 2014 and Chow, 1959. Basemap source: McKim & Creed, 2012a.

Figure 2.1-5: Vehicle Barrier System Gaps Dimensions

Gap ID	Gap Dimension (ft)	Gap Discharge Rating Curve ID	Gap ID	Gap Dimension (ft)	Gap Discharge Rating Curve ID	Gap ID	Gap Dimension (ft)	Gap Discharge Rating Curve ID	Gap ID	Gap Dimension (ft)	Gap Discharge Rating Curve ID	Gap ID	Gap Dimension (ft)	Gap Discharge Rating Curve ID
1	2.67	2.5	51	2.89	2.5	101	2.67	2.5	151	1.17	0.5	201	1.68	1.4
2	2.68	2.5	52	2.78	2.5	102	2.69	2.5	152	1.74	1.4	202	1.68	1.4
3	2.69	2.5	53	2.84	2.5	103	0.89	0.5	153	0.92	0.5	203	0.56	0.5
4	2.59	2.5	54	2.88	2.5	104	1.19	0.5	154	1.24	0.5	204	1.34	0.5
5	2.78	2.5	55	2.95	2.5	105	1.15	0.5	155	1.54	1.4	205	1.53	1.4
6	2.83	2.5	56	2.88	2.5	106	0.99	0.5	156	1.8	1.4	206	2.38	2.5
7	2.71	2.5	57	2.82	2.5	107	0.5	0.5	157	2.06	1.4	207	2.17	2.5
8	2.69	2.5	58	2.77	2.5	108	0.59	0.5	158	2.54	2.5	208	2.1	2.5
9	2.79	2.5	59	2.78	2.5	109	0.4	0	159	2.54	2.5	209	1.88	2.5
10	2.7	2.5	60	2.72	2.5	110	0.53	0.5	160	2.8	2.5	210	1.77	2.5
11	2.69	2.5	61	2.82	2.5	111	0.73	0.5	161	1.91	1.4	211	1.72	2.5
12	2.54	2.5	62	2.85	2.5	112	0.38	0	162	0.47	0.5	212	1.74	2.5
13	2.66	2.5	63	2.67	2.5	113	0.39	0	163	0.93	0.5	213	1.91	2.5
14	2.97	2.5	64	2.85	2.5	114	0.39	0	164	1.08	0.5	214	1.95	2.5
15	2.79	2.5	65	2.83	2.5	115	0.47	0.5	165	1.07	0.5	215	2.03	2.5
16	2.65	2.5	66	2.79	2.5	116	0.43	0	166	1.13	0.5	216	1.88	2.5
17	2.71	2.5	67	2.86	2.5	117	0.45	0.5	167	1.14	0.5	217	2.17	2.5
18	2.7	2.5	68	2.72	2.5	118	0.33	0	168	1.06	0.5	218	2.03	2.5
19	2.71	2.5	69	2.75	2.5	119	0.38	0	169	1.13	0.5	219	2.11	2.5
20	2.62	2.5	70	2.86	2.5	120	0.46	0.5	170	0.64	0.5	220	1.98	2.5
21	2.85	2.5	71	2.87	2.5	121	0.69	0.5	171	1.11	0.5	221	1.68	2.5
22	2.79	2.5	72	2.63	2.5	122	0.31	0	172	1.18	0.5	222	0.55	1.4
23	2.77	2.5	73	2.91	2.5	123	0.37	0	173	0.45	0.5	223	1.23	1.4
24	2.78	2.5	74	2.85	2.5	124	0.46	0.5	174	0.51	0.5	224	1.29	1.4
25	2.76	2.5	75	3.01	2.5	125	0.31	0	175	1.01	0.5	225	1.08	1.4
26	2.61	2.5	76	2.71	2.5	126	0.33	0	176	1.6	1.4	226	1.5	2.5
27	2.65	2.5	77	2.8	2.5	127	0.6	0.5	177	1.68	1.4	227	1.06	1.4
28	2.57	2.5	78	2.92	2.5	128	0.36	0	178	1.66	1.4	228	1.02	1.4
29	2.99	2.5	79	2.66	2.5	129	0.27	0	179	1.55	1.4	229	1.2	1.4
30	2.5	2.5	80	2.98	2.5	130	0.32	0	180	1.56	1.4	230	1.2	1.4
31	2.73	2.5	81	3.12	2.5	131	0.37	0	181	1.5	1.4	231	0.86	1.4
32	2.86	2.5	82	1.65	1.4	132	0.33	0	182	1.66	1.4	232	0	0
33	2.63	2.5	83	1.82	1.4	133	0.3	0	183	1.69	1.4	233	0.63	1.4
34	2.69	2.5	84	1.79	1.4	134	0.41	0	184	1.67	1.4			
35	2.89	2.5	85	1.96	1.4	135	0.34	0	185	1.51	1.4			
36	2.8	2.5	86	1.86	1.4	136	0.38	0	186	1.61	1.4			
37	2.75	2.5	87	2.17	1.4	137	0.28	0	187	1.57	1.4			
38	2.67	2.5	88	1.8	1.4	138	1.62	1.4	188	1.62	1.4			
39	2.95	2.5	89	2.21	1.4	139	0.54	0.5	189	1.4	1.4			
40	2.73	2.5	90	2.49	2.5	140	0.65	0.5	190	0.75	0.5			
41	2.99	2.5	91	2.32	1.4	141	0.57	0.5	191	1.35	1.4			
42	2.85	2.5	92	2.51	2.5	142	0.51	0.5	192	1.44	1.4			
43	2.81	2.5	93	2.5	2.5	143	0.66	0.5	193	0.96	0.5			
44	3.01	2.5	94	2.57	2.5	144	0.75	0.5	194	1.81	1.4			
45	2.87	2.5	95	2.62	2.5	145	0.53	0.5	195	1.92	1.4			
46	2.93	2.5	96	2.34	1.4	146	1	0.5	196	1.7	1.4			
47	2.84	2.5	97	2.57	2.5	147	0.66	0.5	197	1.73	1.4			
48	2.94	2.5	98	2.64	2.5	148	1.71	1.4	198	1.75	1.4			
49	2.9	2.5	99	2.52	2.5	149	1.61	1.4	199	1.59	1.4			
50	2.71	2.5	100	2.87	2.5	150	1.37	1.4	200	1.71	1.4			

Note: Building Outlines from McKim & Creed, 2012a. Orthophoto from McKim & Creed, 2012a. See Table 2.1-2 for Gap Discharge Rating Curves. Gap Discharge Rating Curve ID of 0 indicates opening was conservatively not modeled.

Figure 2.1-6: HMR-51/52 - Six-Hour Incremental Hyetograph – End Loading Temporal Distribution

Figure 2.1-7: Flow Chart Showing Major Steps Involved in Calculating the Site-Specific LIP

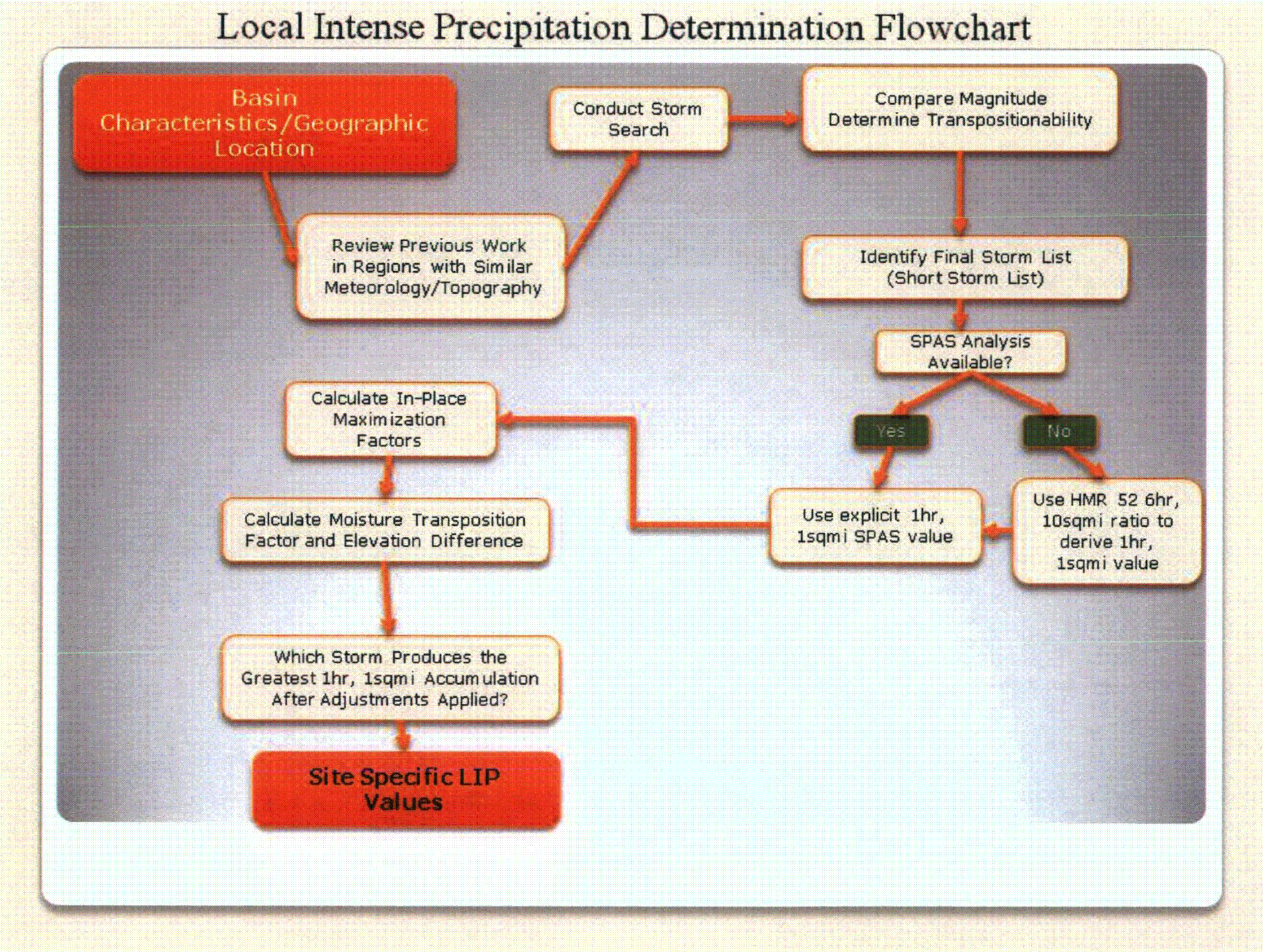


Figure 2.1-8: Storm Locations Used for Site-Specific LIP Development in Relation to Millstone

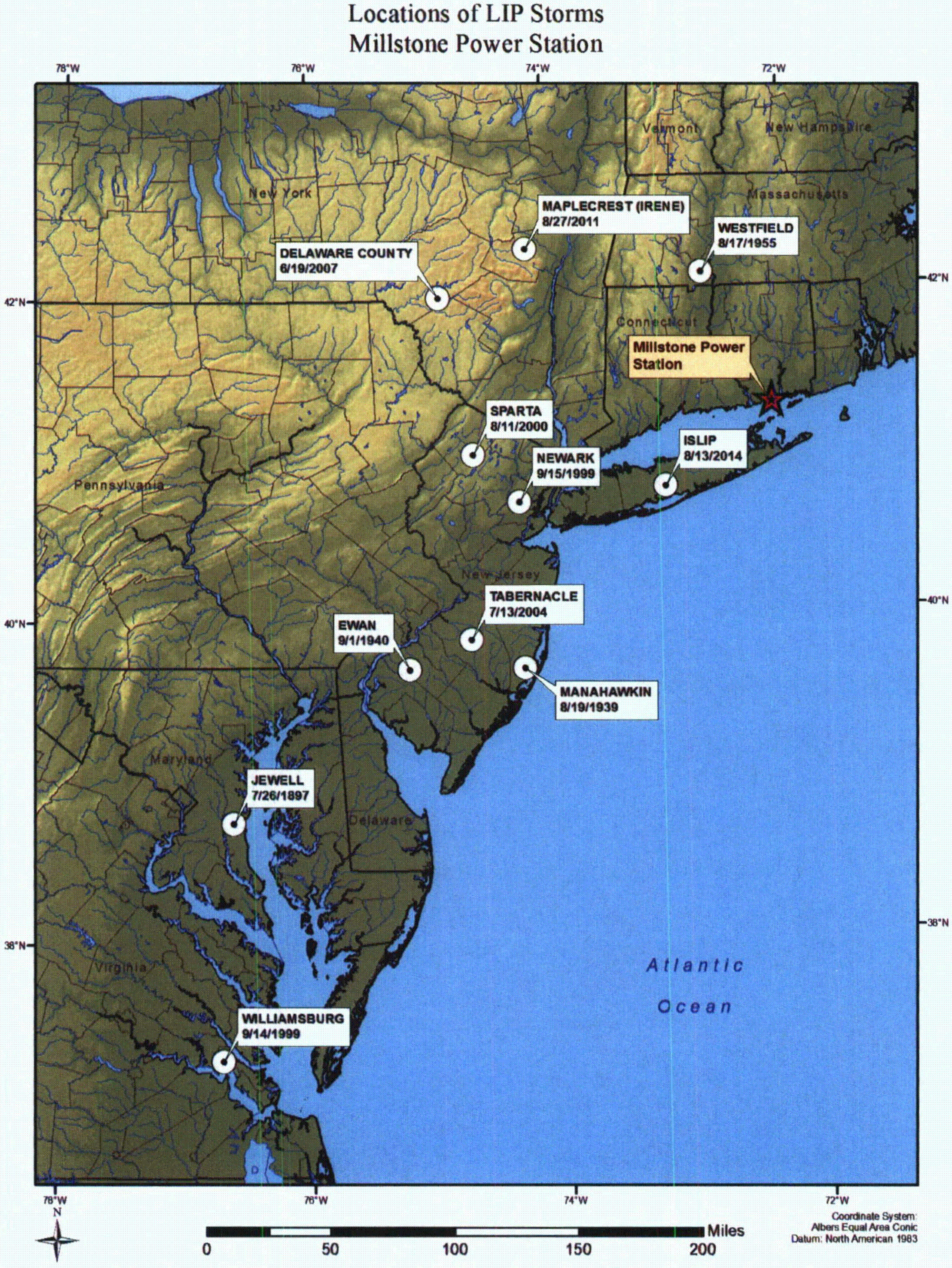


Figure 2.1-9: Site-Specific Meteorology Study-Six-Hour Incremental Hyetograph – Critical Loading Temporal Distribution

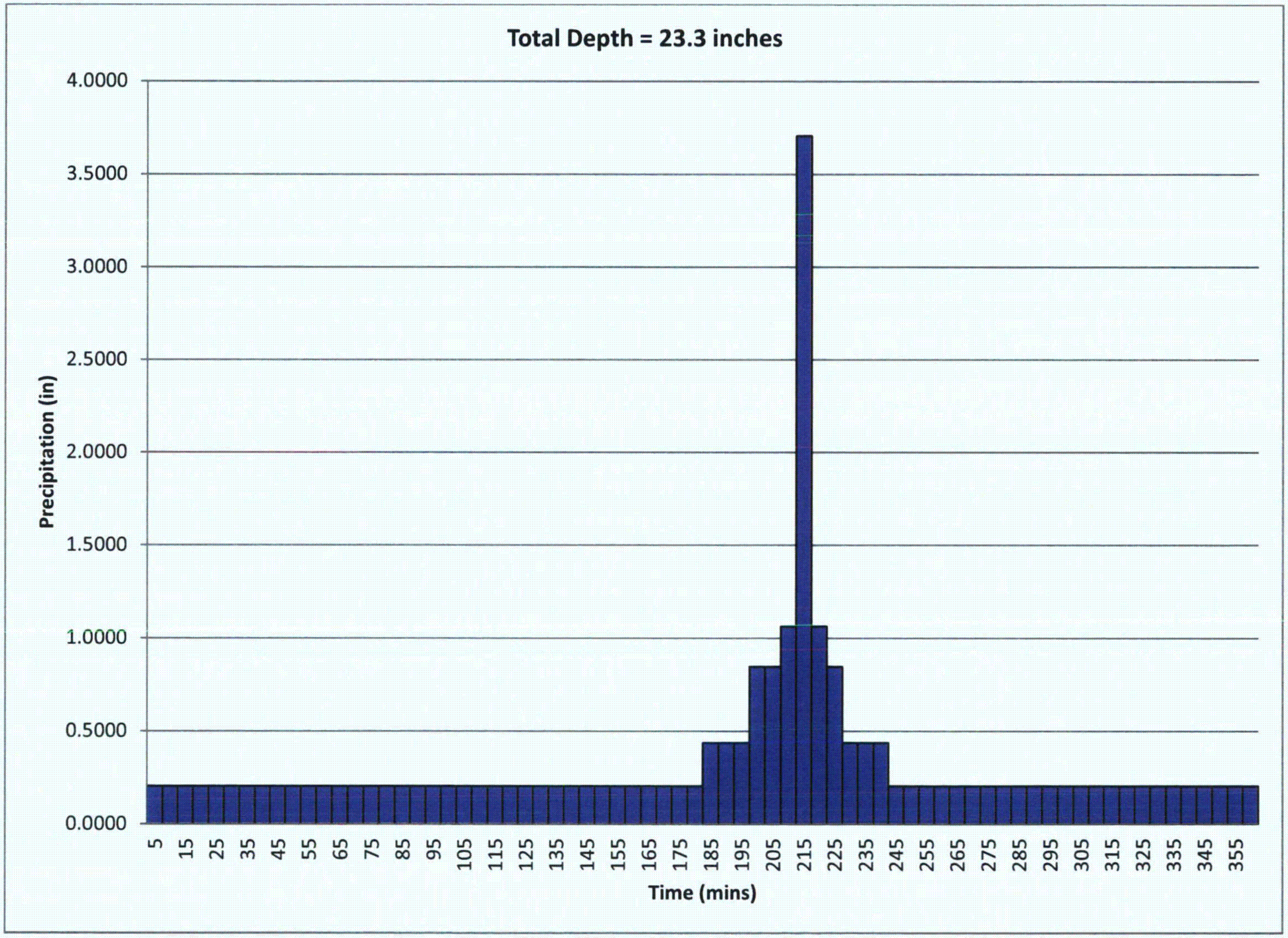
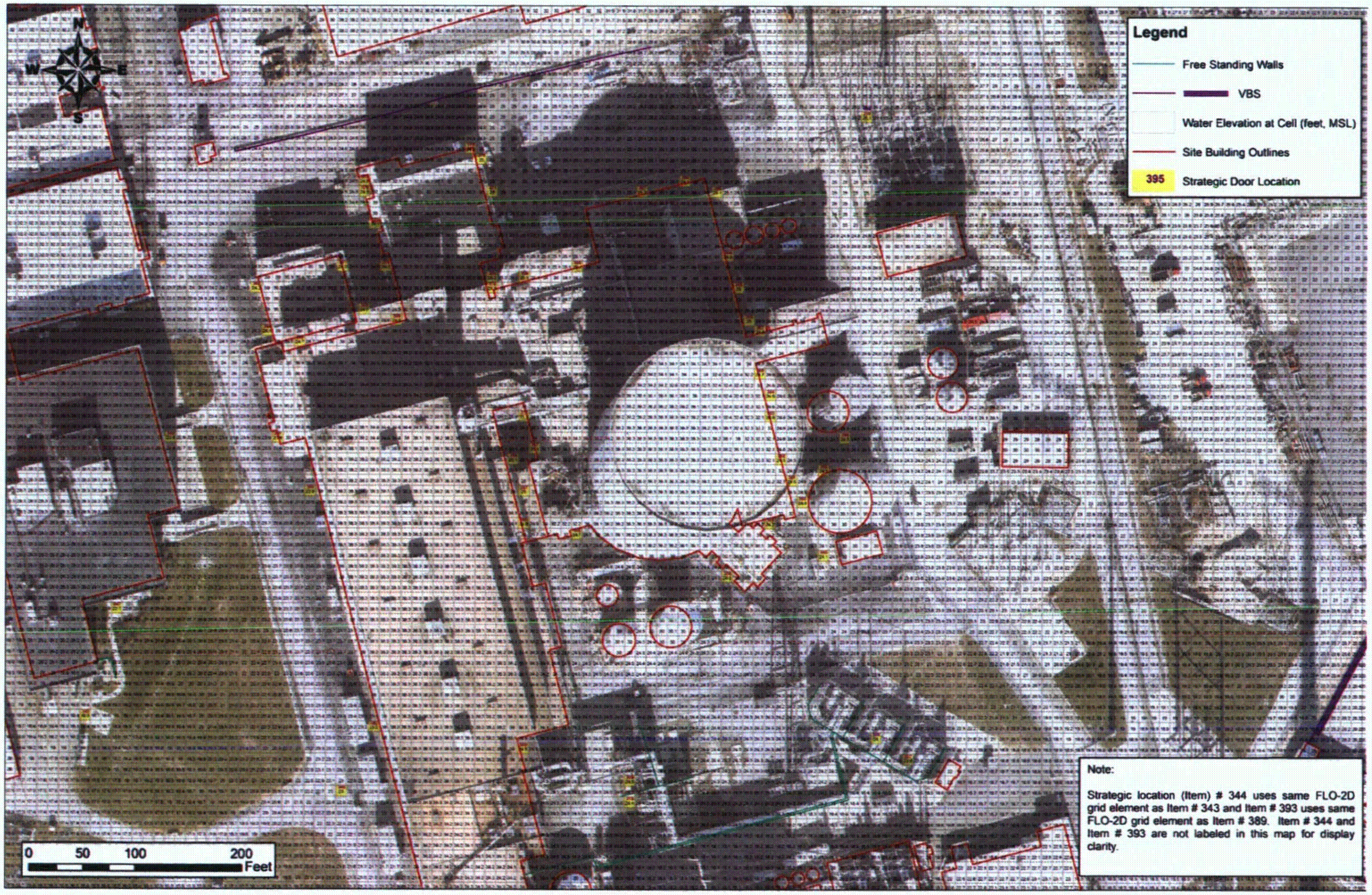
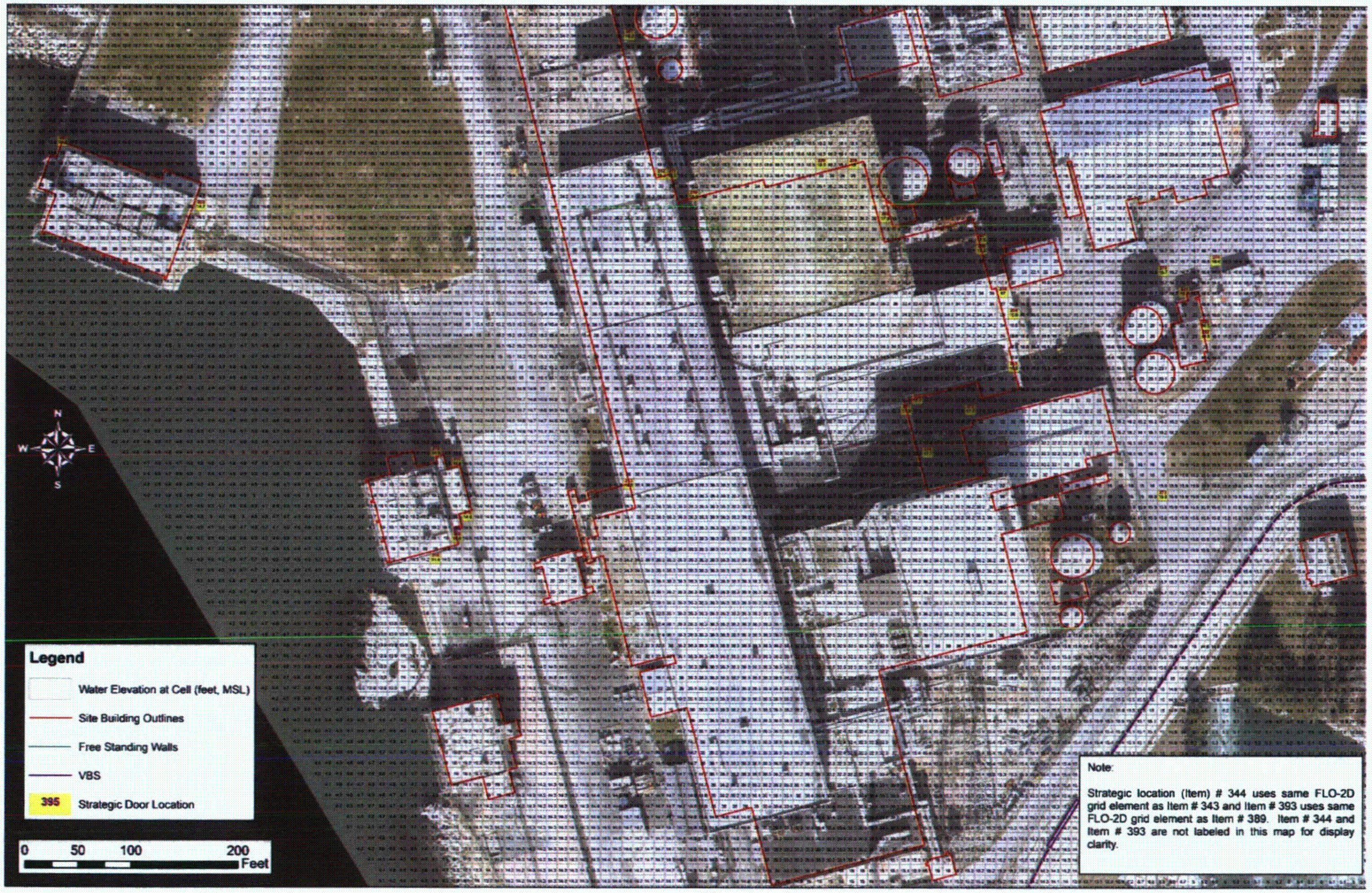


Figure 2.1-10: HMR-51/52 Simulation-Grid Element Maximum Water Surface Elevations – MPS3 Area (feet, MSL)



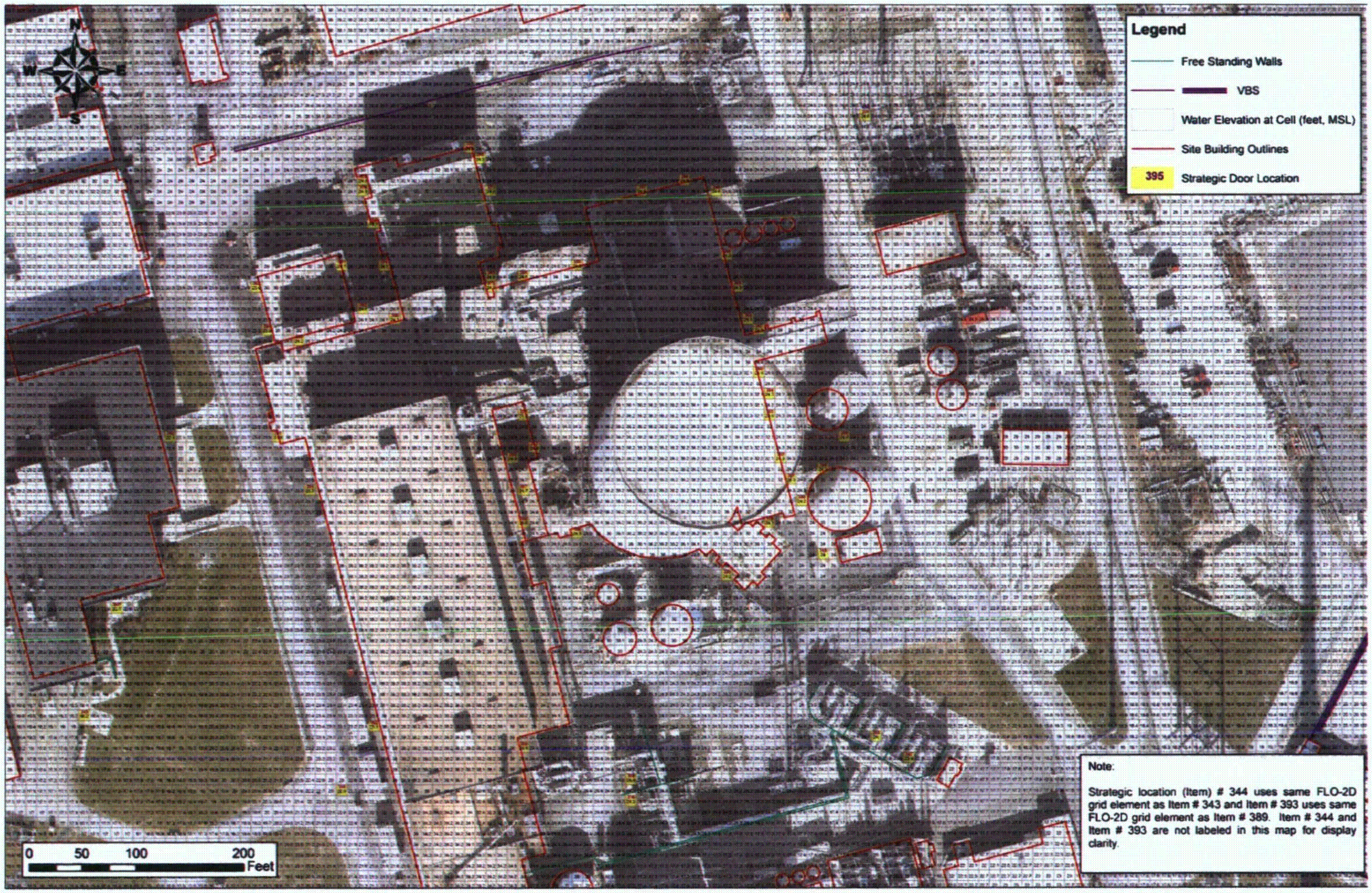
Note: Building Outlines from McKim & Creed, 2012a. Building rooftops do not represent actual roof elevations. Orthophoto from McKim & Creed, 2012a. Buildings shown in orthophotos in an oblique view.

Figure 2.1-11: HMR-51/52 Simulation-Grid Element Maximum Water Surface Elevations – MPS2 Area (feet, MSL)



Note: Building Outlines from McKim & Creed, 2012a. Building rooftops do not represent actual roof elevations. Orthophoto from McKim & Creed, 2012a. Buildings shown in orthophotos in an oblique view.

Figure 2.1-12: Site-Specific Meteorology Study Simulation-Grid Element Maximum Water Surface Elevations – MPS3 Area (feet, MSL)

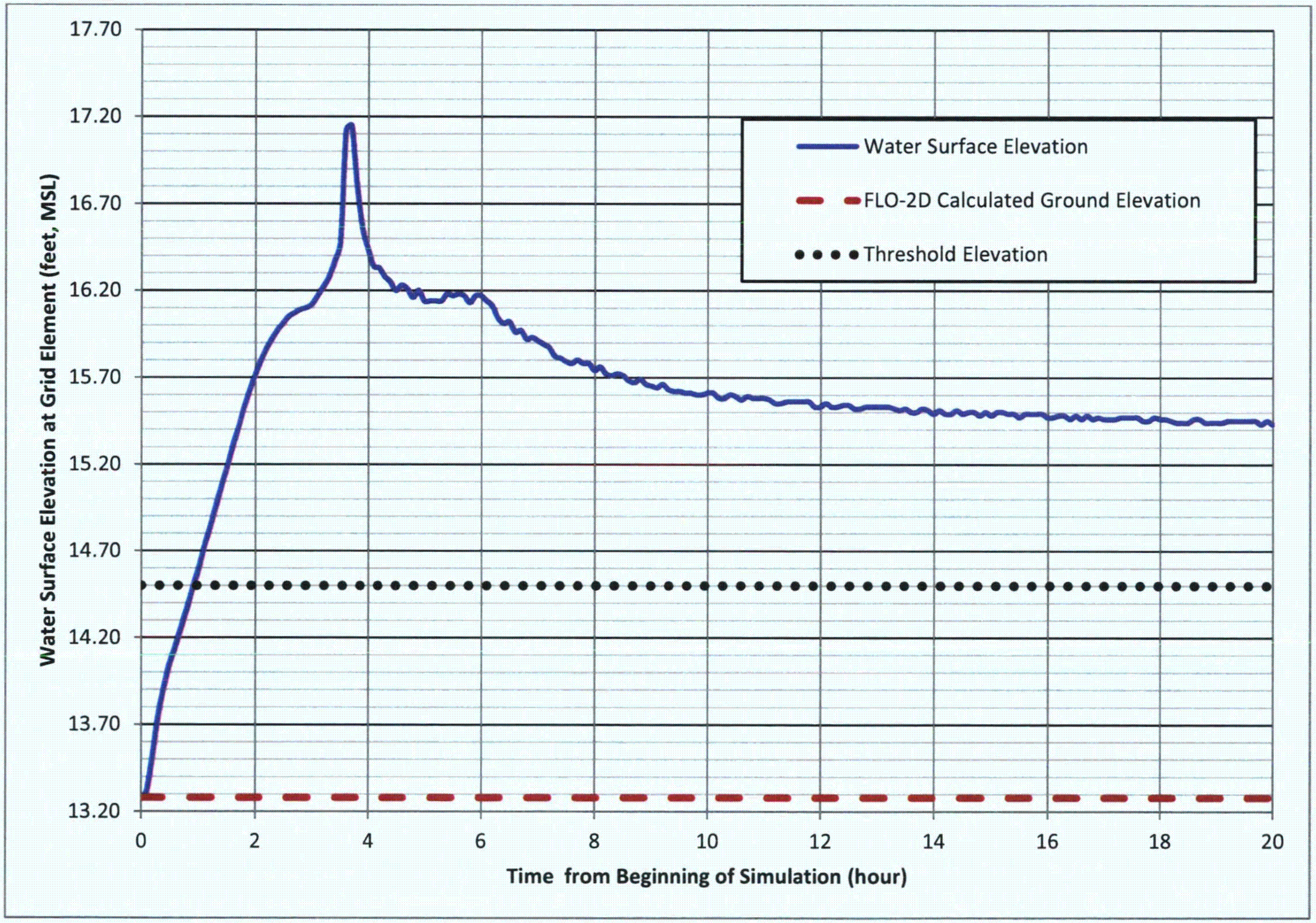


Note: Building Outlines from McKim & Creed, 2012a. Building rooftops do not represent actual roof elevations. Orthophoto from McKim & Creed, 2012a. Buildings shown in orthophotos in an oblique view. Larger figure encompassing entire model domain in Appendix B4

Figure 2.1-13: Site-Specific Meteorology Study Simulation-Grid Element Maximum Water Surface Elevations – MPS2 Area (feet, MSL)

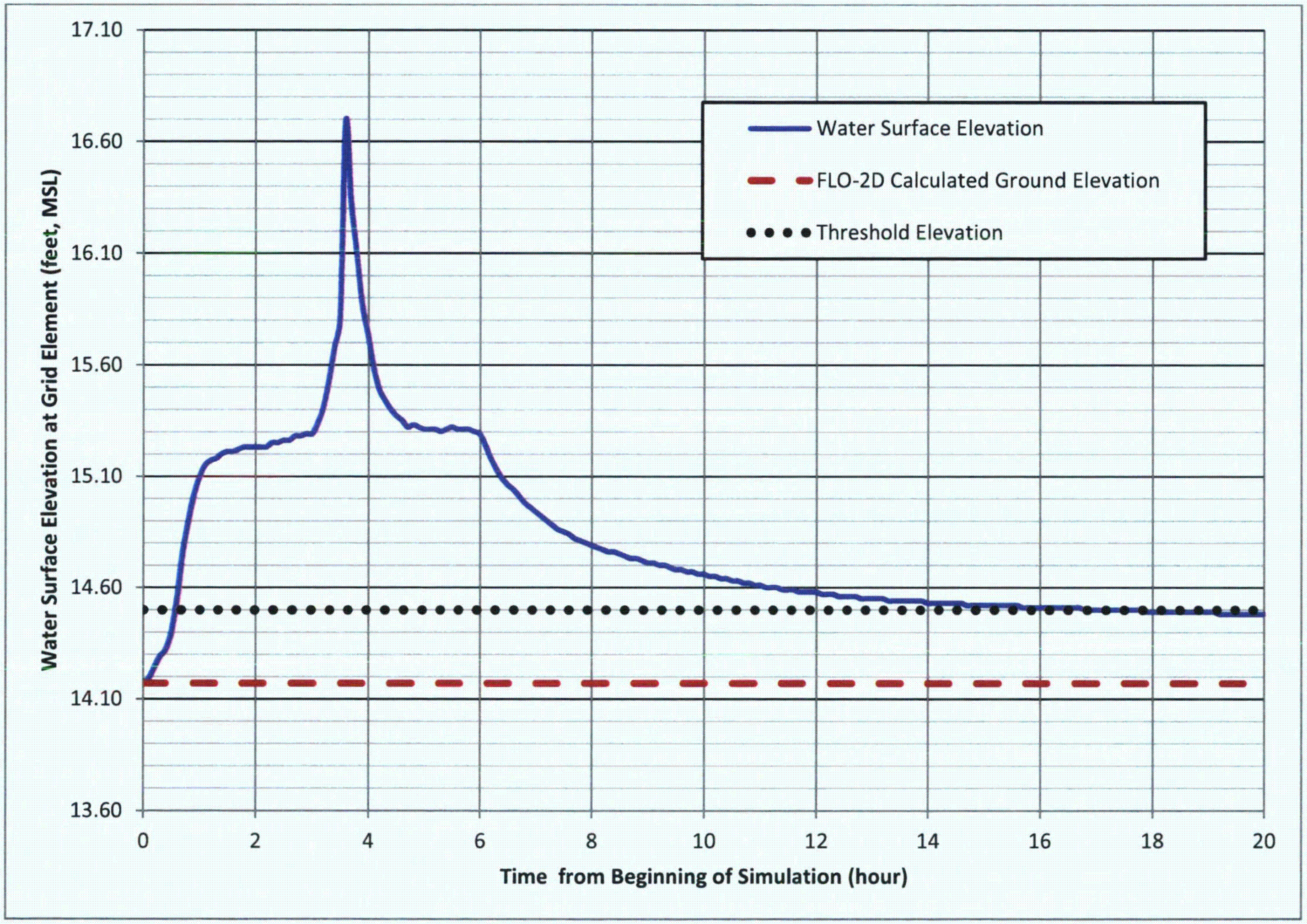
Note: Building Outlines from McKim & Creed, 2012a. Building rooftops do not represent actual roof elevations. Orthophoto from McKim & Creed, 2012a. Buildings shown in orthophotos in an oblique view. Larger figure encompassing entire model domain in Appendix B4.

Figure 2.1-14: Stage-Hydrograph for Item #210 (Flood Gate #12) Based on Site-Specific Meteorology Study Rainfall Input



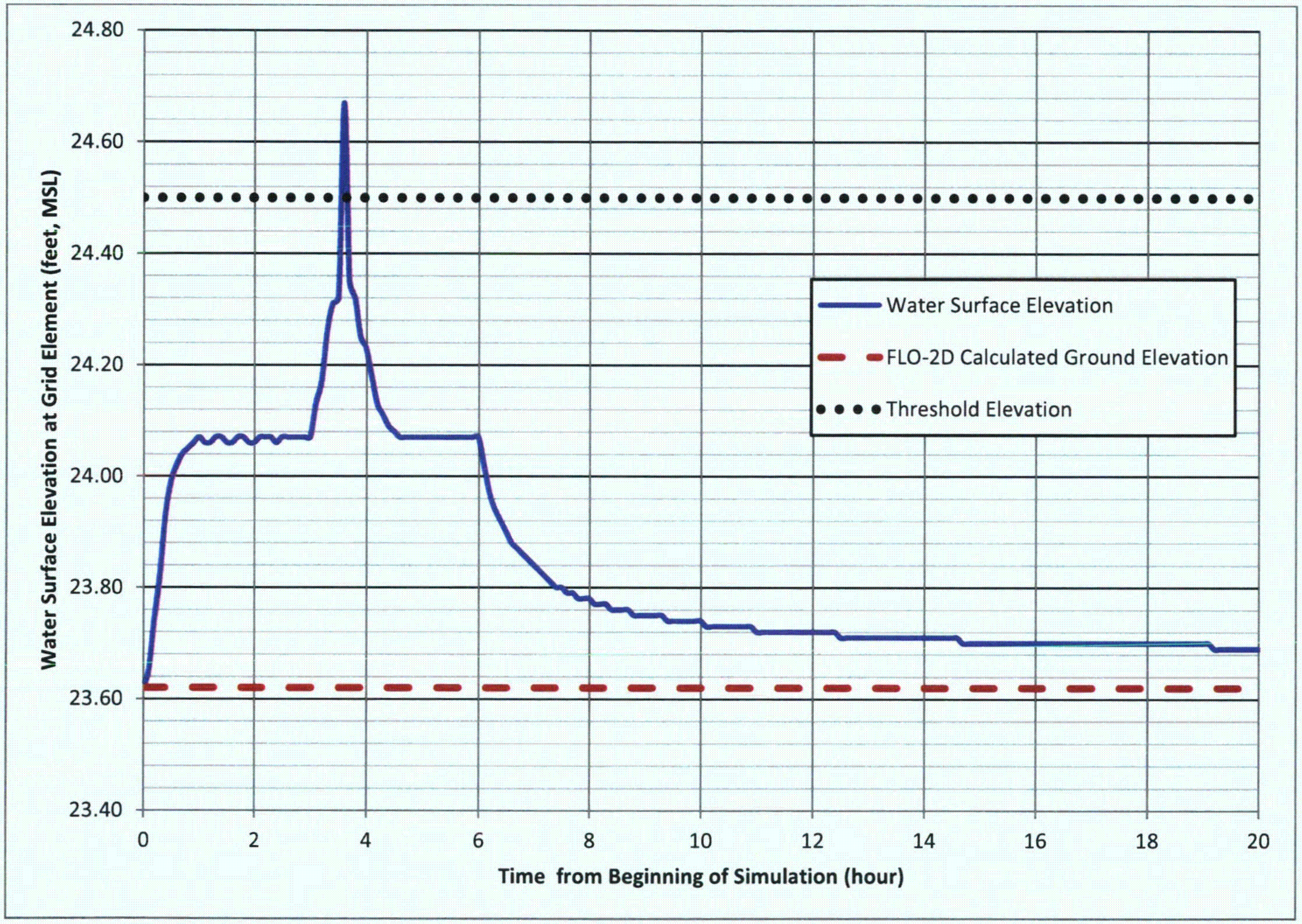
Note: For stage-hydrograph item # location, refer to "Strategic Door Location" in Figure 2.1-13.

Figure 2.1-15: Stage-Hydrograph for Item #214 (Flood Gate #21) Based on Site-Specific Meteorology Study Rainfall Input



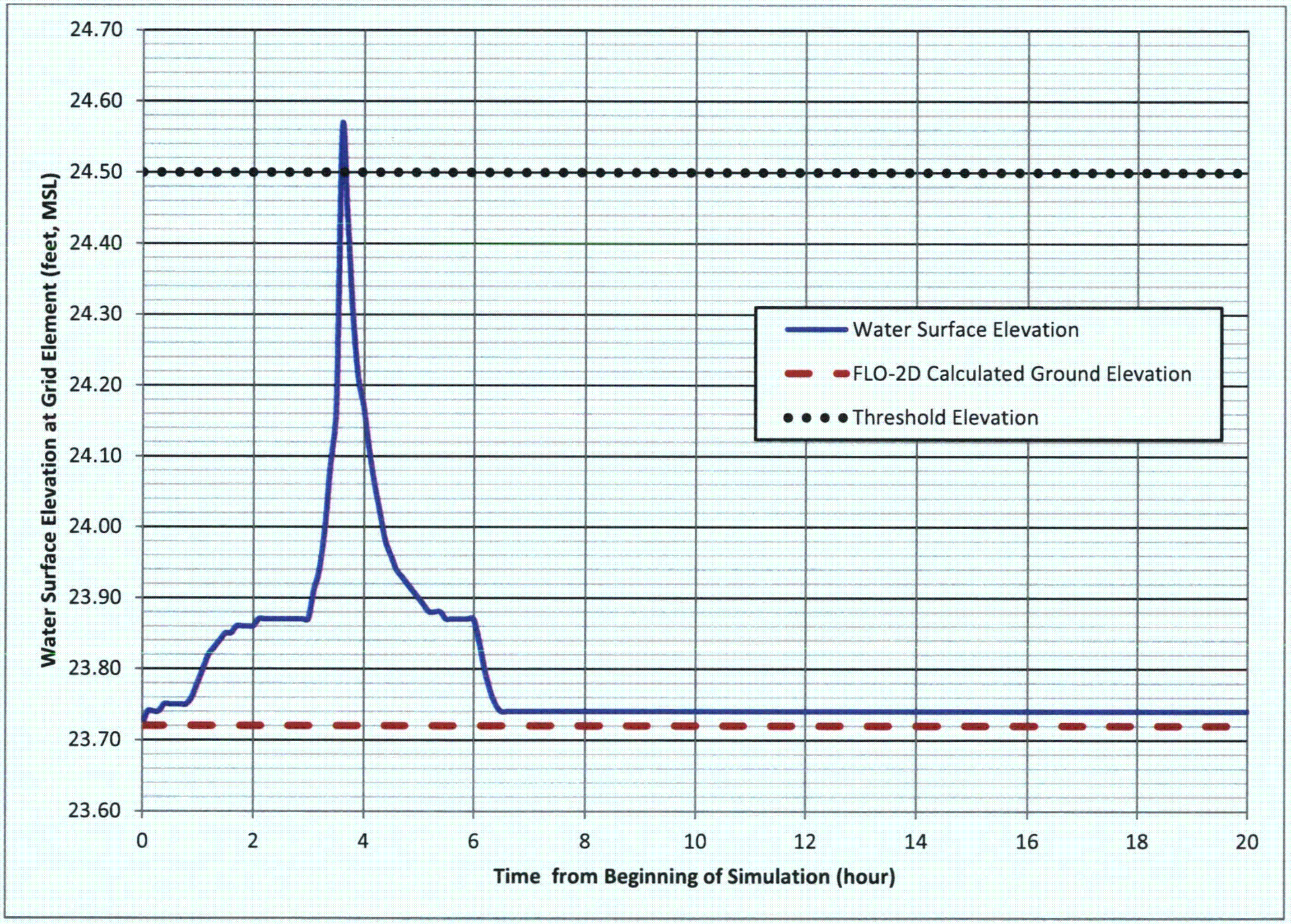
Note: For stage-hydrograph item # location, refer to "Strategic Door Location" in Figure 2.1-13.

Figure 2.1-16: Stage-Hydrograph for Item #357 Based on Site-Specific Meteorology Study Rainfall Input



Note: For stage-hydrograph item # location, refer to "Strategic Door Location" in Figure 2.1-12.

Figure 2.1-17: Stage-Hydrograph for Item #395 Based on Site-Specific Meteorology Study Rainfall Input



Note: For stage-hydrograph item # location, refer to "Strategic Door Location" in Figure 2.1-12.

2.2. Probable Maximum Flood on Rivers and Streams

This section addresses the potential for flooding at MPS due to the Probable Maximum Flood (PMF) on the small unnamed coastal stream near MPS. "The PMF is the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of the probable maximum precipitation (PMP) and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt." (NRC, 2011)

2.2.1. Method

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC, 2011; Section 2) was used for the evaluation of the PMF on the small coastal stream near MPS and resultant water surface elevation at MPS. Simplifying conservative assumptions such as treating the watershed as essentially impervious were used (see Section 2.2.2). The evaluation used the following steps:

1. Delineate the watershed for the small coastal stream near MPS.
2. Calculate the all-season PMP using BOSS HMR52 computer program and Hydrometeorological Report Numbers 51 and 52 (NOAA, 1978 and NOAA, 1982).
3. Calculate the seasonal PMP using HMR-53 (NOAA, 1980). Calculate snow melt potential in the watershed using the energy budget method. Add the snow melt to the seasonal rainfall to calculate the largest combined seasonal PMP and snow melt value (cool-season PMP).
4. Develop rainfall-runoff model using the U.S. Army Corps of Engineers HEC-HMS v3.5 computer model. The Soil Conservation Service (SCS, now known as the Natural Resources Conservation Service or NRCS) method was used to simulate the hydrology of the watershed. Model input parameters are drainage area, curve number (CN) and lag time (L).
5. Perform PMF hydrologic simulations with HEC-HMS, including required non-linearity adjustments.
6. Calculate the PMF Elevation on the small coastal stream near MPS.

Riverine flooding in the Niantic River was not analyzed because Millstone is a tidally influenced site located on the Niantic Bay shoreline. Therefore, flooding from the Niantic River is expected to dissipate into Niantic Bay and have a negligible effect on Millstone.

2.2.2. Results

2.2.2.1. Delineate Watersheds

The small coastal stream watershed near MPS was delineated (Figure 2.2-1) based on topography information (CTDEEP, 2011 and McKim & Creed ,2012). The watershed drainage area is approximately 87 acres.

2.2.2.2. Calculate the All Season PMP

The all-season PMP was calculated for the contributory watershed using the methodology of HMR-51 and HMR-52. The BOSS HMR52 computer program was used for the calculations. Inputs included the basin boundary coordinates, initial storm orientation, depth-area-duration values, and storm temporal order. The maximum duration of 72-hours used in HMR-51 and HMR-52 was conservatively adopted for the evaluation. The total rainfall depth for the all-season 72-hour PMP was 39.2 inches.

2.2.2.3. Calculate the Cool Season PMP

The seasonal variation of the PMP was evaluated in combination with snowmelt. Monthly PMPs were calculated using HMR-53 (NOAA, 1980) to develop the seasonal PMP depths for the 72-hour, 10 square mile (mi^2) storm by month of occurrence. Statistical monthly snow cover records at the New London and Groton precipitation gages (approximately 6 and 9 miles northwest of MPS, respectively) were obtained from NCDC (NCDC, 2012). The records indicate that there is possible snow cover between November and April.

The energy budget approach was used to determine snow melt for the seasonal PMP. The melt rate components for "Case 5" included in the U.S. Army Corps of Engineers (USACE) guideline (Table 5-4 of USACE, 1998) was conservatively used for the calculation, given the absence of site-specific snow melt meteorological parameters in the small watershed.

The rainfall for the rain-on-snow melt component was calculated as the product of the 6-hour incremental all-season PMP and the ratio of the cool season maximum monthly (November) 10 mi^2 PMP and all-season 10 mi^2 PMP. The twelve 6-hour increments that make up the 72-hour PMP were calculated individually and then summed to determine the total snow melt. The cool-season PMP, the sum of the snow melt and the cool season maximum monthly (November) 72-hour, 10-mi^2 seasonal PMP, was calculated to be 38 inches (Table 2.2-1).

2.2.2.4. Select the Controlling PMP

Comparison of the 72-hr, 10-mi^2 all-season PMP (39.2 inches) and the cool-season (combined precipitation and snow melt) 72-hr, 10-mi^2 PMP (38 inches) indicates that the all-season PMP would result in the more severe PMF. This is based on the all-season PMP total rainfall being larger than the sum of snowmelt and the seasonal PMP, and also upon the relative (higher) intensity of the rainfall-only all-season PMP in comparison to the snowmelt-influenced cool-season PMP. The snowmelt portion of the cool-season PMP is calculated in inches per day and distributed evenly over the 6-hr PMP increments used by HMR-52.

2.2.2.5. Develop HEC-HMS Model

The HEC-HMS hydrologic model was used to model the rainfall-runoff response of the watershed to the PMP. A Curve Number (CN) of 99 was conservatively used to assume no infiltration (NRCS, 1986). The time of concentration (Tc) flow path for the small coastal stream watershed is presented in Figure 2.2-2. The calculated lag time for the watershed, 60-percent of the Tc, was 50 minutes.

2.2.2.6. PMF Simulations

Nonlinearity adjustments were made to the HEC-HMS Dimensionless Unit Hydrograph to include a 20-percent increase in peak discharge of the unit hydrograph, a 33-percent reduction in time to peak of the unit hydrograph, and adjustments to the falling limb of the unit hydrograph to conserve the volume under the unit hydrograph (NRC, 2011). Using the calculated parameters and the adjusted unit hydrograph, the PMF was simulated using the controlling all-season PMP. An antecedent storm, 40-percent of the full 72-hour all season PMP, was also modeled.

The calculated peak discharge at MPS without non-linearity adjustment from the small coastal stream is 1,000 cubic-feet-per-second (cfs). The PMF peak discharge calculated at MPS incorporating non linearity adjustments is 1,100 cfs (Figure 2.2-3).

2.2.2.7. Calculate PMF Elevation

The PMF elevation for the small coastal stream was calculated at Millstone Road as shown in Figure 2.2-4. Millstone Road creates a small impoundment at the confluence of the small stream and Long Island Sound. Based on site observation, no culverts are apparent. A 3.5-feet-high concrete barrier runs along Millstone Road in this location. Therefore, the roadway was modeled as a broad crested weir using the weir equation.

Solving for H (hydraulic head) in the weir equation yields:

$$H = \left(\frac{Q}{CL}\right)^{2/3}$$

Flow (Q) was 1,100 cfs, as found with HEC-HMS. Weir length (L) was calculated as 190 feet per the site survey (McKim & Creed 2012, Figure 2.2-4). The weir coefficient (C) was conservatively assigned a value of 2.6 to represent an inefficient broad crested weir (USACE, 2010). Solving the weir equation yielded a head, or overflow depth above the concrete barriers of 1.7 feet. The invert elevation of the weir (Millstone Road) was 9.5 feet MSL. The resultant PMF elevation was therefore 11.2 feet MSL.

2.2.3. Conclusion

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 (see Section 3.2).

2.2.4. References

2.2.4-1 **NOAA, 1978.** Probable Maximum Precipitation Estimates – United States East of the 105^{th} Meridian, Hydrometeorological Report No.51 (HMR-51) by U. S. Department of Commerce & USACE, National Oceanic and Atmospheric Administration, June 1978.

2.2.4-2 NOAA, 1980. Seasonal Variation of 10-square-mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No.53 (HMR-53) by US Department of Commerce and US Nuclear Regulatory Commission, National Oceanic and Atmospheric Administration, April 1980.

2.2.4-3 NOAA, 1982. Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No.52 (HMR-52) by U. S. Department of Commerce & USACE, National Oceanic and Atmospheric Administration, August 1982.

2.2.4-4 NRC, 2011. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants, NUREG/CR-7046, United States Nuclear Regulatory Commission (USNRC), November 2011.

2.2.4-5 CTDEEP, 2011. Connecticut 2-foot contours vector digital data for the State of Connecticut, State of Connecticut, Department of Energy and Environmental Protection, May 2011.

2.2.4-6 McKim & Creed, 2012. Topographic information contained in the AutoCAD file "MPS_Contours.dwg," McKim & Creed, Inc., December 2012.

2.2.4-7 NCDC, 2012. Snow Climatology Data, National Climatology Data Center, National Oceanic and Atmospheric Administration, revised February 8, 2012, accessed February 8, 2012 http://www.ncdc.noaa.gov/ussc/index.jsp.

2.2.4-8 USACE, 1998. Runoff from Snowmelt, EM-1110-2-1406 by U.S. Army Corps of Engineers, March 1998.

2.2.4-9 NRCS, 1986. "Urban Hydrology of Small Watersheds", USDA Technical Release 55 (TR-55), Soil Conservation Service, June 1986.

2.2.4-10 USACE, 2010. HEC-RAS River Analysis System, Hydraulic Reference Manual, Version 4.1, January 2010.

Table 2.2-1: Summary of Probable Maximum Precipitation Inputs

Time Interval (6 hours each)	All Season Incremental Precipitation (in)	Cool Season Incremental Precipitation (in)	Case 5 Parameters					Snowmelt (in/6-hr)
			T (F)	Msw (in/day)	Ml (in/day)	Mce (in/day)	Mg (in/day)	
0-6	0.43	0.29	50	0.05	0.52	2.27	0.02	0.75
6-12	0.52	0.35	50	0.05	0.52	2.27	0.02	0.76
12-18	0.66	0.44	50	0.05	0.52	2.27	0.02	0.77
18-24	0.92	0.61	50	0.05	0.52	2.27	0.02	0.79
24-30	1.49	0.99	50	0.05	0.52	2.27	0.02	0.84
30-36	4.06	2.71	50	0.05	0.52	2.27	0.02	1.06
36-42	25.97	17.31	50	0.05	0.52	2.27	0.02	2.90
42-48	2.17	1.45	50	0.05	0.52	2.27	0.02	0.90
48-54	1.13	0.75	50	0.05	0.52	2.27	0.02	0.81
54-60	0.77	0.51	50	0.05	0.52	2.27	0.02	0.78
60-66	0.58	0.39	50	0.05	0.52	2.27	0.02	0.76
66-72	0.47	0.31	50	0.05	0.52	2.27	0.02	0.75
Total 72 hours	39.2	26.1						11.9

All Season PMP	39.2	in
Cool Season PMP	38.0	in

Figure 2.2-1: Watershed Delineation Map

Figure 2.2-2: Time of Concentration Flow Path

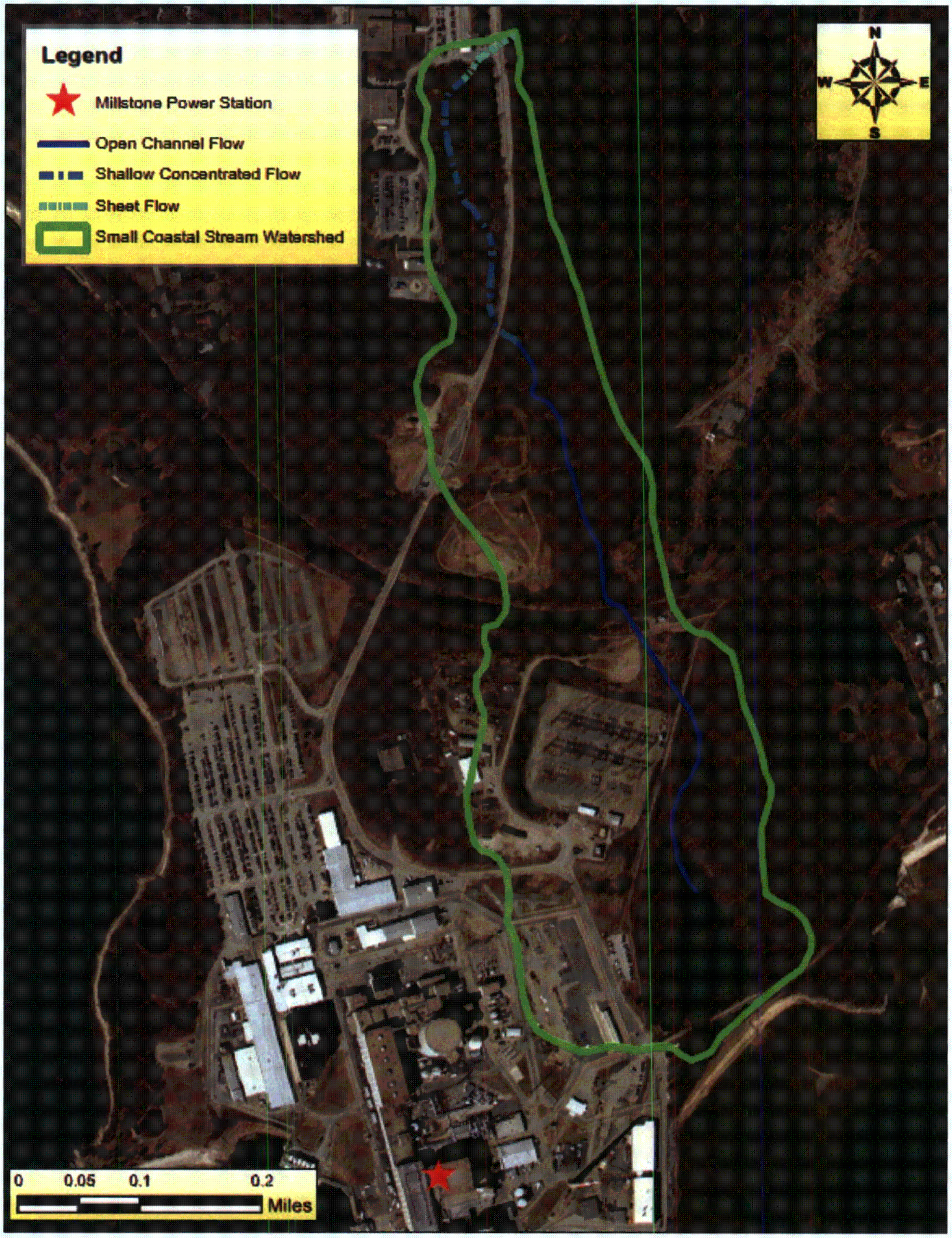


Figure 2.2-3: Results of 72 hour PMP With Nonlinearity Adjustments

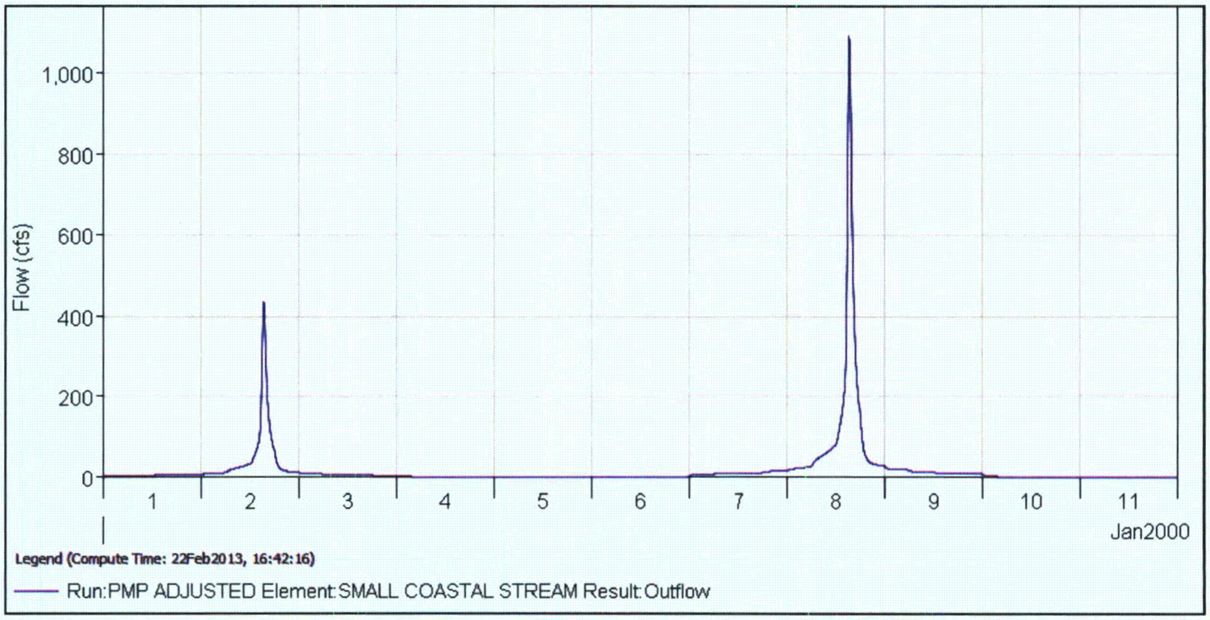


Figure 2.2-4: Millstone Road Location Map

2.3. Dam Failures

This section assesses the potential for flooding at MPS resulting from upstream dam failure. The local drainage area of Millstone and the watershed contributing to a small coastal stream located approximately 200 feet east of the ISFSI were evaluated for potential dam failures.

2.3.1. Method

The method of analysis consisted of identifying dams near MPS based on publically available dam database information (CT DEP, 1996 and USACE, 2010). The State of Connecticut Dams' GIS shapefile (CT DEP, 1996) and the National Inventory of Dams (NID) database (USACE, 2010) were used for identifying dams near Millstone in conjunction with the GIS layer / shapefile "Local Drainage Basins" (CT DEP, 2006), which includes the Connecticut statewide mapping of natural drainage basins.

2.3.2. Results

2.3.2.1. Dam Databases

A comparison was made between the State of Connecticut Dams' database and the NID database for the total number of dams in the State of Connecticut. The State of Connecticut Dam's database includes 3,646 dams and the NID's database includes 726 dams in Connecticut, state-wide. Figure 2.3-1 presents the location of the two dams in the region of MPS based on the State of Connecticut Dams' and the NID databases.

2.3.2.2. Identify Dam(s) near Millstone

The review of the databases did not identify any dams within the local drainage basins near MPS. The closest dam to MPS is Gardners Wood Road Pond Dam (GWRP Dam), which is located about 0.9 miles northeast of Millstone on a separate small coastal stream and watershed. The GWRP Dam is located about 1,000 feet upstream of Long Island Sound. Therefore, failure of GWRP Dam would flow directly into Long Island Sound without impact to Millstone based on its distinct local drainage basin location.

Note that upstream dam failures that drain to the Niantic River, upstream of Niantic Bay were not considered potential flood hazards to Millstone because Niantic Bay is freely and hydraulically connected to Long Island Sound. Therefore, upstream dam failure flows that reach Niantic Bay will dissipate quickly in the Bay (i.e., Long Island Sound), which is approximately two miles wide, and no significant increase in water surface elevation in Niantic Bay is expected as a result of upstream dam failures in Niantic Bay.

2.3.3. Conclusions

The potential for flooding at Millstone resulting from upstream dam failure is not applicable based on the lack of dams within the Millstone local drainage basin.

2.3.4. References

2.3.4-1 **CT DEP, 1996**. State of Connecticut Department of Energy and Environmental Protection (CT DEP), Connecticut Dams, Connecticut Department of Energy and Environmental Protection, GIS Data (http://www.ct.gov/deep/cwp/view.asp?a=2698&q=322898&deepNav_GID=1707) – data downloaded in February 2013. Data published in 1996.

2.3.4-2 **USACE, 2010**. National Inventory of Dams (NID), U.S. Army Corps of Engineers (USACE), http://geo.usace.army.mil/pgis/f?p=397:12 - data downloaded in February 2013. Data published in 2010.

2.3.4-3 **CT DEP, 2006**. Connecticut Local Drainage Basins, Connecticut Department of Energy and Environmental Protection, GIS Data http://www.ct.gov/deep/cwp/view.asp?a=2698&q=322898&deepNav_GID=1707 - data downloaded in February 2013. Data published in 2006.

Figure 2.3-1: Dams near Millstone