ATTACHMENT 1

BVY 13-021

VERMONT YANKEE

FLOOD HAZARD REEVALUATION REPORT

AREVA

AREVA NP Inc.

Engineering Information Record

Document No.: 51 - 9195290 - 000

Entergy Fleet Fukushima Program
Flood Hazard Reevaluation Report for
Vermont Yankee Nuclear Power Station (VYNPS)

Entergy Fleet Fukushima Program
Flood Hazard Reevaluation Report for Vermont Yankee Nuclear Power Station (VYNPS)

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Name and Title/Discipline	Signature	P/LP, R/LR, A-CRF, A	Date	Pages/Sections Prepared/Reviewed/ Approved or Comments
J. H. Snooks Advisory Scientist	[signature]	LP	3/7/2013	All
D. T. Brown Scientist III	[signature]	LR	3/7/2013	All
D. M. Leone GZA Hydraulic Engineer	[signature]	R	3/7/2013	Section 3
M. A. Rinckel, Technical Manager, Radiological & Environmental Analysis	[signature]	A	3/7/13	All
Leo Lessard, Project Manager	[signature]	A	3/7/2013	All

Note: P/LP designates Preparer (P), Lead Preparer (LP)
R/LR designates Reviewer (R), Lead Reviewer (LR)
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Overview

This report describes the approach, methods, and results from the reevaluation of flood hazards at the Vermont Yankee Nuclear Power Station (VYNPS). It provides the information, in part, requested by the U.S. Nuclear Regulatory Commission (NRC) to support the evaluation of the NRC staff recommendations for the Near-Term Task Force (NTTF) review of the accident at the Fukushima Dai-chi nuclear facility.

Section 1 provides introductory information related to the flood hazard. The section includes background regulatory information, scope, general method used for the reevaluation, assumptions, the vertical datum used throughout the report, and a conversion table to determine elevations in other common datum.

Section 2 describes detailed VYNPS site information, including present-day site layout, topography, and current licensing basis flood protection and mitigation features. The section also identifies relevant changes since license issuance to the local area and watershed as well as flood protections.

Section 3 presents the results of the flood hazard reevaluation. It addresses each of the eight flood-causing mechanisms required by the NRC as well as a combined effect flood. In cases where a mechanism does not apply to the VYNPS site, a justification is included. The section also provides a basis for inputs and assumptions, methods, and models used.

Section 4 compares the current and reevaluated flood-causing mechanisms. It provides an assessment of the current licensing and design basis flood elevation to the reevaluated flood elevation for each applicable flood-causing mechanism evaluate in Section 3.

Section 5 presents an interim evaluation and actions taken, or planned, to address those higher flooding hazards identified in Section 4 relative to the current licensing and design basis. Section 6 describes the additional actions taken to support the interim actions described in Section 5.

The report also contains one appendix, Appendix A, which describes the software model FLO-2D used in the reevaluation, including the quality assurance criteria and a discussion of validation of model-derived results.

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Acronyms and Abbreviations

Acronym/Abbreviation	Description
ANS	American Nuclear Society
ANSI	American National Standards Institute
CLB	Current License Basis
CFR	Code of Federal Regulations
cfs	cubic feet per second
DBD	Design Basis Document
FEMA	Federal Emergency Management Agency
FOST	Fuel Oil Storage Tank
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
IPEEE	Individual Plant Examination for External Events
ISFSI	Independent Spent Fuel Storage Installation
LIP	Local Intense Precipitation
MSL	Mean Sea Level
NAVD 88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NGDC	National Geophysical Data Center
NGVD 29	National Vertical Datum of 1929
NOAA	National Oceanic And Atmospheric Administration

Acronyms and Abbreviations
(continued)

Acronym/Abbreviation	Description
NRC	U.S. Nuclear Regulatory Commission
NTTF	Near-Term Task Force
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RHR	Residual Heat Removal
SSC	Structures, Systems and Components
SRP	Standard Review Plan
UFSAR	Updated Final Safety Analysis Report
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
VBS	Vehicle Barrier System
VYNPS	Vermont Yankee Nuclear Power Station

1.0 INTRODUCTION

Following the Fukushima Daiichi accident on March 11, 2011, which resulted from an earthquake and subsequent tsunami, the U.S. Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF) to review the accident. The NTTF subsequently prepared a report with a comprehensive set of recommendations.

In response to the NTTF recommendations, and pursuant to Title 10 of the Code of Federal Regulations, Section 50.54 (f), the NRC has requested information from all operating power licensees (NRC 2012). The purpose of the request is to gather information to re-evaluate seismic and flooding hazards at U.S. operating reactor sites.

The Vermont Yankee Nuclear Power Station (VYNPS), located on the Connecticut River in Vernon, Vermont, is one of the sites required to submit information.

The NRC information request relating to flooding hazards requires licensees to re-evaluate their sites using updated flooding hazard information and present-day regulatory guidance and methodologies and then compare the results against the site's current licensing basis (CLB) for protection and mitigation from external flood events.

1.1 Purpose

This report satisfies the "Hazard Reevaluation Report" Request for Information pursuant to 10CFR50.54(f) by the Nuclear Regulatory Commission dated November 12, 2012 NTTF Recommendation 2.1 Flooding Enclosure 2.

The report describes the approach, methods, and results from the reevaluation of flood hazards at the VYNPS.

1.2 Scope

This report addresses the eight flood-causing mechanisms and a combined effect flood, identified in Attachment 1 to Enclosure 2 of the NRC information request (NRC 2012). No additional flood causing mechanisms were identified for VYNPS.

Each of the flood causing mechanisms and the potential effects on the VYNPS site is described in Section 3 and 4 of this report.

1.3 Method

This report follows the Hierarchical Hazard Assessment (HHA) approach, as described in NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America" (NRC 2011) and its supporting reference documents.

A HHA consists of a series of stepwise, progressively more refined analyses to evaluate the hazard resulting from phenomena at a given nuclear power plant site to structures, systems, and components (SSC) important to safety with the most conservative plausible assumptions consistent with the

available data. The HHA starts with the most conservative, simplifying assumptions that maximize the hazards from the maximum probable event. If the assessed hazards result in an adverse effect or exposure to any safety-related SSC, a more site-specific hazard assessment is performed for the probable maximum event.

The HHA approach was carried out for each flood-causing mechanism listed in Section 2 and 3, with the design-basis flood being the event that resulted in the most severe hazard to the safety-related SSC at VYNPS. The steps involved to estimate the design-basis flood typically included the following:

1. Identify flood-causing phenomena or mechanisms by reviewing historical data and assessing the geohydrological, geoseismic, and structural failure phenomena in the vicinity of the site and region.

2. For each flood-causing phenomenon, develop a conservative estimate of the flood from the corresponding probable maximum event using conservative simplifying assumptions.

3. If any safety-related SSC is adversely affected by flood hazards, use site-specific data and/or more refined analyses to provide more realistic conditions and flood analysis, while ensuring that these conditions are consistent with those used by Federal agencies in similar design considerations.

4. Repeat Step 2; if all safety-related SSC are unaffected by the estimated flood, or if all site-specific data have been used, specify design bases for each using the most severe hazards from the set of floods corresponding to the flood-causing phenomena.

Section 3 of this report provides additional HHA detail for each of the flood-causing mechanisms evaluated.

Due to use of the HHA approach, the results (water elevation) for any given flood hazard mechanism may be significantly higher than results that could be obtained using more refined approaches. Where initial, overly conservative assumptions and inputs result in water elevations bounded by the CLB, no subsequent refined analyses are required to develop flood elevations that are more realistic or reflect a certain level of probability.

1.4 Assumptions

Assumptions used to support the flood reevaluation are described below in Section 3 and its subsections. Details relating to assumption justifications are discussed further in referenced documentation. Otherwise, none of the assumptions require verification, i.e., need to be confirmed prior to use of the results.

1.5 Elevation Values

Reference to elevation values in this report are based on mean sea level (MSL), unless otherwise stated.

MSL at VYNPS is equivalent to the National Geodetic Vertical Datum of 1929 (NGVD 29) (NOAA 2013a). To determine elevations in another datum, use the conversion table below (NOAA 2013b).

		To:		
	Datum	MSL (ft.)	NGVD 29 (ft.)	NAVD 88 (ft.)
From:	MSL	0	0	-0.46
	NGVD 29	0	0	-0.46
	NAVD 88	0.46	0.46	0

Where:

MSL = Mean Sea Level
NGVD 29 = National Geodetic Vertical Datum of 1929
NAVD 88 = North American Vertical Datum of 1988

1.6 References

NOAA 2013a. National Oceanic Atmospheric Administration, National Geodetic Survey Frequently Asked Questions, Website http://www.ngs.noaa.gov/faq.shtml, accessed January 9, 2013.

NOAA 2013b. National Oceanic Atmospheric Administration, National Geodetic Survey, VERTCON, North American Vertical Datum Conversion, Website http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html, accessed January 9, 2013.

NRC 2011. NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America" U.S. Nuclear Regulatory Commission (U.S. NRC), Springfield, VA: National Technical Information Service, 2011.

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

2.0 INFORMATION RELATED TO THE FLOOD HAZARD

2.1 Detailed Site Information

VYNPS is located in the town of Vernon, Vermont, on the west shore of the Connecticut River immediately upstream of the Vernon Hydroelectric Station (Figure 2.1-1).

The areas adjacent to the station are primarily farm and pasture lands. Downstream of the plant are the Vernon Hydroelectric Station and the town of Vernon, Vermont. The area within a 5-mile radius is predominantly rural with the exception of a portion of the town of Brattleboro, Vermont, and the town of Hinsdale, New Hampshire. Between 75% and 80% of the area within 5 miles of the station is wooded. The remainder is occupied by farms and small industries.

The Vernon Hydroelectric Station is the furthest downstream of a series of six hydroelectric projects on the reach of the river at or upstream of VYNPS. Storage reservoirs are also usable for power generation. Three of the dams, at 32, 75, and 132 miles upstream from the site, are relatively low structures developing heads of from 29 to 62 feet, with small amounts of storage. The large storage reservoirs are from 150 to 260 miles upstream from the Vernon Hydroelectric Station.

Under normal conditions, the flow of river water is largely determined by operation of the hydroelectric stations and by the upstream reservoirs and lakes.

2.1.1 Present-Day Site Layout

Figure 2.1-2 shows the VYNPS site layout and topography. The VYNPS site contains about 125 acres and is bounded on the north, south, and west by privately-owned land and on the east by the Connecticut River. In addition to the Reactor and Turbine Building, major site features include two switchyards and parking area northwest of the plant, the ISFSI north of the Reactor Building, the intake and discharge structures to the east and southeast, respectively, and the alternate cooling tower system to the south. The West Cooling Tower includes one cell that forms part of the Alternative Cooling System.

The site incorporates a concrete vehicle barrier system (VBS) as a security measure. Flow penetrations in the VBS allow surface water to flow through the VBS into natural drainage areas, which ultimately drain into the Connecticut River. Otherwise, the site is in the direct path of natural drainage to the west from the local watershed, with surface drainage flowing toward the river.

2.1.2 Site Topography

The site is located on a relatively flat area (Figure 2.1-2). Area topography generally slopes west to east toward the river. Nominal plant grade is at elevation 252.0 ft., but access to many plant SSCs important to safety, such as the Turbine, Reactor, Radwaste, and Control buildings, is at elevation 252.5 ft. The ISFSI is at elevation 254.0 ft. and the curb around the cooling tower basin is at elevation 250.5 ft. (VYNPS 2012b, Section 2.4.3.4).

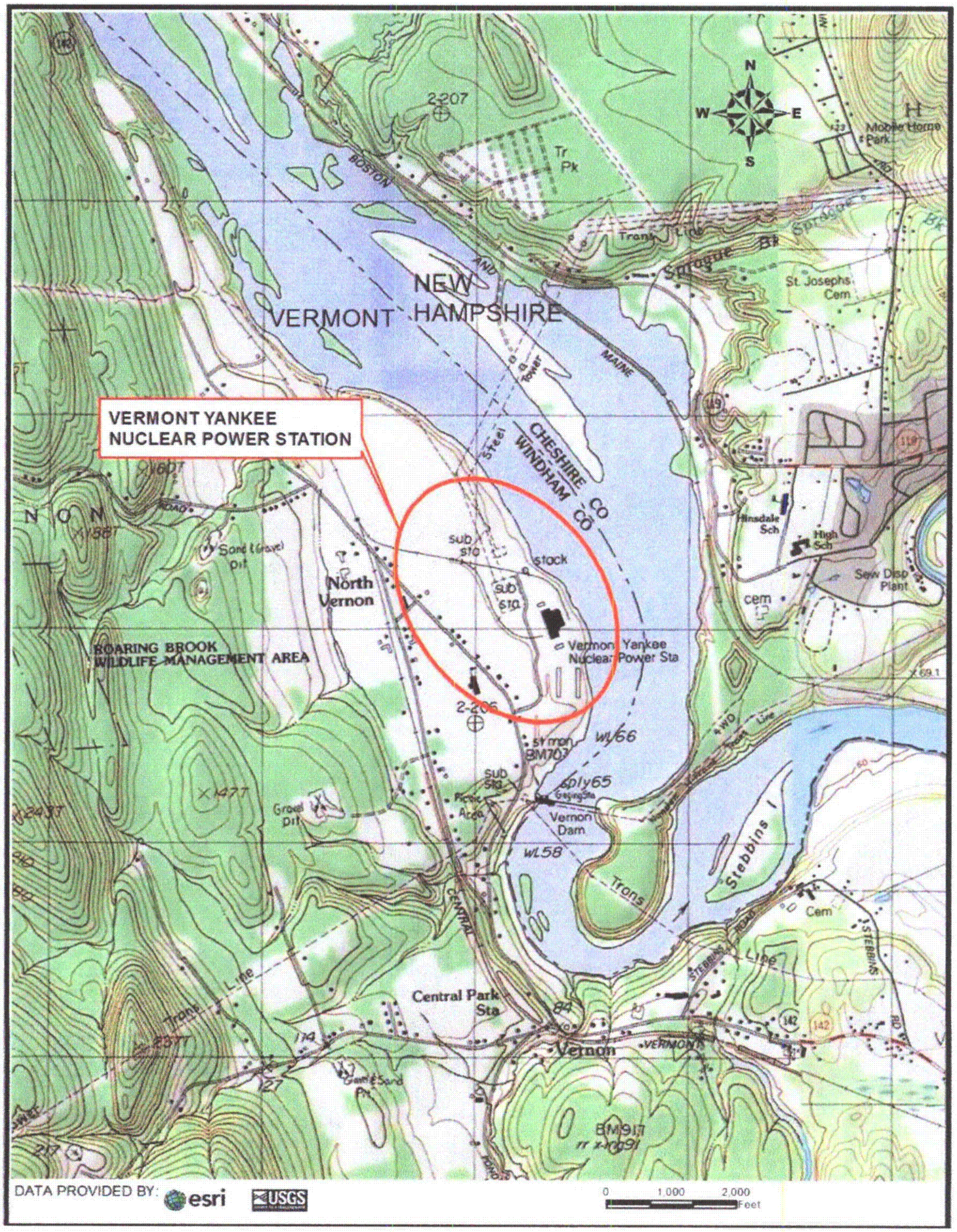
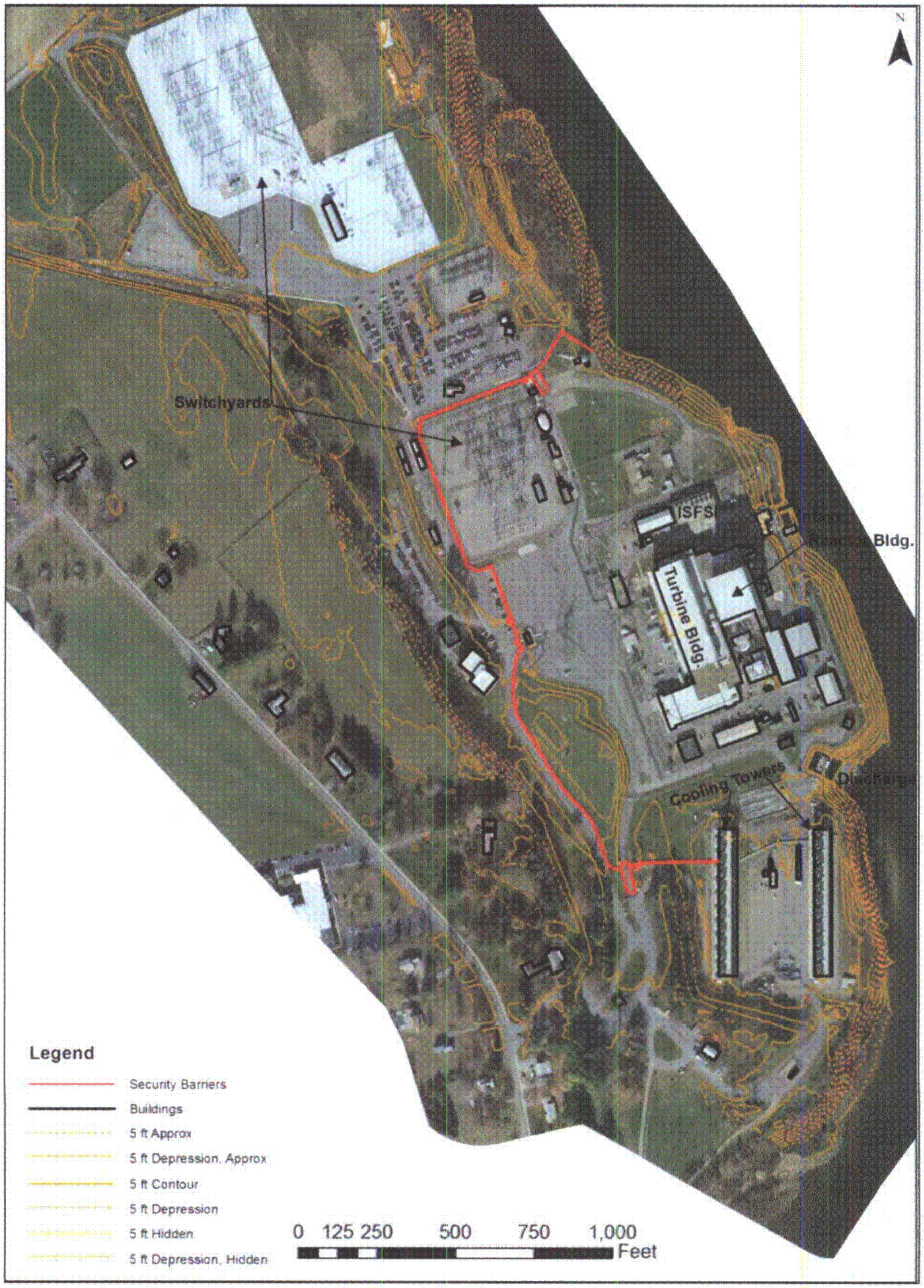


Figure 2.1-1: VYNPS Location Map



(Source: Sanborn 2013)

Figure 2.1-2: VYNPS Site Map and Topography

2.2 Current Design Basis Flood Elevations

The VYNPS current design basis and related flood elevations are described in the Topical Design Basis Document for External Events (Entergy 2005). The Design Basis Document (DBD) presents the external event licensing and design bases used in the design and construction of VYNPS. External flooding is part of the DBD scope. Additional discussion on the design basis flood hazard level is located in the recent walkdown report required as part of the 10 CFR 50.54(f) letter (Entergy 2012a) and the UFSAR, Section 2.4 (Entergy 2012b).

2.2.1 Background

VYNPS was issued a construction permit in 1967 and an operating license in 1972. During the period of initial design and construction, there was no regulatory criteria/guidance on implementing external flood design. The only external flooding design requirement at the time for design and construction was described in the Draft AEC General Design Criterion 2, which specified that nuclear power plants would be designed to withstand the forces imposed by natural phenomena including flooding conditions. (Entergy 2005, Section 1.2.3.1)

Initially, no special design considerations were required for external flooding (Entergy 2005, Section 1.2.3.2), which was based on the Plant Design & Analysis Report (PDAR) and the Atomic Energy Commission's Safe Evaluation Report (SER), both issued in 1967. However, the subsequent SER issued in1971, based on the contents of the FSAR, established the current design basis for external flooding.

2.2.2 Design Basis

Based on the DBD review of external flooding issues at VYNPS, the maximum PMF[1] flood level is 252.5 ft. at the main plant structures. No changes have been made to this value since the time of original plant licensing (Entergy 2005, Section 1.2.3.2).

2.3 Current Licensing Basis Flood Protection and Mitigation Features

The DBD (Entergy 2005) consolidates the current external event design and licensing bases for the VYNPS. The DBD includes a compilation of the licensing bases associated with external events. These licensing bases include the original licensing positions, as amended with formal VYNPS licensing commitments since issuance of the Operating License.

2.3.1 Current Licensing Basis (CLB)

The 1971 SER accepted the maximum PMF level of 252.5 ft. as the stillwater elevation. However, the SER concluded that wave effects could produce plant flooding as high as 254 ft.

[1] In the original FSAR, VYNPS referred to this flood as the Maximum Probable Flood (MPF). The term PMF used in the DBD is synonymous with MPF and consistent with the updated FSAR.

As a result, the CLB requires that VYNPS be prepared to emplace sandbags so as to protect vital systems from flooding at elevations up to 254 ft. (Entergy 2005, Section 1.2.3.2). This requirement is controlled by VYNPS operating procedure (Entergy 2012c).

2.3.2 Flood Causing Mechanisms

The following is a summary of the flood causing mechanisms that are part of the CLB. The reference sources cited provide more detail for each mechanism.

1. Local Intense Precipitation (LIP) – The **CLB excludes LIP**. However, as part of the Individual Plant Examination of External Events (IPEEE), updated local probable maximum precipitation (PMP) criteria were used to perform a detailed evaluation of site ponding due to LIP. The analysis estimated local site flooding depths around the major plant structures using a flood runoff model and determined that the plant is protected by a combination of design features and procedure action up to the local Probable Maximum Precipitation (PMP) (Entergy 2004, Section 1.4.4).

2. Probable Maximum Flood (PMF) – The CLB controlling source of flooding at VYNPS is a PMP-induced Probable Maximum Flood (PMF) with stillwater elevation of 252.5 ft. (Entergy 2005, Section 1.2.3.2 and Entergy 2012b).

3. Wind and Wave Activity – The CLB includes the effect of wind-generated waves on the PMF stillwater elevation per the 1971 SER, which postulates that the site could be subjected to a wave run up of 254 ft.

4. Potential Dam Failures – The CLB includes flooding at VYNPS due to upstream dam failures; however, the resulting stillwater elevations are well below the PMF level at the site (Entergy 2005, Section 1.2.3.2).

5. Probable Maximum Surge and Seiche - The **CLB excludes flood levels at VYNPS due to hurricane surge or seiche**. These events were not considered because the site is not located in a coastal region (Entergy 2012a, Section 2.1.6).

6. Probable Maximum Tsunami – The **CLB excludes flooding at VYNPS due to tsunamis**. These events were not considered because the site is far removed from the Atlantic Ocean and any source of tsunami activity (Entergy 2012a, Section 2.1.7).

7. Ice Effects - The **CLB excludes flood levels at VYNPS due to ice effects**. Flooding due to Ice Effects was addressed during IPEEE, which concluded that the PMF level of 252.5 ft., which is about 32 ft. above the normal maximum Connecticut River levels, is much greater than any conceivable ice flooding in the river adjacent to the site (Entergy 2004, Section 5.2.1.2).

8. Channel Diversions - The **CLB excludes flooding at VYNPS due to channel diversions**. The effects of Channel Diversion were address during IPEEE, which concluded that there are no detrimental effects because there is no known mechanism for the Connecticut River to be diverted around or away from Vernon Pond, which is located along the main stem of the Connecticut River and where the plant draws its cooling water (Entergy 2004, Section 5.2.1.2).

9. Cooling Water Canal and Reservoir - The **CLB excludes flooding at VYNPS due to failure of a cooling water canal or reservoir** because all active components of the alternate cooling system are designed for the PMF event (Entergy 2012a, Section 2.1.9).

2.3.3 CLB Flood Protection and Mitigation Features

The equipment required for operation during a PMF include the residual heat removal (RHR) pumps and heat exchangers, the RHR service water pumps, a cooling tower cell, and the electrical and piping systems required for operation of these components. If normal electrical power is unavailable, diesel generators, fuel, and fuel oil pumps are also required (Entergy 2012b, Section 2.4.3.4).

- The RHR pumps, the RHR heat exchangers and RHR service water pumps are located within the Reactor Building. The emergency diesel generators are located within the Turbine Building and the electrical equipment is located within the Turbine and Control Buildings. The fuel oil pumps are located within a structure that forms the tornado and flood protection around the fuel oil tank. Since the entrances to all of these structures are at elevation 252.5 ft., they are at the maximum flood stage and, thus, are protected against the PMF.
- The curb around the cooling tower basin is at elevation 250.5 ft. The basin will be inundated by the PMF, but since no active components are within the basin, there should be no interruption of cooling water flow. All yard valves will have been lined up to permit alternate cooling water system operation by the time the flood stage reaches 237.0 ft., the point at which the station would have to shut down due to inundation of the circulating water pump motors. Service water pumps would provide for normal reactor cooling. When their service is terminated due to river water leakage into the intake structure, the alternate cooling water system would be put into service from within the plant to provide for reactor cooling.
- The diesel generators fuel oil is supplied from the Fuel Oil Storage Tank (FOST). The fuel oil pumps are located within a structure which forms the tornado protection around the FOST. Since the entrances to all of these structures are at elevation 252.5 ft., they are at maximum flood stage and, thus, are protected against the PMF. In addition, these entrances are protected against wave run up to 254 ft. by sandbag and plywood barricades emplaced per plant operating procedure.

In addition, several enhancements were made to the plant flooding procedure along with several plant modifications during the IPEEE process for external flooding. The enhancements included installing removable plugs on switchgear room floor drains, placing sandbags around entrances, such as the exterior doors to the East Switchgear Room, providing pumps for manways in the switchgear room, and installation of flood seals (Entergy 2004, Section 1.2.3.2).

As a result, the plant is suitably protected against the PMF. The equipment required for reactor shutdown is within buildings that would not be inundated by the flood. Both the service water system (required portions of) and the alternate cooling water system would be available for shutdown cooling.

2.4 Licensing Basis Flood-Related and Flood Protection Changes

The plant design features and their functional requirements that provide protection against the design basis external flood mechanisms are provided in the DBD (Entergy 2005). Attributes of the overall plant configuration that support the design for external flooding are also identified.

The only change that has been made to those flood protection mitigations and enhancements described in Section 2.2.3 above is the recent revision to the natural phenomena (sandbagging) procedure to address an observation identified in the walkdown report (Entergy 2012a, Section 7.3.3). Otherwise, as noted in the walkdown report, no flood protection features were determined to be nonfunctional and no deficiencies were observed (Entergy 2012a, Section 7.1).

2.5 Watershed and Local Area Changes

2.5.1 Watershed Changes

The watershed boundaries have not changed but four new dams have been constructed in the watershed since 1971. However, none of these dams were among the selected significant dams used in calculations.

The downstream Vernon Dam underwent significant modifications in 1985-1986, with crest gates and other controls added to the spillway. Demographics of the watershed have also changed.

In 1970, the population of the Upper Connecticut Watershed (in New Hampshire and Vermont) was approximately 275,000. In 2010, the population was approximately 375,000. Increasing population is commonly accompanied with development of land, and thus, increased impervious area may be inferred.

2.5.2 Local Area Changes

Local area changes have been minimal since plant operation began at the site. Offsite areas within 5 miles of the plant to the north, south and, west remain largely rural with privately-owned land that is mostly wooded. The remainder is occupied by farms and small industries. (Entergy 2006, Section 2.1)

On site, major changes include the addition of a security barrier around the plant, known as the Vehicle Barrier System (VBS). Examples of other significant construction projects include

- another switchyard and adjacent parking lot outside the VBS and north of the plant,
- ISFSI, inside the VBS and north of the Reactor Building, and
- Plant Support Building and Power Uprate Building outside the VBS and west of the plant.

Other minor changes have been made, but none significant enough to affect site drainage.

2.6 Additional Site Details – Walkdown Results

A total of 220 walkdown flood protection features were reviewed during with the walkdown completed at VYNPS in November 2012. None of the features was inaccessible or restricted access. See Entergy 2012a, Section 7.

Of the total flood protection features, 217 were defined as passive-incorporated, and one each as passive-temporary, active-incorporated, and active-temporary. None of the flood protection features was determined to be non-functional and no deficiencies were observed.

All observations resulting in a potential deficiency were entered into the plant's corrective action program and an operability determination associated with the observation. None of the flooding conditions observed during the walkdown was determined to pose a risk to the safe operation of the plant and no safety-related or safe-shutdown equipment is adversely impacted by these conditions.

Three of the observations required corrective action. Two related to repairing penetrations and the other to revising the procedure relating to providing detailed sandbagging guidance.

2.7 References

Entergy 2004. VYNPS Individual Plant Examination of External Events (IPEEE), Rev. 2, November 2004 (AREVA Document No. 38-9193644-001).

Entergy 2005. VYNPS Design Basis Document "EXEV," VYNPS Topical Design Basis Document for External Events, August 2005 (AREVA Document No. 38-9193644-001).

Entergy 2006. Environmental Report, Operating License Renewal Stage, Appendix E, January 2006 (AREVA Document No. 38-9193644-001).

Entergy 2012a. Engineering Report Number VY-RPT-12-00020, Rev. 0, VYNPS Walkdown Submittal Report for Resolution of Fukushima Near Term Task Force Recommendation 2.3: Flooding per NEI-12-07 and NRC 10CFR50.54(f), November 2012 (AREVA Document No. 38-9193644-001).

Entergy 2012b. VYNPS Updated Final Safety Analysis Report, Revision 25, May 2012.

Entergy 2012c. VYNPS Operating Procedure OPOP-PHEN -3127, Rev 10, Natural Phenomena, March 2013 (AREVA Document No. 38-9193644-002).

Sanborn 2013. Vermont Yankee (VY) Topographic Survey, Sanborn Map Company, Inc., January 2013 (AREVA Document No. 38-9196957-000).

3.0 FLOOD HAZARD REEVALUATION

This section details the evaluation of the eight flood causing mechanisms and combined effects for VYNPS as detailed in Attachment 1 to Enclosure 2 of the NRC information request. No additional flood causing mechanisms were identified for VYNPS.

Each of the flood causing mechanisms and the potential effects on the VYNPS site is described in the subsections below. None of the mechanisms result in a hazard to SSCs important to safety due to debris loading and transportation (See Sections 3.1.3 and 3.9.2.3).

3.1 Local Intense Precipitation

The Local Intense Precipitation event is a distinct flooding mechanism that consists of a short duration, locally heavy rainfall centered upon the plant site itself. Based on NUREG-7046, the LIP is deemed equivalent to the 1-hr, 1-mi^2 PMP (NRC 2011, Section 3.2).

3.1.1 Method

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011) was used for the evaluation of the LIP and resultant water surface elevation at VYNPS.

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
   a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
   b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

With respect to LIP, the HHA 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.
4. Perform flood simulations in FLO-2D and estimate LIP maximum water surface elevations throughout the VYNPS site.

In addition, the LIP evaluation included the assumption that 1) all active and passive storm drainage structures are non-functional, including catch basins, storm drains, channels, and culverts, and 2) the contributory drainage area is completely impervious (i.e., no infiltration losses). The vehicle barrier system, which surrounds most of the site, was not included; however, model test runs were performed to verify that excluding its effects was conservative.

3.1.2 Results

3.1.2.1 FLO-2D Model Limits

Due to anticipated unconfined flow characteristics, a two-dimensional hydrodynamic computer model, FLO-2D, was used for the LIP Analysis. 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. The watershed applicable for the LIP Analysis was computed internally within FLO-2D based on the limits of the digital terrain model (DTM) input into FLO-2D. The DTM was extracted from the site topographic survey prepared by photogrammetric methods using aerial photography supplemented by surveyed information included in the topographic map of the site (AREVA 2012). The DTM limits were defined based on the local watershed divides that defines the contributory drainage area to VYNPS. The total drainage area found to contribute rainfall to the VYNPS site is approximately 152 acres. More information on the FLO-2D software, including model validation, is provided in Appendix A.

The VYNPS site is generally graded to convey runoff towards the river (i.e., east direction) directly via overland flow and through an underground storm drainage system. The site is in the direct path of natural drainage from higher ground to the west of VYNPS.

3.1.2.2 FLO-2D Model Site Features

A FLO-2D model was developed for the LIP analysis based on VYNPS site features including: topography, site location, and building structures. The model grid size selected was 40 feet by 40 feet. The model was thus comprised of 5,162 grid elements. Grid element elevations were established on the basis of the site DTM data. Outflow grid elements along the model computational boundary were selected as outflow grid elements. Buildings were incorporated as completely blocked grid elements based on assessment of aerial photography (AREVA 2013). The computational boundary, outflow elements, and incorporated buildings within the VYNPS site are shown in Figure 3.1-2.

The land use categories were used to assign Manning's roughness coefficients to each model grid element and were selected based on aerial photography assessment. The 2-D Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete or paved areas to 0.4 for wooded areas. The Manning's n values selected were based on two references including: Chow (Chow 1959) and FLO-2D Software documentation (FLO-2D 2012).

Concrete security barriers (VBS) which serve to redirect flows away from VYNPS were not considered, as they are not currently credited as flood mitigation structures. Model runs that included the VBS as structures capable to re-direct overland flow away from safety-related SSC were found to be less conservative, resulting in lower water elevations (AREVA 2013a). All active and passive drainage

structures were modeled as non-functional in accordance with the guidance in NUREG/CR-7046, Appendix B.

3.1.2.3 LIP/PMP Inputs

NUREG/CR-7046 (NRC 2011, Section 3.2) directs that the LIP parameters be defined using National Weather Service Hydrometeorological Reports #51(NOAA 1978) and #52 (NOAA 1982) (HMR-51 and HMR-52). However, VYNPS is located within a "stippled" region of HMR-51 and HMR-52 (i.e., an area where orographic effects were not studied in detail as part of the HMR studies). As per the recommendations of HMR-51/HMR-52, a detailed site-specific study was therefore conducted to establish Probable Maximum Precipitation (PMP) parameters, including LIP parameters. This analysis is detailed in AREVA Calculation No. 32-9196324-000 "Vermont Yankee Nuclear Power Station Flood Hazard Re-evaluation - Probable Maximum Precipitation" (AREVA 2013b).

The 6-hour site specific PMP, which includes the 1-hour and sub-1-hour PMPs, was used in the LIP analysis. The total PMP depth for the 6-hr PMP was calculated as 14.5 inches and the greatest 1-hr depth was 9 inches. The rainfall was distributed as per NUREG/CR-7046, Appendix B (Figure B-5). Figure 3.1-3 displays the 6-hour PMP - incremental hyetograph at five minute intervals. Figure 3.1-4 is the 6-hour PMP cumulative hyetograph showing the full LIP depth of 14.5 inches. The model conservatively utilizes no rainfall losses (initial abstraction or infiltration) during the LIP event.

3.1.2.4 FLO-2D Flood Simulations and Maximum LIP Elevation Estimates

The 6-hour PMP simulation was performed using FLO-2D. The results of the two-dimensional hydrodynamic simulation, displayed as maximum water surface elevations resulting from the LIP at the VYNPS site, are shown on Figure 3.1-5. In the immediate vicinity of VYNPS main structures (i.e., reactor building and turbine building), the maximum water surface elevations predicted by the FLO-2D model range from El. 252.0 east of the Reactor Building to approximately El. 253.0 ft. (NGVD29) west of the turbine building. The resultant high water surface elevation on the west side of the Turbine Building is likely due to onsite runoff coming from the higher elevations adjacent to the power block, the flat slopes in the area west of the Turbine Building, and the constriction to overland flow formed by the buildings themselves. Flood elevations and depths are lower moving to the north and south sides of the VYNPS power block building complex and lowest east of the buildings

Calculated flow velocity is up to 9.2 ft./sec at the discharge structure. The highest flow velocities near the main buildings (2.9 ft./sec) occur on the side roads north and south of the main buildings, which serve as discharge pathways for runoff flowing around the Reactor and Turbine buildings.

Calculated maximum depths mostly range from 0.1 to 1.0 feet on the flat area west of the Turbine Building. The calculated PMF elevation is up to 0.5 feet higher than the typical building entrance elevation of 252.5 ft. NGVD29.

3.1.3 Conclusions

The maximum LIP flood elevations at VYNPS are a result of the 6-hour PMP. In the immediate vicinity of the main buildings, the maximum water surface elevations predicted by the FLO-2D model range

from El. 252.0 NGVD29 east of the Reactor Building to approximately El. 253.0 ft. NGVD29 west of the Turbine Building.

The peak water surface elevation at VYNPS (253.0 ft. NGVD29) for the LIP analysis is about 0.5 feet higher west of the Turbine Building than the typical building entrance elevation of 252.5 ft. NGVD29.

Significant debris loading and transportation is not a safety hazard due to the relatively low velocity and depth of LIP flood waters in the vicinity of safety-related SSC at VYNPS, in addition to the lack of natural debris sources on site.

3.1.4 References

AREVA 2013. AREVA Document No. 38-9196957-000, Vermont Yankee (VY) Topographic Survey, Sanborn Map Company, Inc.

AREVA 2013a. AREVA Document No. 32-9196328-000, "Local Intense Precipitation – Generated Flood Flow and Elevations at Vermont Yankee Nuclear Power Station."

AREVA 2013b. AREVA Calculation No. 32-9196324-000 "Vermont Yankee Nuclear Power Station Flood Hazard Re-evaluation - Probable Maximum Precipitation."

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NRC 2011. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011 (ADAMS Accession No. ML11321A195).

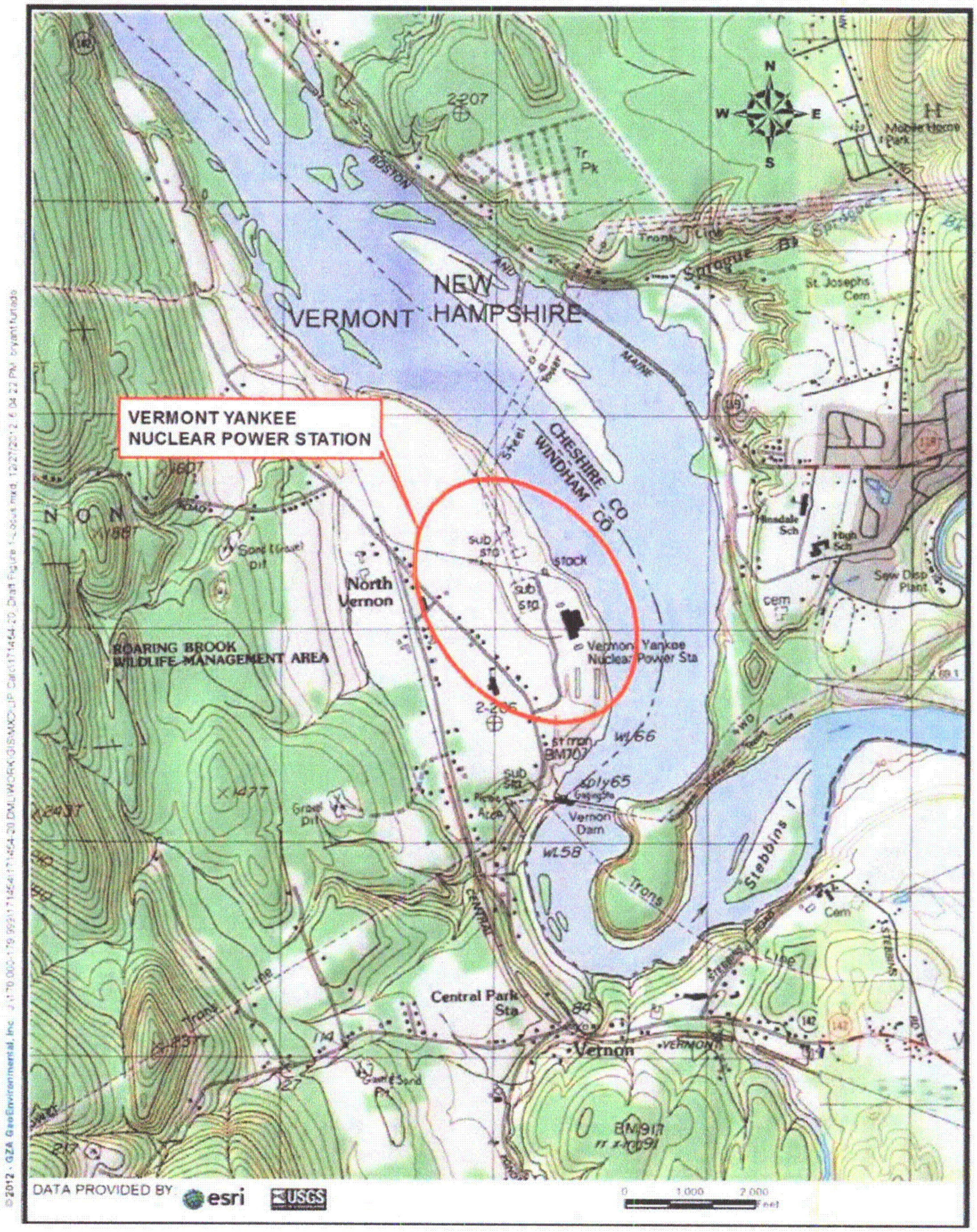

Figure 3.1-1: VYNPS Location Map

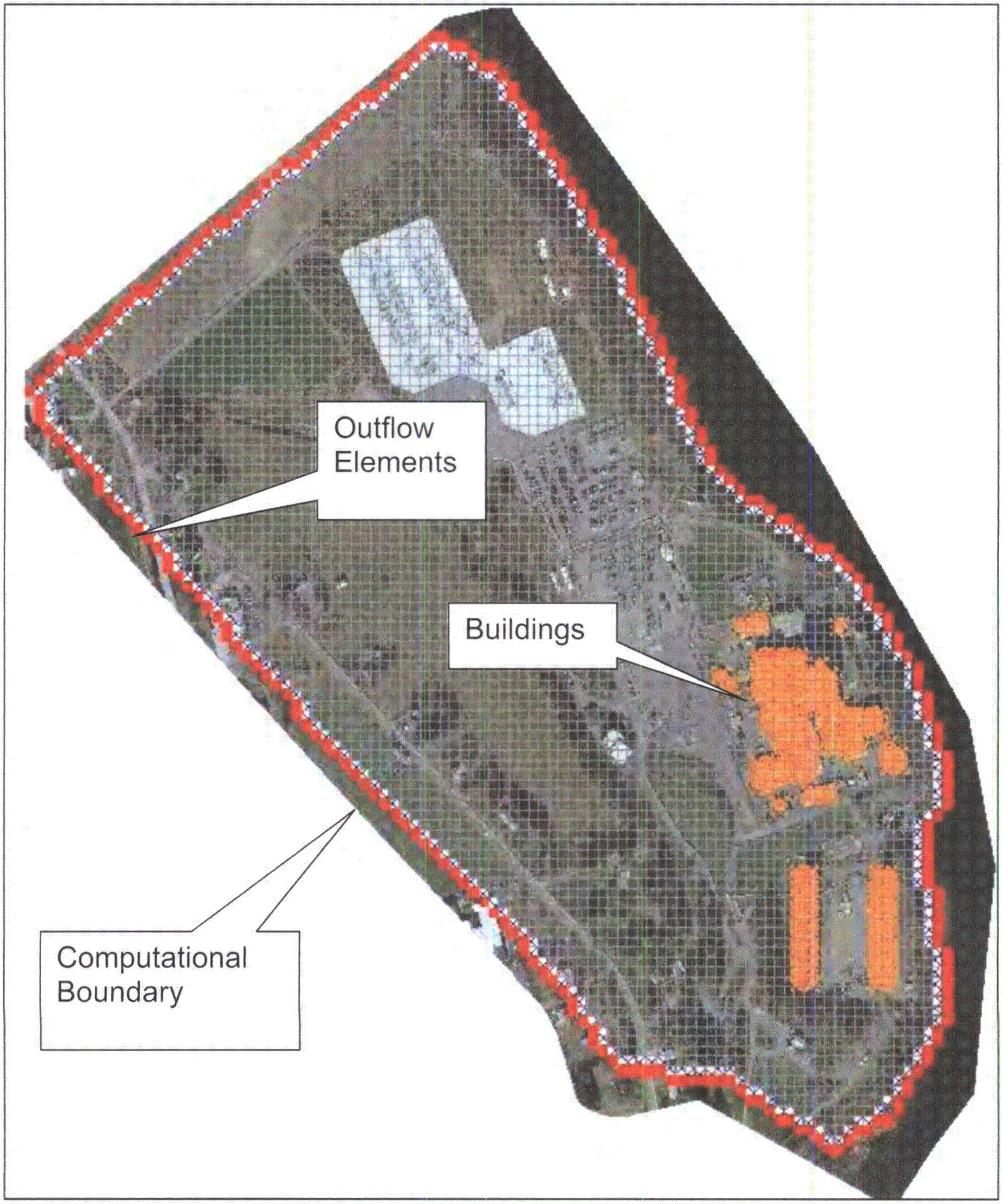

Figure 3.1-2: FLO-2D Computational Boundary, Outflow Nodes, Building Elements and Channels

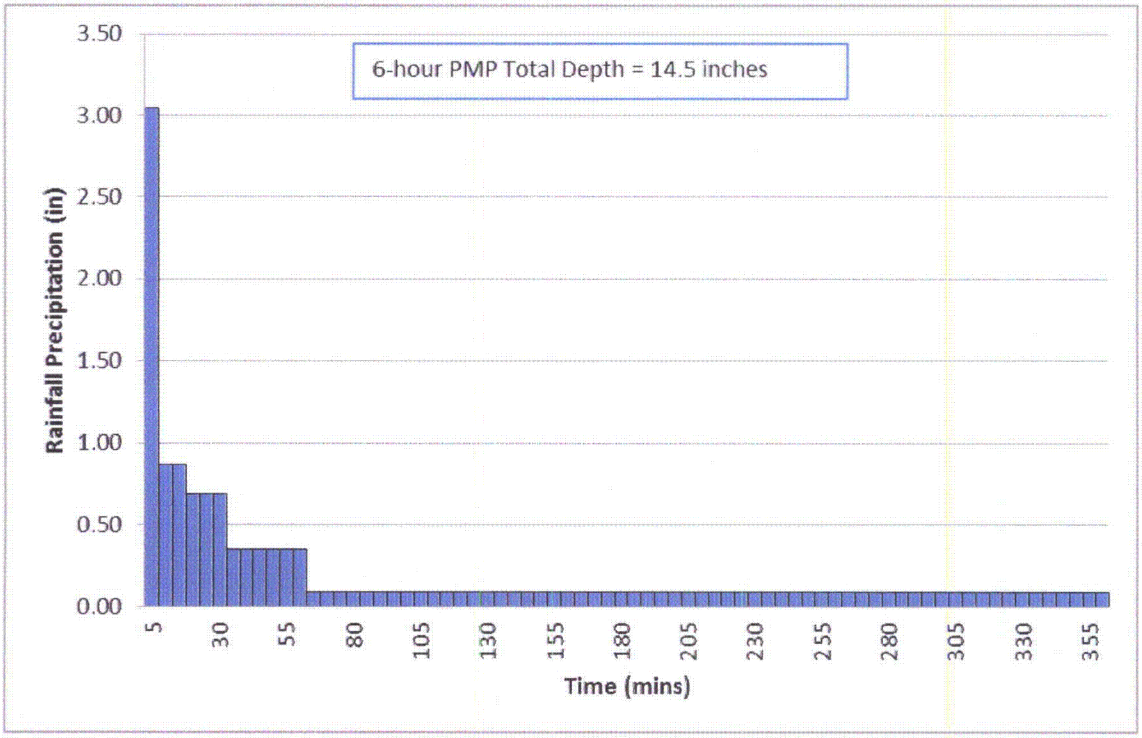

Figure 3.1-3: 6-hour PMP - Incremental Hyetograph

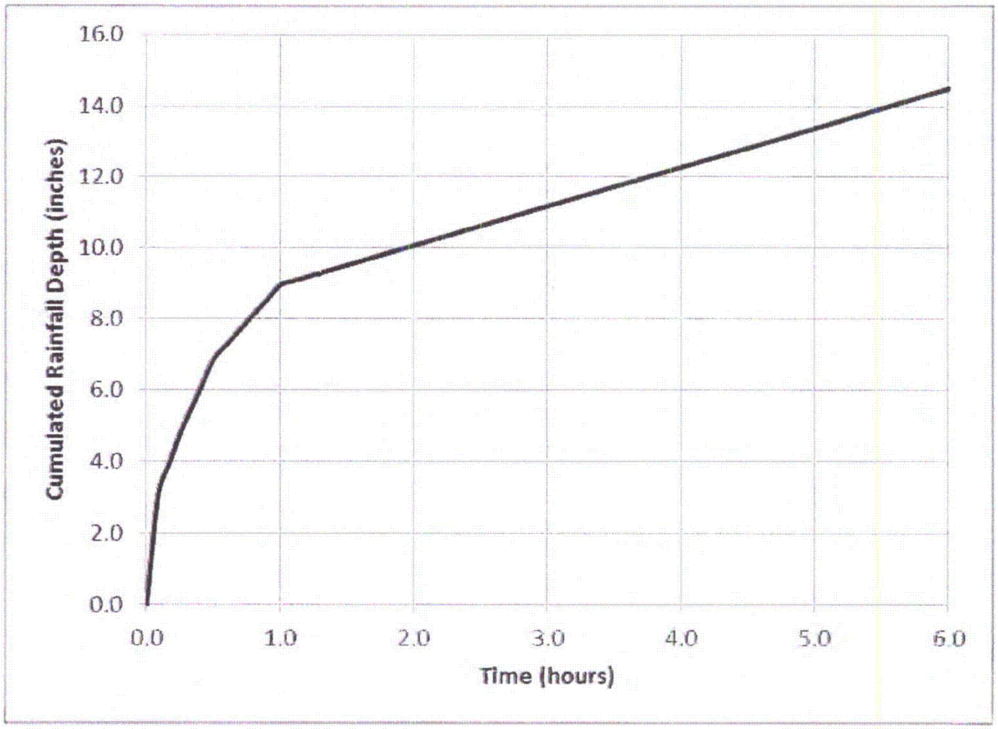

Figure 3.1-4: 6-hour PMP - Cumulative Hyetograph

Figure 3.1-5: Grid Element LIP Maximum Water Elevation (ft., NGVD 29) around VYNPS

3.2 Flooding in Rivers and Streams

This section addresses the potential for flooding at Vermont Yankee Nuclear Power Station (VYNPS) due to the probable maximum flood (PMF) on the Connecticut River. 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).

The Connecticut River forms the eastern boundary of the site. VYNPS is located on the west bank of the Connecticut River in the town of Vernon, VT, about 0.5 miles upstream of Vernon Dam. No additional significant streams are located immediately near VYNPS (Figure 3.2-1).

3.2.1 Method

3.2.1.1 Probable Maximum Flood – Connecticut River

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011) was used for the evaluation of the Connecticut River PMF and resultant water surface elevation at VYNPS.

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
   a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
   b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

With respect to PMF on the Connecticut River, the HHA approach used the following steps:

1. Delineate the Connecticut River watershed contributory to VYNPS. Delineate major sub-basins and identify major dams.
2. Perform the meteorological site-specific Probable Maximum Precipitation (PMP) Study for the watershed upstream of the VYNPS.
3. Perform HMR52 Computer Model for All Season and Cool Season Probable Maximum Precipitation (PMP).
4. Add 100-year snowpack melt rate to Cool Season PMP.

5. Develop HEC-HMS model subwatersheds, dams, tributary streams, and Connecticut River reaches.

6. Calibrate and verify HEC-HMS Model using historic rainfall and flow data.

7. Perform PMF hydrologic simulations with HEC-HMS, including required non-linearity adjustments.

8. Develop HEC-RAS unsteady flow hydraulic computer model.

9. Calibrate and verify HEC-RAS Model using historic flow and river stage data.

10. Perform PMF hydraulic simulations to generate water surface elevations in response to routed PMF hydrographs.

3.2.2 Results

3.2.2.1 Connecticut River Watershed Contributory to VYNPS

The contributory drainage area to Connecticut River at VYNPS was delineated into 21 subwatersheds (Figure 3.2-2) with a total area of 6,270 square miles. 11 significant dams were modeled based on engineering judgment, structure height, and reservoir storage capacity (Figure 3.2-3).

3.2.2.2 Meteorological Site-Specific PMP

The purpose of the study was to calculate site-specific PMP depth-area-duration values over the watershed. Site-specific PMP study-derived depth-area-duration information was applied as per the methodology of HMR-52. Use of a site-specific PMP is judged to be appropriate for the Connecticut River watershed at VYNPS based on its inclusion in a so-called "stippled region" as per HMR-51.

3.2.2.3 HMR52 Model

The PMP was calculated for the 6,270-square-mile contributory watershed using the methodology of HMR-51 and HMR-52. The 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. Total area-averaged rainfall depth for the 72-hr All Season PMP and Cool Season PMP on the contributory watershed at VYNPS were 12.3 inches and 8.8 inches, respectively.

3.2.2.4 100-Year Snowpack Melt Rate

Seasonal variation of the PMP was evaluated in combination with snowmelt. The 100-year snowpack was calculated using historical snow depth data recorded at climate stations throughout the watershed and effective forest canopy cover. The maximum 100-year snowpack calculated for the climate stations was conservatively used throughout the VYNPS watershed. The 100-year snowpack melt rate was calculated using the energy budget method as outlined in the U. S. Army Corps of Engineers EM1110-2-1406 (USACE, 1998).

The 100-year snowpack depth was 188.8 inches. An additional 5.7 inches of snow water equivalent was added to the 72-hour Cool Season PMP, resulting in a total of 14.5 inches of precipitation. The Cool Season PMP was found to be the controlling PMP because it is greater than the All Season PMP (12.3 inches).

3.2.2.5 HEC-HMS Model

The US Army Corps of Engineers HEC-HMS hydrologic model was used to model the rainfall-runoff response of the watershed to the PMP. Sub-watersheds and significant dams upstream of the VYNPS which were previously identified were incorporated into the HEC-HMS model. Reservoir elements were used in the HEC-HMS model to account for attenuation due to storage behind the dam. Reach elements were used in the HEC-HMS model to convey flows downstream and to account for travel time and flow translation (i.e., river routing) within the river.

3.2.2.6 HEC-RAS Model Calibration and Verification – Rainfall Data

USGS 15-minute streamflow data were gathered for three calibration storms and three verification storms for gaged subwatersheds. Corresponding NCDC precipitation data was gathered to create area-weighted precipitation hyetographs corresponding to each calibration and verification storm. The hydrologic model used the Snyder Unit Hydrograph method to model the process of transforming excess precipitation into direct runoff. The rainfall-runoff model was calibrated to observe USGS streamflow data by optimizing the following model input parameters: (1) Snyder basin lag time, (2) Snyder peaking coefficient, (3) initial loss, (4) constant loss rate, (5) Muskingum K, (6) Muskingum X, and (6) Muskingum number. Verified subwatershed and reach parameters are shown in Table 3.2-1 and Table 3.2-2. The input parameters for the ungaged subwatersheds were estimated based on the calibrated / verified parameters for the gaged subwatersheds.

3.2.2.7 PMF Hydrologic Simulations

Nonlinearity adjustments were made to the HEC-HMS Snyder Unit Hydrograph to include a 20% increase in peak discharge of the unit hydrograph, a 33% 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 calibrated parameters and the adjusted unit hydrograph, the PMF was simulated using the PMP. An antecedent storm, 40% of the full 72-hour all season PMP, was modeled for the all season PMP. The cool season PMF combined with snow melt from the 100-year snow pack peak discharge calculated using HEC-HMS and incorporating non-linearity adjustments was 418,900 cfs. The cool season PMF was the controlling condition (Table 3.2-3) and the hydrograph at VYNPS for this flood is shown in Figure 3.2-4.

3.2.2.8 HEC-RAS Unsteady Flow Hydraulic Model

A hydraulic computer model (HEC-RAS v4.1) was developed for an 84-mile-long reach of the Connecticut River near VYNPS. A total of 132 cross sections were used in the HEC-RAS model of which 122 cross sections were originated from a HEC-RAS model developed as part of the Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FIS) for Windham County and Windsor County in Vermont (FEMA 2007a; FEMA 2007b), and 10 cross sections were developed from

Digital Elevation Models (NED 2012; VCGI 2012; MOGI 2012) in GIS. The HEC-RAS hydraulic model extends approximately 64 miles upstream of the site and 20 miles downstream from the site.

3.2.2.9 HEC-RAS Model Calibration and Verification – River Flow Data

Model calibration is the process of selecting and refining HEC-RAS input parameters to produce a simulated profile for a given flood that shows good agreement with an accepted water surface profile for the given flood. The three largest floods of record that resulted in the three highest peak water surface elevations at the USGS Station 01154500, Connecticut River at North Walpole, NH (USGS 2012a) were used as calibration floods. Daily stream flow data was supplemented by peak discharge information for each flood. The HEC-RAS model was calibrated by uniformly adjusting the Manning's-n values of the main channel and until the resultant peak water surface elevations at USGS Station 01154500 and at USGS Station 01156500, Connecticut River at Vernon, VT (USGS 2012b) were generally within one foot of the peak observed historical data. The downstream boundary condition was modeled in HEC-RAS as the Turners Falls Dam elevation versus discharge rating curve, which was calculated based on the dam and spillway geometry description that was published as part of the Notification of Intent to File an Application to the Federal Energy Regulatory Commission (FirstLight 2012). The Vernon Dam was modeled in HEC-RAS based on spillway geometry and other information contained in GEI's 1987 Fifth Quinquennial Safety Inspection (GEI 1987).

3.2.2.10 PMF Hydraulic Simulations

The PMF hydrograph output from HEC-HMS was routed in the calibrated HEC-RAS model to establish flood elevations. The PMF flow at the site after routing was calculated to be 412,600 cfs. The peak PMF stage on the Connecticut River near VYNPS was calculated to be 249.7 ft., which is 2.8 feet below the CLB elevation of 252.5 feet (Entergy 2012). See Table 3.2-4.

3.2.3 Conclusions

At VYNPS, impacts to the site from flooding on the Connecticut River resulting from the PMF are judged to not affect site safety-related SSCs, beyond those already considered for the CLB PMF, for the following reasons:

- The probable maximum flood on the Connecticut River near VYNPS is conservatively estimated at 418,900 cfs (412,600cfs after hydraulic routing). Historical records do not indicate flooding in excess of this PMF flow.

- The peak PMF water surface elevation at Vermont Yankee Nuclear Power Station is 249.7 ft., which is 2.8 ft. below the CLB elevation of 252.5 ft. (Entergy 2012).

Based on the re-evaluated peak PMF elevation on the Connecticut River at VYNPS, the peak PMF water surface elevation from the Connecticut River flood is below the CLB elevation and would not affect safety-related structures, systems, or components at VYNPS beyond those already considered for the CLB PMF.

3.2.4 References

Entergy 2012. "Updated Final Safety Analysis Report, Vermont Yankee Nuclear Power Station, Revision 25", May 2012.

FEMA 2007a. "Flood Insurance Study – Windham County, Vermont (All Jurisdictions)" Federal Emergency management Agency (FEMA), September, 2007.

FEMA 2007b. "Flood Insurance Study – Windsor County, Vermont (All Jurisdictions)" Federal Emergency management Agency (FEMA), September 28, 2007.

FirstLight 2012. FirstLight Power Resources, "Turners Falls Hydroelectric Project, FERC Project No. 1889, Notification of Intent to File an Application for New License and Request for Designation as Non-Federal Representative," October 30, 2012.

GEI 1987. GEI Consultants, Inc., Fifth Quinquennial Safety Inspection – Vernon Project, Published by New England Power Company, October 1987. See Electronic References.

MOGI 2012. MA Digital Elevation Model (DEM), MassGis: Massachusetts Office of Geographic Information – 5-meter resolution (http://www.mass.gov/anf/research-and-tech/it-serv-and-support/application-serv/office-of-geographic-information-massgis/).

NED 2012. NH Digital Elevation Model (DEM), National Elevation Dataset – 10-meter resolution, (http://ned.usgs.gov).

NRC 2011. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011 (ADAMS Accession No. ML11321A195).

TransCanada 2007. TransCanada Hydro Northeast Inc. Reservoir and Minimum Flow Operations and Monitoring Plan, Vernon Hydroelectric Project FERC Project No. 1904, December 2007.

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

USGS 2012a. U.S. Geological Survey (USGS) Stream Gage 01154500 Connecticut River at North Walpole, NH (http://waterdata.usgs.gov/nwis/nwisman/?site_no=01154500&agency_cd=USGS).

USGS 2012b. U.S. Geological Survey (USGS) Stream Gage 01156500 Connecticut River at Vernon, VT (http://waterdata.usgs.gov/nwis/inventory?agency_code=USGS&site_no=01156500).

VCGI 2012. VT Digital Elevation Model (DEM), VCGI: Vermont Center of Geographic Information – 10-meter resolution, (http://www.vcgi.org/).

Table 3.2-1: Verified Subwatershed Parameters

Subwatershed	Standard Lag (hr)	Peaking Coefficient, Cp	Constant Loss Rate (in/hr)	Initial Loss Rate (in)
First Connecticut	14.0	0.70	0.11	0
Indian Stream	22.0	0.55	0.04	0
North Stratford	16.0	0.40	0.05	0
Upper Ammonoosuc	22.0	0.43	0.05	0
Ammonoosuc	5.5	0.53	0.10	0
Passumpsic	12.0	0.40	0.17	0
Moore	4.6	0.53	0.06	0
Comerford	3.3	0.53	0.08	0
Wells	10.1	0.50	0.12	0
Union Village	7.0	0.70	0.04	0
White	12.0	0.70	0.02	0
West Lebanon	26.0	0.60	0.05	0
North Hartland	18.0	0.40	0.16	0
North Springfield	9.4	0.70	0.10	0
Sugar	15.5	0.40	0.16	0
Williams	6.6	0.70	0.15	0
Saxtons	9.0	0.40	0.08	0
North Walpole	30.0	0.40	0.10	0
Ball Mountain	14.0	0.40	0.16	0
Townshend	7.9	0.40	0.11	0
Vermont-Yankee	21.2	0.40	0.20	0

Table 3.2-2: Verified Muskingum Routing Parameters

Reach	Travel time, K (hr)	Weight, X	Subwatersheds, N
Reach 1 - Indian Stream to North Stratford	5	0.100	5
Reach 2 – North Stratford to Moore Dam	20	0.025	20
Reach 3 – Comerford Dam to Wilder Dam	24	0.021	24
Reach 4 – West Lebanon to Sugar	6	0.083	6
Reach 5 – Sugar Junction to Bellows Falls	6	0.083	6

Table 3.2-3: Adjusted PMF Peak Discharges at VYNPS

All Season Peak Discharge	Cool Season Peak Discharge
389,300	418,900

Table 3.2-4: Routed PMF Results at VYNPS

Routed Peak Discharge	Peak Water Surface Elevation
412,600	249.7

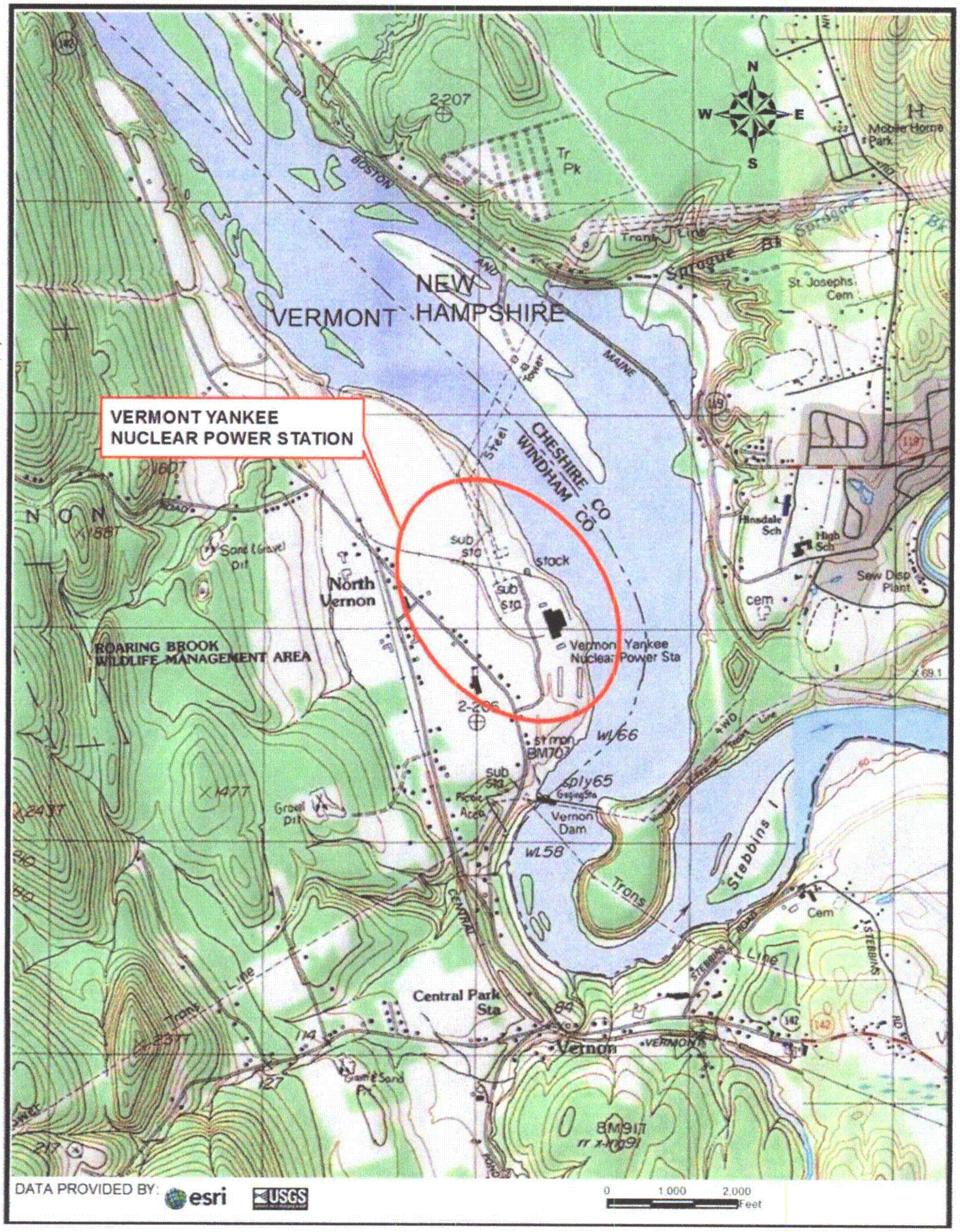

Figure 3.2-1: VYNPS Location Map

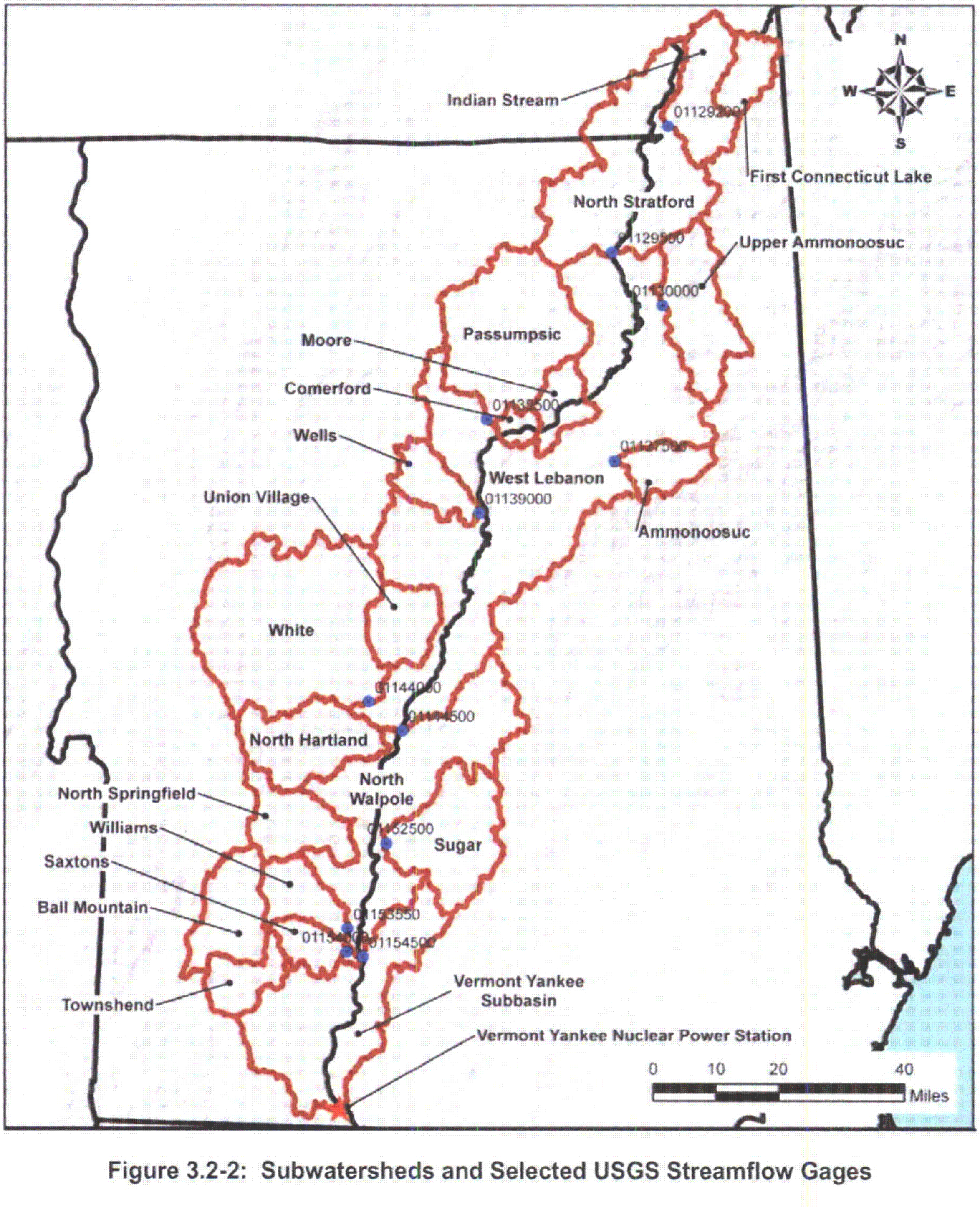

Figure 3.2-2: Subwatersheds and Selected USGS Streamflow Gages

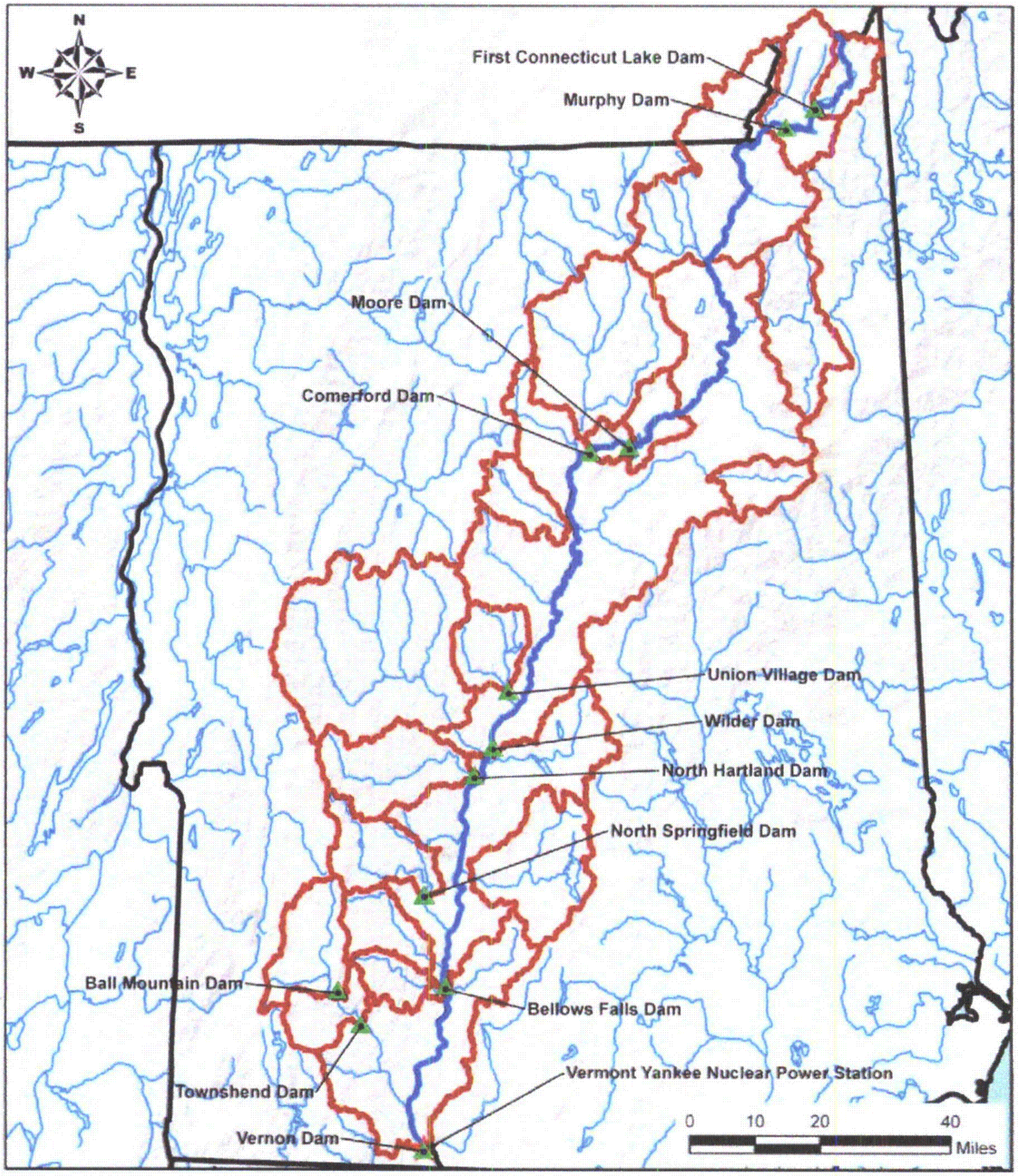

Figure 3.2-3: Incorporated Dams

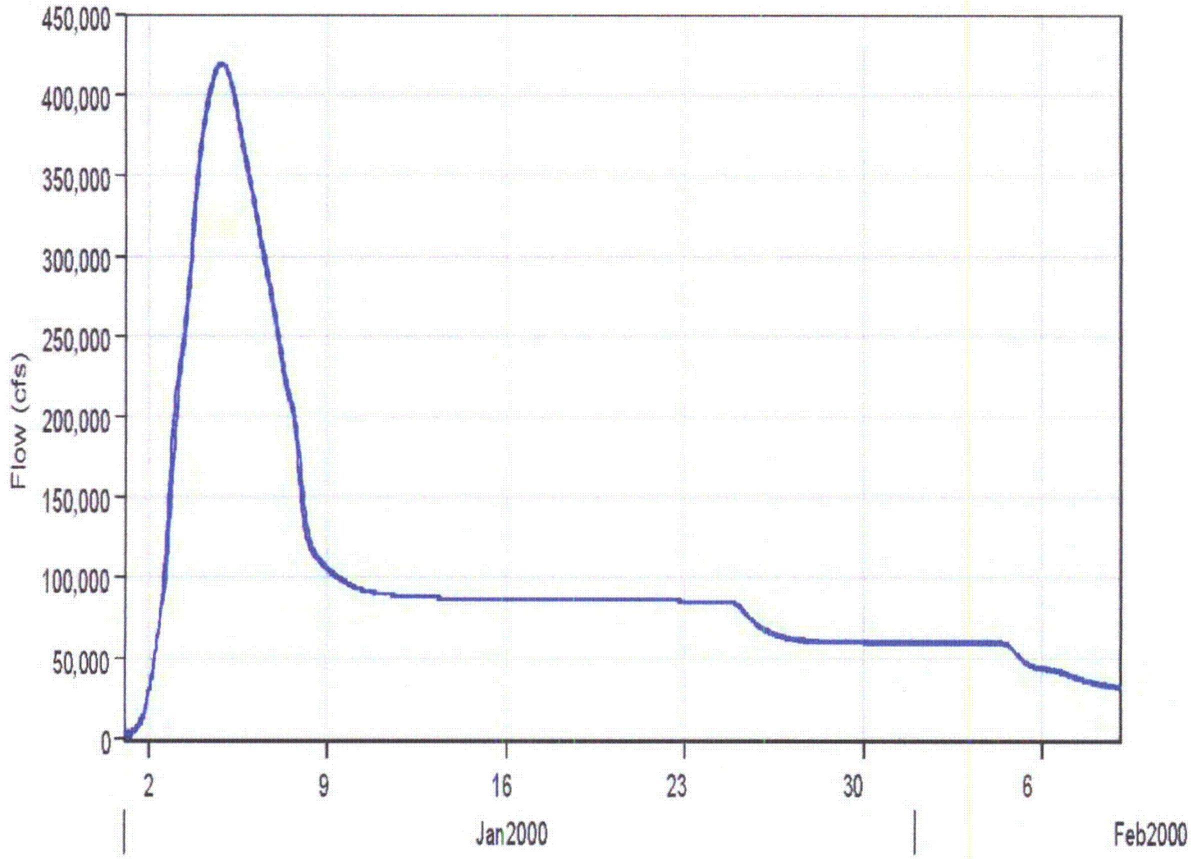

Figure 3.2-4: Cool Season PMF Hydrograph Adjusted for Non-Linearity at VYNPS

3.3 Dam Failures

Dam breaches and failures may cause flood waves that impact the PMF level of the receiving water body. Dam breaches and failures of on-site water-storage or water-control structures (such as onsite cooling or auxiliary water reservoirs) can also impact site safety (NRC 2011, Section 3.4).

There are no significant on-site water-storage or water-control structures that could impact site safety if breached (VYNPS 2012, Section 2.4).

3.3.1 Method

3.3.1.1 Dam Failures - Connecticut River

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011) was used for the evaluation of the effects of upstream dam breaches and failures on the maximum water surface elevation at VYNPS (AREVA 2013c).

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
4. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

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

1. Failure of individual dams (i.e., failure of dams on separate tributaries without domino-like failures) upstream of site; and
2. Cascading or domino-like failures of dams upstream of the site.

The dam failure evaluation was performed under the following events:

a. Hydrologic (i.e., dam failures due to the PMF which may overtop some dams) and

b. Seismically-induced events (i.e., operating basis earthquake dam failure scenario coincident with ½ PMF as per NUREG/CR-7046, Appendix H.2, NRC 2011).

With respect to dam failures on the Connecticut River, the HHA used the following steps:

1. Identify and screen upstream dams with significant height (50 feet or higher) and storage (25,000 acre feet or greater maximum storage) on the Connecticut River and its tributaries within the contributory watershed upstream of VYNPS.
2. Develop a conservative, representative upstream dam failure scenario and estimate the peak breach outflow using HEC-HMS.
3. Perform hydraulic simulation for each event to calculate the peak water surface elevation resulting from the combined dam breach and appropriate river flow (either PMF or ½ PMF) at VYNPS using the VYNPS HEC-RAS model developed under AREVA Document No. 32-9196326-000 "Probable Maximum Flood on Connecticut River – Hydraulics" (AREVA 2012b).

3.3.2 Results

The Connecticut River forms the eastern boundary of the site. VYNPS is located on the west bank of the Connecticut River in the town of Vernon, VT, about 1/2 mile upstream of Vernon Dam. See Figure 3.3-1.

3.3.2.1 Upstream Dams Screening

The total number of dams in the Connecticut River watershed exceeds 1,600 (NHDES 2012 and VCGI 2012). Using the map layer Major Dams of the United States (NA 2012) a total of 30 major dams were identified within the watershed. From this list of dams, 11 upstream dams (see Figure 3.3-2) were selected as follows:

1. Dams located within the VYNPS watershed with a calculated peak breach flow greater than 1,000,000 cfs.
2. Dams located on the Connecticut River upstream of VYNPS with a calculated peak breach flow greater than 100,000 cfs.
3. Dams located within 25 river miles of VYNPS.

3.3.2.2 Upstream Dam Failure Scenario and Peak Breach Flow Estimate

Separate HEC-HMS models were used to develop a conservative, representative dam breach model for each selected dam and to develop a VYNPS Watershed model for the Connecticut River. Data from the dam breach models was input into the overall watershed model to analyze flows caused by the Hydrologic and Seismically-induced upstream dam failure events. Conservative dam breach parameters were used to estimate peak breach flows and are shown in Table 3.3-1 and Table 3.3-2.

Domino failure was deemed the bounding failure scenario for both hydrologic and seismic events due to the proximity and correlation of modeled dams to one another within the watershed. This condition indicates that domino failure with concurrent increased outflow will exceed the individual failure scenario. The controlling event, cold season probable maximum precipitation on 100-year snowpack for the full VYNPS watershed (AREVA 2012a), was used as the hydrologic event to estimate peak

breach outflow from a representative dam failure scenario. The full PMF was used for the hydrologic event and the ½ PMF was used for the seismic event.

Hydrologic (PMF) Event - Pool elevations did not reach the top of dam during the VYNPS watershed PMF for nine out of the eleven modeled dams; therefore, dam failure of these structures was not initiated during the PMF. Two dams on the Connecticut River, Wilder and Bellows Falls Dam, overtopped during the PMF. The peak dam breach outflow for these two dams, which are in series on the Connecticut River, was 337,500 cfs and 459,000 cfs, respectively. See Table 3.3-1.

Seismically-Induced Event - The seismic dam breach event used the ½ PMF, and was similar to the hydrologic event except all modeled dams were failed regardless if they were overtopped during the ½ PMF. Each dam was failed when it reached the maximum water surface elevation resulting from the ½ PMF inflow or upstream dam failure. The dam breach outflows for each dam are shown in Table 3.3-2.

3.3.2.3 Hydraulic Simulation to Calculate Peak Water Surface Elevation

Hydraulic modeling was performed for the hydrologic and seismic events to calculate the water surface elevation at VYNPS resulting from the two dam failure events. The calculated peak water surface elevation after dam failure from the hydrologic event (PMF) in the Connecticut River at Vermont Yankee Nuclear Power Station, based on a maximum discharge after hydraulic routing within HEC-RAS of 461,600 cfs, is 252.0 feet NGVD29, which is 0.5 feet below the minimum SSC entrance elevation of 252.5 feet (UFSAR 2012). The stage hydrograph for the Connecticut River at the VYNPS for this event is shown in Figure 3.3-3. The calculated peak water surface elevation from seismically induced dam failures in Connecticut River at Vermont Yankee Nuclear Power Station, based on a maximum discharge after hydraulic routing within HEC-RAS of 396,400 cfs, is 247.9 feet NGVD 1929, which is 4.6 feet below the minimum SSC entrance elevation of 252.5 feet. The controlling dam failure scenario is the hydrologic event (PMF). The stage hydrograph for the Connecticut River at the VYNPS for this event is shown in Figure 3.3-4.

3.3.3 Conclusions

The dam failure results from the hydrologic event demonstrate that it would be unlikely for the failure during the PMF (resulting from the PMP centered over the VYNPS watershed) of additional dams not included in the dam breach analysis to affect the maximum flood water surface elevation at VYNPS. The storage volume impounded by the selected major dams represents more than 86 percent of the total storage volume in all 30 major dams upstream of the site.

The modeling results from the failure of the selected Major Dams during the seismic event demonstrated that due to the natural timing differences (from varying travel times of the dam breach and natural flood wave from disparate upstream sources), failure of additional dams not incorporated into the dam breach analysis would be unlikely to affect the maximum flood water surface elevation at VYNPS. These results also demonstrate that "sunny day" dam failures (i.e. failures under non-flood conditions) will result in peak water surface elevations that will be bounded by other flood mechanisms.

The results indicate that the peak water surface elevation resulting from failure of the dams on Connecticut River is 252.0 feet NGVD29, which is below the CLB PMF elevation and would not affect safety-related SSCs at VYNPS beyond those already considered for that event.

3.3.4 References

AREVA 2013a. AREVA Document No. 32-9196325-000, "Probable Maximum Flood Connecticut River – Hydrology."

AREVA 2013b. AREVA Document No. 32-9196326-000, "Probable Maximum Flood Connecticut River – Hydraulics."

AREVA 2013c. AREVA Document No. 32-9196327-000, "VYNPS Dam Failures."

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

NA 2012. Major Dams of the United States map layer by the National Atlas of the United States. (http://nationalatlas.gov/mld/dams00x.html). Accessed 23 October 2012.

NHDES 2012. New Hampshire Department of Environmental Services - Office of Dam Safety, State of New Hampshire Dam's GIS shapefile. Accessed 2 November 2012.

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UFSAR 2012. "Updated Final Safety Analysis Report, Vermont Yankee Nuclear Power Station, Revision 25", Entergy, 2012.

VCGI 2012. Vermont Center for Geographic Information (http://www.vcgi.org/dataware/), State of Vermont Dam's GIS shapefile. Accessed 23 October 2012.

Table 3.3-1: PMF Breach Summary

Dam	Top of Dam (NGVD)	Structure Failed	Height of Dam (ft.)	Length of Dam (ft.)	Trigger Elevation* (NGVD, ft.)	Breach Bottom El. (NGVD, ft.)	Average Breach Width** (ft.)	Breach Bottom Width (ft.)	Breach Slope** (H:V)	Development Time** (hrs)	Breach Outflow (cfs)
Wilder	393	Concrete	39	2100	399.5	354	39	39	0	0.1	337,500
Bellows Falls	394	Concrete	48	670	311.5	346	48	48	0	0.1	459,000

* Shaded cell indicates trigger elevation available from initial PMF Run.
** Dam breach parameters based on published guidance (FERC, 1993)

Table 3.3-2: Seismic Breach Summary

Dam	Top of Dam (NGVD)	Structure Failed	Height of Dam (ft.)	Length of Dam (ft.)	Trigger Elevation* (NGVD, ft.)	Breach Bottom El. (NGVD, ft.)	Average Breach Width** (ft.)	Breach Bottom Width (ft.)	Breach Slope** (H:V)	Development Time** (hrs)	Breach Outflow (cfs)
First Connecticut Lake	1647	Earth	56	1117	1636.2	1591	280	224	1	0.5	235,300
Murphy	1400	Earth	106	2200	1401.7	1294	530	424	1	0.5	1,541,200
Moore	820	Earth	149	2920	821.9	671	745	596	1	0.5	3,703,400
Comerford	661	Concrete	180	2253	703.0	481	180	180	0	0.1	2,662,300
Union Village	584	Earth	164	1100	539.7	420	820	656	1	0.5	1,029,500
Wilder	393	Concrete	39	2100	400.2	354	39	39	0	0.1	355,400
North Hartland	572	Earth	182	1640	441.4	390	910	728	1	0.5	132,000
North Springfield	570	Earth	118	2940	504.7	452	590	472	1	0.5	444,300
Bellows Falls	301	Concrete	48	670	308.5	253	335	335	0	0.1	373,100
Ball Mountain	1052	Earth	247	915	913.2	805	668	421	1	0.5	403,600
Townshend	583	Earth	126	1700	508.3	457	630	504	1	0.5	372,700

* Shaded cell indicates trigger elevation available from initial ½ PMF Run. Due to numerical instability within HEC-HMS, 0.1 feet was subtracted in several cases.
** Dam breach parameters based on published guidance (FERC, 1993).

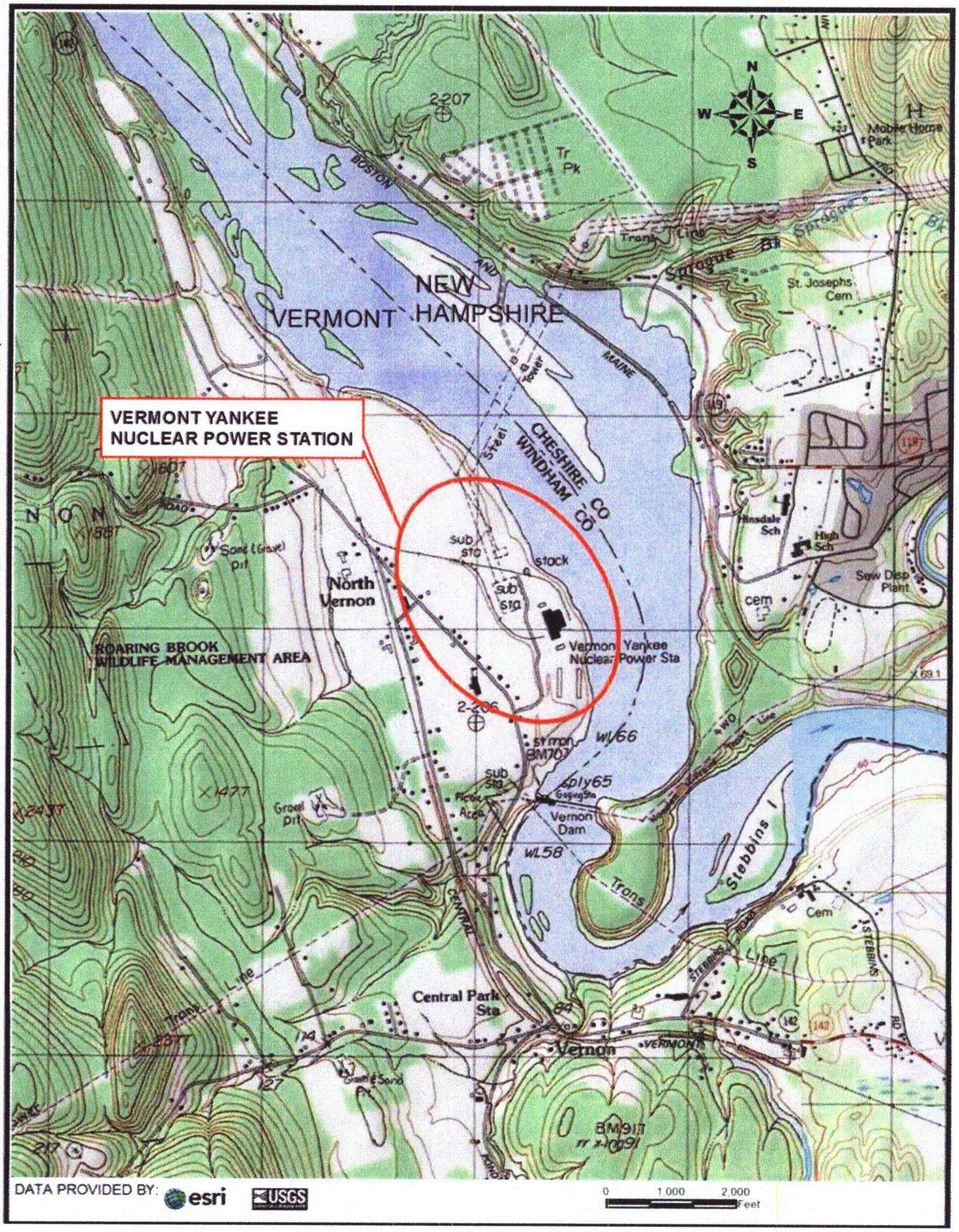

Figure 3.3-1: VYNPS Location Map

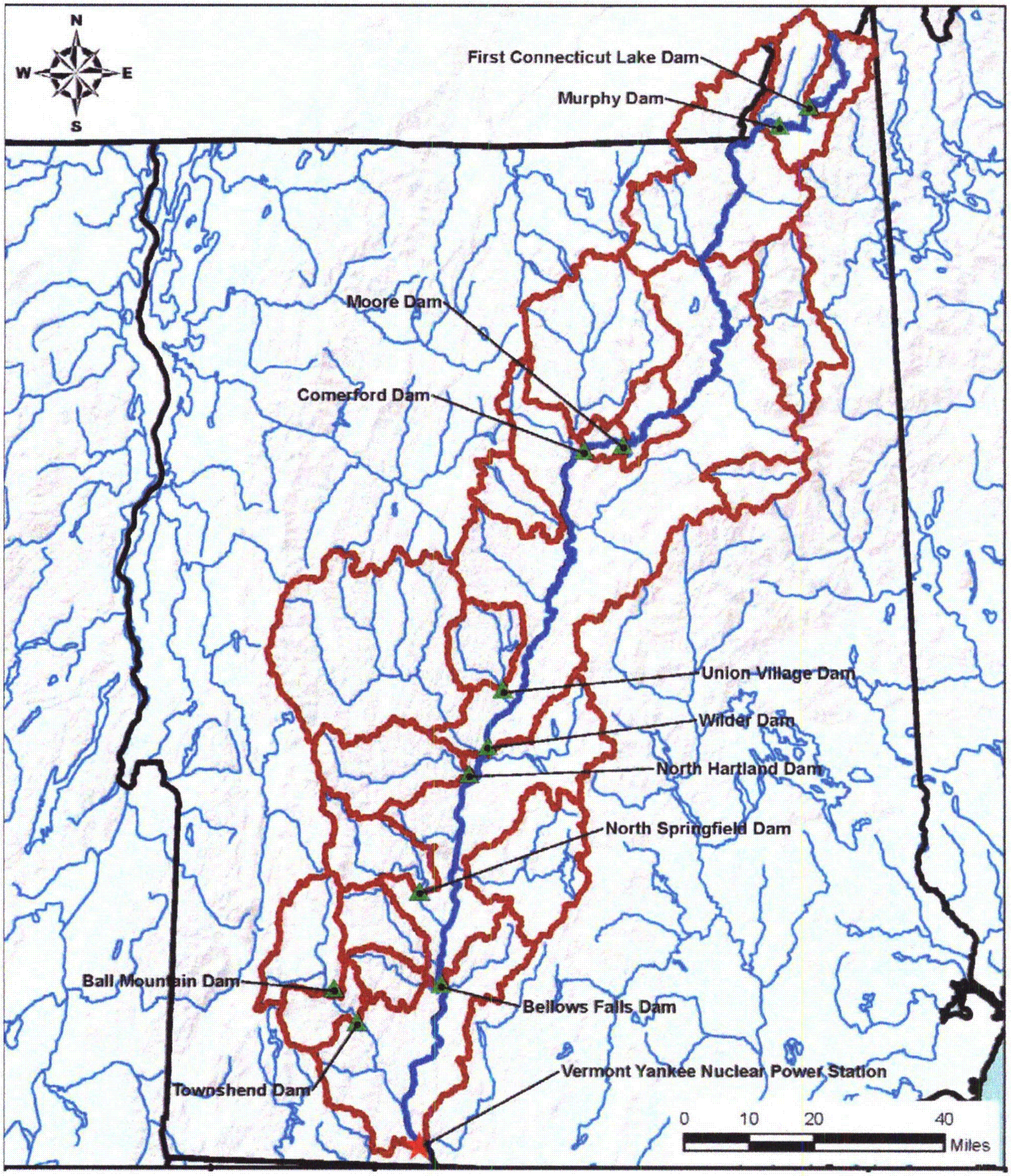

Figure 3.3-2: Selected Dams

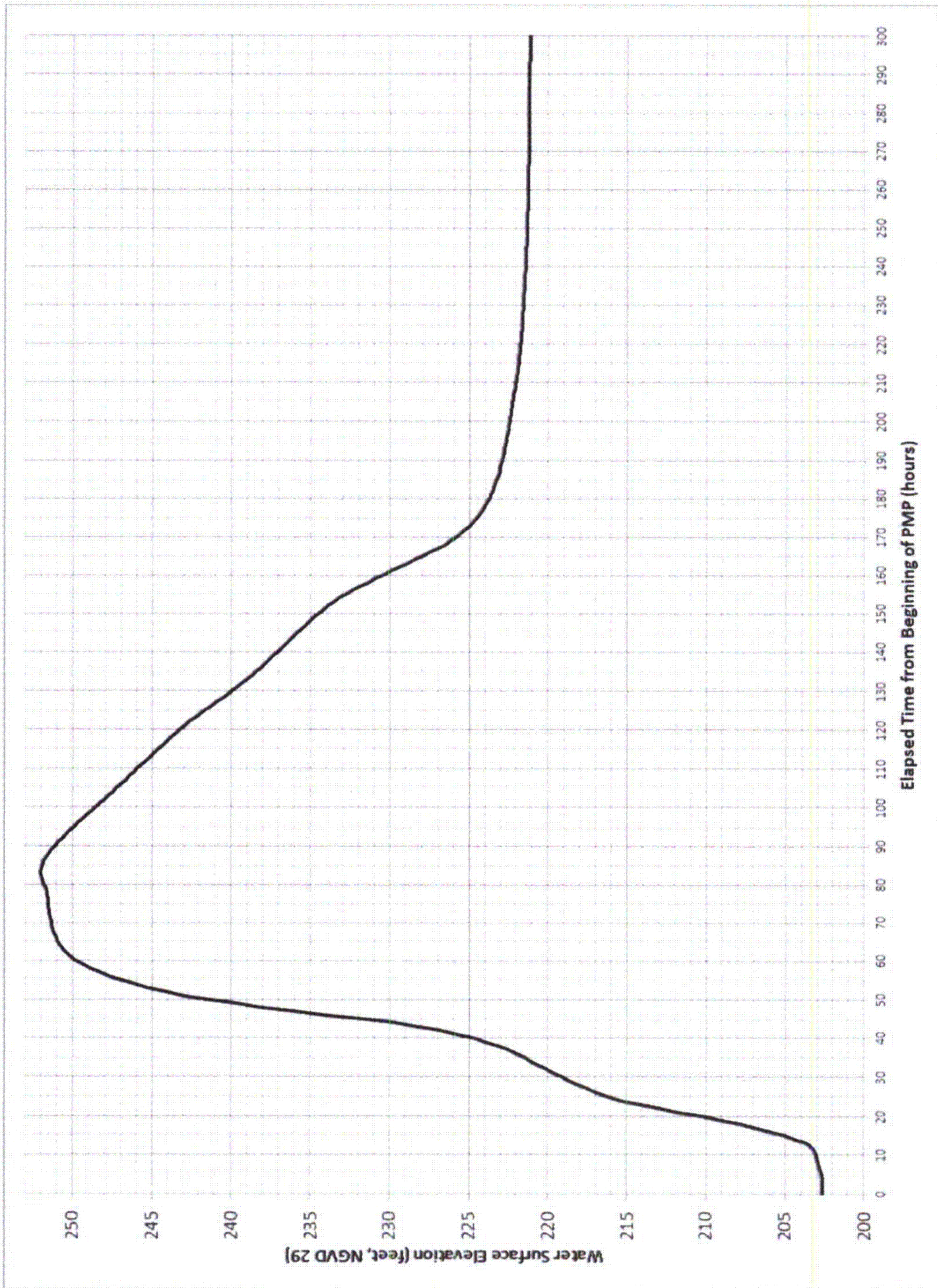

Figure 3.3-3: Hydrologic (PMF) Scenario - Stage Hydrograph at VYNPS

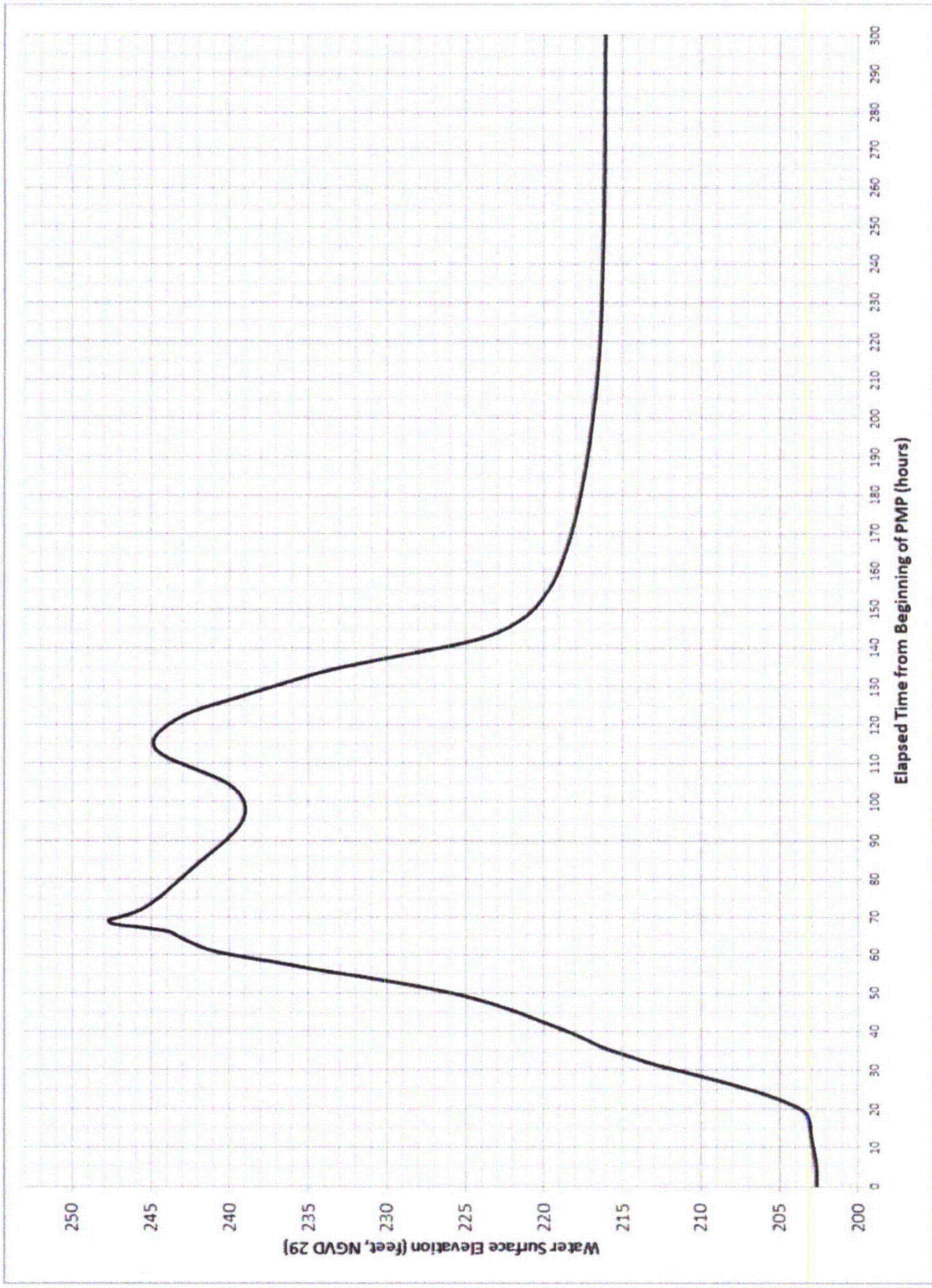

Figure 3.3-4: Seismic Scenario - Stage Hydrograph at VYNPS

3.4 Storm Surge

Storm surges are defined as rises in offshore water elevations caused principally by the sheer force of winds acting on the water surfaces, typically associated with hurricanes (NRC 2011, Section 3.5).

3.4.1 Conclusion

The potential flooding hazard from storm surge at VYNPS is judged to be negligible for the following reasons.

VYNPS is on the hydrologically controlled Connecticut River that includes downstream dams. As such, regional storm surge swells propagating from Atlantic Ocean coastal waters upstream to VYNPS via the river will not occur.

In addition, the hydrometeorlogical conditions locally limit the development of storm surges. The Connecticut River in the VYNPS area is both narrow (less than 1 mile) and meandering, which reduces the broad and extensive water surface area needed to generate a storm surge. Also, the generation of sustained, hurricane-type winds (including from tropical depressions and storms) at VYNPS is minimized due to its inland location, which is more than 100 miles from the nearest Atlantic Ocean coastline.

3.4.2 References

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

3.5 Seiche

A seiche is an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause. Once started, the oscillation may continue for several cycles; however, over time it gradually decays because of friction (NRC 2011, Section 3.6).

3.5.1 Conclusion

The potential flooding hazard from a seiche at VYNPS is judged to be negligible because of the site's riverine setting.

The Connecticut River in the VYNPS area is narrow (less than 1 mile), shallow (40 feet or less), and meandering, which constrains and limits the geometry needed to develop a seiche and its oscillation propagation. The river geometry also limits the height of any seiche oscillations and causes rapid attenuation of any seiche oscillations.

Thus, given a seiche, there would be little, if any, effect on the VYNPS site.

3.5.2 References

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

3.6 Tsunami

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources (NRC 2009, Section 1.1). As an inland site, VYNPS is not susceptible to oceanic tsunamis (NRC 2009, Section 2.1). Instead, there is the potential of tsunami-like waves in the Connecticut River.

3.6.1 Methodology

The VYNPS tsunami evaluation followed the hierarchical hazard assessment (HHA) approach described in NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America (NRC 2009).

With respect to tsunamis, the progressive HHA is considered as a series of three tests:

1. Is the site region subject to tsunamis?

2. Is the plant site affected by tsunamis?

3. What are the hazards posed to safety of the plant by tsunamis?

At VYNPS, however, only the first test needed to be considered. The second and third tests were unnecessary based on the results of the first test.

The first test was answered by performing a regional survey and assessment of potential tsunamigenic sources. The regional survey was in four parts and included the relevant mechanisms that generate tsunamis. The first part was to review the Global Historical Tsunami Database, maintained by the National Oceanic Atmospheric Administration's National Geophysical Data Center (NGDC), to determine the history of tsunamis. The second, third, and fourth parts of the regional survey included an assessment of the mechanisms likely to cause a tsunami.

3.6.2 Results

3.6.2.1 Regional Survey

Tsunamis are generated by rapid, large-scale disturbance of a body of water. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by submarine and subaerial landslides. (NRC 2009, Section 1.3) Meteorite impacts, volcanoes, and ice falls can also generate tsunamis, but were excluded from the regional survey because meteorite impacts and volcanoes are very rare events and ice falls are generally associated with glacial ice processes.

3.6.2.1.1 NGDC Database Review

The NGDC tsunami-source-event database is global in extent with information dating from 2000 B.C. to the present. As an inland site, the VYNPS regional survey considered tsunami-like waves in the area along the Connecticut River, extending from 41.5° to 45° N Latitude and 71° to 75° W Longitude.

No events have been recorded in the NGDC database for the region (NOAA 2012).

3.6.2.1.2 Earthquakes

To generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis (NRC 2009, Section 1.3.1).

Based on the geological and tectonic history information presented in the VYNPS Update Final Safety Analysis Report (UFSAR), the region is relatively quiescent with low magnitude seismic events (VYNPS 2012, Section 2.5.4). The largest New England earthquakes occurred some 85 to 90 miles from the site. If observed at the site, the earthquakes would have been as Modified Mercalli Intensity IV or V (VYNPS 2012, Section 2.5.4). By comparison, this intensity range is less than 5.0 magnitude (USGS 2012b).

As a result, the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent from the region.

Seismic activity outside the region can also produce seismic seiches (USGS 2012a). Seismic waves from the Alaska earthquake of 1964, for example, caused water bodies to oscillate at many places in North America. Seiches were recorded at hundreds of surface-water gaging stations. The seismic seiche distribution did not have an obvious dependence on distance or azimuth from the epicenter. Instead, the distribution had a regional pattern, which reflected the influence of major geologic features. The southeastern part of the United States had the greatest density of seiches, while areas west of the Rockies, the Middle Atlantic States, and New England experienced few or no seiches. A favorable environment for seismic seiche generation includes thrust faults and locations controlled by structural uplifts and basins (USGS 2012a). The VYNPS region, however, lacks such features. Seiche flood hazard for the site is discussed in Section 2.5.

3.6.2.1.3 Landslides

There are two broad categories of landslides: (1) subaerial that are initiated above the water and impact the water body during their progression or fall into the water body, and (2) subaqueous that are initiated and progress beneath the surface of the water body.

In addition, landslide-generated tsunami-like waves have a very strong directivity in the direction of mass movement. Therefore, the outgoing wave from the landslide source propagates in the direction of the slide. The most common landslide mechanism is an earthquake (NRC 2009, Section 1.3.2).

Subaerial Landslide – Area Topography

The geographical areas where subaerial landslides occur are generally limited to areas of steep shoreline topography (NRC 2009, Section 1.3.2).

The USGS classifies the Connecticut River region in Vermont as having a high susceptibility to landslides, but with a low incidence rate (USGS 2012c). Locally, the potential for a subaerial landslide, however, is conspicuously less because the land area along the VYNPS east and west shoreline is either flat or gently rolling (Figure 3.6-1).

Upstream from VYNPS, there are several areas with steep shoreline gradients that have a greater potential to produce a subaerial landslide. Nonetheless, the direction of a landslide in these areas and resultant tsunami-like wave, if it occurred, would be toward the opposite river shoreline and upstream from VYNPS. Any redirection of such a tsunami-like wave downstream would be attenuated prior to reaching VYNPS due to the river's meandering feature and numerous small, inter-river islands and land spits.

Thus, given a subaerial landside, there would be little, if any, effect to the VYNPS site.

Subaqueous Landslide –Connecticut River Bathymetry

The outgoing wave from a subaqueous landslide source propagates in the direction of the slide with its amplitude affected by the terminal velocity of the movement, which in turn is a function of the repose angle, i.e., the slope angle (NRC 2009, Section 1.3.2).

The Connecticut River bathymetry in the VYNPS area is non-uniform (FEMA 2007). There are limited areas with steep bathymetric gradients that have the potential to produce a subaqueous landslide. The maximum slope of these steeper gradient areas is less than 8 degrees. Thus, given a landslide, its velocity would be limited due to the low-angle slope.

As a result, these steeper gradient areas are judged unlikely to generate a subaqueous landslide and resultant tsunami-like wave that could affect the VYNPS site.

3.6.3 Conclusions

As an inland site, the VYNPS site is not subject to oceanic tsunamis. Based on the NGDC tsunami-source-event database regional survey screening results:

- No tsunami-like waves have been recorded in the region.
- Tsunami-like waves generated from
  - an earthquake are limited because the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent from the region;
  - a subaerial landslide is limited because of the flat or gently rolling topography along VYNPS' east and west shorelines, or given a subaerial landside upstream, there would be little, if

any, effect due to attenuation prior to reaching VYNPS because of the river's meandering features and many small, inter-river islands and land spits; and

- a subaqueous landslide is limited because of the low-angle slope of the bathymetric gradients.

As a result, the flooding hazard potential at the VYNPS site from tsunami-live waves is judged to be negligible.

3.6.4 References

FEMA 2007. Federal Emergency Management Agency, Flood Insurance Study for Windham County (all Jurisdictions), Vermont,-September 28, 2007.

NOAA 2012. National Oceanic Atmospheric Administration, National Geophysical Data Center, Tsunami Database Website: http://www.ngdc.noaa.gov/hazard/tsu.shtm; accessed December 4, 2012.

NRC 2009. NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America, U.S. NRC, March 2009 (ADAMS Accession No. ML091590193).

NRC 2011. NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. NRC, November 2011 (ADAMS Accession No. ML11321A195).

USGS 2012a. U.S. Geological Survey, Seismic Seiches, Website: http://earthquake.usgs.gov/learn/topics/seiche.php, accessed August 23, 2012.

USGS 2012b. U.S. Geological Survey, Magnitude/Intensity Comparison, Website: http://earthquake.usgs.gov/learn/topics/mag_vs_int.php, accessed November 27, 2012.

USGS 2012c. Landslide Overview Map of the Conterminous United States, Website: http://landslides.usgs.gov/learning/nationalmap/, accessed December 5, 2012.

VYNPS 2012. Vermont Yankee Nuclear Power Station, Undated Final Analysis Report (UFSAR), Rev. 25, May 2012.

WTBM 2012. World Terrain Base Map, Website: http://www.arcgis.com/home/item.html?id=c61ad8ab017d49e1a82f580ee1298931; accessed December 4, 2012.

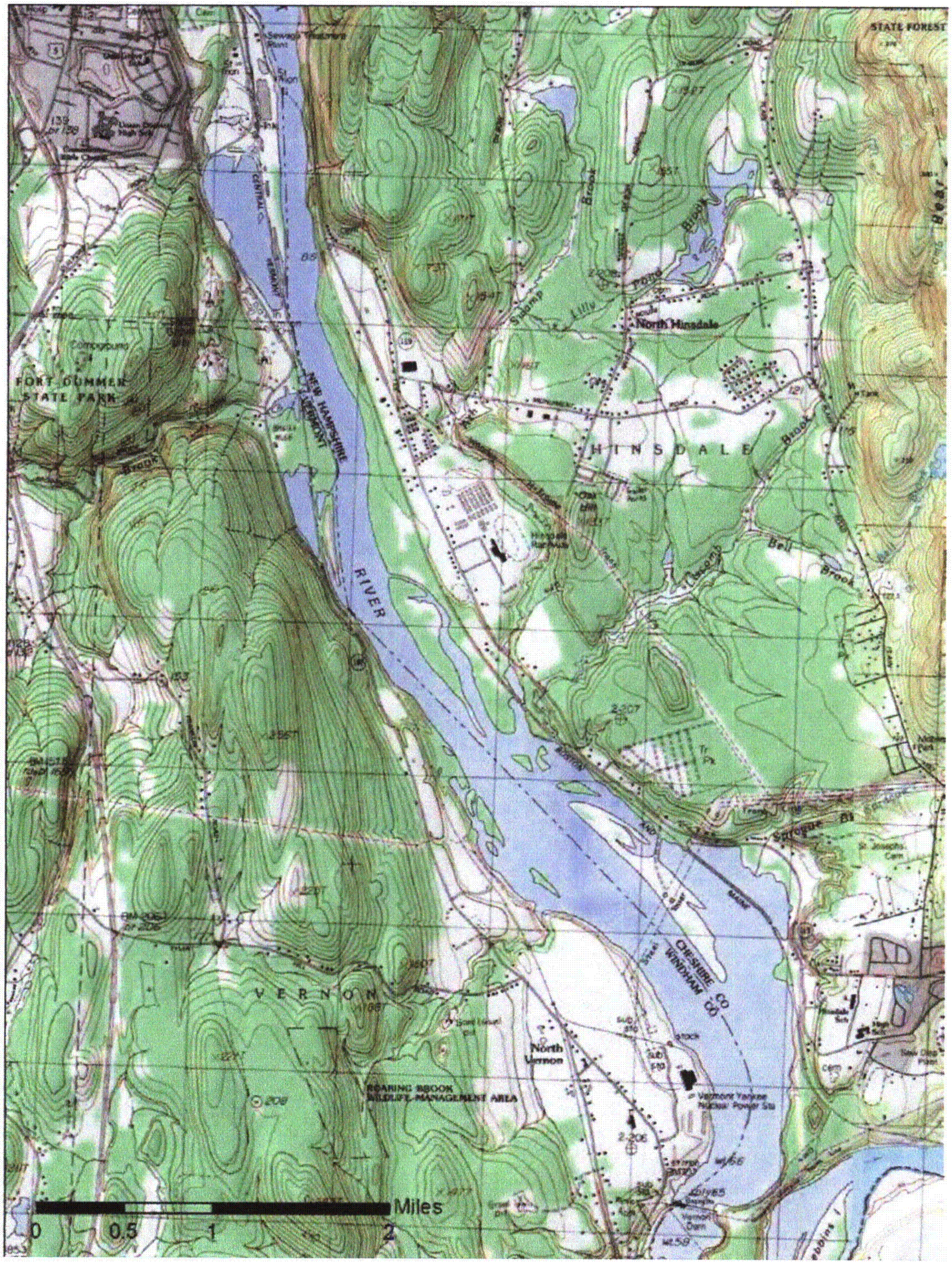

Figure 3.6-1: VYNPS Area Topography

3.7 Ice Induced Flooding

Ice jams and ice dams can form in rivers and streams adjacent to a site and may lead to flooding by two mechanisms: (1) collapse of an ice jam or a dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site and (2) an ice jam or a dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects (NRC 2011).

3.7.1 Method

3.7.1.1 Ice-Induced Flooding

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011) was used for the evaluation of the effects of ice-induced flooding on water surface elevation at VYNPS (AREVA 2012b).

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

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

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

With respect to ice-induced flooding on the Connecticut River, the HHA used the following steps:

1. Identify largest historic ice-induced flooding event and calculate water depth;
2. Identify critical locations where ice jams might form upstream and downstream of the VYNPS site.
3. Conservatively calculate peak water surface elevation resulting from failure of upstream ice jam

4. Conservatively calculate peak water surface elevation from backwater effects resulting from a downstream ice jam

3.7.2 Results

Vermont Yankee Nuclear Power Station (VYNPS) is located on the west bank of the Connecticut River in Vernon, Vermont. VYNPS is situated adjacent to the impoundment formed by the Vernon Dam at approximately river mile 138 from the mouth of the river and approximately 0.5 miles upstream of the Vernon Dam. The nominal site grade is elevation 252 ft., which is approximately 32 ft. above the maximum normal pool elevation of 220.1 feet NGVD 29 at Vernon Dam (TransCanada 2007).

3.7.2.1 Historic Ice-Induced Flooding Events

Records of historic ice jam flood stages were downloaded from the U.S. Army Corps of Engineers (USACE) ice jam database (USACE 2013) for the Connecticut River in Vermont, New Hampshire, and Massachusetts. The largest historic ice-induced flooding event occurred March 10, 1946 at North Walpole, New Hampshire, 30.2 miles upstream of VYNPS. The resultant calculated flood stage water depth behind the ice jam was 15.6 feet.

3.7.2.2 Peak Water Surface Elevation from Failure of Upstream Ice Jam

The peak water surface elevation at VYNPS was calculated for an ice jam forming and breaching at the Boston and Maine Railroad Bridge, the first bridge upstream of VYNPS and within the upstream limits of the Vernon Dam impoundment. The ice jam at the Boston and Maine Railroad Bridge was conservatively considered to be equivalent to the largest historic ice jam recorded at North Walpole, New Hampshire. The maximum normal pool elevation at both VYNPS and at the Boston and Maine Railroad Bridge is assumed to be coincident with the maximum normal pool elevation at the Vernon Dam Pool (220.1 ft., NGVD 29) because they are both located within the upstream limits of the Vernon Dam impoundment. The resultant calculated ice jam flood stage water elevation at the Boston and Maine Railroad Bridge, as a result of the historic 15.6 feet high ice jam, is thus 235.7 feet NGVD 29. However, this resulting ice jam top elevation is greater than the bridge low chord of 231.9 feet. As a result, a conservative approach was taken and it is assumed that the ice jam will accumulate even higher behind the bridge superstructure, resulting in a peak water surface elevation that will be at the same elevation as the bridge deck (242.3 feet, NGVD 29). To compute the peak water surface elevation at VYNPS resulting from the failure of this hypothetical ice dam, the peak flood wave height was conservatively kept constant (i.e., did not allow for attenuation as the flood wave traveled 4.7 river miles southerly or downstream to VYNPS). Therefore, the peak water surface elevation at VYNPS due to an ice jam forming and breaching at the Boston and Maine Railroad Bridge is 242.3 feet (NGVD 29), or 10.2 feet below the CLB maximum stillwater elevation at VYNPS resulting from a flood (Entergy 2012).

3.7.2.3 Peak Water Surface Elevation from Backwater Effects

Peak water surface elevation was calculated for an ice jam forming at the Vernon Dam, the first dam downstream of VYNPS (about 0.5 river miles south or downstream of VYNPS). Peak flood height at VYNPS was considered to be equal to the top elevation of the downstream ice jam. The resultant peak

water surface elevation at VYNPS was equal to the maximum normal pool elevation at VYNPS (described above) combined with the water depth of the backwater from the ice jam.

Similar to Section 2.3.2.1.2, the ice jam at the Vernon Dam was initially considered to be equivalent to the largest historic ice jam recorded at North Walpole, New Hampshire, as discussed above. The top elevation of the hypothetical maximum historic ice jam of 15.6 feet, when translated to the Vernon Dam, results in a peak water surface elevation of 235.7 feet NGVD 29 at the Vernon Dam. This ice jam would block the spillway gates and exceeds the bridge deck that spans over the spillway gates. Because the Vernon Dam is operated in a manner which accounts for the potential for ice jams, this result was judged to be conservative. Therefore, the resulting peak water surface elevation at VYNPS was also calculated to be 235.7 feet NGVD 29, which is 16.8 feet below the existing CLB maximum stillwater elevation of 252.5 feet, NGVD29 (Entergy 2012).

3.7.3 Conclusions

The peak water surface elevation at VYNPS, resulting from the upstream ice jam / ice dam breach was conservatively calculated to be 242.3 feet (NGVD 29), 10.2 feet below the CLB PMF elevation of 252.5 ft. This was also bounded by the PMF peak water surface elevation of the Connecticut River at VYNPS of 249.7 feet (AREVA 2013).

The peak water surface elevation at VYNPS as a result of backwater caused by the ice jam at the Vernon Dam was conservatively calculated to be 235.7 feet (NGVD 29), 16.8 feet below the CLB maximum stillwater elevation. This was also bounded by the PMF peak water surface elevation on the Connecticut River at VYNPS of 249.7 feet (AREVA 2013).

Ice-induced flooding is not specifically included as a mechanism to be combined with other extreme events as per NURGE/CR-7046 (NRC 2011).

3.7.4 References

AREVA 2013. AREVA Document No. 32-9196325-000, “Probable Maximum Flood on Connecticut River – Hydrology.”

Entergy 2012. “Updated Final Safety Analysis Report, Vermont Yankee Nuclear Power Station, Revision 25”, Entergy, May 2012.

NRC 2011. “Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046”, U.S. Nuclear Regulatory Commission, November 2011 (ADAMS Accession No. ML11321A195).

USACE 2013. U.S. Army Corps of Engineers (USACE), Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, Website: http:icejams.crrel.usace.army.mil/, accessed January 2013.

TransCanada 2007. TransCanada Hydro Northeast Inc. Reservoir and Minimum Flow Operations and Monitoring Plan, Vernon Hydroelectric Project FERC Project No. 1904, December 2007.

3.8 Channel Migration or Diversion

Natural channels may migrate or divert either away from or toward the site. The relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site (NRC 2011).

3.8.1 Method

The channel migration and diversion flooding evaluation followed the HHA approach described in NUREGCR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC 2011).

The HHA approach used is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977;
3. NUREG/CR-7046 – Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, dated November 2011; and

With respect to channel migration and diversion, the HHA used the following two steps:

1. Review historical records and geologic data to assess whether the Connecticut River exhibits the tendency to migrate towards the site.
2. Evaluate the foundation type at critical structures to assess potential susceptibility to erosion caused by possible channel migration.
3. Evaluate present-day channel stabilization and maintenance measures in place to mitigate channel migration of the Connecticut River.

3.8.2 Results

3.8.2.1 Historical Records

There have been extensive studies of riverbank erosion along nearly the entire length of the Connecticut River dating back to 1954 (Simons & Associates 2012). While erosion has been and continues to be of concern at many points along the river, a literature review did not yield evidence suggesting there have been significant diversions of the river near VYNPS. The Vernon Hydroelectric

Dam, constructed in 1909, is located immediately downstream of VYNPS and acts to restrict the Connecticut River's velocity near VYNPS where the water is impounded (Figure 3.8-1). A comparison of a 1935 topographic map and a 1984 topographic map illustrates continuity of the river course over the last nearly 50 years, see Figure 3.8-2. The only significant difference between the two maps is the increase in the number and size of islands in the river just upstream of the site since 1935. Following the construction of the Vernon Dam in 1909, sediment deposition likely increased as the river velocity in this area decreased, resulting in increased sediment deposition and the continuous slow growth of these islands.

Although there is a high potential for landslides along the Connecticut River, no evidence of landslide induced channel diversion near VYNPS was found during the literature review.

One significant change in the river bank surface topography caused by the construction of the VYNPS was the extension of the terrace eastward by 200 feet at its greatest point, adding approximately 30 to 35 feet to the existing elevation to match the site grade (GZA 2011). During construction, the riverbank of the terrace was stabilized with a free draining gravel blanket covered with a riprap slope (VYNPC, 1970). The riprap surface was constructed with a slope of 1:2 (UFSAR 2012). An inspection of the slope in 2006 found that the riprap is 2 to 3 feet thick near the waterline grading to about 6 inches in thickness near the top of the slope, consistent with the original installation plans. A diving inspection confirmed that the riprap extends about 5 feet out past the waterline as specified in the original plans. The other banks along VYNPS are also armored with riprap except in the cases where bedrock outcrops are present, providing stabilization and protection against erosive forces (GEI 2009).

Vernon Neck is a natural geologic feature that forms the east abutment of the Vernon Dam (Figure 3.8-1). The potential for the breaching of Vernon Neck, although judged unlikely, has been examined. Vernon Neck is armored where the neck is the thinnest and following the 1936 flood, which approximated a 500-yr flood, the Neck did not experience any adverse erosion (GEI 2009).

3.8.2.2 Foundation Types and Susceptibility to Erosion

VYNPS was constructed on an existing terrace deposit. However, the major structures at VYNPS (Cooling Water Intake, Turbine Building, Control Building, Stack, and Reactor Building) are all founded in bedrock or piles to bedrock. The placement of the buildings was formulated to capitalize on an existing high area of the bedrock surface. During construction, targeted areas of the natural soils and bedrock were excavated from the power block area to allow the reactor building and other major structures to be founded on bedrock to meet the required construction design (GZA 2011). Due to this design, the major structures at VYNPS are not vulnerable to the impacts of erosion.

A previous study evaluated the potential for erosion along the riverbank adjacent to VYNPS in the unlikely event of the breaching of the Vernon Neck. The results showed a maximum lateral scour of 25 feet along VYNPS's riverbank (Figure 3.8-4). As there are no structures in this area, only some local damage to the riverbank would need to be repaired following such an event. The results of this study are conservative as many elements that would factor into reducing scour, including riprap slopes, heavy vegetation, and the location of the VYNPS on an inside bend (depositional area), were ignored (GEI 2009).

3.8.2.3 Present-Day Channel Stabilization and Maintenance

The banks of the river near VYNPS are well stabilized with riprap and there has been little evidence of erosion (UFSAR 2012). The Connecticut River channel proximate to VYNPS is periodically dredged to maintain a river depth profile similar to the pre-operational profile. Normandeau Associates conducts a yearly bathymetric survey of the Connecticut River near the intake structure for VYNPS. Dredging of the river is triggered when the survey indicates that greater than 3,000 cubic yards of sediment has filled in the 120 yd by 120 yd survey area compared to the pre-operational depth or more than 30% of the intake structure is blocked. The pre-operational river depth in this area is about 30 feet (Vermont Yankee, 2007). The most recent dredging occurred during the summer of 2011 (Normandeau 2011).

3.8.3 Conclusions

Historical data indicate that the Connecticut River has not exhibited a tendency to meander towards VYNPS. The Vernon Dam, located just downstream of VYNPS, impounds the Connecticut River and acts to restrict the river's velocity and thus its erosive power near VYNPS. A 2009 flood study at VYNPS by GEI showed that even in the unlikely event of a breach at Vernon Neck combined with the PMF, flood flows over the Site would not be powerful enough to cause erosion and the maximum lateral scour of the VYNPS's riverbank would be 25 feet. None of VYNPS's structures would be impacted by such an event as they are all located over 200 feet from the impacted bank. In the highly unlikely event that the Connecticut River does migrate towards VYNPS, the structures of VYNPS's power block are founded on bedrock which is judged to be unsusceptible to erosion caused by potential channel migration.

3.8.4 References

ESRI ArcGIS Online World Topographic Map Service. Published December 2012 by ESRI ARCIMS Services.

GEI 2009. "Flood Study of the Connecticut River," Geotechnical, Environmental and Water Resources Engineering (GEI), February 2009.

GZA 2011. "Hydrogeologic Investigation of Tritium in Groundwater," GZA GeoEnvironmental, Inc., May 2011.

Normandeau 2011. Letter to Entergy Nuclear Vermont Yankee, LLC., Ref No. 21333.033 Task 4, Normandeau Associates, October 2011.

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

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University of New Hampshire. "Historic USGS Maps of New England & NY" , 1935 Burlington, VT Quadrangle, http://docs.unh.edu/nhtopos/nhtopos.htm.

Vermont Yankee, 2007. "Conduct Bathymetric Survey of River in Vicinity of Intake Structure," Entergy, February 2007.

VYNPC, 1970. "General Excavation and Grading and Performance of Related Work for Vermont Yankee Nuclear Power Station 1970 – Installation." Vermont Yankee Nuclear Power Corporation, Ebasco Services Incorporated, Agent, and Morrison-Knudsen Company, Inc., 1970.

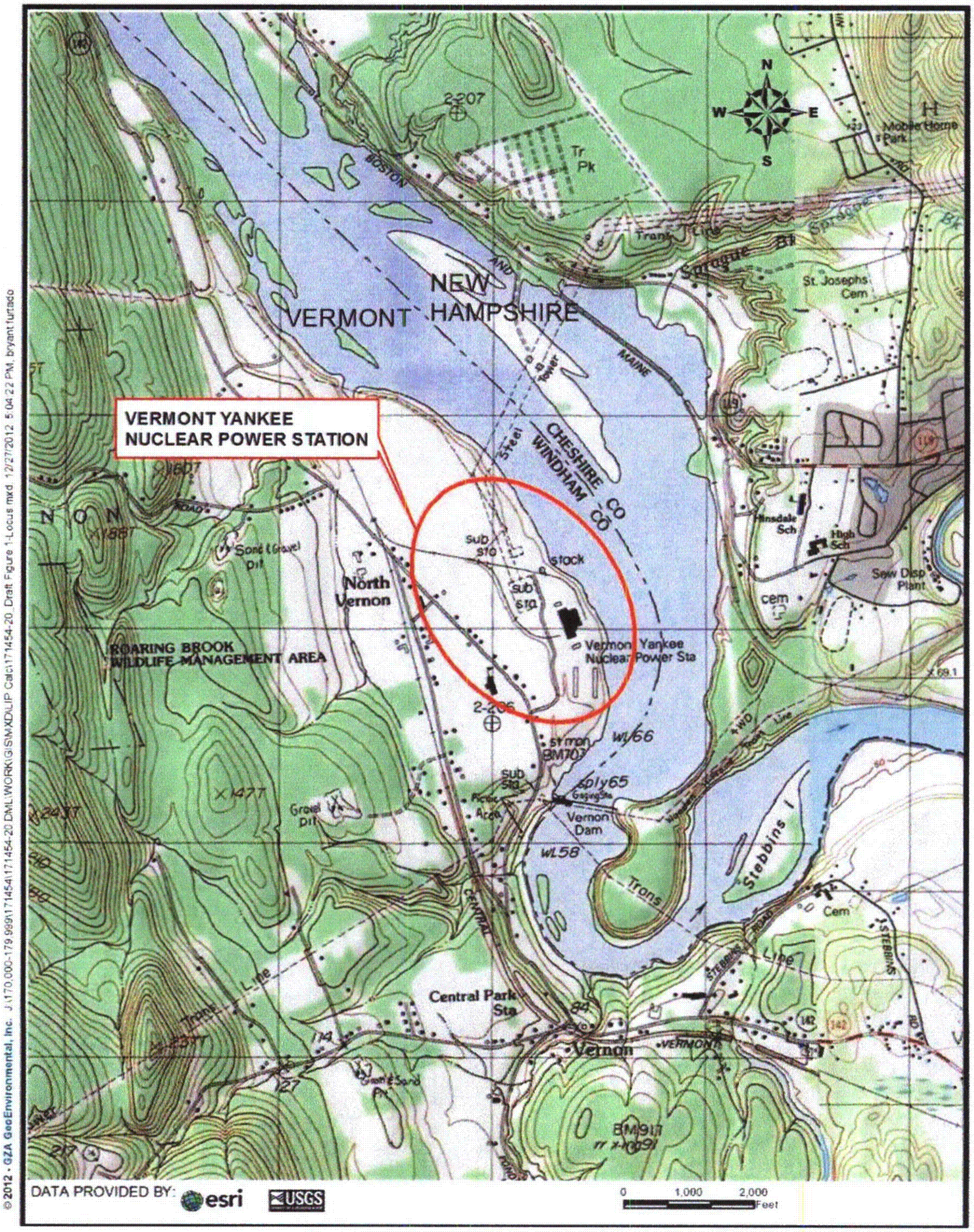
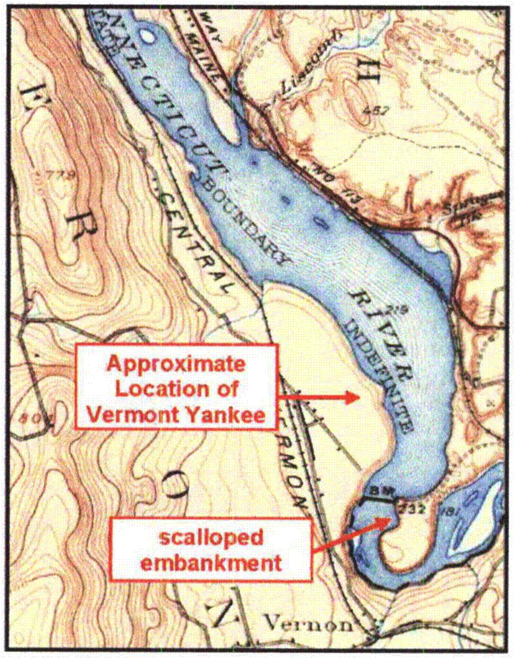
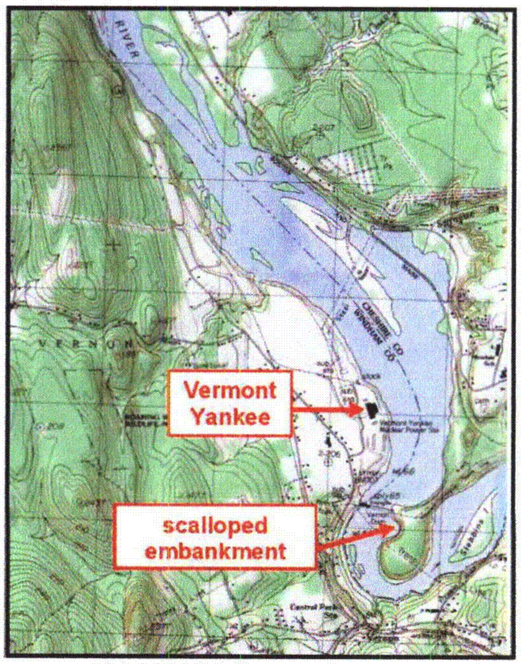


Figure 3.8-1: Locus Map



1935 USGS Topographic Map (Source: UNH)

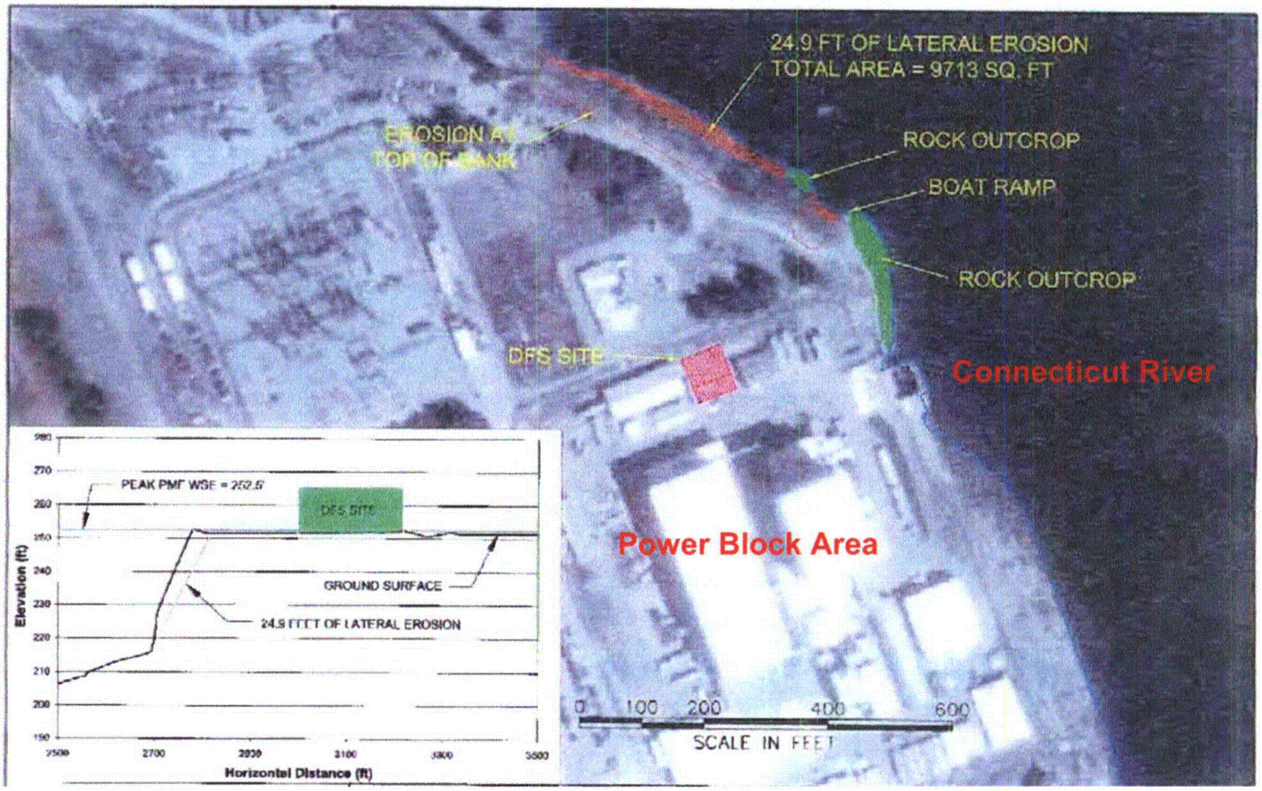


1984 USGS Topographic Map
(Source: ESRI ArcGIS Online World Topographic Map service)

Figure 3.8-2: Comparison of USGS Topographic Maps

Riprap slope along northern portion of Vermont Yankee Nuclear Power Station shoreline.
(Source: GEI, 2009)

Figure 3.8-3: Riprap Slope



(Adapted from GEI 2009)

Figure 3.8-4: Potential Maximum Scour

3.9 Combined Effect Flood

This section addresses combined effect flooding at VYNPS and includes the impacts of the effect of the PMF on the Connecticut River coincident with wind generated waves at VYNPS.

3.9.1 Method

The Hierarchical Hazard Assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011) was used for the evaluation of the effects of the combined-effects floods on the Connecticut River at VYNPS.

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

The criteria for combined events are provided in NUREG/CR-7046, Appendix H, of which two apply to VYNPS: Floods caused by precipitation events (H.1) and Floods caused by seismic events (H.2). Other criteria for the determination of the effects of the combined-effect flood described in NUREG/CR-7046 (NRC 2011, Appendix H, Sections H.3 – H.5) do not apply to VYNPS because the site is not a coastal site.

Floods Caused by Precipitation Events

The criteria for floods caused by precipitation events were used as one input to the combined event calculation (NUREG/CR-7046, Appendix H, Section H.1). The criteria include the following:

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

Floods Caused by Seismic Dam Failures

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

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

The results of the dam failure calculation, AREVA Document No. 32-9196327-00 (AREVA 2013b), indicate that flood elevations resulting from seismic dam failures (NUREG/CR-7046, Appendix H.2) are bounded by those resulting from the PMF with coincident hydrologic dam failure on the Connecticut River at VYNPS. Therefore, further calculations to address NUREG/CR-7046 Appendix H.2 are not necessary.

Other criteria for the determination of the effects of the combined-effect flood described in NUREG/CR-7046 (Appendix H, Sections H.3 – H.5) do not apply to VYNPS given the site is not along the shore of an open, semi-enclosed or enclosed body of water. Therefore, "Alternative 1" under the "Floods Caused by Precipitation Events" sub-section of Appendix H has been judged to be the controlling scenario for Combined-Effects Floods. The Maximum Water Elevation for the combined effects flood is thus the sum of the Probable Maximum Still Water Elevation from the bounding hydrologic event and the wind generated wave runup resulting from a 2-year wind speed applied along the critical direction.

The combined event evaluation for VYNPS used the following steps:

- Calculate the wind wave effects and wave runup on the Connecticut River at VYNPS using the CEDAS-ACES v4.3 Computer Program (AREVA 2013c);
- Calculate the Probable Maximum Water Elevation on the Connecticut River at VYNPS resulting from the combined-effect flood.

3.9.2 Results

3.9.2.1 Wind-Wave Effects

As per NUREG/CR-7046, Appendix H, Section H.1, wind-wave effects to be used as inputs to the combined effects flood are based on wind generated wave runup resulting from a 2-year wind speed applied along the critical direction (Figure 3.9-1).

3.9.2.1.1 Straight Line Fetch

The baseline PMF water surface elevation was calculated using the USACE HEC-RAS model developed in AREVA Document No. 32-9196327-000 (AREVA 2013a). The effect of hydrologically-induced dam failures on that water surface elevation was calculated in AREVA Document No. 32-9196327-000, "VYNPS Dam Failures", January 2013 (AREVA 2013b). The PMF maximum still water surface elevation with coincident hydrologic upstream dam failure during the PMF for the VYNPS watershed is Elevation 252.0 feet NGVD29. This water surface elevation was used along with elevation contours along the river bank to estimate the wetted top widths for five (5) selected cross sections through the site and river, as depicted in Figure 3.9-2 (AREVA 2013). A maximum fetch of 2,650 feet perpendicular to the shore at the site (cross section 2) was determined to be a conservative input for the CEDAS- ACES v4.03 module.

3.9.2.1.2 Sustained Wind Speed

Using the Gumbel Distribution on the 2-minute duration wind speed data from NCDC station at Orange Municipal Airport (NOAA 2012), the 2-year return period wind speed was determined to be 37 miles per hour.

3.9.2.1.3 Wave Height and Period

The Wave Prediction application of the CEDAS - ACES v.4.03 was used to determine the deep water significant wave height and period. A negative twenty-seven degrees Fahrenheit air sea temperature was selected as a conservative input for wave prediction. Using a negative value indicates air temperatures colder than the water temperatures, which is likely during an extreme precipitation event such as the PMF. The duration of the final wind speed was selected to be twenty minutes. This is a conservative estimate used for a two-year return period wind speed. The deepwater significant wave height for the cross-section with the maximum fetch was determined to be 1.06 feet with a period of 1.63 seconds.

3.9.2.1.4 Wave Runup

The Wave Runup and Overtopping on Impermeable Structures application was selected to calculate the wave runup at VYNPS from the CEDAS-ACES v.4.03 program. Nearshore slopes were estimated from the HEC-RAS hydraulic model cross section nearest the site as developed in AREVA Document No. 32-9196326-000, "Probable Maximum Flood on Connecticut River – Hydraulics" (AREVA 2013a). Because the water depths are deep and the wave periods are short, wave growth is governed by deep open water conditions. The slopes of the shore along wave runup cross sections 1, 4, and 5 (as conservatively selected from visual observation during the site walk) were developed using the site topography drawing (AREVA 2013). Wave runups (including setup) for cross sections 1, 4, and 5 were calculated in the program based on equations developed for rough slopes and using roughness coefficients describing riprap. Wave runup (including setup) for cross sections 2 and 3 were conservatively calculated in the program based on equations developed for smooth slopes. Wave runup calculation inputs are shown in Table 3.9-1.

Wave runup and overtopping rates were calculated for each cross section. The maximum wave runup and overtopping rate (at cross section 2) were calculated to be 1.4 feet and 1.3 cfs per linear foot, respectively.

3.9.2.2 Maximum Water Elevation Resulting from the Combined Effect Flood

The 100-year snowpack and a site-specific cool-season PMP were identified in AREVA Document No. 32-9196325-000, "Probable Maximum Flood on Connecticut River - Hydrology" (AREVA 2012d) to result in the controlling PMF at VYNPS. The controlling PMF included mean monthly cool season baseflow. Therefore the Alternative 3 (including hydrologic dam failure) scenario of the Flood Caused by Precipitation Events category was judged to be the bounding condition to which wind wave effects would be added to calculate the Maximum Water Elevation resulting from a combined effects flood. The Probable Maximum Water Elevation at VYNPS is the combination of the probable maximum still water elevation and runup from waves induced by the 2-year wind speed:

Probable Maximum Water Elevation Resulting from Combined Events =

Probable Maximum Still Water Elevation + 2 Year Wind Wave Runup Elevation

The probable maximum stillwater elevation on the Connecticut River at Vermont Yankee Nuclear Power Station is 252.0 ft. NGVD29 (AREVA 2013a). The wave runup induced by the 2-year wind speed was calculated to be 1.4 feet.

The Maximum Water Elevation on the Connecticut River at VYNPS is the combination of this stillwater elevation and wave runup induced by the 2-year wind speed applied along the critical direction:

252.0 ft. + 1.4 ft. = 253.4 ft. (NGVD29)

3.9.2.3 Debris Loading and Transportation

Debris loading is a function of flood flow velocity and flood flow depth at the site. Debris loading is not significant when the flood flow velocity or depth are low. The hydrodynamic forces for low velocity flow (less than 10 feet per second) are considered equivalent to hydrostatic force, increased by the head due to the low velocity flood flow (FEMA 2012). Debris loading on structures above the VYNPS site grade is expected to be negligible given that the Probable Maximum Stillwater Elevation is at site grade and no hydrostatic or hydrodynamic loading would occur. The debris loading caused by the 1.4 foot wind wave runup is also expected to be minimal due to the low velocity river flow of 1 ft./sec at VYNPS (during the PMF with upstream dam failure).

3.9.3 Conclusions

At VYNPS, the bounding condition for impacts to the site from combined event flooding on the Connecticut River result from PMF water surface elevations combined with wind generated waves.

The wind generated wave runup resulting from a 2-year wind speed applied along the critical direction on the Connecticut River at VYNPS was calculated to be 1.4 feet.

The bounding still water elevation for riverine flood events is 252.0 feet NGVD29. This flood level results from the peak water surface elevation on the Connecticut River at VYNPS due to the Probable Maximum Flood (PMF) including hydrologic upstream dam failure. This scenario, with the inclusions of wave runup induced by 2-year wind speeds, is described as Alternative 3 of Section H.1 of Appendix H of NUREG/CR-7046.

The Maximum Water Elevation resulting from combined events on the Connecticut River at VYNPS was calculated as the sum of the bounding flood event (still water elevation 252.0 NGVD29) and the 2-year wind wave runup (1.4 feet). This Maximum Water Elevation resulting from combined riverine events was calculated to be elevation 253.4 feet NGVD29.

The results indicate that the combined effects flood maximum water elevation on the Connecticut River at VYNPS resulting from combined-effect flood is above the plant grade elevation of 252.0 feet NGVD29 but below the CLB PMF plus wind activity elevation of 254.0 NGVD29.

3.9.4 References

AREVA 2012. AREVA Document No. 38-9196957-000, Vermont Yankee (VY) Topographic Survey, Sanborn Map Company, Inc.

AREVA 2013a. AREVA Document No. 32-9196326-000, "Probable Maximum Flood on Connecticut River – Hydraulics."

AREVA 2013b. AREVA Document No. 32-9196327-000, "VYNPS Dam Failures."

AREVA 2013c. AREVA Document No. 38-9196713-000, "GZA Computer Program Certification for CEDAS-ACES Version 4.03 PC."

AREVA 2013d. AREVA Document No. 32-9196325-000, "Probable Maximum Flood on Connecticut River – Hydrology."

FEMA 2012. "Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures", Federal Emergency Management Agency (FEMA) P-259, pg. 4-10 through 4-25, January 2012.

NOAA 2012. NOAA National Climatic Data Center Fastest 2-minute wind speed data: Verified Data, Orange Municipal Airport, MA, Station ID GHCND: USW00054756. Retrieved 17 January 2013. Available at: http://www.ncdc.noaa.gov/cdo-web/review.

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

UFSAR 2012. "Updated Final Safety Analysis Report, Vermont Yankee Nuclear Power Station, Revision 25", Entergy, 2012.

Table 3.9-1: Wave Runup Inputs

Input Parameter	Cross Section 1	Cross Section 2	Cross Section 3	Cross Section 4	Cross Section 5
Fetch	2535	2650	2430	2320	2000
Wave Type	Irregular	Irregular	Irregular	Irregular	Irregular
Slope Type	Rough	Smooth	Smooth	Rough	Rough
Breaking Criteria (k)	0.78	0.78	0.78	0.78	0.78
Incident Significant Wave Height (H_i) [feet]	1.04	1.06	1.01	0.99	0.92
Peak Wave Period (T) [seconds]	1.61	1.63	1.59	1.56	1.49
COTAN of nearshore slope (cot phi)	20	20	20	20	20
Water depth at the structure toe (d_s) [feet]	27	12.0	33	22.0	24
COTAN of structure slope (cot theta)	3.95	2.65	2.75	2.65	3.32
Structure height above toe (h_s) [feet]	27.001	12.1	33.1	22.1	24.1
Rough Slope coeff. "a"	0.956	--	--	0.956	0.956
Rough Slope coeff. "b"	0.398	--	--	0.398	0.398
Onshore Wind Speed [feet/sec]	54.22	54.22	54.22	54.22	54.22

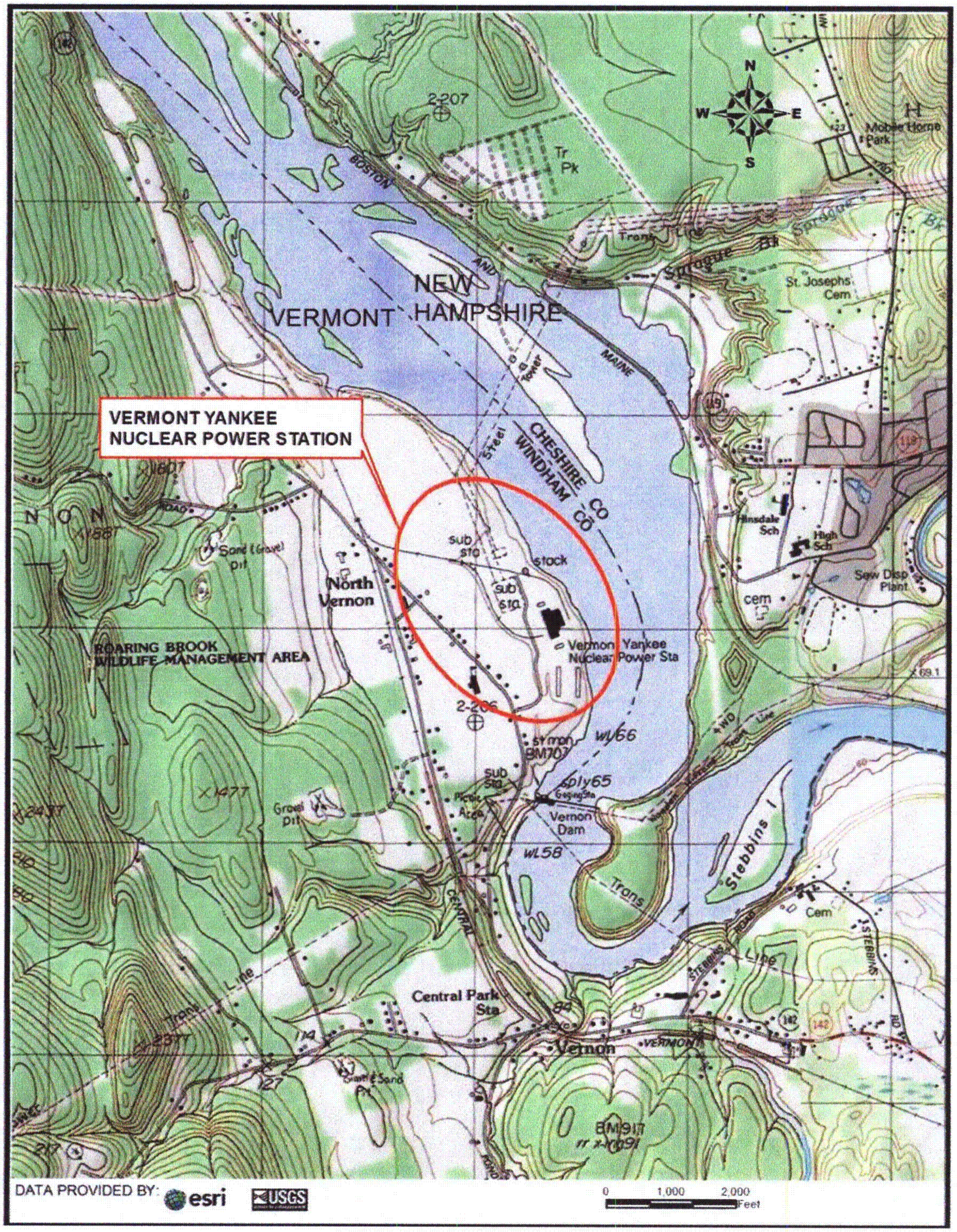


Figure 3.9-1: VYNPS Location Map

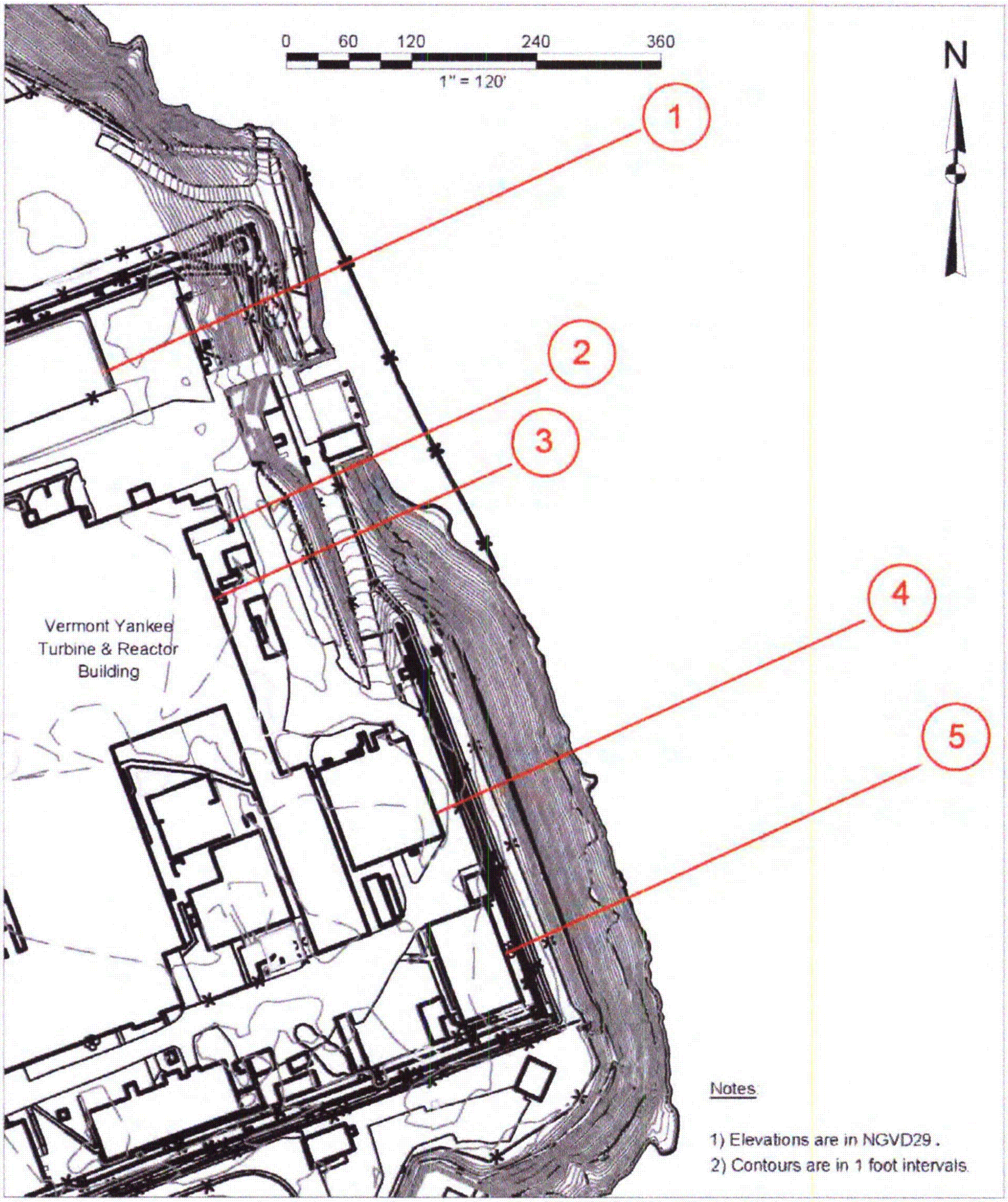


Figure 3.9-2: Wave Runup Cross Section Locations

4.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

This section provides a comparison of the CLB flood elevation to the reevaluated flood elevation for each applicable flood causing mechanism. The CLB flood levels are described in Section 2.2.2 and the reevaluation flood levels using present-day methodologies and regulatory guidance are described in the respective subsections in Section 3.

4.1 Results

Table 4.1-1 summarizes the comparison between CLB and reevaluation flood levels for each flood mechanism. Flood mechanisms noted as "Screened" have been evaluated at a high level and determined to not be applicable to the flooding hazard for VYNPS or negligible and, therefore, bounded by the PMF.

The results of the comparison between the two are described below.

1. Local Intense Precipitation (LIP) – The LIP was not evaluated as part of the CLB. The reevaluation flood levels varied on site, depending on location. The maximum LIP flood level of 253.0 ft., which is greater than the CLB PMF level of 252.5 ft. However, regardless of the site location, all LIP flood elevations are below 254 ft., the CLB PMF plus wave runup elevation. For protection against the PMF plus wave activity, plant operating procedure OPOP-PHEN-3127 must be implemented to protect SSC important to safety. Presently, the advance notice needed prior to implementation of OPOP-PHEN-3127 is 96 hours. However, the meteorological conditions that produce a LIP event are such that a shorter advanced notice will likely be needed.

   Thus, the CLB for the PMF plus wave activity flood level will continue to be the bounding value, provided the plant operating procedure OPOP-PHEN-3127 is revised to address the LIP and the shorter advanced notice prior to a LIP event.

2. Probable Maximum Flood (PMF) –The CLB for the PMF is 252.5 ft. The reevaluation PMF level is 249.7 ft., 2.8 ft. less than the CLB. Thus, the CLB for the PMF level is the bounding value.

3. Dam Breaches and Failures – The CLB for Dam Failure is less than the PMF of 252.5 ft. The reevaluation Dam Breaches and Failures flood level is 252.0 ft., 0.5 ft. less than the CLB. Thus, the CLB for the PMF level is the bounding value.

4. Storm Surge - Storm Surge was screened during both the CLB and reevaluation. As a result, the CLB for the PMF level is the bounding value.

5. Seiche - Seiche was screened during both the CLB and reevaluation. As a result, the CLB for the PMF level is the bounding value.

6. Tsunami – Tsunami was screened during both the CLB and reevaluation. As a result, the CLB for the PMF level is the bounding value.

7. Ice-Induced Flooding – Ice-Induced Flooding was screened during the CLB. The reevaluation Ice-Induced flood level is 242.3 ft., 10.2 ft. less than the CLB for the PMF. Thus, the CLB for the PMF level is the bounding value.

8. Channel Migration or Diversion - The CLB for Channel Migration or Diversion, which also included the failure of cooling water canals and reservoirs, was screened. The reevaluation for Channel Migration or Diversion also was screened. Thus, the CLB for the PMF flood level is the bounding value.

9. Combined Effect – The CLB for Combined Effect, which is the equivalent of the PMF plus wave activity, is 254.0 ft. The reevaluation for Combined Effect flood for PMF with Dam Failures results in a flood level of 253.4 ft., 0.6 ft. less than the CLB for the PMF plus wave activity. Thus, the CLB for the PMF plus wave activity flood level is the bounding value.

4.2 Conclusion

Based on the results shown in Table 4.1-1, and described in Section 4.1 above, all reevaluation flood levels are either below

a) the CLB PMF flood elevation of 252.5 ft., or

b) the CLB PMF flood elevation plus wave activity of 254.0 ft., with revised implementation of plant operating procedure OPOP-PHEN-3127 to address the LIP maximum of 253.0 ft.

As a result, the plant is suitably protected against the existing CLB flood levels. Section 5 describes the interim action that will be taken, and its approach, to revise plant operating procedure OPOP-PHEN-3127 to ensure the required operational and administrative controls are implemented in the case of a pending LIP event.

Table 4.1-1: Flood Elevation Comparison

Flood Mechanism	CLB Elev. (ft.)	Reeval Elev. (ft.)
1. Local Intense Precipitation (LIP)	Not Evaluated	≤253.0
2. Probable Maximum Flood (PMF)	252.5	249.7
3. Dam Breaches and Failures	<252.5	252.0
4. Storm Surge	Screened	Screened
5. Seiche	Screened	Screened
6. Tsunami	Screened	Screened
7. Ice-Induced Flooding	Screened	242.3
8. Channel Migration or Diversion	Screened	Screened
9. Combined Effect (PMF + Wind Wave)	254.0	253.4

Note: Flood mechanisms noted as "Screened" have been evaluated at a high level and determined to not be applicable to the flooding hazard for VYNPS or negligible and bounded by the PMF.

5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

Local Intense Precipitation (LIP) is the sole event determined to exceed the Current Licensing Basis (CLB) flood level. This section describes the actions taken to mitigate the effects of the LIP. This section also addresses the two events that were either not included in the CLB, or due to the results of the evaluation in Section 3, may have different response times than previously considered. Those events are Ice-Induced Flooding and Dam Failures, respectively.

Activities to mitigate the effects of CLB External Flood events are performed in accordance to Procedure OPOP-PHEN-3127 "Natural Phenomena".

5.1 Local Intense Precipitation

The results of the evaluation for LIP in Section 3.1 estimate that the maximum water level along the West side of the main plant structures reaches Elevation 253.0 ft. This level exceeds the maximum CLB stillwater Elevation 252.5 ft. resulting from the Probable Maximum Flood (PMF) on the Connecticut River. LIP flood water elevations diminish as the water flows around the structures toward the Connecticut River (see Figure 3.1-5).

VYNPS currently has mitigative actions in place to protect vital SSC's, including the Switchgear Rooms for the PMF plus wave run-up, to Elevation 254.0 ft. LIP flooding is not included in the CLB; however, the runoff carrying capacity of the site grading design due to LIP was addressed as part of the Individual Plant Examination External Events (IPEEE). The IPEEE evaluation examined the PMP using three separate criteria: (1) the NWS PMP, (2) a 10^{-6} probability rainfall, and (3) the 1975 SRP Criteria.

The resulting one-hour rainfall PMP's were 16.4 inches, 5.6 inches, and 8.4 inches respectively. Utilizing these PMP's and the resulting peak flow rates for each, maximum water surface elevations on the site were determined to be 252.4 ft., 252.0 ft., and 251.9 ft. respectively.

The Design Basis Document for External Events states the following:

> *Equipment required for operation during a PMF include the RHR pumps and heat exchangers, the RHR service water pumps, a cooling tower cell, and the electrical and piping systems required for operation of these components. If normal electrical power is unavailable, diesel generators and fuel oil pumps are also required. The RHR pumps, the RHR heat exchangers and RHR service water pumps are located within the Reactor Building and the electrical equipment is located within the Turbine and Control Buildings. The fuel oil pumps are located within a structure that forms the tornado and flood protection around the fuel oil tank.*

Equipment required for operation during a CLB PMF is protected by actions implemented by Plant Procedure OPOP-PHEN-3127. Although the CLB does not include the LIP event, the procedure does recognize that this type of event could affect plant equipment, as acknowledged in the following NOTE:

> *Localized site flooding due to heavy hurricane rainfall could occur early on in a Probable Maximum Flood condition well before appreciable rise in the river level. This condition could result in the backup of the yard storm drain system to yard grade levels. Therefore, the actions described below associated with protecting the Switchgear Room and the Administration*

Building North door during flooding should be considered at the beginning of the heavy site hurricane rainfall.

Certain actions described in the procedure are applicable in protecting the plant equipment from the effects of the LIP event evaluated as part of the NTTF recommended evaluation.

5.1.1 Protection of the Switchgear Rooms (Located in the Control Building)

There are two Switchgear Rooms (one for each train of Safety Related equipment). One is identified as the East Switchgear Room and one is identified as the West Switchgear Room. There are common doors providing access between the two rooms. The East Switchgear Room can be accessed by a set of double doors leading to the East exterior of the Control Building or by the common doors to the West Switchgear Room. The West Switchgear Room can be accessed by an interior single door (North) in the Administration Building vestibule hallway, or by a set of interior double doors (South) that open into the hallway separating the Control and Reactor Buildings from the Turbine Building. The floor of the Switchgear Rooms is at Elevation 248.5 ft. and is below nominal site grade Elevation 252.0 ft.

The main pathways for LIP water to reach the Switchgear Rooms are via certain of the doors discussed above, the floor drain in the East Switchgear Room, or manholes which communicate with exterior electrical manholes.

Water from the LIP could reach the single North door to the West Switchgear Room via two exterior entrances to the Administration Building. The sills for these exterior doors are at Elevation 252.5 ft. (West entrance) and Elevation 250.25 ft. (North entrance). The projected water elevations at these doors for the LIP are Elevation 253.0 ft. and Elevation 252.7 ft., respectively. It is noted also that the North entrance to the Administration Building is accessed via exterior stairs from grade to the door sill Elevation 250.25 ft. Additional stairs lead from inside this door to the first floor Elevation of 248.5 ft.

Water could reach the South double doors via access pathways from the Turbine Building exterior loading bay roll-up door and personnel door on the West side of the building where the LIP water elevation is 253.0 ft. Water could also reach those doors via doors in the Maintenance Building, however the LIP water elevation at those doors is less than 253.0 ft.

Note that the East Switchgear Room East exterior double doors sill elevation of 252.5 ft. is above the projected LIP water elevation of 252.4 ft.

Current OPOP-PHEN-3127 mitigative actions which would protect the Switchgear Rooms during the LIP are:

- SEAL off the Administrative Building North door entrance with a three inch plumber type floor drain plug. [OPOP-PHEN-3127, Section 5.8.5.D]
- SEAL off doors from the Admin and Turbine Building that lead(s) into SWGR. [OPOP-PHEN-3127, Section 5.8.5.E]
  - SEAL on the TB side using RTV, the SWGR double Door that opens into the Turbine Building

- SEAL on the vestibule (sic) side using RTV, the SWGR Single Security (sic) Door that opens into the Admin Building.
- SEAL on the TB side using RTV, the Single Door to the Rad Waste Hallway that opens into the Turbine Building.

- ESTABLISH fire watches as required, then open the following Switchgear Room manholes per the requirements of EN-IS-102: [OPOP-PHEN-3127, Section 5.8.5.G]
  - MH-P2
  - MH-23 (SI)
  - MH-25 (SII)
  - MH-S1 (SI)
  - MH-P1 (SI)
  - MH-22 (SI)
- OBTAIN sump pumps (Porta-Pump-1A/B) from the first floor of the Construction Storage Building and rigid suction hoses and outlet hose from the Alternate Fire Brigade Room to enable pumping each Switchgear Room manhole listed above. OBTAIN (2) five gallon gasoline storage containers filled with gasoline and stage near each pump. [OPOP-PHEN-3127, Section 5.8.5.H]
- MONITOR manholes in the Switchgear Room for water in-leakage. [OPOP-PHEN-3127, Section 5.8.5.I]
- If required to pump the Switchgear Room manholes, PERFORM the following: [OPOP-PHEN-3127, Section 5.8.6]
  - VERIFY required fire watches are established, then PLACE the Switchgear Room Low Pressure CO2 System in ABORT
  - Request Maintenance INITIATE actions as required to pump the manholes.
- If the East Switchgear Room floor drain is to be plugged, PERFORM the following: [OPOP-PHEN-3127, Section 5.9.1]
  - ISSUE a Barrier Control Permit in accordance with AP 0077 for the Switchgear Room Floor and Manholes which includes appropriate compensatory measures.
  - INSTALL the removable plug in the East Switchgear Room floor drain which is pre-staged next to the East outside double doors.

These existing actions will adequately protect the equipment located in the Switchgear Rooms. Although it is acknowledged that they are intended to provide protection from the CLB still-water

elevation which is 0.5 ft. less than the maximum LIP water elevation, the CLB precipitation event lasts for approximately 72 hours, while the LIP precipitation event only lasts 6 hours and the time that the exterior door sill elevations are exceeded is only 2 hours, thus the volume of water to be handled is significantly less for the LIP. It is not expected that a hydrostatic head equivalent to the difference in the LIP water elevation and Switchgear Room floor elevation will occur since there are also available pathways for the water to find its way to other areas in the buildings, which are at much lower elevations than the switchgear rooms.

5.1.2 Protection of the Fuel Oil Pumps and Fuel Oil Storage Tank (FOST)

The Fuel Oil Pumps are located in a concrete structure adjacent to the FOST at floor Elevation 241.5 ft. The FOST is surrounded by a concrete moat which has a top Elevation of 252.5 ft. The entrance to the building containing the Fuel Oil Pumps is also at Elevation 252.5 ft. and the lower elevation of the building is accessed by a ladder inside the building. The maximum elevation of water in the vicinity of the FOST due to the LIP is at Elevation 252.4 ft. Therefore, both the Fuel Oil Pumps and the FOST are protected by the existing structures.

5.1.3 Protection of Equipment Located Inside the Reactor Building

The RHR pumps, RHR heat exchangers, and the RHR service water pumps are located in the Reactor Building. The ground floor of the Reactor Building and its entrances are at Elevation 252.5 ft. Although the entrances to the Reactor Building are 0.5 ft. below the maximum LIP water elevations, no specific mitigative actions are necessary to protect the equipment from the LIP event. All doors to the Reactor Building require relatively tight seals to maintain the required negative pressure for secondary containment integrity. Water that would enter through the adjacent buildings that can communicate with Reactor Building entrances would seek alternate paths that are available and flow into the lower areas of the adjacent buildings prior to challenging Reactor Building entrances. Regardless, should any volume of water enter the Reactor Building, it would be bounded by the Internal Flooding event and those permanent features that have been implemented to protect equipment during that event.

5.1.4 Protection of the Cooling Tower Cell (Alternate Cooling Cell)

The curb around the Cooling Tower basin which provides the water inventory for the Alternate Cooling System is at Elevation 250.5 ft. The basin is currently inundated by CLB PMF water level of 252.5 ft. There are no actions required for the PMF event since no active components are within the basin. The additional 0.5 ft. of water for the LIP event will not require any new mitigative actions.

5.1.5 Protection of the Emergency Diesel Generators and Day Tanks

The Emergency Diesel Generators (EDG) are located in self-contained rooms within the Turbine Building. Entrance doors to each room are elevated on an 0.67 ft. curb which places the door sill at Elevation 253.17 ft., or 0.17 ft. above the maximum LIP water elevation on the West exterior side of the Turbine Building. In addition, water levels inside the Turbine Building would be limited to what could enter at closed doors resulting in minor ingress flowing through the building and gravitating toward the lower levels of the Condenser Bay and Feedwater Pump rooms. There are also floor and equipment drains in the EDG rooms that tie into a common drain line which is routed to an oil separator in a manhole (Manhole B) whose discharge is normally isolated, and the yard drains. Any water which might backflow into the EDG room through these drains would not exceed the maximum LIP water

elevation of 253.0 ft. There are no safe shutdown components in the EDG room that would be adversely affected until water in the room reaches approximately Elevation 253.63 ft. (1.13 ft. above the floor), thus the EDG's are protected from the maximum water level for the LIP event.

The Day Tank Rooms are also self-contained rooms within the Turbine Building. Although the doors to these rooms do not have a curb similar to the EDG rooms, there is no equipment located in the rooms that would be affected by the maximum water level for the LIP event of 253.0 ft., thus no additional protection is required. Should water ingress occur into the Day Tank Rooms, there are existing open floor penetrations that would convey any flow of water to lower levels of the Turbine Building as well.

5.1.6 Interim Actions Planned to Address the Local Intense Precipitation Event

Mitigation activities to address the Current Licensing Basis Probable Maximum Flood are applicable to the Local Intense Precipitation event. It is acknowledged that the LIP maximum water elevation that can have an effect on plant equipment exceeds the CLB PMF stillwater elevation by 0.5 ft., however the volume of water that could affect the equipment during the PMF far exceeds that of the LIP. In addition, for the CLB PMF event a wave runup of 1.5 ft. is postulated to occur and is added to the stillwater level yielding a total exterior flood water height to Elevation 254.0 ft. that is addressed by the current procedure and mitigative actions defined therein. Conservatisms utilized in the reevaluation for LIP such as not taking credit for the Vehicle Barrier System acting to provide some rerouting of the water should also result in lower water elevations at the plant structures than predicted by the evaluation. Similar reductions in the predicted water levels would be expected due to other conservatisms such as not crediting the stormwater drainage system or infiltration. The PMF precipitation event is proposed to last 72 hours while the LIP precipitation event lasts only 6 hours, with the time that the maximum water level exceeds typical plant access elevations being 2 hours. It is also acknowledged that warning for the LIP event is expected to be less than the 96 hours afforded by the CLB PMF, however a storm capable of producing the amount of rainfall resulting in the LIP would be expected to be predicted with at least 24-hour notice. Evaluations are currently being performed to determine the best methods for notice that a LIP event is probable (see Section 6). OPOP-PHEN-3127 will require revision to align current actions to the LIP event. The specific revision elements have not been determined at this time. However, when completed they will provide sufficient notice to enact the mitigative actions required without being overly conservative so as to warrant these actions to be prescribed unnecessarily or under conditions that the LIP event is underway. These actions should be completed by July 1, 2013.

5.2 Ice-Induced Events

The maximum floodwater elevation at the plant as the result of a break-up of an upstream ice dam is predicted to be Elevation 242.3 ft. This is significantly below the CLB Stillwater PMF level of 252.5 ft. However, the distance to the location of the ice jam is only a few miles upstream of the plant. This resulting river level would occur with much shorter notification than the 96 hours afforded for the CLB PMF. Ice-Induced Flooding is not included in the CLB; however, Ice Effects were briefly addressed as part of the Individual Plant Examination External Events (IPEEE). Only flooding as a result of a downstream ice dam (impoundment) was considered for the IPEEE.

OPOP-PHEN-3127 "Natural Phenomena" directs that before the Connecticut River level reaches El. 237.0 ft. MSL (which would result in a loss of the Service Water Pumps), to start the Alternate Cooling

System. OPOP-2181 "Service Water/Alternate Cooling Operating Procedure" states that Service Water inoperability can result from the following design basis events:

- Appendix R fire in the SW intake structure which disables all four SW pumps
- Loss of the Vernon Dam coincident with the loss of site electric power
- Flooding of the SW intake structure due to precipitation or upstream dam failure

OPOP-2181 also states that lineup to ACS should commence as soon as it is evident that SW cannot be restored. Approximately 90 to 120 minutes should be allowed for the lineup of ACS.

The determination of the Ice-Induced flood elevation at VYNPS, described in Section 3.7, is very conservative. It is assumed that: (1) there is no attenuation of the resulting 'wave' due to the failure of the ice jam, and (2) the failure of the ice jam occurs simultaneously across the entire width of the river. Given the conservative evaluation of the ice-induced flood and existing actions in place to address loss of the Service Water pumps, no additional actions are necessary to address the predicted flood level at the Intake Structure containing the pumps.

5.3 Dam Failures

The maximum floodwater elevation at the plant as a result of a Dam Failure is Elevation 252.0 ft. This is 0.5 ft. below the CLB PMF flood elevation of 252.5 ft. However, the notification time for the maximum water level at the plant to occur is less than that for the PMF. Dam failures are considered in the CLB, but the specific scenarios evaluated are not the same as those directed by NUREG/CR-7046.

The dam break described in Section 3.3 is assumed to occur during the PMF event. The maximum floodwater elevation for the dam break occurs approximately 85 hours after the initiation of the PMP storm, while the CLB PMF level peaks in approximately 96 hours (Figure 5.3-1). Even though this is a shorter response time, it is judged adequate to take all the same actions as directed for the PMF for this reevaluated scenario.

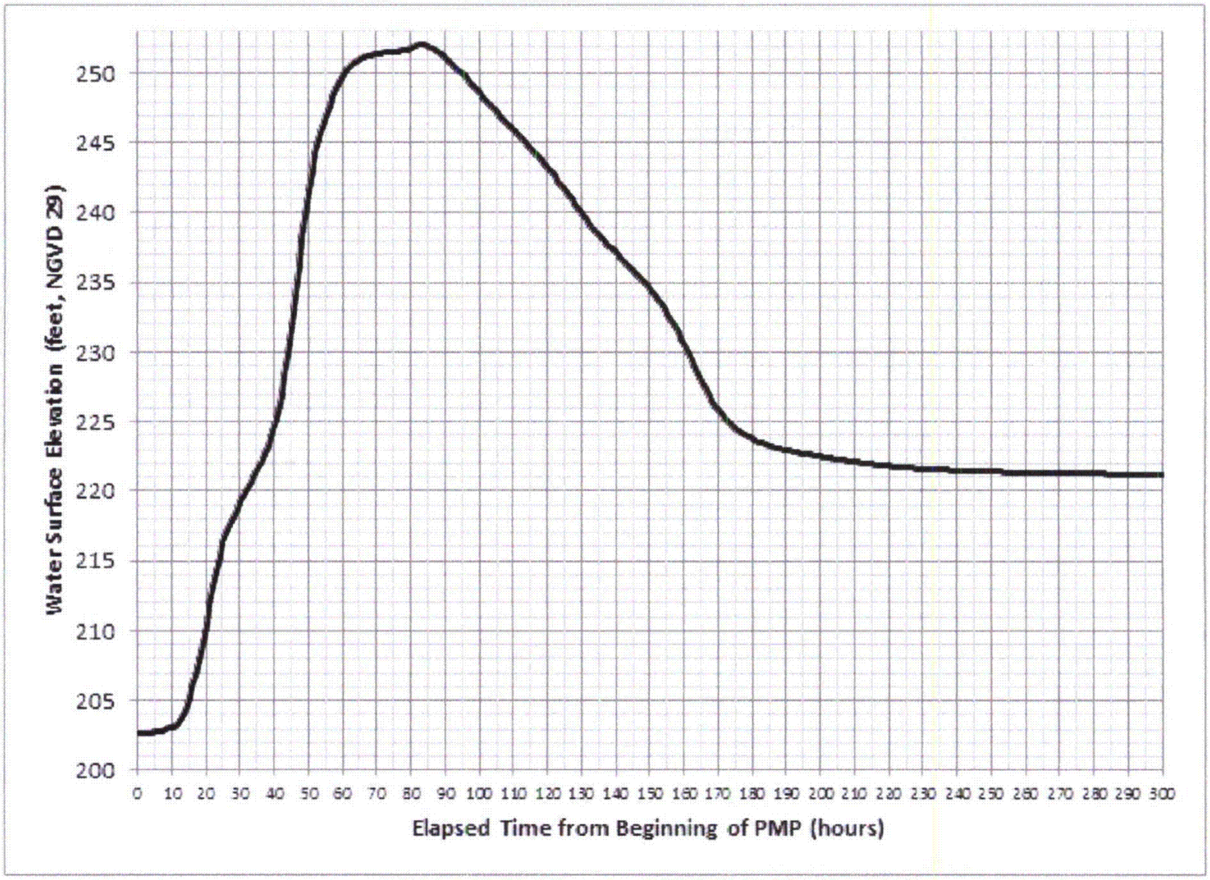


Figure 5.3-1: Stage Hydrograph for PMF Plus Dam Failure Flood Mechanism

6.0 ADDITIONAL ACTIONS

This section describes the additional actions taken to support the interim actions described in Section 5 relating to a LIP event.

Specifically, a LIP event could produce site flood levels onsite that are above the CLB PMF level of 252.5 ft., but below the CLB PMF plus wave activity level of 254 ft. Nevertheless, plant safety can be ensured by implementing operating procedure OPOP-PHEN-3127, which places sandbags and other barricades at strategic locations to protect SSCs important to safety up to elevation 254 ft. However, as noted in Section 4.1.1, the advance notice needed prior to implementing OPOP-PHEN-3127 is 96 hours, but the meteorological conditions that produce a LIP event (high-intensity, short-duration) are such that a determination of reasonable advanced notice is warranted.

Three additional actions are being taken to assist in determining the advanced notice needed and provide recommendations for revising operating procedure OPOP-PHEN-3127:

1) determine the threshold (i.e., minimum) LIP event that could produce on-site flood levels above 252.5 ft.,

2) identify the type of storms that produce a LIP event, and

3) determine the best method(s) to forecast an LIP event.

The goal of these actions is to integrate a forecast method into action level guidance to be used in operating procedure OPOP-PHEN-3127. The forecast method will provide adequate response time to enact the mitigative actions required, while not be overly conservative so as to warrant actions to be prescribed unnecessarily. The response time will also allow mitigative actions to be performed prior to the onset of the LIP event.

6.1 Threshold Precipitation Level

The LIP results in Section 3.1 show that peak water surface elevations vary on site, depending on location, with the maximum being 253.0 ft. The water surface elevations were due to a LIP event of 14.5 inches of rain in six hours centered directly over VYNPS. A lesser rainfall event will produce lower water surface elevations.

To determine the threshold, i.e., minimum, precipitation event that causes water surface elevations to exceed the plant protection level of 252.5 ft., a sensitivity analysis is being performed. The FLO-2D hydrodynamic model used in Section 4.1 is being applied to calculate the lowest total amount of precipitation, which is distributed in the same manner as the LIP per NUREG/CR-7046, Appendix B.

6.2 LIP Storm Type

Precipitation across the VYNPS basin shows very little seasonality, with consistent values spread evenly across most of the year. However, there are preferred months when PMP-type storms are most likely to occur.

The occurrence for the cool-season PMP are the months of March and April, when high levels of moisture begin to affect the region and interact with strong cold fronts. The cool-season PMP rainfall amounts are augmented by melting snowpack. This combination of cool-season PMP and melting snowpack produces a cool-season PMF scenario reflecting the importance of late winter/early spring rainfall on antecedent snowpack in the basin (AREVA 2013, Appendix J, Section1.1). A cool season PMP is not anticipated to generate an LIP event.

The occurrence for the all-season PMP rainfall is from late summer to early fall when tropical systems are most likely to affect the region and combine with frontal boundaries moving through the area (AREVA 2013, Appendix J, Section 2.2). The all-season PMP occurs when the largest amount of moisture is available for precipitation over the region. The major types of extreme precipitation events in the region are produced by thunderstorms often associated with mesoscale convective systems (short durations and small area sizes), synoptic events/fronts (large areas sizes and longer durations), and/or remnant tropical systems. Storms associated with mesoscale convective systems are potential initiators of LIP events (AREVA 2013, Appendix J, Section 2.3).

Thus, a LIP can be expected to be an all-season storm that occurs from May to November (AREVA 2013, Appendix J, Figure 2.5a).

6.3 Precipitation Forecast Method

The National Weather Service, Hydrometeorological Prediction Center, produces various guidance forecast products to assist weather and river forecast centers. Quantitative precipitation forecasts (QPFs) are particularly useful in determining when an LIP event might occur. The QPF provides rainfall over the continental U.S. for up to seven days at various intervals. These forecasts depict isohyets in varying increments of accumulated precipitation expected in each interval. (NOAA 2013)

The QPF can be used in operating procedure OPOP-PHEN-3127 the same way that river stage elevations are used to determine what actions to take and when. For example, Figure 6.3-1 illustrates a longer period forecast, a 5-day QPF, for the total precipitation estimated. The longer term QPF provides an awareness level as to the potential for a LIP event.

Similarly, an alert level requiring standby action could be taken using a shorter period QPF. Again, if the total precipitation estimated in three days is a certain percentage (to be determined) of the minimum LIP (Figure 6.3-2). Finally, protective action would be taken if, for instance, the total precipitation estimated in a one day QPF (Figure 6.3-3) is a larger percentage (to be determined) of the minimum LIP.

6.4 Conclusion

Completing the three actions will be used to ensure that there will be sufficient advance notice in the case of a LIP event to implement operating procedure OPOP-PHEN-3127. The forecast method will provide adequate response time to enact the mitigative actions required, while not be overly conservative so as to warrant actions to be prescribed unnecessarily. The response time will also allow mitigative actions to be performed prior to the onset of the LIP event.

6.5 References

AREVA 2013. AREVA Document No. 32-9196324-000, Vermont Yankee Nuclear Power Station Flood Hazard Re-evaluation - Probable Maximum Precipitation.

NOAA 2013. National Oceanic and Atmospheric Administration, National Weather Service, Hydrometeorological Prediction Center. Website: http://www.hpc.ncep.noaa.gov/; accessed February 19, 2013.

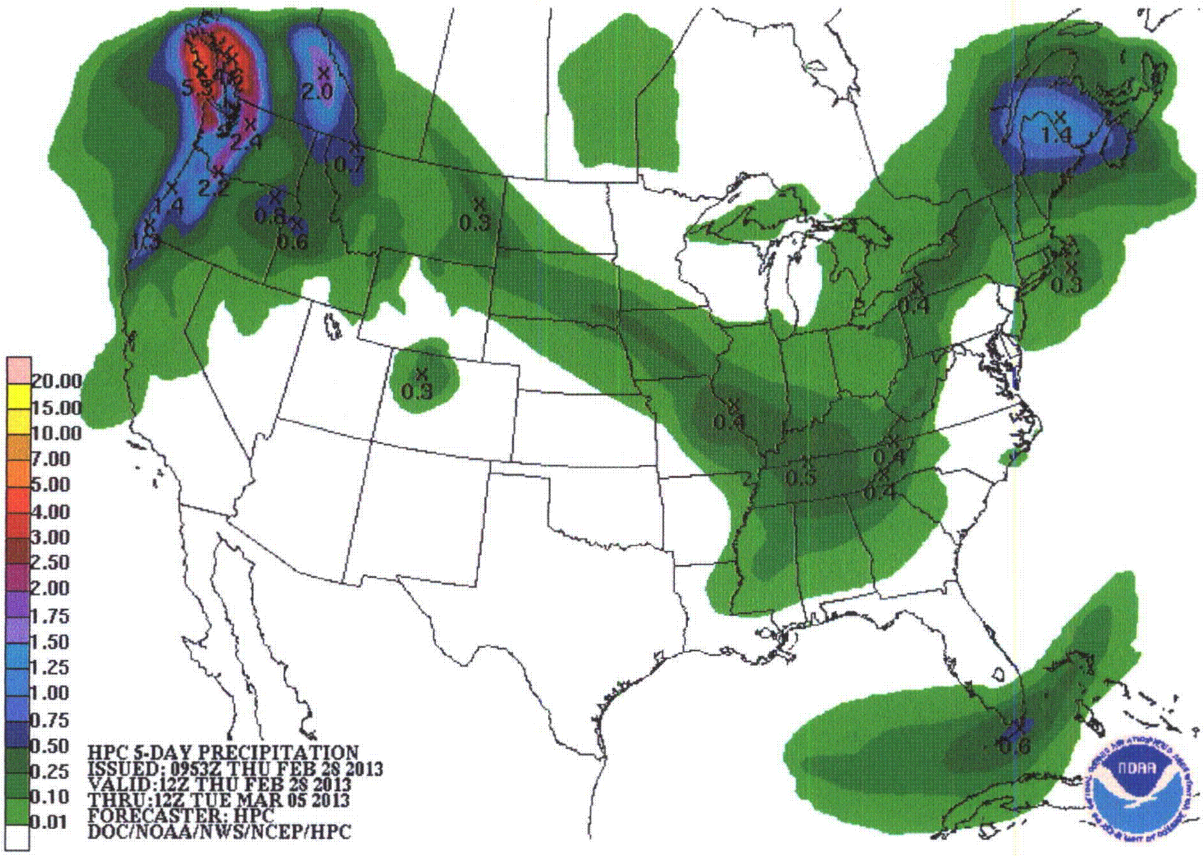


Figure 6.3-1 – Example 5-Day QPF

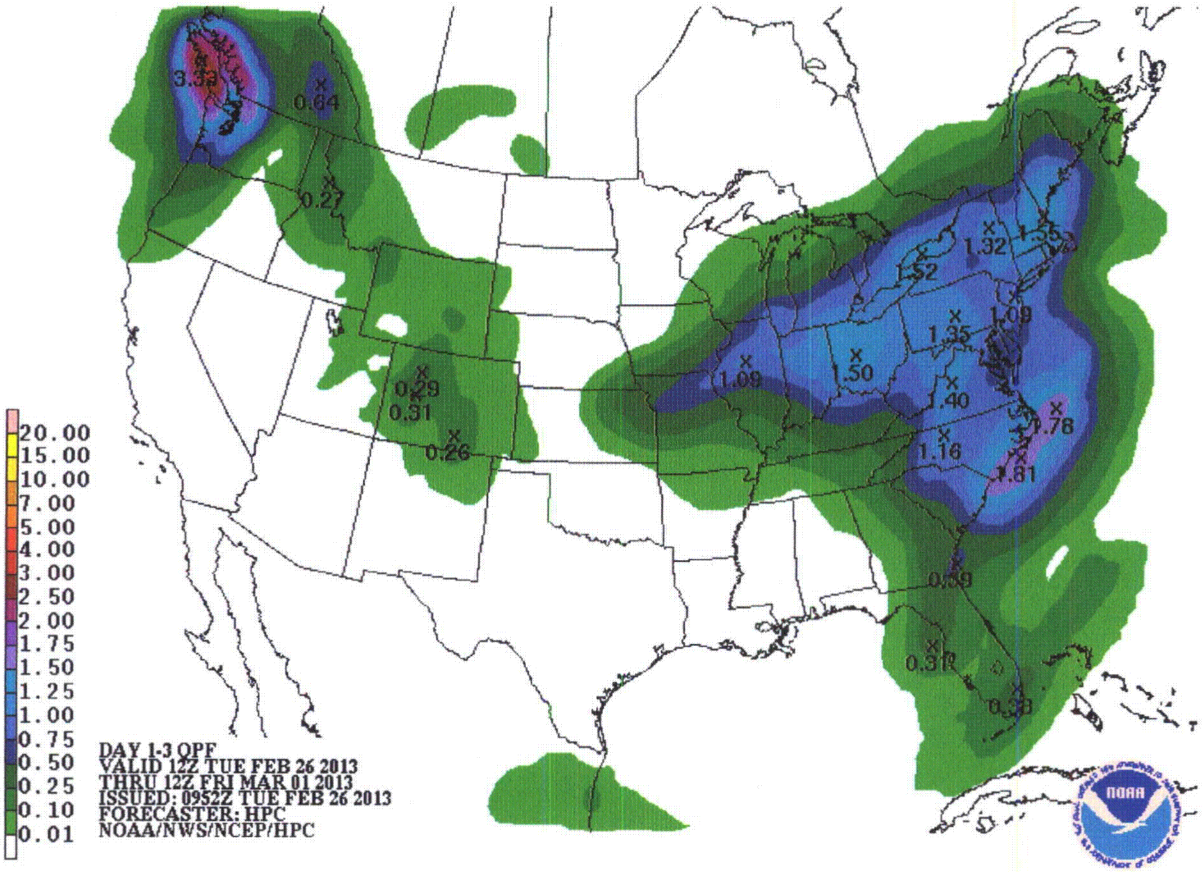


Figure 6.3-2 – Example 3-Day QPF

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Flood Hazard Reevaluation Report for Vermont Yankee Nuclear Power Station (VYNPS)

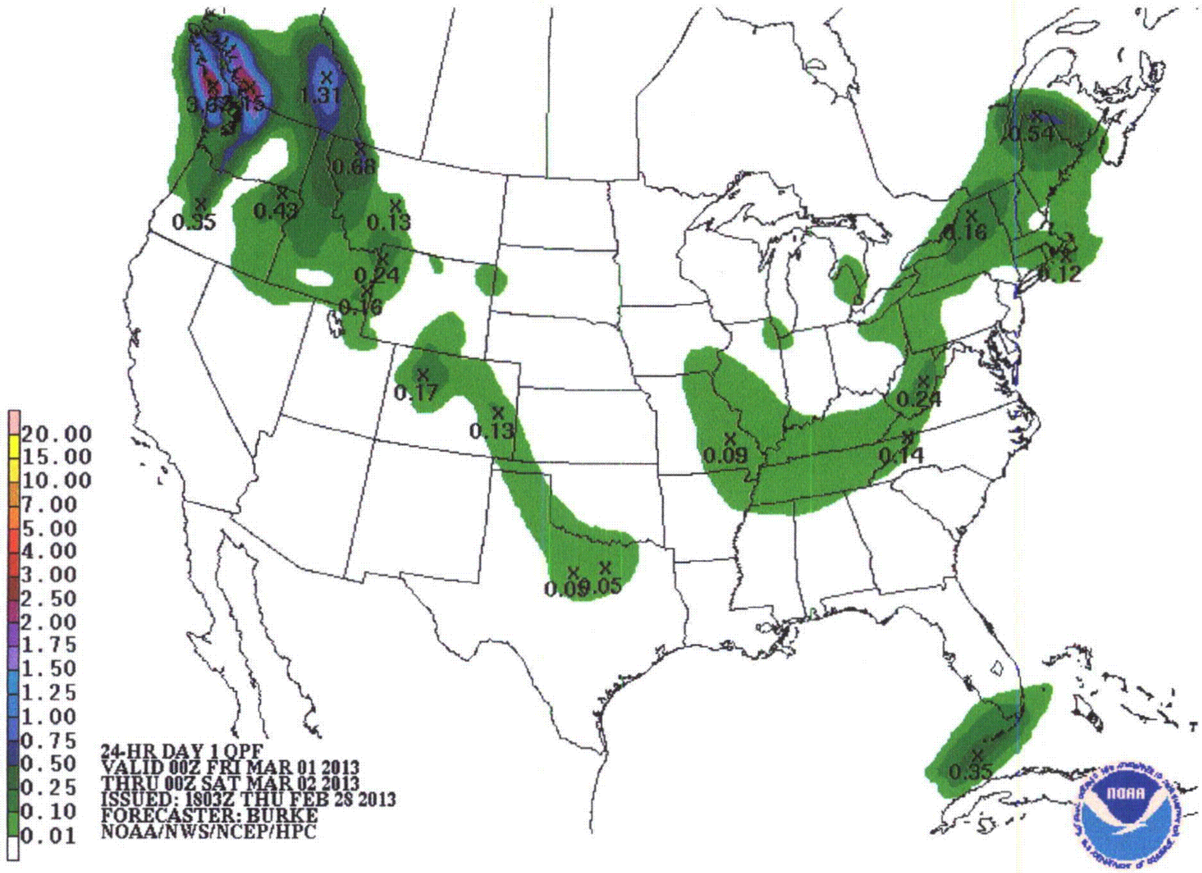

Figure 6.3-3 – Example 1-Day QPF

APPENDIX A: SIMULATION MODEL USE DESCRIPTION

This appendix was prepared as per Sections 5.3 and 5.5 of NUREG/CR-7046 (NRC 2011).

A.1 FLO-2D Computer Program – FLO-2D for LIP Simulations

The example LIP calculation presented in Appendix B of NUREG/CR-7046 (NRC 2011) used HEC-HMS and HEC-RAS, developed by Hydrologic Engineering Center of US Army Corps of Engineers. The hydrologic part of the calculation was performed within HEC-HMS, whereas the hydraulic part of the calculation was performed within HEC-RAS. In this flood re-evaluation study, FLO-2D was selected for calculation of the LIP-induced PMF at NMP and PMF in streams and rivers near NMP. For the LIP calculation, rainfall runoff was calculated internally by FLO-2D and translated into overland flow within FLO-2D.

This appendix was prepared as per Sections 5.3 and 5.5 of NUREG/CR-7046 (NRC 2011).

A.2 Software Capability

The FLO-2D computer program was developed by FLO-2D Software, Inc., Nutrioso, Arizona. FLO-2D is a combined two-dimensional hydrologic and hydraulic model that is designed to simulate river overbank flows as well as unconfined flows over complex topography and variable roughness, split channel flows, mud/debris flows and urban flooding.

FLO-2D is a physical process model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces using the dynamic wave approximation to the momentum equation. The model has components to simulate riverine flow including flow through culverts, street flow, buildings and obstructions, levees, sediment transport, spatially variable rainfall and infiltration and floodways. Application of the model requires knowledge of the site, the watershed (and coastal, as appropriate) setting, goals of the study, and engineering judgment. This software will be used to simulate the LIP, propagation of storm surge, seiches, and riverine flow through overland flow and channels to establish stillwater levels at various Flood Hazard Re-evaluation Project sites.

The major design inputs to the FLO-2D computer model are digital terrain model of the land surface, inflow hydrograph and/or rainfall data, Manning's roughness coefficient and Soil hydrologic properties such as the SCS curve number. The digital terrain model of the land surface is used in creating the elevation grid system over which flow is routed. The specific design inputs depend on the modeling purpose and the level of detail desired.

The following executable modules compose the FLO-2D computer program:

*.exe File	Size
FLO.exe	10.76 MB
GDS.exe	6.00 MB
PROFILES.exe	2.84 MB

*.exe File	Size
HYDROG.exe	2.07 MB
Mapper_2009.exe	3.33 MB
MAXPLOT.exe	2.32 MB

FLO.exe is the model code that performs the numerical algorithms for the aforementioned components of the overall FLO-2D computer model.

GDS.exe graphically creates and edits the FLO-2D grid system and attributes and creates the basic FLO-2D data files for rainfall – runoff and overland flow flood simulation. PROFILES.exe displays the channel slope and permits interactive adjustment of the channel properties. HYDROG.exe enables viewing of channel outputs hydrographs and lists average channel hydraulic data for various reaches of river. Mapper_2009.exe and Maxplot.exe enables graphical viewing of model results and inundation mapping.

A description of the major capabilities of FLO-2D which will be used for this project is provided in Section A.1.2 below.

A.3 Model Components

Overland Flow Simulation

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

Channel Flow Simulation

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

Flood Channel Interface

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

Floodplain Surface Storage Area Modification and Flow Obstruction

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

Rainfall – Runoff Simulation

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

Hydraulic Structures

Hydraulic structures including bridges and culverts and storm drains may be simulated in FLO-2D Pro. Discharge through round and rectangular culverts with potential for inlet and outlet control can be computed using equations based on experimental and theoretical results from the U.S. Department of Transportation procedures (Hydraulic Design of Highway Culverts; Publication Number FHWA-NHI-01-020 revised May, 2005).

Levees

This FLO-2D component confines flow on the floodplain surface by blocking one of the eight flow directions. A levee crest elevation can be assigned for each of the eight flow directions in a given grid element. The model predicts levee overtopping. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad-crested weir flow equation with a 2.85 coefficient. Weir flow occurs until the tailwater depth is 85% if the headwater depth. At higher flows, the water is exchanged across the levees using the difference in water surface elevations.

A.4 FLO-2D Model Theory

Governing equations and solution algorithm are presented in details in FLO-2D Reference Manual (FLO2D 2009). The general constitutive fluid equations include the continuity equation and the equation of motion (dynamic wave momentum equation) (FLO-2D 2009a, Chapter II):

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

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

where

h = flow depth;

V = depth averaged velocity in one of the eight flow directions;

x = one of the eight flow directions;

i = rainfall intensity;

S_f = friction slope based on Manning's equation;

S_0 = bed slope

g = acceleration of gravity

The partial differential equations are solved with a central finite difference numerical scheme, which implies that final results are just approximate solutions to the differential equations. Details on the accuracy of FLO-2D solutions are discussed in FLO-2D Validation Report (FLO-2D 2011).

A.5 Model Inputs and Outputs

Inputs to FLO-2D are entered through a graphical user interface (GUI), which creates ASCII text files used by the FLO-2D model (FLO-2D 2009b). The ASCII text files can be viewed and edited by other ASCII text editors such as MicroSoft WordPad.

Calculated results from FLO-2D simulations are saved in the ASCII text format in a number of individual files. The results can be viewed with the post-processor programs as follows:

- Mapper – to view grid element results such as elevation, water surface elevation, flow depth and velocity, to create contour maps and to generate shapefiles that can later be used by GIS mapping softwares such as ArcMap.
- MAXPLOT – to view grid element maximum flood elevation, flow depth, velocity, channel flow depth/elevation/velocity, and levee minimum free board/overtopping.

- HYDROG – to generate hydrographs for channel elements.
- PROFILES – to plot channel water surface and channel bed profiles.

A.6 Model Validation

As per Section 5.5 of NUREG/CR-7046 (NRC 2011), accuracy of computer models should be validated using site-specific data. Historical observed flood flow / elevation data at NMP is not available. In lieu of site-specific data, the validation of the FLO-2D software used two benchmark case studies presented in the FLO-2D model validation report (FLO-2D Inc., 2011). FLO-2D's model validation report has gained acceptance from a variety of federal, state, and local regulatory agencies and the model itself has been accepted by FEMA. Example 1, a simple flume model, was validated by comparing the results with a hand calculation as shown in Table A-1:

Table A-1: Comparison of Results – Example 1

METHOD OF COMPUTATION	FLO-2D v.2009.06	HAND CALCULATION
Flow Depth (ft.)	6.8	6.8
Velocity (ft./s)	5.9	5.9

Example 2 was a case study for the Truckee River performed by FLO-2D (FLO-2D 2011). The Truckee River FLO-2D model was originally created and calibrated by others to conduct a flood hazard delineation project for the Truckee River in response to recorded flooding of the Truckee River through Reno and the City of Sparks, Nevada between December 31, 1996 and January 6, 1997. The simulated results by FLO-2D were compared with observed USGS gage data during an actual storm. See Table A-2 and Figure A-1. Upon achieving the identical output results as presented in the FLO-2D model validation report, it was concluded that the model validation was completed to the extent practicable.

Table A-2: Comparison of Results – Example 2

	Node	Benchmark	GZA	% Difference
Maximum Flow Depths (ft.)	4936	0.1	0.1	0.0
	4937	6.25	6.25	0.0
	4938	9.09	9.09	0.0
	4968	0.1	0.1	0.0
	4969	4.62	4.62	0.0
	4970	10.75	10.75	0.0
	4998	4.32	4.32	0.0
	4999	9.02	9.02	0.0
	5000	8.77	8.77	0.0
	5022	3.99	3.99	0.0
	5023	3.98	3.98	0.0
	5024	7.08	7.08	0.0
Total Inflow and rainfall Volume (acres)		101028	101028	0.0
Total Outflow and Storage (acres)		101028	101028	0.0
Maximum Inundated Area (acres)		21125	21125	0.0

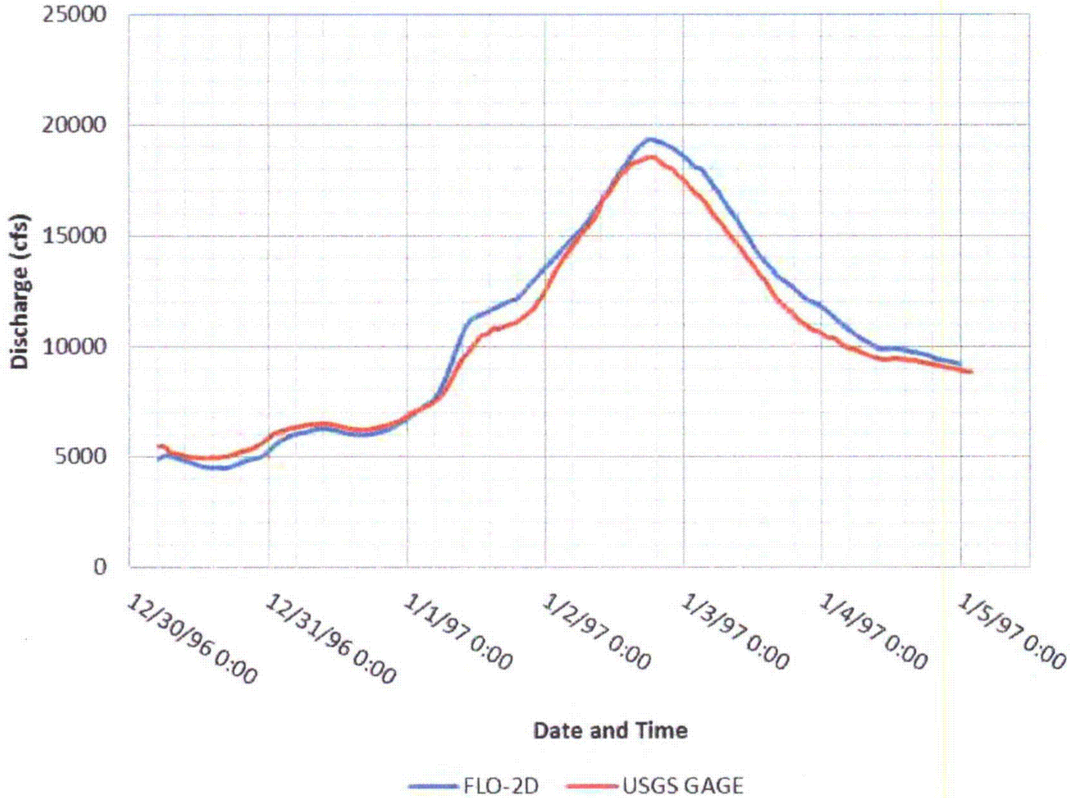

Figure A-1: Observed vs. Predicted Discharge for the 1997 Flood at Truckee River New Vista Gage

A.7 Conclusions

FLO-2D is a FEMA-approved software (FLO-2D 2011). The model validation report prepared for FEMA and the FLO-2D software certification prepared for Flood Re-evaluation Projects (AREVA 2012) have demonstrated its modeling capabilities and numerical accuracy. It is therefore judged to be an appropriate modeling tool for the NMP flood re-evaluation study where 2-dimensional overland flow is predominant.

A.8 References

AREVA 2012. AREVA Document No. 38-9191747-000, Computer Software Certification – FLO-2D v.2009.06, GZA GeoEnvironmental, Inc., October 2012.

FLO-2D 2009a. FLO-2D Reference Manual, FLO-2D Software, Inc.

FLO-2D 2009b. FLO-2D Data Input Manual, FLO-2D Software, Inc.

FLO-2D 2011. FLO-2D Model Validation for Version 2009 and up prepared for FEMA, FLO-2D Software, Inc., June 2011.

NRC 2011. NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. Nuclear Regulatory Commission, November 2011 (ADAMS Accession No. ML11321A195).

USACE 2000. HEC-HMS Hydrologic Modeling System Technical Reference Manual, HEC, USACE, March 2000.

USACE 2010. HEC-RAS River Analysis System Hydraulic Reference Manual, HEC, USACE, January 2010.

ATTACHMENT 2

BVY 13-021

LIST OF REGULATORY COMMITMENTS

List of Regulatory Commitments

The following table identifies those actions committed to by Entergy in this document. Any other statements in this submittal are provided for information purposes and are not considered to be regulatory commitments.

COMMITMENT	TYPE (Check One)		SCHEDULED COMPLETION DATE (If Required)
	ONE-TIME ACTION	CONTINUING COMPLIANCE	
Complete evaluation to determine the entry conditions for a probable Local Intense Precipitation (LIP) event. Implement the guidance and actions in site procedures.	✓		July 1, 2013