ENCLOSURE TO NL-14-148

ENTERGY FLEET FUKUSHIMA PROGRAM FLOOD HAZARD REEVALUATION REPORT FOR INDIAN POINT ENERGY CENTER (IPEC) UNITS 2 AND 3, REVISION 2

ENTERGY NUCLEAR OPERATIONS, INC.
INDIAN POINT NUCLEAR GENERATING UNIT NOS. 2 AND 3
DOCKET NOS. 50-247 AND 50-286

AREVA Inc.

Engineering Information Record

Document No.: 51 - 9195289 - 002

Entergy Fleet Fukushima Program
Flood Hazard Reevaluation Report for Indian Point Energy Center (IPEC) Units 2 and 3

AREVA

20004-021 (01/30/2014)
Document No.: 51-9195289-002

Entergy Fleet Fukushima Program
Flood Hazard Reevaluation Report for Indian Point Energy Center (IPEC) Units 2 and 3

Safety Related? ☒ YES ☐ NO

Does this document establish design or technical requirements? ☐ YES ☒ NO

Does this document contain assumptions requiring verification? ☐ YES ☒ NO

Does this document contain Customer Required Format? ☐ YES ☒ NO

Signature Block

Name and Title/Discipline	Signature	P/LP, R/LR, A-CRF, A	Date	Pages/Sections Prepared/Reviewed/ Approved or Comments
F. X. Bellini Advisory Scientist	F X Bellini	LP	5/1/14	Section headers in 3.0; Figure 3.1-2; Added text to section 6.0 (see ROR)
C.A. Fasano Engineering Supervisor	CA Fasano	LR	5/1/14	Section headers in 3.0; Figure 3.1-2; Added text to section 6.0 (see ROR)
Mark A. Rinckel Technical Manager, Radiological & Environmental Analysis	MA Rinckel	A	5/2/14	Section headers in 3.0; Figure 3.1-2; Added text to section 6.0 (see ROR)

Note: P/LP designates Preparer (P), Lead Preparer (LP)
R/LR designates Reviewer (R), Lead Reviewer (LR)
A-CRF designates Project Manager Approver of Customer Required Format (A-CRF)
A designates Approver/RTM – Verification of Reviewer Independence

Project Manager Approval of Customer References (N/A if not applicable)

Name (printed or typed)	Title (printed or typed)	Signature	Date
N/A			

Record of Revision

Revision No.	Pages/Sections/ Paragraphs Changed	Brief Description / Change Authorization
000	All	Initial issue.
001	Section 1.3	Corrected "SCC" to "SSC"
	Section 1.6	Corrected title of USACE, 2007 reference
	Section 2.1.2	Added "to the dam at Troy (north of Albany, NY)" to the second paragraph
	Section 2.3.2.1	Spelled out LIP and added reference to LIP Section 3.1
	Section 2.4	Added "Rev. 2" to procedure title and "are" to last sentence.
	Section 3.1.2.1.2	Spelled out PMP
	Section 3.1.3	Corrected "PMF" to "LIP"
	Section 3.2.2.1.2	Added "AREVA, 2013c" to end of 1st paragraph
	Section 3.2.2.1.10	**Replaced reference to Figure 4 with reference to AREVA, 2013a.**
	Section 3.2.4	Added AREVA, 2013c (PMP) and ANS, 1992 to reference section
	Section 3.3.1	Corrected "NRC, 2013a" to NRC, 2013
	Section 3.3.4	Added "NRC, 2013" (ISG for Dam Failure) and "ASCE, 2010" to references
	Section 3.4.1, 2nd and 4th para	Added "NUREG/CR-7134" and "AREVA, 2013d" and "AREVA, 2013e" to 2nd para and corrected "USACERC, 1991" to "USACE, 1992" in 4th para.
	Section 3.4.1, 5th paragraph, Item 2.	Added "site and region specific hurricane meteorological study" for better description
	Section 3.4.2.1	Corrected "1955" to "1944" for date of storms referred to in Table 3.4-2.
	Section 3.4.2.2	Corrected reference in eighth para. from "NOAA, 2013a" to "NOAA, 2012c". After ninth para. added new paragraph describing synthetic storm process.
	Section 3.4.2.5.1, 4th para	Corrected NOAA, 2013j to NOAA, 2013a
	Section 3.4.2.5.2	Corrected values in table to match AREVA, 2013b, AREVA Document No. 32-9213356-000, "Flood Hazard Re-evaluation - Combined Effect Floods – Coastal Processes for Indian Point Energy Center" and Section 4.1.9. Note that all corrected values are lower elevations than Rev. 000.
	Section 3.4.2.6.1, 2nd paragraph, 1st and 2nd bullets	Corrected reference "AREVA, 2013c" to "AREVA, 2013a" and replaced "cyclone" with "storms and hurricanes"
	Section 3.4.2.6.1, 2nd paragraph, 1st bullet	In sentence containing…"to a small number of discrete elements" (corrected "AREVA, 2013c" to "AREVA, 2013e")".
	Section 3.4.2.6.1, last bullet of "Part 1"	Added the return period equation for completeness.

Record of Revision
(continued)

Revision No.	Pages/Sections/ Paragraphs Changed	Brief Description / Change Authorization
	Section 3.4.2.6.1, Part II, 2nd paragraph	Corrected Reference NOAA, 2011 to AREVA, 2013b
	Section 3.4.2.6.2,	Corrected values in Scenario Probability table to match AREVA, 2013b, AREVA Document No. 32-9213356-000, "Flood Hazard Re-evaluation - Combined Effect Floods – Coastal Processes for Indian Point Energy Center" and Section 4.1.9. Note that all corrected values are lower elevations than in Rev. 000. Deleted sentence beginning with "Wave setup from deep water..." in twelfth paragraph Deleted "probabilistic" in fourteenth paragraph Corrected values in hydrodynamic loading section (third paragraph to match AREVA, 2013b. Note that all corrected values are lower than in Rev. 000.
	Section 3.4.3, 2nd paragraph	Added "stillwater" to before "results" and removed last sentence regarding deterministic results.
	Section 3.4.4	Added reference for AREVA, 2013d; AREVA, 2013e; FEMA, 2005; NOAA, 2013e; USACE, 1992; USACE, 2002; USDOI, 1982 and USGS, 2013. Deleted Reference NRC, 2013a. Corrected internet links for NOAA 2012c and NOAA 2013b.
	Section 3.4 tables/figures	Added references to figures and tables
	Figure 3.4-15	Added graphic for Figure 3.4-15
	Figures 3.4-47 and 3.4-51	Replaced figures with correct figures from AREVA, 2013e AREVA Document No. 32-9213352-000, "Flood Hazard Re-evaluation - Probabilistic Storm Surge for Indian Point Energy Center".
	Sections 3.5.2.3 and 3.5.4	Corrected citations (editorial) for NOAA, 2005a-d.
	Fig. 3.5.5, caption	Corrected reference typo
	Fig. 3.5.6, caption	Changed FFT to Fast Fourier Transform
	Table 3.5-4	Inserted Table 3.5-4
	Section 3.6.2.2	Replaced text with text from "Indian Point Energy Center Flood Hazard Re-evaluation – Tsunami Screening Analysis", AREVA Document No. 51-9196310-000, 2013
	Section 3.6.3	Deleted the last bullet of section: "The results of a simple, unsteady hydraulic analysis indicated that "wave" amplitudes will likely be significantly attenuated before reaching IPEC, to levels that would not impact IPEC SSCs".
	Section 3.6.4	Removed ref. AREVA, 2013b
	Section 3.9.4	Corrected AREVA, 2013d AREVA document # to 32-9213356-000.
	Section 5.1.6, 1st sentence	Added Figures "3.1-2 and 3.1-3"
	Section 5.2	Added NOAA, 2013 reference

Record of Revision
(continued)

Revision No.	Pages/Sections/ Paragraphs Changed	Brief Description / Change Authorization
	Various	Rev. 001 is in response to AREVA CR 2014-2418 and consists of the above revisions as well as minor editorial changes.
002	Various subheadings in Section 3	Heading title text **bolded.**
	Figure 3.1-2	Plant elevation line corrected to 18'
	Section 6.0	At IPEC's request, these changes were made: Due to changes in the protection approach for the conduits that come from manholes in the transformer yard and turbine buildings that communicate with the 480V switchgear rooms, the following changes should be made to Section 6.0: The first sentence of the 2nd paragraph should be changed to read as follows: To enhance the current protection measures and provide additional protection against normal seepage through sandbag walls and the potential of some groundwater intrusion into manholes, conduits that communicate with the 480V switchgear rooms through manholes 23 & 33 will be sealed. These conduits will be sealed in the manholes with sealant............ Add the paragraph below: Manholes 24 & 34, which are located in the transformer yards and have conduits that communicate with the 480V switchgear rooms, are not subject to the storm surge event, but are subject to ponding caused by the LIP. To prevent water due to the potential ponding from entering these manholes, these manholes will be raised approximately 18". All modifications to these manholes will be performed in accordance to Entergy procedures governing the modification process.

Overview

This report describes the approach, methods, and results from the reevaluation of flood hazards for Units 2 and 3 at the Indian Point Energy Center (IPEC). It provides the information, in part, requested by the U.S. Nuclear Regulatory Commission (NRC) to support the evaluation of the NRC staff Recommendation 2.1 for the Near-Term Task Force (NTTF) review of the accident at the Fukushima Dai-chi nuclear facility.

Section 1 provides information related to the flood hazard. The section begins with an introduction that includes background information, scope, general method used for the reevaluation, the vertical datum used throughout the report, and a conversion table to determine elevations in other common datums.

Section 2 describes detailed IPEC site information, including present-day site layout, topography, and current licensing basis flood protection and mitigation features. The section concludes by identifying 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 including a combined effect flood. In cases where a mechanism does not apply to the IPEC 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.

Section 5 presents mitigation measures necessary to address new flooding elevations developed by this evaluation.

Section 6 outlines additional plant actions that relate to flooding.

The report also contains one appendix. Appendix A describes the software models used in the reevaluation that are not specifically addressed by NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America", including the quality assurance criteria and a discussion of validation of model-derived results.

Table of Contents

Page

SIGNATURE BLOCK 2
RECORD OF REVISION 3
OVERVIEW 6
LIST OF TABLES 10
LIST OF FIGURES 12
ACRONYMS AND ABBREVIATIONS 17
1.0 INTRODUCTION 1-1
1.1 Purpose 1-1
1.2 Scope 1-1
1.3 Method 1-1
1.4 Assumptions 1-2
1.5 Elevation Datums 1-2
1.6 References 1-2
2.0 INFORMATION RELATED TO THE FLOOD HAZARD 2-1
2.1 Detailed Site Information 2-1
2.1.1 Present-day Site Layout 2-1
2.1.2 Site Topography and River Setting 2-1
2.2 Current/Licensing Design Basis Flood Elevations 2-5
2.3 Current Licensing Basis Flood Protection and Mitigation Features 2-5
2.3.1 Flooding Mechanisms 2-5
2.3.2 Flood Protection and Mitigation 2-6
2.4 Licensing Basis Flood-Related and Flood Protection Changes 2-7
2.5 Watershed and Local Area Changes 2-7
2.5.1 General IPEC Site Hydrological Description 2-7
2.5.2 Watershed Changes 2-7
2.5.3 Site Changes 2-7
2.6 Additional Site Details 2-7
2.7 References 2-7
3.0 INFORMATION RELATED TO THE FLOOD HAZARD 3-9
3.1 Local Intense Precipitation 3-9
3.1.1 Method 3-9
3.1.2 Results 3-9
3.1.3 Conclusions 3-12
3.1.4 References 3-12

Table of Contents
(continued)

		Page
3.2	Flooding in Rivers and Streams	3-17
3.2.1	Method	3-17
3.2.2	Results	3-18
3.2.3	Conclusions	3-20
3.2.4	References	3-20
3.3	Dam Breaches and Failures	3-26
3.3.1	Method	3-26
3.3.2	Results	3-27
3.3.3	Conclusions	3-28
3.3.4	References	3-29
3.4	Probable Maximum Storm Surge	3-34
3.4.1	Methodology	3-34
3.4.2	Results	3-36
3.4.3	Conclusions	3-53
3.4.4	References	3-54
3.5	Seiche	3-114
3.5.1	Method	3-114
3.5.2	Results	3-116
3.5.3	Conclusions	3-118
3.5.4	References	3-119
3.6	Tsunami	3-136
3.6.1	Method	3-136
3.6.2	Results	3-137
3.6.3	Conclusions	3-142
3.6.4	References	3-143
3.7	Ice-Induced Flooding	3-153
3.7.1	Method	3-153
3.7.2	Results	3-154
3.7.3	Conclusions	3-154
3.7.4	References	3-155
3.8	Channel Migration or Diversion	3-156
3.8.1	Method	3-156
3.8.2	Results	3-156
3.8.3	Conclusions	3-157
3.8.4	References	3-157

Table of Contents
(continued)

Page

3.9 Combined-Effect Flood ... 3-159
3.9.1 Method ... 3-159
3.9.2 Results ... 3-160
3.9.3 Conclusions ... 3-160
3.9.4 References ... 3-161
4.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS ... 4-1
4.1 Summary of Current Licensing Basis and Flood Reevaluation Results ... 4-1
4.1.1 Local Intense Precipitation ... 4-1
4.1.2 Flooding on Rivers and Streams ... 4-1
4.1.3 Dam Breaches and Failures ... 4-2
4.1.4 Storm Surge ... 4-2
4.1.5 Seiche ... 4-3
4.1.6 Tsunami ... 4-3
4.1.7 Ice-Induced Flooding ... 4-3
4.1.8 Channel Migration or Diversion ... 4-3
4.1.9 Combined Effect Flooding ... 4-4
4.2 Summary of Walkdown Findings ... 4-5
4.2.1 IPEC Unit 2 Walkdown Findings (IPEC, 2012a) ... 4-5
4.2.2 IPEC Unit 3 Walkdown Findings (IPEC, 2012b) ... 4-5
4.3 Impacts of Flood Elevations ... 4-6
4.4 References ... 4-7
5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED ... 5-1
5.1 Storm Surge and Local Intense Precipitation ... 5-1
5.1.1 Intake Structure ... 5-2
5.1.2 Protection of the Unit 2 and Unit 3 480V Switchgear Rooms ... 5-3
5.1.3 Protection of MCC 24A & 34A ... 5-4
5.1.4 Protection of Unit 3 EDG Diesel Cells ... 5-4
5.1.5 Protection of Unit 2 & 3 Service water Valve Pit ... 5-5
5.1.6 Flood Warning Times and Durations ... 5-5
5.2 Reference ... 5-5
6.0 ADDITIONAL ACTIONS ... 6-1
APPENDIX A : ADDITIONAL SIMULATION MODEL USE DESCRIPTIONS ... A-1

List of Tables

Page

TABLE 3.2-1: VERIFIED SUBWATERSHED PARAMETERS 3-23
TABLE 3.2-2: VERIFIED REACH MUSKINGUM ROUTING PARAMETERS 3-23
TABLE 3.2-3: PMF SIMULATIONS 3-24
TABLE 3.3-1: DAM BREACH PARAMETERS (AREVA, 2013) 3-30
TABLE 3.4-1: THE BATTERY, NY CO-OP STATION 8518750 TOP 20 EXTREME WATER LEVELS (AREVA, 2013A) 3-57
TABLE 3.4-2: SANDY HOOK, NJ CO-OP STATION 8531680 TOP 20 EXTREME WATER LEVELS (AREVA, 2013A) 3-58
TABLE 3.4-3: SLOSH MOMS AT THE BATTERY, NY AND SANDY HOOK, NJ (AREVA, 2013A) 3-58
TABLE 3.4-4: ADCIRC SIMULATED PROBABLE MAXIMUM STORM SURGES (AREVA, 2013D) 3-59
TABLE 3.4-5: DISCRETIZED PROBABILITIES FOR CENTRAL PRESSURE DEFICIT (CPD) (AREVA, 2013E) 3-60
TABLE 3.4-6: DISCRETIZED PROBABILITIES FOR FORWARD DIRECTION (Θ) (AREVA, 2013E) 3-60
TABLE 3.4-7: DISCRETIZED PROBABILITIES FOR FORWARD SPEED (VF) (AREVA, 2013E) ... 3-61
TABLE 3.4-8: DISCRETIZED PROBABILITIES FOR RADIUS OF MAXIMUM WINDS (R_{MAX}) (AREVA, 2013E) 3-61
TABLE 3.4-9: FACTOR OF LANDFALL PROBABILITY R_{MAX}/L FOR TRACK OFFSETS (AREVA, 2013E) 3-62
TABLE 3.4-10: TOP 10 JPM SLOSH-CALCULATED STORM SURGE EVENTS (DETERMINED BY SIMULATED SURGE MAGNITUDE AT IPEC) (AREVA, 2013E) 3-63
TABLE 3.4-11: TOP 10 JPM SLOSH-CALCULATED STORM SURGE EVENTS (DETERMINED BY SIMULATED SURGE MAGNITUDE AT THE BATTERY) (AREVA, 2013E) 3-64
TABLE 3.4-12: ADCIRC SIMULATED STORM SURGE STILLWATER ELEVATIONS (AREVA, 2013E) 3-65
TABLE 3.5-1: SEICHE PARAMETERS FOR THE HUDSON RIVER 3-120
TABLE 3.5-2: SEICHE PARAMETERS FOR THE IPEC DISCHARGE CANAL 3-121
TABLE 3.5-3: TIDAL CONSTITUENT AT THREE SITES ON THE HUDSON RIVER FROM NOAA 3-122
TABLE 3.5-4: RATIO OF TIDAL CONSTITUENTS ON THE HUDSON RIVER TO SANDY HOOK, NJ FROM NOAA 3-123

List of Tables
(continued)

Page

TABLE 3.6-1: HISTORIC TSUNAMI RUN-UP EVENTS ALONG THE U.S. EAST COAST 3-145
TABLE 3.9-1: COMBINED EFFECT FLOOD SCENARIO SUMMARY RESULTS 3-162
TABLE 4.1-1: FLOOD ELEVATION COMPARISONS 4-8
TABLE 4-2: UNIT 2 TRANSFORMER YARD DOORWAY LIP FLOOD IMPACTS 4-10

List of Figures

Page

FIGURE 2.1-1: IPEC LOCATION MAP 2-2

FIGURE 2.1-2: SITE TOPOGRAPHY AND LAYOUT 2-3

FIGURE 2.1-3: HUDSON RIVER FROM IPEC TO THE RIVER MOUTH 2-4

FIGURE 3.1-1: FLO-2D COMPUTATIONAL BOUNDARY, OUTFLOW NODES, BUILDING ELEMENTS AND CHANNELS (AREVA, 2013A) 3-14

FIGURE 3.1-2: LIP HYDROGRAPH FOR UNIT 2 TRANSFORMER YARD DOOR (AREVA, 2013A) 3-15

FIGURE 3.1-3: LIP HYDROGRAPH FOR UNIT 3 TRANSFORMER YARD DOOR (AREVA, 2013A) 3-16

FIGURE 3.2-1: IPEC SUBWATERSHEDS (AREVA, 2013A) 3-25

FIGURE 3.3-1: DAM LOCATIONS (AREVA, 2013) 3-31

FIGURE 3.3-2: DOMINO DAM LOCATIONS (AREVA, 2013) 3-32

FIGURE 3.3-3: PMF SCENARIO FORCED CONKLINGVILLE DAM FAILURE OUTFLOW HYDROGRAPH (AREVA, 2013) 3-33

FIGURE 3.3-4: SEISMIC SCENARIO OUTFLOW HYDROGRAPH (AREVA, 2013) 3-33

FIGURE 3.4-1: LOCATION OF NOAA CO-OP STATIONS IN THE VICINITY OF IPEC (AREVA, 2013A) 3-67

FIGURE 3.4-2: HURRICANE STRIKES TO NEW YORK AND NEW JERSEY (AREVA, 2013A) 3-68

FIGURE 3.4-5: NEW YORK, VERSION 3 (NY3) BASIN DIGITAL ELEVATION MODEL (DEM) DEFINING BASE OF MODEL DOMAIN (AREVA, 2013D) 3-71

FIGURE 3.4-6: NEW YORK, VERSION 3 (NY3) BASIN DIGITAL ELEVATION MODEL (DEM) DEFINING BASE OF MODEL DOMAIN IN IPEC VICINITY (UPPER BAY, NEW YORK AND HUDSON RIVER) (AREVA, 2013D) 3-72

FIGURE 3.4-7: NEW YORK, VERSION 3 (NY3) GRID CELLS CORRESPONDING TO THE IPEC RIVER FRONTAGE (AREVA, 2013D) 3-73

FIGURE 3.4-8: LANDFALL POINTS FOR SLOSH 3.97 STORM TRACKS (AREVA, 2013D) 3-74

FIGURE 3.4-9: SLOSH 3.97 BEARING RANGE FOR NORTHERLY STORM TRACKS (AREVA, 2013D) 3-75

FIGURE 3.4-10: SLOSH 3.97 BEARING RANGE FOR WESTERLY STORM TRACKS (AREVA, 2013D) 3-76

FIGURE 3.4-11: ADCIRC FEMA REGION II FINITE ELEMENT MESH – NORTHERN ATLANTIC (AREVA, 2013D) 3-77

FIGURE 3.4-12: ADCIRC FEMA REGION II FINITE ELEMENT MESH – NEW JERSEY/NEW YORK (AREVA, 2013D) 3-78

List of Figures
(continued)

FIGURE 3.4-13: ADCIRC FEMA REGION II FINITE ELEMENT MESH – IPEC VICINITY (AREVA, 2013D) ... 3-79

FIGURE 3.4-14: ADCIRC MODULE MESH ELEVATION (M, NAVD88) – NORTHERN ATLANTIC (AREVA, 2013D) ... 3-80

FIGURE 3.4-16: ADCIRC MESH MODEL ELEVATION (M, NAVD88) – IPEC VICINITY (AREVA, 2013D) ... 3-82

FIGURE 3.4-17: TRACK DIRECTIONS AND LANDFALL LOCATIONS FOR ADCIRC SIMULATIONS (AREVA, 2013D) ... 3-83

FIGURE 3.4-18: ADCIRC ENVELOPE OF MAXIMUM WINDS (M/S) OF STORM NO. 941 (AREVA, 2013D) ... 3-84

FIGURE 3.4-19: ADCIRC ENVELOPE OF MAXIMUM WINDS (M/S) OF STORM NO. 985* (AREVA, 2013D) ... 3-85

FIGURE 3.4-20: ADCIRC MAXIMUM STORM TIDE STILLWATER ELEVATIONS (M, NAVD88) OF STORM NO. 941 (AREVA, 2013D) ... 3-86

FIGURE 3.4-21: ADCIRC MAXIMUM STORM TIDE STILLWATER ELEVATIONS (M, NAVD88) OF STORM NO. 941 – IPEC VICINITY (AREVA, 2013D) ... 3-87

FIGURE 3.4-22: ADCIRC MAXIMUM STORM TIDE STILLWATER ELEVATIONS (M, NAVD88) OF STORM NO. 985*(AREVA, 2013D) ... 3-88

FIGURE 3.4-23: MAXIMUM STORM TIDE STILLWATER ELEVATIONS (M, NAVD88) OF STORM NO. 985* - IPEC VICINITY (AREVA, 2013D) ... 3-89

FIGURE 3.4-24: COMPARISON OF WIND FIELD OF ADCIRC AND SLOSH MODELS – STORM NO. 941 (AREVA, 2013D) ... 3-90

FIGURE 3.4-25: COMPARISON OF WIND FIELD OF ADCIRC AND SLOSH MODELS – STORM NO. 985* (AREVA, 2013D) ... 3-91

FIGURE 3.4-26: COMPARISON OF WIND PROFILES OF ADCIRC AND SLOSH MODELS – STORM NO. 941 ... 3-92

FIGURE 3.4-27: COMPARISON OF WIND PROFILES OF ADCIRC AND SLOSH MODELS – STORM NO. 985* (AREVA, 2013D) ... 3-92

FIGURE 3.4-28: COMPARISON OF WATER LEVELS ON HUDSON RIVER OF ADCIRC AND SLOSH MODELS – STORM 941 (AREVA, 2013D) ... 3-93

FIGURE 3.4-29: COMPARISON OF WATER LEVELS ON HUDSON RIVER OF ADCIRC AND SLOSH MODELS – STORM 985* (AREVA, 2013D) ... 3-94

FIGURE 3.4-30: TIME SERIES OF WIND AND STORM TIDE STILLWATER ELEVATION – STORM NO. 941 ... 3-95

FIGURE 3.4-31: TIME SERIES OF WIND AND STORM TIDE STILLWATER ELEVATION – STORM NO. 985* ... 3-96

List of Figures
(continued)

FIGURE 3.4-32: EXAMPLE STORM TRACKS (Θ=-40 DEGREE BEARING) WITH OFFSET CALCULATION (OFFSET IDS INDICATED) FROM BASE TRACK (AREVA, 2013E) 3-97

FIGURE 3.4-33: LANDFALL POINTS FOR THE JPM SYNTHETIC STORMS SHOWN BY ASSOCIATED STORM TRACK BEARING (Θ) IN DEGREES (AREVA, 2013E) 3-98

FIGURE 3.4-34: STORM TRACKS (R_{MAX} = 16 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-99

FIGURE 3.4-35: STORM TRACKS (R_{MAX} = 20 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-100

FIGURE 3.4-36: STORM TRACKS (R_{MAX} = 24 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-101

FIGURE 3.4-37: STORM TRACKS (R_{MAX} = 28 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-102

FIGURE 3.4-38: STORM TRACKS (R_{MAX} = 32 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-103

FIGURE 3.4-39: STORM TRACKS (R_{MAX} = 36 NAUTICAL MILES) FOR THE JPM SYNTHETIC STORMS (AREVA, 2013E) 3-104

FIGURE 3.4-40: TRACKS FOR TOP 10 SLOSH-CALCULATED SURGE EVENTS AT IPEC SHOWN BY NUMERIC STORM IDENTIFICATION AND SURGE RANKING (AREVA, 2013E) 3-105

FIGURE 3.4-41: TRACKS FOR TOP 10 SLOSH-CALCULATED SURGE EVENTS AT THE BATTERY SHOWN BY NUMERIC STORM IDENTIFICATION AND SURGE RANKING (AREVA, 2013E) 3-106

FIGURE 3.4-42: HISTOGRAM OF MAXIMUM STORM SURGE AT IPEC AT LOW TIDE (AREVA, 2013E) 3-107

FIGURE 3.4-43: HISTOGRAM OF MAXIMUM STORM SURGE AT IPEC AT HIGH TIDE (AREVA, 2013E) 3-107

FIGURE 3.4-44: HISTOGRAM OF MAXIMUM STORM SURGE AT THE BATTERY AT LOW TIDE (AREVA, 2013E) 3-108

FIGURE 3.4-45: HISTOGRAM OF MAXIMUM STORM SURGE AT THE BATTERY AT HIGH TIDE (AREVA, 2013E) 3-108

FIGURE 3.4-46: SLOSH-CALCULATED STORM TIDE (STILLWATER) STAGE-FREQUENCY CURVE AT IPEC 3-109

FIGURE 3.4-47: SLOSH-CALCULATED STORM TIDE (STILLWATER) STAGE-FREQUENCY CURVE AT THE BATTERY (AREVA, 2013E) 3-109

FIGURE 3.4-48: COMPARISON OF ADCIRC TIDAL RESULTS TO PREDICTED TIDES AT THE BATTERY (AREVA, 2013E) 3-110

List of Figures
(continued)

FIGURE 3.4-49: ADCIRC TIDAL RESULTS WITH 25-YEAR FLOOD IN THE HUDSON RIVER AT IPEC (AREVA, 2013E) 3-110

FIGURE 3.4-50: ADCIRC-CALCULATED STORM TIDE (STILLWATER) STAGE-FREQUENCY CURVE AT IPEC WITHOUT SEA LEVEL RISE (AREVA, 2013E) 3-111

FIGURE 3.4-51: ADCIRC-CALCULATED STORM TIDE (STILLWATER) STAGE-FREQUENCY CURVE AT IPEC WITH SEA LEVEL RISE (AREVA, 2013E) 3-112

FIGURE 3.4-52: INUNDATION MAP – COMBINED EFFECT FLOOD - PROBABILISTIC STORM SURGE 3-113

FIGURE 3.5-1: SITE LOCUS MAP WITH LOCATION OF THE IPEC DISCHARGE CANAL (ESRI, 2012). 3-124

FIGURE 3.5-2: NYS DEC LOWER HUDSON RIVER ESTUARY PROGRAM BATHYMETRY. 3-125

FIGURE 3.5-3: NYS DEC UPPER HUDSON RIVER ESTUARY PROGRAM BATHYMETRY. 3-126

FIGURE 3.5-4: DIAGRAM OF THE FIRST THREE RESONANT MODES OF AN ENCLOSED (LEFT) AND SEMI-ENCLOSED (RIGHT) BASIN FROM SCHEFFNER 2008 3-127

FIGURE 3.5-5: MAP OF THE HUDSON RIVER INCLUDING THE LOCATION OF IPEC AND NOAA TIDAL CONSTITUENTS STATIONS (NOAA, 2005A-D) 3-128

FIGURE 3.5-6: MAP OF THE HUDSON RIVER INCLUDING THE LOCATION OF IPEC AND USGS STREAM GAGES USED FOR A FAST FOURIER TRANSFORM (USGS, 1991; NOAA, 2012) 3-129

FIGURE 3.5-7: SPECTRAL ANALYSIS OF SURFACE HEIGHT TIME SERIES AT ALBANY ON THE HUDSON RIVER 3-130

FIGURE 3.5-8: SPECTRAL ANALYSIS OF SURFACE HEIGHT TIME SERIES AT HASTINGS ON THE HUDSON RIVER. 3-131

FIGURE 3.5-9: SPECTRAL ANALYSIS OF SURFACE HEIGHT TIME SERIES AT POUGHKEEPSIE ON THE HUDSON RIVER. 3-132

FIGURE 3.5-10: SPECTRAL ANALYSIS OF SURFACE HEIGHT TIME SERIES AT WEST POINT ON THE HUDSON RIVER 3-133

FIGURE 3.5-11: SPECTRAL ANALYSIS OF SURFACE HEIGHT TIME SERIES AT SANDY HOOK ON THE HUDSON RIVER. 3-134

FIGURE 3.5-12 PROFILE VIEW OF THE IPEC DISCHARGE CANAL, (IPEC, 1970) 3-135

FIGURE 3.6-1: LOCATION OF IPEC RELATIVE TO HUDSON RIVER FEATURES AND THE CONTINENTAL SLOPE 3-147

FIGURE 3.6-2: POTENTIAL TSUNAMI SOURCE LOCATIONS FOR THE U.S. EAST COAST IN THE NORTH ATLANTIC OCEAN BASIN 3-148

FIGURE 3.6-3: SHADED RELIEF MAP OF THE EAST COAST OF THE U.S. WITH INTERPRETATION SHOWING THE 48 LANDSLIDE AREAS MAPPED BY AGMTHAG (AGMTHAG, 2008) 3-149

List of Figures
(continued)

FIGURE 3.6-4: THE ADDITIONAL 46 SHELF TRANSECTS USED IN KRAUSE (KRAUSE, 2011) EACH USED FOR 15,000 STOCHASTIC STABILITY ANALYSES OF SUBMERGED SLOPES 3-150

FIGURE 3.6-5: TIME SERIES OF TSUNAMI AMPLITUDE AT A NEARSHORE LOCATION BROADSIDE OF THE CURRITUCK LANDSLIDE FOR THE THREE DIFFERENT SLIDE SCENARIOS. 3-151

FIGURE 3.6-6: MAXIMUM SURFACE ELEVATION COMPUTED WITH FUNWAVE IN 30" REGIONAL GRIDS FOR THE 19 MI^3 CVV CASE (HARRIS ET AL, 2012). 3-152

Acronyms and Abbreviations

Acronym/Abbreviation	Description
ANS	American Nuclear Society
ANSI	American National Standards Institute
CFR	Code of Federal Regulations
cfs	Cubic feet per second
CLB	Current Licensing Basis
COLA	Combined License Application
DEM	Digital Elevation Model
DPF	Design Project Flood
DTM	Digital Terrain Model
IPEC	Indian Point Energy Center
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
ISFSI	Independent Spent Fuel Storage Installation
LIP	Local Intense Precipitation
MSL	Mean Sea Level, also known as Sea Level Datum of 1929
NAVD88	North American Vertical Datum of 1988
NGDC	National Geophysical Data Center
NGVD29	National Geodetic Vertical Datum of 1929, also known as Mean Sea Level
NOAA	National Oceanic And Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission

Acronyms and Abbreviations
(continued)

Acronym/Abbreviation	Description
NRCS	Natural Resources Conservation Service
NTTF	Near-Term Task Force
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PMWE	Probable Maximum Water Elevation
SCS	Soil Conservation Service
SSC	Structures, systems and components
SSW	Standby Service Water
UFSAR	Updated Final Safety Analysis Report
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
VBS	Vehicle Barrier System

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.

Indian Point Energy Center (IPEC), located in Buchanan, New York, is one of the sites required to submit information. IPEC consists of two operational units (Units 2 and 3) and one unit that is decommissioned (Unit 1). This report addresses flooding conditions for Units 2 and 3 only.

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 IPEC.

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 IPEC.

Each of these flood causing mechanisms and the potential effects on the IPEC site is described in Section 3 and 4 respectively 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 relevant flood-causing mechanism listed in Section 3 and 4. A maximum flood level was determined as the event that resulted in the most severe hazard to the safety-related SSC at IPEC. The steps involved to calculate a maximum flood level typically included the following:

1. Identify flood-causing phenomena or mechanisms by reviewing historical data and assessing the hydrometeorological, geoseismic, and structural failure phenomena in the vicinity of the site and region.
2. For each flood-causing phenomenon, calculate the flood elevation from the corresponding probable maximum event using initial 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. If any safety-related SSC is adversely affected by flood hazard, repeat steps 2 and 3 using further refined analyses; if all safety-related SSC are unaffected by the calculated flood, or if all justified site-specific refinements have been used, identify the most severe flood hazard from each flood mechanism.

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 calculation may be significantly higher than results that could be obtained using more refined, realistic 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

Any assumptions indicated in this report are discussed and justified in the body of this report, by topic, and in supporting calculations.

1.5 Elevation Datums

Elevations in this report refer to the National Geodetic Vertical Datum of 1929 (NGVD29) unless otherwise noted. As per United States Army Corps of Engineers document EM 1110-1-1005 Appendix C (USACE, 2007), the NGVD29 was originally named the Mean Sea Level Datum of 1929 (MSL). Elevations referenced in the current IPEC Unit 2 and Unit 3 Updated Final Safety Analysis Reports (UFSAR) (IPEC, 2010, IPEC 2011) that use MSL are equivalent to NGVD29. To convert elevations from NAVD88 to NGVD29, at the site, 0.99 feet was added to the NAVD88 elevations (VERTCON, 2013).

1.6 References

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

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.

USACE, 2007. "EM 1110-1-1005, Appendix C – Development and Implementation of NAVD 88", United States Army Corps of Engineers, January 2007.

VERTCON, 2013. North American Vertical Datum Conversion, by National Geodetic Survey, http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html, accessed January 17, 2013. See AREVA Document No. 32-9196316-000.

2.0 INFORMATION RELATED TO THE FLOOD HAZARD

2.1 Detailed Site Information

IPEC is located approximately 43 miles north of "the Battery," the southern tip of Manhattan Island, New York, NY. The site is situated on the east bank of the Hudson River on a sloping, terraced bedrock exposure at about river mile 43 (Figure 2.1-1). The plant site property boundary encompasses approximately 239 acres.

2.1.1 Present-day Site Layout

IPEC site layout and topography is shown in Figure 2.1-2 (Sanborn, 2013).

Unit 2 and Unit 3 are located along the Hudson River shoreline, with a grade elevation at the turbine buildings for both units of approximately 15 ft. Unit 2 is located to the northeast of Unit 3 along the shoreline. Both reactor containment buildings are set into the bedrock slope with a base elevation near 18 ft rising up to approximately 85 ft.

2.1.2 Site Topography and River Setting

The plant is founded on a terraced bedrock slope. The IPEC site grade elevation varies from 15 ft at the Hudson River shoreline to about 140 ft with surrounding high ground ranging from 600 to 1,000 feet. The drainage for the entire site flows into the Hudson River. A vehicle barrier system (VBS) redirects off-site surface water flow around the IPEC site.

Near IPEC, the Hudson River consists of a fiord-like water body, or an extended estuary, about 1 mile wide. Tidal influence on the river extends to the dam at Troy (north of Albany, NY), about 125 miles upstream from New York City. River flow is generally confined to a width of about two to four miles by high bedrock slopes on the east and west bank. Adjacent to IPEC, the Hudson River has a width of 4,500 ft to 5,000 ft. The river width varies from about 0.3 mile to 3 miles across downstream of the plant site (Figure 2.1-3).

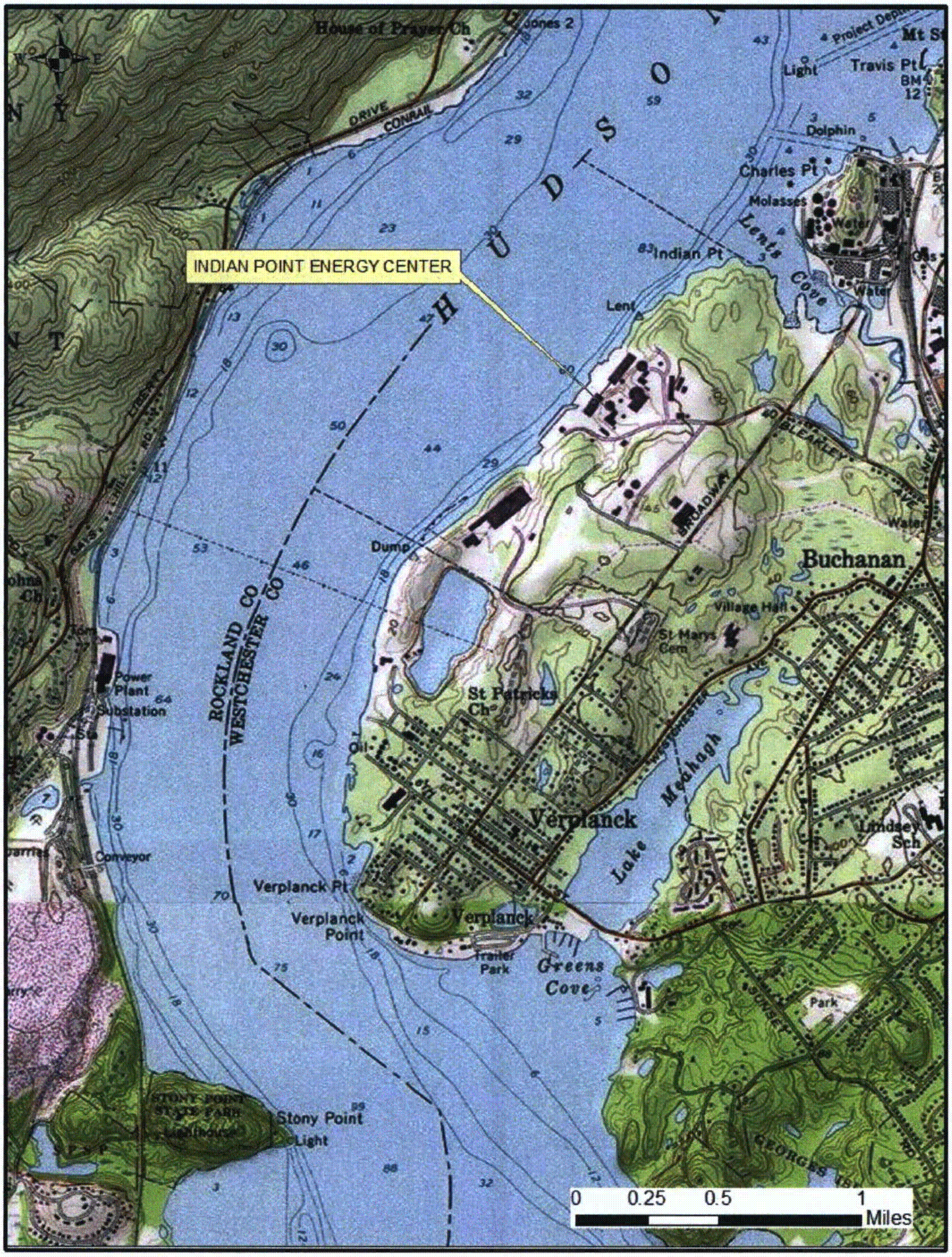

Figure 2.1-1: IPEC Location Map

Figure 2.1-2: Site Topography and Layout

Figure 2.1-3: Hudson River from IPEC to the River Mouth

2.2 Current/Licensing Design Basis Flood Elevations

The current design basis and related flood elevations for IPEC are described in the IPEC Updated Final Safety Analysis Reports (UFSARs) (IPEC, 2010 and IPEC, 2011) as well as the recent walkdown report (IPEC, 2012a and 2012b) required based on the NRC's 10 CFR 50.54(f) letter.

2.3 Current Licensing Basis Flood Protection and Mitigation Features

The CLB for IPEC is defined from the IPEC UFSAR documents (IPEC, 2010 and IPEC, 2011) and supporting plant documents.

2.3.1 Flooding Mechanisms

Flooding hazard evaluations for IPEC include an assessment for the following flood mechanisms:

1. Probable Maximum Flood (PMF) on Hudson River – 12.7 ft (IPEC 2010, Table 2.5-1)
    a. PMF with Coincident Wind-Wave Activity - 13.7 ft (IPEC 2010, Table 2.5-1)
2. PMF and High Tide – 12.4 ft (IPEC 2010, Table 2.5-1)
    a. Coincident Wind-Wave Activity – 13.4 ft (IPEC 2010, Table 2.5-1)
3. PMF and Low Tide – 13.0 ft (IPEC 2010, Table 2.5-1)
    a. Coincident Wind-Wave Activity – 14.0 ft (IPEC 2010, Table 2.5-1)
4. River Flooding with Dam Failures – 7.2 ft (IPEC 2010, Table 2.5-1)
    a. Coincident Wind-Wave Activity – 8.2 ft (IPEC 2010, Table 2.5-1)
5. Probable Maximum Hurricane (PMH) and Spring High Tide – 13.5 ft (IPEC 2010, Table 2.5-1)
    a. Coincident Wind-Wave Activity – 14.5 ft (IPEC 2010, Table 2.5-1)
6. Standard Project Hurricane and Standard Project Flood on Hudson River – 13.0 ft (IPEC 2010, Table 2.5-1)
    a. Coincident Wind-Wave Activity – 14.0 ft (IPEC 2010, Table 2.5-1)
7. Standard Project Flood plus Standard Project Hurricane plus Dam Failure – 14.0 ft
    a. Coincident Wind-Wave Activity – 15.0 ft (IPEC 2010, Table 2.5-1)

The NRC did an independent evaluation of flooding from the Hudson River which calculated a maximum stillwater elevation of 15.0 ft with a 3.25 ft wave runup for the Unit 2 intake structure (NRC, 1988). This evaluation concluded that the service water pumps (the nearest safety-related SSC) located 20 to 30 ft away from the bank of the river would not be impacted by direct wave runup.

In addition, an evaluation of the impacts of a localized probable maximum precipitation event was performed during the Individual Plant Examination of External Events (IPEEE) for Unit 2 in 1995 (IPEEE, 1995). Although not part of the CLB, the results of this analysis indicated localized ponding could occur in the vicinity of IPEC structures.

2.3.2 Flood Protection and Mitigation

2.3.2.1 IPEC Unit 2

Flood protection features at IPEC Unit 2 include both permanent and temporary features. The permanent features consist of door seals, dikes, exterior walls and floors of structures containing safety-related SSC, backflow prevention valves, penetration seals, and conduit seals. The temporary protection features include portable gas-powered pumps and submersible electric pumps. Temporary protection equipment is kept in the Unit 1 Turbine Building at an elevation of 33 ft (IPEC, 2012a). Repairs have been made to door seals to mitigate the effects of flooding due to reevaluated local intense precipitation (LIP) (see Section 3.1); relevant doors are above the CLB flood elevation.

The Intake Structure and Control Building contain safety-related equipment requiring protective features below the CLB flood elevation of 15.0 ft. The 1 ft dike around MCC 24A in the Turbine Building protects this structure during flooding events. In response to the IPEEE assessed PMP event for the Transformer Yard, although not part of the CLB, the north wall of the Control Building (480V Switchgear) is inspected to a height of 18.7 ft (IPEC, 2012a).

The external flooding procedure for Unit 2 is initiated in the event of Hudson River water levels rising above 4.5 ft, a high tide advisory, or a NOAA flood warning. This procedure includes actions to plug the floor drains on the 12 ft elevation of the Turbine Building, open the breaker on the feed to the Service Water Strainer Pit heaters, and to install temporary pumps when necessary to assist the sump pump maintaining the strainer pit dry. If river levels reach 5.75 ft, the procedure has an action to close the Zurn Strainer Pit drain valve and to continuously monitor the strainer pit for leakage (IPEC, 2012a).

2.3.2.2 IPEC Unit 3

Flood protection features at IPEC Unit 3 include both permanent and temporary features. The permanent features consist of door seals, dikes, exterior walls and floors of structures containing safety-related SSC, backflow prevention valves, penetration seals, and conduit seals. The temporary protection features include portable gas-powered pumps and submersible electric pumps. Temporary protection equipment is kept in the Unit 1 Turbine Building at an elevation of 33 ft (IPEC, 2012b). Repairs have been made to door seals to mitigate the effects of flooding due to reevaluated LIP; relevant doors are above the CLB flood elevation.

The Intake Structure, Control Building, Diesel Generator Building, and Primary Auxiliary Building contain safety-related SSC at or below the CLB flood level of 15 ft that require protection. The north wall of the Control Building is inspected up to the grade level in the transformer yard due to the number of conduit penetrations (IPEC, 2012b).

Flooding procedures for Unit 3 provide steps to mitigate external flooding. In the event of external flooding, this procedure is initiated in the event of Hudson River levels above 7 ft, a high tide advisory, or a NOAA flood warning. If the Hudson River exceeds 7 ft 4 inches, the flood procedure requires that the strainer pit be monitored for leakage and installation of temporary pumps to assist the sump pump in maintaining the strainer pit dry. Also, sandbags are placed around the strainer pit as necessary and temporary pumps are installed as necessary to maintain the Service Water Valve pits dry. When the Hudson River level reaches 10 ft, power is removed from the Service Water Strainer Pit sump pump and the pump discharge valve is closed. When the Hudson River reaches 11 ft, sandbags are installed around the Service Water Pump area, requiring this action to be completed before the river level reaches 15 ft. The plant is to be shut-down when the river level reaches 12 ft 5 inches. A maintenance procedure also establishes the requirements for stacking and location of sandbags in order to protect the Service Water Pumps, the Turbine Building, and the Administration Building in the event the river exceeds 11 ft (IPEC, 2012b).

2.4 Licensing Basis Flood-Related and Flood Protection Changes

Maintenance procedure 0-MET-402-GEN, Rev. 2 has been issued that protects the service water pumps and all vulnerable areas for external flooding up to an elevation of 17'-11" for both Units (AREVA, 2013). Current mitigation measures are in place up to 17'-11".

2.5 Watershed and Local Area Changes

2.5.1 General IPEC Site Hydrological Description

The dominant hydrologic feature in the vicinity of the site is the Hudson River. The Hudson River below the dam at Troy (immediately below the confluence of the Hudson and Mohawk Rivers) is a tide-influenced, estuarine waterway. Fresh water from the combined Hudson and Mohawk Rivers, as well as from numerous tributaries, discharges directly into the tidal portion of the river. Seawater enters the extreme lower reaches of the river through the Narrows and the Harlem/East River. The distribution of saltwater is influenced by fresh water flow, tides, physical characteristics of the river channel, and weather.

Flow in the Hudson River is normally controlled more by the tides than by the runoff from the tributary watershed. River width adjacent to the plant ranges from 4,500 ft to 5,000 ft. Water depths within 1,000 ft of the shore near the site are variable with an average depth of 65 ft with depths exceeding 85 ft.

2.5.2 Watershed Changes

The Hudson River is a large watershed with numerous tributaries that are often regulated by dams and other measures. There have been no changes to the watershed in the vicinity of the site that impact flood hazard.

2.5.3 Site Changes

There have been no site changes pertinent to flood modeling or hazard.

2.6 Additional Site Details

N/A

2.7 References

AREVA, 2013. AREVA Document No. 38-9216740-000, Entergy IPEC 0-MET-402 GEN, Rev. 2 Location of Sand bags in Flood Warning Conditions.

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

IPEEE, 1995. "Individual Plant Examination of External Events for Indian Point Unit No. 2 Nuclear Generating Station", Consolidated Edison Company of New York, Inc., December 1995.

IPEC, 2012a. "Indian Point Energy Center Unit 2 Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", Entergy Nuclear, Engineering Report No. IP-RPT-12-00036, November 2012, See AREVA Document No. 38-9193643-000.

IPEC, 2012b. "Indian Point Energy Center Unit 3 Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", Entergy Nuclear, Engineering Report No. IP-RPT-12-00038, November 2012, See AREVA Document No. 38-9193643-000.

NRC, 1988. "External Flooding Condition Technical Specifications for Indian Point Nuclear Generating Unit No. 2 (TAC No. 51921)", U.S. Nuclear Regulatory Commission, November 15 1988.

Sanborn, 2013. Indian Point Energy Center (IPEC) Topographic Survey, Sanborn Map Company, Inc., January 2013, AREVA Document No. 38- 9196956-000.

3.0 INFORMATION RELATED TO THE FLOOD HAZARD

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

3.1 Local Intense Precipitation

This section addresses the potential for flooding at IPEC due to the Local Intense Precipitation (LIP) event. The Local Intense Precipitation event is a distinct flooding mechanism that consists of locally heavy rainfall centered upon the plant site itself.

3.1.1 Method

3.1.1.1 Local Intense Precipitation

The hierarchical hazard assessment (HAA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the LIP and resultant water surface elevation at IPEC. The HHA approach is a progressively refined, stepwise estimation of site-specific flood hazards, starting with the most conservative simplifying assumptions that maximize flood hazards. If the site is not inundated by the flood mechanism evaluated, a conclusion that the SSC are not susceptible to flooding is valid and no further analyses were completed (NRC, 2011). Please note that the methodology used in this study differs from what was used in the calculations supporting IPEC's IPEEE, primarily due to subsequent changes in regulatory guidelines (i.e., NUREG/CR-7046). This calculation includes: 1) the assumption that all 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 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. Develop hydraulic computer model for LIP analysis
2. Develop LIP/PMP inputs.
3. Perform flood simulations in FLO-2D and calculate maximum water surface elevations throughout the IPEC site.

3.1.2 Results

3.1.2.1 Local Intense Precipitation (AREVA, 2013a)

3.1.2.1.1 Develop hydraulic model for LIP analysis

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 digital terrain model (DTM) limits input into FLO-2D (Sanborn, 2013).

The FLO-2D model includes topography, site location, and building structures. Grid elements along the model computational boundary were selected as outflow grid elements.

3.1.2.1.2 Develop LIP/Probable Maximum Precipitation (PMP) inputs

The main input parameters for the IPEC FLO-2D model include:

The 6-hour (Probable Maximum Precipitation) (PMP), which includes the 1-hour and sub-1-hour PMPs, was calculated from the site specific PMP study (AREVA, 2013b) using the methodology of HMR -52 (HMR-52, 1982). This was used as the controlling storm for the LIP analysis. The total PMP depth was calculated as 15.3 inches.

Elevation: The elevation data used to develop the FLO-2D model consist of DTM data (Sanborn, 2013) prepared by photogrammetric methods using aerial photography supplemented by surveyed information included in the topographic map of the site (Sanborn, 2013). Elevation data for the area within the model limits but beyond the extent of the topographic survey was supplemented with data extracted from the Peekskill DEM file (CUGIR, 2012).

Manning's n Coefficient: Land use categories were selected based on aerial photography (Sanborn, 2013) and visual assessment. The Manning's n value selected for each land cover category was based on two references: Chow, 1959 and FLO-2D Software manual.

Buildings: Buildings were incorporated as completely blocked grid elements based on assessment of aerial photography (Sanborn, 2013). FLO-2D calculates runoff from blocked grid elements and translates it to the nearest unblocked grid element.

Channel Segments: The model includes one channel segment as shown in Figure 3.1-1, to model the existing discharge canal / channel for IPEC. The channel is approximately 35-feet wide for the first 710 feet and transitions to a 75-feet wide channel paralleling the Hudson River for 530 feet (Sanborn 2013). The cross sectional geometry was assumed to be rectangular. The channel bottom elevations were based on cross sectional information (GZA, 2008).

Transformer Yard Moats: The storage volume of Unit 2 and Unit 3 transformer yard moats (IPEC, 2012, IPEC, 1983 and IPEC, 2006) was used to calculate an elevation adjustment which was applied to grid elements within the footprint of the moat in the transformer yards. The purpose of the reduction in elevation to the selected grid elements was to account for the storage capacity in each moat. The volume in the Unit 2 transformer yard moat is approximately 10,300 cubic feet (IPEC, 2012 and IPEC, 1983). Eight grid cells were adjusted in the transformer yard for Unit 2 by -1.4 vertical feet. The volume in the Unit 3 transformer yard moat was approximately 8,550 cubic feet (IPEC, 2012 and IPEC, 2006). Six grid cells were adjusted in the transfer yard for Unit 3 by -1.5 vertical feet.

3.1.2.1.3 Perform flood simulations in FLO-2D and calculate maximum water surface elevations throughout the IPEC site

Calculated depths typically ranged from 0.1 to 3.6 feet. Several areas of localized greater depths were identified:

- 1.1 to 1.4 feet along the northeast corner of the Unit 2 Turbine Building and between the Unit 2 Turbine Building and the Unit 2 Auxiliary Feedwater building;
- 1.0 to 3.6 feet between the Unit 2 Maintenance and Outage Building and the Primary Auxiliary Building;
- 1.1 to 3 feet along the east corner of the Unit 1 Fuel Storage Building;
- 0.7 to 1 feet along the south corner of the Unit 1 Superheater / Administration Building;
- 1.5 to 1.9 feet along the east edge of the Unit 3 Primary Auxiliary Building; and
- 1.2 to 2.4 feet along the southwest face of the Unit 3 Administration Building.

The resultant high water surface elevations along the northeast side of the Unit 2 Turbine Building and southwest sides of the Unit 3 Primary Auxiliary Building and the Administration Building are likely due to runoff coming from the higher elevations immediately adjacent of these buildings and the constriction to overland flow formed by the buildings themselves. Note that the concrete security barriers were not included in the model because they are not known to be credited or designed to withstand flood flows.

The calculated maximum water surface elevation was 19.0 feet in the interior of the Unit 2 transformer yard. There are five building entrances abutting the Unit 2 transformer yard structure [maximum water surface elevation in feet NGVD29], with flood levels shown below:

- the building entrance (U2-PAB-1) along the west side of the Unit 2 Primary Auxiliary Building is at elevation 18.9 feet (Sanborn, 2013 and IPEC, 2013), [18.9 feet];
- the north door Auxiliary Feedwater Building (U2-ABFP-1) threshold at elevation 18.6 feet (Sanborn, 2013 and IPEC, 2013), [18.5 feet];
- the roll up door Auxiliary Feedwater Building (U2-ABFP-2) at elevation 18.7 feet (Sanborn, 2013 and IPEC, 2013), [between 18.5 and 18.8 feet];
- the south door Auxiliary Feedwater Building (U2-ABFP-3) at elevation 18.7 feet (Sanborn, 2013 and IPEC, 2013), [between 18.5 and 18.8 feet] and
- the exterior double door (U2-CB-1) for the Unit 2 Control Building to the transformer yard is at elevation 18.7 feet (Sanborn, 2013 and IPEC, 2013), [18.9 feet].

The calculated maximum water surface elevation at the northwest corner of the Unit 2 Auxiliary Feedwater Building is 18.5 feet, with a calculated maximum depth of 1.4 feet.

The calculated maximum water surface elevation along the northwest face of the Unit 2 Turbine Building is between 15 and 15.6 feet, with a calculated maximum depth between 0.1 and 0.6 feet.

The calculated maximum water surface elevation along the southwest face of the Unit 2 Emergency Diesel Generator Building varies between 71.2 and 74.8 feet, with a calculated maximum depth between 0.1 and 1.2 feet.

The calculated maximum water surface elevation between the Unit 2 Fuel Storage Building and Primary Auxiliary Building is 80.8 feet, with a calculated maximum depth between 1 and 3.6 feet.

The calculated maximum water surface elevation was 19.2 feet in the interior of the Unit 3 transformer yard. There are six building entrances abutting the Unit 3 transformer yard structure [maximum water surface elevation in feet NGVD 29], with flood levels shown below:

- the Auxiliary Feedwater Building door (U3-ABFP-1) at 18.6 feet (Sanborn, 2013 and IPEC, 2013), [18.0 feet];
- the Auxiliary Feedwater Building roll up door (U3-ABFP-2) at elevation 18.7 feet (Sanborn, 2013 and IPEC, 2013), [between 18.4 and 18.5 feet];
- the Auxiliary Feedwater Building door (U3-ABFP-3) calculated at elevation 18.7 feet (Sanborn, 2013 and IPEC, 2013), [18.5 feet];
- two Primary Auxiliary Building doors (U3-PAB-1 and U3-PAB-2) sill elevations at 18.9 feet (Sanborn, 2013 and IPEC, 2013) [between 18.5 and 18.6 feet] and
- the Control Building door (U3-CB-1) calculated door elevation at 20 feet, (Sanborn, 2013 and IPEC, 2013), [18.8 feet].

The maximum calculated water surface elevation along the northeast side of the Unit 3 Turbine building varies from 14.0 and 16.4 feet, and the maximum depth varies between 0.2 and 0.4 feet.

The maximum calculated water surface elevation along the northeast side of the Unit 3 Control Building (in the transformer yard) varies from 18.7 to 19.2 feet, and the maximum depth is 0.2 feet.

The maximum calculated water surface elevation along the northwest side of the Unit 3 Primary Auxiliary Building (in the transformer yard) is 18.6 feet.

Representative hydrographs showing LIP flood durations are provided in Figures 3.1-2 and 3.1-3. These are for locations where flood protection is required.

3.1.3 Conclusions

At IPEC, impacts to the site from LIP are discussed below:

In the immediate vicinity of Unit 2, the maximum water surface elevations predicted by the FLO-2D model are up to elevation 19.0 feet in the transformer yard. The calculated LIP elevation varies between 0 and 0.2 feet higher than building entrances abutting the Unit 2 Transformer Yard. Details are provided in Section 4.3.

In the immediate vicinity of Unit 3 the maximum water surface elevations predicted by the FLO-2D model are up to elevation 19.2 feet in the transformer yard. The calculated LIP elevation does not exceed building entrances abutting the Unit 3 Transformer Yard.

3.1.4 References

AREVA, 2013a. "Flood Hazard Re-evaluation – Local Intense Precipitation-Generated Flood Flow and Elevations at Indian Point Energy Center", AREVA Document No. 32-9196321-000, 2013.

AREVA, 2013b. "Probable Maximum Precipitation", AREVA Document No. 32-9196314-000, 2013.

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

Chow, 1964. Chow, Ven Te Handbook on Applied Hydrology, McGraw-Hill, New York, 1964.

CUGIR, 2012. CUGIR: Cornell University Geospatial Information Repository Digital Elevation Model (DEM) – "Peekskill Mapsheet", (http://cugir.mannlib.cornell.edu/mapsheet.jsp?code=DD47), accessed December 2012, See AREVA Document No. 32-9196321-000.

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

GZA, 2008. GZA GeoEnvironmental, Inc., "Hydrogeologic Site Investigation, Figure 6.18 – Unit 1 and 2 Hydrologic Cross Section A-A' and B-B'", January 2008. See AREVA Document No. 32-9196321-000.

HMR-52, 1982. "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", NOAA Hydrometeorological Report No.52 (HMR-52), US Department of Commerce & U.S. Army Corps of Engineers, August 1982.

IPEC, 1983. "Yard Storm Drains for Indian Point Energy Center Indian Point Generating Station Unit No. 2", Westinghouse Electric Corporation, March 17, 1983, See AREVA Document No. 38-9193643-000.

IPEC, 2006. "Yard Storm Drains for Indian Point Energy Center Indian Point Generating Station Unit No. 3", Westinghouse Electric Corporation, December 28, 2006, See AREVA Document No. 38-9193643-000.

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

IPEC, 2012. "Moat volume summary.xls", Entergy, 2012, See AREVA Document No. 38-9193643-000.

IPEC, 2013. "Transformer Yard Doors.jpg", Entergy, 2013, See AREVA Document No. 38-9193643-000.

Sanborn, 2013. Indian Point Energy Center (IPEC) Topographic Survey, Sanborn Map Company, Inc., January 2013, AREVA Document No. 38- 9196956-000.

Figure 3.1-1: FLO-2D Computational Boundary, Outflow Nodes, Building Elements and Channels (AREVA, 2013a)

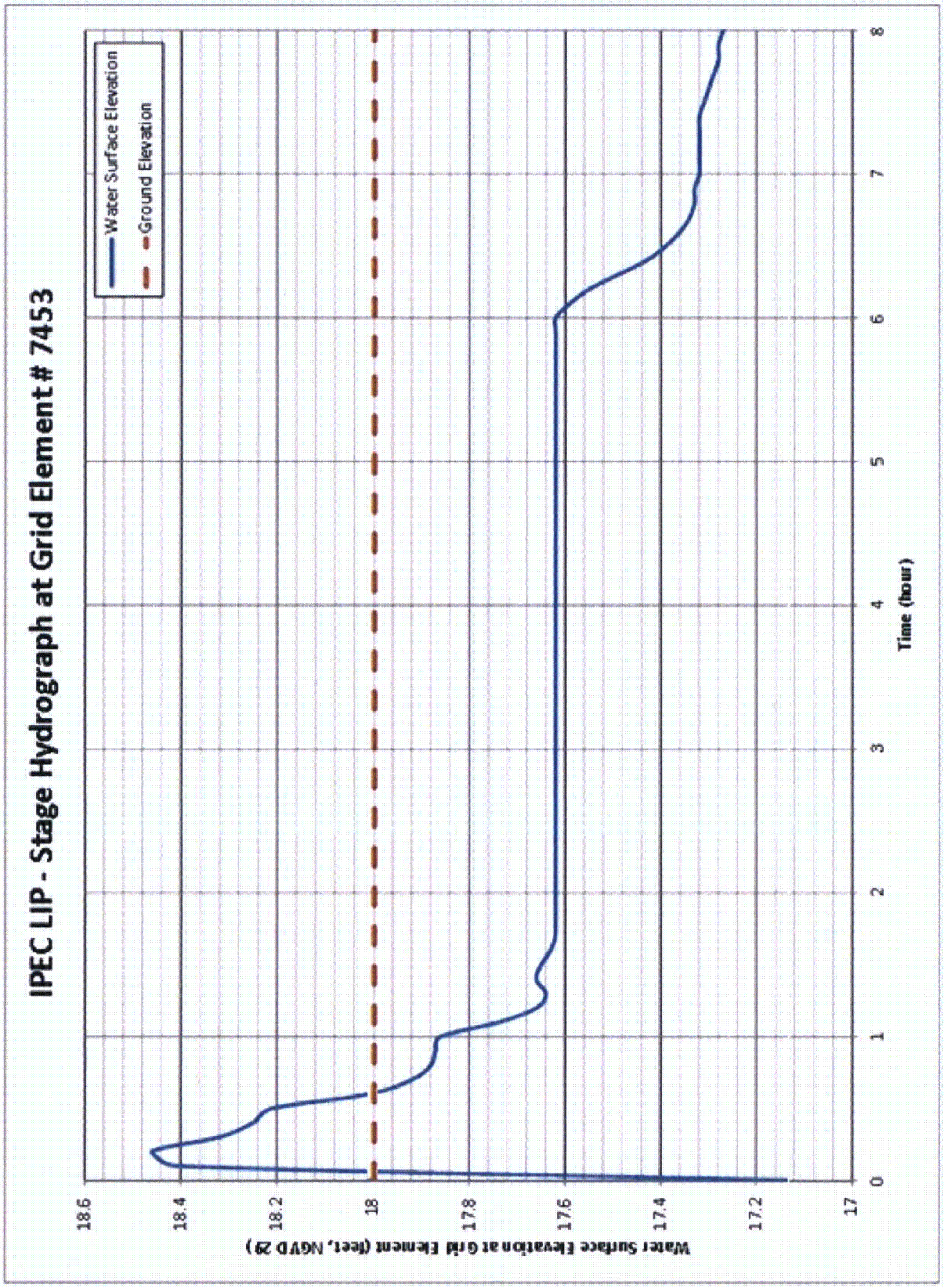

Figure 3.1-2: LIP Hydrograph for Unit 2 Transformer Yard Door (AREVA, 2013a)

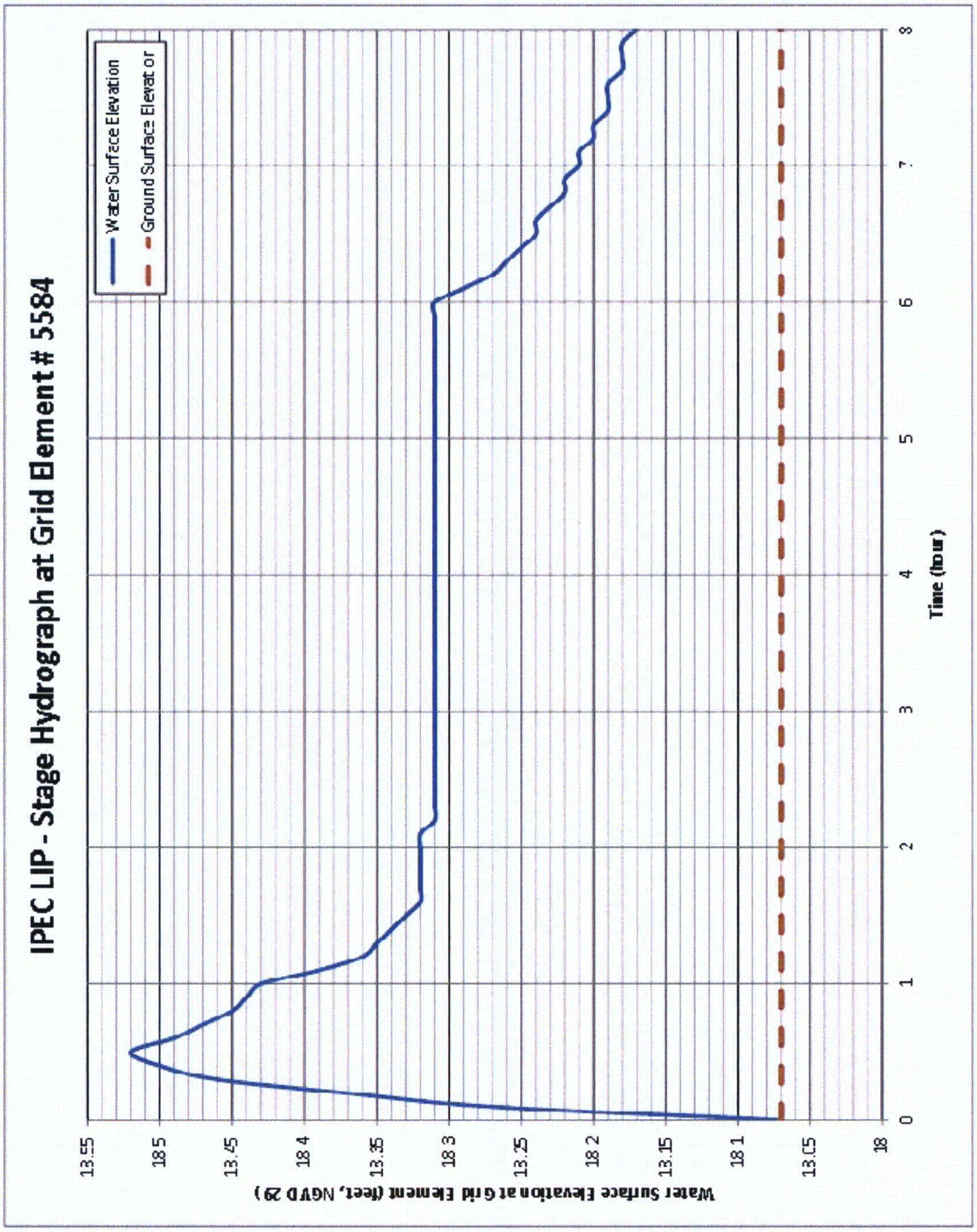

Figure 3.1-3: LIP Hydrograph for Unit 3 Transformer Yard Door (AREVA, 2013a)

3.2 Flooding in Rivers and Streams

This section addresses the potential for flooding at IPEC due to the PMF on streams and rivers. 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, Section 3.3)

3.2.1 Method

HHA approach described in NUREG/CR-7046 (NRC, 2011, Section 2) was used for the evaluation of the PMF on rivers and streams and resultant water surface elevation at IPEC.

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 (ANS,1992)

3.2.1.1 Probable Maximum Flood - Hudson River

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

1. Delineate the Hudson River watershed contributory to IPEC.
2. Perform the meteorological site-specific Probable Maximum Precipitation (PMP) Study.
3. Calculate All Season and Cool Season Probable Maximum Precipitation (PMP) using BOSS HMR52 computer program.
4. Calculate 100-year snowpack melt rate to Cool Season PMP using USACE energy budget methodology.
5. Develop rainfall/runoff model: Model the subwatersheds, dams, tributary streams, and Hudson River reach upstream of the site using the HEC-HMS (USACE, 2000) computer program.
6. Calibrate and Verify HEC-HMS Model using observed stream gage flow data.
7. Perform PMF hydrologic simulations with HEC-HMS for both All Season and Cool Season (with 100-year snowpack) PMP storms. Select event that generates highest peak flow rate at site. PMF flow hydrograph is used as input to HEC-RAS (USACE, 2010) in Step 9.
8. Develop HEC-RAS unsteady flow hydraulic computer model for selected portion of the Hudson River.
9. Calibrate HEC-RAS Model using observed stream and tide gage elevation data.
10. Perform PMF hydraulic simulations using the HEC-RAS model.

3.2.2 Results

3.2.2.1 Probable Maximum Flood – Hudson River (AREVA, 2013a and AREVA, 2013b)

3.2.1.1.1 Delineate the Hudson River watershed contributory to IPEC

The contributory drainage area to Hudson River at IPEC was delineated based on topography and stream flow direction (ESRI, 2011). The total watershed area is 12,750 square miles (Figure 3.2-1).

3.2.2.1.2 Perform the meteorological site-specific Probable Maximum Precipitation (PMP) Study

The location of the Hudson River watershed is within the domain of the National Weather Service / U.S. Army Corps of Engineers Hydrometeorological Report No. 51 (HMR-51, 1978) and Hydrometeorological Report No. 52 (HMR-52, 1982). However, the Hudson River watershed at IPEC is located in the Appalachian Mountains, a "stippled" region of HMR-51 and HMR-52. HMR-51 states that "this stippling outlines areas within which the generalized PMP estimates might be deficient because detailed terrain effects have not been evaluated." Further, HMR-51 states that "...we suggest that major projects within the stippled regions be considered on a case-by-case basis as the need arises." A site-specific Probable Maximum Precipitation (PMP) study for IPEC was therefore performed (AREVA, 2013c).

The purpose of the study was to calculate site-specific PMP depth-area-duration values over the watershed for both the All Season and Cool Season PMPs. The approach used in the study is storm-based utilizing many of the procedures used by the National Weather Service (NWS) in the development of the HMRs. These same procedures are recommended by the World Meteorological Organization (WMO) for PMP determination. All Season and Cool Season extreme rainfall events were identified as storms having similar characteristics to extreme rainfall events that could potentially occur over the Hudson River watershed and could potentially influence PMP values at one or more of the area sizes and/or durations analyzed. These storms are separated into two types: one representing the all season event when no snowpack would be available and one representing the cool season, when antecedent snowpack would potentially supplement the rainfall runoff. The storms of interest were maximized, transpositioned, and elevation-adjusted to the Hudson River at IPEC's watershed centroid. Depth-Area (DA) plots were made for durations of 6-, 12-, 24-, 48-, and 72-hour for area sizes of 10-, 200-, 1,000-, 5,000-, 10,000-, and 20,000-square miles. Enveloping curves were constructed for each set of curves at the Hudson River at IPEC's watershed centroid. Depth-Duration plots were then made and curves constructed.

3.2.2.1.3 Perform HMR52 Computer Model for All Season and Cool Season Probable Maximum Precipitation (PMP)

The PMP was calculated for the 12,750-square-mile Hudson River at IPEC watershed using the methodology of HMR-51 and HMR-52. The BOSS HMR52 computer program was used for the calculations. BOSS HMR52 is based on the original program (HMR52) developed by the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) in 1987, with enhanced input and output user interfaces. 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 areal averaged rainfall depth for the 72-hr All Season PMP and Cool Season PMP on the Hudson River at IPEC watershed to IPEC were 10.8 inches and 8.7 inches, respectively.

3.2.2.1.4 Add 100-year snowpack melt rate to Cool Season PMP

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 Hudson

River at IPEC watershed. Due to the mountainous characteristics of the watershed, which varies in elevation from sea level to approximately 5,000 ft (ESRI, 2011), the 100-year snowpack depth was calculated on a topographic / elevation basis. The elevation bands were defined in 1,000-foot increments within the watershed to represent the variation of snowpack given the topographic range of the watershed and the availability of historic snow climatology data.

The 100-year snowpack melt rate was calculated using the energy budget method as outlined in the Army Corps of Engineers EM1110-2-1406 (USACOE, 1998). Hourly snowmelt rates were calculated for each subwatershed, based on the percentage of each elevation band in the subwatershed. An additional 4.6 inches of snow-water equivalent (varies by subwatershed location) was added to the 72-hour Cool Season PMP. The Cool Season PMP with snowmelt was found to be the controlling PMP because it represents a larger water equivalent input than the All Season PMP.

3.2.2.1.5 Develop HEC-HMS model subwatersheds, dams, tributary streams, and Hudson River reaches

The Hudson River at IPEC watershed was delineated into 21 subwatersheds (Figure 3.2-1) based on the location of (a) major tributaries to the Hudson River; (b) significant dams; and (c) U.S. Geological Survey (USGS) stream gages. Three significant dams were incorporated into the HEC-HMS model based on relatively large structure height and reservoir storage capacity: Olive Bridge Dam (Ashokan Reservoir), Merriman Dam (Rondout Reservoir), and Conklingville Dam (Lake Sacadaga). Reservoir elements were used in the HEC-HMS model to account for attenuation due to storage behind the dam. Significant reaches were incorporated into the HEC-HMS model based on riverine cross section width, depth, and flow length. Reach elements were used to convey flows and account for travel time, attenuation, and flow translation within the Hudson River.

3.2.2.1.6 Calibrate and Verify HEC-HMS model

USGS stream flow records were gathered for three to four calibration storms and three verification storms for each gaged subwatershed. Corresponding NCDC precipitation data was gathered to create area-weighted precipitation hyetographs corresponding for each calibration and verification storm. The HEC-HMS model used the Snyder Unit Hydrograph method to transform excess precipitation into direct runoff. The rainfall-runoff model was calibrated to observed 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 of subreaches. Verification of each of the calibrated subwatershed was performed, and the results were compared to observed USGS stream flow data. The verified parameters were bounded by the calibrated Snyder parameters and constant loss rates. Verification results for each subwatershed illustrated acceptable agreement between observed and modeled peak flow and total flow volume. Verified subwatershed and reach parameters are shown in Table 3.2-1 and 3.2-2. The input parameters for the ungaged subwatersheds were estimated based on the verified parameters for the gaged subwatersheds.

3.2.2.1.7 Perform PMF simulation with PMP input using calibrated and verified HEC-HMS model

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 with the controlling PMP. The Cool Season PMP with snowmelt is larger and is controlling over the All Season PMP with 40% antecedent storm summarized in Table 3.2-3. The PMF peak discharge in the Hudson River at IPEC calculated using HEC-HMS with non-linearity adjustments is 1,213,800 cfs. The PMF flow hydrograph developed in HEC-HMS is then routed using the HEC-RAS hydraulic model described in Section 3.2.2.1.8.

3.2.2.1.8 Develop HEC-RAS unsteady flow hydraulic computer model

A hydraulic computer model (HEC-RAS v4.1) was developed for a 76-mile-long reach of the Hudson River near IPEC. A total of 104 cross sections were used in the HEC-RAS model. 79 cross sections were developed from Digital Elevation Maps (CUGIR, 2012), bathymetric data (NYOGLECC, 2012) and USGS topographic data (ESRI, 2011) using GIS. The HEC-RAS hydraulic model extends 24 miles upstream of the site and 52.1 miles downstream from the site.

3.2.2.1.9 Calibrate HEC-RAS Model

Model calibration was performed to refine 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. Two recorded riverine floods and two tidal cycles were selected for use to calibrate the HEC-RAS hydraulic model. Selection of calibration events was limited by the period of record (i.e., data availability) of available USGS Stream Gage 01372058, Hudson River below Poughkeepsie, NY (USGS, 2012a), USGS Stream Gage 01376304, Hudson River South of Hastings-on-Hudson, NY (USGS, 2012b) and NOAA tidal gage stations 8518924 Haverstraw, NY (NOAA, 2012) within the model limits. The HEC-RAS model was calibrated by uniformly adjusting the Manning's-n values of the main channel until the resultant peak water surface elevations at USGS Station 01376304, Hudson River South of Hastings-on-the-Hudson, NY were generally within one foot of the peak observed historical data.

3.2.2.1.10 Perform PMF Hydraulic Simulations

The controlling PMF output hydrograph (with non-linearity adjustments of 1,213,800 cfs) from HEC-HMS was input and routed in the calibrated HEC-RAS model to establish peak flood elevations (AREVA, 2013a). The peak PMF elevation on the Hudson River near IPEC was calculated to be 14.6 feet, which is 0.4 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

3.2.3 Conclusions

The probable maximum flood in the Hudson River at IPEC after hydraulic routing within HEC-RAS is conservatively calculated at 1,185,300 cfs. Historical records do not indicate flooding in excess of this PMF flow.

The peak PMF water surface elevation at Indian Point Energy Center is 14.6 feet, which is 0.4 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

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

3.2.4 References

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

AREVA, 2013a. "Probable Maximum Flood on Hudson River – Hydrology", AREVA Document No. 32-9196315-000, 2013.

AREVA, 2013b. "Indian Point Energy Center Flood Hazard Re-evaluation – Probable Maximum Flood on Hudson River – Hydraulics", AREVA Document No. 32-9196316-000, 2013.

AREVA, 2013c. "Probable Maximum Precipitation", AREVA Document No. 32-9196314-000, 2013.

Chow, 1964. Chow, Ven Te Handbook on Applied Hydrology, McGraw-Hill, New York, 1964.

CUGIR, 2012. Digital Elevation Model (DEM), CUGIR: Cornell University Geospatial Information Repository, Website: http://cugir.mannlib.cornell.edu/mapsheet.jsp?code=DD47, accessed December, 2012, See AREVA Document No. 32-9196316-000.

ESRI, 2011. ESRI ArcGIS Online World Topographic Map service, Published February 2011 by ESRI ARCIMS services.

HMR-51, 1978. "Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", Hydrometeorological Report No.51 (HMR-51), US Department of Commerce & U.S. Army Corps of Engineers, June 1978

HMR-52, 1982. "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", NOAA Hydrometeorological Report No.52 (HMR-52), US Department of Commerce & U.S. Army Corps of Engineers, August 1982

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

NOAA, 2012. National Oceanic and Atmospheric Administration (NOAA) Station 8518924 Haverstraw Bay, NY, Website: http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=8518924 Haverstraw Bay, NY&type=Historic+Tide+Data, accessed December 2012, See AREVA Document No. 32-9196316-000.

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)

NYOGLECC, 2012. Hudson River Bathymetry, New York Ocean and Great Lakes Atlas, New York Ocean and Great Lakes Ecosystem Conservation Council (NYOGLECC), Website: http://atlas.oglecc.ny.gov/index.cfm, accessed December, 2012, See AREVA Document No. 32-9196316-000.

USACOE, 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 01372058 Hudson River below Poughkeepsie, NY, website: http://waterdata.usgs.gov/nwis/uv?agency_code=USGS&site_no=01372058, accessed December 2012, See AREVA Document No. 32-9196316-000.

USGS, 2012b. U.S. Geological Survey (USGS) Stream Gage 01376304 Hudson River South of Hastings-on-Hudson, NY, website: http://waterdata.usgs.gov/nwis/inventory?agency_code=USGS&site_no=01376304, accessed December 2012, See AREVA Document No. 32-9196316-000.

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

USACE, 2010. HEC-RAS River Analysis System Hydraulic Reference Manual Version 4.1– CPD-69, USACE Institute of Water Resources – Hydrologic Engineering Center, January 2010.

Table 3.2-1: Verified Subwatershed Parameters

	Standard Lag (hr)	Peaking Coefficient, Cp	Constant Loss Rate (in/hr)	Initial Loss Rate (in)
Hudson North Creek	21	0.40	0.040	0.00
Hudson Hadley	18	0.40	0.170	0.00
Upper Sacandaga	8	0.40	0.088	0.00
Upper Mohawk	20	0.40	0.068	0.00
Schoharie	19	0.49	0.060	0.00
Lower Mohawk	15	0.60	0.100	0.00
Hudson Green Island	18	0.55	0.035	0.00
Upper Ashokan	5	0.50	0.120	0.00
Lower Esopus	10	0.50	0.080	0.00
Lower Rondout	9	0.48	0.080	0.00
Wallkill	13	0.40	0.120	0.00
Wappinger	22	0.50	0.105	0.00
Lower Sacandaga – Ungaged	10	0.40	0.130	0.00
Catskill – Ungaged	10.1	0.48	0.050	0.00
Kinderhook – Ungaged	11.2	0.48	0.100	0.00
Lower Ashokan East - Ungaged	1.9	0.48	0.030	0.00
Lower Ashokan West - Ungaged	3.6	0.48	0.030	0.00
Upper Rondout - Ungaged	4.9	0.50	0.108	0.00
Lower Hudson - Ungaged	27.9	0.55	0.034	0.00

Table 3.2-2: Verified Reach Muskingum Routing Parameters

	Travel time, K (hr)	Weight, X	Subreaches, N
Hudson North Creek to Hudson Hadley	6	0.042	12
Upper Mohawk to Schoharie	6	0.042	12
Schoharie to Lower Mohawk	8	0.031	16
Lower Sacandaga to Hudson Green Island	12	0.021	24
Lower Ashokan East to Lower Esopus	15	0.014	36
Upper Rondout to Lower Rondout	4	0.063	8

Table 3.2-3: PMF Simulations

Simulation	All Season PMF (from HEC-HMS)	Cool Season PMF (from HEC-HMS)	Cool Season PMF, Adjusted for Non-linearity (from HEC-HMS)	Cool Season PMF, Adjusted for Non-linearity, Routed Through HEC-RAS
Peak Flow	984,700 cfs	1,142,500 cfs	1,213,800 cfs	**1,185,300 cfs**

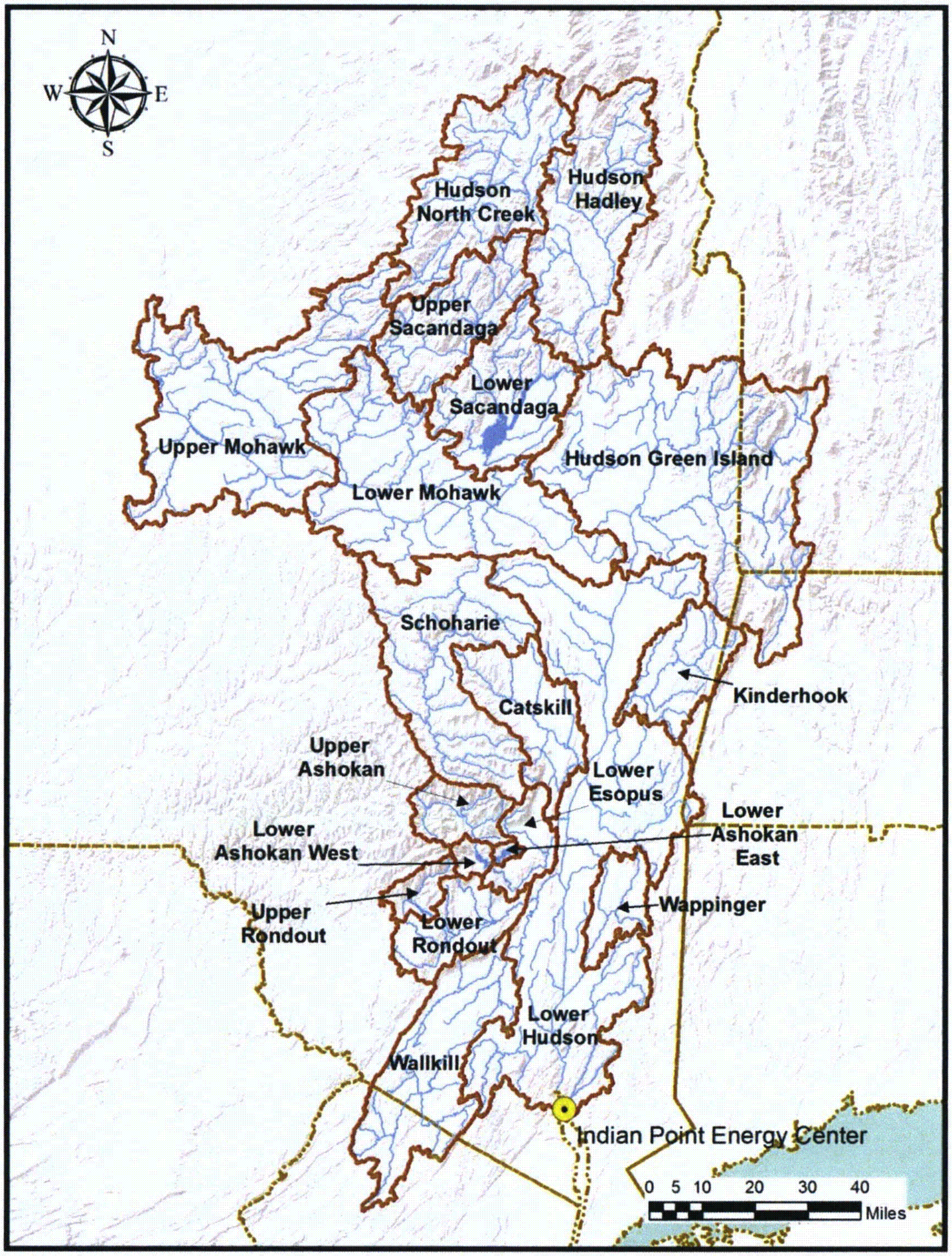

Figure 3.2-1: IPEC Subwatersheds (AREVA, 2013a)

3.3 Dam Breaches and Failures

This section addresses the effects of upstream dam failures on the maximum water surface elevation at Indian Point Energy Center (IPEC). There are no on-site dams or levees which could impact site safety if breached (IPEC, 2012).

3.3.1 Method

The hierarchical hazard assessment (HAA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of upstream dam failures on the maximum water surface elevation at IPEC. The HHA approach is a progressively refined, stepwise estimation of site-specific flood hazards, starting with the most conservative simplifying assumptions that maximize flood hazards. If the site is not inundated by the flood mechanism evaluated, a conclusion that the SSCare not susceptible to flooding is valid and no further analyses were completed (NRC, 2011). Thus, the flood elevations herein are conservative and intended to demonstrate the safety of SSC against flooding from upstream dam failure, and not intended to represent the most realistic estimation of flood elevations due to flooding from upstream dam failure.

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 (ANS,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., group of dams no 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 Hudson River, the following steps were used:

1. Identify upstream dams within the IPEC watershed (NYDEC, 2012) and screen for significant dams with either significant height (100 feet or higher) or storage (200,000 acre-feet or greater). Note that dams that were not overtopped were not assumed to fail under the hydrologic dam failure scenario.
2. Develop a conservative dam failure scenario and calculate the peak breach outflow using HEC-HMS from the screened dams identified in Step 1:

   a. Identify dams located within approximately 200 miles upstream of IPEC with an estimate peak breach flow of approximately 1,000,000 cfs.
   b. Identify dams located 200 or more miles upstream of IPEC with an estimate peak breach flow of approximately 1,000,000 cfs.
   c. Identify a series of dams nearby IPEC which might fail in a domino-like sequence.
3. Perform combined dam breach outflow and PMF hydraulic simulation using HEC-RAS

Flooding due to dam failures was evaluated in a manner consistent with that proposed by NRC, 2013. That includes screening of dams for impact and conservative selection of significant dams and peak outflows, and use of the HEC-HMS and HEC-RAS computer models (USACE, 2000, USACE, 2010). Consistent with NRC 2013a, IPEC studies used ASCE, 2010 for evaluation of debris loads.

3.3.2 Results

3.3.2.1 Dam Failures (AREVA, 2013)

3.3.2.1.1 Identify upstream dams and screen for overtopping potential

The total number of dams in the Hudson River watershed exceeds 1,000 (NYDEC, 2012). From this list of dams, there were eleven upstream dams with either significant height (100 feet or higher) or storage (200,000 acre-feet or greater). Three dams (Merriman Dam, Olive Bridge Dam, and Conklingville Dam) were included in the dam failure analysis based on proximity to IPEC and estimated breach flow (1,000,000 cfs or higher) shown in Figure 3.3-1. An additional four dams (Lake Te-Ata Dam, Lake Popolopen Dam, Mine Lake Dam and Stillwater Dam) in series near IPEC, shown in Figure 3.3-2, were included for the cascade failure simulation.

3.3.2.1.2 Develop a conservative dam failure scenario and estimate peak breach outflow using HEC-HMS

Separate HEC-HMS models were used to develop a conservative, representative dam breach model and an IPEC Watershed model for the hydrologic and seismically-induced upstream dam failure events. Conservative dam breach parameters were used to estimate peak breach flow and are shown in Table 3.3-1 (FERC, 1993).

Hydrologic (PMF Event): Pool elevations behind Olive Bridge and Merriman Dams did not reach the top of dam during the PMF; therefore, dam failure of these structures was not initiated during the PMF. Popolopen Dam and Mine Lake Dam were overtopped during the PMF simulation. The peak dam breach outflow for the dams in series was 15,670 cfs, 3,960 cfs, 7,690 cfs and 41,880 cfs for Lake Te-Ata Dam, Lake Popolopen Dam, Mine Lake Dam and Stillwater Dam respectively.

Hydrologic (PMF Event with Forced Conklingville Dam Failure): Conklingville Dam also did not overtop during the PMF. However, an additional dam failure analysis was performed to assess the effect of a representative upstream dam failure on the water surface elevation at IPEC. Failure was forced at Conklingville Dam during the PMF as it impounds the largest volume of water in the IPEC watershed. The peak dam breach outflow from the Conklingville Dam breach was 969,500 cfs. The peak dam breach outflow for the dams in series was the same as listed above.

Seismically-Induced Event: Peak dam breach flow for individual large dams under the seismic failure scenario (1/2 PMF) was 4,080,000 cfs, 1,647,400 cfs and 733,800 cfs for the Merriman Dam, Olive Bridge Dam, and Conklingville Dam respectively.

A separate ½ PMF domino failure of the four dams in series was not performed. The peak dam breach outflow from the full PMF was selected for use because it was conservative

3.3.2.1.3 Perform combined dam breach outflow and PMF hydraulic simulation using HEC-RAS

Hydrologic (PMF Event): The calculated peak water surface elevation from the hydrologic event (the combined dam breach outflow and PMF) in the Hudson River at Indian Point Energy Center, based on a maximum discharge after hydraulic routing within HEC-RAS of 1,186,300 cfs, was 14.6 feet, which is 0.4 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

Hydrologic (PMF Event with Forced Conklingville Dam Failure): The calculated peak water surface elevation from the PMF and forced failure of the Conklingville Dam (during the PMF) in the Hudson River at Indian Point Energy Center, based on a maximum discharge after hydraulic routing within HEC-RAS of 1,217,700 cfs, was 14.9 feet, which is 0.1 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

Seismically-Induced Event: The calculated peak water surface elevation from seismically induced dam failures (the combined dam breach outflow and ½ PMF) in the Hudson River at Indian Point Energy Center, based on a maximum discharge after hydraulic routing within HEC-RAS of 1,121,300 cfs was 13.5 feet, which is 1.5 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

3.3.3 Conclusions

The controlling dam failure scenario is the hydrologic event (PMF) with the forced failure of Conklingville Dam. Combination of the dam breach outflow and PMF at IPEC after hydraulic routing is calculated to be 1,217,700 cfs. The resultant peak water surface elevation from the combined dam breach peak outflow and PMF in the Hudson River at Indian Point Energy Center is 14.9 feet, which is 0.1 feet below the plant grade elevation of 15.0 feet.

The hydrologic event (PMF) with the forced failure of the Conklingville Dam was representative of the individual dams upstream of the Conklingville Dam. This scenario demonstrated that a failure of a dam upstream of the Conklingville Dam would have a minimal effect on the peak water surface elevation at IPEC. The total volume of the impoundments for the significant screened-out dams is approximately 510,000 acre-feet which is less than the large reservoir impounded by Conklingville Dam, Lake Sacandaga, about 1.2 million acre-feet at normal pool. Given the considerable distance upstream of IPEC, the peak dam breach outflow for dams 200 miles upstream and beyond will attenuate as the flood wave travels downstream. Therefore, the combined volume of the dam failures was judged to be more important than the localized peak breach outflow from the dam. The impacts of the failure of the Conklingville Dam are therefore judged to bound the impacts of the failure of the other significant dams 200 or more miles upstream of IPEC. The outflow hydrograph at IPEC for the forced failure of Conklingville Dam is shown in Figure 3.3-3.

The dams included in the domino failure event were the closest dams in series upstream of IPEC. The combined failure of these four dams resulted in a much smaller peak flow compared to the peak flow of the PMF and the peak is not coincident with the PMF peak demonstrating that it is unlikely that the failure of other small dams would affect the peak flow of the PMF at IPEC.

The seismic scenario (dam failure coincident with the ½ PMF) demonstrated that due to the natural timing differences (from varying travel times of the dam breach and natural flood wave from disparate upstream sources) the failure of significant dams which are closer to IPEC is expected to result in peak water surface elevations that are bounded by the other dam failure scenarios. The outflow hydrograph at IPEC for the seismic scenario is shown in Figure 3.3-4.

Based on the re-evaluation of upstream dam failures on the Hudson River, the peak water surface elevations on the Hudson River at IPEC resulting from the failure of dams under both the Hydrologic and seismically-induced events are below the plant grade elevation.

3.3.4 References

ASCE, 2010. "Minimum Design Loads for Buildings and Other Structures", ASCE/SEI 7-10, American Society of Civil Engineers (ASCE), 2010.

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

AREVA, 2013. "Dam Failures", AREVA Document No. 32-9196320-000, February 2013.

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.

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

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)

NRC, 2013. "Japanese Lessons Learned Project Directorate, JLD-ISG-2013-01, Guidance for Assessment of Flooding Hazard Due to Dam Failure, Interim Staff Guidance, Revision 0, July 2013 (ADAMS Accession No. ML13151A153)

NYDEC, 2012. New York State Dam Inventory, New York State Department of Environmental Conservation, file titled "nysdamslink.kmz", accessed October 24, 2012. http://www.dec.ny.gov/maps/nysdamslink.kmz, see AREVA Document No. 32-9196320-000.

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

USACE, 2010. HEC-RAS River Analysis System Hydraulic Reference Manual Version 4.1– CPD-69, USACE Institute of Water Resources – Hydrologic Engineering Center, January 2010.

Table 3.3-1: Dam Breach Parameters (AREVA, 2013)

	Top Elevation	Bottom Elevation	Side Slopes Horizontal Component	Bottom Width	Development Time	Trigger Method	PMF(forced) Trigger Elevation[1]	PMF (forced) Trigger Elevation	Seismically Induced Trigger Elevation[2]	Progression Method
	(ft MSL + 1000)	(ft MSL + 1000)		(ft)	(hour)		(ft MSL + 1000)	(ft MSL + 1000)	(ft MSL + 1000)	
Conklingville Dam	1795	1699	0.5	310.8	0.5	Elevation	N/A	1781.3	1776.7	Linear
Olive Bridge Dam (Ashokan)	1610	1397	0	196.1	0.1	Elevation	N/A	N/A	1593.1	Linear
Merriman Dam	1860	1665	0.5	710.4	0.5	Elevation	N/A	N/A	1842.6	Linear
Lake Te-Ata Dam	1870	1835	0.5	147.6	0.5	Elevation	1867.8	1867.8	1867.8	Linear
Lake Popolopen Dam	1679.8	1663.7	0	16.1	0.1	Elevation	1679.8	1679.8	1679.8	Linear
Mine Lake Dam	1651.1	1633.7	0	17.4	0.1	Elevation	1651.1	1651.1	1651.1	Linear
Stillwell Dam	1617.8	1560.5	0	46.3	0.1	Elevation	1606.8	1606.8	1606.8	Linear

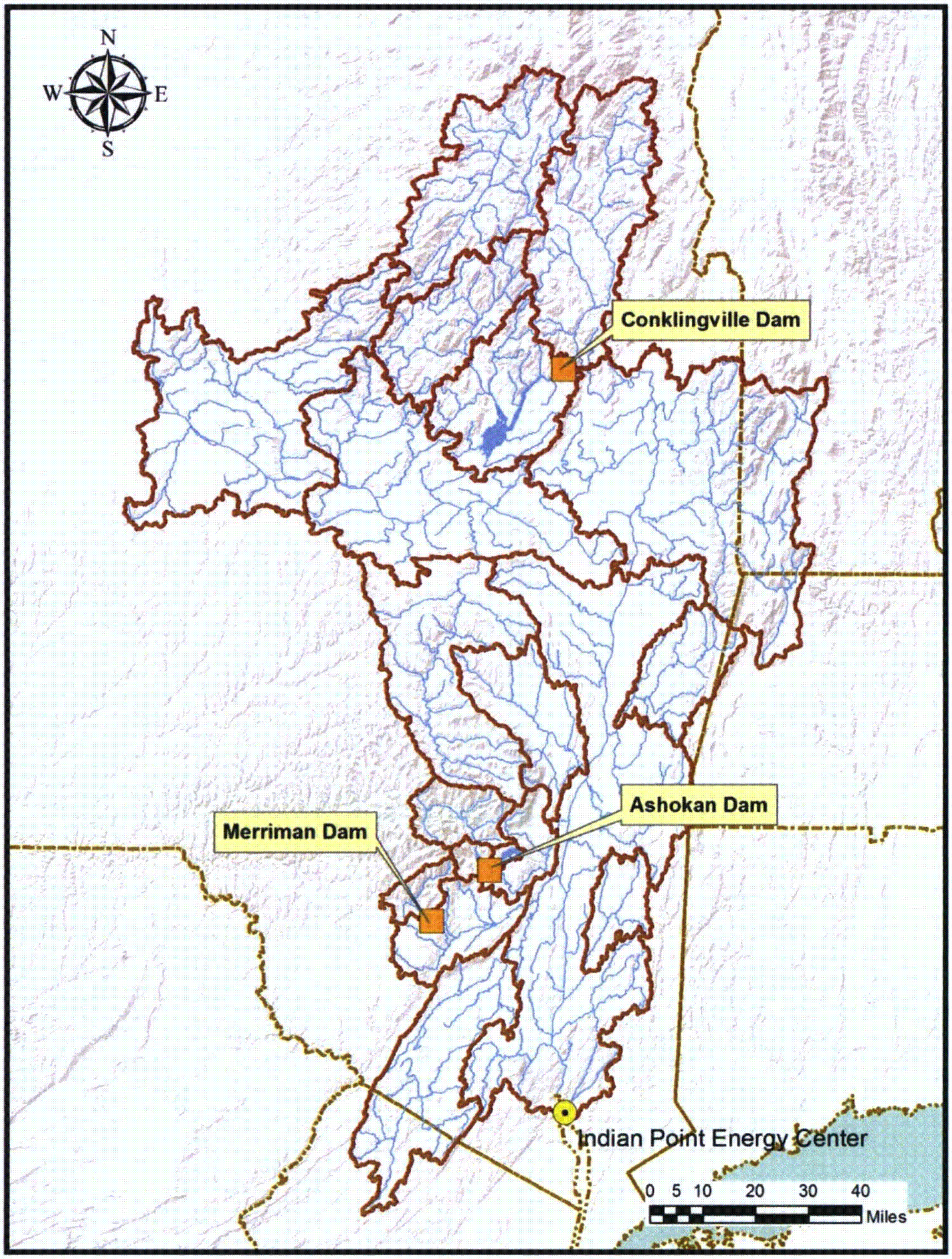

Figure 3.3-1: Dam Locations (AREVA, 2013)

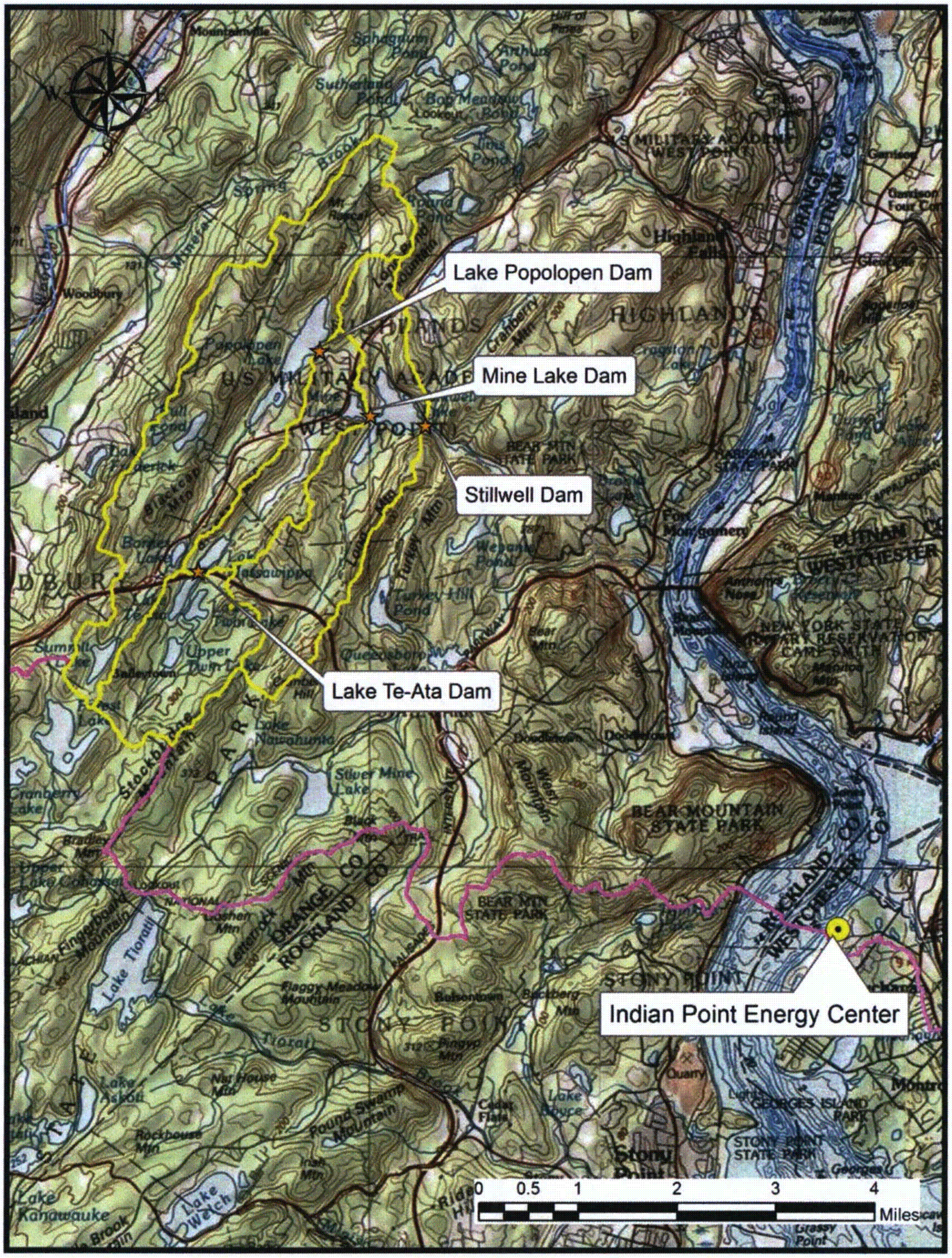

Figure 3.3-2: Domino Dam Locations (AREVA, 2013)

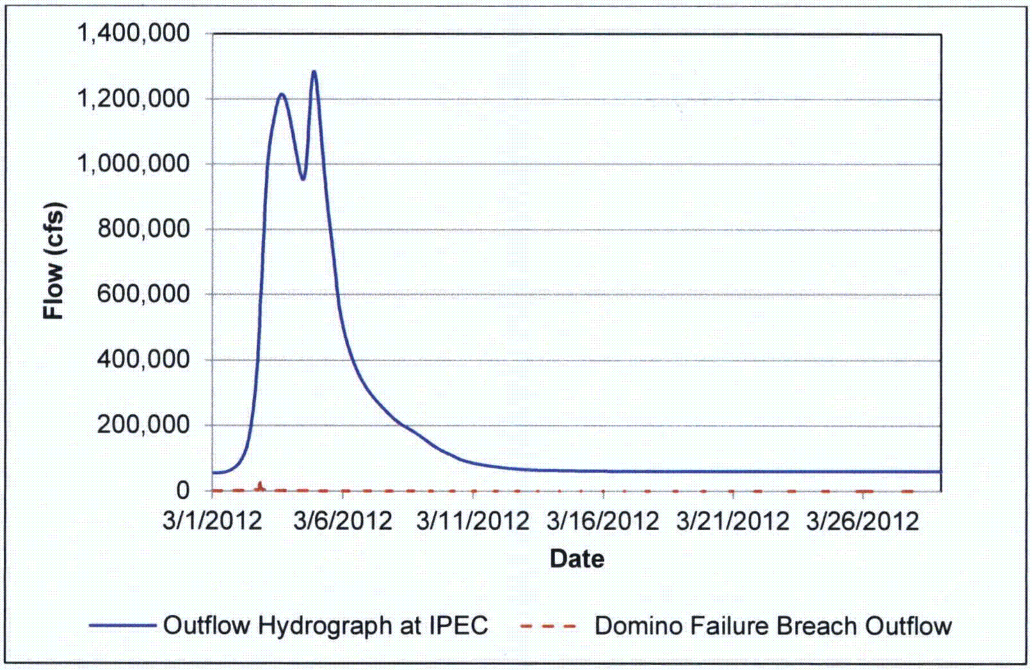

Figure 3.3-3: PMF Scenario Forced Conklingville Dam Failure Outflow Hydrograph (AREVA, 2013)

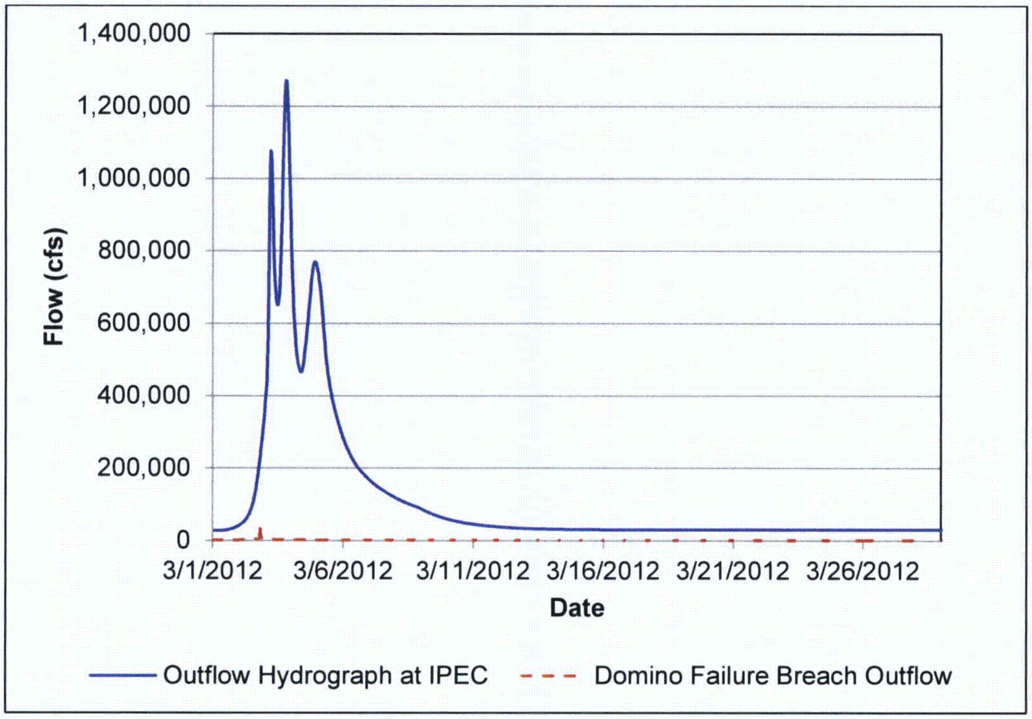

Figure 3.3-4: Seismic Scenario Outflow Hydrograph (AREVA, 2013)

3.4 Probable Maximum Storm Surge

3.4.1 Methodology

The methodology used to evaluate flooding due to storm surge was chosen to be consistent with the IPEC's hydrologic setting (see Section 2.1.2), a complex riverine environment influenced by coastal processes. Hydrologic features pertinent to the methodology used for storm surge assessment are described in Section 2.0. The area of New York Harbor, the New York Bight and the Hudson River have unique characteristics that influence storm surge forces.

Consistent with guidance presented in NRC, 2011, NRC 2012, and NRC, 2013 (NUREG/CR-7046, NUREG/CR-7134 and ISG JLG 2012-06), two analytical approaches were used to evaluate Probable Maximum Storm Surge (PMSS) at IPEC: 1) a deterministic analysis (AREVA, 2013d); and 2) a probabilistic-deterministic analysis (AREVA, 2013e). The deterministic analysis determined a bounding flood level without a defined probability of occurrence. The probabilistic-deterministic analysis developed a flood stage-frequency relationship and identified the flood-stage associated with an annual exceedance probability range of from about $2x10^{-6}$ to $1.5x10^{-7}$.

Unless otherwise noted, elevations cited in this report section refer to both NGVD29 and NAVD88 vertical datums. All IPEC plant documents use NGVD29. Computer analyses performed for this report and their results use NAVD88 as a standard. To convert elevations from NAVD88 to NGVD29 at IPEC, add 1.0 foot to the NAVD88 elevations. To convert elevations at The Battery, NY, to the NGVD29 add 1.1 feet to NAVD88 elevations (NGS, 2013).

As discussed below, a hurricane (tropical cyclone) was identified as the controlling storm event resulting in the largest storm surge elevation at IPEC. A site and region specific hurricane meteorological study (AREVA, 2013a) was performed to define the meteorological storm parameters for both the deterministic analysis and probabilistic-deterministic analyses. AREVA, 2013a involved a statistical analysis of the National Oceanic and Atmospheric Administration (NOAA) historical HURDAT hurricane database (HURDAT2) (NOAA, 2013a). Due to the relatively few historical hurricanes that have impacted the vicinity of IPEC, the historical database was supplemented by a set of synthetic storms that were generated using intensity and atmospheric models verified through statistical comparison to the historical storm database (AREVA, 2013a). Two-dimensional hydrodynamic computer software programs, were used to simulate the storm surge: 1) the Sea, Lakes and Overland Surges from Hurricanes (SLOSH) model (NOAA, 2012a, b); and 2) the ADvanced CIRCulation (ADCIRC) model (USACE, 1992).

The methodologies used to determine the PMSS for IPEC are those defined in NRC, 2011:

1. Controlling Storm Event. Selection of the controlling storm event resulting in the largest storm surges at IPEC.
2. Meteorological Parameters. Development of the meteorological hurricane parameters (central pressure, forward speed, forward direction, radius of maximum winds), initially using the National Weather Service Report NWS 23 (NOAA, 1979) and subsequently based on the results of the site and region specific hurricane meteorological study (AREVA, 2013a), including:
    a. The range of meteorological parameters defining the Probable Maximum Hurricane (PMH) for the deterministic analysis of PMSS; and
    b. Parameter probability density functions used in the probabilistic-deterministic analysis of storm surge.

3. Antecedent Water Levels. Development of the antecedent water levels used for the storm surge simulations.
4. Analysis of the 25-year recurrence period river flood using statistical analysis of observed water level data in the Hudson River.
5. Deterministic analysis of the PMSS stillwater and storm-tide elevations, including a storm surge sensitivity analysis using the SLOSH model and final storm surge analyses using the ADCIRC model including storm surge, tide and river flooding.
6. Probabilistic/deterministic analysis using the Joint Probability Method (JPM), including about 32,000 storm surge simulations performed using the SLOSH model, and final storm surge simulations using the ADCIRC model, which included storm surge, tide and river flooding components.

The net effect of the methodology used was to add conservatism to the analyses such that analytical biases are directed toward over-predicting, not under-predicting, flood elevations.

Sources of uncertainty include several general components, or variables:

a) Storm characteristics which define the meteorological forcing causing the storm surge;
b) Physical characteristics of the shoreline, bottom, conditions, etc. that hydraulically affect the propagation of the storm surge;
c) Tidal flow that acts commensurate with the storm surge;
d) River flood flow (which is a function of the precipitation and run-off occurring during the storm) that acts commensurate with the storm surge; and
e) Wind-generated waves that act commensurately with the storm surge and wind conditions.

Each of these components has both random variability and uncertainty.

Additional uncertainty is associated with meteorological and climatological data used to quantitatively characterize the meteorological storm parameters including:

f) Storm track direction;
g) Pressure deficit;
h) Radius of maximum wind; and
i) Storm translational speed.

Finally, additional sources of uncertainty include:

j) Use of hydrodynamic models used with inherent epistemic uncertainty
k) Limited hurricane history for the site region
l) Generation of synthetic storms as the major evaluated storm data set.

Although the methodology for the storm surge analysis was designed to mitigate uncertainties and variability to the extent practical, results presented have used parameters which are on or somewhat above the upper bounds of what is considered meteorologically possible and therefore additional analyses may provide further refinement of the results, which would only result in lower surge elevations. In addition, an established consensus method, an industry standard approach, or a regulatory-based standard do not exist for probabilistic analysis of very low probability flood events at this time. IPEC chooses to adopt the results for a $2x10^{-6}$ annual probability of occurrence, a probability level which is considered to be suitably conservative to establish a beyond-design-basis storm surge event for IPEC.

3.4.2 Results

3.4.2.1 Controlling Storm Event

The controlling storm event (i.e., the type of storm that is expected to result in the largest storm surge at IPEC) was identified by: 1) review of historical extreme water level data from the NOAA Co-Op Stations from both The Battery, New York City, NY (NOAA, 2013b) and Sandy Hook, NJ (NOAA, 2013c); and 2) comparison of hurricane storm surge elevations predicted by NOAA to the observed water levels at the two NOAA Co-Op stations. See Figure 3.4-1 for the NOAA station locations.

The water levels represented by the 20 highest monthly water levels measured at The Battery, New York City, NY (data available for the period of 1932 to 2013) and Sandy Hook, NJ (data available for the period of 1944 to 2012) stations, are presented on Tables 3.4-1 and 3.4-2, respectively. The storm type associated with these water levels was determined based on reference to NOAA historical hurricane data (NOAA, 2013a) and other sources (NOAA, 2008a; NASA, 1998). As shown on Tables 3.4-1 and 3.4-2, three types of events have resulted in coastal storm surges at the two NOAA Co-Op stations: a) extra-tropical storms (aka Nor'easters), b) tropical storms and c) hurricanes. Although most (fourteen) of the twenty highest water levels at The Battery and Sandy Hook were caused by extra-tropical storms, three of the five highest water levels (including the highest) were caused by tropical storms or hurricanes.

Figure 3.4-2 shows the locations of historical hurricane strikes for the New York and New Jersey area. Figure 3.4-3 shows the hurricanes that resulted in the recorded high water levels at The Battery and Sandy Hook.

The historic storm surge data is limited. Thus to evaluate whether larger hurricanes or extra-tropical storms are likely to represent the controlling storm, storm surge elevations predicted by NOAA, 2012b for Category 1 through 4 hurricanes, were compared to the historical water level data.

Predicted storm surge elevations were provided in the NOAA "SLOSH Display Program" run by NOAA (NOAA, 2012b). The SLOSH Display Program provides two types of composite output: 1) the Maximum Envelope of Water (MEOWs); and 2) the Maximum of MEOWs (MOMs). MEOWs are a composite that represent the maximum storm surge height at each SLOSH grid cell for hypothetical hurricanes with parallel tracks and constant category, forward speed, landfall direction, and initial tide level (NOAA, 2012b and NRC, 2011). MOMs represent the maximum storm surge height at each SLOSH grid cell for all hurricanes simulated for a given hurricane category, regardless of other hurricane parameters, and represent potential worst case flooding for each hurricane category (NOAA, 2012b and NRC, 2011).

Table 3.4-3 presents the MOM values at mean tide for hurricane Categories 1 through 4 for the SLOSH cells containing the NOAA Co-Op stations for the Battery and Sandy Hook, NJ. A comparison of the MOM values to the recorded water levels in Table 3.4-1 indicates that historic extra-tropical storms have caused storm surges roughly equivalent to those predicted for a simulated Category 1 hurricane, and that the recorded water levels resulting from historic hurricanes have caused storm surges roughly equivalent to those predicted for simulated Category 1 and 2 hurricanes. The predicted surges due to more intense hurricanes, representative of the Probable Maximum Hurricane, create larger surges than those documented for extra-tropical storms, tropical storms or Category 1 and 2 hurricanes.

In summary, it is concluded that a major hurricane will likely be the controlling storm event creating storm surge at IPEC. This is based on: 1) review of historical extreme water level data from the NOAA Co-Op Stations from both The Battery, NY and Sandy Hook, NJ; and 2) examination of the extreme water level events associated with predicted hurricane storm surge elevations produced by NOAA.

3.4.2.2 Development of the Meteorological Parameters

The meteorological hurricane parameters (e.g., central pressure, forward speed, etc.) were developed to establish: 1) the range of parameters defining the PMH for the deterministic analysis; and 2) the parameter probability density functions used for the probabilistic-deterministic analysis of storm surge. The meteorological hurricane parameters developed using the National Weather Service Report NWS 23 (NWS 23) (NOAA, 1979) were subsequently revised based on the results of the Site and Region Specific Hurricane Meteorological Study (AREVA, 2013a).

The PMH is the "hypothetical steady-state hurricane with a combination of values of meteorological parameters that will give the highest sustained winds speed that can probably occur at a specified coastal location" (NOAA, 1979). The meteorological parameters that define the PMH wind field include a) the hurricane peripheral pressure, b) central pressure, c) radius of maximum winds, d) forward speed, and e) track direction. Although IPEC is located on the Hudson River approximately 50 miles from the mouth of the river at Raritan Bay, the storm surge at IPEC is induced principally by the coastal storm surge in the vicinity of the mouth of the river. Thus the ranges of meteorological parameters (using NWS 23) used to evaluate storm surge were initially selected for that coastal location. The range of meteorological parameter values determined using NWS 23 are presented below:

Parameter	Lower Bound	Upper Bound	Units
Peripheral Pressure (P_w)	30.12	30.12	Inches
Central Pressure (P_o)	26.65	26.80	Inches Hg
Radius of Maximum Winds (R_w)	12	31	Nautical miles
Forward Speed (T)	12	31	Knots
Track Bearing	-35	10	Degrees
Track Direction (θ)[1]	75	190	Degrees

Note: 1. Track direction is the direction from which the storm comes, measured clockwise from north.

The parameters selected from NWS 23 were compared with the comprehensive set of hurricane climatology statistics developed in NWS 38 (NOAA, 1987) for the Atlantic and Gulf Coasts of the United States for hurricanes during the period of 1900 through 1984. The trends and statistics associated with historic land-falling hurricanes confirm that the meteorological parameters developed in NWS 23 for the PMH are conservative; however, certain of the parameters appear to fall beyond the ranges indicated by the historical hurricane database and do not appear to be consistent with the current meteorological science.

A site and region specific study was performed to further evaluate and refine the meteorological parameters based on the current state of knowledge (AREVA, 2013a). That study performed a detailed statistical analysis of a hurricane database including both historical hurricanes and synthetic storms generated using atmospheric and intensity models.

Observed Atlantic hurricane data for the IPEC site region, documented in the HURDAT2 database (NOAA, 2012c), covers the historical period of record from 1851 through 2012. The HURDAT2 database contains 162 years of 6-hourly storm track positions and corresponding 5-minute average maximum winds. Central pressure data is also available for the period from 1979-2012. There are 1,705 Atlantic tropical storms and hurricanes documented in the full data set, and, in the post-1979 period, 505 storms are documented.

A substantially larger sample of synthetic hurricane data, filtered to include storms that pass within a 200km radius of IPEC, was developed using hurricane intensity models and atmospheric models (AREVA, 2013a). The synthetic data served to supplement the relatively limited period of record and the limited number of high intensity storms in the HURDAT2 database that impacted the IPEC site region.

The statistical analysis of the hurricane database utilized two processes.

The first process involved analysis of a subset of storm parameters available from the HURDAT2 database (NOAA, 2012c), focused on storm parameters within a 5° x 5° coastal region that encompasses the New Jersey, New York and the southern New England coasts (referred to as the IPEC study region). First, univariate probability distributions (PDFs) were calculated for the following storm parameters:

1. Maximum Sustained (1-minute average) Wind Speed
2. Forward Translation Speed
3. Storm Bearing
4. Central Pressure Deficit (CPD)
5. 6-hourly changes in intensity as indicated by the change in maximum sustained wind speed

Extreme value statistics (EVS) were performed to estimate the (very low) probability of strong hurricanes in the IPEC study region. Extreme value distributions were fitted from the subset of hurricanes equal to, or exceeding, Category 3 on the Saffir-Simpson hurricane scale. The EVS analysis supplemented the PDF estimates of the probability of extreme winds. Next, the co-variability of relevant storm parameter combinations was examined, since substantial co-variability among hurricane parameters could result in an increase in the probability of parameter combinations over that which would be determined for independent parameters.

The second process used the synthetic hurricane data to develop a large sample of hurricanes directly impacting the vicinity of New York Harbor. The sample size of over 5,000 synthetic storms allowed direct calculation of storm surge probabilities with all parameter co-variability inherently accounted for. The representativeness of these hurricane parameter data was examined within the context of the co-located HURDAT2 data to confirm the consistency with the empirical data (i.e., to validate the synthetic storm set relative to the historical data).

Based on AREVA, 2013a, revision of certain hurricane parameters presented in NWS 23 (NOAA, 1979) is recommended. Two different sets of parameters were developed based on two storm approach angles (northerly and westerly) to capture the range of storms that could cause the PMSS. The two sets of parameter ranges reflect the significant change in hurricane parameters as the storm bearings change from a northerly direction to a westerly direction.

The following PMH parameter ranges are applicable for storms moving in a northerly direction (northeast to northwest, with 10 to -35 degrees bearing):

Parameter	Lower Bound	Upper Bound	Units
Peripheral Pressure (P_w)	30.12	30.12	Inches
Central Pressure (P_o)	27.17	27.17	Inches
Maximum 1-min Sustained Wind	128	128	knots
Radius of Maximum Winds (R_w)	15	25	Nautical miles
Forward Speed (T)	30	45	knots
Track Bearing	-35	10	Degrees
Track Direction (θ)	145	190	Degrees

The following PMH parameter ranges are applicable for storms moving in a westerly direction (west of bearing -40 degrees bearing):

Parameter	Lower Bound	Upper Bound	Units
Peripheral Pressure (P_w)	30.12	30.12	Inches
Central Pressure (P_o)	28.17	28.17	Inches
Maximum 1-min Sustained Wind	86	86	knots
Radius of Maximum Winds (R_w)	10	40	Nautical miles
Forward Speed (T)	10	35	knots
Track Bearing[1]	-105	-35	Degrees
Track Direction (θ)	75	145	Degrees

Note 1: Track bearing indicates direction the storm is moving toward (where 0 is due north); Track direction indicates the direction from which the storm is moving (clockwise from north).

3.4.2.3 Development of the Antecedent Water Level

Antecedent water levels were developed for performing storm surge simulations for both the deterministic analysis and the probabilistic-deterministic analyses.

In accordance with NRC, 2011, the PMSS is evaluated coincident with an antecedent water level equal to the 10 percent exceedance high tide plus long term sea level rise. The 10 percent exceedance high tide is defined as the high tide that is equaled or exceeded by 10 percent of the maximum monthly tides over a continuous 21-year period (ANS, 1992). The 10 percent exceedance high tide at The Battery was calculated (AREVA, 2013a) using the Weibull plotting position equation (Stedinger et al., 1993) to be 3.4 feet NAVD88, the sum of 10 percent exceedance high tide (2.75 feet NAVD88) based on predicted astronomical tides, the calculated sea level anomaly (0.18 feet) and the expected 50-year sea level rise (0.46 feet).

The tidal range varies between The Battery and IPEC, with a tidal attenuation factor of 0.64 (NOAA, 2013b). The 10 percent antecedent high tide at IPEC was calculated as:

$$2.75 \text{ feet NAVD88} \times 0.64 + 0.18 \text{ feet} + 0.46 \text{ feet} = 2.40 \text{ feet NAVD88}$$

The average value (elevation 2.9 feet NAVD88) between these two locations was selected as the antecedent water level for SLOSH analyses. For the storm surge simulations using the ADCIRC model, tide input consistent with the 10 percent antecedent high tide, was developed using historical tide data.

The antecedent water levels for the probabilistic-deterministic storm surge model simulations included tides and the potential, long term rate of sea level rise. The peak storm surge at IPEC was modeled to occur with equal probability at high and low tide, with each tide level assigned a probability of occurrence of 0.5. The Mean High Water (MHW) and Mean Low Water (MLW) levels at The Battery (NOAA, 2013a) were used corresponding to elevations 2.0 feet NAVD88 and -2.6 feet NAVD88, respectively.

3.4.2.4 Deterministic Probable Maximum Storm Surge

The deterministic storm surge analysis was performed to: 1) identify the combination of tropical cyclone meteorological parameters and storm tracks that result in the PMSS; and 2) calculate the resulting storm surge stillwater elevation (i.e., storm-tide elevation), including the combined mechanisms of storm surge and tide (AREVA, 2013b). The PMSS is defined as "that surge that results from a combination of meteorological parameters of a probable maximum hurricane, a probable maximum wind storm, or a moving squall line and has virtually no probability of being exceeded in the region involved" (NRC, 2011). Inputs to the PMSS analysis are presented in Section 3.4.2.2 Development of the Meteorological Parameters, and Section 3.4.2.3 Antecedent Water Level).

The Hierarchical Hazard Assessment (HHA, NRC, 2011) approach was used to evaluate the PMSS. Specifically, a storm surge screening level/sensitivity analysis (approximately 1,000 simulations) was performed using SLOSH to identify: 1) the sensitivity of storm surge at IPEC to different storm parameters (i.e., storm track, radius of maximum winds, etc.); and 2) the specific combinations of storm parameters and storm tracks that result in the largest predicted storm surges at IPEC. The SLOSH model is computationally efficient allowing performance of many simulations; however, it has some important limitations including its relatively coarse, structured model grid and it allows only a constant antecedent tide level. Therefore, additional, more detailed simulations were performed using ADCIRC to further evaluate the storms identified by the SLOSH model screening/sensitivity analysis that resulted in large surges at IPEC. ADCIRC results were used to develop the final PMSS elevations, including the effects of tidal flow and river flooding.

Two constraints for ADCIRC inputs are described below. First, the pressure difference input, between minimum central pressure in the hurricane and peripheral pressure was manually adjusted for the critical storms to limit the maximum 10-meter height, 1-minute duration sustained wind velocity to the maximum PMH wind velocity of 128 knots (AREVA, 2013a). Second, while most simulations were performed for steady-state storm conditions, a non-steady state storm surge simulation was performed using ADCIRC for a critical northerly storm to evaluate the effects of overland weakening of the hurricane (i.e., wind velocity) on the storm surge at IPEC. Further the combined effects of storm surge at IPEC and tidal phasing was also evaluated to confirm which combination of phasing storm surge and tide created the highest storm-tide elevation at IPEC.

SLOSH is a dynamic, two-dimensional numerical finite-difference computer model used to calculate storm surge heights and winds resulting from historical, hypothetical, or predicted hurricanes. The SLOSH model uses storm track, pressure deficit, radius of maximum winds, and forward speed to calculate storm surge heights. The model accounts for both the hurricane wind field and the pressure differential when calculating storm surge (NOAA, 1992). The NOAA SLOSH New York Operational Basin (ny3) was utilized for the SLOSH 3.97 simulations. The ny3 SLOSH 3.97 model grid is presented in Figure 3.4-4. The New York basin model Digital Elevation Model (DEM) is presented in Figure 3.4-5.

Figure 3.4-6 presents the portion of the DEM in the site vicinity (IPEC and Upper Harbor, New York). This DEM defines the elevations associated with the base of the SLOSH model domain used in this calculation (relative to the NAVD88 datum). The grid cells around IPEC are shown in Figure 3.4-7.

The primary SLOSH model inputs include:

a. Landfall location
b. Delta-P (difference between central and peripheral barometric pressures) in millibars;
c. T- translational storm speed in knots
d. R_w- radius of maximum winds in miles
e. θ - storm track direction in degrees (°) from North

Landfall points were selected to represent a range of landfall locations both north and south of the New York Bight. The landfall points were established at approximately 20-mile intervals along the coastline as shown in Figure 3.4-8. The synthetic storm tracks analyzed are presented in Figures 3.4-9 and 3.4-10.

The following trends were apparent based on the SLOSH storm surge simulations:

1. The surge elevation at IPEC is very sensitive to storm track direction, with the higher surge elevations calculated for hurricanes tracking from the east-southeast. By inspection, this trend is expected as storms tracking toward Raritan Bay, located at the apex between the Long Island and New Jersey shoreline, will create large surges in New York Harbor with no flow outlet.
2. The surge elevation is also significantly sensitive to storm forward speed, with the largest surge, for storms tracking from the east-southeast, resulting from the fastest-moving hurricane.
3. Storms tracking in a more northerly direction, roughly parallel to and west of the Hudson River can also cause significant storm surges at IPEC, although smaller surges result at New York Harbor from such storm tracks. Slower translational speeds for such storms maximize the wind set-up within the river fetch south of IPEC.
4. The surge elevation is moderately sensitive to the radius of maximum winds. Maximum surges generally occur with a radius of maximum wind on the order of 30 nm.

Based on the results of the sensitivity analyses, twenty-one (21) surge simulations were performed for select storms using the ADCIRC model. ADCIRC is a two-dimensional, depth-integrated, barotropic time-dependent long wave, hydrodynamic circulation model. ADCIRC uses a non-structured, triangulated model grid with variable grid resolution. The ADCIRC grid and boundary information file developed by the Federal Emergency Management Agency (FEMA) for the current FEMA Region II Flood Coastal Analysis and Mapping was used for this project. The ADCIRC model grid (FEMA, 2011a, FEMA 2011b, FEMA, 2011c, FEMA, 2012a) description is presented in Figures 3.4-11 through 3.4-13. The model mesh elevation data are presented in Figures 3.4-14 through 3.4-16. Note: FEMA 2011a, 2011b and 2012a are draft documents; they are cited here for general reference purposes only and were not used as design input for calculations. Data was obtained from FEMA for use as input for storm surge models via AREVA, 2013c.

Tidal constituents, which combine to generate tide potential, are variations in tides that are created by different frequencies of astronomical forces. ADCIRC utilizes developed tide databases as input, and seven tidal potential constituents from the LeProvost tide database (LeProvost et al., 1994) were used to simulate the tides including the potential range of astronomical tides. The tidal constituents were applied along the coastal boundary of the ADCIRC grid. To simulate tides reflective of the antecedent water level, tide gage data at The Battery (NOAA, 2013a) were analyzed to select the date and time for which the historical, astronomical tide approximately equals the antecedent water level. Tidal simulations were performed without storm meteorological forcing to confirm consistency between the simulated and predicted tidal phasing, range and amplitude.

Wind fields were developed for input to the ADCIRC simulations using the Dynamic Holland model (input option NWS 8) per Fleming et al., 2008, which presents the full equations used to define the wind field. In brief, the wind velocity and atmospheric pressure are calculated at every ADCIRC model node using the axisymmetric Dynamic Holland model:

$$p(r) = (p_o + \Delta p \times exp(-R_w/r)^B)/\rho_w g$$

where:

$p(r)$ = surface pressure a distance r from the storm center (Pascals)

p_o = central pressure (Pascals)

Δp = difference between central and background pressures (equivalent to *CPD,* Pascals)

R_w = radius to maximum winds (equivalent to R_{max}, meters)

r = distance from the storm center (meters)

B = pressure profile parameter (Holland B parameter; calculated, unitless)

ρ_w = density of water (kilograms per cubic meter)

g = coefficient of gravity (meters per second squared)

The 10-meter height 1-minute duration wind velocity (s_f, m/sec) was an input to the model. The maximum vortex storm wind speed (s_m, m/sec) was calculated by subtracting the storm translation speed from the maximum wind speed:

$$s_m = s_f - (v_{te}^2 + v_{tn}^2)^{0.5}$$

where:

v_{tn} = the storm translation speed in the north direction (m/sec)

v_{te} = the storm translation speed in the east direction (m/sec)

The gradient wind speed (V_m, in m/sec) at the radius of maximum wind is calculated by dividing s_m by the boundary layer adjustment factor (β=0.9). The Holland B parameter, B, is then calculated:

$$B = (\rho \times e \times V_m^2)/(\Delta p)$$

where:

ρ = density of air (kg/m^3)

e = Euler's number (constant, unitless) (Fleming et al, 2008)

Values of B are limited at the extremes to values between 1.0 and 2.5. The gradient wind velocity at each node (Vg) is calculated as:

$$V_g(r) = \sqrt{\left(\frac{R_w}{r}\right)^B * \exp\left[1-\left(\frac{R_w}{r}\right)^B\right] V_m^2 + \frac{r^2 f^2}{4}} - \frac{rf}{2}$$

where:

f = Coriolis frequency (per second)

The translational speed of the storm is tapered (based on the ratio of the maximum gradient and the local gradient wind) and added back to the vortex speed. The winds are converted to 10-minute winds (by multiplying by 0.88) for analysis of the storm surge (Fleming et al., 2008).

The list of storms and the storm parameters simulated in ADCIRC and predicted storm-tide elevations for The Battery and IPEC are shown in Table 3.4-4. The tracks for the storms are presented in Figure 3.4-17. The simulations were primarily performed for steady-state storm tracks and wind fields, with no storm intensity decay over land. The simulation for storm number 985 was performed for the storm bearing in a northerly direction (approximately parallel to and west of the Hudson River) that resulted in the largest storm surge at IPEC. Storms bearing in a northerly direction have the potential to generate wind set-up along the Hudson River. The additional simulation, storm number 985*, was performed with a non-steady state wind field to account for degradation of the wind field as the storm passed over land. The wind velocities were degraded in accordance with Kaplan & DeMaria 2001.

As indicated on Table 3.4-4, elevation 15.9 feet NAVD88 (16.9 ft NGVD29) was identified as the highest predicted storm-tide stillwater elevation at IPEC (deterministic analysis). This surge elevation is representative of a PMH defined as fast moving (40 to 45 knots), intense (maximum 10-meter height, 1-minute duration wind velocity of 128 knots) tropical cyclone bearing in a northwest direction and making landfall in New Jersey such that the highest velocity winds intersect the apex of the New York and New Jersey shoreline, in the vicinity of the New York Bight. Figure 3.4-18 presents the simulated envelope of maximum winds for this storm. As indicated by the results on Table 3.4-4, variations of this storm relative to landfall point, radius of maximum winds and storm speed resulted in predicted surge stillwater elevations ranging from about elevation 13.2 to 15.9 feet NAVD88 (14.2 to 16.9 feet NGVD29).

A storm tide stillwater elevation of 15.9 feet NAVD88 (16.9 feet NGVD29) is also representative of a PMH defined as a slower (30 knots translational speed – the lower bound for a PMH moving in a northerly direction), intense (maximum 10-meter height, 1-minute duration wind velocity of 128 knots), tropical cyclone bearing in a northerly direction (approximately parallel to and west of the Hudson River) and making landfall in the vicinity of Atlantic City, NJ. The storm surge indicates that this storm (Storm 985) will result in lower storm surges at The Battery but high surges at IPEC, due to additional wind set-up within the river fetch south of IPEC. Storm 985, however, will experience significant degradation of the wind velocity overland as indicated by Storm 985* (relative to Storm 985 that did not include any wind velocity degradation over land). Figure 3.4-19 presents the simulated envelope of maximum winds for this storm.

Figures 3.4-20 through 3.4-23 present the maximum storm-tide stillwater elevations (plan view) for each simulation of these two storms. Figures 3.4-24 and 3.4-25 show the wind fields for storms 941 and 985* (both ADCIRC and SLOSH wind models). Figures 3.4-26 and 3.4-27 show the wind profiles (both ADCIRC and SLOSH wind models). Figures 3.4-28 and 3.4-29 show the maximum storm-tide stillwater elevations within the Hudson River in profile view between The Battery and IPEC. Figures 3.4-30 and 3.4-31 present the storm-tide hydrographs and wind velocity time series for storms 941 and 985* at The Battery and IPEC.

As indicated on Figures 3.4-28 and 3.4-29, the surge at The Battery is significantly attenuated between Hastings On-Hudson and Verplanck Point, located south of IPEC, due to the river's large storage volume. Both SLOSH and ADCIRC show similar attenuation (however, with different storm surge amplitudes at The Battery). As the river channel constricts north of Verplank Point, the surge amplifies. The amount of storm surge attenuation, in particular for the ADCIRC simulations of storms that are not subject to significant wind set-up along the river, is similar to that observed for tidal flow. As expected, greater wind set-up occurred within the river fetch between Hastings On-Hudson and Verplanck Point for Storm 985*. As shown on Figures 3.4-30 and 3.4-31 the peak storm surge precedes the peak wind velocity by about one hour and the storm surge time to peak difference between The Battery and IPEC is about 3 hours.

In summary:

1. Storm surge simulations have been completed to analyze the range of storm tracks and meteorological parameters developed based on a site-specific meteorological and climatological study documented in AREVA 2013a.
2. Two different two-dimensional hydrodynamic numerical computer models (ADCIRC and SLOSH) were utilized to perform the simulations. Approximately 1,000 storm surge simulations were performed using SLOSH for screening/sensitivity purposes to identify the combinations of storm tracks and meteorological parameters that are predicted to cause the largest storm surge at IPEC. Based on the screening/sensitivity analyses, surge simulations were performed using ADCIRC.
3. An antecedent water level of elevation 2.9 feet NAVD88 was calculated (based on the average between the antecedent water levels at The Battery and IPEC) and utilized for the SLOSH storm surge simulations. For the ADCIRC simulations, a documented, historical astronomical tide with amplitudes at The Battery consistent with the calculated antecedent water level was simulated. Based on an evaluation of the effects of tide on the storm surge, the ADCIRC simulations were performed with the storm surge peak at IPEC occurring coincident with the astronomical high tide at IPEC.

Elevation 15.9 feet NAVD88 (16.9 ft NGVD29) was identified as the highest predicted storm surge stillwater elevation at IPEC based on the deterministic analysis. This elevation is representative of a PMH defined as a tropical cyclone that is:

1. Fast moving (40 to 45 knots),
2. Intense (maximum 10-meter height, 1-minute duration wind velocity of 128 knots)
3. Bearing in a northwest direction and
4. Making landfall in New Jersey such that with the highest velocity winds intersect the apex of the New York and New Jersey shoreline (i.e., in the vicinity of the New York Bight).

That same elevation is also representative of a PMH defined as slower (30 knots translational speed – the lower bound for a PMH moving in a northerly direction), intense tropical cyclone bearing in a northerly direction (approximately parallel to and west of the Hudson River) and making landfall in the vicinity of Atlantic City, NJ.

3.4.2.5 Combined Effects Floods - Coastal

3.4.2.5.1 Methodology

In accordance with guidelines presented in NRC, 2011 and ANS, 1992, several combined flood events (applicable to IPEC as a streamside location on an open or semi-enclosed body of water) were evaluated to determine the combined external flood effects resulting in the greatest flooding at IPEC using deterministic analyses:

- Alternative 1 – The combination of the lesser of one-half of the Probable Maximum Flood (PMF) or the 500-year flood, the surge and seiche from the worst regional hurricane or windstorm with wind-wave activity, and the antecedent 10 percent exceedance high tide
- Alternative 2 – The combination of the PMF, the 25-year storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide
- Alternative 3 – The combination of the 25-year flood in the stream, the probable maximum storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide

The analysis of the PMF is described in Section 3.2 and the PMF flow is 1,213,800 cubic feet per second (cfs). The analysis of one-half the PMF is calculated as one-half the PMF flow: 625,200 cfs. The analysis of the antecedent water level is presented in Section 1.1 and the value corresponding to the 10 percent exceedance high tide is elevation 2.9 feet NAVD88.

To evaluate Alternative 1, Hurricane Sandy, which resulted in the largest observed storm surge at IPEC, was selected as the worst regional hurricane. The ADCIRC model was used to simulate the combined storm surge, tidal flow and river flood. The wind field due to Sandy (NOAA, 2013c) was simulated using the NWS19 wind model within ADCIRC. The one-half PMF was used as input for a steady flow per unit width of river. Tides were simulated in ADCIRC such that the peak high tide elevation was coincident with the peak surge from the worst regional hurricane and was comparable with the calculated 10 percent exceedance high tide antecedent water level. The wind-generated deep water significant wave height (H_{mo}) corresponding to Sandy's maximum sustained wind speed were calculated and added (using linear superposition) to the combined water level resulting from the antecedent water level, one-half PMF and the worst regional hurricane (Sandy) simulated in ADCIRC.

To evaluate Alternative 2, the one-dimensional U.S. Army Corps of Engineers HEC-RAS v4.1 computer model (HEC-RAS) was used to simulate the tide, river flood and storm surge. The 25-year storm surge was calculated based on observed National Oceanic and Atmospheric Administration (NOAA) tide station data (NOAA, 2013a). A statistical analysis was performed using the NOAA tide gage at The Battery, New York (NOAA, 2013a). The maximum annual water level from each year was obtained for the 48-year period of record. A frequency analysis was performed by applying the log-Pearson Type III distribution (USDOI, 1982) to the data. The PMF flow hydrograph was used as the upstream boundary condition. A constant downstream boundary condition corresponding to the 25-year surge elevation plus the antecedent 10 percent exceedance high tide was used. Waves corresponding to the 25-year wind were then calculated and added on to combined water level resulting from the antecedent water level, the PMF and the 25-year surge simulated in HEC-RAS.

To evaluate Alternative 3, the ADCIRC model was used to simulate a combined storm surge, tidal flow and the 25-year recurrence period flood. Tides were simulated in ADCIRC such that the peak high tide elevation was coincident with the peak surge from the worst regional hurricane (Sandy) and was comparable with the calculated 10 percent exceedance high tide antecedent water level. The peak 25-year recurrence period flood flow in the Hudson River and was calculated by applying standard hydrological statistical methods to the United States Geological Survey (USGS) annual peak stream flow (USGS Gage No. 01258000 on Hudson River, at Green Island, New York, located approximately 110 miles upstream of IPEC). This flow gage is the closest Hudson River USGS gage to IPEC that is not located in a portion of the river subject to tidal effects. Data was obtained for the entire period of record for the gage (USGS, 2013). A frequency analysis was performed by applying the log-Pearson Type III distribution (USDOI, 1982) to the data. The 25-year flood in the Hudson River was input as a steady flow per unit width of the Hudson River, added in the ADCIRC model over a period of 10 days to maintain computational stability. Wind-generated waves corresponding to the PMH wind speed were then calculated and added to the combined water level resulting from the antecedent water level, the 25-year recurrence period flood and the PMSS. The maximum 1-minute, 10 meter height wind speed associated with the PMH was reduced based on the storm intensity decay over land in accordance with Kaplan and DeMaria, 2001.

Deep water wave height and wave period were calculated using CEDAS-ACES v.4.03. These large waves break at the river bulkhead and do not propagate inland onto the site. Wave setup from these waves will be mitigated due to interference from the structures that are present along the IPEC waterfront. The waves in flood inundation areas were depth-limited (due to the shallow water depth present in inundated areas) and wave heights were calculated per CEM guidance (USACE, 2002). Run-up from the depth limited waves on vertical building surfaces was calculated using CEDAS-ACES v4.03.

Hydrostatic force and hydrodynamic loads were calculated based on methodologies presented in guidance contained in (FEMA, 2011d), (FEMA, 2012b) and (ASCE, 2010). A debris weight of 2,000 pounds was used for computation of debris impact loads.

3.4.2.5.2 Results

The deterministically calculated maximum storm surge elevation at IPEC for Alternative 1, 2 and 3 includes a) a stillwater elevation, b) a wave crest elevation, and c) a maximum run-up on vertical surfaces. The table below provides a summary of those results:

NUREG/CR-7046 Appendix H.3 Alternative No.	Stillwater Elevation (feet, NGVD29)	Wave Crest Elevation (feet, NGVD29)	Limit of Runup Elevation (feet, NGVD29)
Alternative 1	15.2	15.4	16.3
Alternative 2	16.5	17.4	18.6
Alternative 3	20.4	23.4	26.5

In order to assess the probability for these three deterministic storm surge (stillwater) results, a separate probabilistic assessment was made. It includes certain conservative deterministic inputs and results in the construction of probability curves, Figures 3.4-51 and 3.4-52, described in Section 3.4.2.6.2.

3.4.2.6 Probabilistic-Deterministic Storm Surge Analysis

3.4.2.6.1 Methodology

Probabilistic-deterministic analyses were performed to further define the risk associated with very low probability storm surges at IPEC". Consistent with NRC, 2013, the methodology used to calculate the probabilistic storm surge stillwater elevation uses an integrated approach that has both probabilistic and deterministic components and includes:

1. A Site and Region Specific Meteorological Study (AREVA, 2013c) to statistically define the probability of occurrence of individual storm properties, e.g., translational speed, storm direction, etc., as discussed in Section 1.1.
2. Development of simulated storm tracks, each with a determined probability of occurrence using the Joint Probability Method (JPM) (FEMA, 2007).
3. Evaluation of storm surge for each storm utilizing SLOSH and ADCIRC

The probabilistic-deterministic analysis developed the flood stage-frequency relationship and identifies the flood elevation associated with an annual exceedance probability associated with an annual exceedance probability range of from about $2x10^{-6}$ to $1.5x10^{-7}$.
The methodology used in the calculation of the probabilistic-deterministic analysis includes an integrative approach to evaluate storm-tide elevation probabilities. The probabilistic components were developed using the Joint Probability Method (JPM) and included:

- Multiple simulated synthetic storm tracks and landfalls, each developed with a probability of occurrence determined using the JPM and the results of AREVA, 2013a that statistically defines the probability of occurrence of individual storm properties (e.g., translational speed, pressure deficit, etc.);
- Overall rate of tropical storms and hurricanes occurrence in the vicinity of IPEC; and
- Tide condition (high or low tide).

The deterministic components included:

- The storm simulations using two-dimensional hydrodynamic models;
- The volumetric river flow in Hudson River due to river flooding; and
- The long term rate of sea level rise (i.e., projected rate of increase in static water surface elevation), as defined in the PMSS calculation (AREVA, 2013b).

The JPM method was selected as the appropriate calculation methodology due to the sensitivity of large, low probability storm surges to a) storm track direction, b) storm speed, c) landfall location and d) radius of maximum wind. The selection of the JPM method was also based on the finding that equally large storm surges at IPEC can be caused by different combinations of storm tracks, landfalls and parameters. The JPM comprehensively captures the complete range of storm tracks and parameters that could result in large, very low probability storm surges for IPEC.
The calculation was performed in two parts (each with several steps) with increased computational detail focused on storm surges associated with annual exceedance probabilities on the order of $2x10^{-6}$ to $1.5x10^{-7}$. This approach, which is consistent with the Hierarchal Hazard Assessment (HHA) for flood re-evaluation (NRC, 2011), included the following parts:

Part I: The execution of approximately 32,000 synthetic storm surge simulations using the SLOSH model and simplified tidal and river flow conditions to develop an initial flood stillwater stage-frequency curve, including:

- Storm parameter development including probability distributions; subdivision of each of the parameter probability distributions into a small number of discrete elements (AREVA, 2013e).
- Determination of the omni-directional rate of storm occurrence in the vicinity of IPEC based on the historical storms that have made landfall from eastern Long Island to southern New Jersey.
- Development of a set of synthetic storms by combining each of the following elemental storm parameters:

  - 6 pressure deficit values (CPD) values at 10-millibar intervals;
  - 9 forward directions at 10-degree intervals;
  - 7 different forward speeds at 5-knot intervals; and
  - 6 Rmax values at 4-nautical-mile intervals (where Rmax indicates the radius of the maximum wind velocity). Each combination of storm parameters was assigned a joint (elemental) probability of occurrence using the product of the parameter-specific probabilities. The elemental combinations produced 2,268 (6x9x7x6) unique events.
- Creation of storm tracks based on 9 forward directions at 10-degree intervals. A base set of storm tracks was created with 9 tracks converging to the center of Raritan Bay (Figure 3.4-32). Raritan Bay represents vicinity location where coastal storm surges impacting New York City and routing up the Hudson River are generated. Three parallel tracks were created on each side of each base track (producing 7 tracks for each forward direction) using offset distances determined by functional relationships to the R_{max} storm intensity parameter. This process produced a total of 378 unique tracks, with 325 distinct landfall points (note that each track does not have a unique landfall location due to overlap at the target location). The total number of storm simulations (after combining the unique elemental events with all potential tracks) is 15,876 (2,268x7).
- For each storm calculation of the corresponding event rate of occurrence, which is defined as a product of the overall rate of occurrence, the joint (elemental) probability and the probability of any storm track making landfall (calculated as the ratio of R_{max}/L). For each individual track, the event rate of occurrence was then adjusted to reflect the spacing between the two adjacent tracks (i.e., in recognition of potential variance from the defined track). The factors representing this adjustment were individually calculated as the ratio of the sum of the distances from a given track to its two adjacent tracks and $R_{max.}$
- Simulation of the synthetic storm set using SLOSH computer model. SLOSH simulations were performed for both for mean high and low tide antecedent water levels and no-flow river conditions. The total number of storm simulations (considering all the tracks) was 31,752 (15,876x2). The simulations were performed for steady-state storm conditions, i.e., not changing storm parameters or storm direction along the storm track.
- Event occurrence rates were accumulated into bins defined by surge elevation increments.
- For the accumulated occurrence rates (at The Battery and IPEC), the results were summed for all bins to generate estimates of the cumulative surge distribution and flood stage-frequency curve. The return period, the reciprocal of the calculated cumulative probability, was thereby provided as

$$T_i = \frac{1}{P_i}$$

where:

T_i = return period for the ith bin; and

P_i = calculated probability for the ith bin.

Part II: Approximately 50 additional simulations using the ADCIRC model to refine the flood stillwater stage-frequency curve for annual exceedance probabilities ranging from about $5x10^{-5}$ to $1x10^{-8}$ and including:

- The top 50 storms, ranked by SLOSH storm surge elevation at IPEC, were identified as representing the flood stillwater stage-frequency curve for annual exceedance probabilities ranging from about $5x10^{-5}$ to $1x10^{-8}$. A subset of the top 50 storms (21 events, in total) was identified for further analysis with SLOSH stillwater storm surge elevations greater than 17.0 feet NAVD88 at IPEC.

- Surge heights for those 21 storms were simulated using the ADCIRC model. To compare ADCIRC and SLOSH results, storm surge simulations were initially performed assuming conditions similar to the SLOSH simulations. Additional simulations for the top 50 events were performed to include tidal flow, river flooding (25-year return period) and decay of storm intensity (decreasing wind velocity) over land.
- A final flood stage-frequency curve was developed for the ADCIRC results representing a range of annual exceedance probabilities from about 2.9×10^{-6} to 1.4×10^{-7}.

Observed river flooding and storm surge due to large historical hurricanes, including data from Hurricanes Donna, Gloria and Irene, indicate that storm surge and river flooding are not independent events. Hurricane Irene made landfall in North Carolina as a Category 1 hurricane (NOAA 2013e). It eventually moved over Brooklyn, New York. The strongest wind at the time of landfall was 55 knots and storm-tide heights were approximately 9.5 feet NAVD88 at The Battery and 7.92 feet at Poughkeepsie. The rainfall total at Central Park, New York was 6.9 inches (AREVA, 2013b). The USGS stream gage on the Hudson River at Green Island reported a peak flow of 181,000 cfs of August 29, 2011 (USGS, 2013). The peak storm surge at The Battery was recorded as elevation 6.73 feet NAVD 88, occurring on August 28, 2011 at 12:41pm GMT (NOAA, 2013a). At the time of the peak surge observed near IPEC, the flow in the river at Poughkeepsie (approximately 75 miles upriver from The Battery) was approximately 50,000 cfs.

For the probabilistic-deterministic analysis, the volumetric flow for the river flood was used as deterministic input to each of the ADCIRC storm surge simulations. The flood flow was also conservatively modeled as a steady-state river flow condition, assuring that the maximum river flood stage is present at IPEC at the time of the peak storm surge. The rationale for this conservative approach is threefold:

1. This approach is consistent with scenarios for "combined events" as defined in NUREG/CR-7046 (NRC, 2011).
2. Storms were defined as steady-state events with respect to direction. Although, many of these storm tracks (i.e., storms moving in a westerly direction, south of IPEC) are not expected to result in significant precipitation within the contributory watershed, there is a reasonable chance, based on tracking trends of historic storms, that the storm tracks could veer to the northeast after landfall.
3. The approach appears to be supported by observed river and storm surge flooding from large historical hurricanes. Historical data from Hurricanes Donna, Gloria and Irene were used to qualitatively evaluate the timing of the storm surge and the riverine flooding.

3.4.2.6.2 Results

The annual historical storm rate for IPEC was determined to be 0.261 storm/year. That value is representative storms that impact the U. S. coast in the area between southern NJ to Eastern Long Island, NY.

Tables 3.4- 5 through 3.4-8 present the discretized probabilities for pressure deficit, forward direction, forward speed and radius of maximum wind, respectively. Table 3.4-9 shows the factor for adjusting landfall probabilities for each track based on spacing between adjacent tracks.

A base set of storm tracks was created with 9 tracks converging to a point coincident with the center of Raritan Bay (Figure 3.4-32). Storm tracks were developed based on 9 forward directions spaced at 10-degree intervals. Three parallel tracks were created on each side of each base track (producing 7 total tracks for each forward direction) using offset distances determined by functional relationships to the R_{max} storm intensity parameter. This process produced a total of 378 unique tracks, with 325 distinct landfall points. An example of this offsetting scheme is illustrated in Figure 3.4-32. The landfall points for all of the storm tracks are shown in Figure 3.4-33. The tracks are shown in Figures 3.4-34 through 3.4-39 for six R_{max} values, from 16 to 36 nautical miles. The total number of storms evaluated for a given antecedent water level is equal to 15,876.

The high and low tides used in the SLOSH JPM storm simulations were the Mean High Water (MHW) and Mean Low Water (MLW) levels at The Battery (NOAA station 8518750): 1.96 feet NAVD88 and -2.57 feet NAVD88, respectively (NOAA, 2013a). SLOSH simulations were performed for both for MHW and MLW antecedent water levels, and no river flow conditions, resulting in a total of 31,752 results. The highest surge elevations computed for each storm at IPEC and The Battery were recorded for each storm, each with a defined probability of occurrence.

The top ten storms, ranked by surge stillwater elevation at IPEC and The Battery are summarized on Tables 3.4-10 and 3.4-11, respectively; storm tracks for these storms are shown in Figures 3.4-40 and 3.4-41, respectively. Histograms of the SLOSH simulations are presented on Figures 3.4-42 through 3.4-45. Stillwater stage-frequency curves at IPEC and The Battery based on the JPM SLOSH simulations are shown in Figures 3.4-46 and 3.4-47.

The top 21 storm surge events (storm surge greater than elevation 17.0 feet NAVD88), identified by the SLOSH storm surge results at IPEC, were further analyzed using ADCIRC, as a check to compare the ADCIRC and SLOSH results for similar model conditions. The simulations were first performed with similar forcing conditions as the SLOSH simulations, including tidal flow (mean high tide), normal river flow and steady-state wind fields (i.e., no wind decay of storm intensity over land), for comparison to the SLOSH simulation results. The simulations were phased so that the peak storm surge and high tide occurred at the same time at IPEC.

The final ADCIRC simulations were performed to simulate the top 50 storms, defined and ranked by the SLOSH-simulated surge elevation at IPEC. The simulations considered:

1. Tidal flow, with high tide and storm surge occurring simultaneously at IPEC;
2. River flood flow, corresponding to the 25-year flood and applied as steady-state flow to assure that the peak river flood will be present during peak storm surge at IPEC; and
3. Decay of storm intensity after landfall (i.e., for overland portions of the storm track).

The river flood flow was modeled as a steady-state flux of 259,800 cfs (i.e., the 25-year return period peak flood flow rate for Hudson River). The results from the tidal and 25-year riverine flow simulation were then used to establish the initial model conditions for the subsequent storm surge simulations in ADCIRC. Figures 3.4-48 and 3.4-49 shows the simulated tidal flow with and without river flooding. The results indicate that the 25-year river flood reduces the difference between high and low tide but causes an overall elevated water level in the river (by approximately 2 to 2.5 feet near IPEC).

Table 3.4-12 presents the results of the ADCIRC simulations for the top 50 storms. Summaries of storm occurrence rate versus surge elevations were re-created for IPEC using the final ADCIRC simulations by accumulating the storm rates into the bins of surge elevation. The accumulated rates in the bins constitute a density distribution of the surge elevation. Bins sized at 0.25 feet were summed from the top to the bottom, to create a cumulative surge distribution and flood stage-frequency curve. Figures 3.4-50 and 3.4-51 represent the final storm-tide (stillwater) stage-frequency curves at IPEC without and with a 50-year sea level rise of 0.46 feet, respectively (AREVA, 2013b).

Probabilistic analysis of storm surge using the JPM and SLOSH AND ADCIRC models indicates that the storm surge stillwater flood elevation at IPEC associated with a $1x10^{-6}$ annual exceedance probability, including storm surge, tidal flow, projected sea level rise and steady-state river flow associated with a 25-year return period in the Hudson River, is elevation 16.6 feet NAVD88 (17.6 feet NGVD29). [1] Probabilistic analysis of storm surge using the JPM and SLOSH and ADCIRC models indicates that the storm surge Stillwater flood elevation at IPEC associated with a $2x10^{-6}$ annual exceedance probability, including storm surge, tidal flow, projected sea level rise and steady-state river flow associated with a 25-year return period in the Hudson River, is elevation 14.8 feet NAVD88 (15.8 feet NGVD29).

Table 3.4-12 also presents results for combined effect wave action. Similar to the deterministic methodology, deep water wave height and wave period were calculated using CEDAS-ACES v.4.03. These large waves break at the river bulkhead and do not propagate inland onto the site. Wave setup from the deep water waves will be mitigated due to interference from the structures that are present along the IPEC waterfront. The waves in flood inundation areas were depth-limited (due to the shallow water depth present in inundated areas) and wave heights were calculated per CEM guidance (USACE, 2002). Run-up from the depth limited waves on vertical building surfaces was calculated using CEDAS-ACES v4.03.

Scenario Probability (Return Period)	Stillwater Elevation (feet, NGVD29)	Wave Crest Elevation (feet, NGVD)	Limit of Runup Elevation (feet, NGVD29)
$1.0x10^{-6}$ (1,000,000)[1]	17.6	19.1	21.1
$2x10^{-6}$ (500,000)	15.8	16.3	17.7

Inundation maps for the probabilistic-deterministic analysis results are presented in Figure 3.4-52. Areas located east (inland) of the turbine buildings have limited connectivity with the open water west of the turbine building area and by inspection are not affected by wave action. Wave run-up on vertical surfaces (i.e. buildings) is present on buildings along the bulkhead and along the west (i.e., river-facing) side of the turbine buildings.

Limits of wave hazard are delineated based on US Army Corps of Engineers guidance that has been adopted by FEMA (FEMA, 2005). Limits for waves larger than 3 feet and 1.5 feet are shown on Figure 3.4-52 for reference. Waves larger than 3 feet have been determined to cause major damage on contact with conventional structures; waves larger than 1.5 feet have been found to cause failure of traditional stud wall construction (FEMA, 2005).

Typical hydrostatic and hydrodynamic forces were calculated for the deterministic combined flood effects.

Hydrostatic Loads

Hydrostatic loads including vertical downward loads, lateral loads and vertical upward loads (uplift or buoyancy) were calculated per ASCE, 2010.

[1] These results contain conservative assumptions which are undergoing further refinement. Final results may be significantly lower.

The maximum stillwater water surface elevation of 19.4 feet NAVD88 (20.4 feet NGVD29) for the deterministic combined effect flood was used and results in a depth of flood water of 5.4 feet. The typical hydrostatic force was calculated as 940 lbs/linear foot with the force acting at elevation 16.8 feet NGVD29.

Flow Velocity

Flow velocities for water being forced onto the site by the action of waves breaking on the river bulkhead were conservatively calculated per FEMA, 2011d.

An upper bound flow velocity was calculated to be 13.2 feet per second for the controlling deterministic combined effects flood and 9.1 feet per second for the probabilistic combined effects flood.

Hydrodynamic Loads

Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides and suction at the downstream side. Hydrodynamic loads calculations used steady-state flow velocities as per FEMA, 2011d and FEMA, 2012b. The hydrodynamic forces were calculated per FEMA, 2012b.

The hydrodynamic loading analysis (calculated at the Turbine Buildings) resulted in a force of 1,825 pounds per liner foot acting at elevation 16.7 NAVD88 (17.7 feet NGVD29) for the controlling deterministic combined flood effects.

The hydrodynamic loading analysis (calculated at the Turbine Buildings) resulted in an equivalent hydrostatic force of 417 pounds per linear foot acting at elevation 16.3 NAVD88 (17.3 feet NGVD29) for the probabilistic combined effects flood.

Debris Impact Loads

Typical debris impact loads on exterior portions of structures (for debris weight of 2,000 lbs) were calculated as 27,456 pounds for the controlling deterministic combined effects flood and were calculated using the guidelines described in FEMA 2012b and by considering debris weight recommended in ASCE, 2010.

Debris impact loads on exterior portions of structures were calculated as 9,464 pounds probabilistic combined effects flood.

Wave Loads

Wave loads are those loads that result from water waves propagating over the water surface and striking a building (or other structure). The forces from breaking waves are the largest and most severe; therefore this load condition was used as the design wave load as per FEMA, 2011d.

The typical breaking wave load on vertical walls inboard of the river bulkhead generated by the depth-limited waves propagating across the site was calculated as 11,372 pounds per foot for the controlling deterministic combined effects flood.

The maximum breaking wave load on vertical walls inboard of the river bulkhead generated by the depth-limited waves propagating across the site was calculated as 2,636 pounds per foot for the probabilistic combined effects flood.

3.4.3 Conclusions

The storm surge reevaluation is based on the use of current hydrodynamic numerical computer models. The probable maximum hurricane (PMH) (i.e. the hypothetical steady-state hurricane with a combination of values of meteorological parameters that will give the highest sustained winds speed that can probably occur at a specified coastal location) is used as input to the models. Due to the lack of historical hurricane data, the historical database was supplemented by a set of synthetic storms that were generated using intensity and atmospheric models and validated by a region specific hurricane study.

The deterministic storm surge stillwater results given in Section 3.4.2.4 are calculated to be 15.9 feet NAVD88 (16.9 feet NGVD29). Based on the JLD-ISG-2012-06 (NRC, 2013), a probabilistic frequency curve was developed for the storm surge simulations developed for IPEC.

The reevaluated storm surge stillwater elevation is calculated to be 15.8 ft (NGVD29) based on the $2x10^{-6}$ probabilistic-deterministic frequency analysis described in Section 3.4.2.6. A lower probability elevation (e.g. $1x10^{-6}$) will require further analysis, as discussed in Section 3.4.1.

3.4.4 References

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

AREVA, 2013a. AREVA Document No. 32-9207390-000, "Probable Maximum Hurricane for Indian Point Energy Center", 2013.

AREVA, 2013b. AREVA Document No. 32-9213356-000, "Flood Hazard Re-evaluation - Combined Effect Floods – Coastal Processes for Indian Point Energy Center", 2013.

AREVA, 2013c. AREVA Document No. 38-9216321-000, "Acquisition of Coastal Flood Analysis Data from Federal Emergency Management Agency Region II Indian Point Energy Center Flooding Hazard Re-evaluation", August, 2013.

AREVA, 2013d. AREVA Document No. 32-9196319-000, "IPEC Deterministic Probable Maximum Storm Surge Calculation", 2013.

AREVA, 2013e. AREVA Document No. 32-9213352-000, "Flood Hazard Re-evaluation - Probabilistic Storm Surge for Indian Point Energy Center", 2013.

ASCE, 2010. "Minimum Design Loads for Buildings and Other Structures", ASCE/SEI 7-10, American Society of Civil Engineers (ASCE), 2010.

FEMA, 2005. "Flood Hazard Zones: FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report", Federal Emergency Management Agency, February 2005.

FEMA, 2007. Guidelines and Specifications for Flood Hazard Mapping Partners, "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update - Final Draft", Federal Emergency Management Agency, Department of Homeland Security, Washington DC, February 2007.

FEMA, 2011a. "Region II Storm Surge Project, Mesh Development Report" – Draft Report, Federal Emergency Management Agency, Department of Homeland Security, Washington DC, September 2011. Draft document used for information only – not used for design input.

FEMA, 2011b. "Region II Coastal Terrain Processing Methodology Documentation Report" – Draft Report, Federal Emergency Management Agency, Department of Homeland Security, Washington DC, September 2011. Draft document used for information only – not used for design input.

FEMA, 2011c. Redefinition of the Coastal Flood Hazard Zones in FEMA Region II: Analysis of the Coastal Storm Surge Flood Frequencies, Federal Emergency Agency, August 2011

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

FEMA, 2012a. "Region II Storm Surge Analysis – Spatially Varying Nodal Attribute Parameters" – Draft Land Cover Data Report, Federal Emergency Management Agency, Department of Homeland Security, Washington DC, October 2012. Draft document used for information only – not used for design input.

FEMA, 2012b. "Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures", FEMA-P-256, Federal Emergency Management Agency, 2012.

Fleming et al., 2008. "A Real Time Storm Surge Forecasting System using ADCIRC", Fleming, J.G., Fulcher, C.W., Luettich, R.A., Estrade, B.D., Allen, G.D. and Winer, H.S. 2008.

Kaplan & DeMaria, 2001. "On the Decay of Tropical Cyclone Winds after Landfall in the New England Area – Notes and Correspondence", Kaplan J. and DeMaria, M., January and May 2000, published in February 2001, Journal of Applied Meteorology, Vol 40, p280-286.

LeProvost et al., 1994. "Spectroscopy of the ocean tides from a finite element hydrodynamic model", Le Provost, C., Genco, M.L., Lyard, F., Vincent, P. and Canceil, P., Journal of Geophysical Research, 99(C12):24777–24797, 1994.

NASA, 1998. Atlas of Extratropical Storm Tracks (1961-1998), National Aeronautics and Space Administration, Date revised: December 1998, Date Accessed: 2 July 2013. Available at: http://data.giss.nasa.gov/stormtracks/

NGS, 2013. "VERTCON – North American Vertical Datum Conversion", National Geodetic Survey http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html, Date accessed September and October 2013, Date revised January 24, 2013.

NOAA, 1979. Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coast of the United States, Technical Report NWS 23, Schwerdt, R.W., Ho, F.P., & Watkins, R.R.,. Washington D.C., National Oceanic and Atmospheric Administration, 1979.

NOAA, 1987. "Hurricane Climatology for the Atlantic and Gulf Coasts of the United States", National Oceanic and Atmospheric Administration Technical Report NWS 38, April 1987, see Electronic References

NOAA, 1992. SLOSH: Sea, Lake, and Overland Surges from Hurricanes, NOAA Technical Report. NWS 48, Jelesnianski, C.P., Chen, J., & Shaffer, W.A., Silver Spring, Maryland: National Weather Service, National Oceanic and Atmospheric Administration, April 1992.

NOAA, 2008. Sea Lake and Overland Surges from Hurricanes (SLOSH) Basin Development Handbook v2.0. Meteorological Development Lab, National Weather Service, National Oceanic and Atmospheric Administration, 2008.

NOAA, 2008a. NCDC Satellite Gallery of Extra Tropical Cyclones, Perfect Storm 1991, Date Revised: August 2008, Date Accessed: 3 July 2013. Available at: http://www.ncdc.noaa.gov/oa/satellite/satelliteseye/cyclones/pfctstorm91/pfctstorm.html

NOAA, 2012a. "SLOSH Model v3.97" Evaluation Branch/ Meteorological Development Lab National Weather Service/NOAA/U.S. Dept. of Commerce, January 2012.

NOAA, 2012b. "SLOSH Display Program (1.65i)." Evaluation Branch/ Meteorological Development Lab National Weather Service/NOAA/U.S. Dept. of Commerce, January 2012.

NOAA, 2012c. HURDAT2 Data Reanalysis Project, accessed 12/6/2013.
http://www.aoml.noaa.gov/hrd/hurdat/newhurdat-format.pdf
http://www.aoml.noaa.gov/hrd/hurdat/metadata_master.html
http://www.aoml.noaa.gov/hrd/hurdat/Data_Storm.html

NOAA, 2013a. Tides and Currents, Water Levels, 8518750 The Battery, New York, National Oceanic and Atmospheric Administration. Data accessed September 2013. Date modified September 2013. http://tidesandcurrents.noaa.gov/waterlevels.html?id=8518750.

NOAA, 2013b. Tides and Currents, Sea Levels Online, Mean Sea Level Trend, 8518750 The Battery, New York, National Oceanic and Atmospheric Administration. Data accessed September 2013. Date modified September 2013. .
http://tidesandcurrents.noaa.gov/sltrends/sltrends_update.shtml?stnid=8518750

NOAA, 2013c. Tides and Currents, Tide Predictions, 8531680 Sandy Hook, New Jersey, National Oceanic and Atmospheric Administration. Data accessed September 2013. Date modified September 2013.
http://tidesandcurrents.noaa.gov/noaatidepredictions/viewDailyPredictions.jsp?Stationid=8531680.

NOAA, 2013e National Hurricane Center Hurricane Climatology, National Weather Service, National Oceanic and Atmospheric Administration. Data accessed October 2013. Date modified May 2013. http://www.nhc.noaa.gov/aboutsshws.php.

NRC, 2011. U.S. Nuclear Regulatory Commission (U.S. NRC) 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.

NRC, 2012. U.S. Nuclear Regulatory Commission (U.S. NRC) NUREG/CR-7134 "The Estimation of Very Low Probability Surges to Design and Licensing at Nuclear Power Plants in Coastal Areas," May 2012.

NRC, 2013. Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, JLD-ISG-2012-06, U.S. Nuclear Regulatory Commission, January 2013.

Stedinger et al., 1993. Chapter 18: Analysis and Modeling of Hydrologic Time Series, Stedinger, J., Vogel, R., & Foufoula-Georgiu, E., in *Handbook of Hydrology.* McGraw-Hill, 1993.

USACE, 1992. Luettich, R.A., Jr., J.J. Westerink, and N.W. Scheffner, ADCIRC: an advanced three-dimensional circulation model for shelves coasts and estuaries, report 1: theory and methodology of ADCIRC-2DDI and ADCIRC-3DL, Dredging Research Program Technical Report DRP-92-6, U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, 137p.
http://www.unc.edu/ims/adcirc/publications/1992/1992_Luettich02.pdf

USACE, 2002. "Coastal Engineering Manual – Part II", EM 1110-2-1100, U.S. Army Corps of Engineers, April 30, 2002.

USDOI, 1982. "Guidelines For Determining Flood Flow Frequency", Bulletin #17B of the Hydrology Subcommittee, U.S. Department of the Interior, Geologic Survey, Office of Water Data Collection, Revised September 1981, Edited March 1982.

USGS, 2013. USGS Water Data for the Nation, National Water Information System: Web Interface. Accessed in October 2013, U.S. Geological Survey. Available at http://waterdata.usgs.gov/nwis.