Attachment 2 to

W3F1-2015-0042

Flood Hazard Reevaluation Report

(264 pages)

Engineering Report No. WF3-CS-15-00010 Rev 0
Page 1 of 264

Entergy

ENTERGY NUCLEAR
Engineering Report Cover Sheet

Engineering Report Title:
Waterford Steam Electric Station Unit 3 Fukushima Flood Hazard Re-evaluation Report

Engineering Report Type:

New ☒ Revision ☐ Cancelled ☐ Superseded ☐
Superseded by: ______

Applicable Site(s)

IP1	☐	IP2	☐	IP3	☐	JAF	☐	PNPS	☐	VY	☐	WPO	☐
ANO1	☐	ANO2	☐	ECH	☐	GGNS	☐	RBS	☐	WF3	☒	PLP	☐

EC No. 58788

Report Origin: ☐ Entergy ☒ Vendor
Vendor Document No.: 51-9227040-00

Quality-Related: ☒ Yes ☐ No

Prepared by: AREVA / See AREVA Cover Pages Date: 7/9/15
Responsible Engineer (Print Name/Sign)

Design Verified: AREVA / See AREVA Cover Pages Date: 7/9/15
Design Verifier (if required) (Print Name/Sign)

Reviewed by: Stephen Picard / See Associated EC Date: 7/13/15
Reviewer (Print Name/Sign)

Approved by: Chris Talazac / See Associated EC Date: 7/13/15
Supervisor / Manager (Print Name/Sign)

20004-021 (01/30/2014)

AREVA

AREVA Inc.

Engineering Information Record

Document No.: 51 - 9227040 - 000

Waterford Steam Electric Station Flooding Hazard Re-Evaluation Report

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
Daniel T. Brown Scientist IV, Environmental Analysis		LP	7/9/15	All
Kenneth D. Hunu GZA Civil Engineer		P	7/9/15	Sections 3.1, 3.2, 3.3, 3.4, 3.6, 3.7, 3.8, and 3.9
Cynthia A. Fasano Advisory Engineer, Environmental Analysis		LR	7/9/15	All
David M. Leone GZA Hydraulic Engineer	CAH Cox for DML	R	07/09/15	Sections 3.1, 3.2, 3.3, 3.4, 3.6, 3.7, 3.8, and 3.9
Barbara Hubbard First Line Leader, Radiological & Environmental Analysis		A	7/9/15	All

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 release.

Overview

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

Section 1.0 provides introductory information related to the flood hazard. The section includes background regulatory information, scope, general method used for the re-evaluation, assumptions, the elevation datum used throughout the report, and a conversion table to determine elevations in other common datum.

Section 2.0 describes detailed WSES site information, including present-day site layout, topography, and current licensing basis flood protection and mitigation features. The section also identifies relevant changes since license issuance to the local area and watershed as well as flood protections.

Section 3.0 presents the results of the flood hazard re-evaluation. It addresses each of the eight flood-causing mechanisms required by the NRC as well as a combined effect flood. In cases where a mechanism does not apply to the WSES site, a justification is included. The section also provides a basis for inputs and assumptions, methods, and models used.

Section 4.0 compares the current and re-evaluated flood-causing mechanisms. It provides an assessment of the current licensing and design basis flood elevation to the re-evaluated flood elevation for each applicable flood-causing mechanism evaluated in Section 3.0.

Section 5.0 presents an interim evaluation and actions taken, or planned, to address those higher flooding hazards identified in Section 4.0 relative to the current licensing and design basis.

Section 6.0 describes the additional actions taken to support the interim actions described in Section 5.0. Note that no additional actions were identified as necessary.

The report also contains one appendix. Appendix A background information used to determine the specific elevation differencet for the site datum relative to National Geodetic Survey reference stations.

Executive Summary

This report satisfies the "Hazard Reevaluation Report" Request for Information pursuant to 10 Code of Federal Regulations (CFR) 50.54(f) by the NRC dated November 12, 2012, NTTF Recommendation 2.1 Flooding Enclosure 2.

The report describes the approach, methods and results from the re-evaluation of flood hazards at Waterford Steam Electric Station (WSES). 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. No additional flood causing mechanisms were identified for WSES.

Each of the re-evaluated flood causing mechanisms and the potential effects on the WSES site are described in Sections 3.0 and 4.0 of this report.

The methodology of the flood hazard reevaluation documented in this report follows the Hierarchical Hazard Assessment 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 Interim Staff Guidance, as appropriate, and their supporting reference documents.

Screened mechanisms have been evaluated at a high level and determined to not be applicable to the flooding hazard for WSES.

The WSES design basis flood level and associated design basis protections are challenged by two flood mechanisms. The direct precipitation and rooftop drainage during the Local Intense Precipitation event results in ponding in the Dry Cooling Tower Basins, with potential to impact equipment important to safety. Additionally, the WSES design basis is challenged by flooding due to the combined effects of a combined effect flood scenario resulting in wave overtopping the east side of the Nuclear Plant Island Structure. Interim evaluations of the impacts of those two flood mechanisms are addressed in Section 5.0.

An evaluation was performed to determine the impact of inundation due to the two flood mechanisms that challenge the WSES design basis. The results of this evaluation indicate that there are no impacts to equipment important to safety as a result of the re-evaluated flood elevations.

Table of Contents

Page

Table of Contents
(continued)

Table of Contents
(continued)

List of Tables

Page

List of Tables
(continued)

Page

List of Figures

Acronyms and Abbreviations

Acronym/Abbreviation	Description
10CFR50.54(f)	Title 10 of the Code of Federal Regulations, Section 50.54(f)
ANS	American Nuclear Society
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASPRS	American Society for Photogrammetry and Remote Sensing
AWL	Antecedant Water Level
CCEF	Controlling Combined Effect Flood
CEM	Coastal Engineering Manual
CFR	Code of Federal Regulations
cfs	cubic ft per second
CLB	Current License Basis
CO-OPS	Center for Operational Oceanographic Products and Services
DA	Depth-Area
DAD	Depth-Area-Duration
DBFE	Design Basis Flood Elevation
DCT A	Dry Cooling Tower Alpha
DCT B	Dry Cooling Tower Bravo
DEM	Digital Elevation Model
DTM	Digital Terrain Model
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FHB	Fuel Handling Building
FIS	Flood Insurance Study

Acronym/Abbreviation	Description
fps	ft per second
FSAR	Final Safety Analysis Report
FSU	Florida State University
FT	Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coast of the United States, Technical Report FT
GEV	Generalized Extreme Value
GIS	Geographic Information Systems
GoM	Gulf of Mexico
GSOD	Global Surface Summary of Day Data
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
HSDRRS	Greater New Orleans Hurricane and Storm Damage Risk Reduction System
HURDAT	Hurricane Database
HWM	High Water Mark
ISFSI	Independent Spent Fuel Storage Installation
ISG	Interim Staff Guidance (NRC)
LiDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
LMSL	Local Mean Sea Level
LOOP	Loss of Offsite Power
mb	millibars
MCC	Motor Control Center
MPI	Maximum Potential Intensity
MRGO	Mississippi River Gulf Outlet

Acronym/Abbreviation	Description
MSIV	Main Steam Isolation Valve Area
MSL	Mean Sea Level
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NESDIS	National Environmental Satellite, Data, and Information Service
NGDC	National Geophysical Data Center
NGVD29	National Geodetic Vertical Datum of 1929
NID	National Inventory of Dams
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NPIS	Nuclear Plant Island Structure
NRC	U.S. Nuclear Regulatory Commission
NTHMP	National Tsunami Hazard Mitigation Program
NTTF	Near-Term Task Force
NWS	National Weather Service
ODMI	Operational Decision Making Issue
ORCS	Old River Control Structure
PDF	Project Design Flood
PMF	Probable Maximum Flood
PMH	Probable Maximum Hurricane
PMP	Probable Maximum Precipitation
PMSS	Probable Maximum Storm Surge
RAB	Reactor Auxiliary Building
RAMMB	Regional and Mesoscale Meteorology Branch
RMSE	Root Mean Square Error

Acronym/Abbreviation	Description
SLR	Sea Level Rise
SPF	Standard Project Flood
SSCs	Structures, Systems and Components
SST	Sea Surface Temperature
UHS	Ultimate Heatsink
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geological Survey
UTC	coordinated universal time
VBS	Vehicle Barrier System
V_{max}	maximum wind speed in kt
WCT A	Wet Cooling Tower Alpha
WCT B	Wet Cooling Tower Bravo
WMO	World Meteorological Organization
WSES	Waterford Steam Electric Station

1.0 INTRODUCTION

Following the Fukushima Dai-ichi 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.

Waterford Steam Electric Station (WSES), located on the west bank of the Mississippi River in Killona, Louisiana, is one of the sites required to submit information. WSES is located near river mile 130, upstream of and approximately 25 miles west of New Orleans, Louisiana.

The NRC information request 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 10 Code of Federal Regulations (CFR) 50.54(f) by the NRC dated November 12, 2012, NTTF Recommendation 2.1 Flooding Enclosure 2.

The report describes the approach, methods and results from the re-evaluation of flood hazards at WSES.

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 WSES.

Each of the re-evaluated flood causing mechanisms and the potential effects on the WSES site are described in Sections 3.0 and 4.0 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), NRC Interim Staff Guidance (ISG), as appropriate, and their 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 (SSCs) 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 SSCs important to safety, a more site-specific hazard assessment is performed for the probable maximum event.

The HHA approach was carried out for each flood-causing mechanism, with the controlling flood being the event that resulted in the most severe hazard to the SSCs important to safety at WSES. The steps involved to estimate the design-basis flood typically included the following:

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

2. For each flood-causing phenomena, develop a conservative estimate of the flood from the corresponding probable maximum event using conservative simplifying assumptions.
3. If any SSCs important to safety are adversely affected by flood hazards, use site-specific data and/or more refined analyses to provide more realistic conditions and flood analysis, while ensuring that these conditions are consistent with those used by Federal agencies in similar design considerations.
4. Repeat Step 2 until all SSCs important to safety are unaffected by the estimated flood, or if all feasible site-specific data and model refinement options have been used.

Section 3.0 of this report provides additional HHA detail for each of the flood-causing mechanisms evaluated.

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

1.4 Assumptions

Assumptions used to support the flood re-evaluation are described in Section 3.0 and its subsections, and depend on the mechanism being evaluated. Details relating to assumption justifications are discussed further in referenced, supporting documentation. None of the assumptions require verification, i.e., need to be confirmed prior to use of the results.

Discussions in this report which include the terminology "design basis" indicates information developed to determine flooding hazard and requirements for flood protection, as indicated in Section 2.4 of the WSES FSAR (WSES, 2013).

By definition, CLB (per 10CFR54.3(a)) includes any NRC requirements, current and effective licensee commitments, operation, and any design basis information for the site as documented in the most recent final safety analysis report.

For the purposes of the WSES Flood Hazard Re-evaluation Report, the two terms, design basis and licensing basis, can be considered to have the same meaning.

1.5 Elevation Values

Elevations in the WSES Final Safety Analysis Report (FSAR) (WSES, 2013) refer to the Plant Datum, Mean Sea Level (MSL). To convert elevations from the North American Vertical Datum of 1988 (NAVD88)-2004.65 to MSL (Plant Datum), 1.43 ft is added to the NAVD88-2004.65 elevation (see Appendix A). For the purpose of this report, elevations referenced as MSL refer to the Plant Datum. Note that for this location, MSL is not equivalent to the National Geodetic Vertical Datum of 1929 (NGVD29), or the Mean Sea Level Datum of 1929. This is due to ongoing settlement in the Mississippi Delta region.

1.6 References

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

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. (ADAMS Accession No. ML12053A340)

WSES, 2013. WSES Updated Final Safety Analysis Report, 2013, See AREVA Document No. 38-9243507-000.

2.0 INFORMATION RELATED TO THE FLOOD HAZARD

2.1 Detailed Site Information

The WSES site is located on the west (right descending) bank of the Mississippi River near River Mile 130, approximately 25 miles upstream of New Orleans (See Figure 2-1). The site area consists of over 3000 acres with approximately 7500 ft of river frontage. The WSES site grade ranges from approximately 14.5 ft MSL on the south side to 17.5 ft MSL on the north side (WSES, 2013, Section 2.4.1). The river frontage of the WSES site consists of United States Army Corps of Engineers (USACE) maintained levee with a top elevation of approximately 30 ft MSL.

2.1.1 Site Layout

Figure 2-2, Site Topography and Layout, shows the WSES site layout and topography, including important features and locations related to flood hazards (AREVA, 2014).

The following buildings are identified in Figure 2-2 using acronyms:

- Fuel Handling Building (FHB)
- Dry Cooling Tower Alpha (DCT A)
- Dry Cooling Tower Bravo (DCT B)
- Wet Cooling Tower Alpha (WCT A)
- Wet Cooling Tower Bravo (WCT B)
- Reactor Auxiliary Building (RAB)
- Nuclear Plant Island Structure (NPIS)
- Main Steam Isolation Valve Area (MSIV East and MSIV West)

2.2 Current Design Basis Flood Elevation

The current design basis and related flood elevation from natural sources is described in the WSES FSAR (WSES 2013, Section 2.4) and in the Fukushima Flooding Walkdown Report Engineering Report for Entergy Waterford Steam Electric Station Unit 3 NTTF Recommendation: 2.3 Flooding (Walkdown Report) (WSES, 2012) required as part of the response to the 10 CFR 50.54(f) letter.

The design basis flooding event at WSES is a levee failure during a Probable Maximum Flood on the Mississippi River (PMF) and a Probable Maximum Hurricane (PMH) at the mouth of the Mississippi River. This results in a maximum Design Basis Flood Elevation (DBFE) of 27.6 ft MSL (WSES, 2013, Section 2.4).

Note that the information and elevations indicated in Section 2.2 (including subsections) are taken from the WSES FSAR (WSES, 2013) and the WSES Walkdown Report (WSES, 2012).

Additionally, Probable Maximum Precipitation (PMP) induced ponding in the DCT areas is postulated to remain below a height of 1.6 ft, which is below any SSCs important to safety in those areas.

2.2.1 Elevation of Safety Structures, Systems and Components

All SSCs important to safety are flood protected because they are enclosed in a rectangular box-like reinforced concrete structure 380 ft. long, 267 ft. wide, and extending 64.5 ft. below grade known as the NPIS. The NPIS wall has a minimum protection elevation of 29.25 ft MSL. (WSES, 2012)

There are a total of seven exterior, flood-protected access doors below 29.25 ft MSL which prevent flood waters from entering the NPIS. In the Reactor Auxiliary Building there are three doors located in the east exterior wall, and two located in the west exterior wall above 21 ft MSL. In the Component Cooling Water System area there are two flood doors located in the west exterior wall above 21 ft MSL. In the Fuel Building area there is one removable flood-protected gate (modified to be welded shut) located by the spent fuel cask decontamination area above elevation 20 ft MSL. (WSES, 2012)

The Dry Cooling Towers are located within the NPIS wall, but are open vertically to the atmosphere. As a result, there is potential for precipitation to infiltrate directly into the DCT areas. Inside DCT A and DCT B, there are sump pump motors and motor control centers (MCCs) for the ultimate heatsink (UHS) that are potentially vulnerable to flooding. The critical heights (above building slab) for SSCs inside DCT A and DCT B are summarized below (WSES, 2001). Flooding can exceed the height of the sump pump motors without directly impacting plant safety, but exceeding the height of the UHS MCCs would result in loss of the UHS.

Dry Cooling Tower	Sump Pump Motor	Motor Control Center for UHS
Alpha (DCT A)	1.51 ft	1.66
Bravo (DCT B)	1.41 ft	1.65

2.3 Current Licensing Basis Flood Protection and Mitigation Features

The NPIS common foundation mat and exterior wall system are designed to withstand all loadings and postulated floods as well as to minimize water intrusion. All exterior doors of the NPIS at plant grade or below the DBFE, which lead to areas that house and protect SSCs important to safety, are designed as flood protection doors to withstand the hydrostatic pressures due to the DBFE and prevent water intrusion. Four valves form the flood barrier for the FHB by providing a barrier between the Spent Fuel Pool Cask Decontamination Area (open to the train bay which is not flood protected) and the FHB sump. (WSES, 2012)

Additionally, each DCT cell, and open area adjacent to the cells, is provided with area drains. The WCTs are provided with overflows at their high water level elevations, which spill onto the open areas adjacent to them. All area drains in each Cooling Tower area are interconnected by a network of drainage piping which terminates at an area drain sump for DCT A and at an area drain sump for DCT B. Each drain area sump is provided with a set of motor driven sump pumps. Each cooling tower area is also provided with a diesel powered sump pump. During the design basis Probable Maximum Precipitation (PMP) event, it is assumed that one motor driven sump pump is engaged within 30 minutes of the onset of the event, and the diesel powered sump pump is engaged within 3 hours of the onset of the event.

The lowest elevation of the FHB (-35 ft MSL) is considered as rain water storage capability for the DCT areas. Water level equalization between the two areas occurs through four 4 inch pipes installed under two door sills located at each side of the FHB. To maintain negative pressure in the FHB, these pipes have two flappers installed, one per train. These flappers do not impede the flow of water into the FHB. Two-thirds of the pipes need to remain unblocked to maintain the necessary equalization rate.

The FHB, RAB, and Reactor Building, have roof drains. There are a combined 21 drains of various sizes (4, 5, and 6 inch) credited for these three buildings. There are also 14 scuppers on the RAB roof. The FHB and RAB must maintain two-thirds of their roof drainage capacity.

2.3.1 CLB Flood Causing Mechanisms

The potential impacts from several flood causing mechanisms are evaluated in the WSES FSAR (WSES, 2013, Section 2.4). These events include: PMP over the plant site; Levee failure during PMF; and PMH induced Probable Maximum Storm Surge (PMSS) at the mouth of the Mississippi River; PMH PMSS through Barataria Bay (with coincident wind waves); Probable Dam Failures, Seismically Induced; Probable Maximum Surge and Seiche Flooding; Probable Maximum Tsunami Flooding; Ice Effects; and Cooling Water Canals. Of these flood mechanisms, the controlling scenarios are the PMH induced PMSS at the mouth of the Mississippi River coincident with a PMF and Levee Failure, and the PMP over the site with respect to ponding in the DCT areas.

The DBFE for the WSES site is due to the Combined Effect scenario of a PMH induced PMSS at the mouth of the Mississippi River coincident with a PMF and levee failure at the site. The resulting flood level is a stillwater elevation of 27.6 ft MSL. This scenario does not include appreciable wind-generated waves due to the configuration of the flood. The design basis flood level for the DCT areas due to rainfall runoff is 1.6 ft of ponding. (WSES, 2012)

2.4 Licensing Basis Flood-Related and Flood Protection Changes

There have been no significant changes to the licensing basis with respect to flooding or flood protection.

2.5 Watershed and Local Area Changes

2.5.1 Watershed Changes

The Lower Mississippi River (including the Mississippi River segment which borders WSES) is frequently navigated, and the USACE New Orleans District is responsible for maintaining navigable conditions. As part of this responsibility, USACE actively maintains revetments and flood control structures that have been constructed to minimize the risk of channel diversions, bank erosion, and instability. The absence of major morphological changes in the region of the river adjacent to the site indicates that the river channel segment bordering WSES has not migrated in the past even though other parts of the river have exhibited a tendency to migrate. (See Section 3.8)

An extensive levee system has been constructed in southeast Louisiana. The levee system and hydraulic control structures are owned and maintained by the USACE and interconnect the Mississippi River floodplain with the Atchafalaya River floodplain for the purposes of maintaining channel stability, navigation and flood control. The USACE structures along the Mississippi River and Atchafalaya River were designed based on the Mississippi River and Tributaries Project Design Flood. In the past few years (subsequent to Hurricane Katrina in 2004), a new round of levee repairs and improvement projects have been completed, which are referred to as the Greater New Orleans Hurricane and Storm Damage Risk Reduction System. (See Section 3.4.1)

2.5.2 Local Area Changes

During the initial phase of construction from 1975 to 1978 the plant settled approximately 0.75 ft resulting in an NPIS minimum protection elevation change from 30.0 ft MSL to 29.25 ft MSL (WSES, 2012). Since that initial settlement, ongoing regional settlement has resulted in a current NPIS wall minimum height of 29.18 ft MSL (AREVA, 2014).

2.6 Additional Site Details – Walkdown Results

The findings reported in the Walkdown Report (WSES, 2012) indicate that there is sufficient protection available at the site to ensure the safe operation of the plant in the event of a design basis flood. The inspections included all features credited for protection from the design basis flood, including all penetration or door seals below the minimum flood protection elevation of 29.25 ft MSL.

During the walkdowns, conditions that did not meet the acceptance criteria were entered into the Corrective Action Program at WSES. The operability reviews of these conditions determined that the issue did not prevent safe plant operation or create a flooding risk for any safety-related equipment at the site. Based on the results of the visual inspections and the information provided in the current licensing basis at WSES, safe operation of the plant would be maintained in the event of a design basis external flooding event. (WSES, 2012)

2.7 References

AREVA, 2014. Waterford Nuclear Generating Station – WSES: Aerial Mapping Validation Report, prepared by McKim & Creed, July 2014. See AREVA Document No. 38-9226991-000.

WSES, 2001. WSES Calculation EC-M99-010, Revision 0-2, August 2001, See AREVA Document No. 38-9243507-000.

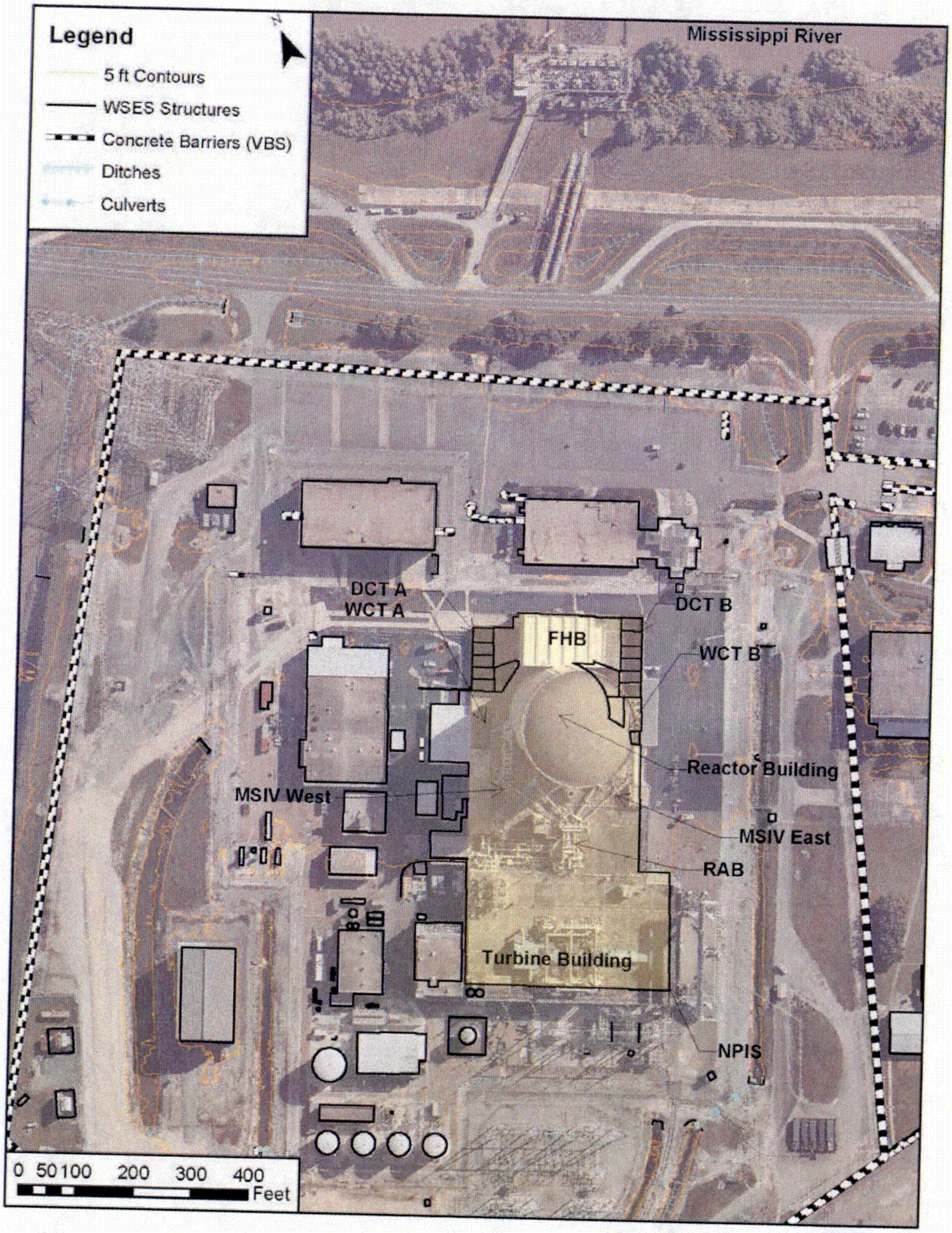
WSES, 2012. Flooding Walkdown Report - Entergy's Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident, Waterford Steam Electric Station, Unit 3 (Waterford 3), Docket No. 50-382, License No. NPF-38, 2012. (ML12333A147)

WSES, 2013. WSES Updated Final Safety Analysis Report, 2013, See AREVA Document No. 38-9243507-000.

Figure 2-1: Site Location Map

Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 2-2: Site Topography and Layout



Any illegible text or features are not pertinent to the technical purposes of this document. Site topography, orthoimagery, and plant structure delineation from AREVA, 2014.

3.0 FLOOD HAZARD RE-EVALUATION

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

3.1 Local Intense Precipitation

3.1.1 Local Intense Precipitation – External to NPIS

This section addresses the potential for flooding at WSES due to Local Intense Precipitation (LIP) outside of the NPIS wall. The potential for flooding due to the LIP inside the NPIS is addressed in Section 3.1.2. The LIP event is a distinct flooding mechanism that consists of a short-duration, locally heavy rainfall centered upon the plant site itself.

This section summarizes the External LIP evaluation documented in AREVA Calculation No. 32-9226993-000 (AREVA, 2014a).

3.1.1.1 Method

The HHA approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the LIP and the resultant water surface elevation at WSES. The HHA approach for external LIP used the following steps:

1. Develop LIP/PMP inputs.
2. Develop the FLO-2D computer model with site features.
3. Perform flood simulations in FLO-2D to calculate maximum flood depths throughout the WSES site (not including ponding inside the NPIS/DCT areas).

3.1.1.2 Results

All SSCs important to safety are located within the NPIS which is a *"reinforced concrete box structure with solid exterior walls"* (WSES, 2013) and is protected from external flooding to elevation 29.18 ft MSL (27.7 ft, NAVD88; AREVA, 2014b). Exterior doors that lead to areas containing safety-related equipment within the NPIS, that are located below the flood protected elevation, are watertight (WSES, 2013). The power block of WSES is virtually enclosed by concrete barriers, referred to herein as the Vehicle Barrier System (VBS), and is constructed from concrete blocks placed end to end to create a continuous barrier.

The FLO-2D model for LIP flooding analysis at WSES uses 2014 topographic mapping results (AREVA, 2014b) to generate ground elevations and associated flood water surface elevations.

3.1.1.2.1 Precipitation

Note that the CLB evaluation of PMP for WSES used rainfall from Hydrometeorological Report No. 33 (HMR-33) (NOAA, 1956): 11.67 inches over one hour and 30.7 inches over 6-hours. HMR-33 has since been superseded by Hydrometeorological Reports No. 51 (HMR-51) (NOAA, 1978) and No. 52 (HMR-52) (NOAA, 1982). HMR-51 and HMR-52 provide generic PMP guidance for areas in the United States east of the 105th meridian. This LIP evaluation conservatively uses rainfall parameters from HMR-51 and HMR-52, in accordance with NUREG/CR-7046 (NRC, 2011, Section 3.2). Note that since the results for the external LIP obtained using HMR-51 and HMR-52 were acceptable, the external LIP analysis did not require using site-specific PMP information to refine the generic HMR-51 and HMR-52 inputs, in accordance with the HHA approach.

A front-loaded temporal distribution was used as per NUREG/CR-7046 (NRC, 2011). The sub-divisions from the second hour to the sixth hour are based on recommendations in NUREG/CR-7046, Appendix B. The one-square-mile, one-hour duration PMP was estimated from HMR-52 (NOAA, 1982) to be 19.4 inches. The one-hour PMP was further subdivided into shorter duration increments based on the methodology of HMR-52. The sub-one hour division ratios are 0.32-, 0.50-, and 0.73 in the first 5-, 15-, and 30-minutes, respectively (Appendix B; NOAA, 1982). The ten-square-mile, six-hour duration PMP was estimated from HMR-51 (NOAA, 1978) as 32 inches. The ten-square-mile, six-hour duration PMP hyetograph is shown in Figure 3-1. The 5-minute incremental rainfall depth from the second hour to the sixth hour was calculated as 0.21 ft (i.e. 2.52 inches) and is the difference between the ten-square-mile, six-hour duration PMP and the one-square-mile, one-hour duration PMP spread evenly in 5-minute increments over the five hour period (NRC, 2011).

3.1.1.2.2 FLO-2D Model Development

Due to anticipated unconfined flow characteristics outside of the NPIS, a two-dimensional hydrodynamic computer model, FLO-2D, was used for this calculation. 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 (AREVA, 2014c). 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 (AREVA, 2014b).

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

3.1.1.2.3 FLO-2D Computer Model with Site Features

The FLO-2D model developed for the LIP analysis was based on WSES site features including: topography, site location, VBS layout, and structures. The selected grid element size for the project was 10 ft by 10 ft. The elevation data used to develop the FLO-2D model consist of 2014 DTM data (AREVA, 2014b) for WSES. Flow obstructions due to buildings were also included in the model. The main input parameters for the WSES FLO-2D model include:

Elevation Data: Elevation grid for the project area was calculated based on the site topographic survey plan for WSES (AREVA, 2014b). The surveyed topographic map is in Louisiana South (1702) State Plane Coordinate System, North American Datum (NAD) 1983 (Conus) (horizontal) datum and elevations are in NAVD88 (GEOID 12A) (vertical) datum. The unit of the survey is U.S. ft. The elevation data imported into the FLO-2D model is supplemented by surveyed information.

The methodology of the topographic survey was Light Detection And Ranging (LiDAR), with resulting data provided in AutoCAD™ format (AREVA, 2014b). The DTM used was extracted from the AutoCAD file. Additional elevation data was used based on the topographic site plan produced along with the DTM. Topographic data for WSES was developed based on a site-specific aerial survey using methodology consistent with the need for first-order level of accuracy. The topographic survey performed in 2014 at WSES was required to meet the American Society for Photogrammetry and Remote Sensing (ASPRS) Class I Accuracy Standard for 1" = 100' planimetrics and 1-foot contour intervals, with +/- 1 ft horizontal accuracy, +/- 0.33 ft Root Mean Square Error (RMSE) vertical accuracy for 1 foot contours and +/- 0.17 ft RMSE vertical accuracy for spot elevations and DTM points, at well-defined points. Additional designated critical structures and locations with respect to site flooding impacts were identified and surveyed with a vertical accuracy of +/- 0.17 ft. The methodology of the topographic survey was aerial LIDAR mapping of the site with sufficient control points for calibration meeting the mapping standard, and conventional ground survey loops for the critical structures and locations (AREVA, 2014b).

FLO-2D grid element elevation data was interpolated based on imported DTM points from the topographic survey of the site that were added to the working region. Interpolation methods available in FLO-2D include:

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

Model grid elevations cannot be more accurate than the survey they are based upon. Therefore model grid elevations have a minimum level of uncertainty of +/- 0.17 ft. A minimum of two closest DTM points within the vicinity of a grid element was used in computing grid elevations. The density of spot elevations on the DTM provided for adequate coverage for each grid element. Interpolated grid elevations at all critical points were spot checked against the survey elevations and adjustments were made as needed. Model interpolation errors are therefore believed to be very minimal.

Uncertainty regarding onsite flood elevations is generally limited to the level of accuracy of the site survey. The nature of the two dimensional flow model is such that the impact of potential inaccuracy in the elevation of any single grid element is generally mitigated by the surrounding grid elements. LIP results were computed as maximum water surface depths, which was then compared to the known height of flood protection at critical elements, thus reducing uncertainty related to potential issues with elevation datum normalization.

Buildings: Buildings at WSES were incorporated into the FLO-2D model by manually adjusting grid element elevations based on the site survey and the high resolution orthoimagery (AREVA, 2014b). Area Reduction Factors and Width Reduction Factors were not used. Grid elements that were completely within the aerial extent of a building were assigned elevations at least 5 ft higher than the surrounding topography. Uniform elevations were assigned to grid elements representing a single building to ensure that runoff from rooftops are uniformly distributed to the surrounding areas. For buildings with different rooftop elevations adjacent to each other (as estimated based on DTM data (AREVA, 2014b), the relative change in rooftop elevations were represented as a 2-ft relative difference in building grid element elevations. This ensures that general flow directions of runoff from rooftops are considered.

Outflow Grid Elements: Grid elements along the computational boundary were selected as outflow grid elements, as shown in Figure 3-2.

Levee Structures: The VBS at WSES was modeled using the levee structure component in FLO-2D. Levee structures in FLO-2D confine flow on the floodplain surface by blocking one or more of the eight available flow directions. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad crested weir flow equation with a weir coefficient of 3.1 (FLO-2D, 2014). The top elevation of the VBS was set at elevation four ft above the underlying grid elevation (WSES, 2012). With the exception of the vehicle opening on the Northeastern end of the site, the site is fully enclosed within the VBS.

Infiltration and Surface Roughness: Rainfall was directly transformed into runoff within FLO-2D. No initial abstractions and/or infiltration were used.

The land use categories were selected based on aerial photography assessment (AREVA, 2014b). The Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete and asphalt surfaces to 0.20 for areas with short trees.

3.1.1.2.4 LIP Simulation Results

The results of the WSES FLO-2D LIP model are summarized in Table 3-1. Locations of interest are shown on Figure 3-3. The LIP maximum water surface elevations outside the NPIS range from 16.4 ft MSL (15.9 ft NAVD88) near the southeast side of the Independent Spent Fuel Storage Installation (ISFSI) pad to 20.5 ft MSL (19.1 ft NAVD88) between the West Side Access and Tool Room. The maximum flow depths range from 0.5 ft at the southeast side of the NPIS to 1.1 ft at the southeast side of the ISFSI. The maximum velocities range from 0.2 ft per second (fps) at the southeast side of the ISFSI to 2.2 fps on the southwest side of the NPIS. Flood water within the VBS generally flows away from the relatively higher grounds in the vicinity of the NPIS toward the relatively lower grounds at the southeast and southwest ends of the site and overtops the VBS to exit the site.

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

3.1.2 Local Intense Precipitation – Internal to NPIS

This section addresses the flooding inside the NPIS (specifically the DCT areas and the MSIV areas) at WSES due to rooftop runoff and direct precipitation from the LIP event. The LIP event results from the PMP event centered over the site area and the local watershed.

This section summarizes the NPIS LIP evaluation documented in AREVA Calculation No. 32-9231496-000 (AREVA, 2015a).

3.1.2.1 Background

The following information is paraphrased from the WSES FSAR (WSES, 2013), unless otherwise noted. All SSCs important to safety at WSES are located within the NPIS. The NPIS is a "*reinforced concrete box structure with solid exterior walls*" according to the WSES FSAR and is protected from external flooding to approximately 13 ft above general site grade. Exterior doors that lead to areas containing safety-related equipment within the NPIS that are located below the flood protected elevation are watertight. Relevant NPIS details are shown on Figure 3-4.

As shown in Figure 3-4, the NPIS consists of DCTs A and B, open areas adjacent to the DCTs, WCTs A and B, the FHB, the Reactor Building, and the RAB. DCTs A and B and the adjacent open areas are not covered by roofs and are open to the atmosphere allowing for falling precipitation and runoff from adjacent roofs to accumulate. The MSIV areas, located in the wings of the RAB, are also susceptible to falling precipitation. The FHB, Reactor Building, and RAB, with the exception of the MSIV areas, are roofed structures with varying roof elevations.

For the purpose of this report, the term "DCT Basins" is defined as the areas in and around the DCTs that are hydraulically-connected at the mat foundation finished floor elevation of -34.75 ft MSL to the DCT sump pumps. These areas include the DCTs and the open areas adjacent to the DCTs (Figure 3-4). Equalization of ponding depths within each DCT Basin occurs via backflow of the floor drainage system (WSES, 1992). It should be noted that four 4-inch-diameter pipes hydraulically connect the sub-basement of the FHB to DCT Basin A and another four 4-inch-diameter pipes connect the sub-basement of the FHB to DCT Basin B (Figure 3-5; WSES, 1986; WSES, 1991c). The pipes have a flapper (i.e., check) valve system that allows water to flow into but not out of the FHB (WSES, 1986).

There are six air intake/exhaust openings on the top of the north side of the FHB that are open to the atmosphere allowing for falling precipitation to accumulate in the sub-basement of the FHB, which has a finished floor elevation of -34.75 ft, MSL (WSES, 2013, Section 9.3.3.2.1.2).

Roof drains, scupper drains, and floor drains convey runoff from the Reactor Building dome, RAB roof, and MSIV areas to pipes that discharge outside of the NPIS. The drains were assumed to be 100-percent open because the area of the grate covering the respective pipe openings is much larger than the pipe openings. In addition, WSES has and will create additional operational plans to ensure that these pipes are free from debris that may block flow through them (WSES 2014a, WSES 2015a, WSES 2015b). These drains and scuppers are assumed to discharge freely away from the NPIS because roof top elevations are at least elevation 41 ft MSL or higher, indicating that backwater effects at the drain/scupper outlet are unlikely given that typical site grade is between 14.5 and 17.5 ft MSL (WSES, 1997).

3.1.2.2 Methodology

This report determines the ponding depths within the Dry Cooling Tower (DCT) Basins and the Main Steam Isolation Valve (MSIV) areas (see Figure 3-4); locations that contain SSCs important to site safety.

The methodology is based on the use of a conservation of mass (i.e., volumetric balance) approach in a simple relation of inflow to, outflow from, and changes in storage within the NPIS areas as follows:

Inflow = Outflow + Change in Storage (Equation 1)

Where:

Inflow = Runoff from contributory areas and/or direct inputs from rainfall, cubic ft;
Outflow = Discharge from pumping and/or drains (connected to piping leading out of the NPIS), cubic ft;
Change in Storage = Ponding within NPIS areas, cubic ft.

This analysis also incorporates the following updates relative to the existing design basis analysis:

- This analysis uses site-specific PMP values developed in a separate calculation (AREVA, 2015b). The 6-hour PMP includes embedded 5-minute, 15-minute, 30-minute, and 60-minute duration PMP values whereas the existing design basis analysis evaluated only the 6-hour and 1-hour duration PMP. Thus, this analysis uses greater precipitation intensities than the existing design basis analysis.
- The mass balance time step increment used for this analysis ranges from one minute to five minutes to allow for calculation of resultant flood depths based on PMP durations as short as five minutes. The existing design basis analysis uses a 30-minute to one-hour time step increment. Thus, this analysis uses shorter time step increments than the existing design basis analysis.
- The variation of pipe flow with flood depth (e.g., hydraulic head) is accounted for in this analysis. The existing design basis analysis used a volumetric approach that does not appear to consider hydraulic limitations due to pipe characteristics and available hydraulic head at each time step.
    - The capacities of the scuppers and roof drains on the Reactor Building dome, RAB Roof A1, RAB Roof A2, RAB Roof A3, RAB Roof B1, RAB Roof B2 and RAB C1 were assumed to be 100-percent open because the area of the grate covering the respective pipe openings is much larger than the pipe openings. In addition, WSES has and will create additional operational plans to ensure that these pipes are free from debris that may block flow through them (WSES 2014a, WSES 2015a, WSES 2015b).

This updated analysis has a modeling period of 7-hours (i.e., one hour longer than the LIP). Although there is no precipitation during the last hour of the modeling period, the outflows from pumping are considered to continue.

3.1.2.2.1 Rainfall

A site-specific meteorology study was performed in accordance with the HHA methodology to calculate the LIP at the 1-square mile area size for duration of 5-, 15-, and 30-minutes and 1- through 6-hours at WSES. This site specific evaluation is documented in a separate calculation (AREVA, 2015b).

3.1.2.2.2 DCT Basins

The method of analysis for determining the ponding depths in the DCT Basins is described in general terms below:

1. Calculate inflows:
    a. Calculate temporally distributed precipitation.
    b. Calculate areas that will directly contribute to ponding inside the NPIS and transform rainfall to runoff for these areas.
    c. Calculate potential overflows from surrounding building roofs by transforming rainfall to runoff and apportioning based on roof configurations and parapet storage.
2. Calculate storage volumes:
    a. Calculate areas where ponding can occur (i.e. storage areas).
    b. Calculate reductions in storage volume due to equipment foundations, internal walls, etc.

3. Calculate outflows:
   a. Calculate the outflows from Reactor Building and RAB roof drains and scupper drains.
   b. Calculate sump pump discharges.
   c. Calculate rating curve for flow into the FHB.
4. Calculate ponding depth in each DCT Basin using the mass balance approach and computed depth-to-volume relationship for both basins. Due to the differing contributing areas and storage areas between DCT Basin A and DCT Basin B, the two areas were modeled separately to account for the possibility of differing ponding depths.

3.1.2.2.3 MSIV Areas

The two MSIV areas of the RAB are not hydraulically connected to either DCT Basin but are potentially susceptible to flooding due to direct precipitation (i.e., the areas are open to the atmosphere allowing for falling precipitation to accumulate). The MSIV areas have a parapet wall height of 24.5 ft MSL (WSES, 2001c), which prevents overflow from the MSIV areas to the DCT Basins because the height of the parapet walls is significantly greater than the LIP depth (Figure 3-4).

The existing design basis calculation for ponding within the MSIV areas, Calculation ECM13-001 (Entergy, 2014), indicates that the MSIV areas are protected from overflowing rainwater from the surrounding RAB roofs (see "Noncontributing Roof Area" in Figure 3-4) by a 15 inch high parapet wall and that the RAB roof drains, located in the surrounding roofs of the RAB, are adequately sized to handle the PMP rainfall (Entergy, 2014). This report follows the same approach (i.e., mass balance computations) used in the existing design basis calculation (Entergy, 2014), but it refines two design inputs to reevaluate the maximum ponding depths within the MSIV areas including:

- Updated PMP values from site-specific meteorology study; and
- Floor drains and scupper drains discharge during LIP.

The method of analysis for calculating the ponding depths in the MSIV areas is described in general terms below:

1. Calculate inflows:
   a. Use temporally distributed precipitation previously calculated.
   b. Calculate areas that will directly contribute to ponding inside the MSIV areas and transform rainfall to runoff for these areas.
   c. Calculate potential overflows from surrounding building roofs by transforming rainfall to runoff and apportioning based on roof configurations and parapet storage.
2. Calculate areas where ponding can occur (i.e. storage areas).
3. Calculate outflows:
   a. Calculate the outflows from Reactor Building and RAB roof drains and scupper drains.
   b. Calculate outflows from the MSIV area drains.
4. Calculate ponding depth in each MSIV area using the mass balance approach.

No infiltration losses were considered in the calculation for the ponding depths within the DCT Basins or the MSIV areas (i.e., the site and contributory areas are assumed to be 100-percent impervious).

3.1.2.3 Results

3.1.2.3.1 Calculate Inflows

<u>Calculate Rainfall</u>

The LIP event is a distinct flooding mechanism that consists of locally heavy rainfall centered over the site area and the immediate local watershed. Initial analyses indicated that using generalized PMP values from HMR-51 and HMR-52 would exceed the current design basis for the DCT SSCs important to safety. Therefore, in accordance with the HHA approach, a site-specific meteorology study was performed to refine the generalized PMP estimates provided by HMR-51 and HMR-52. The site-specific PMP study incorporates numerous improvements over the generalized HMR-51 / HMR-52 guidance, including the usage of 40+ years of additional precipitation records and several technological advancements in the analyses of historic extreme storms. Section 5.2 of American National Standards Institute/ American Nuclear Society (ANSI/ANS)-2.8-1992 (ANS, 1992) indicates that parameters of the PMP should be determined by a meteorological study using a storm based approach. This analysis followed the storm-based approach as followed in HMR-53 (NOAA, 1980) and HMR-51 (NOAA, 1978). The World Meteorological Organization (WMO) Manual for PMP determination (WMO, 2009) recommends this same approach. Figure 3-6 displays the major steps used in the calculation of the 1- and 6-hour, 1-square mile PMP.

The initial step in the development of the site-specific PMP values was to identify a set of storms which represent rainfall events that are PMP-type local storm events. This included all storms used in HMR-51 (NOAA, 1978) and HMR-52 (NOAA, 1982), all storms included in the USACE Storm Studies analyses (USACE, 1973), as well as more recent storms. Emphasis was placed on storms which produced high intensity rainfall over short durations (6 hours or less). The storm search domain included a region from the Central Plains (south of 41°N latitude) through the Gulf of Mexico, east/west to locations within +/- 1,000 ft of the site elevation (Figure 3-7). These general guidelines for the storm search domain and transposition limits are similar to those described in HMR-51, Section 2.4.2 (NOAA, 1978). The storms which are important for LIP development at the WSES site are known from previous storm analyses and storm maximization completed in the region (e.g. NOAA, 1978, NOAA, 1982, AWA, 2008, AWA, 2012, AWA, 2014).

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

This resulted in 22 events being evaluated for use in the site-specific PMP calculation (Figure 3-8 and Table 3-2). Eleven of the storms were not covered by the HMR or USACE analyses. For these newly identified extreme rainfall events without published Depth-Area-Duration (DAD) analyses, hourly rainfall grids and DADs were computed using the SPAS computer program (Parzybok et al., 2014). There are two main steps in the SPAS DAD analysis: 1) the creation of high-resolution hourly precipitation grids and 2) the computation of Depth-Area (DA) rainfall amounts for various durations. Because this process has been the standard for many years (all DAD produced by the NWS in HMR 51 used this procedure) and holds merit, the SPAS DAD analysis process used in this study attempts to mimic the NWS procedure as much as possible. By adopting this approach, consistency between the newly analyzed storms and the hundreds of storms already analyzed by the NWS is achieved.

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

Energy Regulatory Commission (FERC) and state dam safety regulators (AWA, 2008 and AWA, 2013, respectively).

Once each storm is maximized in-place, it is then transpositioned from its original location to the site. Transfer of a storm from where it occurred to a location that is meteorologically and topographically similar is known as storm transpositioning. The transpositioning process accounts for differences in moisture and elevation between the original location and WSES. For a given storm event to be considered transpositionable, there must be of similar meteorological / climatological and topographical characteristics at its original location versus the new location. The general guidelines described in HMR-51 Section 2.4.2 are followed in this analysis.

The process produces a total adjustment factor that is applied to the original rainfall data for each storm. The result represents the maximum rainfall each storm could have produced at WSES had all factors leading to the rainfall been ideal. Table 3-2 provides each value used in this calculation, including the observed or derived 1-hour and 6-hour rainfall, the calculated total adjustment factor for each storm, and the resulting total adjusted 1- and 6-hour rainfall amounts.

After the maximization and transposition factors were calculated for each storm, the results were applied to the maximum 1- and 6-hour value for each storm to calculate the maximized 1- and 6-hour 1-mi^2 value. The largest of these values results in the site-specific LIP for the site (Table 3-3). After adjustments were applied, the Thrall, TX September 1921 storm had the highest 1-hour and 6-hour rainfalls, with several other storms providing support with slightly smaller values. Note that the Smethport, Pennsylvania July 1942 storm, which produced the 4- and 6-hour world record rainfall, was outside of the transposition limits to WSES shown in Figure 3-7, therefore this storm did not influence the LIP values at WSES. The refined transposition limits used in this calculation result in lower LIP values compared to HMR-52 for locations where the Smethport storm apparently influenced PMP values. Smoothing of the PMP/LIP contours in HMR-51 and HMR-52 necessarily had to encompass the Smethport maximized in-place rainfall far beyond its explicit transposition limits. This over-envelopment effect extended well beyond the intended transposition limits of the Smethport storm (e.g. HMR-52 Figure 26) because the PMP/LIP contours required smoothing and fitting over surrounding regions.

For final applications, the 1-hour value is then required to be split into sub-hourly increments of 5-, 15-, 30-minutes. Therefore, the ratios derived in HMR-52 (Figures 36-38 of HMR-52; NOAA, 1982) were applied specific to the site location. The PMP depths results from the site-specific meteorology study are shown in Table 3-3 and Table 3-4.

This site-specific LIP study provided differences in LIP values from those presented in HMR-52. However, this calculation explicitly addressed elevation, used updated in-place maximization factors, and explicitly defined transposition limits for each storm considered. These improved site-specific evaluations result in more accurate PMP values because specific characteristics of meteorology and topography of the site were considered, while in HMR-51 and HMR-52 they were not.

Calculate Directly Contributing Areas

Open areas subject to direct precipitation were considered to be directly contributing areas. The roof tops of surrounding buildings were considered separately since only a portion of rainfall on the rooftops can overflow into the areas of interest (i.e., the DCT Basins and the MSIV areas).

DCT Basins Directly Contributing Areas

The "directly contributing areas" are open to the atmosphere and are subject to direct precipitation-induced ponding. DCT Basin A consists of the open areas on the west side of the NPIS including the wet cooling tower area. Note that the enclosed walkway over DCT Basin A (labeled as "Full Contributing Roof with Storage Beneath" in Figure 3-4) is a flat roof with no parapet wall. Therefore, it was conservatively assumed that the precipitation that fell on this roof would accumulate in the open area below the roof at elevation -34.75 ft MSL and would contribute to flooding at DCT Basin A. DCT Basin B consists of the open areas on the east side of the NPIS including the wet cooling tower area (see Figure 3-4 and Figure 3-10). It was assumed that 100-percent of the internal walls and 50-percent of the external walls of the DCTs contribute to ponding within the DCT Basins based on visual inspection.

There are six air intake/exhaust structures on the north side of the FHB (Figure 3-9; WSES, 2011a and WSES, 2011b). These horizontal openings were considered directly contributing areas since they are open to the atmosphere and allow precipitation to pool within the sub-basement of the FHB (WSES, 2013, Section 9.3.3.2.1.2). The volume of water added to the system by these openings affects the available storage volume within the FHB that can be used by each DCT Basin.

Areas directly contributing to ponding were calculated using AutoCAD® and were based on the site survey data (AREVA, 2014b). Figure 3-10 outlines the directly contributing areas and the calculated values for DCT Basin A, DCT Basin B and the FHB air intake and exhaust openings are summarized in Table 3-5.

MSIV Areas Directly Contributing Areas

The direct contributing area of MSIV East consists of the open area (i.e., covered by pervious overhead grating) on the east wing of the RAB and the contributing area of MSIV West consists of the open area on the west wing of the RAB (see Figure 3-4). Ponding due to direct precipitation inflow is based on the MSIV areas that were taken from the existing design basis calculation for ponding within MSIV areas, Calculation ECM13-001 (Entergy, 2014) because this information was not captured by the site survey (AREVA, 2014b). The area of MSIV East was calculated at 4,051 square ft and the contributing are of MSIV West was calculated at 4,140 square ft (Entergy, 2014).

Subsequent to performance of this evaluation, the MSIV storage areas were revised to 4,088 square ft for each MSIV area.

Calculate Overflow Volumes from Surrounding Roof Areas

The surrounding roof areas with parapet walls adjacent to the open areas of the DCT Basins (Figure 3-4) will contribute to ponding if the depth of pooling on the roof overtops the parapet walls. The volume of water that would overtop the parapet walls and flow into a particular area was calculated using a percentage based on the perimeter of roof that water can overtop and the length of roof that is shared with the area of interest. The overflow from the parapet walls was assumed to occur uniformly around the perimeter available for overflow. It was considered that the roof of the West Side Access Building, adjacent to WCT A, drains away from the NPIS and does not contribute to ponding (see Attachment 8.3 of WSES, 2001b).

DCT Basins Overflow Volumes from Surrounding Roof Areas

The following list includes the roofs and roof sections analyzed that had the potential to overflow and affect the DCT basins:

- FHB Roof: Fuel Handling Building Roof that does not overflow to other areas.
- RAB Roof A1: Roof Section A1 of Reactor Auxiliary Building at El. 106.5 that drains to DCT A.
- RAB Roof A2: Roof Section A2 of Reactor Auxiliary Building at El. 91.0 that drains to DCT A.
- RAB Roof A3: Roof Section A3 of Reactor Auxiliary Building at El. 66.0 that drains to DCT A.
- RAB Roof B1: Roof Section B1 of Reactor Auxiliary Building at El. 80.5 that drains to DCT B.
- RAB Roof B2: Roof Section B2 of Reactor Auxiliary Building at El. 41.0 that drains to DCT B.
- RB Dome: Roof Section of Reactor Building that drains to DCT A and B.

Fuel Handling Building Roof

The parapet walls on the roof of the FHB (Figure 3-4) are 33 inches high (WSES, 1991b) and its roof drains convey flow beyond the NPIS (WSES, 2013). If the roof drains were completely blocked, the maximum depth of ponding would be equal to the 6-hour PMP of 27 inches (AREVA, 2015b). The 6-hour PMP would be contained within the parapet walls of the FHB roof and no overflow into the DCT Basins would occur. Water falling on the FHB was therefore not considered to contribute to ponding within the DCT Basins.

Reactor Auxiliary Building Roofs

The RAB is comprised of roofs with varying elevations. The MSIV areas that are directly adjacent to the Reactor Building at elevation 46 ft MSL (Figure 3-4) have a tall parapet wall that is 24.5 ft high (WSES, 2001a). Flooding in the MSIV areas would not contribute to ponding within the DCT Basins because the 6-hour PMP is 27 inches.

Other RAB roofs adjacent to the DCT Basins were also considered. These roofs are labeled as "RAB Roof A1", "RAB Roof A2", "RAB Roof A3", "RAB Roof B1", and "RAB Roof B2" on Figure 3-4 and Figure 3-10. The roofs labeled as RAB Roof A1, RAB Roof A2, and RAB Roof A3 contribute water to DCT Basin A. The roofs labeled as RAB Roof B1 and RAB Roof B2 contribute water to DCT Basin B.

The RAB roofs adjacent to the DCT Basins are drained by scuppers and roof drains (WSES, 1993). The RAB Roof A1, RAB Roof A2, RAB Roof A3, and RAB Roof B1 are drained by 3-inch-diameter scuppers, and RAB Roof B2 is drained by a 4-inch-diameter roof drain (WSES, 1993). These scuppers and roof drains were considered to be available and 100-percent clear because the area of the grate covering the respective pipe openings is much larger than the pipe openings. In addition, WSES has operational plans to ensure that these pipes and grates are free from debris that may block flow through them (WSES 2014a, WSES 2015a, WSES 2015b). The parapet walls on these roofs are 1 foot (12 inches) high (WSES, 2001b). Overflow would occur if the depth of water exceeded the parapet walls.

RAB Roof A1

RAB Roof A1 is at a higher elevation than the surrounding roofs. If water on RAB Roof A1 reaches the height of the parapet wall, it would uniformly overflow the perimeter of the roof. The northern edge of RAB Roof A1 is adjacent to RAB Roof A2. The water that would overflow this portion of RAB Roof A1 is expected to contribute to the ponding water on RAB Roof A2. The mass balance computations indicate that the 3-inch-diameter scupper combined with the roof top storage is sufficient to prevent overflow to RAB Roof A2 during the LIP.

RAB Roof A2

RAB Roof A2 is at a higher elevation than the roofs to the north, west and east. Uniform overflow over these three sides occurs if water on RAB Roof A2 reaches the height of the parapet wall. The northern edge of RAB Roof A2 is adjacent to RAB Roof A3. The water that would overflow this portion of RAB Roof A2 is expected to contribute to the ponding water on RAB Roof A3. The mass balance computations indicate that the 3-inch-diameter scupper combined with the roof top storage is sufficient to prevent overflow to RAB Roof A3 during the LIP.

RAB Roof A3

RAB Roof A3 is at a higher elevation than the roofs to the north and west. The northern edge of RAB Roof A3 is adjacent to DCT Basin A. The water that would overflow this portion of RAB Roof A3 is expected to contribute to the ponding water in DCT Basin A. The mass balance computations indicate that the 3-inch-diameter scupper combined with the roof top storage is sufficient to prevent overflow to DCT Basin A during the LIP.

RAB Roof B1

RAB Roof B1 is at a higher elevation than the surrounding roofs. A portion of the northern edge of RAB Roof B1 is adjacent to RAB Roof B2. The water that would overflow this portion of RAB Roof B1 is expected to contribute to the ponding water on RAB Roof B2. The mass balance computations indicate that the 3-inch-diameter scupper combined with the roof top storage is sufficient to prevent overflow to RAB Roof B2 during the LIP.

RAB Roof B2

RAB Roof B2 is at a higher elevation than the roof to the north, which is the DCT Basin. The water that would overflow the northern portion of RAB Roof B2 is expected to contribute to the ponding water in DCT Basin B. The mass balance computations indicate that the 4-inch-diameter roof drain combined with the roof top storage is sufficient to prevent overflow to DCT Basin B.

Reactor Building Roof

The Reactor Building is a domed structure surrounded by an approximately 4-ft wide walkway with a 21-inch high parapet wall (WSES, 2002a and Section 2.4.2.3.3 of WSES, 2013). Precipitation falling over the area of the dome will pool along the perimeter at the walkway (WSES, 2002a). Once water reaches the height of the parapet wall, it spills over the parapet wall around the perimeter of the dome. A portion of the overflowing water will contribute to DCT Basin A, DCT Basin B, and the RAB roof. The circumference of the dome was calculated at 484 ft, 263 ft of which would contribute water to either DCT Basin A or DCT Basin B. Approximately 54-percent of the volume of water overflowing the parapet wall will contribute to ponding in the DCT Basins; therefore 27-percent would contribute to each DCT Basin. The volume of water that flows over the southern portion of the RB dome, adjacent to the RAB, will contribute to the MSIV areas on Figure 3-4. The volume of water that overtops this section of the Reactor Building dome will not contribute to ponding in the DCT Basins.

The Reactor Building roof walkway is drained by three 6-inch-diameter roof drains (WSES, 1993). These roof drains were considered to be available and 100-percent clear.

MSIV Areas Overflow Volumes from Surrounding Roof Areas

Reactor Auxiliary Building Roofs

The existing MSIV area calculation ECM13-001 (Entergy, 2014), indicates that the MSIV areas are protected from overflowing rainwater from the surrounding RAB roofs (see "Noncontributing Roof Area" in Figure 3-4) by a parapet wall and that the RAB roof drains, located in the surrounding roofs of the RAB, are adequately sized to handle the CLB PMP rainfall (Entergy, 2014). However, due to the updated PMP values (AREVA, 2015b) that are higher than the PMP values calculated in the existing MSIV area calculation, the overflow potential from the RAB roofs into the MSIV areas was reevaluated in this report. The roofs labeled as "RAB Roof C1", "RAB Roof C2", "RAB Roof C3", and "RAB Roof C4" shown in Figure 3-4 were included as roof areas contributing to flooding to the MSIV areas. Note that RAB Roof C5 does not contribute to flooding into the MSIV areas due to its lower elevation (46.0 ft MSL) than surrounding roof elevations.

The surrounding roof areas with parapet walls adjacent to the open areas of the MSIV areas (Figure 3-12) contribute to ponding if the depth of pooling on the roof overtops the parapet walls. The parapet wall that separates the RAB Roof C1 (former Noncontributing Roof Area" in Figure 3-4) from both the MSIV areas is approximately 17 inches high according to a plant drawing (WSES, 1991d). Note that the parapet wall storages for RAB Roofs C2, C3, and C4 were conservatively not accounted for in the overflow potential from RAB Roof C1 to the MSIV areas calculations (see Figure 3-4).

The RAB Roofs C2, C3, and C4 are located within the RAB Roof C1 extent and are at a higher elevation than the RAB Roof C1 (see Figure 3-4). Contributing inflows from RAB Roofs C2, C3, and C4 were directly translated to

RAB Roof C1 as no storage was credited for these parapet walls. The contributing areas of RAB Roofs C1, C2, C3, and C4 are included in Table 3-6.

RAB Roof C1 is at a lower elevation than the surrounding roofs except for RAB Roof C5 located south of RAB Roof C1. If water on the surrounding RAB Roofs A1, A2, B1, and B2 reach their respective height of the parapet wall, a portion of the overflow would discharge into RAB Roof C1 but the results show that runoff does not overflow RAB Roofs A1, A2, B1, and B2. Direct overflow from the reactor building walkway dome into the RAB Roof C1 was accounted for in the mass balance computations. Approximately 100 ft (21-percent of dome perimeter) of the volume of water overflowing the parapet wall will discharge into RAB Roof C1. The mass balance computations indicate that the two six-inch-diameter roof drains combined with the roof top storage is sufficient to prevent overflow from RAB Roof C1. Note that other available roof drains and scuppers in the RAB Roof C1 were not credited in the mass balance computations because they were not necessary. However, if these unaccounted drains were to be credited, the maximum ponding depth at RAB Roof C1 would be lower.

Reactor Building Roof

The circumference of the dome was calculated at 484 ft, 121 ft of which would directly contribute water to either MSIV East or MSIV West. Approximately 25-percent of the volume of water overflowing the parapet wall will contribute to ponding in the MSIV areas, therefore 12.5-percent would contribute to each MSIV area.

3.1.2.3.2 Calculate Storage Volumes

DCT Basins Storage Areas

Storage areas in which water would pond included the separate DCTs within each DCT Basin, the open areas adjacent to the DCTs in each DCT Basin, and the sub-basement of the FHB. These areas are shown in Figure 3-11. The obstructed areas due to the WCTs, internal walls, equipment foundations, etc. were calculated separately and subtracted from the total storage areas.

Areas were calculated using AutoCAD® based on the site survey (AREVA, 2014b) where available. If a storage area was not captured by the site survey (i.e. it is located under/within a roof area), drawing G-580, Sheet 3 "Nuclear Plant Island Structure – Flood Wall Penetrations" (WSES, 1991a) was used.

Areas 1 through 9 compose DCT Basin B and Areas 10 through 16 compose DCT Basin A. A summary of the calculated storage areas is presented in Table 3-7.

MSIV Areas Storage Areas

Storage areas within the MSIV areas include only the MSIV areas themselves. The storage area of MSIV East of 4,051 square ft and MSIV West of 4,140 square ft were taken from the existing design calculation, ECM13-001 (Entergy, 2014).

Equipment important to safety is located at least one foot off of the ground in the MSIV areas. Cables conduits are potentially located within one foot of the floor and are designed to be temporarily submerged in water without being adversely impacted (Entergy, 2014). These conduits do not significantly affect storage volumes in the MSIV. Therefore, volume reductions are not present in the storage calculations.

Subsequent to performance of this evaluation, the MSIV storage areas were revised to 4,088 square ft for each MSIV area.

Calculate RAB Roofs and Reactor Building Roof Storage Capacity

The maximum storage volume that the Reactor Bulding dome parapet wall can accommodate without overtopping is 4,063 cubic ft. The maximum storage volume the parapet walls can accommodate without overtopping RAB

Roof A1, RAB Roof A2, RAB Roof A3, RAB Roof B1, and RAB Roof B2 is respectively: 1,299, 387, 1,142, 885, and 1,089 cubic ft.

The maximum storage volume the parapet walls can accommodate without overtopping RAB Roof C1 is approximately 25,160 square ft x 1.42 ft (17 inches) = 35,727 cubic ft.

Calculate Storage Volume Reductions

No storage volume reductions were calculated for the MSIV areas. The following discussion covers the storage volume reductions for the DCT Basins and FHB.

The storage volumes for obstructions within each storage area were calculated and subtracted from the storage volume calculated above on a depth-varying basis (e.g., storage volume reductions are different depending on the ponding depth). Obstructions included, but were not limited to, conduit floor penetrations, columns, and equipment foundations. Fifty-percent of the volume occupied by the sump pumps was assumed to be available for ponding to account for the sump pumps and associated equipment (WSES, 2001b). Fifty-percent of the volume occupied by the shelving and storage locations was assumed to be available for ponding (WSES, 2001b). The volume reduction of the air accumulators was assumed to be equal to a rectangular shape of the length and width of the accumulators and a height equal to that of the depth of ponding (WSES, 2001b). The 10-inch-diameter and 12-inch-diameter pipes and the 20-inch-diameter CCW piping that run at elevation -33 ft, MSL within DCT Basins A and B and the FHB were assumed to be fully submerged. The previous calculation assumed these pipes were half submerged, bounded at 24-inches of ponding depth (WSES, 2001b) and therefore, the previous values were conservatively multiplied by 2 in this updated calculation. A reduction in the available storage volume would directly lead to a greater depth of ponding.

The adjusted storage volumes available at a particular ponding depth were calculated by subtracting the cumulative volume reductions from the unadjusted DCT volumes for DCT A and DCT B, respectively. A plot of the adjusted volumes versus the ponding depth was created and a linear regression equation was developed. The linear regression equation was used to calculate the overall ponding depth based on the calculated volume of water in each DCT Basin. The linear regression equations are as follows:

$$Depth\ of\ Ponding_{DCT\ Basin\ A} = 0.00011412 * Volume\ of\ Water_{in\ DCT\ Basin\ A} + 0.10501908 \quad \text{(Equation 2)}$$

$$Depth\ of\ Ponding_{DCT\ Basin\ B} = 0.00012632 * Volume\ of\ Water_{in\ DCT\ Basin\ B} + 0.08710623 \quad \text{(Equation 3)}$$

$$Depth\ of\ Ponding_{FHB} = 0.00021901 * Volume\ of\ Water_{in\ FHB} + 0.00462122 \quad \text{(Equation 4)}$$

3.1.2.3.3 Calculate Outflows

Calculate RAB Roofs and Reactor Building Roof Drain Capacity

The three 6-inch-diameter drains on the Reactor Building dome walkway, the 3-inch-diameter scuppers on RAB Roof A1, RAB Roof A2, RAB Roof A3, and RAB Roof B1, the 4-inch-diameter drain on RAB Roof B2, and the two six-inch-diameter drains on RAB Roof C1 were considered as outflows when calculating the potential overflow from the roofs. The roof drains and scuppers do not contribute to ponding in DCT Basins A or B, or the MSIV areas (e.g., runoff entering the roof drains and/or scuppers flows out of the NPIS system).

The capacity of the drains and scuppers is dependent, in part, on the head of water above the drain (e.g., the depth of water pooling on the Reactor Building dome walkway). The capacities were assumed to be 100-percent open because the areas of the grate covering the pipe openings are much larger than the pipe openings. In addition, WSES has operational plans to ensure that these pipes and grates are free from debris that may block flow through them (WSES 2014a, WSES 2015a, WSES 2015b).

The capacities of the drains and scuppers were calculated based on the general weir equation (Equation 5) and the orifice flow equation (Equation 6). The calculated flows using the two equations at each depth were compared and the lesser of the flows at each depth was used as the limiting flow.

The scuppers and roof drains will act as weirs until the water pooling along the Reactor Building dome walkway and the RAB roof tops reaches a transitional depth into orifice flow. The capacity of the weirs was calculated using the equation below:

$$Q = CLH^{3/2} \qquad \text{(Equation 5)}$$

Where:

Q= flow; cubic ft per second; cfs;
C= weir coefficient; 3.0 (USACE, 2010);
L= perimeter/circumference; ft;
H= height of ponding water; ft.

The capacity of the drains as orifices was calculated using the equation below:

$$Q = CA\sqrt{2gh} \qquad \text{(Equation 6)}$$

Where:

Q= flow; cfs;
C= orifice coefficient; 0.8 (USACE, 2010);
A= area of orifice; square ft;
g= acceleration due to gravity; 32.2 ft per square second;
h= height of ponding water relative to the orifice center-line; ft.

It is possible for water to overflow the 21-inch parapet wall along the Reactor Building dome roof, and/or the 12-inch parapet walls along the RAB Roof A1, RAB Roof A2, RAB Roof A3, RAB Roof B1, and RAB Roof B2, and/or the 17-inch parapet wall along the RAB Roof C1 boundary with the MSIV areas if the volume of water entering the system (i.e. the precipitation) is greater than the volume of water that can be removed from the system by the roof drains and stored within the parapet surrounded area.

The volume of water that can be stored along the perimeter of the dome was calculated by using the cross sectional area of the walkway as it changes with ponding depth and multiplying it by the circumference of the walkway at its center. The curve of the dome was considered to be negligible and the cross sectional area was essentially a trapezoid. The area of the trapezoid was calculated by adding the area of a rectangle (i.e., the walkway) to the area of a triangle (i.e., approximating the shape of the dome).

The volume of water that can be stored on the roof tops of RAB Roof A1, RAB Roof A2, RAB Roof A3, RAB Roof B1, RAB Roof B2, and RAB Roof C1 were calculated based on their respective roof top areas and the 12-inch high parapet walls (except for RAB Roof C1 that has 17-inch parapet walls).

The rate at which precipitation was falling was compared to the rate at which volume was exiting via the drains.

The calculated Reactor Building roof drain capacities in relation to ponding depths are presented in Table 3-8. The drain flow transitioned from weir flow to orifice flow at about 4 inches of depth and the maximum capacity of the three pipes with water depth just reaching the top of the parapet wall (assuming no obstruction) is 5.0 cfs (i.e., 1,500.8 cubic ft per five minutes). The maximum storage volume the parapet wall can accommodate without overtopping is 4,063 cubic ft.

The calculated 3-inch-diameter scuppers capacity in relation to ponding depths on RAB Roof A1, RAB Roof A2, RAB Roof A3, and RAB Roof B1 are presented in Table 3-9. The drain flow transitioned from weir flow to orifice flow at about 2 inches of depth and the maximum capacity of each scupper with water depth just reaching the top of the parapet wall assuming no obstruction) is 0.32 cfs (i.e., 94.5 cubic ft per five minutes).

The calculated 4-inch-diameter roof drain capacity in relation to ponding depths on RAB Roof B2 is presented in Table 3-10. The drain flow transitioned from weir flow to orifice flow at about 3 inches of depth and the maximum capacity of each scupper with water depth just reaching the top of the parapet wall (assuming no obstruction) is 0.56 cfs (i.e., 168.1 cubic ft per five minutes).

The calculated combined capacity of the two six-inch-diameter roof drains in relation to ponding depths on RAB Roof C1 is presented in Table 3-11. The drain flow transitioned from weir flow to orifice flow at about 4 inches of depth and the maximum capacity of the two pipes with water depth just reaching the top of the parapet wall (assuming no obstruction) is 3.0 cfs (i.e., 900 cubic ft per five minutes).

Calculation Pumping Discharge for DCT Basins

Each DCT Basin is hydraulically connected to one sump pump (Figure 3-4). These sump pumps were considered as an outflow from the NPIS. It was assumed that a loss of offsite power (LOOP) would occur during an extreme precipitation event (WSES, 2001b). As a result of a LOOP, the sump pump in the FHB sub-basement was assumed to be offline because it is not typically connected to the emergency diesel generators (WSES, 2001b). Following the LOOP, it was assumed that the sump pumps would not begin to operate until 30 minutes after the start of precipitation to allow for operator actions to restart the pumps (WSES, 2001b).

Each DCT sump pump is rated for 350 gallons per minute (gpm) (WSES, 2014b). The pump flow was conservatively reduced to 300 gpm, while aligned with an inoperable circulating water system, which was assumed to occur during a LOOP (WSES, 2002b). Therefore, a constant pumping rate of 300 gpm was used after the initial 30 minute setup time (WSES, 2001b).

A constant pumping rate of 300 gpm was used for each sump pump throughout the duration of the 6-hour duration LIP and in the hour following the end of precipitation. The pumping rate was converted to cubic ft per minute (cfm; 40.1 cfm) and then multiplied by 5 (200.5 cubic ft per 5 minutes) where necessary to match the time interval used in calculating the ponding depths. The following additional pump combinations / operational assumptions (i.e., sensitivity analysis) were calculated:

1. One installed sump pump for each DCT Basin starting 30 minutes after the onset of precipitation (300 gpm) – Base Case;
2. One sump pump for each DCT Basin starts immediately (i.e., at the first minute of precipitation).
3. Two pumps in each DCT basin starting after 30 minutes: One sump pump and one portable pump in each DCT Basin. The portable pumps are rated at the same capacity as the installed sump pumps. The installed sump pumps and portable pumps started after the initial 30 minute setup time.
4. Two pumps in each DCT basin starting at the onset of precipitation: One installed sump pump and one portable pump starting at the beginning of precipitation.
5. One installed sump pump starting after 30 minutes and one portable pump starting one hour after the beginning of precipitation in each DCT basin.
6. One installed sump pump starting after 30 minutes and one portable pump starting three hours after the beginning of precipitation in each DCT basin.
7. One installed sump pump starting after 30 minutes, one portable sump pump starting 1 hour after the beginning of precipitation, and one portable pump starting 3 hours after the beginning of precipitation (a total of three pumps), in each DCT basin.
8. One installed sump pump at the beginning of precipitation and one portable pump starting 30 minutes after the beginning of precipitation, in each DCT basin.

Calculate Flow into the Fuel Handling Building for DCT Basins

The FHB sub-basement is connected to each DCT Basin via four, 4-inch-diameter pipes, each with a flapper (i.e., check) valve below Doors D204 and D206 (Figure 3-5) (WSES, 1986). The flapper valves allow water to flow into but not out of the FHB sub-basement (WSES, 1986). Thus, the FHB acts as a storage reservoir. The volume of water from each DCT Basin that can be diverted into and stored within the FHB sub-basement was calculated based on the hydraulic capacity of the pipes and the head differential between the water in the DCT Basins and the FHB. Flow through the pipes for a given head differential was calculated using CulvertMaster®. CulvertMaster® is a program designed to aid in analyzing and designing culverts using the US Federal Highway Administrations' Hydraulic Design of Highway Culverts (HDS-5) methodologies. CulvertMaster® calculates the flow through the pipes based on the size, quantity, and roughness coefficient (AREVA, 2012). The four, 4-inch-diameter pipes below Doors D204 and D206 (Figure 3-5) are made of steel and were assigned a Manning's roughness coefficient of 0.013 (USDOT, 2011) with an invert elevation of -34.75 ft, MSL (WSES, 1986) in CulvertMaster®. The pipes were assigned a length of 3 ft (WSES, 1991c).

Note that the FHB sump pump was conservatively assumed to be offline as a result of LOOP because it is not connected to the emergency diesel generators (WSES, 2001b).

Calculate MSIV Area Drains Outflow

MSIV West has two 5-inch roof drains that drain outside the NPIS (Entergy, 2014). MSIV East has one 4-inch and one 5-inch scupper that drain outside the NPIS (Entergy, 2014). The calculated capacity of the two five-inch-diameter floor drains in relation to ponding depths on MSIV West area is presented in Table 3-12. The drain flow transitioned from weir flow to orifice flow at about 3 inches of depth and the maximum computed outflow of the two pipes combined relative to the computed maximum depth (assuming no obstruction) is 1.34 cfs (i.e., 80 cubic ft per five minutes). The maximum storage computed in the MSIV West is approximately 2,560 cubic ft.

The calculated four-inch-diameter and five-inch diameter scupper drains capacity in relation to ponding depths on MSIV East area is presented in Table 3-13. The drain flow transitioned from weir flow to orifice flow at about 3 inches of depth on both scupper drains. The maximum computed outflow of the two pipes combined relative to the computed maximum depth (assuming no obstruction) is 1.17 cfs (i.e., 70 cubic ft per five minutes). The maximum storage computed in the MSIV East is approximately 2,785 cubic ft.

Calculate Ponding Depths

A spreadsheet-based mass (volumetric) balance of the inflows, outflows, and storage volumes was used to calculate a maximum ponding depth in each DCT Basin and MSIV area as a result of the LIP. A variable 1-minute to 5-minute time step was used in the analysis. The spreadsheet-based mass balance is included in the LIP Calculation (AREVA, 2015a).

DCT Basins Ponding Depths

Maximum ponding depths within the DCT Basins due to the LIP for each pumping scenario are shown in Table 3-14. This calculation has a modeling period of 7-hours (i.e., one hour longer than the LIP). Although there is no precipitation during the last hour of the modeling period, the outflows from pumping are considered to continue. The total inflow from precipitation into DCT Basin A was calculated at 26,254 cubic ft. The total inflow from precipitation into DCT Basin B was calculated at 27,138 cubic ft. The total inflow from precipitation into the FHB was calculated at 1,026 cubic ft.

The calculations for the inflows and outflows can be found in Appendix C of the LIP Calculation (AREVA, 2015a).

The inflows, outflows, and capacity of the FHB pipes were combined with the storage volume reduction regression equations to produce resulting ponding depths. Figure 3-13 through Figure 3-20 are time series plots showing the depth of ponding in DCT Basin A, DCT Basin B, and the FHB.

MSIV Areas Ponding Depths

The inflows and outflows were combined with the linear storage volume relation to produce resulting ponding depths.

The equations calculated for each MSIV area are as follows:

$$Volume\ of\ Water_{MSIV\ East} = 4{,}051\ ft^2 * Depth\ of\ Ponding_{MSIV\ East} \quad \text{(Equation 7)}$$

$$Volume\ of\ Water_{MSIV\ West} = 4{,}140\ ft^2 * Depth\ of\ Ponding_{MSIV\ West} \quad \text{(Equation 8)}$$

Resultant maximum flood depths of 0.69 ft and 0.62 ft within the MSIV East and West areas, respectively, were calculated. Ponding depths within the MSIV areas due to the LIP are shown in Table 3-15. Figure 3-21 and Figure 3-22 are time series plots showing the depth of ponding MSIV East and West, respectively. The calculations for the inflows and outflows can be found in Appendix I of the LIP Calculation (AREVA, 2015a).

The updated MSIV storage areas provided after performance of this evaluation do not result in significant changes to the results of the initial MSIV ponding levels reported above.

3.1.3 Conclusions

The results of the LIP analysis at WSES indicate that the computed flood depths are well below the NPIS flood protection level of 12 to 15 ft above the general site grade in the vicinity of the NPIS and, therefore, no external LIP flood impacts are anticipated at WSES.

Maximum computed flood depths within the DCT Basins due to LIP ponding vary depending on the number and start time of pumps available. Maximum flood depths in DCT Basin A range from 1.18 ft to 1.53 ft for the conditions analyzed. Maximum flood depths in DCT Basin B range from 1.27 ft to 1.63 ft for the conditions analyzed. Maximum flood depths in MSIV East and MSIV West are 0.69 ft and 0.62 ft, respectively.

3.1.4 References

AREVA, 2012. AREVA Document No. 38-9192493-000, "Computer Software Certification – Bentley CulvertMaster® v.3.3", GZA GeoEnvironmental, Inc., October 12, 2012.

AREVA, 2014a. AREVA Document No. 32-9226993-000, "Waterford Steam Electric Station Flooding Hazard Re-evaluation – Local Intense Precipitation," GZA GeoEnvironmental, Inc., 2014.

AREVA, 2014b. Waterford Nuclear Generating Station – WSES: Aerial Mapping Validation Report, prepared by McKim & Creed, July 2014. See AREVA Document No. 38-9226991-000.

AREVA, 2014c. AREVA Document No. 38-9225054-000, "Computer Software Certification – FLO-2D® Pro Model, Build No. 14.03.07", GZA GeoEnvironmental, Inc., June 16, 2014.

AREVA, 2015a. AREVA Document No. 32-9231496-000, "Waterford Steam Electric Station Flooding Hazard Re-Evaluation – Nuclear Power Island Structure Local Intense Precipitation," GZA GeoEnvironmental, Inc., 2015.

AREVA, 2015b. AREVA Document No. 32-9233937-000, "Waterford Steam Electric Station Flooding Hazard Re-Evaluation – Probable Maximum Precipitation," GZA GeoEnvironmental, Inc., 2015.

AWA, 2008. Applied Weather Associates, Statewide Probable Maximum Precipitation (PMP) Study for the state of Nebraska, Prepared for Nebraska Dam Safety, Lincoln, NE, 2008

AWA, 2012. Applied Weather Associates, Site-Specific Probable Maximum Precipitation (PMP) Study for the Tarrant Regional Water District-Benbrook and Floodway Basins, Ft Worth, TX, 2012.

AWA, 2013. Applied Weather Associates, Statewide Probable Maximum Precipitation (PMP) Study for the State of Ohio, Ohio Dam Safety, Columbus, OH, 2013.

AWA, 2014. Site-Specific Probable Maximum Precipitation Study for the Duck River Basin, Alabama, prepared for Utilities Board of the City of Cullman, AL, 2014.

Entergy, 2014. "MSIV Area Flooding Analysis," Calculation ECM13-001, Revision 0, Entergy, 2014, See AREVA Document No. 38-9243507-000.

FLO-2D, 2014. FLO-2D® Pro Reference Manual, FLO-2D Software, Inc., Nutrioso, Arizona (www.flo-2d.com).

NOAA, 1956. Seasonal Variation of the Probable Maximum Precipitation east of the 105th Meridian for areas from 10 to 1000 square miles and durations of 6, 12, 24, and 48 hours. Hydrometeorological Report No. 33 by US Department of Commerce, 1956.

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

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

NOAA, 1982. Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No.52 (HMR-52) by US Department of Commerce & USACE, August 1982.

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, Springfield, VA, National Technical Information Service, 2011.

Parzybok, et al., 2006. Parzybok T.W. and E.M. Tomlinson 2006: A New System for Analyzing Precipitation from Storms, Hydro Review, Vol. XXV, No. 3, 58-65, 2006.

USACE, 1973. US Army Corps of engineers (USACE), US Army 1936-1973, Storm Rainfall in the United States, Depth-Area-Duration Data, Office of Chief of Engineers, Washington, D.C.

USACE, 2010. U.S. Army Corps of Engineers Hydrologic Engineering Center, River Analysis System HEC-RAS, Hydraulic Reference Manual, Version 4.1, January 2010.

USDOT, 2011. U.S. Department of Transportation Federal Highway Administration "Introduction to Highway Hydraulics – Appendix B." http://www.fhwa.dot.gov/engineering/hydraulics/pubs/08090/appb.cfm. Updated 4/7/2011. Accessed 11/24/2014.

WMO, 2009. World Meteorological Organization (WMO), Manual for Estimation of Probable Maximum Precipitation, Operational Hydrology Report No 1045, WMO, Geneva, 2009.

WSES, 1986. WSES Drawing LOU-861-HV-100, Revision 0, "Flapper Assembly Detail at FHB Doors D-204 & D-206 for Maintaining Neg. Press.," April 1986. See AREVA Document No. 38-9243507-000.

WSES, 1991a. WSES Drawing G-580 Sheet 3, Revision 2, "Nuclear Plant Island Structure Flood Wall Penetrations – Sh. 3 Plan," April 1991 See AREVA Document No. 38-9243507-000.

WSES, 1991b. WSES Drawing LOU-1564 G-589, Revision 8, "Fuel Handling Building Floor at El. +46.00 & Roof – Masonry," July 1991 See AREVA Document No. 38-9243507-000.

WSES, 1991c. WSES Drawing LOU-1564 G-593 Sheet 1, Revision 13, "Fuel Handling Building Exterior & Interior Walls Masonry Sh. 1," July 1991. See AREVA Document No. 38-9243507-000.

WSES, 1991d. WSES Drawing LOU-1564 G-560 Sheet 1, Revision 9, "Reactor Auxiliary Building Roof Slab at EL. +69.00 Masonry Sh. 1," April 1991. See AREVA Document No. 38-9243507-000.

WSES, 1992. WSES Drawing LOU-1564 G-874 Sheet 1, Revision 11, "Fuel Handling Building Plan El. -34.75 – Sh. 1 Plumbing & Drainage," September 1992. See AREVA Document No. 38-9243507-000.

WSES, 1993. WSES Drawing LOU-1564 G-889, Revision 8, "Roof Plan Plumbing & Draining," August 1993. See AREVA Document No. 38-9243507-000.

WSES, 1997. WSES Drawing LOU-1564 G-880 Sheet 2, Revision 22, "Riser Diagrams & Details – Plumbing & Drainage," October 31, 1997. See AREVA Document No. 38-9243507-000.

WSES, 2001a. WSES FSAR Figure 2.4-8, Revision 11, "Roof Drainage," May 2001. See AREVA Document No. 38-9243507-000.

WSES, 2001b. WSES Calculation EC-M99-010, Revision 0-2, August 2001. See AREVA Document No. 38-9243507-000.

WSES, 2001c. WSES Drawing G-210, Revision 20, "General Arrangement Cooling Towers Plan," April, 2001. See AREVA Document No. 38-9243507-000.

WSES, 2002a. WSES Drawing G-148, Revision 23, "General Arrangement Reactor Building Sections," September 2002. See AREVA Document No. 38-9243507-000.

WSES, 2002b. WSES Calculation EC-M97-029, Revision 0, "Dry Cooling Tower Area Drain Sump Pump Minimum Capacity," May 2002. See AREVA Document No. 38-9243507-000.

WSES, 2011a. WSES Drawing G-580 Sheet 1, Revision 3, "Nuclear Plant Island Structure Flood Wall Penetrations – Sh. 1 Plan," April 2011 See AREVA Document No. 38-9243507-000.

WSES, 2011b. WSES Drawing G-580 Sheet 2, Revision 3, "Nuclear Plant Island Structure Flood Wall Penetrations – Sh. 2 Plan," April 2011. See AREVA Document No. 38-9243507-000.

WSES, 2012. WSES Drawing G493 Sheet 10, Revision 0, "Vehicle Barrier Sections & Details" February 2012 (Appendix H). See AREVA Document No. 38-9243507-000.

WSES, 2013. WSES Updated Final Safety Analysis Report, 2013. See AREVA Document No. 38-9243507-000.

WSES, 2014a. WSES Procedure Number OP-901-521 (Attachment 4) – "Severe Weather and Flooding," Revision 315, 2015. See AREVA Document No. 38-9243507-000.

WSES, 2014b. WSES Specification Document LOU-1564.089A, Revision 6, "Sump Pumps and Accessories – Nonnuclear," February 2014. See AREVA Document No. 38-9243507-000.

WSES, 2015a. WSES CR Number 2015-2125 – "Potential Flooding in Dry Cooling Tower (DCT) Areas based on Fukushima Flood Hazard Reevaluation (FHE) - Local Intense Precipitation Calculation," Revision 0, April 14, 2015. See AREVA Document No. 38-9243507-000.

WSES, 2015b. WSES CR Number 2015-01282, April 28, 2015. See AREVA Document No. 38-9243507-000.

Table 3-1: LIP Model Results

Location	Representative Grid Element Number	Grid Element Elevation (ft, NAVD88)	LIP Peak Water Surface Elevation – With VBS (ft, NAVD88)	Maximum Flow Depth (ft)	Maximum Flow Velocity (fps)
Northwest corner of NPIS	16,886	15.7	16.6	1.0	0.4
Northeast side of NPIS	17,833	15.8	16.6	0.8	0.7
Southeast side of NPIS	30,414	15.7	16.2	0.5	1.0
Between Tool Room and RW Solid Bldg on Southwest side of NPIS	24,720	18.2	19.1	0.9	2.2
Between West Side Access and Tool Room on West side of NIPS	22,524	18.5	19.0	0.5	1.7
Northwest side of NPIS	16,878	15.6	16.6	1.0	1.0
Southeast side of ISFSI	44,883	14.7	15.9	1.1	0.2

Table 3-2: Storms Used to Calculate the Site-Specific LIP Values

Storm Name	State	Lat	Lon	Year	Month	Day	Max Rainfall	Max 6-hour $10mi^2$ Rainfall	Maximum 1-hour $1mi^2$ Rainfall Using HMR 52 Ratio or SPAS Data	Maximum 6-hour $1mi^2$ Rainfall Using HMR 52 Ratio or SPAS Data	Waterford Total Adjustment Factor	Waterford 1-hour $1mi^2$ PMP	Waterford 6-hour $1mi^2$ PMP	Precipitation Source
ST. GEORGE	GA	30.521	-82.038	1911	8	28	19.10	14.90	8.91	15.20	1.22	10.87	18.54	SA 3-11
THRALL	TX	30.591	-97.297	1921	9	9	39.70	22.40	13.40	22.85	1.18	15.81	26.96	GM 4-12
NEOSHO FALLS	KS	38.082	-95.701	1926	9	12	14.00	13.40	8.01	13.67	1.45	11.62	19.82	SW 2-1
JEFFERSON PLAQ	LA	29.855	-89.991	1927	4	12	20.40	13.80	8.25	14.08	1.10	9.08	15.48	LMV 4-8
ELBA	AL	31.417	-86.067	1929	3	12	29.60	10.39	2.50	10.39	1.25	3.13	12.99	SPAS 1305
SIMMESPORT	LA	30.983	-91.800	1935	5	16	14.10	13.80	8.25	14.08	1.21	9.99	17.03	LMV 4-21
BEBE	TX	29.332	-97.682	1936	6	30	21.00	14.00	8.37	14.28	1.20	10.05	17.14	GM 5-6
ENGLE	TX	29.681	-97.009	1940	6	29	22.70	11.00	6.58	11.22	1.29	8.49	14.47	GM 5-11
HALLETT	OK	36.250	-96.610	1940	9	2	24.00	18.29	4.05	18.42	1.21	4.90	22.29	SPAS 1429
TRENTON	FL	29.610	-82.820	1941	10	17	35.00	12.90	7.71	13.16	1.06	8.18	13.95	SA 5-6
MOUNDS	OK	35.877	-96.061	1943	5	16	17.00	16.18	8.21	16.23	1.39	11.41	22.56	SPAS 1320
SILVER LAKE	TX	32.670	-95.596	1943	6	5	16.50	14.20	8.49	14.48	1.19	10.11	17.24	SW 3-3
HOLT	MO	39.453	-94.342	1947	6	18	17.60	12.04	11.93	12.94	1.24	14.79	16.05	SPAS 1434
YANKEETOWN	FL	29.030	-82.720	1950	9	3	45.20	16.00	9.57	16.32	1.10	10.52	17.95	SA 5-8
KELSO	MO	37.191	-89.550	1952	8	11	13.00	13.00	7.77	13.26	1.37	10.65	18.17	UMV 3-30
HARRISONBURG DAM	LA	31.767	-91.817	1953	5	11	25.40	9.20	4.41	9.43	1.15	5.07	10.84	SPAS 1435
GLADEWATER	TX	32.537	-94.943	1966	4	27	25.33	8.82	2.92	8.97	1.31	3.83	11.75	SPAS 1181
ALVIN	TX	29.429	-95.271	1979	7	23	45.49	19.88	6.45	20.47	1.06	6.84	21.70	SPAS 1463
HOUSTON	TX	29.755	-95.275	2001	6	4	40.97	15.00	5.67	15.20	1.09	6.18	16.57	SPAS 1464
NEW ORLEANS	LA	30.290	-89.900	2005	8	28	15.62	9.88	3.77	9.88	1.05	3.96	10.37	SPAS 1229
LARTO LAKE	LA	31.220	-92.130	2008	9	1	23.31	10.91	6.15	11.08	1.24	7.63	13.74	SPAS 1182
WARNER PARK	TN	36.061	-86.906	2010	4	30	19.71	14.98	4.58	15.06	1.16	5.31	17.47	SPAS 1208

Table 3-3: Point (1-mi^2) Local Intense Precipitation Depths at WSES

Time (min)	Waterford PMP Depth (in) at 1-hour 1-square mile
60	15.8
30	11.5
15	7.9
5	5.0
	Waterford PMP Depth (in) at 6-hour 1-square mile
	27.0

Table 3-4: Site-Specific PMP Rainfall Distribution

Time Step Values [1]	Incremental PMP Depths [2]	Cumulative PMP Depths
(hours)	(inches)	(inches)
0.000	0.0000	0.000
0.017	1.0000	1.000
0.033	1.0000	2.000
0.050	1.0000	3.000
0.067	1.0000	4.000
0.083	1.0000	5.000
0.100	0.2900	5.290
0.117	0.2900	5.580
0.133	0.2900	5.870
0.150	0.2900	6.160
0.167	0.2900	6.450
0.183	0.2900	6.740
0.200	0.2900	7.030
0.217	0.2900	7.320
0.233	0.2900	7.610
0.250	0.2900	7.900
0.267	0.2400	8.140
0.283	0.2400	8.380
0.300	0.2400	8.620
0.317	0.2400	8.860
0.333	0.2400	9.100
0.350	0.2400	9.340
0.367	0.2400	9.580
0.383	0.2400	9.820
0.400	0.2400	10.060
0.417	0.2400	10.300
0.433	0.2400	10.540
0.450	0.2400	10.780
0.467	0.2400	11.020
0.483	0.2400	11.260
0.500	0.2400	11.500
0.517	0.1433	11.643
0.533	0.1433	11.787
0.550	0.1433	11.930
0.567	0.1433	12.073
0.583	0.1433	12.217
0.600	0.1433	12.360
0.617	0.1433	12.503
0.633	0.1433	12.647
0.650	0.1433	12.790

Time Step Values [1] (hours)	Incremental PMP Depths [2] (inches)	Cumulative PMP Depths (inches)
0.667	0.1433	12.933
0.683	0.1433	13.077
0.700	0.1433	13.220
0.717	0.1433	13.363
0.733	0.1433	13.507
0.750	0.1433	13.650
0.767	0.1433	13.793
0.783	0.1433	13.937
0.800	0.1433	14.080
0.817	0.1433	14.223
0.833	0.1433	14.367
0.850	0.1433	14.510
0.867	0.1433	14.653
0.883	0.1433	14.797
0.900	0.1433	14.940
0.917	0.1433	15.083
0.933	0.1433	15.227
0.950	0.1433	15.370
0.967	0.1433	15.513
0.983	0.1433	15.657
1.000	0.1433	15.800
1.083	0.1867	15.987
1.167	0.1867	16.173
1.250	0.1867	16.360
1.333	0.1867	16.547
1.417	0.1867	16.733
1.500	0.1867	16.920
1.583	0.1867	17.107
1.667	0.1867	17.293
1.750	0.1867	17.480
1.833	0.1867	17.667
1.917	0.1867	17.853
2.000	0.1867	18.040
2.083	0.1867	18.227
2.167	0.1867	18.413
2.250	0.1867	18.600
2.333	0.1867	18.787
2.417	0.1867	18.973
2.500	0.1867	19.160
2.583	0.1867	19.347
2.667	0.1867	19.533
2.750	0.1867	19.720

Time Step Values [1]	Incremental PMP Depths [2]	Cumulative PMP Depths
(hours)	(inches)	(inches)
2.833	0.1867	19.907
2.917	0.1867	20.093
3.000	0.1867	20.280
3.083	0.1867	20.467
3.167	0.1867	20.653
3.250	0.1867	20.840
3.333	0.1867	21.027
3.417	0.1867	21.213
3.500	0.1867	21.400
3.583	0.1867	21.587
3.667	0.1867	21.773
3.750	0.1867	21.960
3.833	0.1867	22.147
3.917	0.1867	22.333
4.000	0.1867	22.520
4.083	0.1867	22.707
4.167	0.1867	22.893
4.250	0.1867	23.080
4.333	0.1867	23.267
4.417	0.1867	23.453
4.500	0.1867	23.640
4.583	0.1867	23.827
4.667	0.1867	24.013
4.750	0.1867	24.200
4.833	0.1867	24.387
4.917	0.1867	24.573
5.000	0.1867	24.760
5.083	0.1867	24.947
5.167	0.1867	25.133
5.250	0.1867	25.320
5.333	0.1867	25.507
5.417	0.1867	25.693
5.500	0.1867	25.880
5.583	0.1867	26.067
5.667	0.1867	26.253
5.750	0.1867	26.440
5.833	0.1867	26.627
5.917	0.1867	26.813
6.000	0.1867	27.000
6.083	0.0000	27.000
6.167	0.0000	27.000
6.250	0.0000	27.000

Time Step Values [1]	Incremental PMP Depths [2]	Cumulative PMP Depths
(hours)	(inches)	(inches)
6.333	0.0000	27.000
6.417	0.0000	27.000
6.500	0.0000	27.000
6.583	0.0000	27.000
6.667	0.0000	27.000
6.750	0.0000	27.000
6.833	0.0000	27.000
6.917	0.0000	27.000
7.000	0.0000	27.000

Notes:
1. Time steps are in 1 minute increments for durations less than 1 hour. For durations greater than 1 hour, the time step is 5 minutes.
2. PMP depths are from WSES Calculation 32-9233937-000 (AREVA, 2015).

Table 3-5: DCT Basins Summary of Contributing Areas

Contributing Area	(sq. ft.)	Areas from EC-M99-010	Percent Difference
DCT Basin A	11,197		
Exterior walls	312		
DCT Basin A Total [1]	**11,041**	11,122	1%
DCT Basin B	11,720		
Exterior walls	573		
DCT Basin B Total [1]	**11,434**	11,359	-1%
Reactor Building Dome	**18,627**	18,627	0%
RAB Roof B1	**885**	882	0%
RAB Roof B2	**1,089**	1,100	1%
RAB Roof A1	**1,299**	1,248	-4%
RAB Roof A2	**387**	360	-7%
RAB Roof A3	**1,142**	1,170	2%
1	78		
2	78		
3	78		
4	74		
5	74		
6	74		
FHB Air/Exhaust [1,2]	**456**	273	-40%

Notes:
1. Areas are considered directly contributing to ponding.
2. FHB Air/Exhaust areas based of scaled drawing only (WSES, 2011a) since they were not captured by the site survey (AREVA, 2014). Areas includes 50-percent of adjacent exterior walls.
3. Areas were calculated using AutoCAD®.
4. Areas were drawn in AutoCAD® based on the site survey, unless otherwise noted.

Table 3-6: MSIV Areas Summary of Contributing Areas

Contributing Areas	
	Square Ft
MSIV East[1]	4,051
MSIV West[1]	4,140
Reactor Building Dome [2]	18,627
RAB Roof C1 [3]	25,160
RAB Roof C2 [3]	195
RAB Roof C3 [3]	445
RAB Roof C4 [3]	475

Notes:

1. MSIV areas taken from Calculation ECM13-001 (Entergy, 2014).
2. Reactor Building Dome area calculated as described in the DCT Basin and FHB Flooding Calculations. (WSES, 2013)
3. Areas were calculated using AutoCAD® and were based on the site survey data (AREVA, 2014).

Table 3-7: DCT Basins Storage Areas Summary

Storage Area	sq. ft.	Area from WSES, 2001b	Percent Difference
1	409	411	0%
2	527	546	-3%
3	431	434	-1%
4	423	518	-18%
5	427	434	-2%
6	517	532	-3%
7	4,181	4,274	-2%
8	360	353	2%
9	764	765	0%
DCT B	**8,041**	**8,267**	-3%
10	392	397	-1%
11	563	560	0%
12	440	434	1%
13	417	518	-19%
14	432	434	0%
15	522	518	1%
16	6,071	5,952	2%
DCT A	**8,838**	**8,813**	0%
FHB	**5,867**	6,595	-11%

Notes:
1. Areas 1-6, 8 and 11-15 are based off of a combination of the site survey (AREVA, 2014) data and the scaled drawing (WSES, 1991a). Remaining areas, including the FHB, are based off of the scaled drawing (WSES, 1991a) since they are not captured by the site survey.
2. Area 16 in this calculation is a combination of Areas 16, 17, and 18 from Calculation EC-M99-010 (WSES, 2001b).
3. Difference in FHB areas is due to accounting for some interior walls that Calculation EC-M99-010 (WSES, 2001b) takes into consideration in a later part of the calculation.

Table 3-8: Reactor Building Roof Drain Capacity (Three 6-Inch Roof Drains)

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0
0.04	0.12	0.77	0.12	36.1
0.08	0.34	1.09	0.34	102.0
0.13	0.62	1.34	0.62	187.4
0.17	0.96	1.54	0.96	288.6
0.21	1.34	1.73	1.34	403.3
0.25	1.77	1.89	1.77	530.1
0.29	2.23	2.04	2.04	612.7
0.33	2.72	2.18	2.18	655.0
0.38	3.25	2.32	2.32	694.7
0.42	3.80	2.44	2.44	732.3
0.46	4.39	2.56	2.56	768.1
0.50	5.00	2.67	2.67	802.2
0.54	5.64	2.78	2.78	835.0
0.58	6.30	2.89	2.89	866.5
0.63	6.99	2.99	2.99	896.9
0.67	7.70	3.09	3.09	926.3
0.71	8.43	3.18	3.18	954.8
0.75	9.18	3.28	3.28	982.5
0.79	9.96	3.36	3.36	1009.4
0.83	10.75	3.45	3.45	1035.7
0.88	11.57	3.54	3.54	1061.2
0.92	12.41	3.62	3.62	1086.2
0.96	13.26	3.70	3.70	1110.6
1.00	14.14	3.78	3.78	1134.5
1.04	15.03	3.86	3.86	1157.9
1.08	15.94	3.94	3.94	1180.8
1.13	16.87	4.01	4.01	1203.3
1.17	17.81	4.08	4.08	1225.4
1.21	18.78	4.16	4.16	1247.1
1.25	19.76	4.23	4.23	1268.4
1.29	20.75	4.30	4.30	1289.4
1.33	21.77	4.37	4.37	1310.0
1.38	22.79	4.43	4.43	1330.3
1.42	23.84	4.50	4.50	1350.3

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
1.46	24.90	4.57	4.57	1370.0
1.50	25.97	4.63	4.63	1389.5
1.54	27.06	4.70	4.70	1408.6
1.58	28.17	4.76	4.76	1427.6
1.63	29.28	4.82	4.82	1446.2
1.67	30.42	4.88	4.88	1464.6
1.71	31.57	4.94	4.94	1482.8
1.75	32.73	5.00	5.00	1500.8
Note:				
1. "Resulting Value" is the lesser or the weir flow and the orifice flow.				

Table 3-9: One 3-Inch-Diameter Scupper Capacity

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0
0.04	0.02	0.06	0.02	6.0
0.08	0.06	0.09	0.06	17.0
0.13	0.10	0.11	0.10	31.2
0.17	0.16	0.13	0.13	38.6
0.21	0.22	0.14	0.14	43.2
0.25	0.29	0.16	0.16	47.3
0.29	0.37	0.17	0.17	51.1
0.33	0.45	0.18	0.18	54.6
0.38	0.54	0.19	0.19	57.9
0.42	0.63	0.20	0.20	61.0
0.46	0.73	0.21	0.21	64.0
0.50	0.83	0.22	0.22	66.9
0.54	0.94	0.23	0.23	69.6
0.58	1.05	0.24	0.24	72.2
0.63	1.16	0.25	0.25	74.7
0.67	1.28	0.26	0.26	77.2
0.71	1.40	0.27	0.27	79.6
0.75	1.53	0.27	0.27	81.9
0.79	1.66	0.28	0.28	84.1
0.83	1.79	0.29	0.29	86.3
0.88	1.93	0.29	0.29	88.4
0.92	2.07	0.30	0.30	90.5
0.96	2.21	0.31	0.31	92.6
1.00	2.36	0.32	0.32	94.5

Note:
1. "Resulting Value" is the lesser or the weir flow and the orifice flow.

Table 3-10: One 4-Inch-Diameter Roof Drain Capacity

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0
0.04	0.03	0.11	0.03	8.0
0.08	0.08	0.16	0.08	22.7
0.13	0.14	0.20	0.14	41.7
0.17	0.21	0.23	0.21	64.1
0.21	0.30	0.26	0.26	76.7
0.25	0.39	0.28	0.28	84.0
0.29	0.49	0.30	0.30	90.8
0.33	0.60	0.32	0.32	97.0
0.38	0.72	0.34	0.34	102.9
0.42	0.84	0.36	0.36	108.5
0.46	0.97	0.38	0.38	113.8
0.50	1.11	0.40	0.40	118.8
0.54	1.25	0.41	0.41	123.7
0.58	1.40	0.43	0.43	128.4
0.63	1.55	0.44	0.44	132.9
0.67	1.71	0.46	0.46	137.2
0.71	1.87	0.47	0.47	141.5
0.75	2.04	0.49	0.49	145.6
0.79	2.21	0.50	0.50	149.5
0.83	2.39	0.51	0.51	153.4
0.88	2.57	0.52	0.52	157.2
0.92	2.76	0.54	0.54	160.9
0.96	2.95	0.55	0.55	164.5
1.00	3.14	0.56	0.56	168.1

Note:
1. "Resulting Value" is the lesser or the weir flow and the orifice flow.

Table 3-11: Two 6-Inch-Diameter Roof Drains Capacity

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0
0.04	0.08	0.51	0.08	24.0
0.08	0.23	0.73	0.23	68.0
0.13	0.42	0.89	0.42	125.0
0.17	0.64	1.03	0.64	192.4
0.21	0.90	1.15	0.90	268.9
0.25	1.18	1.26	1.18	353.4
0.29	1.48	1.36	1.36	408.5
0.33	1.81	1.46	1.46	436.7
0.38	2.16	1.54	1.54	463.2
0.42	2.53	1.63	1.63	488.2
0.46	2.92	1.71	1.71	512.0
0.50	3.33	1.78	1.78	534.8
0.54	3.76	1.86	1.86	556.6
0.58	4.20	1.93	1.93	577.7
0.63	4.66	1.99	1.99	597.9
0.67	5.13	2.06	2.06	617.5
0.71	5.62	2.12	2.12	636.6
0.75	6.12	2.18	2.18	655.0
0.79	6.64	2.24	2.24	673.0
0.83	7.17	2.30	2.30	690.4
0.88	7.71	2.36	2.36	707.5
0.92	8.27	2.41	2.41	724.1
0.96	8.84	2.47	2.47	740.4
1.00	9.42	2.52	2.52	756.3
1.04	10.02	2.57	2.57	771.9
1.08	10.63	2.62	2.62	787.2
1.13	11.25	2.67	2.67	802.2
1.17	11.88	2.72	2.72	816.9
1.21	12.52	2.77	2.77	831.4
1.25	13.17	2.82	2.82	845.6
1.29	13.84	2.87	2.87	859.6
1.33	14.51	2.91	2.91	873.3
1.38	15.20	2.96	2.96	886.9
1.42	15.89	3.00	3.00	900.2
Note:				
1. "Resulting Value" is the lesser or the weir flow and the orifice flow.				

Table 3-12: MSIV West Drains Capacity (Two 5-Inch-Diameter Floor Drains)

Comparison and Resulting Flow				
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0
0.04	0.07	0.36	0.07	20.0
0.08	0.19	0.51	0.19	56.7
0.13	0.35	0.62	0.35	104.1
0.17	0.53	0.71	0.53	160.3
0.21	0.75	0.80	0.75	224.1
0.25	0.98	0.88	0.88	262.6
0.29	1.24	0.95	0.95	283.7
0.33	1.51	1.01	1.01	303.2
0.38	1.80	1.07	1.07	321.6
0.42	2.11	1.13	1.13	339.0
0.46	2.44	1.19	1.19	355.6
0.50	2.78	1.24	1.24	371.4
0.54	3.13	1.29	1.29	386.6
0.58	3.50	1.34	1.34	401.2
0.63	3.88	1.38	1.38	415.2
0.67	4.28	1.43	1.43	428.9
0.71	4.68	1.47	1.47	442.0
0.75	5.10	1.52	1.52	454.9
0.79	5.53	1.56	1.56	467.3
0.83	5.97	1.60	1.60	479.5
0.88	6.43	1.64	1.64	491.3
0.92	6.89	1.68	1.68	502.9
0.96	7.37	1.71	1.71	514.2
1.00	7.85	1.75	1.75	525.2
1.04	8.35	1.79	1.79	536.1
1.08	8.86	1.82	1.82	546.7
1.13	9.37	1.86	1.86	557.1
1.17	9.90	1.89	1.89	567.3
1.21	10.43	1.92	1.92	577.4

Notes:

1. "Resulting Value" is the lesser or the weir flow and the orifice flow.

Table 3-13: MSIV East Drain Capacity (one 4-Inch-and one 5-Inch Diameter Scupper)

Comparison and Resulting Flow									
	4" Scupper				5" Scupper				4" Scupper + 5" Scupper Resulting Value
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]	
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)	(cubic ft per 5 minutes)
0.00	0.00	0.00	0.00	0.0	0.00	0.00	0.00	0.0	0.0
0.04	0.03	0.11	0.03	8.0	0.03	0.18	0.03	10.0	18.0
0.08	0.08	0.16	0.08	22.7	0.09	0.25	0.09	28.3	51.0
0.13	0.14	0.20	0.14	41.7	0.17	0.31	0.17	52.1	93.7
0.17	0.21	0.23	0.21	64.1	0.27	0.36	0.27	80.2	144.3
0.21	0.30	0.26	0.26	76.7	0.37	0.40	0.37	112.0	188.7
0.25	0.39	0.28	0.28	84.0	0.49	0.44	0.44	131.3	215.3
0.29	0.49	0.30	0.30	90.8	0.62	0.47	0.47	141.8	232.6
0.33	0.60	0.32	0.32	97.0	0.76	0.51	0.51	151.6	248.7
0.38	0.72	0.34	0.34	102.9	0.90	0.54	0.54	160.8	263.7
0.42	0.84	0.36	0.36	108.5	1.06	0.57	0.57	169.5	278.0
0.46	0.97	0.38	0.38	113.8	1.22	0.59	0.59	177.8	291.6
0.50	1.11	0.40	0.40	118.8	1.39	0.62	0.62	185.7	304.5
0.54	1.25	0.41	0.41	123.7	1.57	0.64	0.64	193.3	317.0
0.58	1.40	0.43	0.43	128.4	1.75	0.67	0.67	200.6	328.9
0.63	1.55	0.44	0.44	132.9	1.94	0.69	0.69	207.6	340.5
0.67	1.71	0.46	0.46	137.2	2.14	0.71	0.71	214.4	351.7
0.71	1.87	0.47	0.47	141.5	2.34	0.74	0.74	221.0	362.5
0.75	2.04	0.49	0.49	145.6	2.55	0.76	0.76	227.4	373.0
0.79	2.21	0.50	0.50	149.5	2.77	0.78	0.78	233.7	383.2

Comparison and Resulting Flow										
	4" Scupper				5" Scupper					4" Scupper + 5" Scupper Resulting Value
Depth	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]	Weir Flow	Orifice Flow	Resulting Value [1]	Resulting Value [1]		
(ft)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)	(cfs)	(cfs)	(cfs)	(cubic ft per 5 minutes)		(cubic ft per 5 minutes)
0.83	2.39	0.51	0.51	153.4	2.99	0.80	0.80	239.7		393.2
0.88	2.57	0.52	0.52	157.2	3.21	0.82	0.82	245.7		402.9
0.92	2.76	0.54	0.54	160.9	3.45	0.84	0.84	251.4		412.4
0.96	2.95	0.55	0.55	164.5	3.68	0.86	0.86	257.1		421.6
1.00	3.14	0.56	0.56	168.1	3.93	0.88	0.88	262.6		430.7
1.04	3.34	0.57	0.57	171.5	4.17	0.89	0.89	268.0		439.6
1.08	3.54	0.58	0.58	174.9	4.43	0.91	0.91	273.3		448.3
1.13	3.75	0.59	0.59	178.3	4.69	0.93	0.93	278.5		456.8
1.17	3.96	0.61	0.61	181.5	4.95	0.95	0.95	283.7		465.2
1.21	4.17	0.62	0.62	184.8	5.22	0.96	0.96	288.7		473.4

Note: 1. "Resulting Value" is the lesser or the weir flow and the orifice flow.

Table 3-14: DCT Basins Maximum Ponding Depths Summary (ft)

	1 Pump (1 x 300 gpm)		2 Pumps (2 x 300 gpm)		1 Pump Starting after 30 minutes and 1 Pump Starting after 1 hour	1 Pump Starting after 30 minutes and 1 Pump Starting after 3 hours	1 Pump Starting after 30 minutes, 1 Pump Starting after 1 hour and 1 Pump Starting after 3 hours	1 Pump Starting after 0 minutes and 1 Pump Starting after 30 minutes
	Starting after 30 minutes (Base Scenario)	Starting after 0 minutes	Starting after 30 minutes	Starting after 0 minutes				
DCT Basin A	1.53	1.41	1.40	1.18	1.53	1.53	1.53	1.29
DCT Basin B	1.63	1.51	1.50	1.27	1.63	1.63	1.63	1.38

Table 3-15: MSIV Areas Maximum Ponding Depths Summary

	Maximum Depth of Ponding (ft)
MSIV East	0.69
MSIV West	0.62

Figure 3-1: LIP Hyetograph

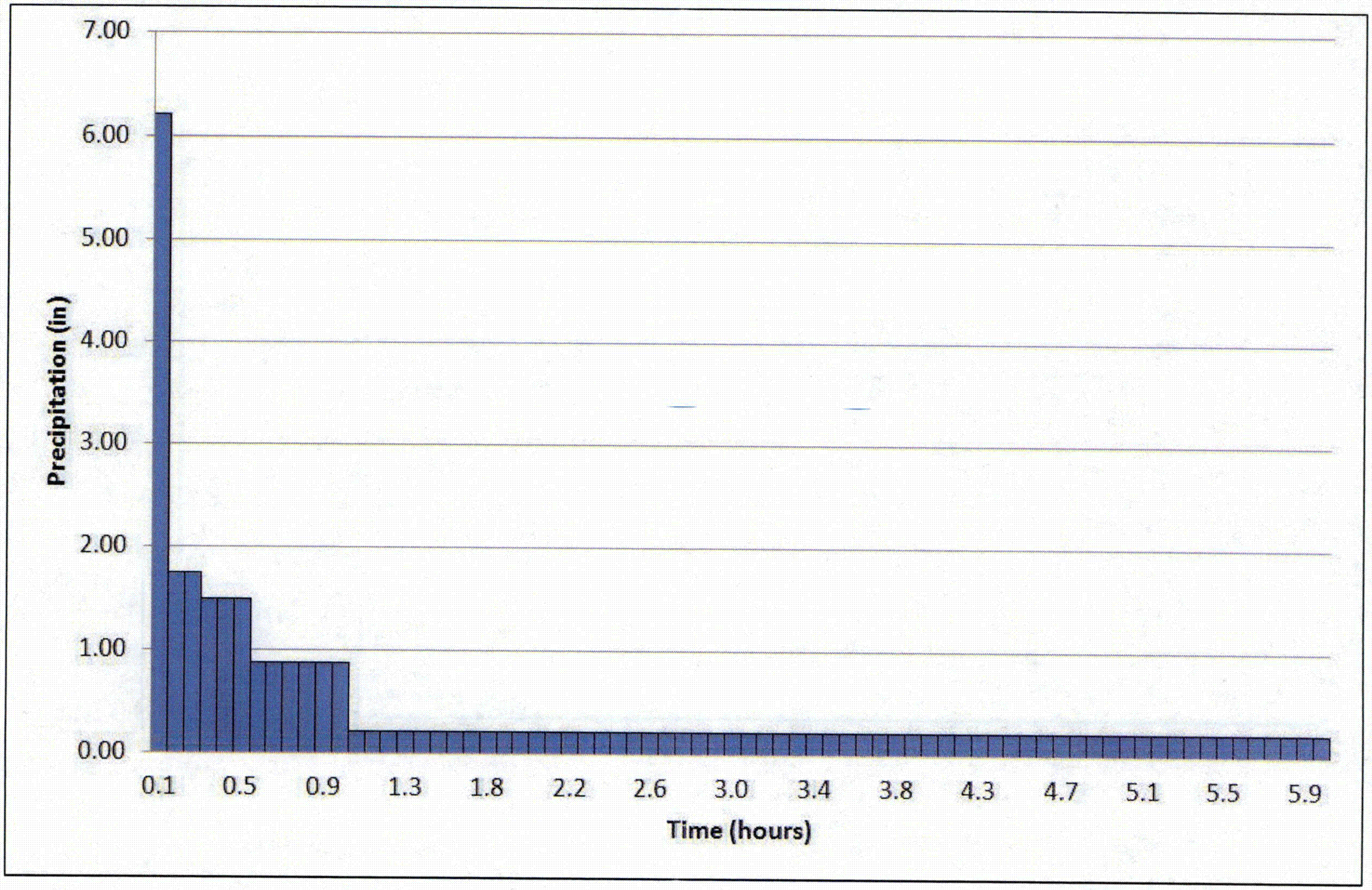


Figure 3-2: FLO-2D Modeled Site Features

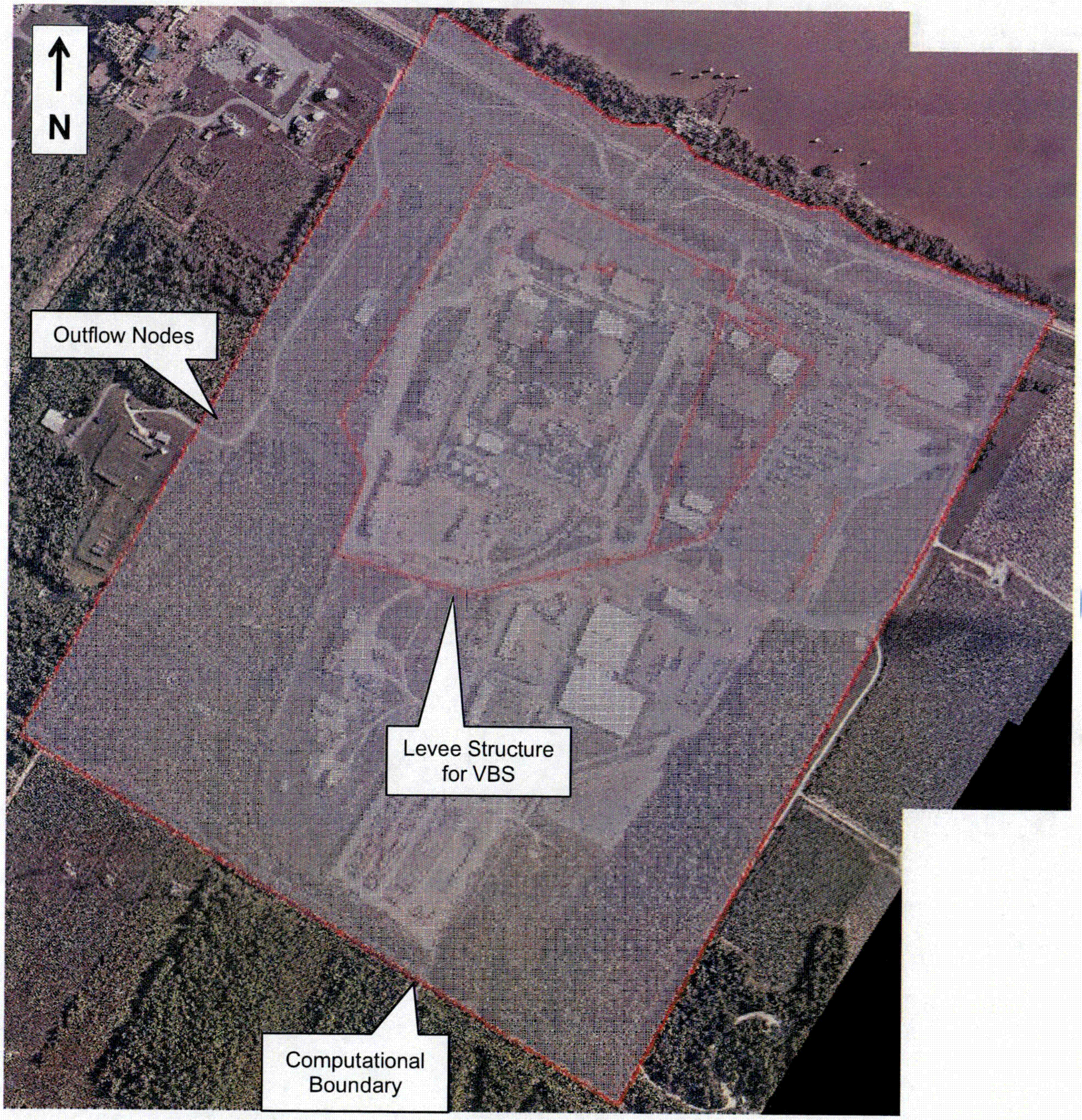
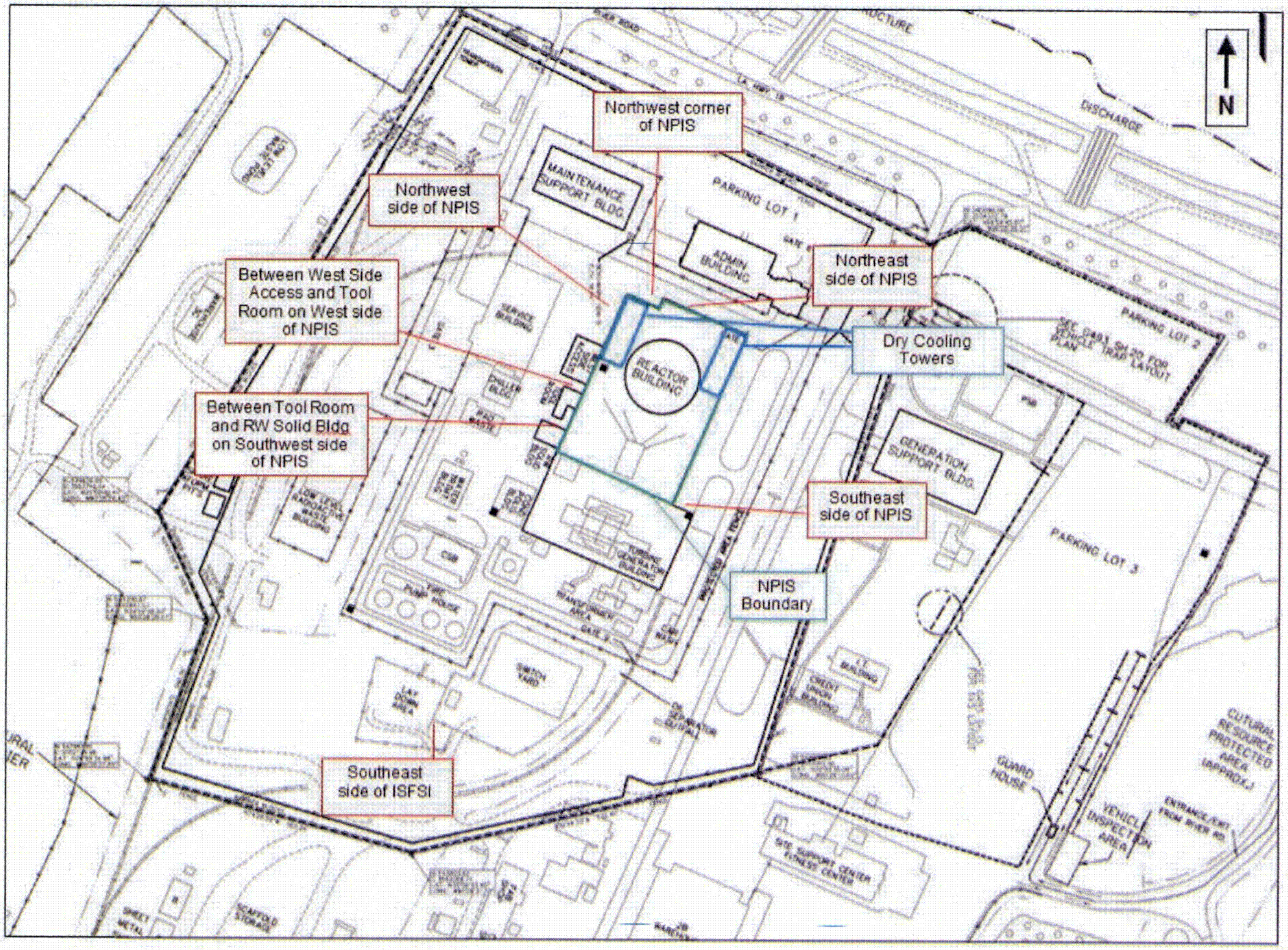


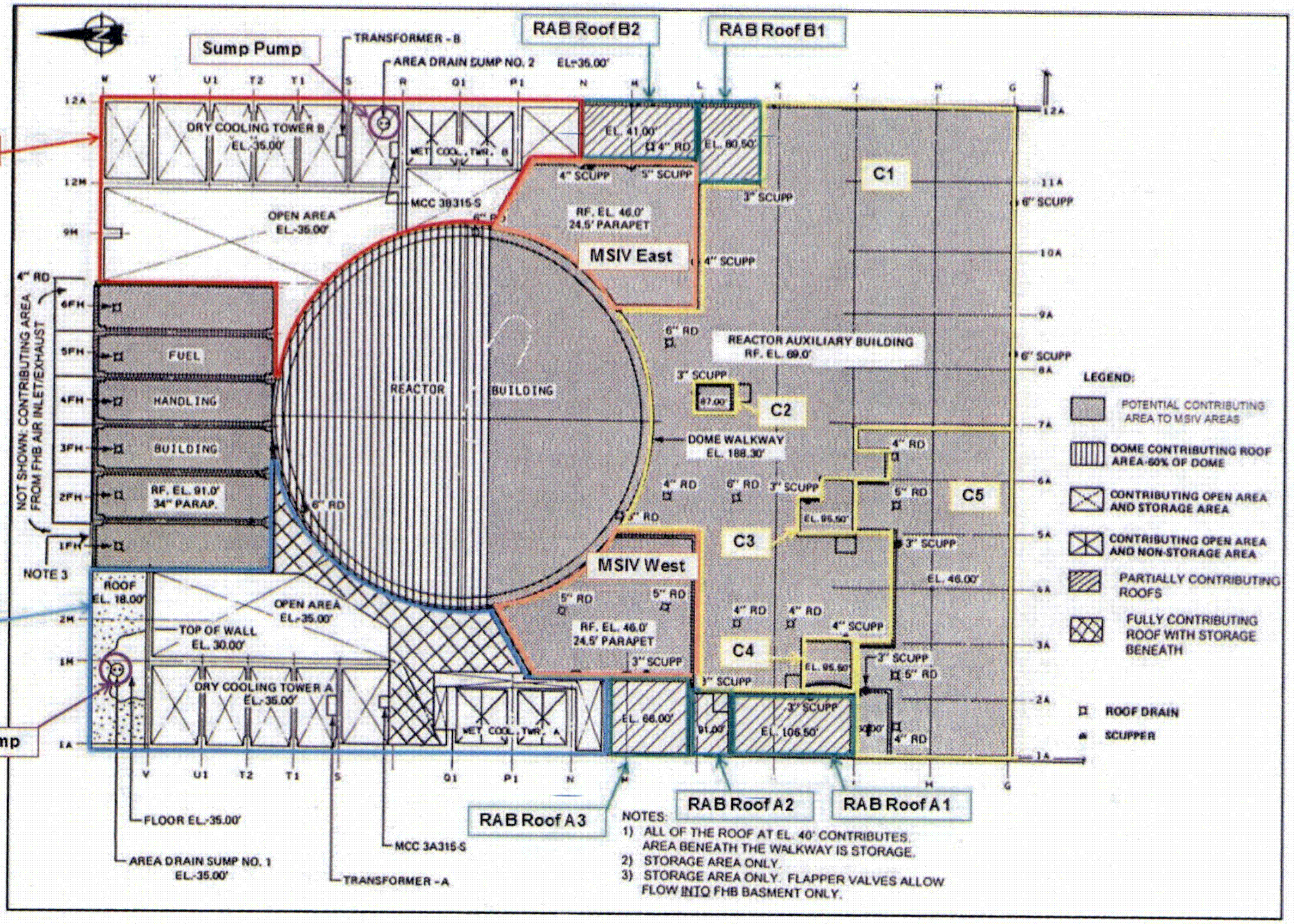
Figure not to scale.

Figure 3-3: WSES LIP Locations of Interest



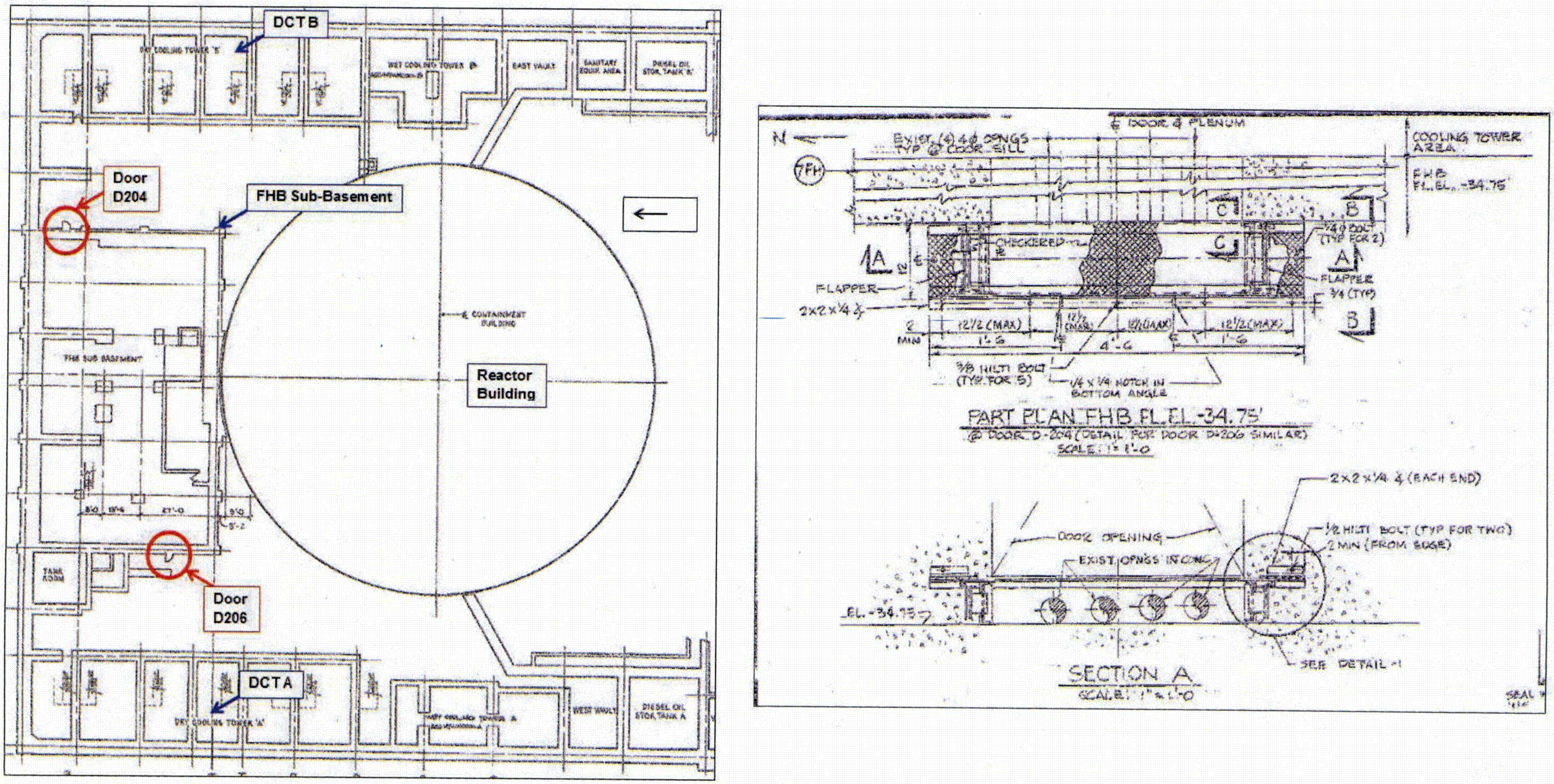
Any illegible text for features in this figure are not pertinent to the technical purposes of this document. Figure not to scale.

Figure 3-4: Relevant NPIS Details



Any illegible text or features in this figure are not pertinent to the technical purposes of this document. Figure not to scale. (WSES, 2001a)

Figure 3-5: Pipes Connecting the FHB to DCT Basins



Any illegible text or features in this figure are not pertinent to the technical purposes of this document. Figure not to scale. (WSES, 1986 and WSES, 1991a)

Figure 3-6: Flow Chart Showing Major Steps Involved in Calculating the Site-Specific PMP for LIP

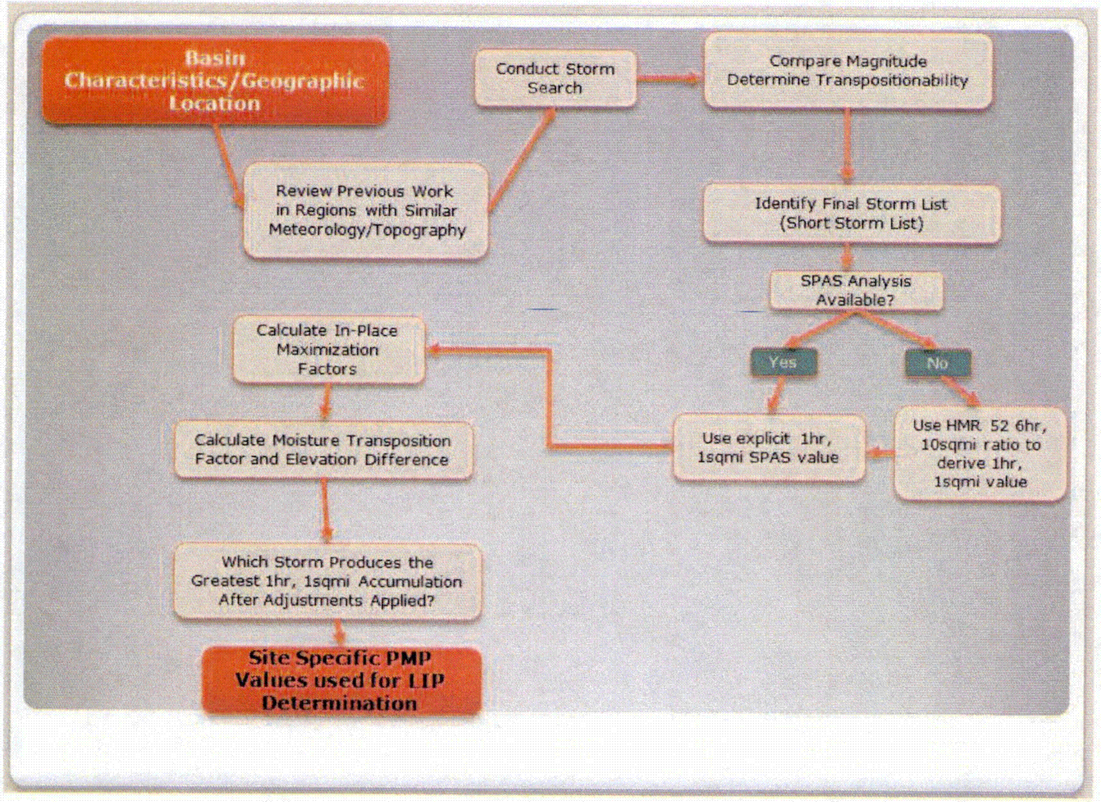


Figure 3-7: Domain Used to Identify Storms Used in the Analysis

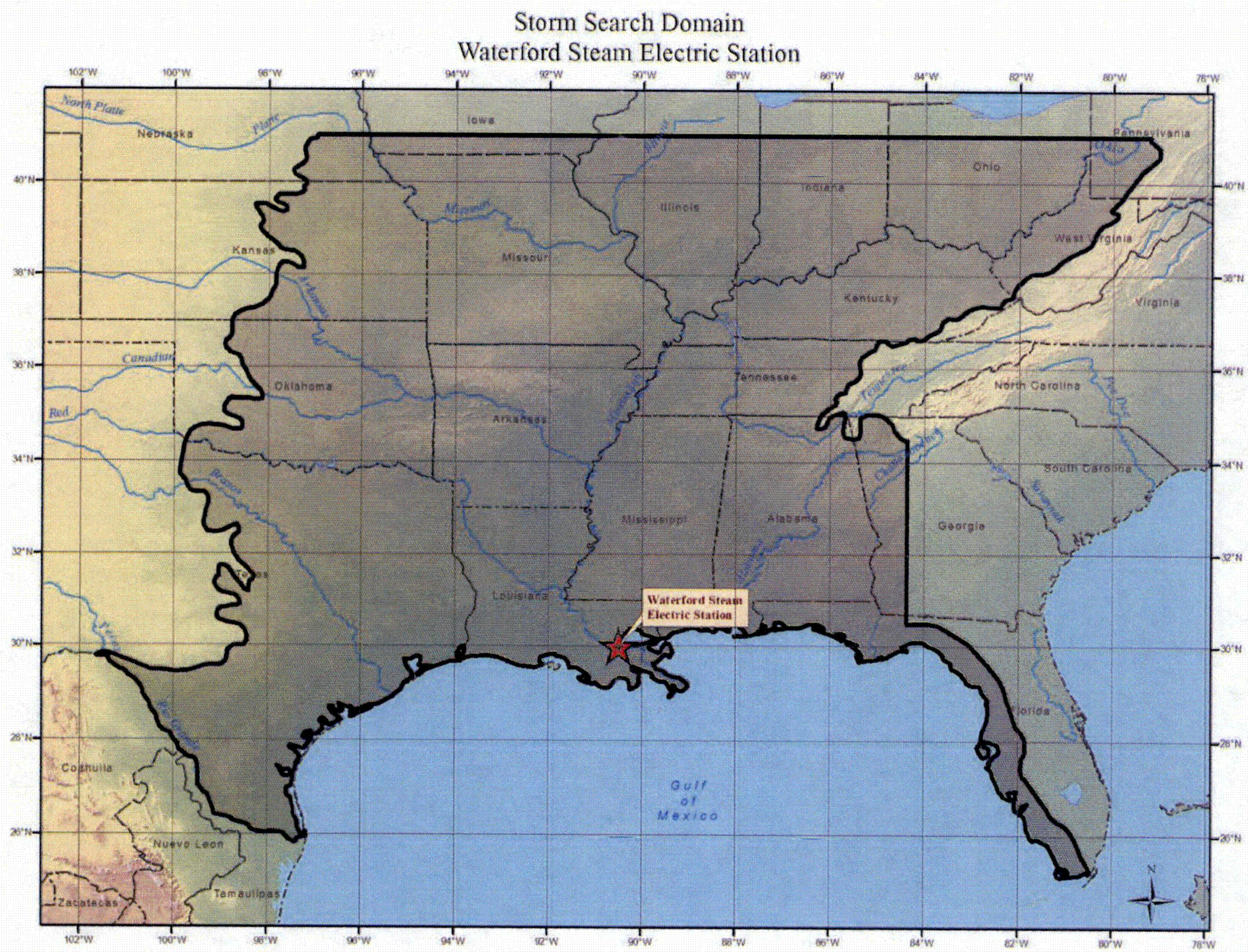
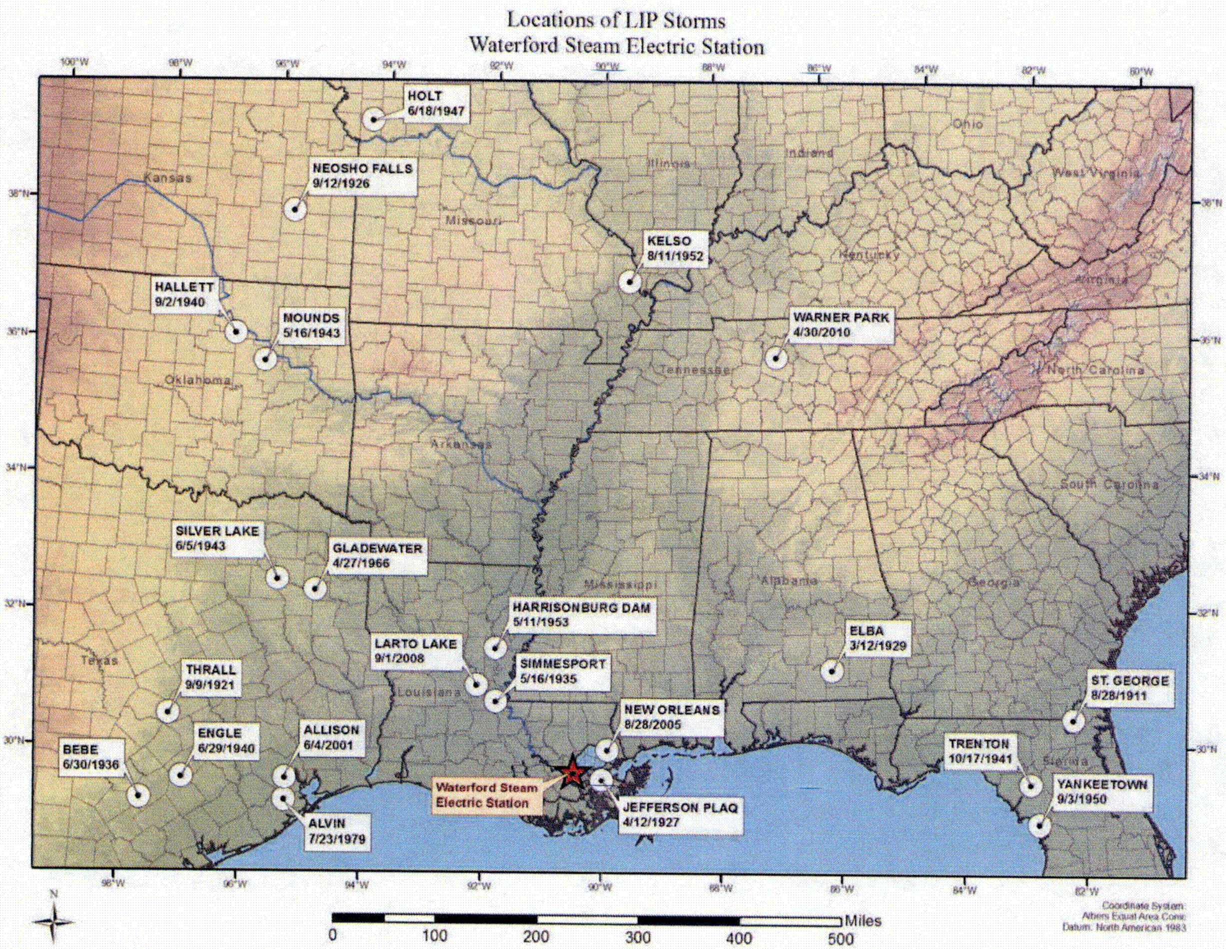
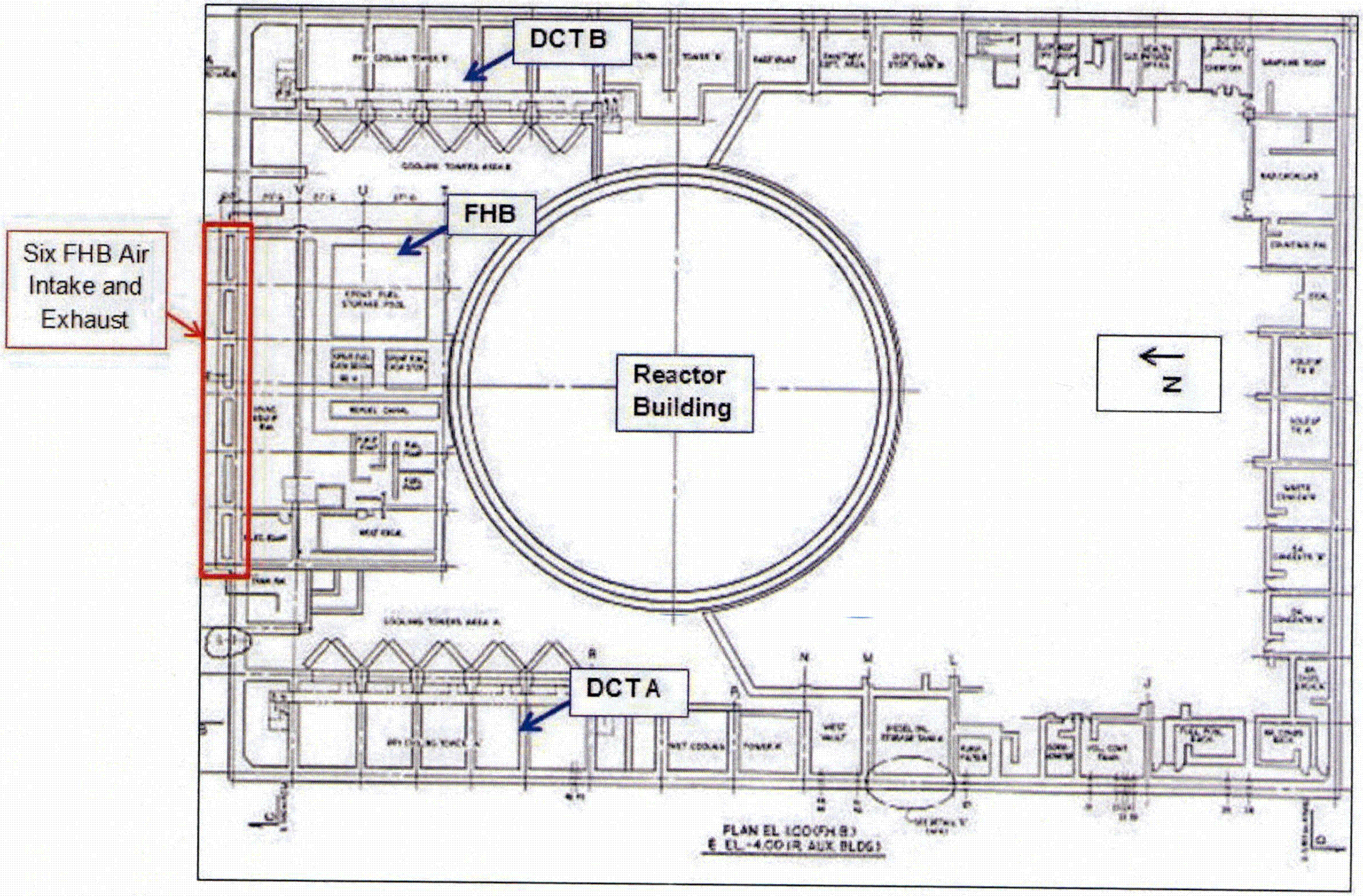


Figure 3-8: Locations of Storms Used to Calculate LIP Values for WSES



Note: Any illegible text or features in this figure are not pertinent to the technical purposes of this document

Figure 3-9: FHB Air Intake and Exhaust Openings Elev. +1 ft, MSL



Any illegible text or features in this figure are not pertinent to the technical purposes of this document. Figure not to scale. (WSES, 2011b)

Figure 3-10: DCT Basins Contributing Areas Layout

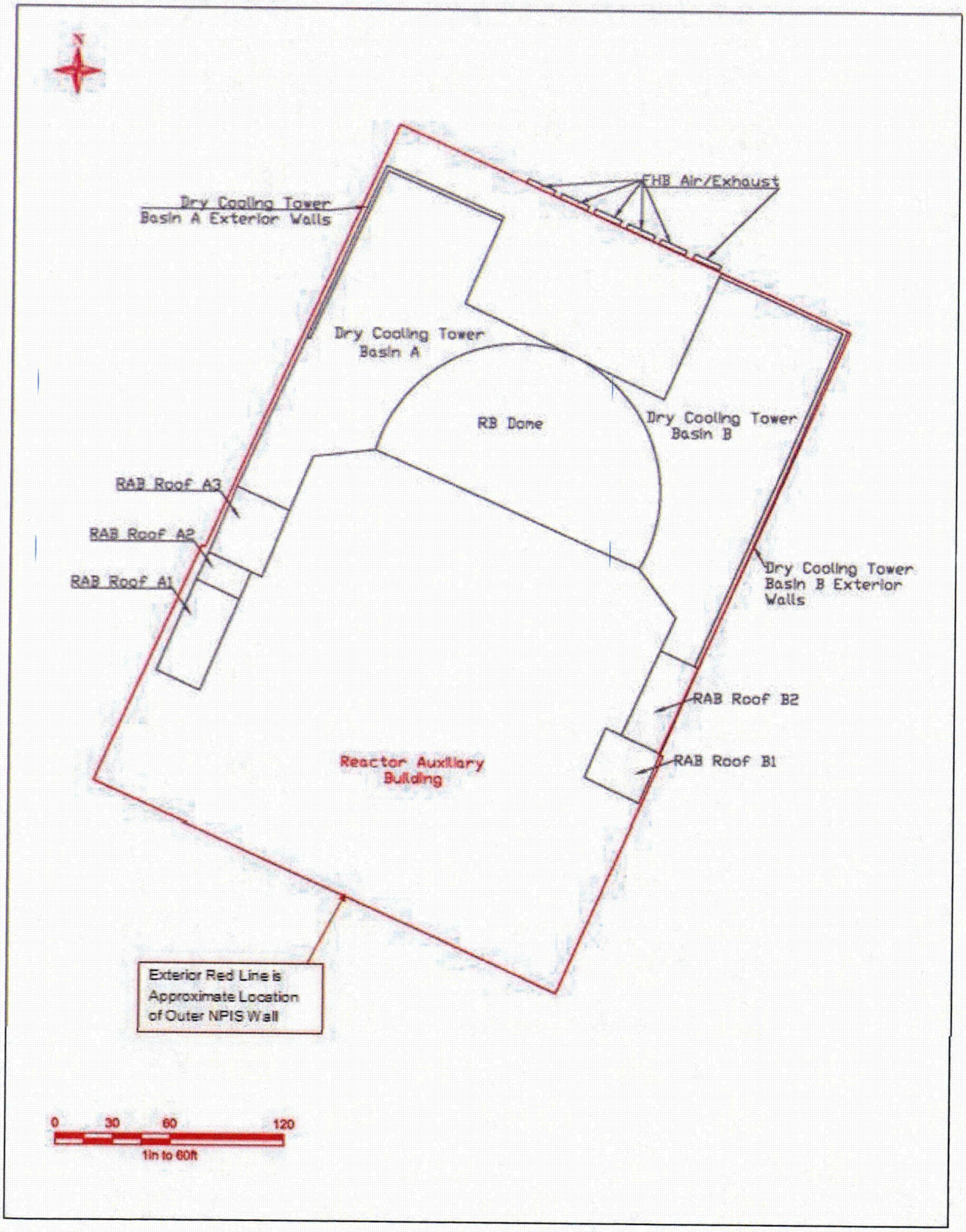


Figure 3-11: DCT Basins Storage Areas Layout

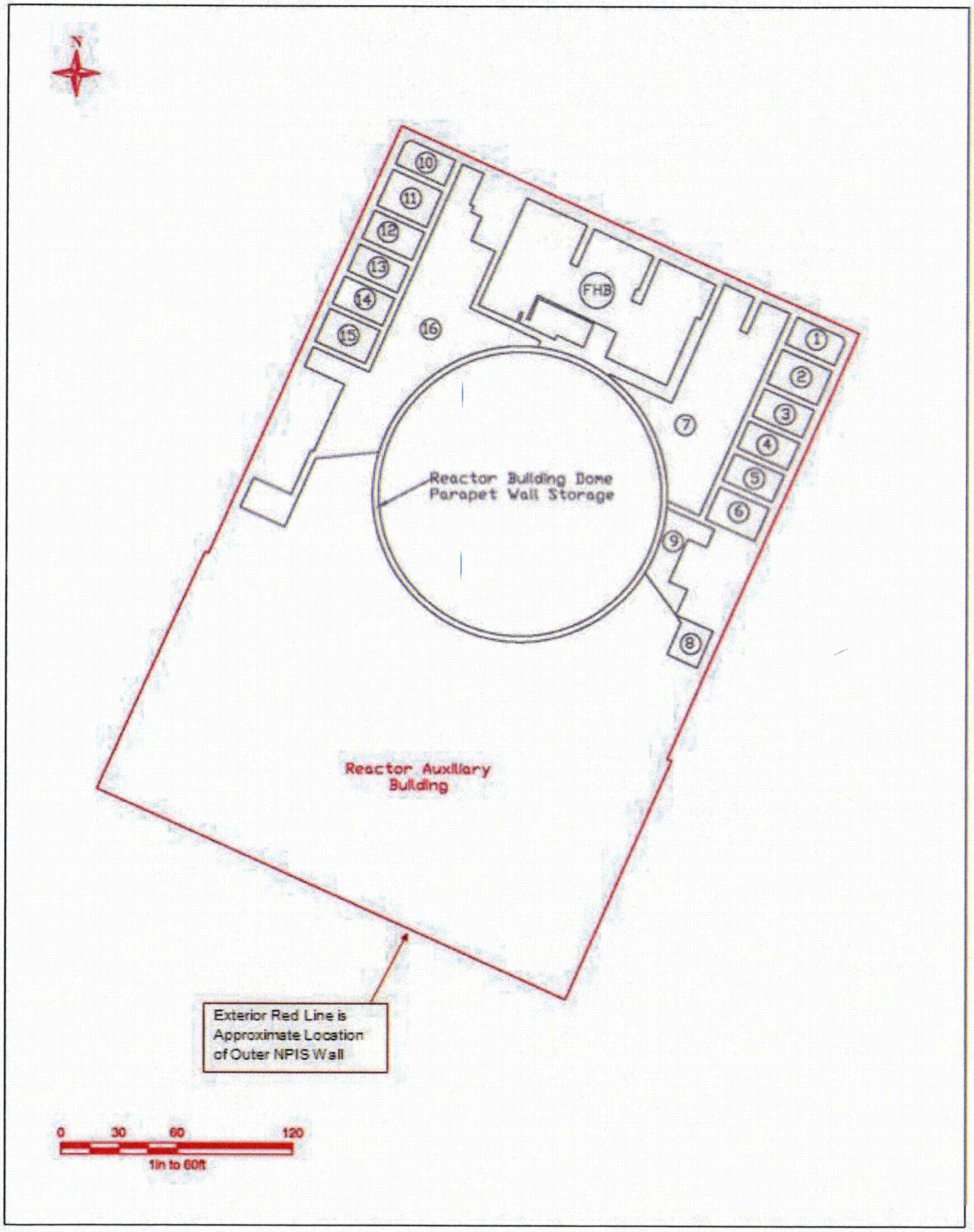


Figure 3-12: MSIV Contributing Areas Layout

Figure 3-13: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 30 Minutes

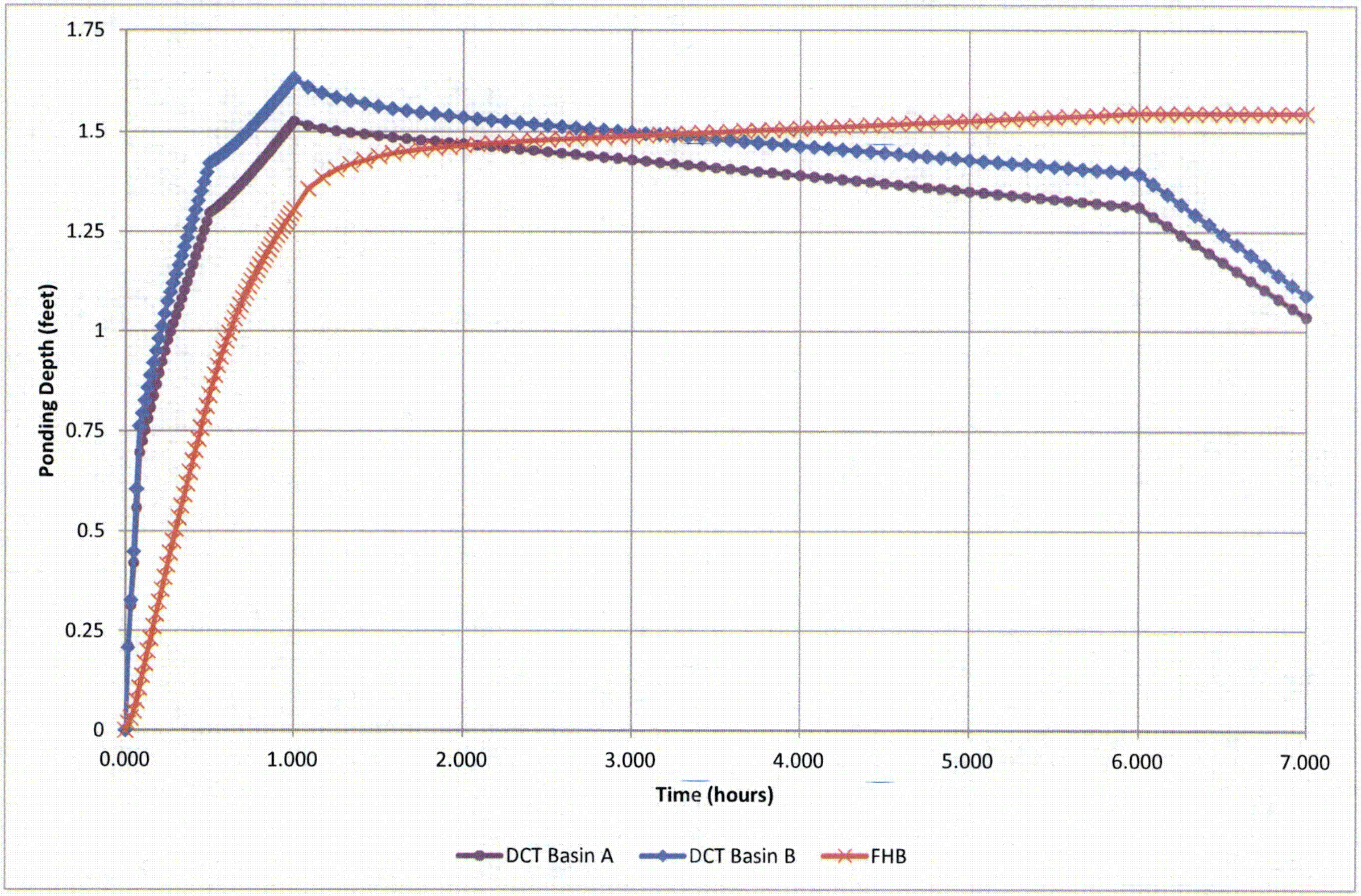


Figure 3-14: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 0 Minutes

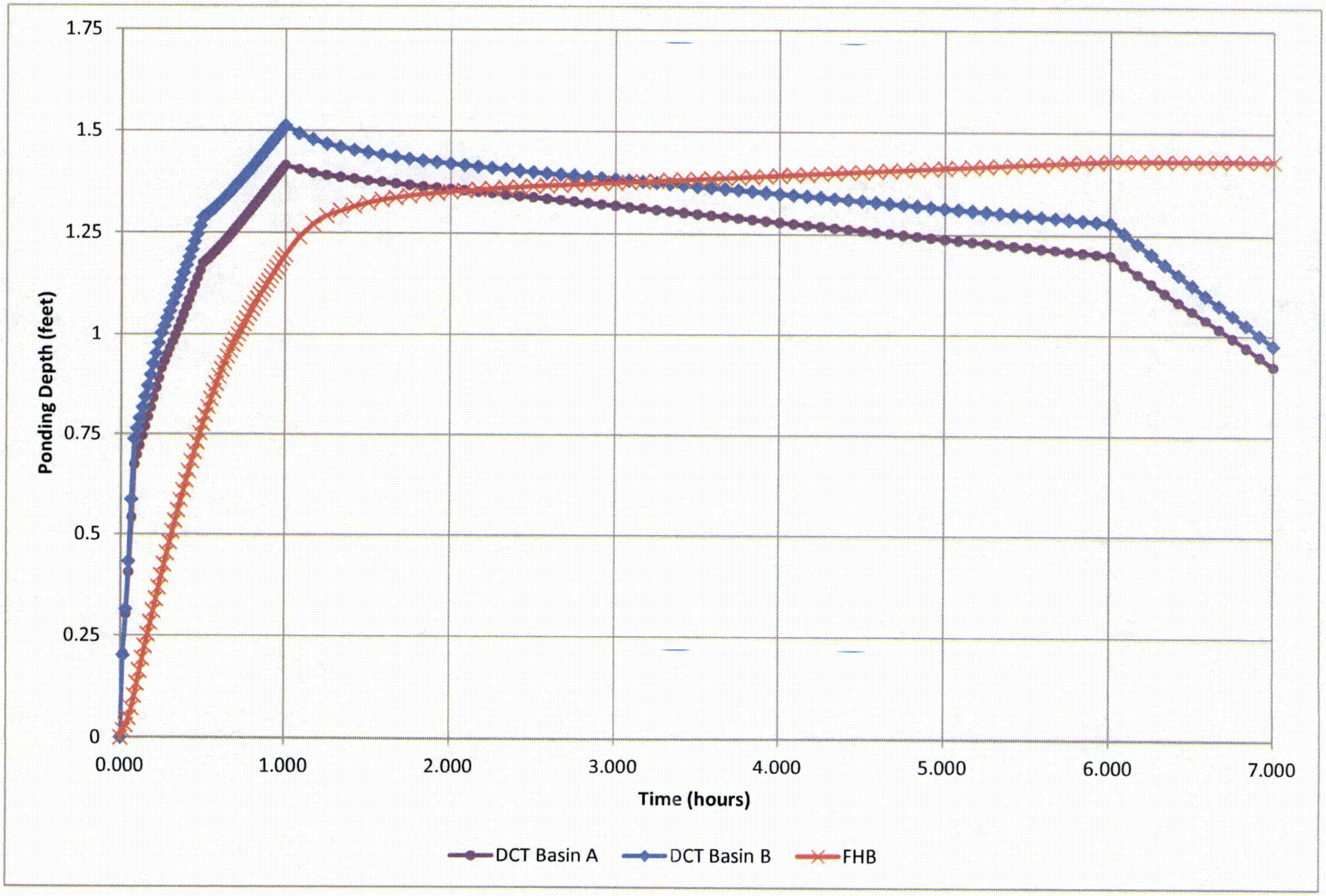


Figure 3-15: DCT Basins and FHB Flood Depths – 1 Sump Pump and 1 Portable Pump Starting after 30 Minutes

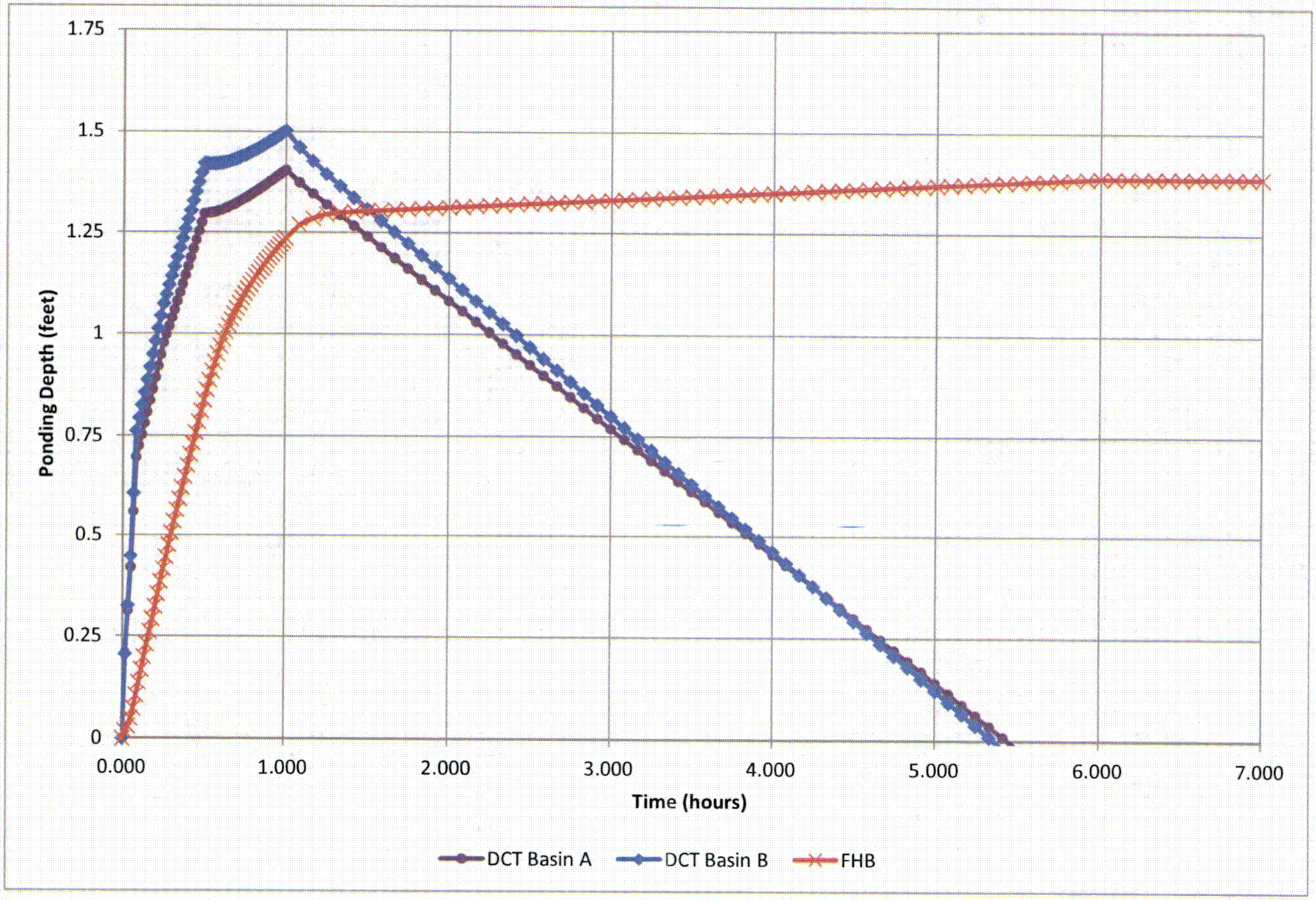


Figure 3-16: DCT Basins and FHB Flood Depths – 1 Sump Pump and 1 Portable Pump Starting after 0 Minutes

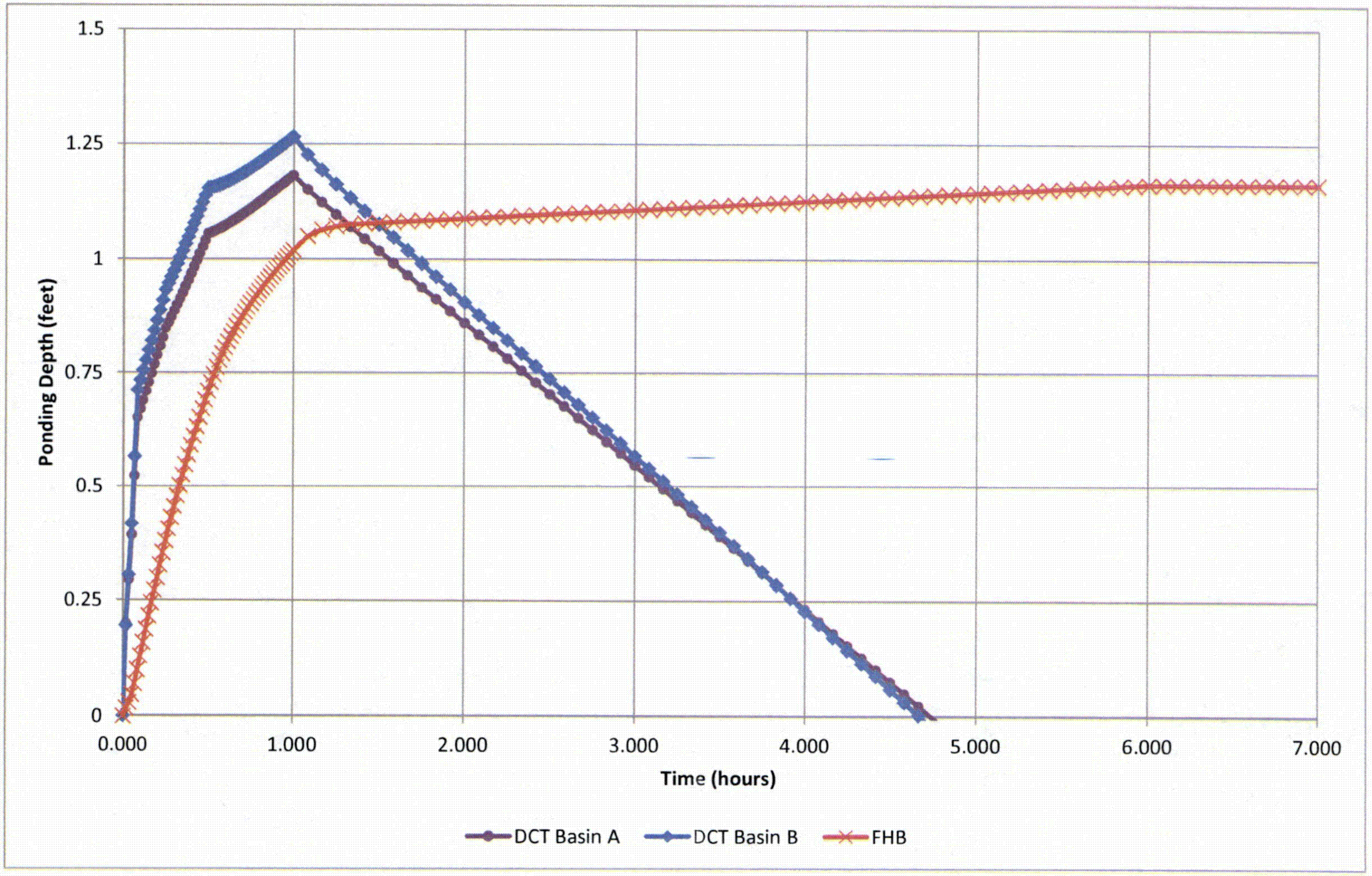


Figure 3-17: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 30 Minutes and 1 Portable Pump Starting after 1 Hour

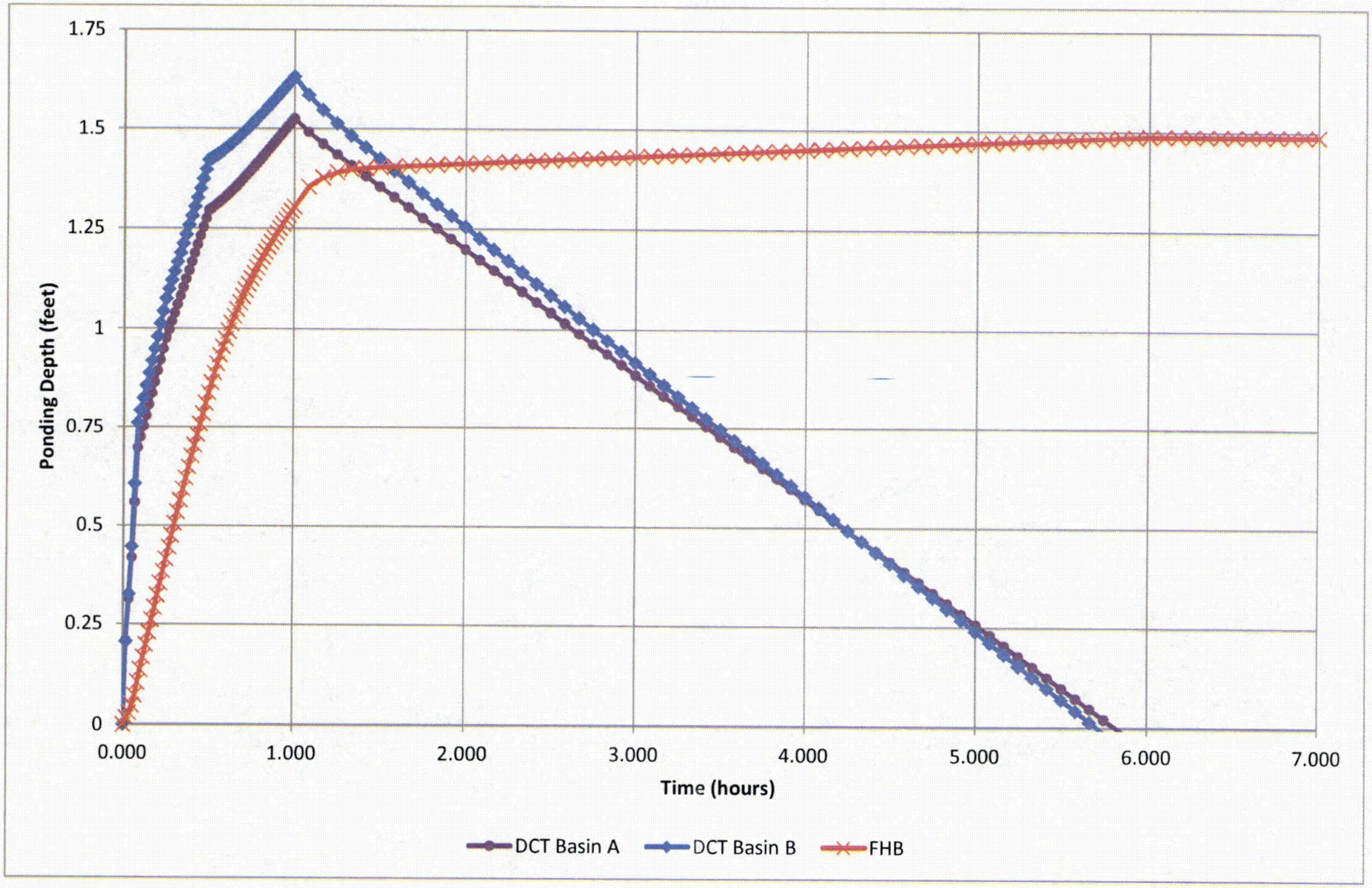


Figure 3-18: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 30 Minutes and 1 Portable Pump Starting after 3 Hours

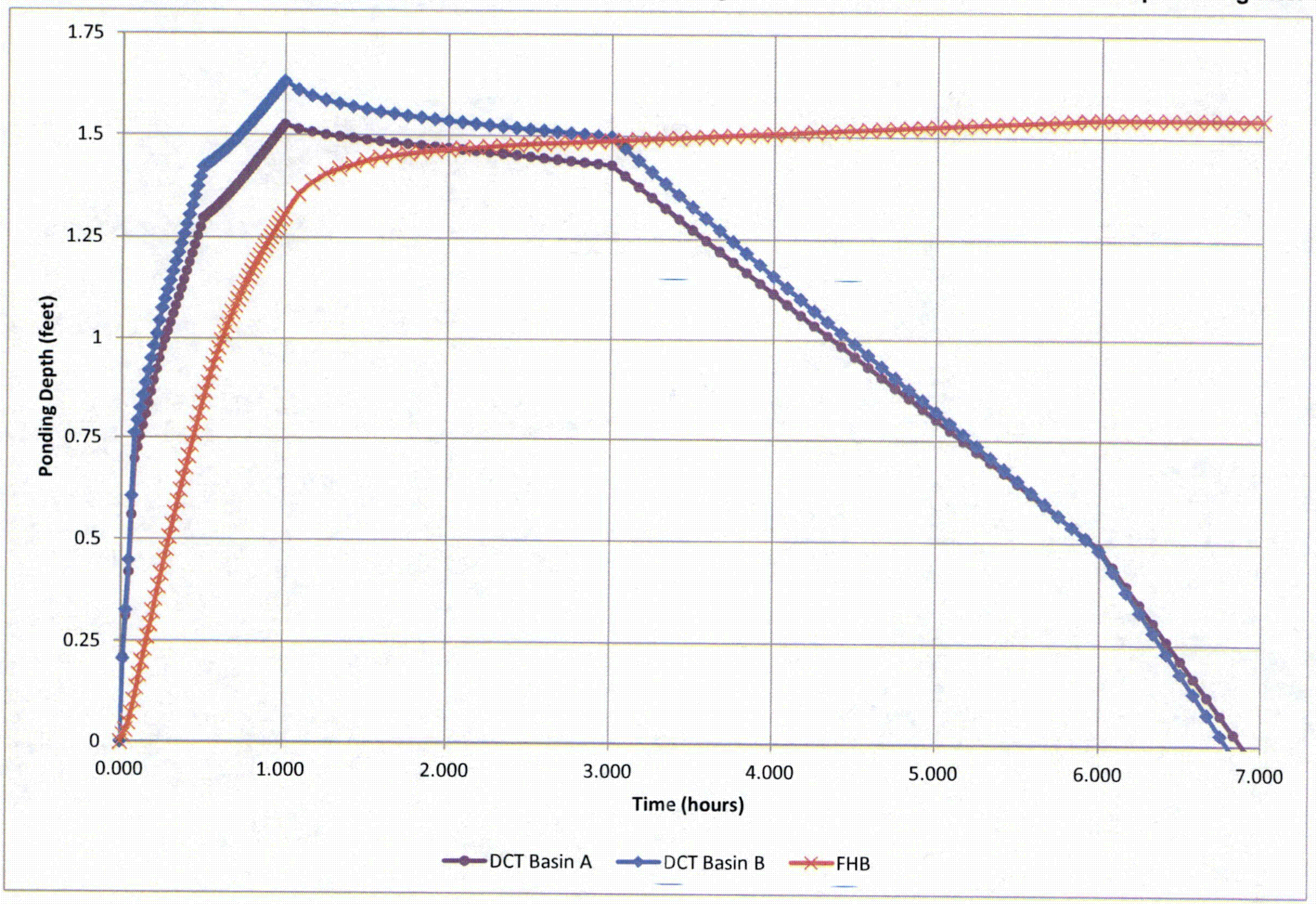


Figure 3-19: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 30 Minutes, 1 Portable Pump Starting after 1 Hour and 1 Portable Pump Starting after 3 Hours

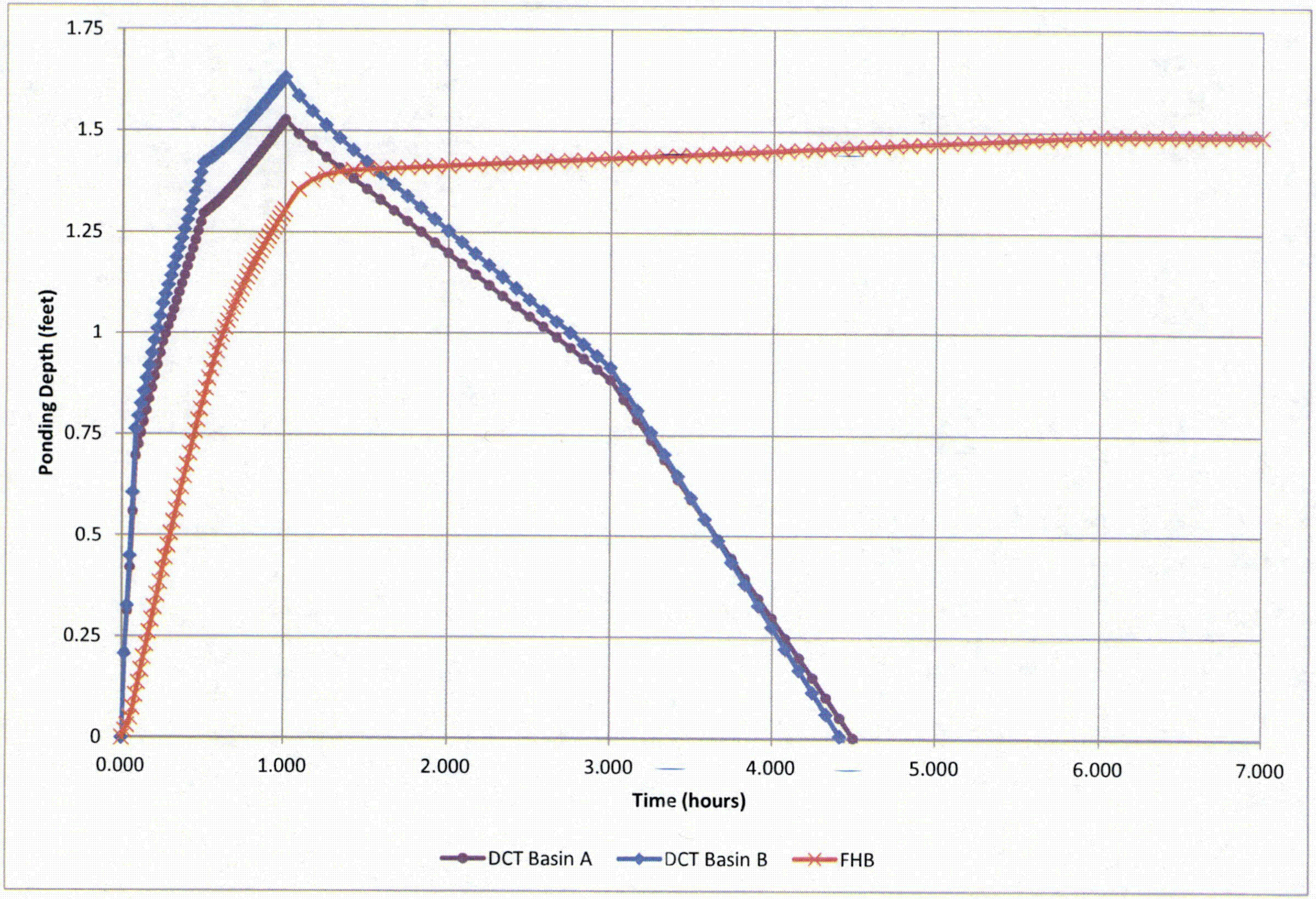


Figure 3-20: DCT Basins and FHB Flood Depths – 1 Sump Pump Starting after 0 Minutes and 1 Portable Pump Starting

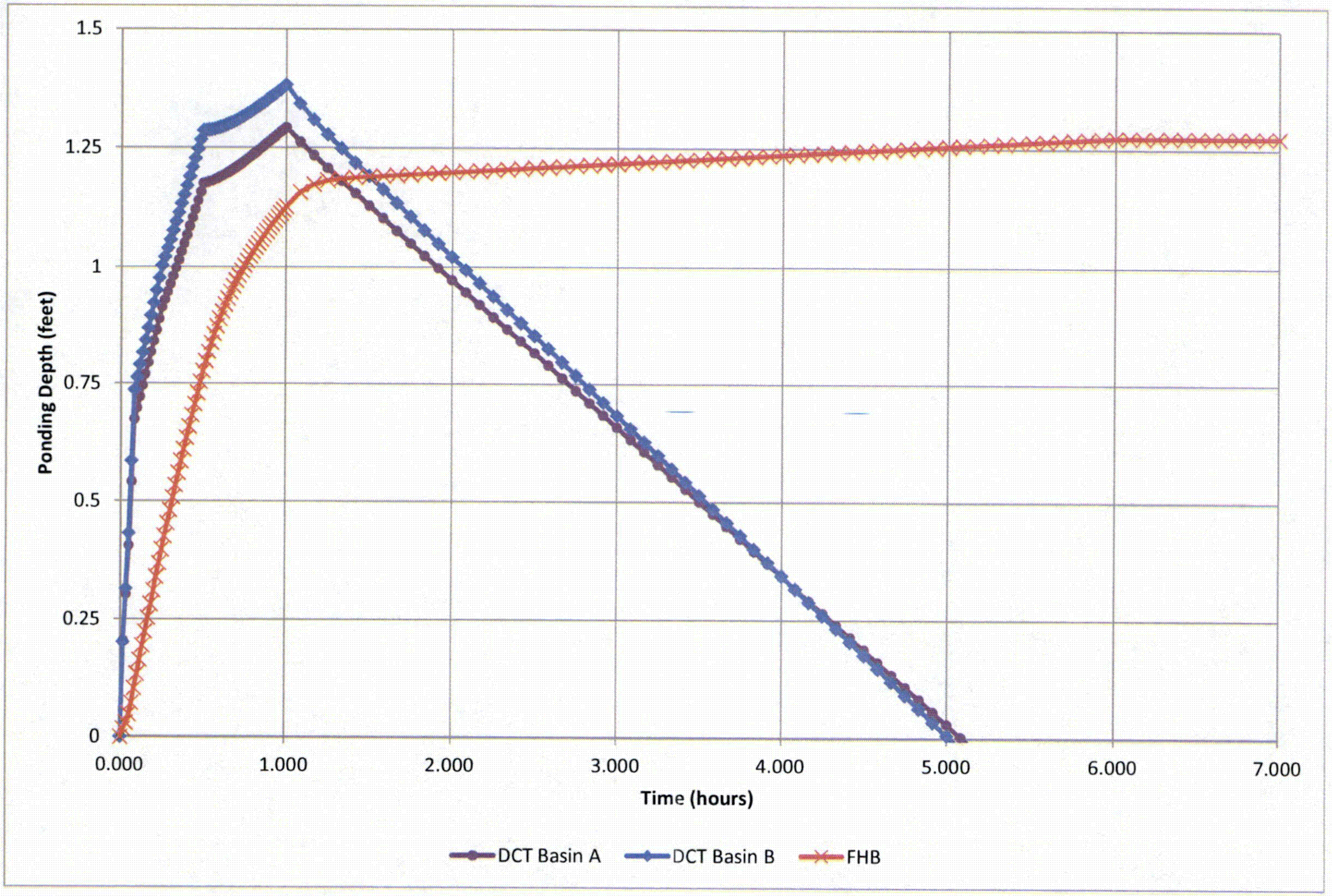


Figure 3-21: MSIV East Flood Depths

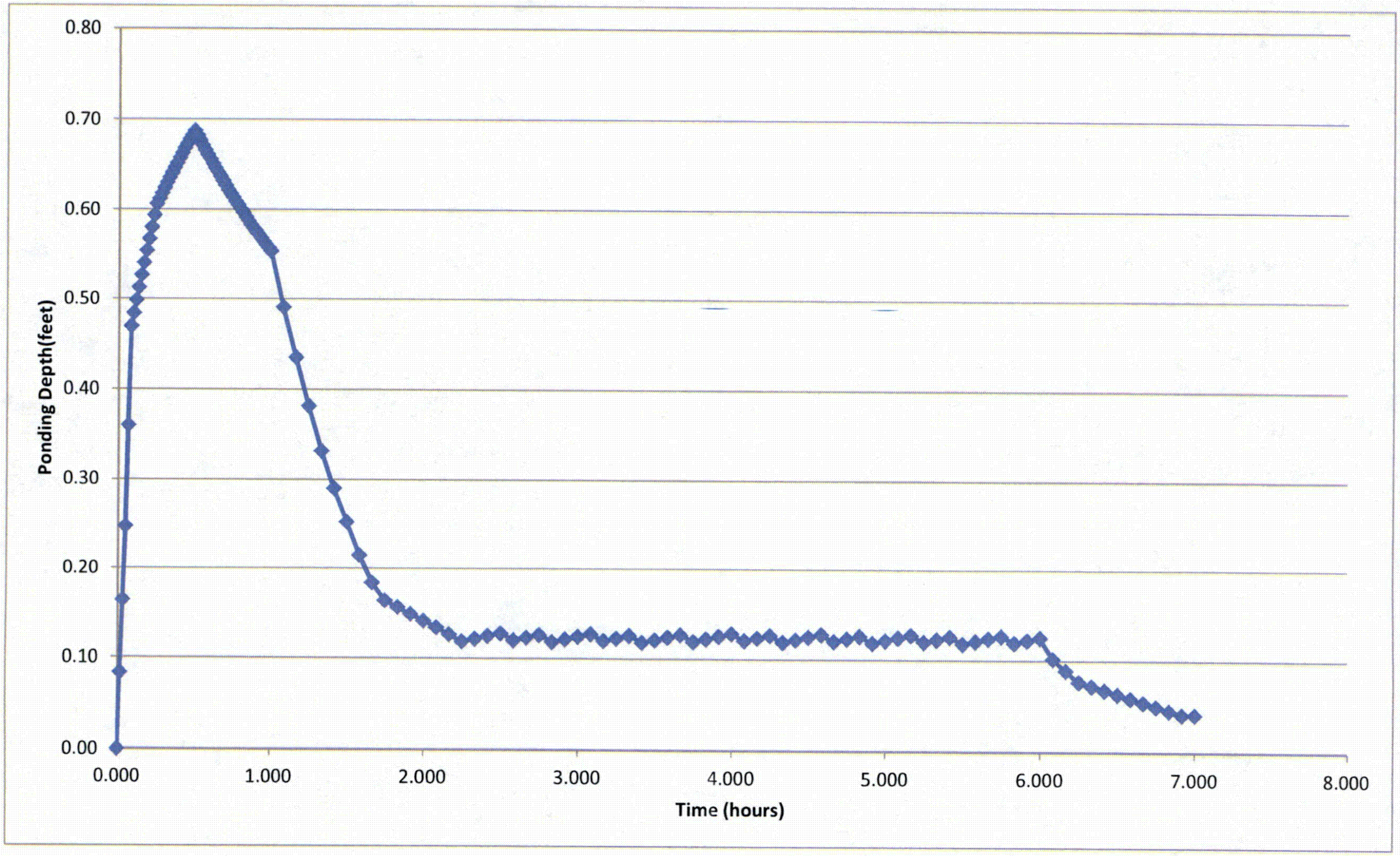


Figure 3-22: MSIV West Flood Depths