**Enclosure to**

**0CAN091602**

**Arkansas Nuclear One Units 1 and 2 Flooding Hazard Re-evaluation Report**

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AREVA

# AREVA Inc.

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## Arkansas Nuclear One Units 1 and 2 Flooding Hazard Re-Evaluation Report

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## Overview

This report describes the approach, methods, and results from the re-evaluation of flood hazards at the Arkansas Nuclear One nuclear power plant (ANO) Units 1 and 2. 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 ANO site information, including present-day site layout, topography, and current design 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 ANO 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 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 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.

## 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 March 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 Arkansas Nuclear One (ANO) Units 1 and 2. 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 the ANO Site.

Each of the re-evaluated flood causing mechanisms and the potential effects on the ANO 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 ANO. These mechanisms are: Probable Maximum Storm Surge, Probable Maximum Seiche, Probable Maximum Tsunami, Ice-Induced Flooding, and Channel Migration.

The ANO Unit 1 and Unit 2 design bases considered Local Intense Precipitation (LIP) bounded by the controlling design basis flood event, and did not specifically evaluate LIP flood elevations. As a result, the ANO Unit 1 and Unit 2 design bases are considered not-bounding for the Local Intense Precipitation (LIP) event. An evaluation of the impacts of LIP flooding at the ANO site concluded that there were no impacts to structures, systems or components important to safety.

The ANO Unit 1 and Unit 2 controlling design basis flood hazard included wind-generated waves and potential wave runup. The design basis peak flood hazard is caused by the combined effect of an Arkansas River Probable Maximum Flood (PMF) with coincident wind-generated waves and associated wave runup. The controlling design basis event for stillwater flood levels is generated by the Arkansas River PMF coincident with a sudden failure of the upstream Ozark Dam.

The re-evaluated flood hazard for the Arkansas River PMF coincident with upstream dam failure and wind-generated waves results in stillwater levels below the controlling design basis stillwater level, and peak flood hazard elevations (combined effect of the Arkansas River PMF coincident with upstream dam failure, wind-generated waves and associated wave runup) below the controlling design basis peak flood hazard. As a result, the re-evaluated flood hazard for flooding from the Arkansas River and associated combined effect floods are bound by the design basis.

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## Acronyms and Abbreviations

| Acronym/Abbreviation | Description |
|---|---|
| 10CFR50.54(f) | Title 10 of the Code of Federal Regulations, Section 50.54(f) |
| ANO | Arkansas Nuclear One (Site or Both Units) |
| ANS | American Nuclear Society |
| ANSI | American National Standards Institute |
| ASCE | American Society of Civil Engineers |
| CDB | Current Design Basis |
| CEM | Coastal Engineering Manual |
| CFR | Code of Federal Regulations |
| CSB | Central Support Building |
| DA | Depth-Area |
| DBE | Design Basis Earthquake |
| DEM | Digital Elevation Model |
| DTM | Digital Terrain Model |
| ECP | Emergency Cooling Pond |
| FEMA | Federal Emergency Management Agency |
| fps | ft per second |
| GEV | Generalized Extreme Value |
| GHCND | Global Historic Climatology Network-Daily |
| GIS | Geographic Information Systems |
| HHA | Hierarchical Hazard Assessment |
| HMR | Hydrometeorological Report |
| ISFSI | Independent Spent Fuel Storage Installation |
| ISG | Interim Staff Guidance (NRC) |

| Acronym/Abbreviation | Description |
|---|---|
| LiDAR | Light Detection and Ranging |
| LIP | Local Intense Precipitation |
| MCS | Mesoscale Convective System |
| MKARNS | McClellan-Kerr Arkansas River Navigation System |
| MSL | Mean Sea Level |
| NAVD88 | North American Vertical Datum of 1988 |
| NCDC | National Climatic Data Center |
| NGDC | National Geophysical Data Center |
| NGVD29 | National Geodetic Vertical Datum of 1929 |
| NOAA | National Oceanic and Atmospheric Administration |
| NRC | U.S. Nuclear Regulatory Commission |
| NRCS | Natural Resources Conservation Service |
| NTTF | Near-Term Task Force |
| NWS | National Weather Service |
| OBE | Operating Basis Earthquake |
| PMF | Probable Maximum Flood |
| PMP | Probable Maximum Precipitation |
| PMS | Probable Maximum Seiche |
| RMSE | Root Mean Square Error |
| SAR | Safety Analysis Report |
| SCS | Soil Conservation Service |
| SPAS | Storm Precipitation Analysis System |
| SSCs | Structures, Systems and Components |
| UHS | Ultimate Heatsink |
| USACE | U.S. Army Corps of Engineers |

| Acronym/Abbreviation | Description |
|---|---|
| USGS | U.S. Geological Survey |
| VBS | Vehicle Barrier System |
| WMO | World Meteorological Organization |

# 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.

Arkansas Nuclear One nuclear power plant (ANO) Unit 1 and Unit 2, located on a peninsula formed by Lake Dardanelle (Pool No. 10, Dardanelle Lock and Dam) is one of the sites required to submit information. ANO is located near river mile 210 of the Arkansas River, approximately 6 miles northwest of Russellville, Arkansas. For the purpose of this report, ANO or ANO site refers to both units, unless otherwise specified.

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 design basis (CDB) 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 March 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 ANO.

## 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 ANO.

Each of the re-evaluated flood causing mechanisms and the potential effects on the ANO 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 ANO. 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 CDB 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.

This report compares the results of the flood hazard re-evaluation to the Current Design Basis of the plant, unless otherwise specified.

Discussions in this report which include the terminology "design basis" (CDB) indicates information developed to determine flooding hazard and requirements for flood protection, as indicated in Section 2.4 and 3.4 of the ANO Unit 1 and Unit Safety Analysis Reports (SARs) (ANO, 2016a; ANO, 2016b).

Note that results reported in this report are applicable to both ANO Unit 1 and Unit 2, unless otherwise specified.

## 1.5 Elevation Values

Elevations in the Safety Analysis Reports (SAR) for both Unit 1 and Unit 2 are reported in terms of Mean Sea Level (MSL). At this location, MSL is considered equivalent to the National Geodetic Vertical Datum of 1929 (NGVD29). To convert elevations reported in MSL or NGVD29 to the North American Vertical Datum of 1988 (NAVD88), add 0.03 ft to the NGVD29/MSL value (AREVA, 2016).

## 1.6 References

**AREVA, 2016.** AREVA Document No. 32-9207382-000, Arkansas Nuclear One Flooding Hazard Re-Evaluation – Local Intense Precipitation – Generated Flood Flow and Elevations, 2016.

**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)

**ANO, 2016a.** Arkansas Nuclear One – Unit 1, Safety Analysis Report, Facility Operating License Number DPR-51, Docket Number 50-313, See AREVA Document No. 38-9257436-000, 2016.

**ANO, 2016b.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

# 2.0 INFORMATION RELATED TO THE FLOOD HAZARD

## 2.1 Detailed Site Information

The ANO site is located on a peninsula formed by Lake Dardanelle (Pool No. 10, Dardanelle Lock and Dam; See Figure 2-1). ANO is located near river mile 210 of the Arkansas River, approximately 6 miles northwest of Russellville, Arkansas (see Figure 2-2). The site is centrally situated on a "peninsula," about two miles wide and two miles long, that extends into Lake Dardanelle (also referred to as Dardanelle Reservoir). On three sides, the site is surrounded by reservoir water; the shortest stretch of which is approximately one mile in a southeast direction. Generally, the site peninsula is at an elevation of about 400 ft NGVD29, but with rises above 500 ft NGVD29.

### 2.1.1 Site Layout

Figure 2-3, Site Topography and Layout, shows the ANO site layout and topography, including important features and locations related to flood hazards (AREVA, 2013).

## 2.2 Current Design Basis Flood Elevation

The current design basis and related flood elevation from natural sources is described in the ANO Unit 1 and Unit 2 SARs (ANO, 2016a and ANO, 2016b, Sections 2.4 and 3.4) and in the Arkansas Nuclear One Flooding Walkdown Submittals (ANO, 2012a and ANO, 2012b).

The controlling design basis flooding event with respect to stillwater flood levels at the ANO site is due to the instantaneous failure of the upstream Ozark Dam combined with the probable maximum flood (PMF) on the Arkansas River. The design basis flood event results in Stillwater flood levels up to [redacted] ft NGVD29 (358 ft NGVD29 due to PMF, (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) [redacted]. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

The effects of Wind-Generated Waves were evaluated as part of the CDB coincident with a PMF on the Arkansas River. A 2.5 foot wave was applied to the Arkansas River PMF elevation of 358 ft NGVD29, for a combined effect flood elevation of 360.5 ft NGVD29. Potential wave runup was identified as being less than 10 ft above the 358 ft NGVD29 stillwater PMF level, for a peak flood hazard level of 368 ft NGVD29. The CDB did not evaluate the combined effects of wind-generated waves coincident with a PMF and upstream dam failure.

Groundwater intrusion into safety related structures is considered a credible source during flooding events. The design of the Seismic Class 1 structures considers the source as credible and provides protection against groundwater intrusion into these structures as low as the base grade level at elevation 317' MSL. Minor groundwater leaks are anticipated and discussed in the ANO Unit 2 SAR.

The effects of local intense precipitation (LIP) on the site are bounded by the hypothetical Ozark Dam failure event. The Auxiliary Building Roof is structurally designed to withstand roof ponding loads; the Auxiliary Building Roof drainage is sufficient to protect safety-related equipment; and the site drainage is not credited for flooding protection. Flood elevations due to LIP were not specifically evaluated as part of the CDB flood hazard evaluation, but were identified as being screened out as a flood hazard due to the height of flood protections at ANO.

### 2.2.1 Elevation of Safety Structures, Systems and Components

The safety-related structures, systems, and components at ANO Unit 2 are capable of withstanding the worst flooding caused by any of a number of hypothetical events. According to the ANO Unit 2 SAR (ANO, 2016b), critical equipment and components are protected from splash effects up to 10 feet above PMF level of 358 ft NGVD29 (i.e. 368 ft NGVD29) as follows: "…critical components and equipment are either protected or are located above Elevation 369 feet".

The Emergency Cooling Pond (ECP) represents the Ultimate Heat Sink (UHS) for the ANO site. The design basis for the ECP included a probable maximum precipitation induced high water level of 350.44 ft (ANO, 2008). The majority of the pond is below the surrounding grade, with the only portion of the pond perimeter above grade occurring in the immediate vicinity of the spillway, located on the northwest side of the pond. The spillway was constructed with a safety factor under flood conditions of 16.14, well beyond the required factor of safety of 1.3. (ANO, 2008)

## 2.3 Current Design Basis Flood Protection and Mitigation Features

SSCs important to safety are flood protected either because of their location above the postulated maximum flood level, or because they are enclosed in reinforced concrete, Seismic Class 1 structures. The Seismic Class 1 structures that may be affected by a design basis flood at the site are designed to withstand the postulated floods for the site using the hardened flood protection approach. The hardened protection approach means structural provisions are incorporated in the plant's design that will protect SSCs important to safety from the static and dynamic effects of a flood. As part of the hardened approach, watertight doors and equipment hatches as well as watertight piping and electrical penetrations are installed below the maximum flood level. (ANO, 2012b)

ANO Unit 1 site adverse weather procedure describes the actions to be taken in the event of plant flooding caused by natural phenomena at the site. This procedure provides actions to be taken based on different levels of the Dardanelle Reservoir as measured at the Unit 1 Intake Structure. The site adverse weather procedure not only addresses plant actions to address flooding, but also provides directions to mobilize outside parties to perform the jumper hookups. (ANO, 2012a)

ANO Unit 2 site adverse weather procedure describes the actions to be taken in the event of plant flooding caused by natural phenomena at the site. This procedure provides actions to be taken based on different levels of the Dardanelle Reservoir as measured at the Unit 2 Intake Structure. The Unit 2 site adverse weather procedure gives specific details for the verification and closure of the various openings and penetrations in the flood protected buildings. (ANO, 2012b)

ANO site preventative maintenance procedures address the required maintenance for components such as doors and hatches that require procedural actions in order to maintain flood protection. The inspection interval of the doors and seals is not associated with a specific water level or flood situation at ANO but is established as part of the maintenance work order planning system. (ANO, 2012a)

There are three temporary actions required to be installed prior to a significant flooding event at ANO. The actions are: to install jumpers from the 161 kV Pleasant Hill transmission line to the Startup Transformer #2; any screened opening of the Startup 2 Transformer Bus Duct below [redacted] ft NGVD29 must be sealed; and a blind flange must be installed in the discharge of drumming station and hot fan supply 2VSF-38. No other temporary active or passive flood protection measures are required to be installed for protection of SSCs important to safety during flooding conditions at ANO Units 1 and 2. Based on the current design basis, several Unit 1 and Unit 2 doors are assumed to remain closed or be verified as closed during a flooding event. ANO's site adverse weather procedure provides instructions to verify and/or close the doors as part of the response in accordance with anticipated and measured flood levels. All doors, with the exception of the Emergency Diesel Fuel Vault doors for both Units 1 and 2, can be accessed without requiring personnel to travel outdoors. Therefore, only the Emergency Diesel Fuel Vault doors would require personnel to travel outside into potentially adverse weather conditions. However, these doors are maintained in a normally closed position, thus, adverse weather impacts are not considered a credible hindrance to the flooding response.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

### 2.3.1 CDB Flood Causing Mechanisms

The potential impacts from several flood causing mechanisms are evaluated in the ANO Unit 1 and Unit 2 SARs (ANO, 2016a and ANO, 2016b, Section 2.4). These events include: PMF on the Arkansas River; coincident wind generated waves; and failure of upstream dams (both hydrologic failure and seismic failure). The SARs also include discussion of the combined effect scenario of Wind-Generated Waves coincident with PMF on the

Arkansas River, as well as the combined effect of the PMF on the Arkansas River coincident with the failure of the upstream Ozark Dam. Other flooding mechanisms were not addressed in the SARs, such as: tsunami; seiche, storm surge; channel migration; ice-induced flooding; and various combined event scenarios. Local intense precipitation is discussed in Section 2.4.2.3 of the Unit 2 SAR (ANO, 2016b), indicating that it is bound by the design basis flood event and not considered a hazard to plant safety.

## 2.4 Design Basis Flood-Related and Flood Protection Changes

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

## 2.5 Watershed and Local Area Changes

### 2.5.1 Watershed Changes

Prior to the completion of the McClellan-Kerr Arkansas River Navigation System in 1970, the Arkansas River occasionally naturally meandered into new courses creating oxbows and cutoffs. The impoundments created by the dams built as part of the navigation system added stability by maintaining the width of the river and creating a semi-permanent edge with flow staying within a relatively fixed channel. As a result, changes in the watershed subsequent to the completion of the navigation system are limited. See Section 3.8 for additional details.

## 2.6 Additional Site Details – Walkdown Results

The findings reported in the Walkdown Reports (ANO, 2012a and ANO, 2012b) 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.

## 2.7 References

**ANO, 2008.** "Emergency Cooling Pond Spillway Replacement", EC-443, July 25, 2008. See AREVA Document No. 38- 9257436-000.

**ANO, 2012a.** Arkansas Nuclear One Unit 1 Flooding Walkdown Submittal; Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding, Engineering Report No. CALC-ANO1-CS-12-00003, See AREVA Document 38-9207373-000, 2012.

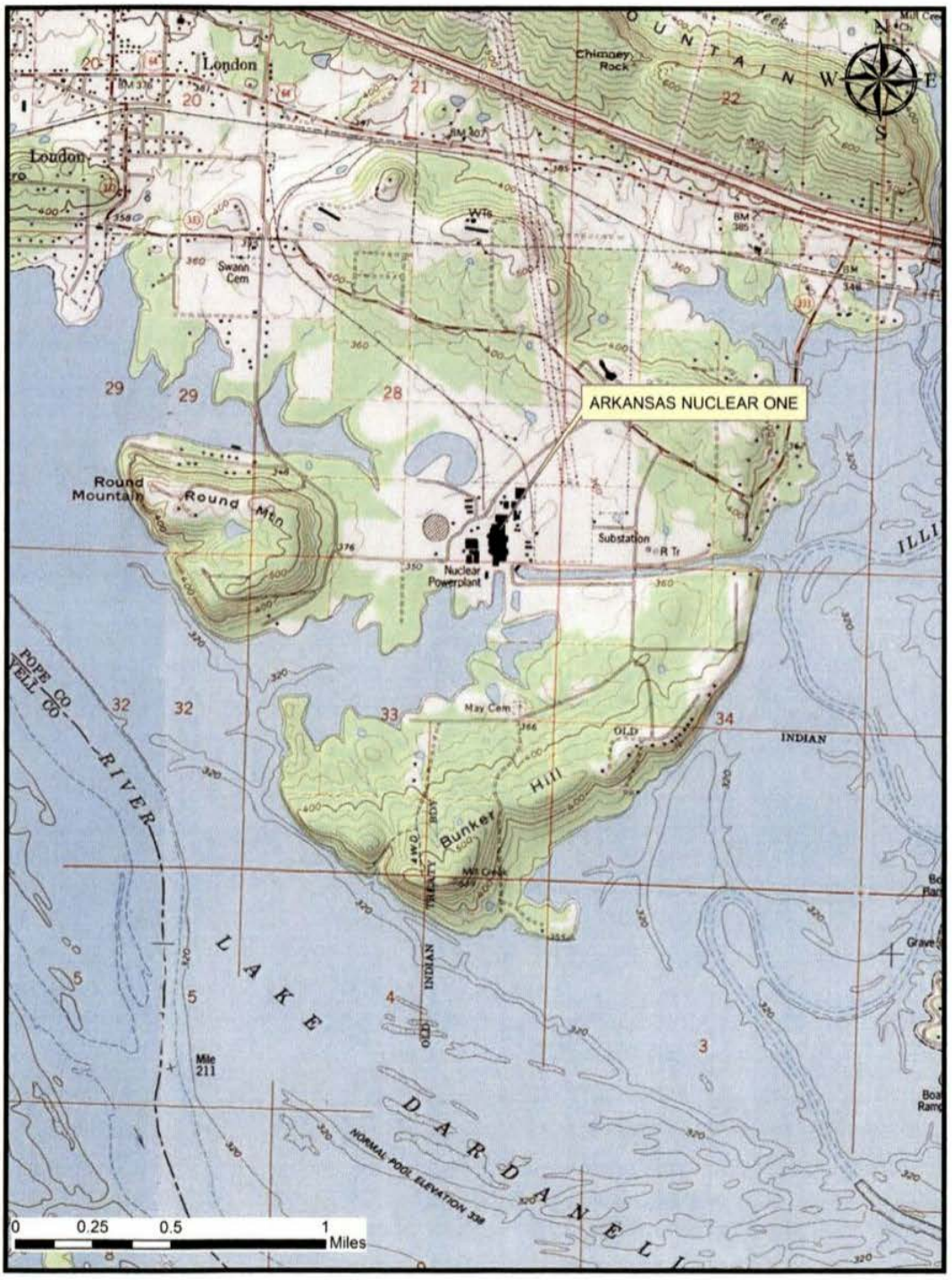
**ANO, 2012b.** Arkansas Nuclear One Unit 2 Flooding Walkdown Submittal; Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding, Engineering Report No. CALC-ANO2-CS-12-00002, See AREVA Document 38-9207373-000, 2012.

**ANO, 2016a.** Arkansas Nuclear One – Unit 1, Safety Analysis Report, Facility Operating License Number DPR-51, Docket Number 50-313, See AREVA Document No. 38-9257436-000, 2016.

**ANO, 2016b.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

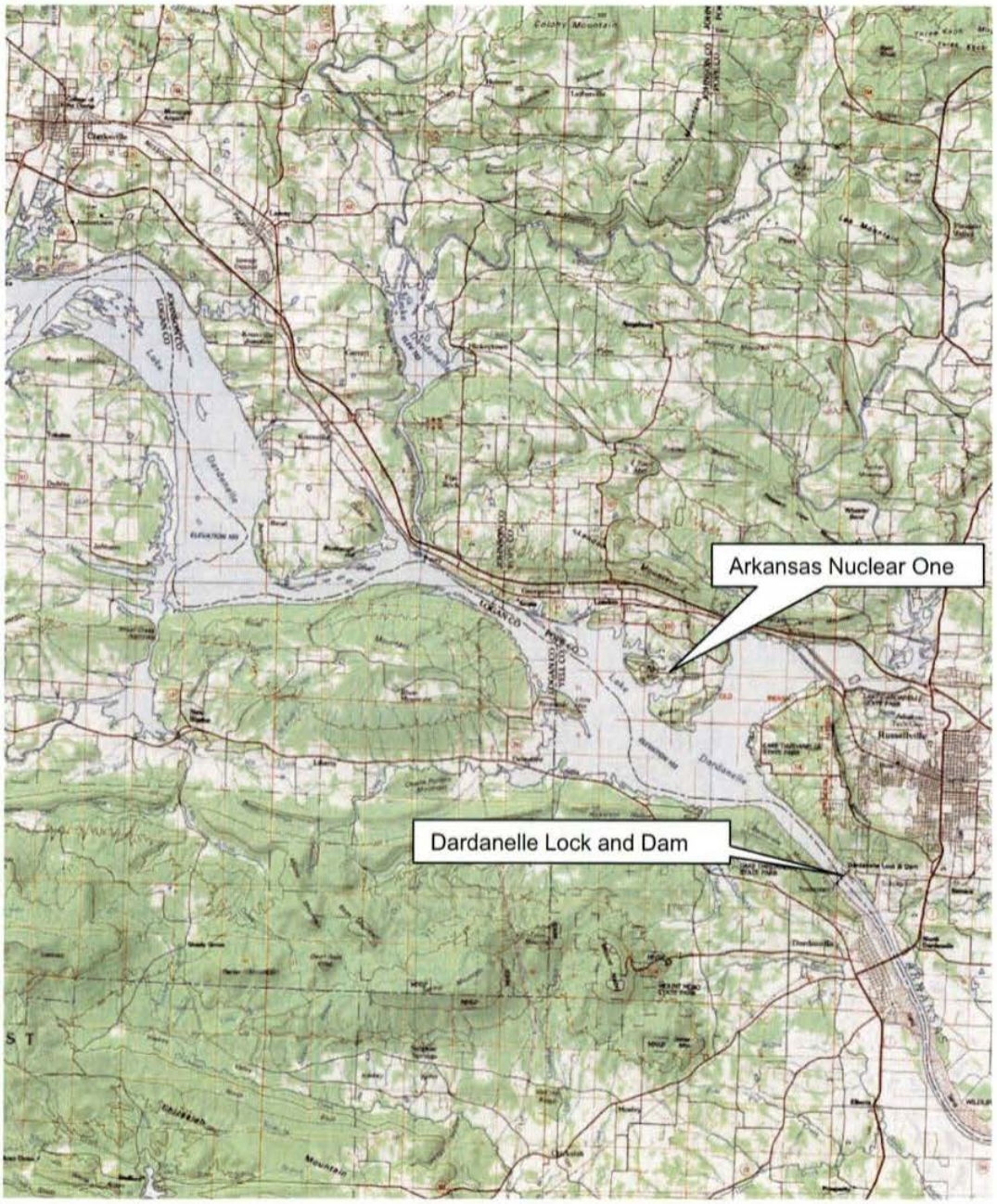
**AREVA, 2013.** Arkansas Nuclear One Topographic Survey Deliverables, 2013. See AREVA Document No. 38-9208201-001.

**Figure 2-1: Site Location Map #1**



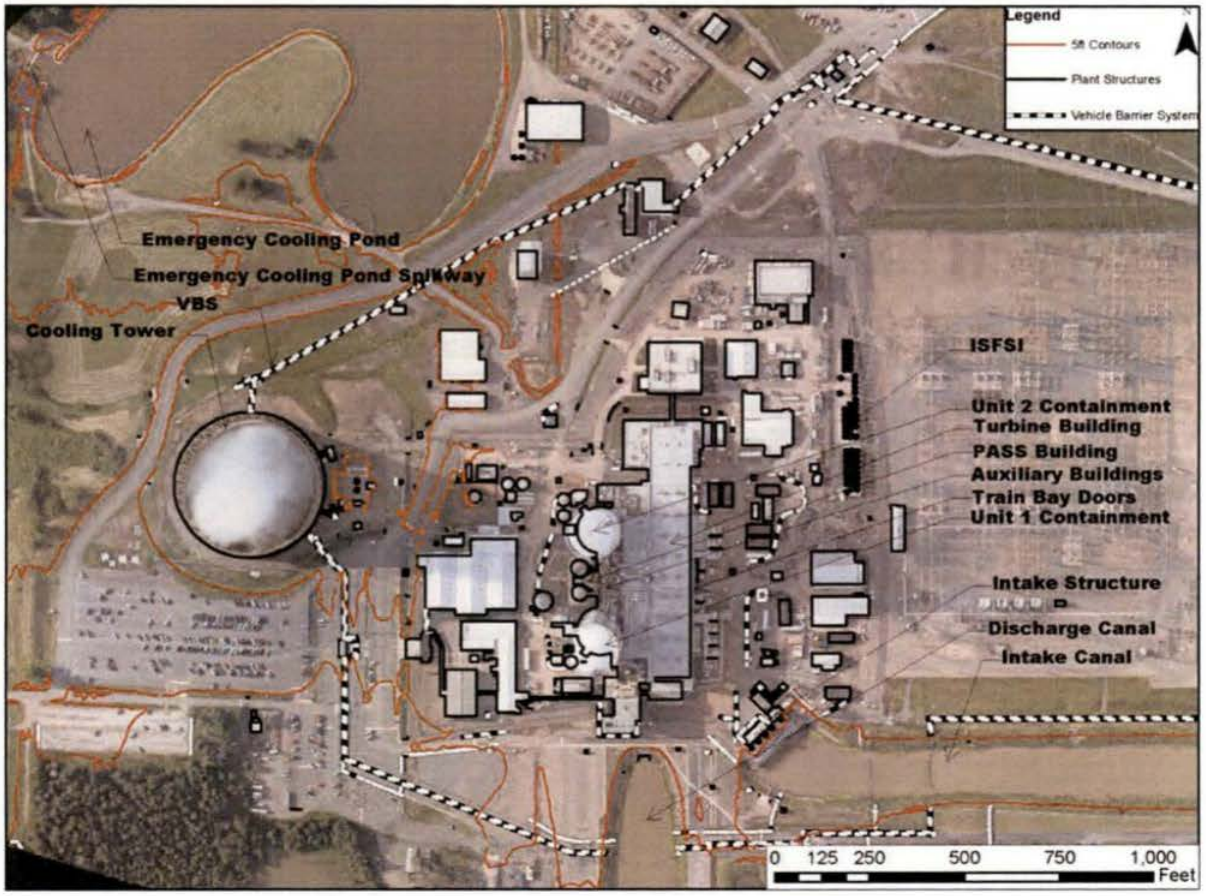

Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 2-2: Site Location Map #2**




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

**Figure 2-3: 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, 2013.

## 3.0 FLOOD HAZARD RE-EVALUATION

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

## 3.1 Local Intense Precipitation

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

This section summarizes the LIP and site-specific PMP evaluation performed in AREVA Calculation Nos. 32-9207382-000 (AREVA, 2016b) and 32-9207374-000 (AREVA, 2014).

### 3.1.1 Site Description

ANO is located on a peninsula formed by Lake Dardanelle (Pool No. 10, Dardanelle Lock and Dam), near river mile 210 of the Arkansas River, approximately 6 miles northwest of Russellville, Arkansas. The peninsula ANO is located on has high ground located to the west, from the north to the northeast, and from the southeast to the south. Round Mountain which has an elevation of 500 feet is located to the west of ANO. Several hills ranging from 400 to 500 feet in elevation are located from the north to the northeast of the site. Bunker Hill which has a maximum elevation of 520 feet is located from the southeast to the south of the site. Within the confines of the hills and Round Mountain the land slopes gently toward two major coves; one is on the west bank and one is a small cove of the Illinois Bayou on the east bank. The discharge canal is located on the southern side of the peninsula between Round Mountain and Bunker Hill. The intake canal is located on the eastern side of the peninsula just north of Bunker Hill. ANO is a two-unit facility. Unit 1 is located on the south side of the site and Unit 2 is adjacent to Unit 1, on the north side. The ANO site grade is typically at elevation 353 feet MSL and has natural drainage slopes toward Lake Dardanelle (ANO, 2016), with higher elevations (approximately 356 feet MSL) in the northern portion of the power block. The finished plant floor is at elevation 354 feet MSL.

Site drainage is normally accomplished through a system of catch basins, underground storm drains and surface drainage ditches. Surface drainage is constricted in some locations by the perimeter Vehicle Barrier System (VBS) which fully encompasses the site and is generally about 4 feet high. There is a gap in the VBS for the intake canal, the discharge canal, the northwest access road and several pedestrian openings on the south access road southeast of the intake building.

### 3.1.2 Method

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

The LIP calculation used the following steps:

1. Perform a site specific PMP study to calculate the one-hour, one-square mile and six-hour, ten-square mile PMP. The PMP values provided in the National Oceanic and Atmospheric Administration (NOAA) and U.S. Army Corps of Engineers (USACE) Hydrometeorological Report No. 51 (HMR-51) for ANO provide values starting at 6-hours and 10-square-miles. There are no explicit values provided at the 1- and 6-hour durations for 1-square-mile. Hydrometeorological Report No. 52 (HMR-52) provides information to derive the 1-hour 1- and 10- square-mile values based on the 6-hour 10-square-mile PMP in HMR-51. Unfortunately, the most recent storm evaluated in HMR-51 occurred in 1972. In addition, because HMR-51 and 52 cover large domains, generalization and conservatism were employed in the development of their respective PMP values that do not necessarily reflect the site-specific characteristics of ANO. Thus, an up-to-date site-specific study was performed.

2. Define FLO-2D model limits for LIP analysis.
3. Develop the FLO-2D computer model with site features.
4. Perform flood simulations in FLO-2D and estimate maximum water surface elevations at ANO.

NUREG/CR-7046 recommends that runoff losses be ignored during the LIP event to maximize runoff (NRC, 2011). Therefore, infiltration (i.e., constant loss) is conservatively not simulated in FLO-2D. Initial abstraction was also set to be zero. Time of concentration or lag time is not a direct input in FLO-2D (i.e. the time component of flow routing is computed internally in FLO-2D) because FLO-2D computes overland flow (i.e., routing) based on ground surface conditions, such as elevations and roughness coefficients, within FLO-2D (FLO-2D, 2013).

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

### 3.1.3 Results

#### 3.1.3.1 FLO-2D Model Development

FLO-2D is a physical process model that routes flood hydrographs and rainfall-runoff over unconfined flow surfaces or in channels using the dynamic wave approximation to the momentum equation (FLO-2D, 2013). The watershed applicable for the LIP analysis was computed internally within FLO-2D based on the digital terrain model (DTM) (AREVA, 2013) and the Digital Elevation Models (DEM) for Russellville West (USGS, 2009) input into FLO-2D (AREVA, 2013). The FLO-2D model includes topography, site location, and building structures. Grid elements along the model computational boundary were selected as outflow grid elements.

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

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

The FLO-2D computational area is approximately 811.5 acres. The selected grid element size for the model was 20 feet by 20 feet, which was selected based on the level of detail judged appropriate for the project. The final model computational boundary is shown in Figure 3.1-1. The FLO-2D model developed for the LIP analysis was based on ANO site features including: topography, site location, VBS layout, and structures (Figure 3.1-2).

Step 2: Assign Elevation Data to Grid Elements

The elevation data used to develop the FLO-2D model consist of DTM data which was extracted from an AutoCAD file (AREVA, 2013). Additional elevation data was used based on the topographic site plan (AREVA, 2013) produced along with the DTM. The topographic survey (AREVA, 2013) performed in 2013 at ANO met the American Society for Photogrammetry and Remote Sensing Class I Accuracy Standard for 1" = 100' planimetrics and 1-foot contour intervals, with +/- 1 feet horizontal accuracy, +/- 0.33 feet Root Mean Square Error (RMSE) vertical accuracy for 1 foot contours and +/- 0.17 feet RMSE vertical accuracy for spot elevations 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 +/- 0.1 feet. 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, 2013). This elevation data was supplemented with DEM for Russellville West (USGS, 2009). The final interpolated elevations for the grid elements are shown in Figure 3.1-3.

A minimum of two closest DTM points within the vicinity of a grid element was used in computing grid elevations. FLO-2D interpolated elevations for grid elements were spot checked for accuracy and modified as necessary based on site survey. Some FLO-2D ground interpolated elevations near locations with abrupt changes in elevations such as near the sloped sides of the intake canal and discharge canal were manually adjusted to conform with the site survey (AREVA, 2013).

Step 3: Define Surface Roughness Parameters

Manning's n-values used in FLO-2D are composite values that represent flow resistance. Grid element Manning's n-values were assigned based on land cover types at the site, and recommended Manning's roughness coefficients (i.e., Table 1 of FLO-2D, 2013 and Table 5-6 of Chow, 1959). Table 3.1-1 shows the relationship between Manning's n values and selected land cover categories. The Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete or paved areas to 0.40 for wooded areas.

FLO-2D uses the continuity and dynamic wave momentum equations in its computations. The friction slope component of the dynamic wave momentum equation as implemented in FLO-2D is based on Manning's equation (FLO-2D, 2013). Manning's roughness coefficients used in FLO-2D for overland flow accounts for surface roughness conditions (vegetation, concrete, etc.) in addition to non-uniform and unsteady flow conditions, which are the flow conditions being simulated. Manning's roughness coefficients are a function of flow depth (FLO-2D, 2013) with shallow flows (generally depths less than 0.5 feet) requiring higher Manning's roughness coefficients to account for increased resistance due to relatively greater surface roughness conditions. FLO-2D assumes a limiting depth for complete submergence of surface roughness to be 3 feet (FLO-2D, 2012). FLO-2D has the option of using depth variable Manning's roughness coefficients in its computations. When the depth variable roughness option is used, the following rules apply to the Manning's roughness coefficients used in computations (FLO-2D, 2012):

| | |
|---|---|
| *0.0 < flow depth < 0.2 ft (0.06 m)* | *n = SHALLOWN value* |
| *0.2 ft (0.06 m) < flow depth < 0.5 ft (0.15 m)* | *n = SHALLOWN / 2.* |
| *0.5 ft (0.15 m) < flow depth < 3 ft (1 m)* | $n = Specified\ n - value\ x\ 1.5\ x\ e^{-(\frac{0.4xDepth}{3})}$ |
| *3 ft (1 m) < flow depth* | *n = specified n-value* |

The variable roughness option was used in the FLO-2D model and a "SHALLOWN" value of 0.2 was used. Unlike the specified Manning's n values, the "SHALLOWN" is a global value and does not vary spatially (Chapter 4 of FLO-2D, 2012). The selected "SHALLOWN" value of 0.2 is a conservative representation of varying floodplain land cover types ranging from concrete to trees. The "SHALLOWN" value of 0.2 is also conservative because most of the site area (where shallow flow conditions are most likely to occur) is paved. The specified Manning's roughness coefficients were applicable for flow depths greater than 3 feet, indicating flow conditions where surface roughness is completely submerged (FLO-2D, 2012). It is noted that flood depths adjacent to the intake canal and along surface drainage ditches are well above 3 feet, occasionally exceeding 10 feet deep. Therefore, the specified Manning's roughness values used are judged to be appropriate for LIP-type flood analyses.

Figure 3.1-4 shows the Manning's coefficients selection for each land cover. The land cover type upon which the Manning's roughness coefficients assignments were made was based on the high resolution aerial imagery of the site, obtained as part of the ANO topographical survey (AREVA, 2013).

Step 4: Represent Building Rooftops and Other Flow Obstructions

Buildings at ANO were incorporated into the FLO-2D model based on the surveyed topographic site plan (AREVA, 2013) by manually adjusting (i.e., increasing) grid element elevations. Buildings were represented by grid elements with a ground elevation at least five feet higher than surrounding areas to ensure that runoff from the roofs freely flows to adjacent ground grid elements and flows around the building footprint (i.e. not through the building) see Figure 3.1-5. Building roof top elevations were assigned to represent the approximate runoff pattern (i.e., runoff flows from higher elevation roof top to lower elevation roof top), based on visual inspection of oblique aerial photographs (Bing, 2014). A relative differential height of 5 feet was used to separate adjacent building roof tops. The peak 1-hour duration LIP depth of 16.3 inches is less than the relative change in elevation of at least 5 feet. Therefore, water is not expected to build-up high enough to drive flow from rooftops with lower elevations to adjacent rooftops with higher elevations. This ensures that general flow directions of runoff from rooftops are considered.

Elevating grid elements to represent buildings has the potential to create an artificially high hydraulic gradient that may decrease flow depths and increase velocities at the intersection of building grid elements and grid elements representing the adjacent grade. Through examination of velocities in grid elements bordering the buildings this potential appears to have no appreciable effect on the results. However, review of the SUPER.OUT file indicates that there are several grid elements at intersection of building grid elements and grid elements representing the adjacent grade that have super critical flow. Runoff coming from the building roof discharges to paved areas where erosion is unlikely. None of the grid elements reporting supercritical flow appear to be located near critical locations.

Step 5: Define Initial Water Surface Elevations

The normal pool elevation of the Lake Dardanelle varies between 336 and 338 feet NGVD29, and is controlled by the downstream Dardanelle Lock and Dam No. 10 on the Arkansas River (ANO, 2016). The water surface elevation of Lake Dardanelle, the intake canal and the discharge canal was set to be the upper limit of the normal pool range with the elevation of Lake Dardanelle at 338 feet NGVD29 (ANO, 2016). The initial water surface elevation of the Emergency Cooling Pond was set to the normal pool elevation at 347.0 feet NGVD29 (ANO, 2010).

Step 6: Define "Levees" for Vehicle Barrier Systems

The FLO-2D model was first executed with no VBS (levee components) in the model. From examining the flow velocity vectors, portions of the VBS that would re-direct overland flow away from the site were conservatively not modeled and VBS that contained flow were modeled in the final FLO-2D model. The VBS at ANO was modeled in FLO-2D using the levee structures component of the model. 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 2.85 (FLO-2D, 2013). The top elevation of the modelled portions of the VBS was set at elevation 4 feet above the underlying grid element elevation (ANO, 2013b). The pedestrian opening on the south access road southeast of the intake building was modeled as an opening through the levee. The jersey barriers located immediately to the west of the intake structure were modeled using the same geometry as the VBS. The model layout including levees is as shown in Figure 3.1-2.

Step 7: Define "Channels" for Intake Canal

The intake canal was modeled using a one-dimensional channel segment in FLO-2D, as shown in Figure 3.1-2. As simplified channel cross sectional geometry was based an as-built channel cross section (ANO, 1987). The drainage area of the intake canal is a large potion the local watershed modeled in FLO-2D. The intake canal was therefore modeled using a channel to incorporate the conveyance capacity of the intake canal. Note the discharge canal was modeled as part of the FLO-2D grid, and not by using channel segments. The drainage area of the discharge canal is a small portion of the local watershed modeled in FLOD-2D. Modeling the discharge canal as part of the grid was judged to capture an adequate level of detail.

Step 8: Define "Hydraulic Structures"

The existing spillway at the emergency cooling pond and the covered walkway spanning from the courtyard between the Unit 1 Reactor Building and the Administration Building to south of the Unit 1 Reactor Building were incorporated into the FLO-2D model (see Figure 3.1-2) using rating tables. The covered walkway was modeled using a hydraulic structure as a method to convey runoff from the courtyard through the covered walkway, to reduce the potential of unrealistically high ponding in the courtyard. The rating curve for the spillway and the walkway were calculated based on structure geometry (ANO, 2016 and ANO, 2014) using the weir equation (USACE, 2010). The discharge coefficient for the Emergency Cooling Pond spillway varied with water surface elevation and was calculated based on formulas described in the Design of Small Dams (USBR, 1973).

Step 9: Estimate the maximum water surface elevation in the Emergency Cooling Pond

The maximum water surface elevation in the Emergency Cooling Pond was estimated using HEC-HMS. FLO-2D may not compute reasonable or accurate water surface elevations in ponded areas unless a very small time step is applied (FLO 2D, 2013). Therefore HEC-HMS was used to evaluate the water surface elevation in the Emergency Cooling Pond. Two HEC-HMS models were developed to evaluate the water surface elevation in the ECP. The first model was a simplified case developed using conservative inputs. Following the HHA approach the second model was developed using refined site-specific inputs to estimate a more realistic maximum ECP elevation during the PMF. The PMF at the cooling pond is a result of the LIP on its small contributory watershed (less than one-square mile). The 6-hour 10-square mile LIP (short duration, high intensity rainfall) was used to model the PMF on the ECP due to the small contributory watershed and short watershed lag time. The LIP was calculated based on a site-specific precipitation study, as described in Section 3.1.3.2 (AREVA, 2014). The maximum stillwater elevation on the Arkansas River (at ANO) during the PMF (including the effects of upstream dam failure) was calculated as (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (AREVA, 2016a), resulting in the ECP being submerged.

ArcMap was used to delineate the contributory watershed (0.23 square miles) to the cooling pond by identifying drainage divides (topographic high points) using the existing DEM for Russellville West (USGS, 2009).

The Soil Conservation Service (SCS) now known as the Natural Resources Conservation Service (NRCS), Unit Hydrograph method was used within HEC-HMS. For the simplified model, conservative values were used for lag time (6 minutes, i.e., nearly instantaneous) and curve number (99, i.e., impervious surface). For the refined model a 17 minute lag time was calculated as input for the SCS Unit Hydrograph method and a curve number of 87.2 was calculated to represent precipitation losses in the watershed. Nonlinearity adjustments were not used because the inputs are very conservative (i.e., the watershed is virtually impervious / no appreciable infiltration occurs and travel time is nearly instantaneous), are not derived from observed hydrographs and the watershed area is relatively small.

A reservoir element was used in the HEC-HMS model to account for cooling pond storage and discharge. The elevation-storage relationship was calculated based on a regression equation calculated during the

Emergency Cooling Pond spillway hydraulics evaluation (ANO, 2008). The starting elevation of the pond was set at 347 feet NGVD29 (ANO, 2010).

#### 3.1.3.2 Rainfall Inputs

A site specific PMP study was performed for ANO. The approach used in the study is storm-based using many of the procedures used in HMR-51 and HMR-52 (NOAA, 1978 and NOAA, 1982). The World Meteorological Organization (WMO) Manual for PMP determination (WMO, 2009) recommends this same approach.

The initial step in the development of the LIP values was to identify a set of storms which represent rainfall events that are LIP-type local storm events. This included storms where extreme heavy rainfall accumulated over short durations and small area sizes. These include observed rainfall amounts associated with Mesoscale Convective System (MCS) and individual thunderstorms. This included fourteen storms used in HMR-33 (USWB, 1956) and HMR-51 (NOAA, 1978), as well as more recent storms through April 2010.

The storm-based approach utilizes observed rainfall data from rainfall events which have occurred over the site and in regions where storms are considered to be transpositionable to the ANO site location. These rainfall data are maximized in-place following standard maximization procedures, then transpositioned to the ANO location. The transpositioning process accounts for differences in moisture and elevation between the original storm location and the ANO site. 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 the site had all factors leading to the rainfall been ideal and maximized.

This resulted in 23 events being evaluated for use in LIP calculations (Figure 3.1-6 and Table 3.1-2). Nine of the storms were not covered by the HMR-33, HMR-51 or the USACE Storm Studies analysis (USACE, 1973). These were analyzed using Storm Precipitation Analysis System (SPAS). 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 and holds merit, the DAD analysis process developed within the SPAS program attempts to mimic it as much as possible. By adopting this approach, some level of consistency between the newly analyzed storms and the hundreds of storms already analyzed can be achieved. Using SPAS to analyze the nine storms allowed for hourly rainfall to be evaluated with a spatial resolution of 1/3rd square mile. This provided data for the storm rainfall 1-hour 1-square mile area sizes to be explicitly evaluated.

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. Maximization is accomplished by increasing surface dew points to some climatological maximum and calculating the enhanced rainfall amounts that could potentially be produced. 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.

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 ANO. For a given storm event to be considered transpositionable, there must be similar meteorological / climatological and topographical characteristics at its original location versus the new location. The general guidelines described in HMR 51 Section 2.4.2 are followed in this analysis. The area considered to contain storms which were potentially

transpositionable extended from the Continental Divide of the Rocky Mountains south of 48°N east through the first upslopes on the west side of the Appalachians, south the southern Plains to approximately 50 miles north of the Gulf of Mexico. Further analysis indicated that storms that occurred within a +/- 1,000 feet of elevation were transpositionable.

After the maximization and transposition factors were calculated for each of the storms, the results were applied to the maximum 1-hour value for each storm to calculate the maximized 1-hour 1-square mile values. The largest of these values results in the site-specific LIP for the ANO site (see Table 3.1-2). After adjustments were applied, the Thrall, TX September 1921 storm had the highest 1-hour rainfall, with four other storms providing slightly smaller values and support for this value. Note that use of the Thrall, TX storm at the ANO site is beyond the transposition limits noted by the National Weather Service (NWS). Therefore, use of this storm at the site conservatively produces LIP values that are higher than would be calculated had Thrall, TX not been transpositioned. However, the transposition limits of the storm, as well as the meteorology which led to the estimated rainfall, were deemed similar enough during this analysis to allow it to be transpositioned.

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 ANO had all factors leading to the rainfall been ideal. 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 3.1-3. The 1-square-mile, 1-hour-duration PMP depth was calculated based on a site specific study as 16.3 inches.

The 1-square-mile, 1-hour duration PMP hyetograph used a front-loaded distribution with the most severe 5-minute and 1-hour duration PMP at the beginning of the 6-hour time series as per NUREG/CR-7046 (NRC, 2011), and is shown in Figure 3.1-7. The LIP also included the 10-square-mile, 6-hour duration PMP, which was calculated as part of the site specific study as 23.0 inches. The sub divisions of the 10-square-mile 6-hour PMP are calculated based on methodology of HMR-52 and are the same as the sub-divisions for the 1-square-mile 1-hour PMP for the first hour. The sub-divisions from the second hour to the sixth hour are based on recommendations in NUREG/CR-7046 (NRC, 2011). The 10-square-mile, 6-hour duration PMP hyetograph is shown in Figure 3.1-8.

Note that the cool-season PMP for ANO was not evaluated because the climatology of the local site watershed does not support a bounding or controlling cool-season PMF consisting of a cool-season PMP in combination with snowmelt.

#### 3.1.3.3 FLO-2D Model Simulation Results

Results of the ANO FLO-2D LIP run for representative locations (see Figure 3.1-9) are summarized in Table 3.1-4. The maximum LIP flood depths within the ANO power block area are shown in Figure 3.1-10. Flood depths in the power block area are generally less than 2 feet. Several areas of localized greater depths were identified near the power block area:

- Flood depths around the cooling tower range from 2.0 to 9.9 feet (north of the cooling tower). Flood depths in this area are relatively high due to the topography of the area combined with contribution of runoff from the cooling tower (modeled as a solid building for simplicity) and overland flow into the low drainage system area north of the cooling tower.

- Flood depths to the north and northeast of the intake structure generally range from 2.0 to 3.6 feet. Flood depths in this area are relatively high due to the contribution of runoff from the Unit 1 Turbine Building and smaller adjacent buildings into a slightly depressed area.
- Flood depths approximately 200 feet to the east of the Unit 2 Turbine Building range from 2.0 to 3.6 feet (see Figure 3.1-9). Flood depths in this area are relatively high due to due to runoff from adjacent impervious areas being conveyed into this locally lower area.
- Flood depths on the south bank of the intake canal near the intake structure range from 2.0 feet to 10.6 feet. Flood depths in this area are relatively high due to the contribution of runoff from the surrounding impervious area into the intake canal and overflow from the intake canal.
- Flood depths in the drainage ditches to the west of the Warehouse and Engineering / Modification Building (also known as the Systems Engineering Building) range 2.0 to 2.8 feet. Flood depth in this area are relatively high due to due to runoff from adjacent impervious areas being conveyed into these low lying ditches.
- Flood depths along the VBS southwest of the Warehouse and Engineering / Modification Building (also known as the Systems Engineering Building) range from 2.0 to 4.7 feet. Flood depths in this area are relatively high due to runoff from the surrounding impervious area ponding against the VBS.
- Flood depths to the northwest of the diesel oil storage tank range from 2.0 to 2.7 feet. Flood depths in this area are relatively high due to runoff from the surrounding impervious area and from the roadway into this locally lower area.
- Flood depths in the drainage ditches to the west of the Central Support Building (CSB) and Maintenance Facility range from 2.0 to 2.5 feet. Flood depths in this area are relatively high due to runoff from the surrounding impervious areas conveyed into these low lying ditches.
- Flood depths within the drainage ditches along the west and northwest portion of the power block range from 2.0 to 9.9 feet. Flood depths in these areas are relatively high due to runoff being conveyed into these low lying ditches.
- Flood depths near the Turbine Building Train Bay Doors are 1.2 feet, with a peak flood elevation of 354.4 ft NGVD29.

The maximum LIP flood elevations near plant structures that were modeled generally range from 351.1 feet (NGVD29) along the south side of the warehouse building to 357.5 feet (NGVD29) along the north side of the CSB, see Figure 3.1-11.

Maximum velocities near plant structures range from 0.03 feet per second (fps) to 4.5 fps, see Figure 3.1-12. Locally higher velocities (7.0 to 9.6 fps) occurred along the southwest side of the intake structure, likely due to runoff being constricted along the side of the building and into the confluence with the intake canal. These locally higher velocities are occurring within a paved area. The permissible velocity for rough asphalt is 12 feet per second (USACE, 1984). Erosion in this area is not anticipated because the calculated velocity is lower than the USACE permissible velocity. Maximum velocities throughout the model domain range from 0.01 fps to 12.1 fps, with the highest velocities (higher than 10 fps) occurring in several localized areas outside of the power block.

The maximum LIP flood elevations in the Independent Spent Fuel Storage Installation (ISFSI) expansion area (located to the north of the CSB) generally ranges from 356.2 feet NGVD29 in the southwest corner

to 358.2 feet NGVD29 in the northern corner. Maximum velocities range from 0.2 feet per second (fps) to 2.2 fps, with the highest velocity occurring in the southwest corner.

The SUPER.OUT file reports grid elements that are supercritical and indicated that supercritical flow may occur at several locations near plant structures as displayed in Figure 3.1-13 and described as follows:

- At intersection of building grid elements and grid elements representing the adjacent grade possibly due to the artificially high hydraulic gradient created by elevating grid elements to represent buildings. According to FLO-2D results (Figure 3.1-13), runoff coming from building roofs discharges to either lower building roofs or onto paved areas where erosion is unlikely. None of the grid elements reporting supercritical flow appear to be located near critical locations. Building grid elements may not appear centered on the buildings relative to aerial photography due to distortion and displacement of aerial photography caused by the angle of the image acquisition. Additionally, the building grid elements consist of 20 ft by 20 ft grid cells, and as such may not conform exactly to actual building shapes.
- South of the intake structure along the steep overbank slope of the intake channel. The intake channel is armored with riprap in this area making erosion unlikely.
- Near the discharge structure along the steep overbank slope of the discharge channel. The discharge channel is armored with riprap in this area making erosion unlikely.
- Localized area south of the Eng. / Mod. Building along the paved sloped parking area. Erosion that might occur in this area is unlikely to affect critical structures as it is not located near any critical structures.

The FLO-2D model results at grid elements reporting super critical flow that are not adjacent to buildings are considered conservative. The FLO-2D model results in conservative estimates for flow depth, because supercritical is shallower, and the program limits supercritical flow by reducing the velocity which increases the flow depth. Note that FLO-2D identified supercritical flow on one reporting grid location (grid number 26902). Therefore, the maximum flood depth and water surface elevation at this location is conservative.

PMF Calculation for Emergency Cooling Pond

The PMF peak water surface elevation in the cooling pond was calculated to be 351.5 feet NGVD29 and 350.9 feet NGVD29 for the simplified and refined models, respectively. The PMF peak water surface elevation is 2.5 feet and 3.1 feet below the top of the cooling pond impounding dike embankment for the simplified and refined models, respectively. The ECP is not expected to overtop during the local ECP PMF.

### 3.1.4 Conclusions

The maximum water surface elevations at all locations on the site due to the LIP at ANO result from a PMP depth of 16.3 inches in 1 hour and 23.0 inches within 6 hours. The maximum flood depths in the power block area range from 0.1 feet to as high as approximately 2.0 feet above grade as shown in Table 3.1-4. Time-series plots resulting from the LIP for specific locations are provided in Appendix A.

### 3.1.5 References

**ANO, 1987**. "Drawing Number C-35, Intake Canal Plan, Profile & Details, Revision 8", March 11, 1987. See AREVA Document No. 38- 9207373-000.

**ANO, 2008**. "Emergency Cooling Pond Spillway Replacement", EC-443, July 25, 2008. See AREVA Document No. 38- 9257436-000.

**ANO, 2010**. "Drawing Number C-65, Emergency Cooling Reservoir, Sheet 1, Revision 12", March 9, 2010, See AREVA Document No. 38- 9207373-000.

**ANO, 2013**. "EC #2 Security Upgrade Yard Modifications to Meet 73.55", Entergy Nuclear, 2013, See AREVA Document No. 38-9207373-000.

**ANO, 2016.** Arkansas Nuclear One – Unit 1, Safety Analysis Report, Facility Operating License Number DPR-51, Docket Number 50-313, See AREVA Document No. 38-9257436-000, 2016.

**AREVA, 2013**. "Arkansas Nuclear One Topographic Survey Deliverables", 2013, See AREVA Document No. 38-9208201-001.

**AREVA, 2014**. "Arkansas Nuclear One Flooding Hazard Re-Evaluation - Probable Maximum Precipitation", Revision 0, 2014, See AREVA Document No. 32-9207374-000.

**AREVA, 2016a**. "ANO Combined Effect Flood Analysis Calculation", Revision 0, 2016, See AREVA Document No. 32-9207388-000.

**AREVA, 2016b**. "Arkansas Nuclear One Flooding Hazard Re-Evaluation – Local Intense Precipitation – Generated Flood Flow and Elevations", Revision 0, 2016, See AREVA Document No. 32-9207382-000.

**Bing, 2014**. "Pictometry Bird's Eye", Pictometry International Corp, Microsoft Corporation, http://www.bing.com/maps/, Date Accessed: July 18, 2014, Date Modified: 2012.

**Chow, 1959**. "Open-Channel Hydraulics", Ven Te Chow, Reprint of the 1959 Edition.

**FLO-2D, 2012**. FLO-2D Professional Version Data Input Manual, FLO-2D Software Inc., Nutrioso, Arizona, 2012.

**FLO-2D, 2013**. FLO-2D Professional Version Reference Manual, FLO-2D Software Inc., Nutrioso, Arizona, 2013.

**NGS, 2013**. "VERTCON – North American Vertical Datum Conversion", National Geodetic Survey, http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html, Date accessed: December 3, 2013, Date Modified: January 24, 2013.

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

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

**NOAA, 1988.** "Probable Maximum Precipitation Estimates - United States Between the Continental Divide and the 103rd Meridian", Hydrometeorological Report No.55A (HMR-55A) by US Department of Commerce National Oceanic and Atmospheric Administration & U.S. Army Corps of Engineers, June 1988.

**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, November 2011.

**USACE, 1973**. US Army Corps of engineers, Storm Rainfall in the United States, Depth-Area-Duration Data, 1936-1973.

**USACE, 1984**. Drainage and Erosion Control Mobilization Construction", U.S. Army Corps of Engineers, EM 1110-3-136, April 1984.

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

**USACE, 2013b**. "Project Data", United States Army Corps of Engineers Little Rock District Water Management, http://www.swl-wc.usace.army.mil/pages/Ark_riv_project_data.htm, Date accessed: March 7, 2014, Date modified: December 2013.

**USBR, 1973**. "Design of Small Dam", United States Department of the Interior Bureau of Reclamation, 1973.

**USGS, 2009.** "1/3-Arc Second National Elevation Dataset, Russellville West, AR", U.S. Geological Survey, http://nationalmap.gov, Date Accessed: August 2013, Date Modified: 2009.

**USWB, 1956.** US Weather Bureau, 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 Number 33*, 1956.

**WHO, 2009.** World Meteorological Organization, Manual for Estimation of Probable Maximum Precipitation, Operational Hydrology Report No 1045, 2009.

**Table 3.1-1: Manning's n Values for Selected Land Use Categories**

| Land Use Category | Manning's n |
|---|---|
| Paved / Concrete | 0.02 |
| Grass | 0.20 |
| Trees (Shrubs and forest litter) | 0.40 |
| Water | 0.025 |

**Table 3.1-2: Storms used in the 1-hour 1-square mile Local Intense Precipitation analysis**

| Storm Name | State | AWA Storm Number | Lat | Lon | Year | Month | Day | Max Rainfall | Precipitation Source | ANO Total Adjustment Factor | ANO 1-hour $1mi^2$ PMP |
|---|---|---|---|---|---|---|---|---|---|---|---|
| WARNER PARK | TN | 2 | 36.0611 | -86.9056 | 2010 | 4 | 30 | 19.71 | SPAS 1208 | 1.18 | 5.40 |
| LARTO LAKE | LA | 4 | 31.220 | -92.130 | 2008 | 9 | 1 | 23.31 | SPAS 1182 | 1.15 | 7.07 |
| FALL RIVER | KS | 5 | 37.6500 | -96.0500 | 2007 | 6 | 30 | 25.50 | SPAS 1228 | 1.23 | 5.76 |
| ALBANY | TX | 18 | 32.7260 | -99.3500 | 1978 | 8 | 3 | 32.50 | SPAS 1179 | 1.26 | 15.36 |
| ENID | OK | 21 | 36.3805 | -97.8583 | 1973 | 10 | 10 | 19.45 | SPAS 1034 | 1.20 | 8.88 |
| WOOSTER | OH | 24 | 40.9146 | -81.9729 | 1969 | 7 | 4 | 14.95 | SPAS 1209 | 1.30 | 6.01 |
| GLADEWATER | TX | 25 | 32.5365 | -94.9427 | 1966 | 4 | 27 | 25.33 | SPAS 1181 | 1.24 | 3.62 |
| EDGERTON | MO | 26 | 40.4125 | -95.5125 | 1965 | 7 | 18 | 20.76 | SPAS 1183 | 1.27 | 4.67 |
| COLLEGE HILL | OH | 30 | 40.0854 | -81.6479 | 1963 | 6 | 3 | 19.39 | SPAS 1226 | 1.84 | 4.67 |
| CAMP POLK | LA | 81 | 31.067 | -93.200 | 1955 | 4 | 23 | 21.10 | LMV 5-3 | 0.97 | 4.97 |
| HARRISONBURG DAM | LA | 79 | 31.767 | -91.817 | 1953 | 5 | 11 | 25.40 | LMV 5-4 | 1.02 | 6.01 |
| KELSO | MO | 37 | 37.1906 | -89.5495 | 1952 | 8 | 11 | 15.09 | UMV 3-30 | 1.27 | 10.57 |
| HOLT | MO | 40 | 39.4528 | -94.3422 | 1947 | 6 | 18 | 17.60 | MR 8-20 | 1.18 | 14.16 |
| COLLINSVILLE | IL | 42 | 38.6717 | -89.9800 | 1946 | 8 | 12 | 18.70 | MR 7-2B | 1.19 | 4.57 |
| MOUNDS | OK | 44 | 35.8770 | -96.0610 | 1943 | 5 | 16 | 17.00 | SW 2-21 | 1.39 | 14.14 |
| SILVER LAKE | TX | 45 | 32.6700 | -95.5950 | 1943 | 6 | 5 | 16.50 | SW 3-3 | 1.15 | 10.45 |
| HALLETT | OK | 52 | 36.2000 | -96.6000 | 1940 | 9 | 2 | 24.00 | SW 2-18 | 1.17 | 13.78 |
| ENGLE | TX | 83 | 29.681 | -97.009 | 1940 | 6 | 29 | 22.70 | GM 5-11 | 1.22 | 8.59 |
| BEBE | TX | 84 | 29.332 | -97.652 | 1936 | 6 | 30 | 21.00 | GM 5-6 | 1.11 | 9.95 |
| NEOSHO FALLS | KS | 61 | 38.0820 | -95.7010 | 1926 | 9 | 12 | 14.00 | SW 2-1 | 1.34 | 11.49 |
| THRALL | TX | 77 | 30.591 | -97.297 | 1921 | 9 | 9 | 39.70 | GM 4-12 | 1.14 | 16.34 |
| BONAPARTE | IA | 70 | 40.7667 | -91.7500 | 1905 | 6 | 10 | 12.10 | UMV 2-5 | 1.29 | 8.26 |

**Table 3.1-3: Point (1 square mile) Probable maximum Precipitation Depths**

| Time (min) | PMP Depth (in) |
|---|---|
| 60 | 16.3 |
| 30 | 12.2 |
| 15 | 8.4 |
| 5 | 5.4 |

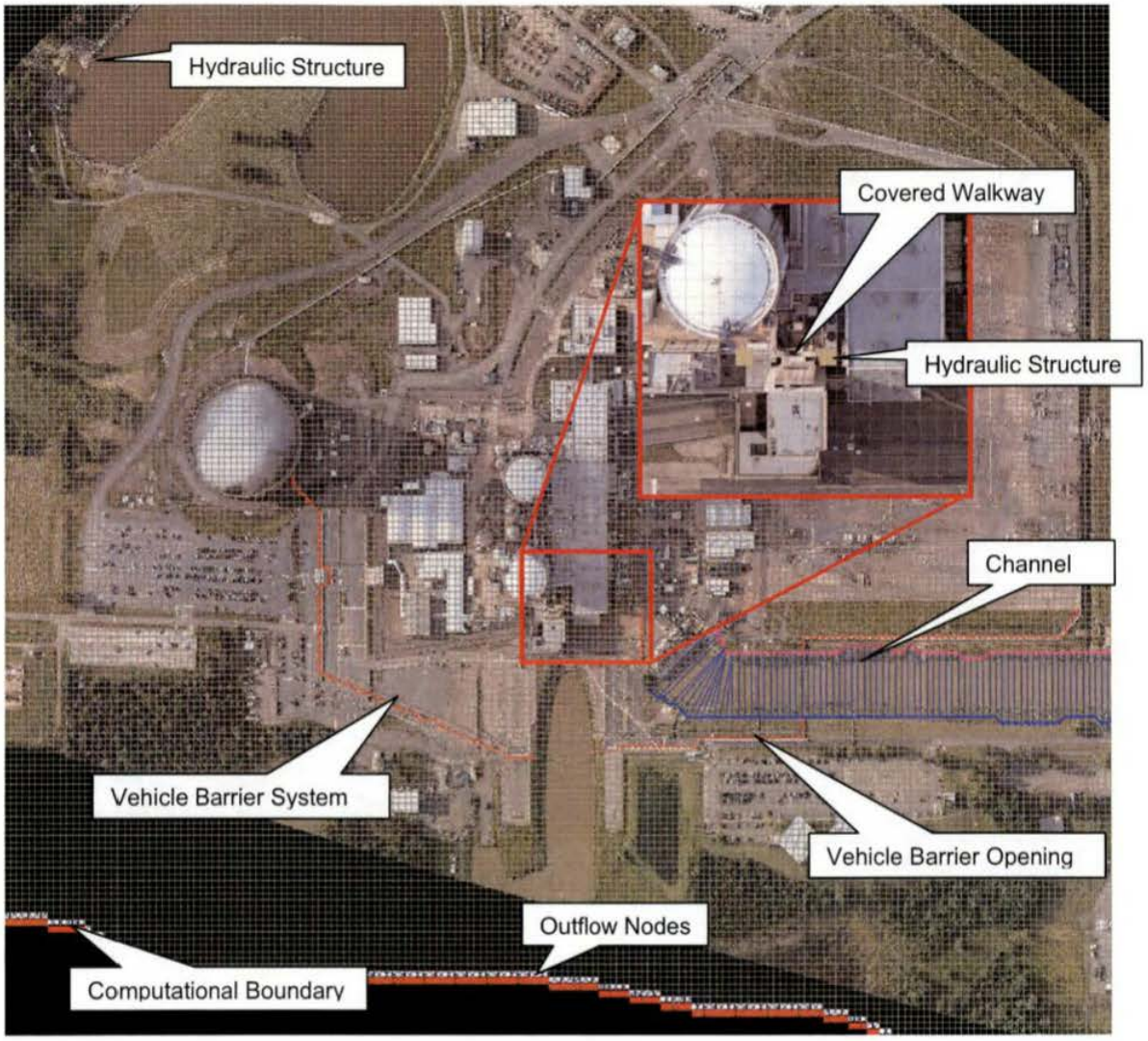
**Table 3.1-4: FLO-2D Results at General Locations**

| General Location | Grid Element Number | Maximum Water Surface Elevation (feet, NGVD29) | Maximum Water Depth (feet) |
|---|---|---|---|
| East of Cooling Tower | 19763 | 351.4 | 1.7 |
| West of Warehouse | 21787 | 351.2 | 1.5 |
| South of Warehouse | 23311 | 351.4 | 0.3 |
| West of Diesel Oil Storage Tank | 23326 | 354.5 | 0.1 |
| West of Engineering / Modification Building | 24918 | 352.2 | 0.6 |
| Between Engineering / Modification Building and Reactor Building Unit 1 | 26902 | 352.7 | 0.7 |
| East of Diesel Fuel Storage Vault | 26918 | 353.7 | 0.2 |
| Between Warehouse and Reactor Building Unit 2 | 27912 | 355.0 | 0.2 |
| West of Maintenance Building | 30261 | 353.7 | 1.3 |
| North of Turbine Building Unit 2 | 31260 | 353.7 | 0.9 |
| South of Turbine Building Unit 2 | 32903 | 355.1 | 0.1 |
| South of Central support Building | 33281 | 354.0 | 1.1 |
| North of Central Support Building | 33292 | 357.7 | 0.1 |
| **North Train Bay Door** | **33930** | **354.4** | **1.2** |
| **South Train Bay Door** | **33931** | **354.4** | **1.2** |
| Northeast of Turbine Building Unit 2 | 34620 | 354.4 | 1.5 |
| Transformer Yard | 35288 | 354.4 | 1.0 |
| East of Turbine Building Unit 1 | 35610 | 354.3 | 1.2 |
| Northwest of Intake Structure | 36273 | 354.1 | 1.7 |
| North of Intake Structure | 37289 | 354.2 | 2.0 |
| North of Independent Spent Fuel Storage Installation | 40664 | 356.3 | 0.4 |
| South of Independent Spent Fuel Storage Installation | 40968 | 355.6 | 0.1 |

**Figure 3.1-1: FLO-2D Computational Boundary**

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

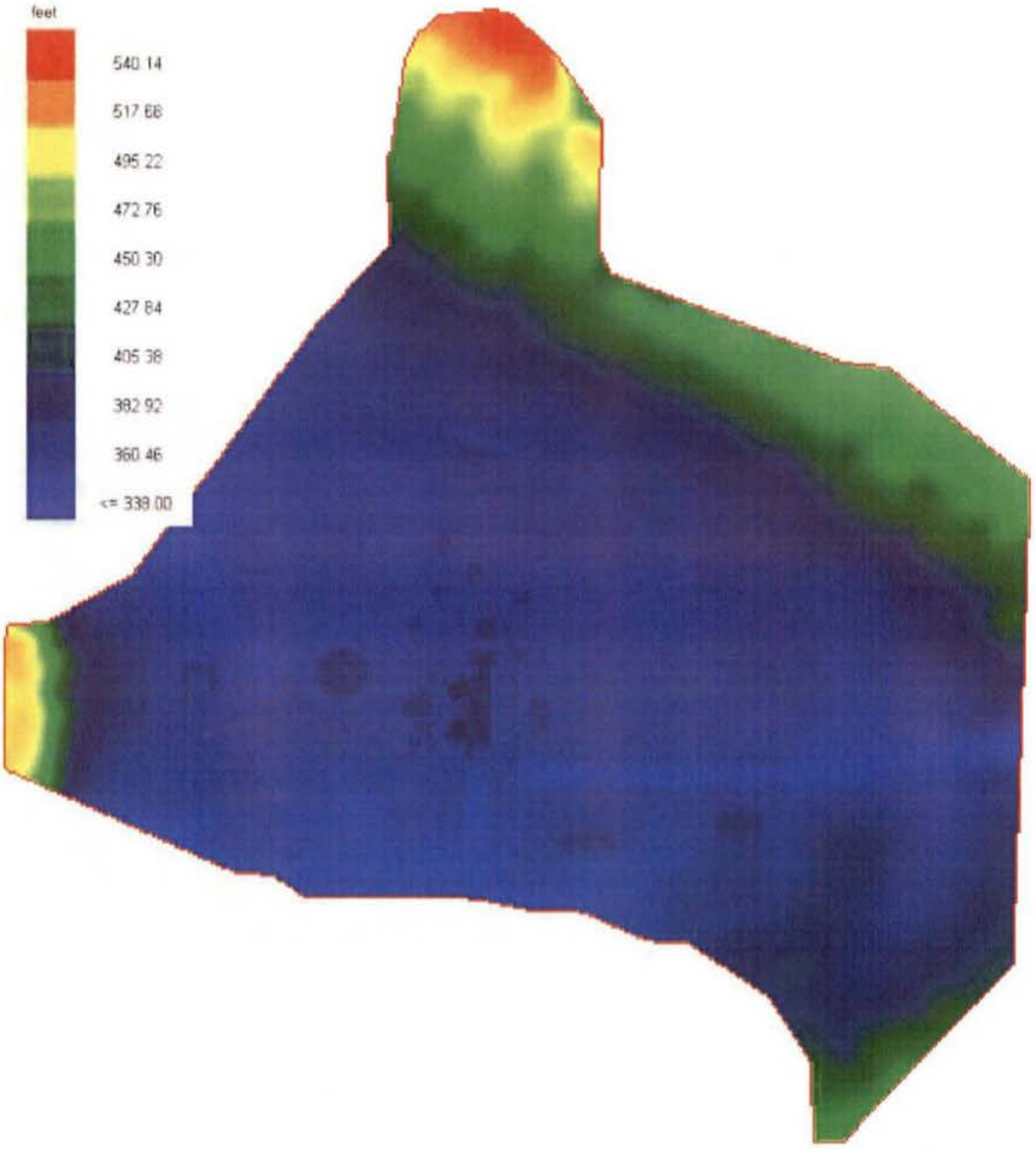
**Figure 3.1-2: FLO-2D Model Layout**




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

Note: VBS that would re-direct overland flow away from the site were conservatively not modeled.

**Figure 3.1-3: Grid Element Elevation Rendering (feet, NGVD29)**




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

**Figure 3.1-4: Grid Element Elevation Rendering – Powerblock Area (feet, NGVD29)**

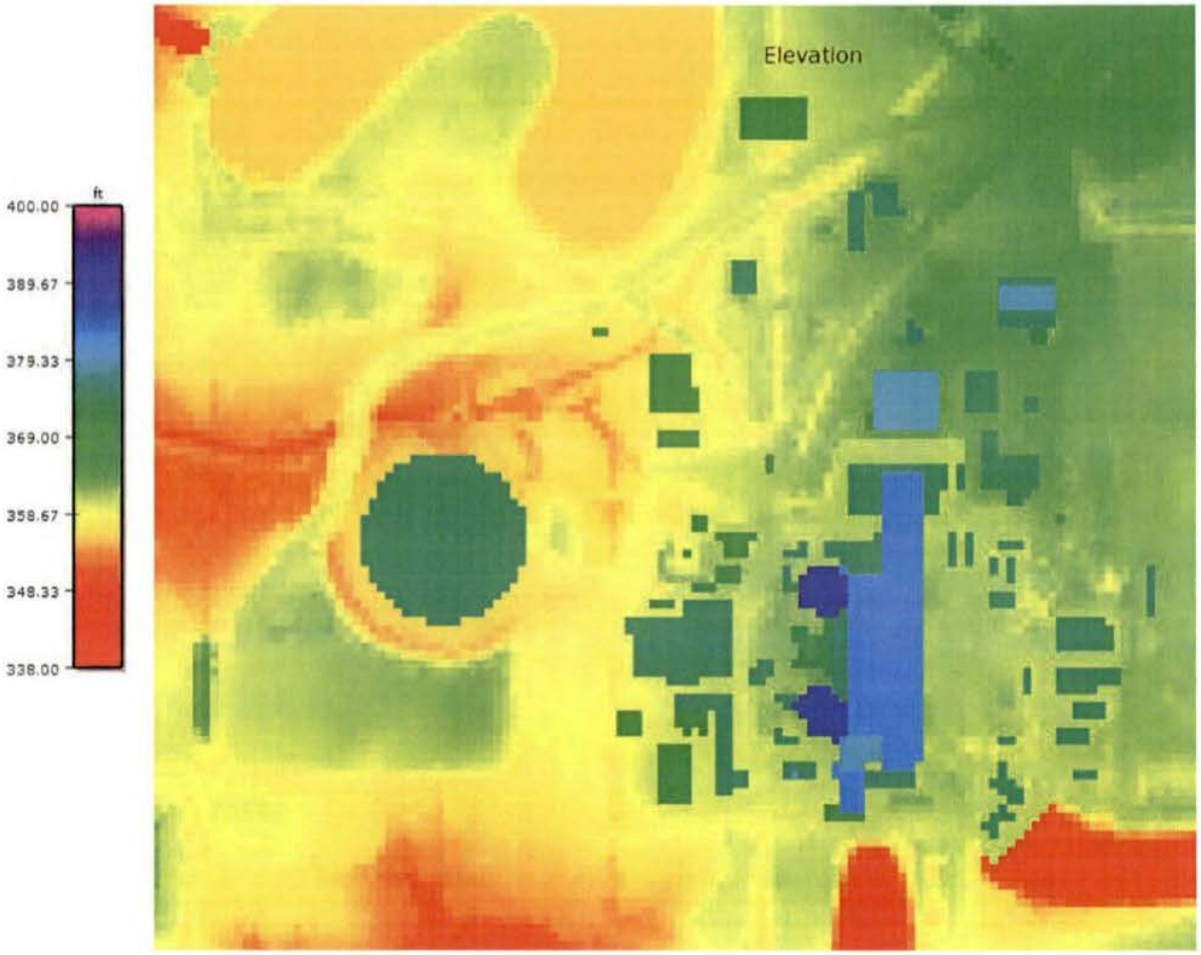
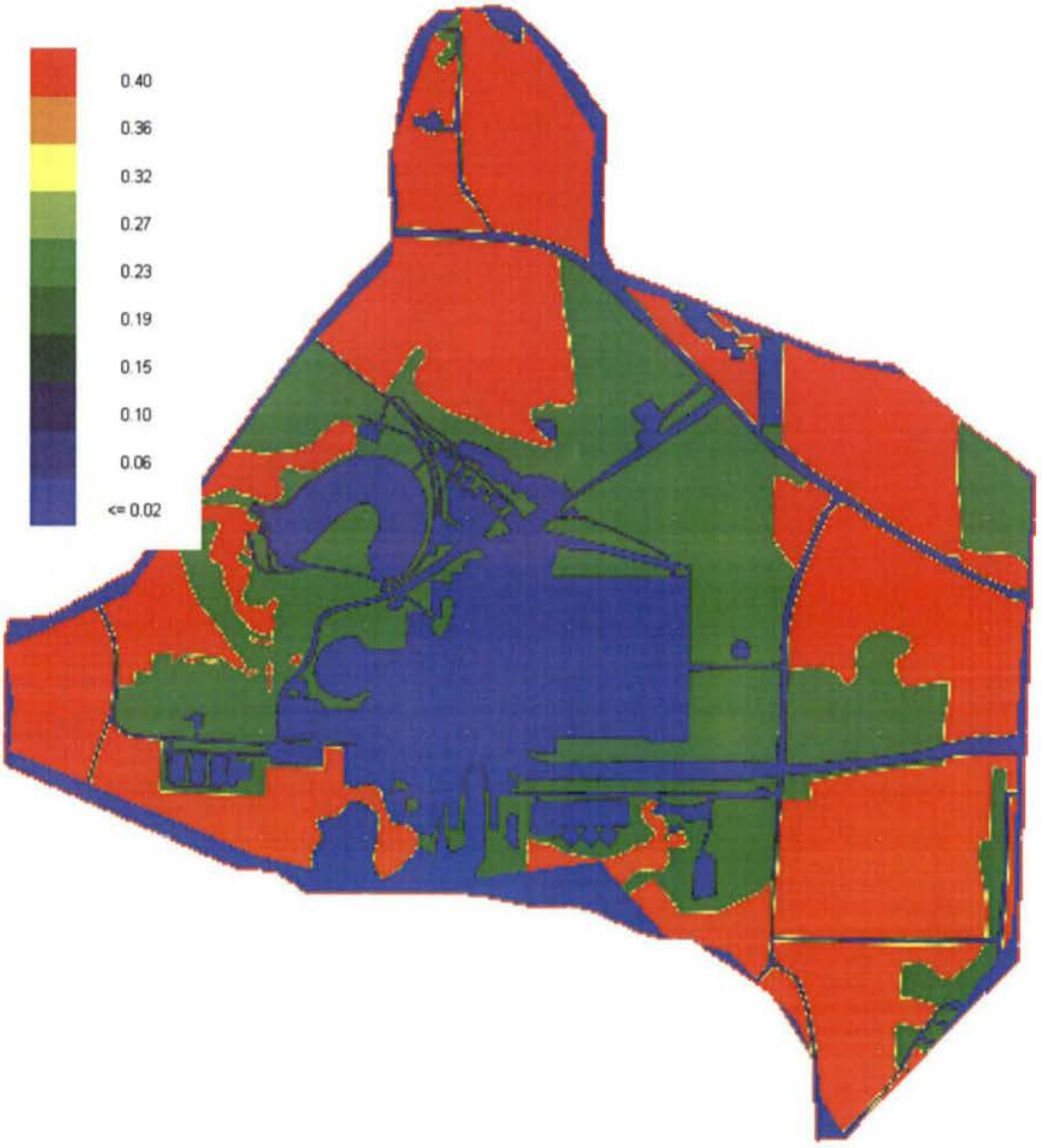


**Figure 3.1-5: FLO-2D Grid Element Manning's Coefficient Rendering**



## Figure 3.1-6: Storms used in the LIP calculations

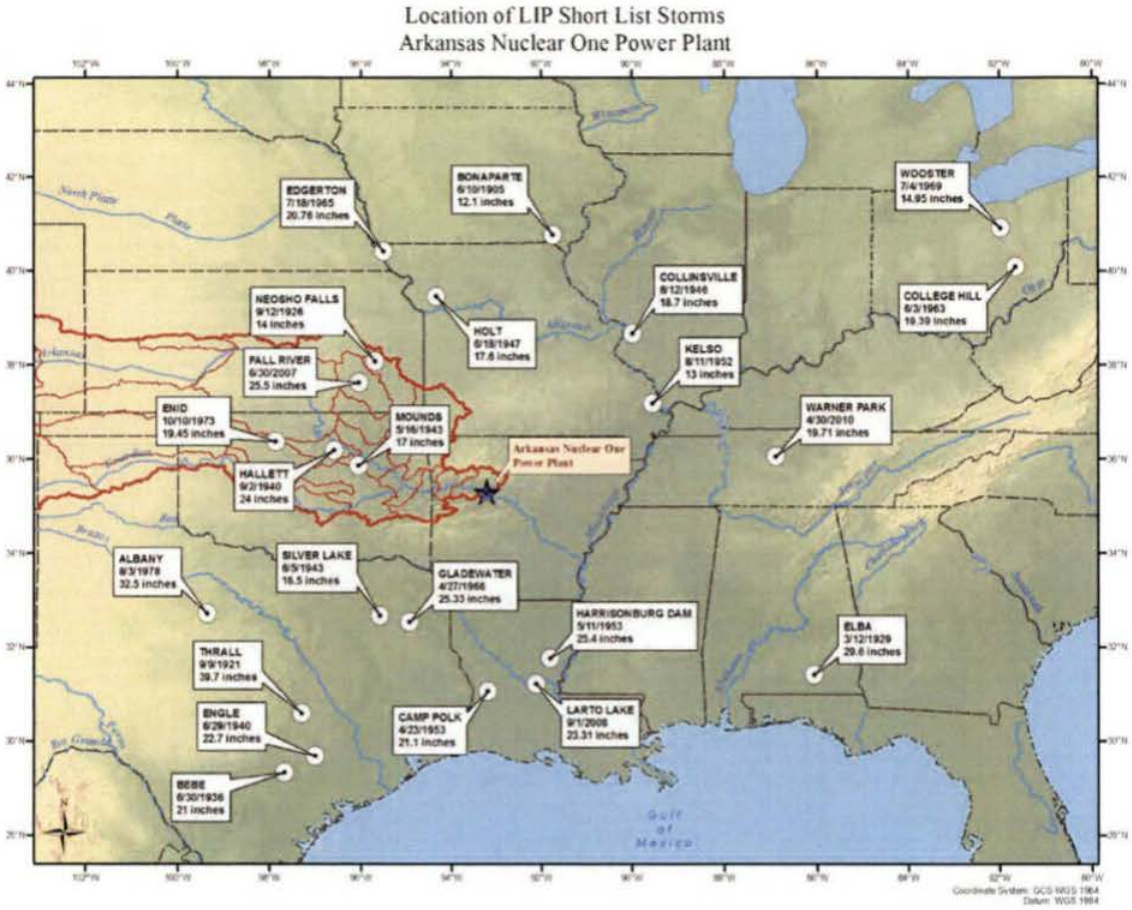


**Figure 3.1-7: 1-hour PMP – Incremental Hyetograph**

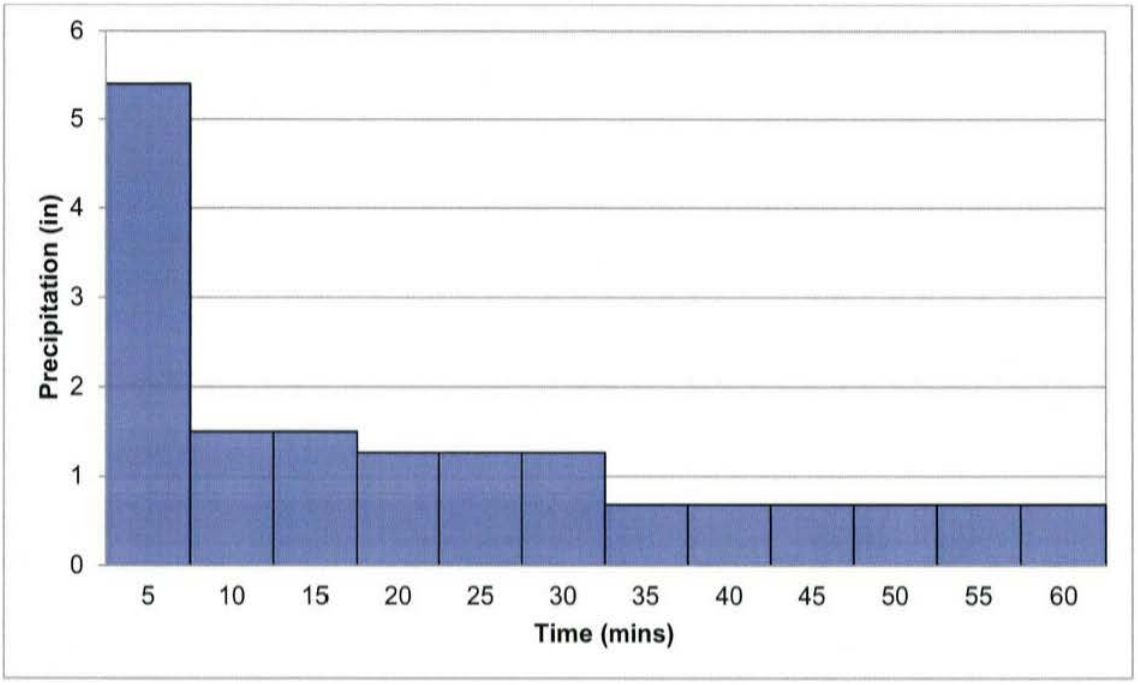



**Figure 3.1-8: 6-hour PMP – Incremental Hyetograph**

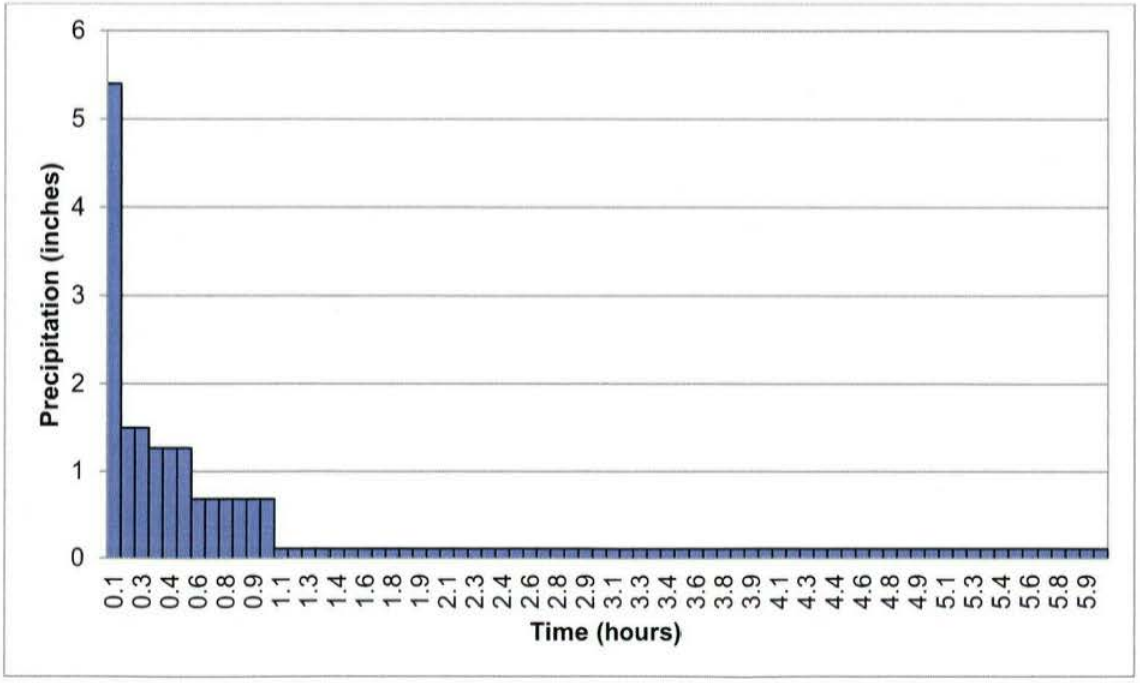
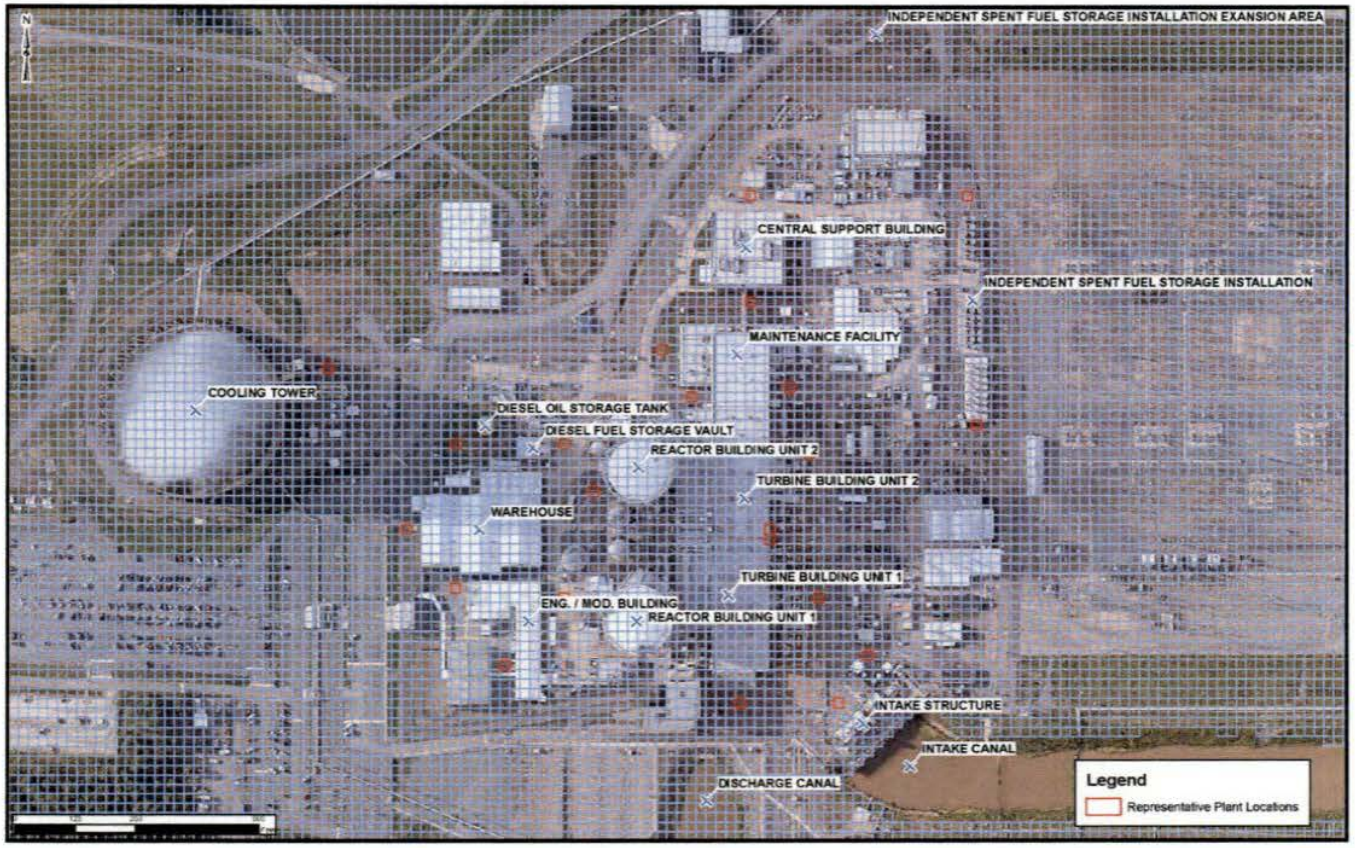


**Figure 3.1-9: Representative Locations for Reporting Results**

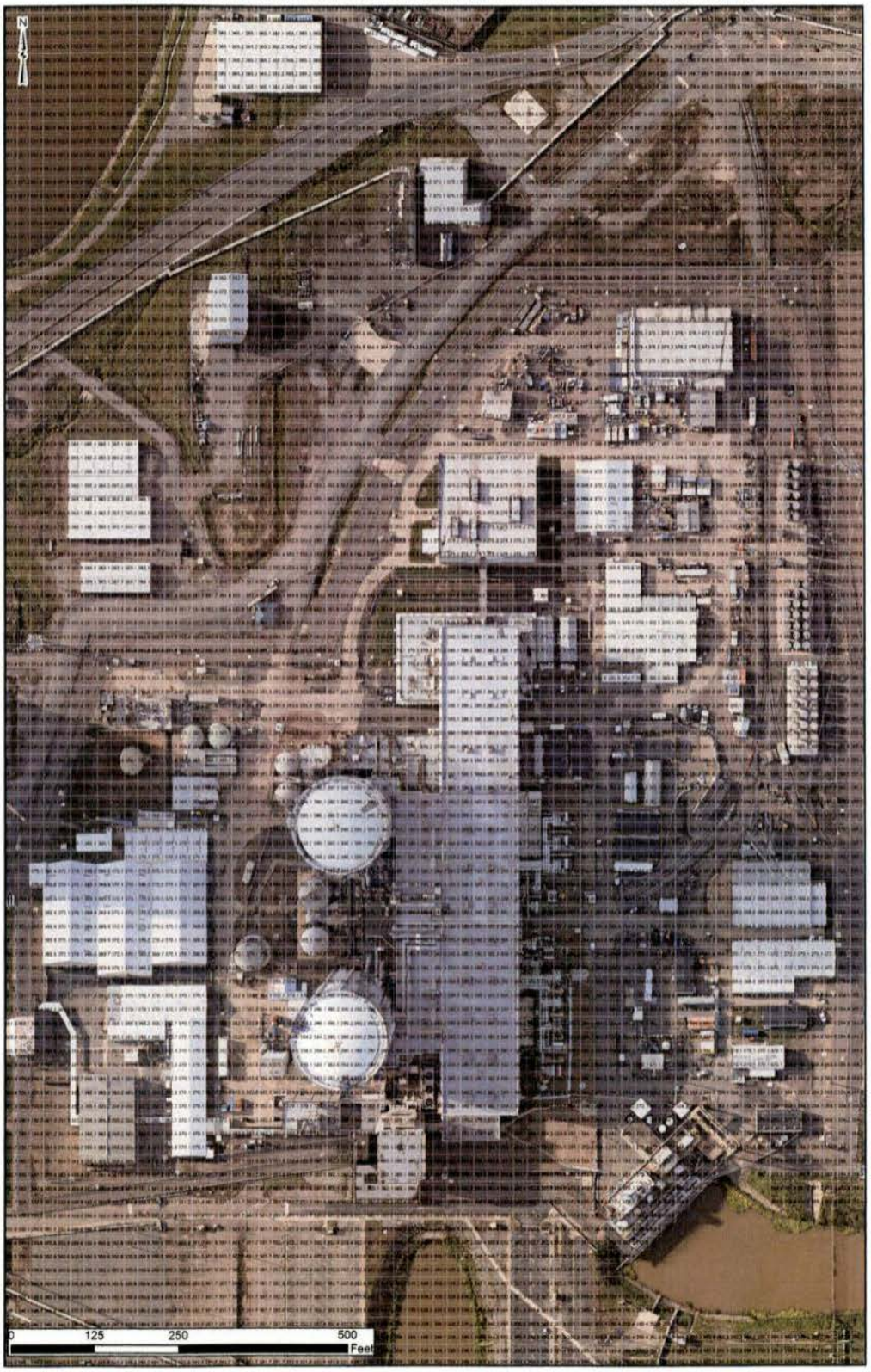



Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3.1-10: Maximum Flow Depth (feet) within Power Block Area**

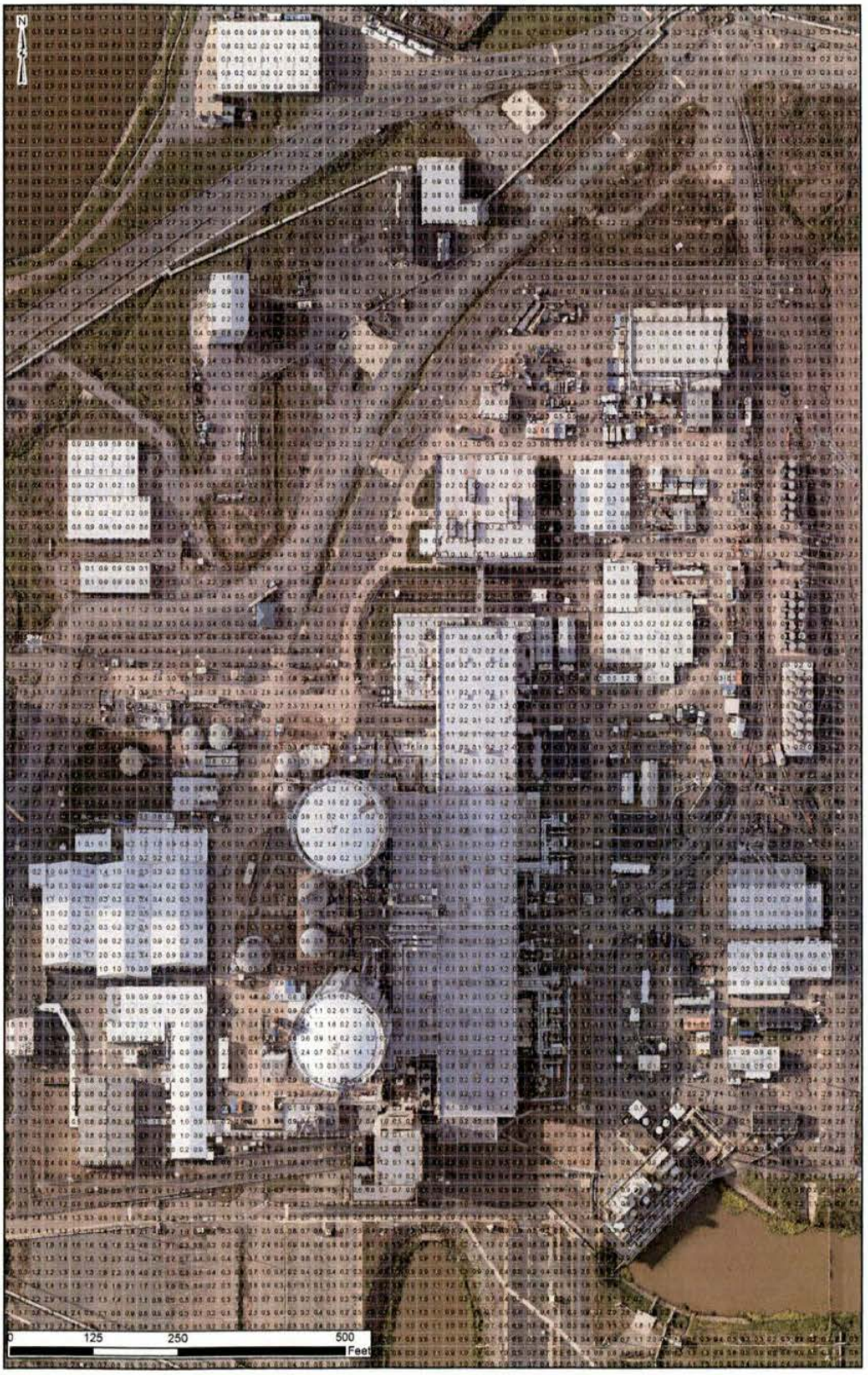
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3.1-11: Maximum Water Surface Elevation (feet, NGVD29) within Power Block Area**




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

**Figure 3.1-12: Maximum Velocities (feet per second) within Power Block Area**




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

**Figure 3.1-13: Grid Elements Reporting Super Critical Flow within Power Block Area**

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

## 3.2 Flooding in Rivers and Streams

### 3.2.1 USACE Input

The USACE, on behalf of NRC, performed the Probable Maximum Flood on the Arkansas River analysis using USACE precipitation, hydrologic and hydraulic models; including USACE-owned and operated lock and dam information unavailable to the public. The NRC/USACE provided results for "Operational Release" (PMF only).

(b)(3): 16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

### 3.2.2 References

**USACE, 2016**. "ANO USACE Hydrographs", United States Army Corps of Engineers, 2016. See AREVA Document No. 38-9257436-000.

## 3.3 Dam Breaches and Failures

### 3.3.1 USACE Input

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

### 3.3.2 References

**USACE, 2016.** "ANO USACE Hydrographs", United States Army Corps of Engineers, 2016. See AREVA Document No. 38-9257436-000.

## 3.4 Storm Surge

### 3.4.1 Storm Surge Screening Discussion

Storm surges are defined as rises in offshore water elevations caused principally by the sheer force of winds acting on the water surfaces, typically associated with hurricanes (NRC, 2011, Section 3.5). Additional NRC guidance recommends that nuclear plants adjacent to cooling ponds or reservoirs subject to potential hurricanes, windstorms, and squall lines must consider the potential for storm surge related flooding (NRC, 2013, Section 3).

ANO is located on the hydrologically controlled Arkansas River which includes downstream dams. As such, regional storm surge swells propagating from the Gulf of Mexico upstream to ANO via the river will not occur.

In addition, the hydrometeorological conditions locally limit the development of storm surges. The Arkansas River and Dardanelle Reservoir in the ANO area are relatively narrow and meandering, which reduces the broad and extensive water surface area needed to generate a storm surge. Also, the generation of sustained, hurricane-type winds (including from tropical depressions and storms) at ANO is minimized due to its inland location.

Wind-generated waves on the Dardanelle Reservoir are evaluated as part of the combined effect analysis detailed in Section 3.9.

Oscillations of the water body due to meteorological events are evaluated as part of the seiche analysis detailed in Section 3.5.

### 3.4.2 References

**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, November 2011.

**NRC, 2013.** "JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, Interim Staff Guidance", Revision 0, January 2013. (ADAMS Accession No. ML12314A412)

## 3.5 Seiche

A seiche is defined in NUREG/CR-7046 as an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause (NRC, 2011). Once started, the oscillation may continue for several cycles; however, over time it gradually decays because of friction.

This section addresses the potential for flooding at ANO due to the Probable Maximum Seiche (PMS) in: 1) Lake Dardanelle, 2) emergency cooling pond, 3) intake canal, and 4) discharge canal which were identified as being susceptible to low frequency water surface oscillations or seiches.

This section summarizes the PMS evaluation performed in AREVA Calculation No. 32-9207387-000 (AREVA, 2015).

### 3.5.1 Method

The approach and methodologies used in this calculation to evaluate the seiche hazard at ANO include the following:

1. Literature review of historical seiche instances;
2. Estimating the natural period of oscillation (primary seiche mode) of the surface water bodies. The natural periods of oscillation of the surface water bodies were calculated using Merian's formula (Scheffner, 2008). Merian's formula is derived from an analytic solution to the primitive equations for fluid motions in an idealized basin (rectangular basin). It provides a method for estimating the natural periods for the seiche modes in enclosed and semi-enclosed basins; and
3. Comparing the natural period of the surface water bodies to the periods of potential forcing mechanisms, including meteorological, seismic forcing, and wind generated waves to determine the potential for resonance. The external forcing mechanisms must resonate with the natural frequency of the surface water bodies for seiches to potentially present a flood risk to ANO,

All surface water bodies have been treated as rectangular basins. This simplification is consistent with the guidance provided in Appendix F of NUREG/CR-7046 (NRC, 2011).

### 3.5.2 Results

#### 3.5.2.1 Historical Seiches in the Vicinity of ANO

Within the United States, 763 of 6,435 United States Geological Survey (USGS) surface water gages registered seiches as a result of the 1964 Alaska earthquake. The Dardanelle Lock and Dam was not yet completed in 1964, and the Arkansas River had not been impounded to create Lake Dardanelle. However, several seiches were recorded in the vicinity of ANO. The largest seiche recorded in Arkansas was 1.45 feet on Lake Ouachita near Hot Springs (USGS, 1967) approximately 51 miles from ANO. On the Piney Creek near Dover, Arkansas, approximately 17 miles from ANO, a 0.48 foot seiche was recorded. While there is an absence of significant observed seismic seiches at the site during the Alaska earthquake, Lake Dardanelle could be susceptible to seiches induced by seismic events based on seiches observed in the region. Therefore further screening to investigate potential seiche interaction with water body configuration was warranted.

### 3.5.2.2 Lake Dardanelle

#### 3.5.2.2.1 Natural Period

Generally, it has been found that natural seiche periods range from tens of seconds to several hours (Rabinovich, 2009). Periods for Lake Dardanelle computed below are consistent with this general finding.

Lake Dardanelle is evaluated as a semi-enclosed basin in the longitudinal direction, and as an enclosed basin at the banks in the transverse direction, see Figure 3.5-14. The bottom of the reservoir is at approximately elevation 320 feet NGVD 29 (USGS, 1994a and USGS, 1994b). The resulting average depth is 18 feet (assuming a maximum lake elevation of 338 feet). In the longitudinal direction, Lake Dardanelle measures approximately 62,075 feet from the confluence of the Arkansas River and the Big Piney Creek to the Dardanelle Lock and Dam. The estimated period for the primary mode is approximately 2.9 hours using the Dardanelle Lock and Dam as a reflection point. In the transverse direction, the width of Lake Dardanelle was based on the two representative cross sections nearest ANO as approximately 15,100 feet and 16,950 feet. The estimated period for the primary mode in the range of between approximately 20.9 to 23.5 minutes. A summary of reservoir geometry and resulting seiche periods is provided in Table 3.5-5.

#### 3.5.2.2.2 External Forcing Periods

Earthquakes

Periods from typical earthquakes do not exceed 10 seconds, and are outside of the range of the natural period of the Lake. Both the longitudinal and transverse periods calculated for Lake Dardanelle are well outside the response spectra of the Operating Basis Earthquake (OBE) and the Design Basis Earthquake (DBE) for both Units 1 and 2 at ANO (ANO, 2016) and in the range where expected accelerations would be insignificant.

Meteorological

Meteorological forcing does not have sufficient energy at this frequency to drive a seiche in Lake Dardanelle. Local convection drives wind gusts with a period of about one minute (Wells, 1997), are outside the range of primary periods for Lake Dardanelle and does not have sufficient energy to drive a seiche in the reservoir. Diurnal heating and cooling also drive relatively weak periodic motions. A high energy band at the synoptic scale is the spatial and temporal scale of temperate weather systems, which in the United States is about 3 to 7 days (Wells, 1997). The period of the synoptic variability is too long and outside the range of primary periods in Lake Dardanelle to force a seiche in the reservoir.

Waves

Wind generated wave periods generally range from approximately 0.1 seconds to 5 minutes (Kinsman, 1965), and are too short to drive a resonant seiche in the longitudinal or transverse directions of Lake Dardanelle.

### 3.5.2.3 Intake Canal and Discharge Canal

#### 3.5.2.3.1 Natural Period

Intake Canal

In the longitudinal direction, the period was estimated as a semi-enclosed basin with the anti-node at the intake opening, and the transverse period was estimated as an enclosed basin due to rip-rap slopes enclosing both sides (ANO, 2016) see Figure 3.5-15. The intake channel length measures approximately 4,120 feet long (USGS, 1994a), 136.8 feet wide and 14 feet deep (ANO, 1987). The period of the primary mode of the discharge canal is approximately 12.9 minutes. The primary mode in the transverse direction is approximately 12.9 seconds. An overview of the channel dimensions and associated periods is provided in Table 3.5-6.

Discharge Canal

The longitudinal direction the period was estimated as a semi-enclosed basin with the anti-node at the discharge opening, and the transverse period was estimated as an enclosed basin due to rip-rap enclosing both sides (ANO, 2016) see Figure 3.5-15. The discharge channel length measures approximately 690 feet long (USGS, 1994a), 100 feet wide and is 11 feet deep (ANO, 1982). The period of the primary mode of the discharge canal is about 2.4 minutes. The primary mode in the transverse direction is approximately 10.6 seconds. An overview of the channel dimensions and associated periods is provided in Table 3.5-7.

#### 3.5.2.4 External Forcing Periods

Earthquakes

The response spectra for the OBE and DBE at Units 1 and 2 (ANO, 2016) show only small ground acceleration at and above the 10 second range indicating that significant seiches in the transverse direction are not expected to occur within the canals. The longitudinal periods in both the intake and discharge canals are outside the frequency content of earthquakes. Earthquake motions with periods greater than 10 seconds create accelerations less than 1% of gravity and, therefore, are not considered mechanisms which are likely to drive seiches in the canals.

Meteorological

Meteorological forcing such as wind gusts, typically have a period of approximately 1 minute (Wells, 1997). Wind gusts do not have a period that is in alignment with the period of the primary seiche mode in the transverse or longitudinal direction for both the intake and discharge canals.

Waves

Wind-generated waves have periods ranging from 0.1 seconds to about 5 minutes (Kinsman, 1965). The primary period for the intake canal is outside the expected range of wind generated wave periods (Kinsman, 1965). The wind generated wave periods are within the ranges of transverse discharge and intake canal periods and the longitudinal discharge canal period. However, the geometries of the discharge and intake canals do not allow waves to enter in the transverse direction. By inspection, the length of the discharge canal (about 690 feet) does not provide a fetch length long enough to produce substantial wind-wave activity within the discharge canal. If waves were to occur in the discharge canal and form seiche-like oscillations, any water which flowed onto the site would immediately flow back towards the lake, thus limiting flooding.

#### 3.5.2.5 Emergency Cooling Pond

##### 3.5.2.5.1 Natural Period

The emergency cooling pond in both the longitudinal and transverse direction is considered to be an enclosed basin (ANO, 2016; ANO, 2009; and ANO, 2010). The primary periods of the emergency cooling pond along two axes were evaluated for an enclosed basin (see Figure 3.5-16). The average depth is 6 feet (ANO, 2016). The average lengths of the pond in the transverse and longitudinal directions were calculated using pond geometry (ANO, 2016) and top widths measured in ArcMap (USGS, 1994a). The average emergency cooling pond lengths were calculated as 1,015 feet long (axis 1) and 370 feet wide (axis 2). Figure 3.5-16 illustrates another potential seiche axis (axis 3); however, the length of the axis is bounded by axes 1 and 2 and, therefore, does not require further evaluation. The primary mode along axis 1 is 2.4 minutes and 53.2 seconds along axis 2. An overview of the channel dimensions and associated periods is provided in

Table 3.5-8.

Earthquakes

The periods for both axis 1 and axis 2 in the emergency cooling pond are outside the response spectra of the OBE and DBE for Units 1 and 2 at ANO.

Meteorological and Waves

Wind gusts have an approximately 1-minute period and are not in alignment with the period of the primary seiche mode in the direction of axis 1 for the emergency cooling pond. While wind gusts are approximately in alignment with the axis 2 period of the emergency cooling pond, a significant seiche is not expected to be generated due to the limited volume of water in the pond and the spillway. If a seiche were to occur in the pond, the water could either flow over the spillway or the pond banks, but such water would flow back towards Lake Dardanelle rather than in the direction of the plant due to topography in the area.

Waves

Wind-generated waves have periods within the ranges of both the directions of axis 1 and axis 2 emergency cooling pond periods; however, a significant seiche is not expected to be generated due to the limited volume of water in the pond and the spillway. If a seiche were to occur in the pond, the water could either flow over the spillway or the pond banks, but in any event such water would flow back towards Lake Dardanelle rather than in the direction of the plant.

### 3.5.3 Conclusions

The natural periods of Lake Dardanelle, the emergency cooling pond, and intake and discharge canals at ANO are generally not in alignment with those due to external forcing mechanisms. Thus, resonance is not expected. In certain cases where external forcing mechanisms may align with natural periods of water bodies near ANO, the low intensity of the forcing mechanisms at these periods combined with the characteristics of the water bodies and physical setting of the site precludes seiche as a flooding mechanism.

### 3.5.4 References

**ANO, 1982.** "Drawing Number C-36, Discharge Canal Plan, Profile & Details, Revision 5", September 13, 1982. See AREVA Document No. 38- 9207373-000.

**ANO, 1987**. "Drawing Number C-35, Intake Canal Plan, Profile & Details, Revision 8", March 11, 1987. See AREVA Document No. 38- 9207373-000.

**ANO, 2009**. Arkansas Nuclear One, "ECP Spillway Replacement Design Drawings SPEC-08-0001-A Spillway Sections and Detail", Drawing No. C-663, Sheet 6, Revision 0, June 2009.

**ANO, 2010**. Arkansas Nuclear One, "Emergency Cooling Reservoir", Drawing No. C-65, Revision 12, March 2009.

**ANO, 2016.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

**AREVA, 2015**. AREVA Document No. 32-9207387-000, "Arkansas Nuclear One Flooding Hazard Re-Evaluation – Probable Maximum Seiche Screening Level Calculation", 2015.

**Kinsman, 1965.** Blair Kinsman, "Wind Waves- Their Generation and Propagation on the Ocean Surface", Prentice-Hall, 1965.

**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.

**Rabinovich, 2009**. Rabinovich A.B. 2009. "Seiches and Harbor Oscillations". In: Kim, Y.C. ed. Handbook of Coastal and Ocean Engineering. World Scientific, Singapore, 2009, 193–236.**

**Scheffner, 2008**. Scheffner N.W. 2008. "Water Levels and Long Waves." In: Demirbilek, Z., Coastal Engineering Manual, Part II, Coastal Hydrodynamics Chapter 5-6, Engineer Manual 1110-2- 1100, U.S. Army Corps of Engineers, Washington, D.C.

**USGS, 1967**, "Hydrologic Effects of the Earthquake of March 27, 1964 Outside Alaska", Geological Survey Professional Paper 544-C, 1967.

**USGS, 1994a**. United States Geological Survey, Russellville West Quadrangle, Arkasas, 7.5 Minute Series (Topographic), May 1994.

**USGS, 1994b**. United States Geological Survey, Delaware Quadrangle, Arkasas, 7.5 Minute Series (Topographic), January 1994.

**Wells, 1997**. Wells, N., "The Atmosphere and Ocean, A Physical Introduction." John Wiley & sons Ltd, 1997.

**Table 3.5-5: Parameters for length, depth and the resulting period of oscillation for Lake Dardanelle**

| Direction | Length (feet) | Depth (feet) | Calculated Period | Units | Geometry Sources: |
|---|---|---|---|---|---|
| Longitudinal | 62075 | 18 | 2.9 | hours | USGS, 1994a |
| Transverse | 16950 | 18 | 23.5 | minutes | USGS, 1994a |
| Transverse | 15100 | 18 | 20.9 | minutes | USGS, 1994a |

**Table 3.5-6: Parameters for length, depth and the resulting period of oscillation for the Intake Canal**

| Direction | Length (feet) | Depth (feet) | Calculated Period | Units | Geometry Sources: |
|---|---|---|---|---|---|
| Longitudinal | 4120 | 14 | 12.9 | minutes | Length: USGS, 1994a Depth: ANO, 1987 |
| Transverse | 136.8 | 14 | 12.9 | seconds | Length: Drawing C-35 Depth: ANO, 1987 |

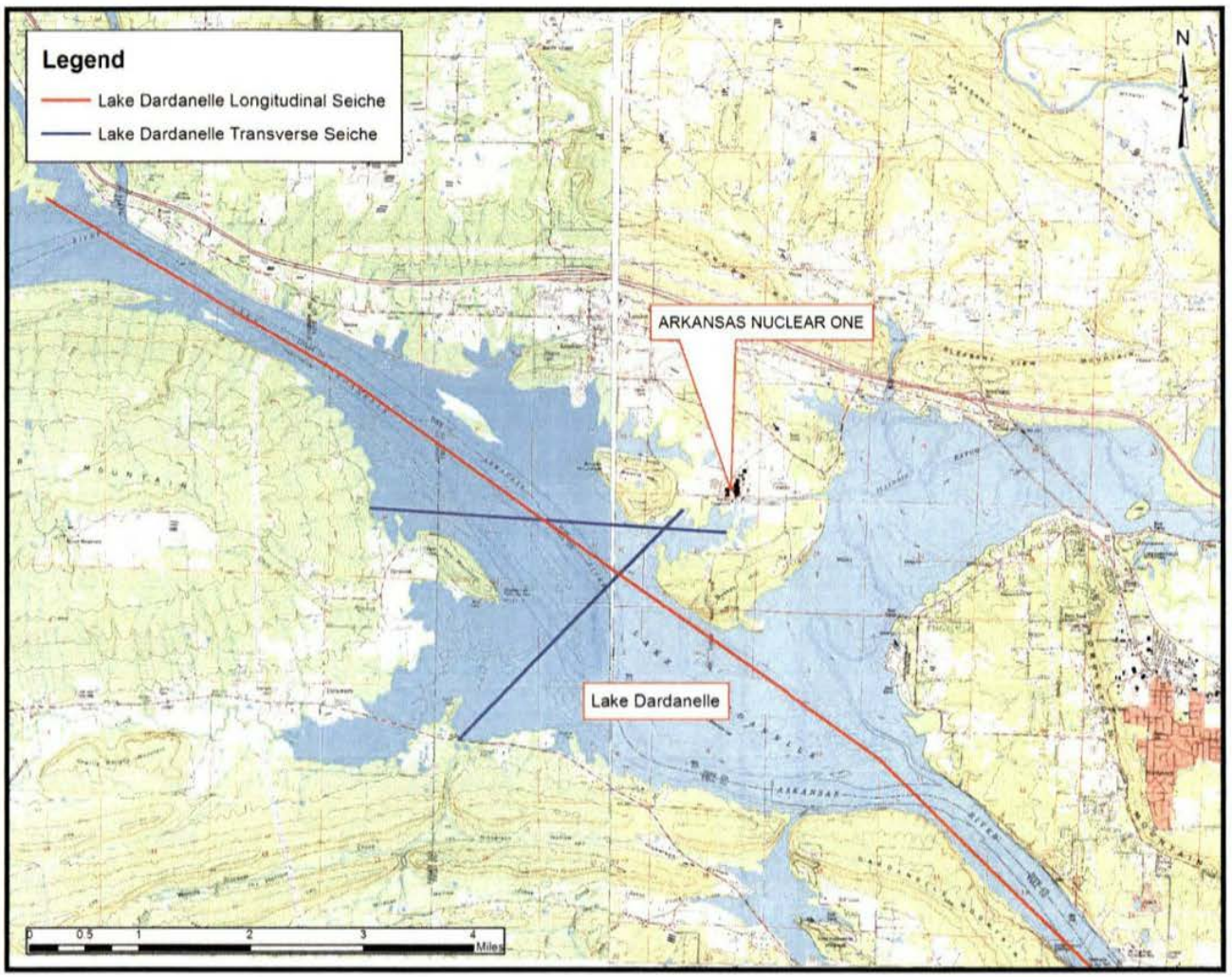
**Table 3.5-7: Parameters for length, depth and the resulting period of oscillation for the Discharge Canal**

| Direction | Length (feet) | Depth (feet) | Calculated Period | Units | Geometry Sources: |
|---|---|---|---|---|---|
| Longitudinal | 690 | 11 | 2.4 | minutes | Length: USGS, 1994a Depth: ANO, 1982 |
| Transverse | 100 | 11 | 10.6 | seconds | Length: ANO, 1982 Depth: ANO, 1982 |

**Table 3.5-8: Parameters for length, depth and the resulting period of oscillation for the Emergency Cooling Pond**

| Direction | Length (feet) | Depth (feet) | Calculated Period | Units | Geometry Sources: |
|---|---|---|---|---|---|
| Axis 1 | 1015 | 6 | 2.4 | minutes | Length: USGS, 1994a Depth: ANO SAR, 2013 |
| Axis 2 | 370 | 6 | 53.2 | seconds | Length: USGS, 1994a Depth: ANO SAR, 2013 |

**Figure 3.5-14: Dardanelle Reservoir Seiche Period Calculation Locations**




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

**Figure 3.5-15: ANO Intake and Discharge Canal Seiche Period Calculation Locations**

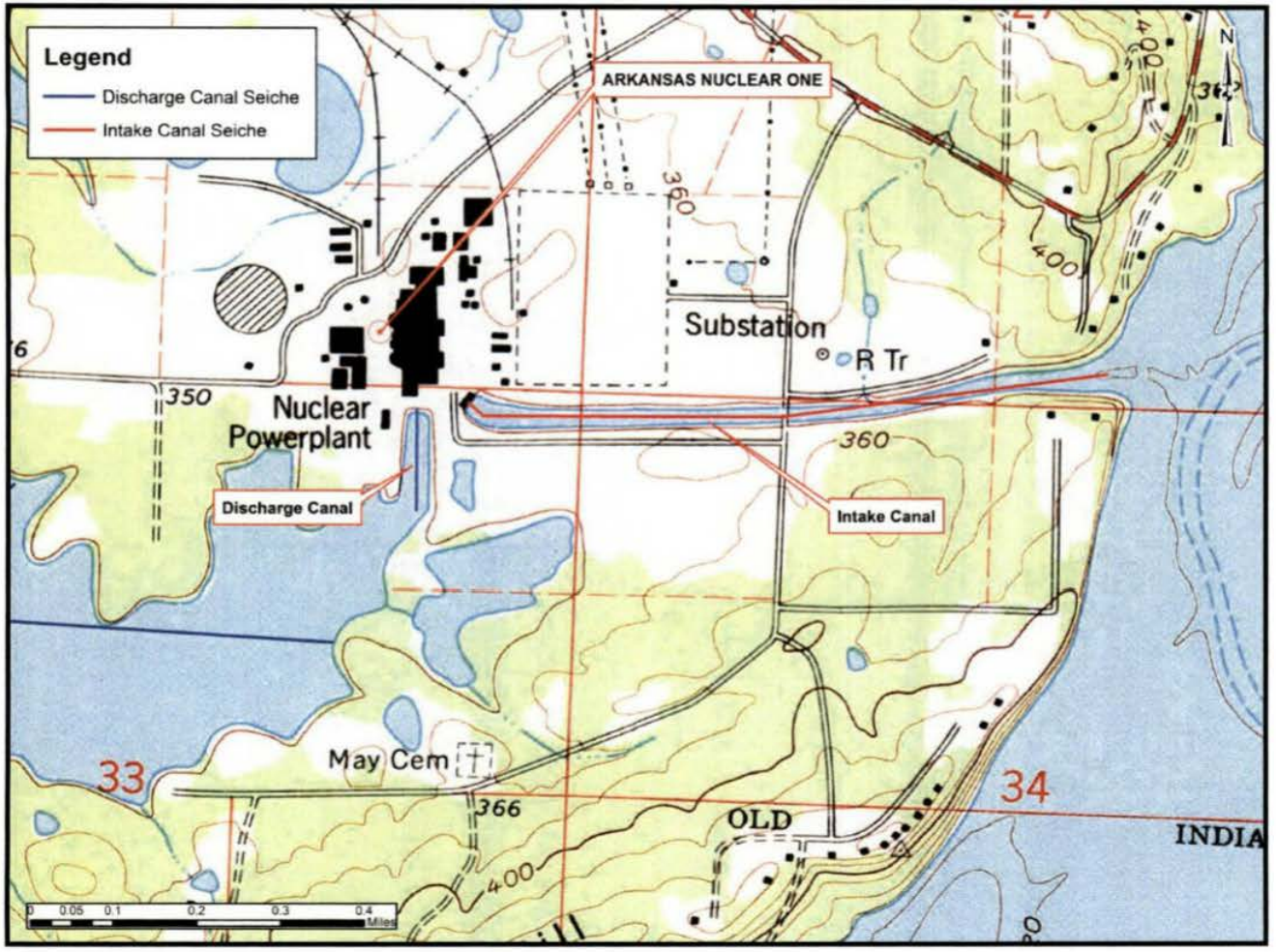
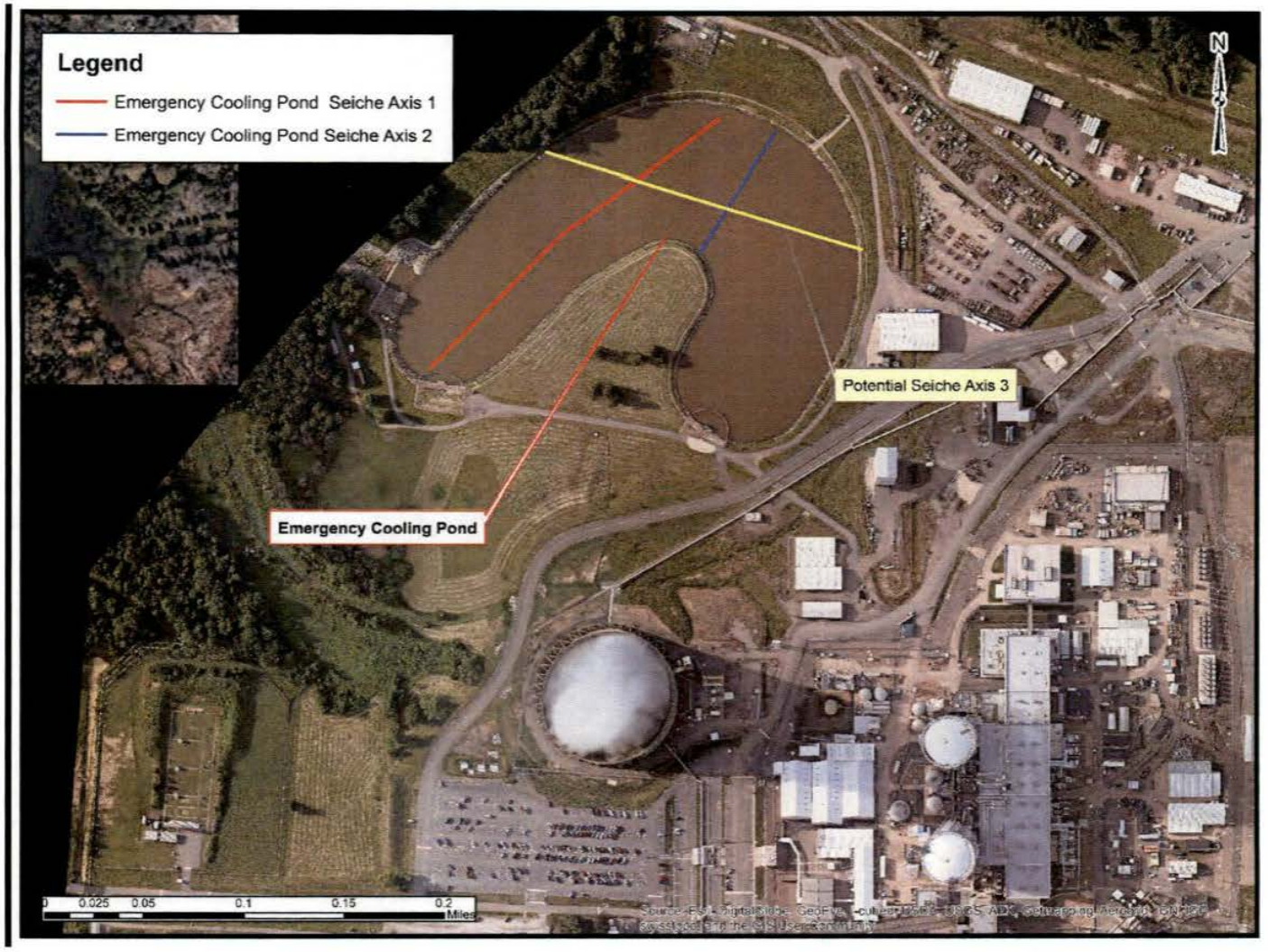


**Figure 3.5-16: ANO Emergency Cooling Pond Seiche Calculations**



## 3.6 Tsunamis

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources (NRC, 2009, Section 1.1 and 1.2). As an inland site, ANO is not susceptible to oceanic tsunamis. Instead, there is the potential of tsunami-like waves in the Dardanelle Reservoir (Lake Dardanelle) along the Arkansas River (NRC, 2009, Section 1.1).

### 3.6.1 Methodology

The ANO tsunami evaluation followed the HHA approach described in NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America (NRC, 2009) and Interim Staff Guidance JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment (NRC, 2013).

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

1. Is the site region subject to tsunamis?
2. Is the plant site affected by tsunamis?
3. What are the hazards posed to safety of the plant by tsunamis?

At ANO, the first two tests were considered. The third test was unnecessary based on the results of the first two tests.

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

The second test was addressed by evaluating the vulnerability of the site location relative to potential tsunami sources.

### 3.6.2 Tsunami Results

At ANO, the first two tests (regional and site tsunami potential) were considered. The third test (tsunami hazard to site) was unnecessary based on the results of the first two tests.

#### 3.6.2.1 Regional Survey (Test 1)

Tsunamis are generated by rapid, large-scale disturbance of a body of water. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by submarine and subaerial landslides. (NRC, 2009, Section 1.3)

Meteorite impacts, volcanoes, and ice falls can also generate tsunamis, but were excluded from the regional survey because meteorite impacts and volcanoes are very rare events and ice falls are generally associated with glacial ice processes.

##### 3.6.2.1.1 NGDC Database Review

The NGDC tsunami-source-event database is global in extent with information dating from 2,000 B.C. to the present. As an inland site, the ANO regional survey considered tsunami-like waves and observations in an area

consisting of the non-coastal southcentral/southeast part of the United States (latitude 30.5°N to 41°N, longitude 81°W to 100°W). Five events due to earthquakes with eight wave runup observations were recorded in the database, occurring between 1811 and 1895 (NGDC, 2013). No wave heights were recorded (AREVA, 2015). These events were associated with earthquakes in the New Madrid fault system.

#### 3.6.2.1.2 Earthquakes

To generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis (NRC, 2009, Section 1.3.1). This magnitude would approximately translate to a modified Mercalli Intensity (subsequently refered to as Intensity) of greater than VIII.

Based on the geological and tectonic history information presented in the ANO Unit 1 Safety Analysis Report, there are no known active or recently active faults in the immediate vicinity of the site. The closest earthquake with an Intensity greater than V was 48 miles away in 1882, which was observed to be Intensity VI to VIII at the estimated epicenter (ANO, 2016, Section 2.7.3 and Table 2-7). A more recent Intensity V earthquake occurred in 1969 approximately 50 miles away. No other earthquakes with an Intensity greater than IV were observed within 50 miles of the site. (ANO, 2016, Section 2.7.3 and Table 2-7)

The site region (within 50 miles) has historical evidence of earthquake activity with potential to cause tsunami like events (ANO, 2016, Section 2.7.3). As a result, the immediate site vicinity must be evaluated for potential impacts. The site is located near the southern end of the Dardanelle Reservoir. Due to the meandering nature of the Dardanelle Reservoir along the Arkansas River flow path, potential tsunami events generated in upstream portions of the reservoir would be subject to several changes in direction in order to propagate to the site. The potential area where a tsunami-like event would be able to directly impact the site is the pool immediately upstream of the Dardanelle Lock and Dam surrounding the site peninsula.

Based on the ANO Unit 1 SAR, there are no known active or recently active faults in the immediate vicinity of the site (within 5 miles) (ANO, 2016, Section 2.7.3 and Table 2-7). As a result, there is no evidence that the Dardanelle Reservoir in the immediate vicinity of the ANO site has sufficient seismic activity to be subject to earthquake generated tsunami events.

Oscillations of the water bodies adjacent to the site due to earthquakes are evaluated as part of the seiche analysis detailed in Section 3.5.

### 3.6.2.2 Landslides (Test 2)

There are two broad categories of landslides: (1) subaerial that are initiated above the water and impact the water body during their progression or fall into the water body, and (2) subaqueous that are initiated and progress beneath the surface of the water body (NRC, 2009, Section 1.3.2).

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

#### 3.6.2.2.1 Subaerial Landslide – Area Topography

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

The USGS landslide hazard overview classifies the Dardanelle Reservoir in the vicinity of ANO as low-incidence and low susceptibility (USGS, 2013) due to relatively flat topography (also see USGS, 1993a and USGS, 1993b).

The potential for landslides occurring adjacent to upstream portions of the Dardanelle Reservoir were not evaluated for impacts to the ANO site due to the meandering configuration of the Arkansas River and the Dardanelle Reservoir, which would mitigate tsunami propagation. Potential events occurring upstream from the

site would be subject to direction changes as the tsunami wave propagated downstream towards the site and lose significant energy. Shallow water depths in the immediate vicinity of ANO (USGS, 1993a and USGS, 1993b), including downstream to the Dardanelle Lock and Dam, mitigates the potential for subaerial landslides to induce a significant tsunami.

Thus, given a subaerial landside, it is judged that there would be little, if any, effect to the ANO site.

#### 3.6.2.2.2 Subaqueous Landslide

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

Based on review of the United States Geological Survey 7.5 Minute Topographic mapping of the Dardanelle Reservoir in the vicinity of the ANO site, the bathymetry has relatively shallow relief and is judged to not be susceptible to landslides (USGS, 1993a and USGS, 1993b). Additionally, due to the configuration of the reservoir, subaqueous landslides outside of the immediate vicinity of ANO are unlikely to propagate in the direction of the site.

### 3.6.3 Conclusions

Tsunami type events in the Dardanelle Reservoir are not likely to occur due to the lack of tsunamigenic mechanisms in the vicinity of the ANO site. Additionally, the geometry of the reservoir relative to the location of the site limits the potential for tsunami like waves to impact the site directly.

### 3.6.4 References

**AREVA, 2015.** "Arkansas Nuclear One Units 1 and 2 Flooding Hazard Re-Evaluation –Tsunami and Storm Surge", AREVA Document No. 51-9210160-000, 2015. Amended 2016 by AREVA Document No. 159-9256649-000.

**ANO, 2016.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

**NRC, 2009.** "NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America", U.S. Nuclear Regulatory Commission, March 2009.

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

**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.

**NRC, 2013.** "JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, Interim Staff Guidance, Revision 0", U.S. Nuclear Regulatory Commission, January 2013.

**NGDC, 2013.** National Oceanic Atmospheric Administration, National Geophysical Data Center, Tsunami Database. Originally accessed at website: http://www.ngdc.noaa.gov/hazard/tsu.shtm; date accessed August 22, 2013. See AREVA, 2015.

**USGS, 1993a.** "Russellville West Quadrangle Arkansas 7.5 Minute Series (Topographic)", U.S. Geological Survey, 35093-C2-TF-024, 1993. See AREVA, 2015.

**USGS, 1993b.** "Delaware Quadrangle Arkansas 7.5 Minute Series (Topographic)", U.S. Geological Survey, 35093-C3-TF-024, 1993. See AREVA, 2015.

**USGS, 2013.** Landslide Overview Map of the Conterminous United States, U.S. Geological Survey, Open File Report 97-289, 1982. Originally accessed at website: http://landslides.usgs.gov/learning/nationalmap/; date accessed August 8, 2013. See AREVA, 2015.

**USGS, 2015.** Magnitude / Intensity Comparison, U.S. Geological Survey, Originally accessed at website: http://earthquake.usgs.gov/learn/topics/mag_vs_int.php, accessed May, 2015. See AREVA, 2015.

## 3.7 Ice-Induced Flooding

As per NUREG/CR-7046, Appendix G (NRC, 2011), ice-induced events may lead to flooding at a site due to two scenarios:

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

This section summarizes the Ice-Induced Flooding evaluation performed in AREVA Calculation No. 32-9207386-000 (AREVA, 2013).

### 3.7.1 Method

The ice-induced flooding evaluation followed the HHA described in NUREG/CR-7046 (NRC, 2011, Section 2). With respect to ice effects at ANO, the HHA used the following steps:

1. Identified the largest historic ice-induced flooding event from the ice jam database (USACE, 2013a) on the Arkansas River and calculated the approximate resulting water depth.
2. Estimated peak water surface elevation at ANO resulting from failure of upstream ice jam that hypothetically formed at the first bridge upstream of ANO.
3. Evaluated the potential for ice jams that could form at the first in-stream structure downstream of ANO.

### 3.7.2 Ice-Induced Flooding Results

#### 3.7.2.1 Review of historical ice events

The USACE ice jam database (USACE, 2013a) was queried to obtain the record of ice jams that have occurred on the Arkansas River. The period of record available was from 1857 through October 2013. The results from the ice jam database query are shown in Table 3.7-9 and illustrated in Figure 3.7-17. The closest reported ice jam occurred in Nickerson and Sterling, Kansas approximately 420 miles upstream of ANO in 1949, and the most severe occurred about 500 miles upstream of ANO in Dodge City, Kansas in 1924 (USACE, 2013a). There have been no ice jams on the Arkansas River since the completion of the McClellan-Kerr Arkansas River Navigation System (MKARNS). This indicates an ice jam in the vicinity of ANO is unlikely.

The highest historic ice-induced flood was calculated based on the February 8, 1924 ice jam occurring at Dodge City, Kansas (USGS Gage 07139500). The reported river flood stage was converted into a water depth, by subtracting the river stage prior to the ice jam from the river flood stage caused by the ice jam. A river stage of 11.3 feet was recorded at USGS Gage 07139500 at Dodge City, Kansas (USACE, 2013a). There was no flow data available from February 1924. The river near Dodge City has historically gone intermittently dry (USGS, 2014), resulting in a river stage of 0.0 feet. The river was, therefore, conservatively assumed to have been dry prior to the formation of the ice jam. The resultant calculated water depth behind the ice jam was 11.3 feet.

### 3.7.3 Results for Water Depth from Upstream Ice Jam

The first structure upstream of ANO is the Morrison Bluff Bridge (Route 109 Bridge) located approximately 21 miles upstream of ANO, was used as the location of the formation for the upstream ice jam (see Figure 3.7-18). The Morrison Bluff Bridge has a minimum clearance of 20 feet above the Arkansas River (ASHC, 1977). See Table 3.7-10 for pertinent information (ASHC, 1977).

The peak water surface elevation at ANO, resulting from the upstream ice jam breach was equal to the elevation of maximum normal pool level at ANO combined with the water depth of the peak flood wave from the ice jam failure. This was considered conservative because the peak flood wave height is kept constant, and does not allow for attenuation as the flood wave travels downstream to ANO. The maximum normal pool elevation at ANO is approximately 338.0 feet NGVD29 (ANO, 2016). The resulting flood wave elevation at the site is 349.3 feet, which is approximately 3.7 feet below the site grade of 353 feet NGVD 29 (ANO, 2016) and below the current design basis flood elevation at ANO, [redacted] feet NGVD 29 (ANO, 2016).

(b)(3):16 U.S.C. § 824o-1(d);(b)(4),(b)(7)(F)

### 3.7.4 Results for Surface Elevation from Downstream Ice Jam

The first structure downstream of ANO is the Dardanelle Lock and Dam, located approximately 5 miles downstream (see Figure 3.7-18). During long periods of sub-freezing air temperature, it is possible that shallow areas of the river with low flows would freeze, but it is unlikely that the navigable channel would be allowed to freeze as the USACE performs maintenance of the MKARNS as necessary to maintain the authorized project dimensions. The Dardanelle Lock and Dam also functions as a hydroelectric generation facility (USACE, 1995), which needs continuous flow to operate during cold periods. Ice jams could be flushed through the gates in the unlikely event that ice collected along the Dardanelle Lock and Dam. The Dardanelle pool level could be frequently fluctuated to prevent the formation of ice within Dardanelle Lake. For these reasons, it is unlikely that an ice jam would form on the Dardanelle Lock and Dam, which is consistent with the statements provided within correspondence with USACE (USACE, 2013b).

### 3.7.5 Conclusions

At ANO, the potential of ice-induced flooding impacting the site is judged to be negligible for the following reasons:

1. No ice jams within 400 miles of ANO have been recorded.
2. Even if an ice jam were to form, the peak water surface elevation at ANO resulting from an upstream ice jam breach was estimated to be 349.3 feet NGVD 29, 3.7 feet below site grade.
3. Ice jams downstream of ANO are unlikely to form and would not have a significant impact at ANO due to the operation of Dardanelle Lock and Dam and the maintenance of MKARNS by the USACE.

### 3.7.6 References

**ANO, 2016.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

**AREVA, 2013.** "Arkansas Nuclear One Flooding Hazard Re-Evaluation – Ice-Induced Flooding Calculation", AREVA Document No. 32-9207386-000, 2013.

**ASHC, 1977.** "Layout of Bridge over Arkansas River", Arkansas State Highway Commission, Revised October, 1976.

**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.

**USACE, 1995**. "25 Years Later: A History of the McClellan-Kerr Arkansas River Navigation System in Arkansas", U.S. Army Corps of Engineers, Little Rock District, 1995.

**USACE, 2013a**. "Ice Jam Database", U.S. Army Corps of Engineers, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, http://icejams.crrel.usace.army.mil/, Date modified October 11, 2013, Date accessed October 11, 2013.

**USACE, 2013b**. "Water Temperature Data for Arkansas River", U.S. Army Corps of Engineers, Little Rock District correspondence to GZA, GeoEnvironmental, October, 10, 2013.

**USGS, 2014**. "USGS 07139500 Arkansas River at Dodge City, KS", U.S. Geologic Survey, http://waterdata.usgs.gov/nwis/nwisman/?site_no=07139500&agency_cd=USGS, Date modified January 21, 2014, Date accessed January 21, 2014.

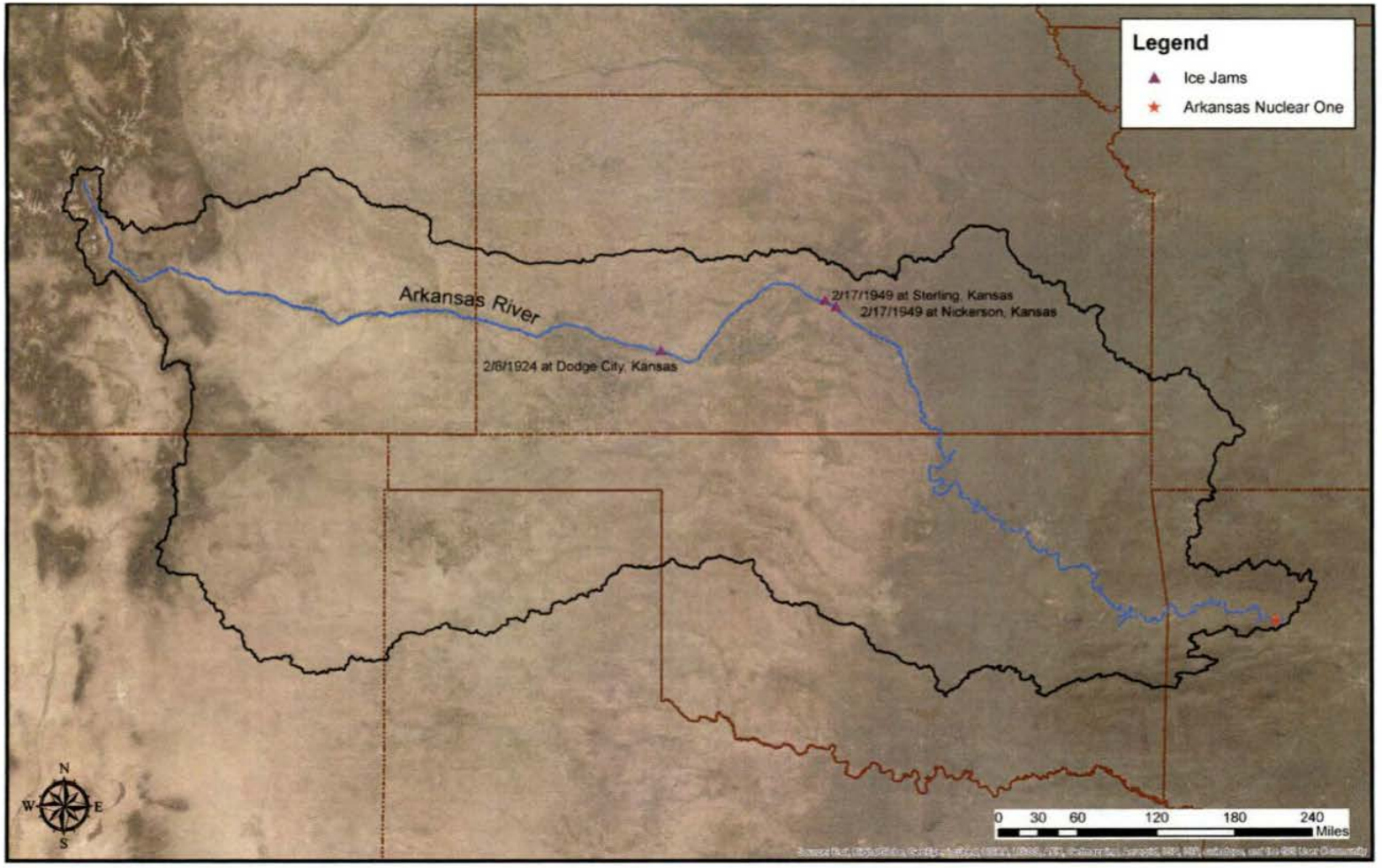
**Table 3.7-9: Historic Ice Jams Data**

| Date | Location | Minimum Gage Height (feet, NGVD29) | River Stage (feet, NGVD29) | Water Depth from Ice Jam (feet) |
|---|---|---|---|---|
| 2/8/1924 | 07139500 Dodge City, KS | 2468.71 | 2480.01 | 11.3 |
| 2/17/1949 | Unknown Gage Sterling, KS | Unknown | Unknown | Unknown |
| 2/17/1949 | Unknown Gage Nickerson, KS | Unknown | Unknown | Unknown |

**Table 3.7-10: Upstream Structure Information**

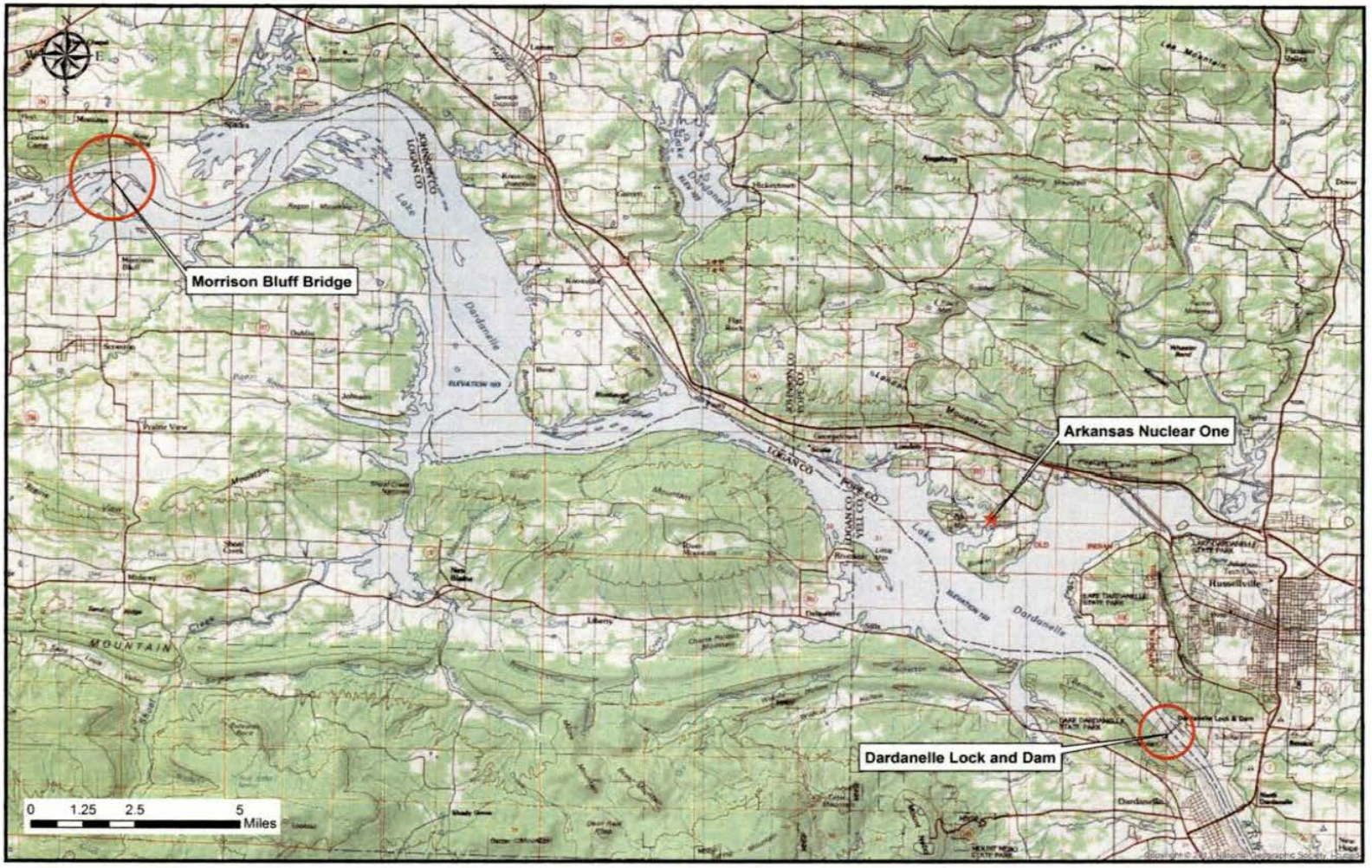
| Structure Description | Distance Upstream from ANO (miles) | Low Chord at Centerline of Navigation Channel (feet, NGVD 29) | Bridge Deck at Centerline of Navigation Channel (feet, NGVD 29) |
|---|---|---|---|
| Route 109 Bridge | 21 | ±404.0 | ±438.0 |

**Figure 3.7-17: Locations of Historic Ice Jams**




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

**Figure 3.7-18: Locations of Nearest Upstream and Downstream Structures**




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

## 3.8 Channel Migration or Diversion

Natural channels may migrate or divert either away from or toward the site. The relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydro-geomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site. (NRC, 2011, Section 3.8)

This section summarizes the Channel Migration or Diversion evaluation performed in AREVA Document No. 51-9209601-000 (AREVA, 2013).

### 3.8.1 Method

The channel migration and diversion flooding evaluation followed the HHA approach described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC, 2011). The approach for the re-evaluation of flooding resulting from channel migration (or diversion) is summarized in general terms below:

1. Review historical records and hydro-geomorphological data to assess whether the Arkansas River has exhibited the tendency to meander towards the site.
2. Evaluate present-day channel protection and stabilization measures in place to mitigate channel diversion of the Arkansas River.

### 3.8.2 Results

Historic Channel Diversion

Prior to the completion of the McClellan-Kerr Arkansas River Navigation System in 1970, the Arkansas River occasionally naturally meandered into new courses creating oxbows and cutoffs (USACE, 1995). The impoundments created by the dams built as part of the MKARNS added stability by maintaining the width of the river and creating a semi-permanent edge with flow staying within a relatively fixed channel (USACE, 1995). A comparison of 1890 (USGS, 1890) and 1984 (USGS, 1984) topographic maps illustrates the changes to the Arkansas River after the completion of the MKARNS, see Figures 3.8-19 and 3.8-20.

Historic orthophotos were obtained from 2006 (ASLIB, 2006) and 2013 (USDA, 2013) for the Arkansas River near ANO. Two major floods, in April 2008 and April 2011 were recorded at USGS Dardanelle (USGS, 2014) stream gage between the two historic orthophotos. The April 2008 flood was the second largest flood (367,000 cfs) and the April 2011 flood (332,000 cfs) was the fourth largest flood in the 45 recorded peak flows from 1969 to 2013. The visual comparison of the 2006 and 2013 historic orthophotos, representing river condition before and after the April 2008 and April 2011 major floods, illustrated that no notable channel migration had occurred in the vicinity of ANO, see Figures 3.8-21 and 3.8-22.

Channel migration typically occurs as the outer bank of a river is eroded while a lateral sandbar is deposited along the inside bank. As migration advances the inner bank can become a series of swales and ridges that comprises a point bar landform (USACE, 1986). Figure 3.8-23 (USACE, 1986) denotes a point bar landform along the inner bank along the peninsula housing the site, indicating that ANO is located on the bank that is less susceptible to erosion.

The USACE Engineering Research and Development Center determined that head-cutting (the upstream movement of a locally steep channel bottom due to the erosion caused by rapidly flowing water) is currently not an issue in the MKARNS (USACE, 2005a).

Site Geology

The ANO Unit 2 Safety Analysis Report describes the foundations of the safety related structures as follows: "*All Category 1 structures except the Emergency Cooling Pond (ECP) inlet and outlet structures, electrical manholes and the Condensate Storage Tank T41B pipe trenches are founded on competent unweathered bedrock of the McAlester Formation of Pennsylvanian age.*" (ANO, 2016). The safety related structures that are founded on bedrock are less vulnerable to erosion caused by channel migration than if they were founded upon soil.

Landslide Diversions

The slopes adjacent to Lake Dardanelle between the Dardanelle Lock and Dam and Hartman, Arkansas range from less than 1 degree to up to 12 degrees (ANO, 2016). The slopes lack sufficient steepness to develop a landslide that could potentially affect the water surface elevation of Lake Dardanelle. Approximately 2 miles upstream of the Dardanelle Lock and Dam, the maximum slopes adjacent to the reservoir are up to 36 degrees. These slopes decrease to less than 6 degrees within 500 feet of the reservoir (ANO, 2016). It is unlikely that a landslide in this area would contain enough volume to affect flow through the reservoir. If the reservoir did become obstructed by a landslide there should be sufficient time for the debris to be removed before the water levels in the reservoir upstream of the slide would be significantly raised (ANO, 2016).

The USGS compiled a map of landslide incidence and susceptibility for the contiguous United States. Susceptibility to a landslide is classified as high, medium or low based on the probable degree of response of soil and rock to cutting or loading of slopes or abnormally high precipitation. Landslide incidence is classified based on the percentage of land within a particular area involved in landslide processes. Landslide incidence is classified as high (greater than 15-percent), medium (1.5- to 15-percent), and low (less than 1.5-percent). ANO is located within an area considered to have a low landslide incidence, see Figure 3.8-24 (USGS, 2001).

Channel Maintenance

The USACE Little Rock and Tulsa Districts perform maintenance of the MKARNS as necessary to maintain the authorized project dimensions (USACE, 2005b). The navigational channel near ANO is 250 feet wide with a minimum channel depth of 9 feet. The maintenance of the navigation channel is a combination of a series of river training structures as well as maintenance dredging. The river training structures for river stability include shore-normal dikes, wing dikes, levees, and revetments. For example, there are 86 dikes and 23.9 miles of revetment on Pool 10 (Lake Dardanelle). The location of the dikes and revetments near ANO are depicted in USACE navigation charts from river mile 202.6 to 238.5 (USACE, 2003). The revetments are comprised of placed stone or broken concrete along a slope (USACE, 2005a). Typically, there are 13 river reaches within the Little Rock District that require dredging to maintain the 9-foot channel depth. Nine of the ten lock approaches require clamming, dredging and pool manipulations (deviations) up to three times a year to maintain the 9-foot channel depth. As a result of these maintenance procedures the 9-foot channel depth is available almost 100 percent of the time (USACE, 2005b). The impoundments created by the dams built as part of the MKARNS also add stability by maintaining the width of the river and creating a semi-permanent edge with flow staying within a relatively fixed channel. The increased cross sectional flow area resulting

from the impoundment created by Lake Dardanelle reduces flow velocities and thus the potential for channel migration.

Canals and Cooling Pond

The intake canal, discharge canal and emergency cooling pond have erosion protection measures. The entire length of the intake canal is founded on sound rock (ANO, 1987). The intake canal has riprap armoring on the bottom portion of the slopes (from elevation 340 feet to the top of sound rock) from the intake structure for approximately 150 feet. The 2.5 horizontal to 1 vertical intake canal slope is armored with a 9-inch-thick layer of riprap. An additional 1,750 feet of a 1-foot-thick layer of riprap armors the bottom portion of the left slope (from elevation 340 feet to the top of sound rock). The discharge canal has riprap up the entire height of the slopes of the canal from the discharge structure for approximately 200 feet. An additional 400 feet of riprap armors the bottom half of the slopes. The 2.5 horizontal to 1 vertical slope is armored with 12 inches of riprap over filter fabric (ANO, 1982 and ANO, 1987). Visual observation of the emergency cooling pond indicates that there is riprap along the perimeter of the pond (AREVA, 2013b).

### 3.8.3 Conclusions

The potential for river channel migration to impact the site is judged to be negligible at ANO. ANO is located on the bank of the Arkansas River that is less susceptible to erosion due to the presence of a point bar landform along the inner bank along the peninsula. The majority of the safety-related structures are founded on bedrock and are less vulnerable to the impacts of erosion caused by channel migration. The intake canal, discharge canal and emergency cooling pond have erosion protection measures. These erosion protection measures reduce the vulnerability to channel migration.

The MKARNS is heavily navigated and USACE is responsible for maintaining navigable conditions. As part of this responsibility, USACE actively performs maintenance dredging and maintains revetments and dikes that were constructed to minimize the risk of channel diversions, bank erosion, and instability in the Arkansas River and Lake Dardanelle.

### 3.8.4 References

**ANO, 1982**. "Drawing Number C-36, Discharge Canal Plan, Profile & Details, Revision 5", September 13, 1982. See AREVA Document No. 38- 9207373-000.

**ANO, 1987**. "Drawing Number C-35, Intake Canal Plan, Profile & Details, Revision 8", March 11, 1987. See AREVA Document No. 38- 9207373-000.

**ANO, 2016**. Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

**AREVA, 2013**. AREVA Document No. 51-9209601-000, "Arkansas Nuclear One Flooding Hazard Re-Evaluation – Channel Migration or Diversion", 2013.

**ASLIB, 2006**. "2006 Natural Color Country Mosaics: Pope County, Yell Country and Logan County", Arkansas State Land Information Board", http://www.geostor.arkansas.gov/G6/Home.html?id=b128bebaf62914c8b81083b3474dcbe6,ftp://ftp.geostor.arkansas.gov/geostor_raster_02/ADOP2_COUNTY_MOSAICS_RGB_MRSID/, \\cmsas-fp-02b\D$\Geostor6\metadata\FTP\FTP.2006 Natural Color County Mosaic, Date accessed: October 29, 2014, Date modified March 31, 2006.

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

**USACE, 1986**. "Geomorphological Reconnaissance of the Lake Dardanelle and Ozark Lake Project Areas, Arkansas River, Arkansas", U.S. Army Corps of Engineers, Little Rock District, March 1986.

**USACE, 1995**. "25 Years Later: A History of the McClellan-Kerr Arkansas River Navigation System in Arkansas", U.S. Army Corps of Engineers, Little Rock District, 1995.

**USACE, 2003**. "McClellan-Kerr Arkansas River Navigation System, Navigation Charts, Chart Numbers 36 – 41, Cover and Legend", U.S. Army Corps of Engineers, Little Rock District and Tulsa District, 2003.

**USACE, 2005a**. "Arkansas River Navigation Study, Final Environmental Impact Statement, Chapter 4," U.S. Army Corps of Engineers, Little Rock District and Tulsa District, 2005.

**USACE, 2005b**. "Arkansas River Navigation Study, Navigation Channel Depth, Appendix A, Hydrology and Hydraulics Report, Navigation Mile 0.0 to 445.0", U.S. Army Corps of Engineers, Little Rock District and Tulsa District, June 30, 2005.

**USDA, 2013**. "NAIP Aerial imagery" U.S. Department of Agriculture, http://earthexplorer.usgs.gov/, Date accessed: October 29, 2014, Date modified July, 13, 2013.

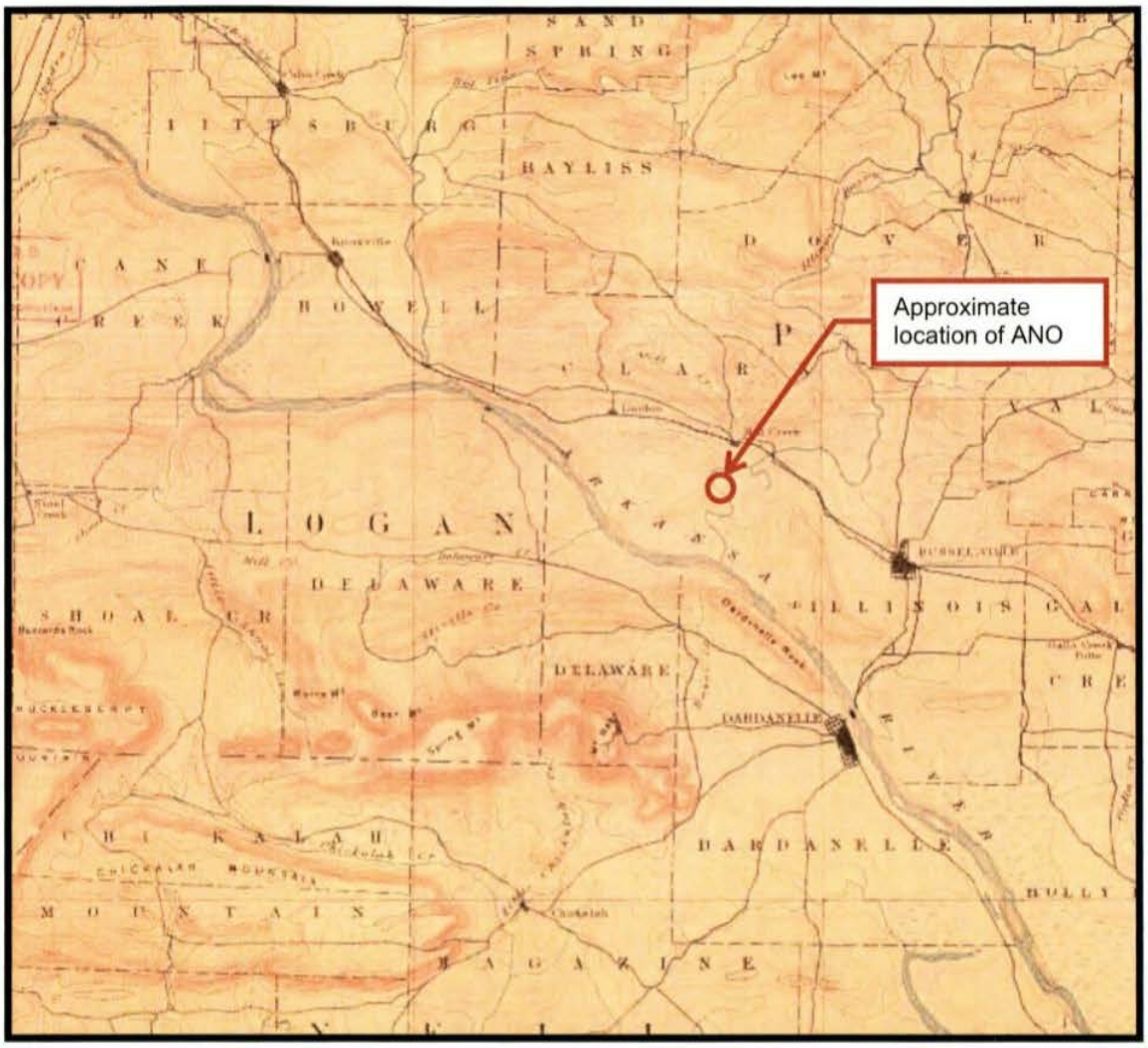
**USGS, 1890**. "Arkansas Dardanelle Sheet", U.S. Geological Survey Historical File Topographic Division, October 1890.

**USGS, 1984**. "Russellville Arkansas" 30 x 60 Minute Topographic Map, U.S. Geological Survey, 1984.

**USGS, 2001**. "Landslide Incidence and Susceptibility in the Conterminous United States U.S. Geological Survey Open-File Report 97-289", U.S. Geological Survey, January 2001.

**USGS, 2014**. "Stream Gage 07258000 Arkansas River at Dardanelle, AR, http://waterdata.usgs.gov/usa/nwis/uv?07258000, Date accessed: October 29, 2014, Date modified October 29, 2014.

**Figure 3.8-19: USGS Topographic Map (1890)**




1890 USGS Topographic Map (USGS, 1890)

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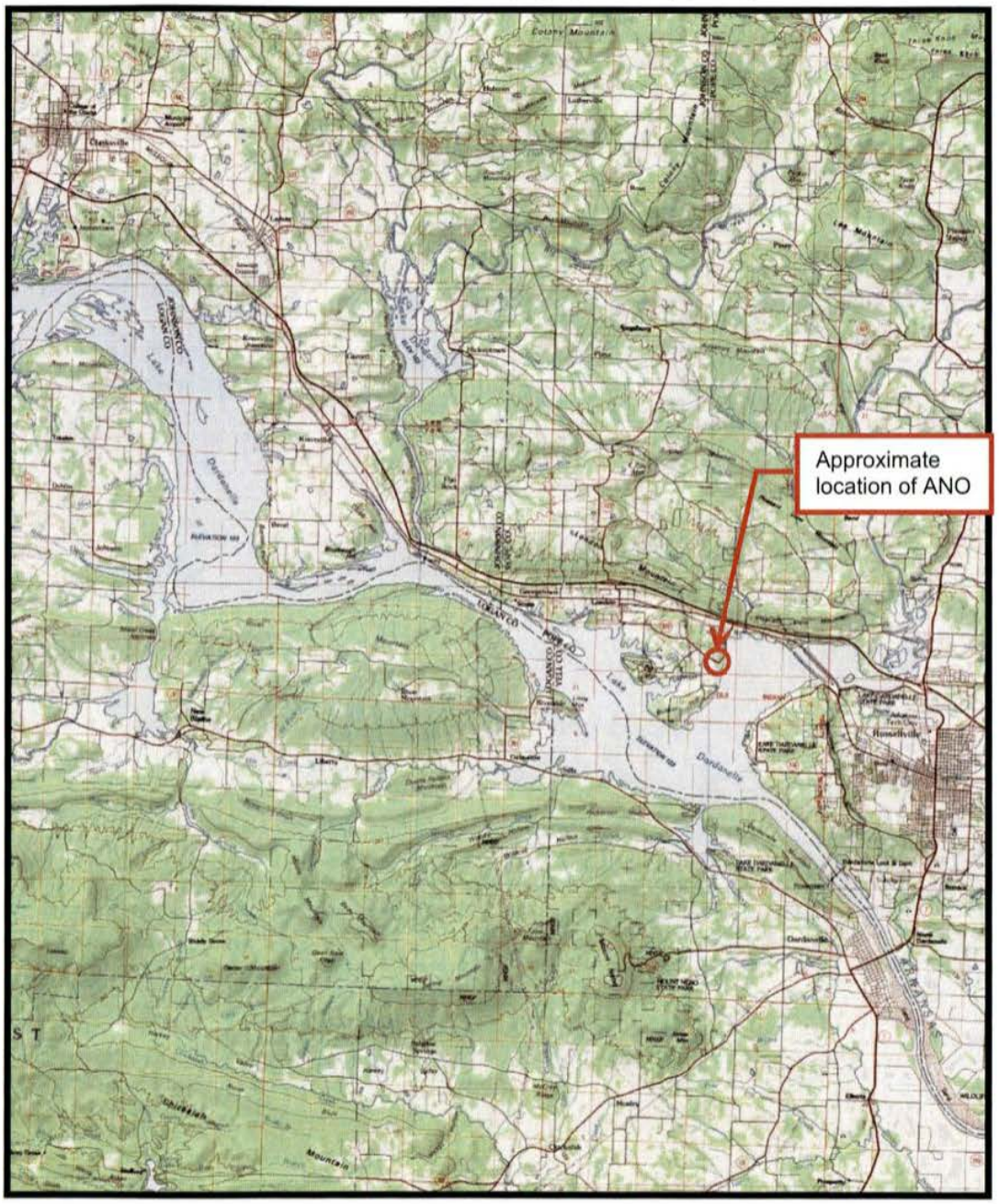
**Figure 3.8-20: USGS Topographic Map (1984)**




1984 USGS Topographic Map (USGS, 1984)

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

**Figure 3.8-21: 2006 Orthophoto (ASLIB, 2006)**

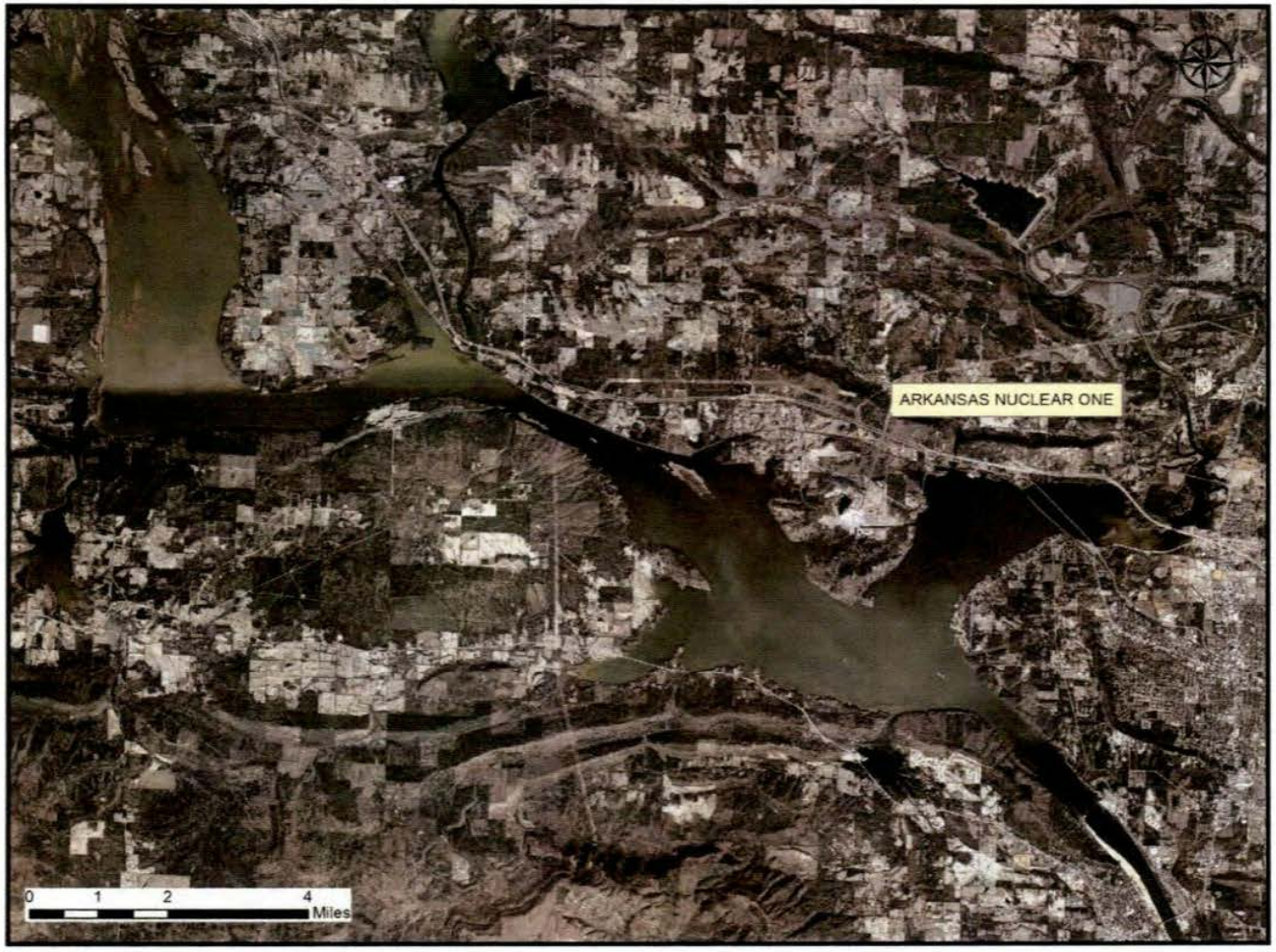


**Figure 3.8-22: 2013 Orthophoto (USDA, 2013)**

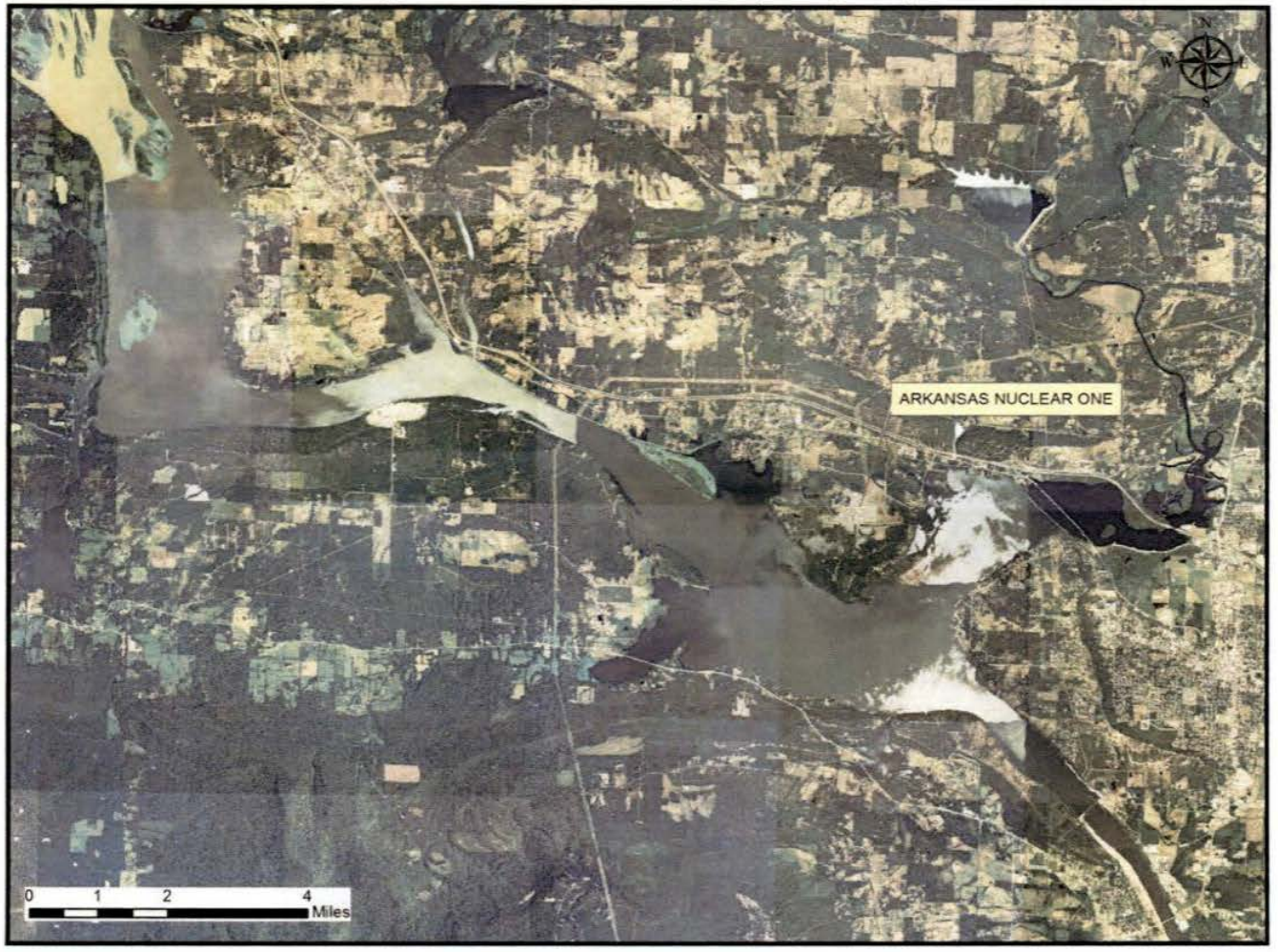
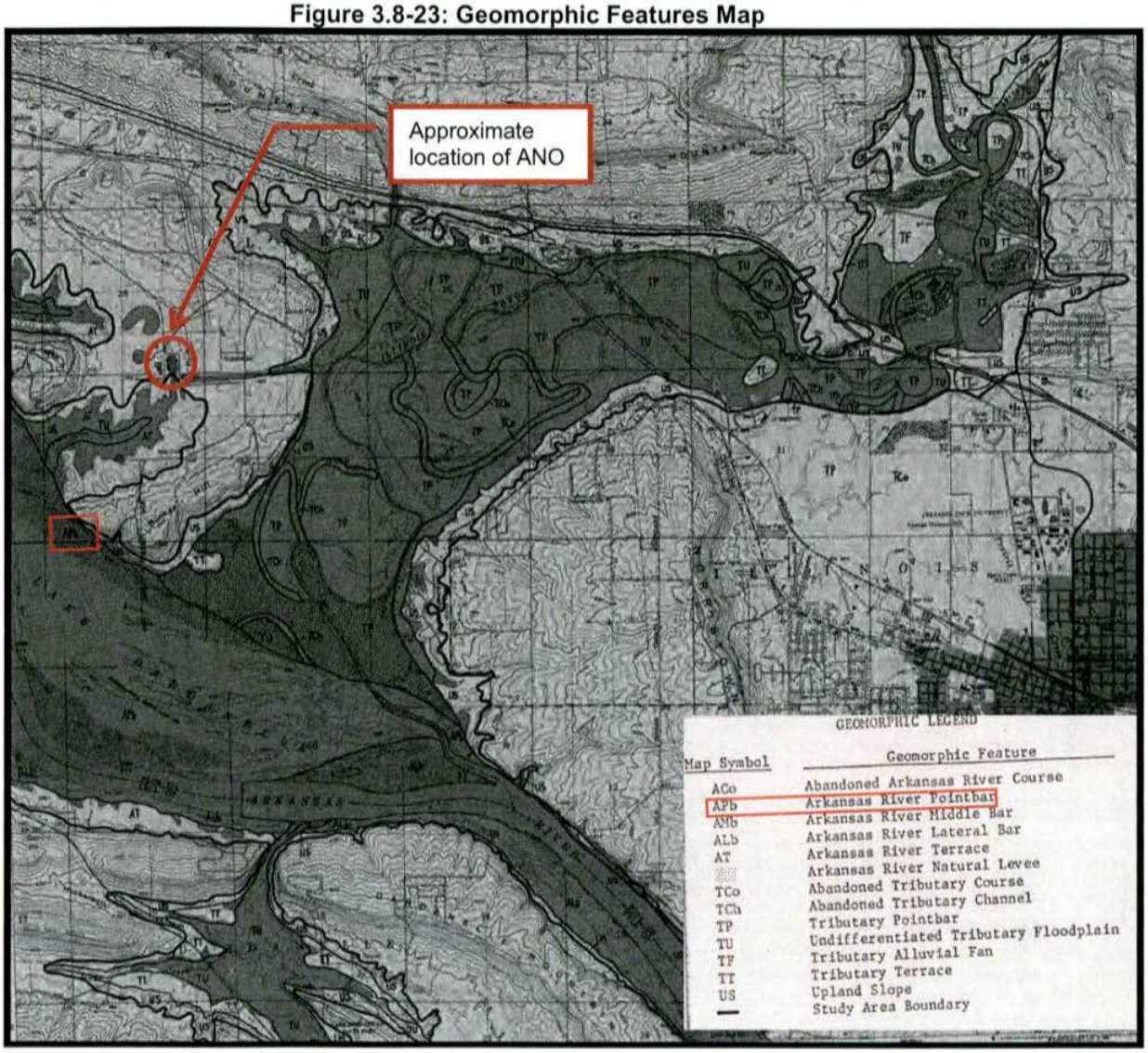


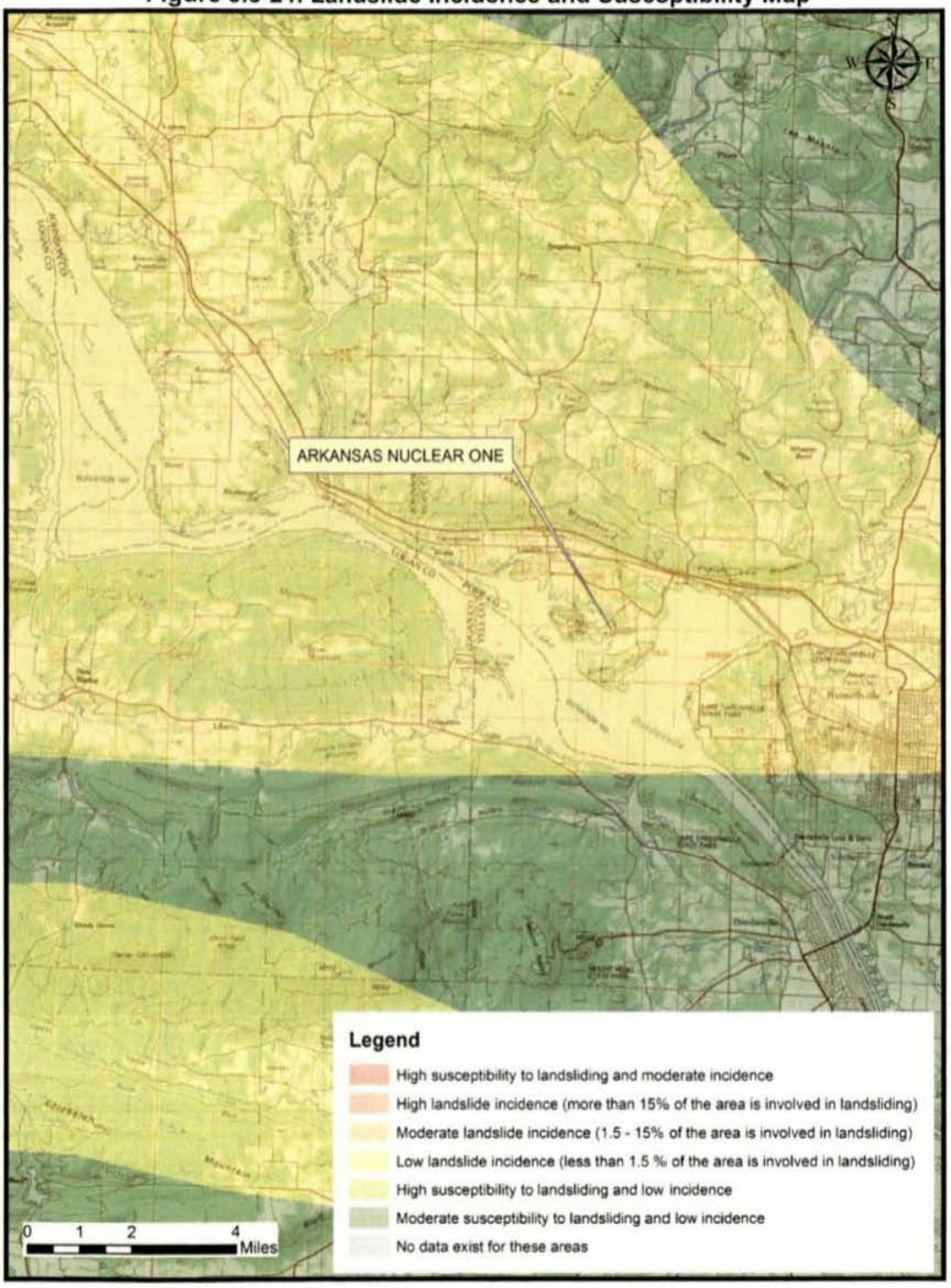
**Figure 3.8-23: Geomorphic Features Map**




(USACE, 1986)

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

**Figure 3.8-24: Landslide Incidence and Susceptibility Map**




(USGS, 2001)

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

## 3.9 Combined Effect Flood

An evaluation of the combined external flood effects associated with riverine flooding at ANO was performed. Note that results of the PMF, and PMF including effects of upstream dam failure were provided directly by the NRC (USACE, 2016). USACE performed the PMF and dam failure calculations which were used as design input to the wind-generated waves analysis. This section also reports associated hydrostatic, hydrodynamic and debris loading forces.

This section summarizes the combined effect flooding evaluation performed in AREVA Calculation No. 32-9207388-000 (AREVA, 2016).

### 3.9.1 Combined Effect Floods Background

#### 3.9.1.1 Hydrologic Setting

ANO is located on a peninsula formed by Lake Dardanelle (Pool No. 10, Dardanelle Lock and Dam). ANO is located near river mile 210 of the Arkansas River, in southwestern Pope County approximately 6 miles northwest of Russellville, Arkansas. The contributory watershed of the Arkansas River at ANO is approximately 153,000 square miles. There are over 5,200 major dams within the entire Arkansas River drainage basin upstream of ANO according to the National Inventory of Dams (USACE, 2013a). The Arkansas River has been highly regulated between circa 1964 and 1971 when the Arkansas River Basin Project, the McClellan-Kerr Arkansas River Navigation System (MKARNS), and its locks and dams, including Dardanelle Lock and Dam went online. MKARNS is operated by the USACE. The system of locks and dams on the Arkansas River regulates normal water levels and creates a series of navigation pools. The MKARNS does not provide flood control (USACE, 2014). Separate dams and reservoirs in the Arkansas River watershed provide flood control benefits (OKDOT, 2012). The hydrologic response of the Arkansas River watershed prior to construction of the MKARNS and the flood control dams is much different than after construction. There 1,772 dams within the watershed which list flood control as one of their functional purposes (USACE, 2013a).

The water surface elevation in Lake Dardanelle is controlled by the operation of the 20 large radial gates (each 50 feet wide and 39 feet high) and other associated low level outlet / hydropower releases (USACE, 2005) within the normal range of Lake Dardanelle elevations, including floods. The Dardanelle Lock and Dam can maintain normal pool (between elevation 336.0 and 338.0 NGVD29) through the operation of the spillway gates for flow rates up to 600,000 cfs, which is the open river flow rate (USACE, 2013b).

#### 3.9.1.2 Historic Floods

The USGS Stream Gage No. 07258000, Arkansas River at Dardanelle, AR, is located 2 miles downstream of the Dardanelle Lock and Dam. Annual peak stream flow rates at the USGS Gage No. 07258000 Arkansas River at Dardanelle were obtained for the entire 77 years of record as shown in Figure 3.9-25 and Table 3.9-11 (USGS, 2016). The flood of record was the May 1943 flood with a peak flow of 683,000 cfs. The flood of record after the completion of the MKARNS at the USGS Stream Gage is the May 1990 flood with a recorded peak flow of 435,000 cfs. The maximum water surface elevation recorded during the May 1990 flood was 337.8 feet NGVD29 (USACE, 2013c), within the normal operating pool elevations of Lake Dardanelle. There are no records of floods after the completion of the MKARNS at the USGS Stream Gage which exceed the 600,000 cfs open river flow rate.

#### 3.9.1.3 NRC / USACE PMF and Dam Failure Analyses

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

### 3.9.2 Method

#### 3.9.2.1 Combined Effect Flood

The criteria for combined effect flooding are provided in NUREG/CR-7046, Appendix H. Note that this calculation includes riverine combined effect flooding only. Coastal flood mechanisms including storm surge, tsunami, and seiche have been screened out (AREVA, 2014a and AREVA, 2014b). There are five combined effect flood mechanisms (riverine and coastal) discussed in NUREG/CR-7046. Only one of the mechanisms is relevant to inland riverine flooding:

Alternative H.1 - Floods Caused by Precipitation Events

The criteria for combined effect floods caused by precipitation events (including applicable hydrologic dam failure effects) were used (NUREG/CR-7046, Appendix H, Section H.1). The maximum stillwater elevation at ANO was determined by NRC/USACE to be a result of upstream hydrologic dam failure during the PMF (USACE, 2016). Waves induced by 2-year wind speed applied along the critical direction was added to the NRC/USACE PMF elevation.

2-year Wind Speed

The 10-meter (measurement height), 2-year annual recurrence interval wind speed was required for the coincident wind-generated wave calculations. Data for 10-meter, 2-minute duration wind speeds from nearby National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center (NCDC) Station Global Historical Climatology Network-Daily (GHCND) stations, was used as the statistical basis for generating 2-year annual recurrence wind speed at the site. Two wind gage stations were evaluated to determine the most conservative 2-year wind speed: 1) Fort Smith Regional Airport, AR (GHCND USW00013964; NOAA, 2016a) and 2) Russellville Municipal Airport, AR (GHCND USW00053920; NOAA, 2016b).

For both stations, the greatest wind speed from each year during the period of record was selected. The Gumbel Distribution, a Generalized Extreme Value (GEV) Distribution (Maidment, 1993), was applied to the annual maximum wind speeds to calculate the 2-year recurrence wind speed.

Wave Height and Period

Wave heights and periods were calculated using the methodology presented in the USACE Coastal Engineering Manual (CEM) for wave growth with fetch (USACE, 2008a). Fetch limited conditions were used in this calculation, consistent with the USACE CEM guidance.

ANO is located on a peninsula and is largely sheltered by hills and high ground. This limits wave propagation along potentially longer fetches across Lake Dardanelle to the site. In addition, wooded areas are located from north to southeast of the site (see Figure 3.9-27). These wooded areas would limit wave propagation, as the wooded areas would break waves coming from the Lake and/or interfere with wave development. The inundated area from the north to the east of the site is characterized by shallow flood depths and a limited areal extent of flooding, which also limits the fetch lengths. Therefore, due to the limited open area around ANO, fetch-limited conditions are considered appropriate for calculation of wave effects.

The equations governing wave growth with fetch are collectively referred to as Equation II-2-36 in the CEM (USACE, 2008a), and were used to calculate the significant wave height and period. The wind speed used was the 2-minute duration, 2-year annual recurrence interval wind speed. The USACE CEM guidance states that for small lakes and reservoirs or in riverine environments, fetch-limited conditions may be attained from a 1-minute to 5-minute wind speed (USACE, 2008a). Therefore, the 2-minute duration wind speed is judged appropriate for use.

The significant wave height is used to estimate the maximum wave height. The maximum wave height was estimated as 1.67 times the significant wave height (ANS, 1992).

Five transects were used to calculate wave effects at areas of interest (ANO, 2016) and are shown in Figure 3.9-28. Waves along other appreciably different fetches than shown would break on the high ground located to the west, from the north to the northeast, and from the southeast to the south of ANO. The fetch was considered to be the largest wetted top width. Wave growth is assumed to occur along the fetch in the direction of the wind. This assumption is conservative as it will generate the largest wave height. It is conservatively assumed that the strongest wind is perfectly aligned with the longest fetch as the worst case scenario.

Deep-water wave equations are recommended in the CEM for estimating wave effects for all depths, as long as the wave period does not exceed the limiting value (Equation II-2-39 from USACE, 2008a). Deep water waves are therefore used herein. Depth-limited wave heights (maximum wave possible for the depth of water present) were assumed whenever the computed wave heights exceeded 0.78 times the flood depth at a given location (Equation II-1-97 from USACE, 2008b).

Wave Setup

Wave setup is an associated hydrodynamic process with shallow-water wave breaking (USACE, 2008b). Therefore, wave setup is not calculated for non-breaking waves. If the calculated wave height is less than the depth-limited wave height the waves are considered non-breaking. In addition, the location of the areas of interest are sheltered by structures, obstacles, and barriers throughout the ANO site (see Figure 3.9-27), which would interfere with the development of wave action and setup.

Wave Effects

The areas of interest are located within structures that are constructed with vertical walls.
The transformation of non-breaking waves at a vertical face results in wave reflection and standing waves with crest elevations greater than the incident waves. Reflected wave crest elevations were calculated using the Sainflou Formula consistent with guidance presented in the USACE CEM (USACE, 2006). Wavelengths at the structure toe (an input to the Sainflou Formula), for a given flood depth and wave period, were calculated using the Hunt method to approximate the solution of the dispersion equation (Dean, 1991).

#### 3.9.2.2 Hydrostatic Force and Hydrodynamic Loading and Debris

Resulting flood depths were used to develop hydrostatic force and hydrodynamic loads. Ground surface elevations used to calculate the stillwater depth at the base of the structures where the areas of interest were located were estimated using the digital terrain model (AREVA, 2013).

Hydrostatic Loads

Hydrostatic loads are those caused by water above or below the ground surface, free or confined which is either stagnant or moves at velocities less than 5 feet per second (fps) (ASCE, 2010). These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. The hydrostatic lateral forces (per linear foot of surface) were calculated using ASCE guidance (ASCE, 2010).

Flow Velocity

Floodwater flow velocities include velocity components due to flooding and wind-generated waves. Flood velocities were estimated conservatively by assuming that floodwaters can approach from the most critical direction relative to the site and by assuming that flow velocities can be high, equivalent to critical velocity (FEMA, 2011). This critical or upper bound velocity (FEMA, 2011) was used to conservatively calculate hydrodynamic and impact loads.

Hydrodynamic Loads

Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides and suction at the downstream side. Hydrodynamic loads calculated here used steady-state flow velocities consistent with Federal Emergency Management Agency (FEMA) guidance (FEMA, 2011; FEMA, 2012). Note that the hydrodynamic loads applied above are for rigid structures. Dividing the horizontal drag force by the building width yields a force per length (pounds per linear foot).

Debris Impact Loads

Debris impact loads are imposed on a building (or structure) by objects carried by moving water. Debris impact loads at the water surface were calculated using the guidelines described in FEMA P-259 (FEMA, 2012) and by considering debris weight recommended in ASCE-7-10 (ASCE, 2010).

Per American Society of Civil Engineers (ASCE) 7-10 (ASCE, 2010), riverine debris weights may range from 1,000 to 2,000 pounds. A debris object weight of 1,000 pounds is a reasonable average for flood-borne debris representing trees, logs and other large woody debris (ASCE, 2010). A debris weight of 2,000 pounds was conservatively used. Shallow depths and surrounding topography serve to prevent larger objects from entering the site area and impacting structures important to safety.

Standing (non-breaking) Wave Loads

Wave loads are those loads that result from water waves propagating over the water surface and striking a building (or other structure), including: a) waves breaking on any portion of the building; and b) wave run-up striking any portion of the building. Loads due to non-breaking waves are calculated as the hydrostatic and hydrodynamic loads. Based on the wave heights calculated, the wind waves at ANO are expected to be non-breaking waves. The hydrodynamic standing wave pressures are calculated based on CEM Chapter 5, Section VI-5-4, based on the Sainflou formula for head-on, fully reflected, standing regular waves (USACE, 2006).

Load / Pressure Combinations

According to FEMA guidelines (FEMA, 2011), the following flood loads are appropriate to be combined including: hydrostatic, hydrodynamic, and debris impact. The hydrodynamic loads in our calculations consist of two components including: 1) current velocity, and 2) added forces due to non-breaking wave impacts.

Probable Maximum Water Elevation

The probable maximum water elevation was calculated by adding the calculated standing wave crest height to the stillwater elevation on Lake Dardanelle at ANO caused by dam failure during the PMF (USACE, 2016). The probable maximum water elevation was calculated at the five areas of interest as shown in Figure 3.9-29 (ANO, 2016).

### 3.9.3 Results

#### 3.9.3.1 Combined Effect Flood

2-year Wind Speed

The 2-minute duration, 2-year annual recurrence interval wind speed was calculated to be 47.0 miles per hour at the Fort Smith Regional Airport, AR NCDC Station. The 2-minute duration, 2-year annual recurrence interval wind speed was calculated to be 33.6 miles per hour at the Russellville Municipal Airport, AR NCDC Station. The 2-minute duration, 2-year wind speed of 47.0 miles per hour was conservatively used for subsequent wave calculations.

Wave Height and Period

Table 3.9-12 provides the calculated fetch lengths used as input to determine the wave height at ANO. The significant wave height calculated varies from 0.3 foot to 3.1 feet, and the period varies from 0.6 seconds to 2.9 seconds. The maximum wave height varies from 1 foot to 5.2 feet. The depth-limited wave height varies from 3.5 feet to 5.2 feet. In instances where the significant wave height or maximum wave height was greater than the depth-limited wave height (maximum wave possible for the depth of water present), the depth-limited height was used. The results are shown in Table 3.9-13.

The calculated wave heights indicate that non-breaking waves are expected at the areas of interest because the wave heights are less than the depth-limited wave heights. One potential exception was identified at the southeast side of Intake Structure No. 1 and Intake Structure No. 2 (i.e., Transect 4)

within the narrow inundation area of the Intake Canal. However, waves are anticipated to be non-breaking at the Intake Structures because:

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Therefore, the waves at the Intake Structures are limited to a maximum of 4.5 feet high. The depth-limited wave immediately upstream of the Intake Structures, within the Intake Canal, is theoretically 5.1 to 5.3 feet high. This results in a non-breaking wave because the maximum wave height of 4.5 feet is less than the depth-limited wave height of 5.1 to 5.3 feet.

Wind Wave Effects

The reflected/standing wave crest heights (above the maximum stillwater elevation) resulting from the significant wave range from a minimum of 0.4 foot at the PASS Building to 4.1 feet at the southeast sides of Intake Structure No. 1 and Intake Structure No. 2. The reflected/standing wave crest heights resulting from the maximum wave range from a minimum of 0.9 foot at the PASS Building to 6.7 feet at the southeast side of Intake Structure No. 1. A summary of the results is provided in Table 3.9-14.

Probable Maximum Water Elevation

The reflected/standing wave crest elevation resulting from the significant wave range from feet NGVD29 at the PASS Building to feet NGVD29 at the southeast sides of Intake Structure No. 2. The reflected/standing wave crest elevation resulting from the maximum wave range from feet NGVD29 at the PASS Building to feet NGVD29 at the southeast sides of Intake Structure No. 1. A summary of the results is provided in Table 3.9-14.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

### 3.9.3.2 Hydrostatic Force and Hydrodynamic Loading and Debris

Hydrostatic

The results of the hydrostatic loading calculations are presented in Table 3.9-15. The maximum water surface elevation of feet NGVD29 results in a depth of flood water that ranges from feet at the PASS Building to feet at the southeast side of Intake Structure No. 2. The hydrostatic force results range from a minimum of approximately pounds per foot acting at elevation feet NGVD29 at the PASS Building to a maximum of approximately pounds per foot acting at elevation feet NGVD29 at the southeast side of Intake Structure No. 2. The maximum hydrostatic pressure (at the base of the structures) results range from a minimum of approximately pounds per square foot at the PASS Building to a maximum of approximately pounds per square foot at the southeast side of Intake Structure No. 2.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Flow Velocity

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

It should be noted that these velocities are judged to be conservative

for use in estimating hydrodynamic and debris impact loading per the HHA approach. Critical velocities used herein significantly exceed reported overbank velocities from the USACE hydrographs (USACE, 2016). Actual velocities at ANO are anticipated to be much lower than critical velocity, due to the partially sheltered location of ANO relative to the main flow conveyance areas within Lake Dardanelle and the Arkansas River.

Hydrodynamic Loads

The results of the hydrodynamic loading calculations are presented in Table 3.9-17. The hydrodynamic force results range from a minimum of approximately [redacted] pounds per foot at the PASS Building to a maximum of approximately [redacted] pounds per foot at the southeast side of Intake Structure No. 2. The hydrodynamic pressure results range from a minimum of approximately [redacted] pounds per square foot at the PASS Building to a maximum of approximately [redacted] pounds per square foot at the southeast side of Intake Structure No. 2.

(b)(3): § 824o- (4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

Debris Impact Loads

The debris impacts results range from a minimum of approximately [redacted] pounds at the PASS Building to a maximum of approximately [redacted] pounds at the southeast side of Intake Structure No. 2, see Table 3.9-18. Debris loads assumed to act at the maximum stillwater elevation ([redacted] feet NGVD29).

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

Standing (non-breaking) Wave Loads

Using the significant wave height, the wave pressures at the stillwater elevation range from approximately 25 pounds per square foot at the PASS Building to a maximum of approximately 201 pounds per square foot at the southeast side of Intake No. 1 and southeast side of Intake No. 2. Using the significant wave height, the wave pressures at the base of the structures range from less than 1 pound per square foot at the PASS Building to a maximum of approximately 111 pounds per square foot at the southeast side of Intake No. 1.

Using the maximum wave height, the wave pressures at the stillwater elevation range from approximately 47 pounds per square foot at the PASS Building to a maximum of approximately 288 pounds per square foot at the southeast side of Intake No. 1. Using the maximum wave height, the wave pressures at the base of the structures range from less than 1 pound per square foot at the PASS Building to a maximum of approximately 161 pounds per square foot at the southeast side of Intake No. 1. The results of the wave load calculation are presented in Table 3.9-19. The results of the load / pressure calculations are presented in Table 3.9-20.

### 3.9.4 Conclusions

Combined effect flooding was evaluated as per the guidance in Appendix H of NUREG/CR-7046. The results of the evaluation of the combined-effect flood at ANO are summarized below:

1. The maximum still water elevation for riverine flood, including the effects of upstream dam failure during the Probable Maximum Flood (PMF) was calculated by the USACE as [redacted] feet NGVD29.

(b)(3):16 U.S.C. § 824o-1(d),(b) (4),(b)(7)(F)

2. The probable maximum water elevation at ANO including wind wave effects resulting from a 2-year wind speed was calculated to be as follows:

| | Diesel Fuel Vault | PASS Building | Intake 1 – Southwest Side | Intake 1 – Southeast Side | Intake 2 - Southeast Side | Start Up 2 Transformer (SU-2) Bus Duct |
|---|---|---|---|---|---|---|
| Significant Wave Height (feet) | 1.0 | 0.3 | 1.3 | 3.1 | 3.1 | 0.9 |
| Reflected Significant Wave Crest El. (feet, NVGD29) | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | | | | | |
| Maximum Wave Height (feet) | 1.7 | 0.5 | 2.2 | 4.5 | 4.5 | 1.5 |
| **Reflected Maximum Wave Crest El. (feet, NGVD29)** **PEAK FLOOD HAZARD** | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | | | | | |

Note that the reflected/standing wave crest elevation at each area of interest was calculated using the controlling wind transect for that specific location.

### 3.9.5 References

**ANO, 2016.** "Drawing Number C-2026A, Sheet 1, Underground Utilities Plot Plan, Revision 25", February 17, 2016. See AREVA Document No. 38- 9257436-000.

**ANS, 1992.** "ANSI/ANS-2.8-1992: American National Standard for Determining Design Basis Flooding at Nuclear Reactor Sites", American National Standards/American Nuclear Society, 1992.

**AREVA, 2013.** Arkansas Nuclear One Topographic Survey Deliverables", 2013, AREVA Document No. 38-9208201-001.

**AREVA, 2014a.** AREVA Document No. 32-9207387-000, "ANO Flooding Hazard Re-Evaluation – Probable Maximum Seiche Screening Level Calculation", 2014.

**AREVA, 2014b.** AREVA Document No. 51-9210160-000, "Arkansas Nuclear One Units 1 and 2 Flooding Hazard Re-Evaluation - Tsunami and Storm Surge", 2014.

**AREVA, 2016.** AREVA Document No. 32-9207388-000, "ANO Combined Effect Flood Analysis Calculation", 2016.

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

**Dean, 1991.** "Water Wave Mechanics for Engineers and Scientists", Dean, Robert G. and Dalrymple, Robert A., 1991.

**FEMA, 2011.** "Coastal Construction Manual: Principles and Practices of Planning, Siting, Designing, Constructing and Maintaining Residential Buildings in Coastal Areas", FEMA 55, 2011.

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

**Maidment, 1993.** "Handbook of Hydrology", Maidment, David R. February, 1993.

**NOAA, 2016a.** "GHCND:USW00013964– Fort Smith Regional Airport, AR US: Fastest 2-Minute Wind Speed Data", National Climatic Data Center, National Oceanic and Atmospheric Administration, http://www.ncdc.noaa.gov/cdo-web/, Date accessed April 20, 2016, Date modified April 20, 2016.

**NOAA, 2016b.** "GHCND: USW00053920 – Russellville Municipal Airport, AR US: Fastest 2-Minute Wind Speed Data", National Climatic Data Center, National Oceanic and Atmospheric Administration, http://www.ncdc.noaa.gov/cdo-web/, Date accessed April 20, 2016, Date modified April 20, 2016.

**OKDOT, 2012.** "Arkansas River Navigation System Inland Waterway Fact Sheet," Oklahoma Department of Transportation, 2012.

**USACE, 2005.** "Arkansas River Navigation Study, Final Environmental Impact Study, Chapter 4 Affected Environments", United States Army Corps of Engineers, August 2005.

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

**USACE, 2008a.** Meteorology and Wave Climate, Coastal Engineering Manual, Part II, Chapter 2, Engineering Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. August, 2008.

**USACE, 2008b.** Water Waves Mechanics, Coastal Engineering Manual, Part II, Chapter 1, Engineering Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. August, 2008.

**USACE, 2013a.** National Inventory of Dams, United States Army Corps of Engineers, http://geo.usace.army.mil/pgis/f?p=397:1:250834646188401::NO, Date accessed: March 6, 2014, Date modified: February 2013.

**USACE, 2013b.** "Project Data", United States Army Corps of Engineers Little Rock District Water Management, http://www.swl-wc.usace.army.mil/pages/Ark_riv_project_data.htm, Date accessed: March 7, 2014, Date modified: December 2013.

**USACE, 2013c.** "Monthly Reservoir Reports, Dardanelle Lock and Dam No. 10", United States Army Corps of Engineers Little Rock District Water Management, http://www.swl-wc.usace.army.mil/pages/mcharts.htm, Date accessed: March 7, 2014, Date modified: December 2013.

**USACE, 2014.** "McClellan-Kerr Arkansas River Navigation System", United States Army Corps of Engineers, Tulsa Rock District, http://www.swt.usace.army.mil/Missions?Navigation.aspx, Date accessed: January 10, 2014, Date modified: January 10, 2014.

**USACE, 2016.** "ANO USACE Hydrographs", United States Army Corps of Engineers, 2016. See AREVA Document No. 38-9257436-000.

**USGS, 2014.** "Stream Gage 07258000 Arkansas River at Dardanelle, AR, United States Geological Survey, http://waterdata.usgs.gov/usa/nwis/uv?07258000, Date accessed: June 8, 2016, Date modified June 8, 2016.

**Table 3.9-11: Peak Flow USGS Stream Gage 07258000**

| Year | Peak Flow (cfs) | Rank |
|---|---|---|
| 1943 | 683,000 | 1 |
| 1945 | 579,000 | 2 |
| 1957 | 471,000 | 3 |
| 1941 | 433,000 | 4 |
| 1990 | 433,000 | 5 |
| 1938 | 396,000 | 6 |
| 2015 | 393,000 | 7 |
| 1959 | 390,000 | 8 |
| 1950 | 382,000 | 9 |
| 2008 | 367,000 | 10 |
| 1986 | 338,000 | 11 |
| 2011 | 332,000 | 12 |
| 1982 | 325,000 | 13 |
| 1973 | 318,000 | 14 |
| 1946 | 303,000 | 15 |
| 1949 | 303,000 | 16 |
| 1973 | 302,000 | 17 |
| 2004 | 302,000 | 18 |
| 1948 | 300,000 | 19 |
| 1971 | 298,000 | 20 |
| 1941 | 295,000 | 21 |
| 1945 | 285,000 | 22 |
| 1993 | 279,000 | 23 |
| 1961 | 271,000 | 24 |
| 2012 | 262,000 | 25 |
| 1998 | 259,000 | 26 |
| 2002 | 258,000 | 27 |
| 2009 | 252,000 | 28 |
| 1944 | 245,000 | 29 |
| 1985 | 245,000 | 30 |
| 1987 | 244,000 | 31 |
| 1974 | 243,000 | 32 |
| 2001 | 240,000 | 33 |
| 2007 | 240,000 | 34 |
| 1970 | 235,000 | 35 |
| 1985 | 233,000 | 36 |

| Year | Peak Flow (cfs) | Rank |
|---|---|---|
| 1951 | 227,000 | 37 |
| 1977 | 221,000 | 38 |
| 1996 | 221,000 | 39 |
| 1989 | 218,000 | 40 |
| 2000 | 217,000 | 41 |
| 1966 | 209,000 | 42 |
| 1995 | 205,000 | 43 |
| 2005 | 205,000 | 44 |
| 1968 | 203,000 | 45 |
| 1969 | 200,000 | 46 |
| 1999 | 199,000 | 47 |
| 1954 | 194,000 | 48 |
| 1982 | 192,000 | 49 |
| 2009 | 192,000 | 50 |
| 1991 | 176,000 | 51 |
| 2013 | 176,000 | 52 |
| 1959 | 157,000 | 53 |
| 1958 | 156,000 | 54 |
| 1961 | 154,000 | 55 |
| 1994 | 152,000 | 56 |
| 1952 | 145,000 | 57 |
| 1970 | 145,000 | 58 |
| 1979 | 143,000 | 59 |
| 1939 | 142,000 | 60 |
| 1976 | 141,000 | 61 |
| 1984 | 139,000 | 62 |
| 1978 | 138,000 | 63 |
| 1996 | 138,000 | 64 |
| 1953 | 137,000 | 65 |
| 1965 | 127,000 | 66 |
| 2014 | 125,000 | 67 |
| 1991 | 124,000 | 68 |
| 1955 | 113,000 | 69 |
| 1955 | 109,000 | 70 |
| 2003 | 108,000 | 71 |
| 1940 | 103,000 | 72 |

| Year | Peak Flow (cfs) | Rank |
|---|---|---|
| 2006 | 96,900 | 73 |
| 1964 | 96,700 | 74 |
| 1967 | 88,300 | 75 |
| 1980 | 75,300 | 76 |
| 1981 | 63,100 | 77 |
| 1962 | 62,600 | 78 |

**Table 3.9-12: Transect Information**

| Transect | Locations | Length (feet) |
|---|---|---|
| 1 | Diesel Fuel Vault | 2,200 |
| 2 | PASS Building | 200 |
| 3 | Intake Structures – Southwest Side | 3,600 |
| 4 | Intake Structures – Southeast Side | 20,200 |
| 5 | Start Up 2 Transformer (SU-2) Bus Duct | 1,800 |

**Table 3.9-13: Wave Height and Peak Period**

| Location | Transect | Bottom Elevation (feet, NGVD29) | Flood Depth (feet) | Depth Limited Wave Height (feet) | Significant Wave Height (feet) | Peak Period (seconds) | Wavelength (feet) | Maximum Wave Height (feet) |
|---|---|---|---|---|---|---|---|---|
| Diesel Fuel Vault | 1 | 354.5 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | 1.0 | 1.4 | 9.7 | 1.7 |
| PASS Building | 2 | 354.7 | | | 0.3 | 0.6 | 2.0 | 0.5 |
| Intake Structure No. 1 – Southwest Side | 3 | 352.5 | | | 1.3 | 1.6 | 13.5 | 2.2 |
| Intake Structure No. 1 – Southeast Side | 4 | 352.7 | | | 3.1 | 2.9 | 35.3 | 5.2 |
| Intake Structure No. 2 – Southeast Side | 4 | 352.4 | | | 3.1 | 2.9 | 35.8 | 5.2 |
| Intake Structure No. 1 and 2 – Southeast Side (including Bridge at Intake Canal) | 4 | 353.8 | | | 3.1 | 2.9 | 35.3 | 5.2 |
| Start Up 2 Transformer (SU-2) Bus Duct | 5 | 353.6 | | | 0.9 | 1.3 | 8.6 | 1.5 |

Note: * - The depth-limited wave height at Intake Structure No. 1 on the southeast side and Intake Structure No. 2 on the southeast side were calculated in reference to the bridge deck over the intake canal.

**Table 3.9-14: Probable Maximum Water Surface Elevation at ANO**

| Location | Transect | Significant Wave Height (feet) | Reflected Significant Wave Crest Height (feet) | Reflected Significant Wave Crest Elevation (feet, NGVD29) | Maximum Wave Height (feet) | Reflected Maximum Wave Crest Height (feet) | Reflected Maximum Wave Crest Elevation (feet, NGVD29) |
|---|---|---|---|---|---|---|---|
| Diesel Fuel Vault | 1 | 1.0 | 1.3 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | 1.7 | 2.6 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) |
| PASS Building | 2 | 0.3 | 0.4 | | 0.5 | 0.9 | |
| Intake Structure No. 1 – Southwest Side | 3 | 1.3 | 1.7 | | 2.2 | 3.3 | |
| Intake Structure No. 1 – Southeast Side | 4 | 3.1 | 4.1 | | 4.5* | 6.7 | |
| Intake Structure No. 2 – Southeast Side | 4 | 3.1 | 4.1 | | 4.5* | 6.6 | |
| Start Up 2 Transformer (SU-2) Bus Duct | 5 | 0.9 | 1.2 | | 1.5 | 2.3 | |

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(

Notes:

1. The maximum stillwater elevation at each location is [redacted] feet, NGVD29.
2. * - the maximum wave height at Intake Structure No. 1 on the southeast side and Intake Structure No. 2 on the southeast side were calculated in reference to the bridge deck over the intake canal and are equal to the depth-limited wave height at the bridge.

**Table 3.9-15: Hydrostatic Load Results**

| | Still Water Elevation (feet) | Ground Surface Elevation (feet) | Depth (feet) | Hydrostatic force (pounds per linear foot) | Resultant Force Elevation (feet) | Maximum Hydrostatic Pressure (pounds per square foot) |
|---|---|---|---|---|---|---|
| Diesel Fuel Vault | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | 354.5 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) |
| PASS Building | | 354.7 | | | | |
| Intake Structure No. 1 – Southwest Side | | 352.5 | | | | |
| Intake Structure No. 1 – Southeast Side | | 352.7 | | | | |
| Intake Structure No. 2 – Southeast Side | | 352.4 | | | | |
| Start Up 2 Transformer (SU-2) Bus Duct | | 353.6 | | | | |

**Table 3.9-16: Flow Velocity Results**

| | Critical Velocity (feet/sec) |
|---|---|
| Diesel Fuel Vault | (b)(3):16 U.S.C. § 824o-1 (d),(b)(4),(b)(7)(F) |
| PASS Building | |
| Intake Structure No. 1 – Southwest Side | |
| Intake Structure No. 1 – Southeast Side | |
| Intake Structure No. 2 – Southeast Side | |
| Start Up 2 Transformer (SU-2) Bus Duct | |

**Table 3.9-17: Hydrodynamic Load Results**

| | Horizontal Drag Force (pounds) | Hydrodynamic Load (pounds per feet) | Resultant Force Elevation (feet) | Hydrodynamic Pressure (pounds per square feet) |
|---|---|---|---|---|
| Diesel Fuel Vault | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) | | | |
| PASS Building | | | | |
| Intake Structure No. 1 – Southwest Side | | | | |
| Intake Structure No. 1 – Southeast Side | | | | |
| Intake Structure No. 2 – Southeast Side | | | | |
| Start Up 2 Transformer (SU-2) Bus Duct | | | | |

**Table 3.9-18: Debris Impact Results**

| | Impact Force (pounds) |
|---|---|
| Diesel Fuel Vault | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) |
| PASS Building | |
| Intake Structure No. 1 – Southwest Side | |
| Intake Structure No. 1 – Southeast Side | |
| Intake Structure No. 2 – Southeast Side | |
| Start Up 2 Transformer (SU-2) Bus Duct | |

**Table 3.9-19: Standing Wave Loads**

| | Significant Wave | | Maximum Wave | |
|---|---|---|---|---|
| | Wave pressure at the still water level (pounds/feet$^2$) | Wave pressure at the base of Structure (pounds/feet$^2$) | Wave pressure at the still water level (pounds/feet$^2$) | Wave pressure at the base of Structure (pounds/feet$^2$) |
| Diesel Fuel Vault | 66 | 6 | 108 | 10 |
| PASS Building | 25 | <1 | 47 | <1 |
| Intake Structure No. 1 – Southwest Side | 86 | 7 | 141 | 12 |
| Intake Structure No. 1 – Southeast Side | 201 | 111 | 288 | 161 |
| Intake Structure No. 2 – Southeast Side | 201 | 108 | 287 | 156 |
| Start Up 2 Transformer (SU-2) Bus Duct | 62 | 2 | 104 | 3 |

**Table 3.9-20: Combined Flood Load Results**

| Structure | Location | Hydrostatic Pressure (pounds per square foot) | Hydrodynamic Pressure (pounds per square foot) | Significant Wave Height | | Maximum Wave Height | | Debris Load Considering a Log of 2,000 lb (pounds) |
|---|---|---|---|---|---|---|---|---|
| | | | | Standing Wave Pressure (pounds per square foot) | Composite Pressure (pounds per square foot) | Standing Wave Pressure (pounds per square foot) | Composite Pressure (pounds per square foot) | |
| Diesel Fuel Vault | Stillwater Elevation | 0 | (b)(3):16 U.S.C. § 824o-1(d), (b)(4),(b)(7)(F) | 66 | 249 | 108 | 291 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 6 | 482 | 10 | 486 | 0 |
| PASS Building | Stillwater Elevation | 0 | | 25 | 200 | 47 | 222 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 0 | 456 | 0 | 456 | 0 |
| Intake Structure No. 1 – Southwest Side | Stillwater Elevation | 0 | | 86 | 348 | 141 | 403 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 7 | 687 | 12 | 692 | 0 |
| Intake Structure No. 1 – Southeast Side | Stillwater Elevation | 0 | | 201 | 456 | 288 | 543 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 111 | 772 | 161 | 822 | 0 |
| Intake Structure No. 2 – Southeast Side | Stillwater Elevation | 0 | | 201 | 467 | 287 | 553 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 108 | 798 | 156 | 846 | 0 |
| Start Up 2 Transformer (SU-2) Bus Duct | Stillwater Elevation | 0 | | 62 | 280 | 104 | 322 | (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)( |
| | Bottom of Structure | | | 2 | 569 | 3 | 570 | 0 |

Notes: 1. Debris loads assumed to act at the maximum stillwater elevation.
2. The composite pressure at a given location is the sum of the hydrostatic pressure, current velocity hydrodynamic pressure and standing wave pressure.

**Figure 3.9-25: Peak Flow USGS Stream Gage 07258000**

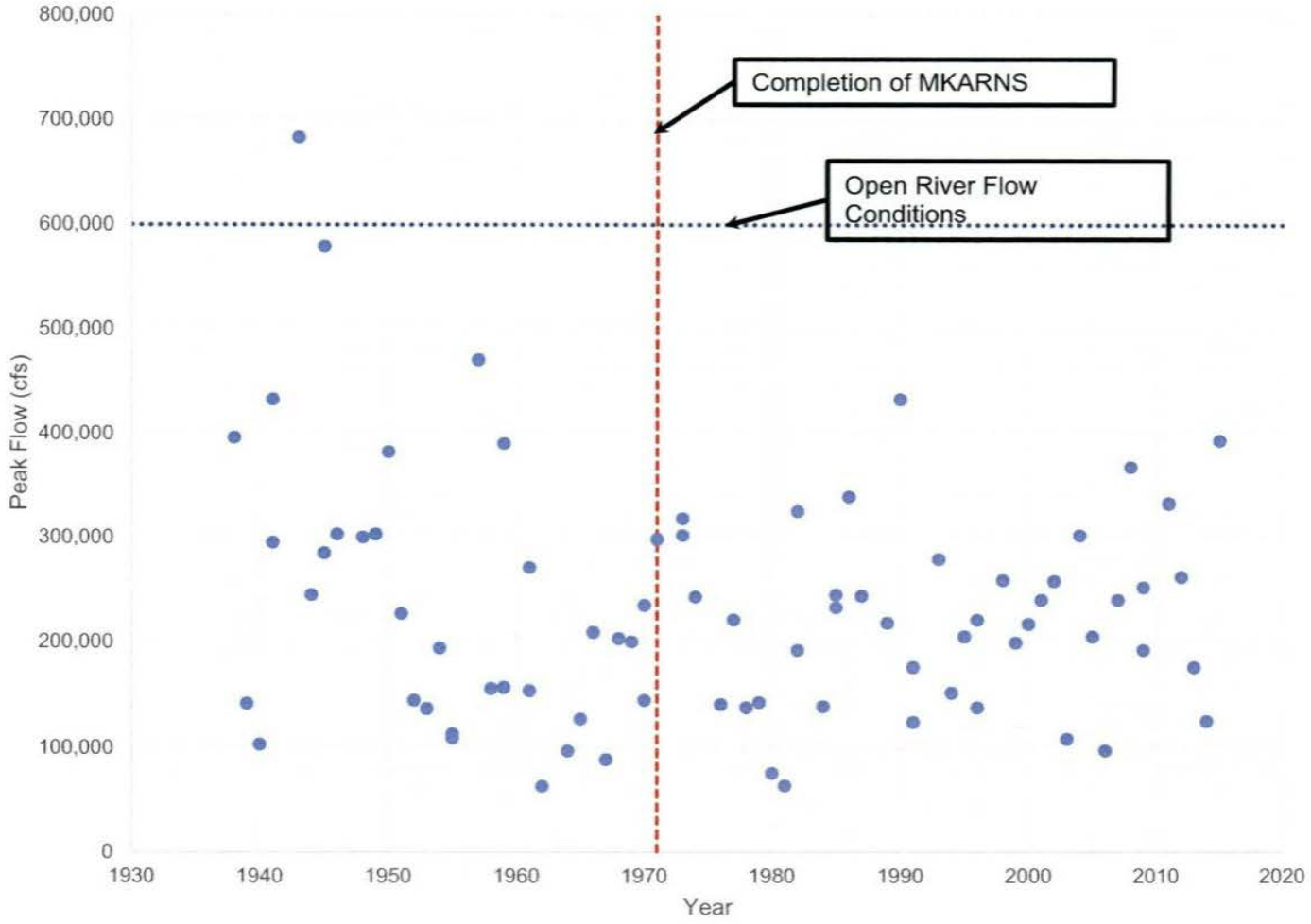


**Figure 3.9-26: USACE-Provided Hydrograph Locations**

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

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

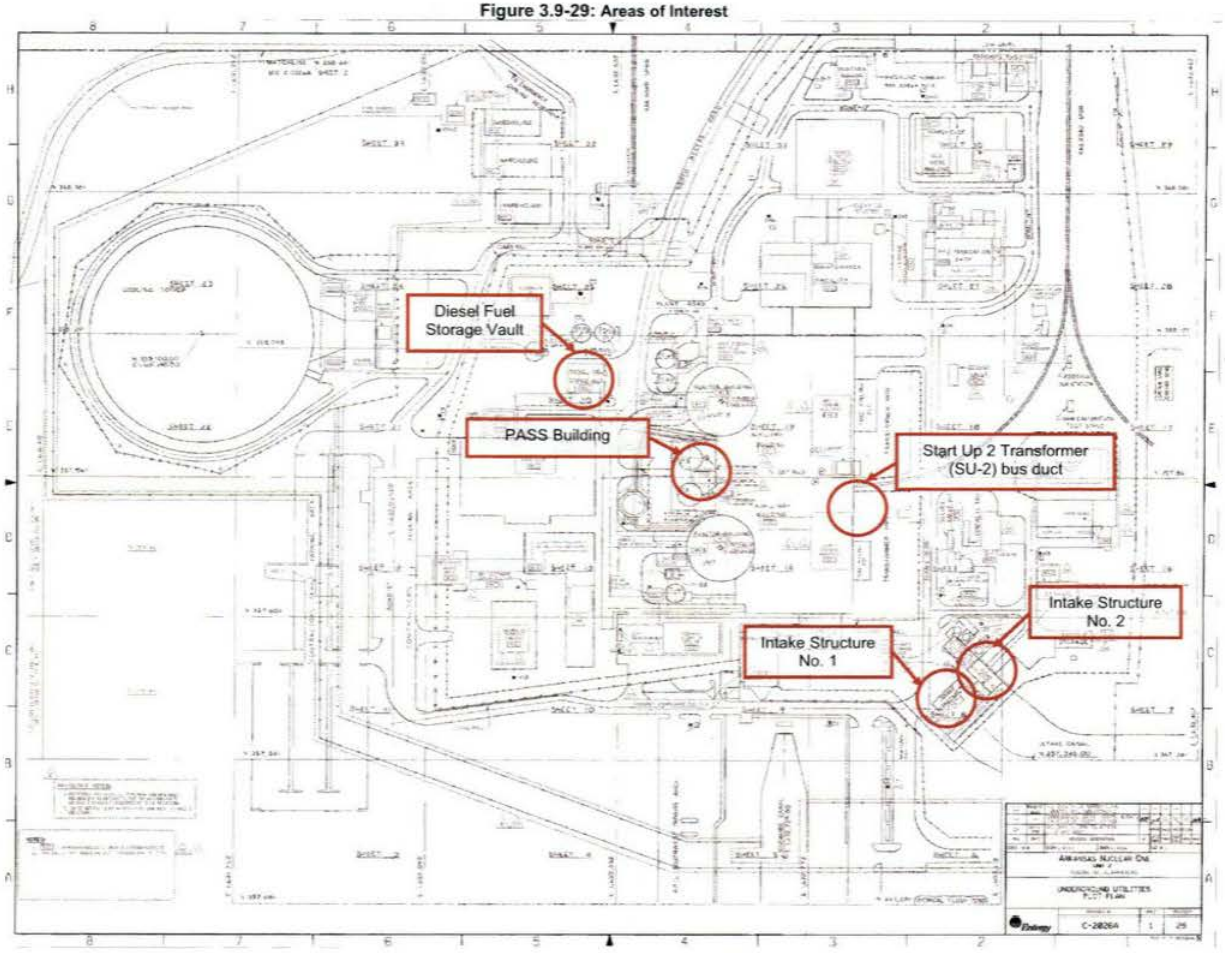
**Figure 3.9-27: Transects for Wave Calculations (Lake Dardanelle)**

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

**Figure 3.9-28: Transects for Wave Calculations (Site)**

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

**Figure 3.9-29: Areas of Interest**




Source: (ANO, 2016). Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

## 4.0 FLOOD PARAMETERS AND COMPARISON WITH CURRENT DESIGN BASIS

Per the March 12, 2012, 50.54(f) letter (NRC, 2012a), Enclosure 2, the following flood-causing mechanisms were considered in the flood hazard reevaluation for ANO.

1. Local Intense Precipitation;
2. Flooding in Streams and Rivers;
3. Dam Breaches and Failures;
4. Storm Surge;
5. Seiche;
6. Tsunami;
7. Ice Induced Flooding; and
8. Channel Migration or Diversion.

Some of these individual mechanisms are incorporated into alternative 'Combined Effect Flood' scenarios per Appendix H of NUREG/CR-7046 (NRC, 2011).

Based on the results of the flood hazard re-evaluation, the ANO design basis flood level and associated design basis protections are potentially challenged by LIP and the combined effect flood scenario of Wind-Generated Waves coincident with Arkansas River PMF and Upstream Dam Failure.

### 4.1 Summary of Current Design Basis and Flood Reevaluation Results

A summary of the CDB flood elevations is provided in Table 4-1. A summary of the re-evaluated flood hazards that have potential to impact the site (not screened out) is provided in Table 4-2.

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

The re-evaluated PMF elevation for the Arkansas River (evaluated by the USACE) of 358.0 ft NGVD29 is the same as the CDB flood elevation for the Arkansas River PMF.

A discussion of results for LIP is provided in Section 4.1.1. Potential for impacts of the flooding due to LIP are addressed in Section 5.0.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

A comparison of the CDB controlling stillwater flood height to the reevaluated controlling stillwater flood height yields a reduction from [redacted] ft NGVD29 to [redacted] ft NGVD29. This comparison is for the PMF coincident with Dam Failure, evaluated in both the CDB and the Re-Evaluation.

A discussion of results for the combined effect flood scenario including wind-generated waves is provided in Section 4.1.2. Impacts of the combined effect flood scenario including wind-generated waves are addressed in Section 5.0.

#### 4.1.1 LIP

Local Intense Precipitation was addressed, but not specifically evaluated as part of the CDB. As a result, flooding due to LIP as evaluated as part of the flood hazard re-evaluation is not considered bound by the CDB. The peak flood levels resulting from LIP are well below the flood levels resulting from the controlling flood mechanism, Arkansas River PMF coincident with Dam Failure and Wind-Generated Waves. By nature, the LIP event may not have as much warning time as a large scale Arkansas River flood, and active or temporary features credited for flood protection during the controlling combined effect and riverine flood event may not be in place or available

during an LIP event. Potential for impacts due to LIP induced flooding was evaluated and is discussed in Section 5.2.

### 4.1.2 Wind-Generated Waves During Combined Effects

The CDB addressed a single wind-generated wave scenario, which is the Arkansas River PMF (358.0 ft NGVD29 stillwater) coincident with Wind-Generated Waves resulting in a wave level of 360.5 ft NGVD29 and splash effects (wave runup) not exceeding 368 ft NGVD29. This flood hazard elevation (368 ft NGVD29) represents the peak design basis flood hazard for ANO Units 1 and 2. Protection from splash effects (wave runup) is identified in the CDB as being up to 369 ft NGVD29.

The re-evaluated flood hazard scenario including wind-generated waves is more conservative than the CDB wind-generated wave scenario, as it includes multiple wind directions and a dam failure component. The re-evaluated hazard for combined effects including wind-generated waves is the Arkansas River PMF with upstream dam failure (still water elevation [redacted] ft NGVD29) with waves (including runup) up to [redacted] NGVD29. This flood hazard elevation ([redacted] ft NGVD29) represents the peak re-evaluated flood hazard for ANO Units 1 and 2.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

A comparison of the CDB to the reevaluated hazard for wind generated waves on top of the stillwater flood level is a reduction from [redacted] ft NGVD29 (PMF coincident with waves, including runup) to [redacted] ft NGVD29, therefore the CDB bounds the reevaluated hazard in terms of peak flood levels and flood hazard.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Potential for wind-generated waves during the controlling re-evaluated combined effect scenario to impact SSCs important to safety is addressed in Section 5.1.

## 4.2 References

**ANO, 2016.** Arkansas Nuclear One – Unit 2, Safety Analysis Report, Facility Operating License Number NPF-6, Docket Number 50-369, See AREVA Document No. 38-9257436-000, 2016.

**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.

**NRC, 2012a**. "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.

**NRC, 2012b**. "JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, Interim Staff Guidance", Revision 0, 2012. (ADAMS Accession No. ML12311A214)

**Table 4-1: Current Design Basis Flood Hazards**

| Mechanism | Stillwater Elevation (ft, NGVD29) | Wave Runup (wave height) (ft) | Design Basis Peak Hazard Elevation (ft, NGVD29) |
|---|---|---|---|
| **Local Intense Precipitation** | Not included in CDB | Not included in CDB | Not included in CDB |
| **Streams and Rivers** | | | |
| PMF on Arkansas River | 358.0 ft | Minimal | 358.0 ft |
| **Combined Effect Scenarios** | | | |
| **PMF on Arkansas River with Coincident Wind-Generated Waves** | 358.0 ft | **10 ft** (2.5 ft) | **368.0 ft** |
| Arkansas River PMF + Ozark Dam Failure | [redacted] ft | Not included in CDB | [redacted] ft |
| Arkansas River ½ PMF + Ozark Dam Failure | Evaluated, not bounding* | Evaluated, not bounding* | Evaluated, not bounding* |
| Arkansas River ½ PMF + Robert S. Kerr Dam Failure | Evaluated, not bounding* | Evaluated, not bounding* | Evaluated, not bounding* |
| **Storm Surge** | Not included in CDB | Not included in CDB | Not included in CDB |
| **Seiche** | Not included in CDB | Not included in CDB | Not included in CDB |
| **Tsunami** | Not included in CDB | Not included in CDB | Not included in CDB |
| **Ice-Induced Flooding** | Not included in CDB | Not included in CDB | Not included in CDB |
| **Channel Migration or Diversion** | Not included in CDB | Not included in CDB | Not included in CDB |

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

*Note: These flood scenarios were evaluated in the CDB and river flow rates were determined to be less than the river flow rates from the controlling combined effect flood scenario (PMF + Ozark Dam Failure). Flood hazard elevations for these scenarios were not determined, and are bound by the controlling combined event flood elevation of [redacted] ft NGVD29.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Note: Bolded results indicate the controlling design basis flood parameter for stillwater elevation, wave runup, and peak flood hazard elevation.

**Table 4-2: Re-Evaluated Flood Hazards for Flood Causing Mechanisms**

| Mechanism | Stillwater Elevation (ft, NGVD29) | Wave Runup (wave height) (ft) | Re-Evaluated Peak Flood Hazard Elevation (ft, NGVD29) |
|---|---|---|---|
| ***Local Intense Precipitation*** | | | |
| East of Cooling Tower | 351.4 ft | Minimal | 351.4 ft |
| West of Warehouse | 351.2 ft | Minimal | 351.2 ft |
| South of Warehouse | 351.4 ft | Minimal | 351.4 ft |
| West of Diesel Oil Storage Tank | 354.5 ft | Minimal | 354.5 ft |
| West of Engineering / Modification Building | 352.2 ft | Minimal | 352.2 ft |
| Between Engineering / Modification Building and Reactor Building Unit 1 | 352.7 ft | Minimal | 352.7 ft |
| East of Diesel Fuel Storage Vault | 353.7 ft | Minimal | 353.7 ft |
| Between Warehouse and Reactor Building Unit 2 | 355.0 ft | Minimal | 355.0 ft |
| West of Maintenance Building | 353.7 ft | Minimal | 353.7 ft |
| North of Turbine Building Unit 2 | 353.7 ft | Minimal | 353.7 ft |
| South of Turbine Building Unit 2 | 355.1 ft | Minimal | 355.1 ft |
| South of Central support Building | 354.0 ft | Minimal | 354.0 ft |
| North of Central Support Building | 357.7 ft | Minimal | 357.7 ft |
| **North Train Bay Door** | **354.4 ft** | **Minimal** | **354.4 ft** |
| **South Train Bay Door** | **354.4 ft** | **Minimal** | **354.4 ft** |
| Northeast of Turbine Building Unit 2 | 354.4 ft | Minimal | 354.4 ft |
| Transformer Yard | 354.4 ft | Minimal | 354.4 ft |
| East of Turbine Building Unit 1 | 354.3 ft | Minimal | 354.3 ft |
| Northwest of Intake Structure | 354.1 ft | Minimal | 354.1 ft |
| North of Intake Structure | 354.2 ft | Minimal | 354.2 ft |
| North of Independent Spent Fuel Storage Installation | 356.3 ft | Minimal | 356.3 ft |
| South of Independent Spent Fuel Storage Installation | 355.6 ft | Minimal | 355.6 ft |
| ***Streams and Rivers*** | | | |
| PMF on Arkansas River | 358.0 ft | Minimal | 358.0 |
| ***Combined Effect Scenarios*** | | | |
| Arkansas River PMF + Upstream Dam Failure | [redacted] ft | Minimal | [redacted] |
| **Arkansas River PMF + Upstream Dam Failure + Wind Generated Waves** | [redacted] **ft** | **6.7 ft** (4.5 ft) | [redacted] **ft** |

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Note: Flood mechanisms determined to result in no potential hazard for the site are not included in this table.

Note: Bolded results for LIP results indicate locations where there are potential impacts. Bolded results for Combined Effect Scenarios indicate the controlling re-evaluated parameters for stillwater elevation, wave runup, and peak flood hazard elevation.

# 5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED[1]

## 5.1 Potential Impacts of Combined Effects on External Flood Features

The CDB addressed a single wind-generated wave combined effect scenario, which is the Arkansas River PMF (358.0 ft NGVD29 stillwater) coincident with Wind-Generated Waves from a single direction and depth limited, resulting in a wave level of 360.5 ft NGVD29 and splash effects (wave runup) not exceeding 368 ft NGVD29. Protection from flooding is identified in the CDB as being up to [redacted] ft and protection from splash effects (wave runup) is identified in the CDB as being up to 369 ft NGVD29. A comparison of the CDB (PMF coincident with dam failure; PMF coincident with wind-generated waves) to the reevaluated hazard (PMF coincident with dam failure and wind-generated waves from multiple directions) reveals a reduction from [redacted] ft NGVD29 to [redacted] ft NGVD29, therefore the CDB bounds the reevaluated hazard in terms of flood hazard. However, the wave heights exceed the CDB described top of wave elevation of 360.5 ft NGVD29. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Due to the higher waves from multiple directions (without runup), evaluated as part of the flood hazard re-evaluation, areas above [redacted] ft NGVD were evaluated to determine if there were any locations in the CDB "splash protection" elevation range that might be vulnerable to the waves. The re-evaluated hazard for combined effects that includes wind-generated waves (including runup) is up to [redacted] ft NGVD29 and varies by specific location. The ANO Flood Protection Design Basis Calculation, CALC-ANOC-CS-15-00003, identifies the credited splash protection features from [redacted] ft NGVD29 to [redacted] ft NGVD29. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

CALC-ANOC-CS-15-00003 identifies only 2 credited splash protection features above [redacted] ft NGVD29 and below [redacted] ft NGVD29 which are potentially impacted by the re-evaluated wave hazard. The flood hazard re-evaluation evaluated specific wave heights for different areas of the ANO site to account for structures and features which might limit wave activity. The two features identified as vulnerable are located at the PASS building. At this location, the peak wave height is [redacted] ft NGVD29, which is below the CDB minimum flood protected elevation of [redacted] ft NGVD29. Therefore, the increased wave heights will have no impact on the splash protection features. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

As a result, no interim actions are required for the impacts of the Combined Effects.

## 5.2 Potential Impacts of Local Intense Precipitation

Potential impacts due to LIP flooding were evaluated for the site. Table 3.1-4 shows the maximum water surface elevation throughout the power block. Areas that were potentially vulnerable to flooding (maximum water surface elevation above 354 ft NGVD29) were evaluated and it was determined that the only location where water ingress may potentially impact SSCs important to safety is the Turbine Building via the Train Bay Doors. The LIP model was used to dertermine specific flood parameters at that location and a maximum water surface elevation of 354.4 ft NGVD29 was determined (see Section 3.1). The potential pathways to SSCs important to safety were evaluated, and it was determined that no SSCs important to safety were impacted by LIP flood water ingress into the Turbine Building via the Train Bay Doors. Temporary protection features credited during the CDB controlling flood event were not credited as part of this evaluation.

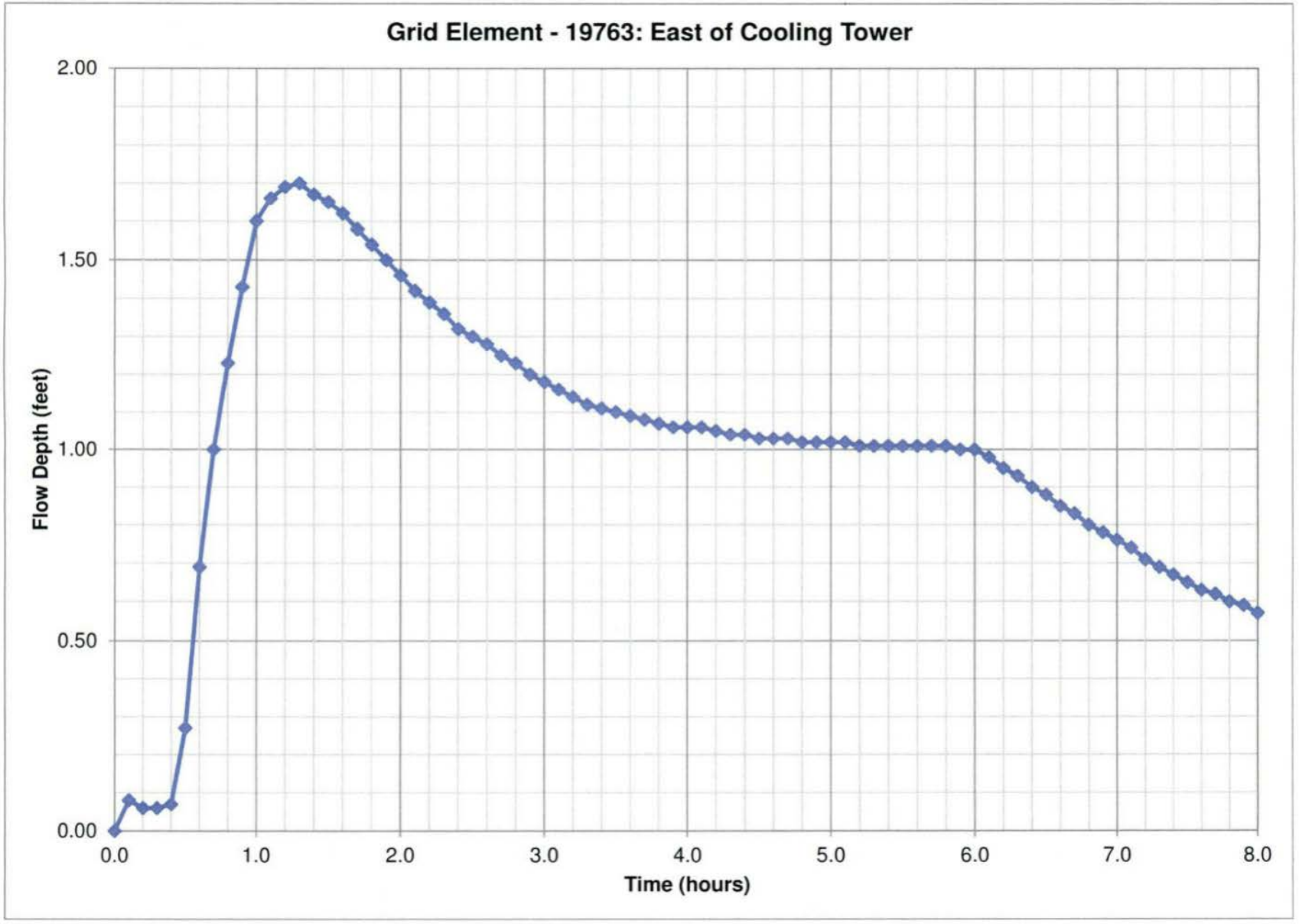
As a result, no interim actions are required for LIP flooding.

[1] Contents of this Section were provided in whole by Entergy, and approved by the project manager for use (see Signature Page).

## 6.0 ADDITIONAL ACTIONS

No additional actions are necessary.

# APPENDIX A: LOCAL INTENSE PRECIPITATION TIME-SERIES PLOTS


Grid Element - 19763: East of Cooling Tower
Flow Depth (feet)
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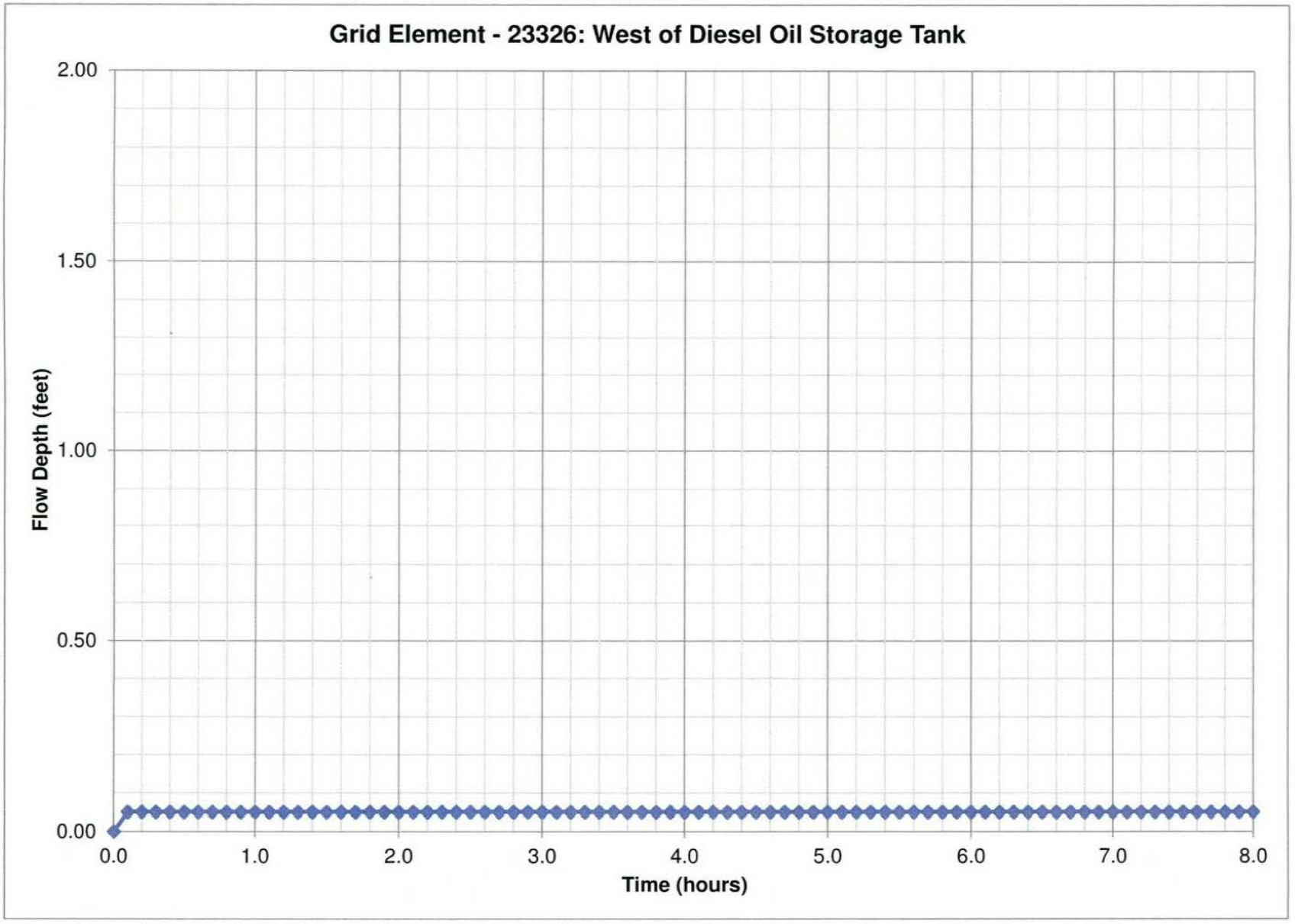
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 23326: West of Diesel Oil Storage Tank
Flow Depth (feet)
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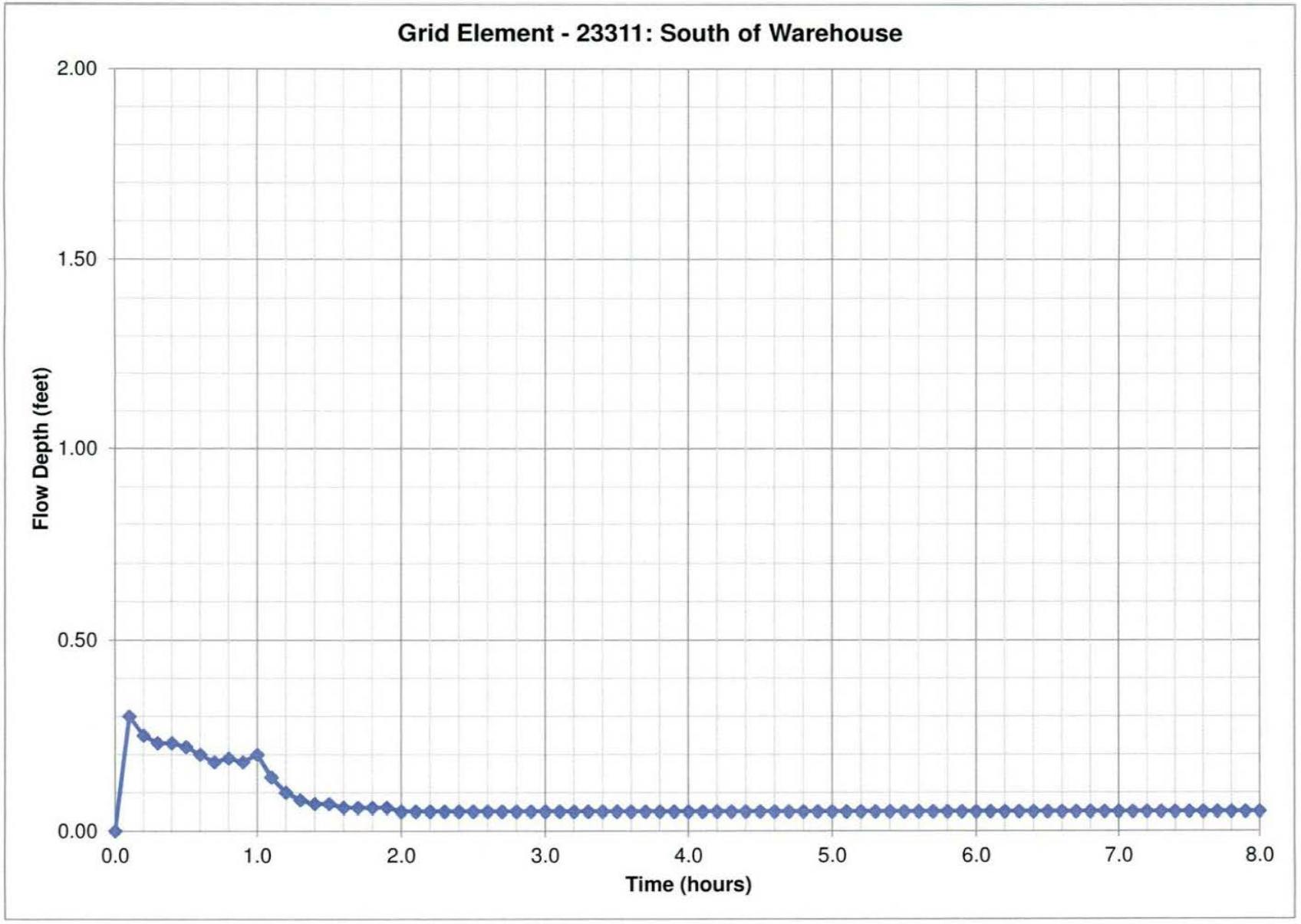
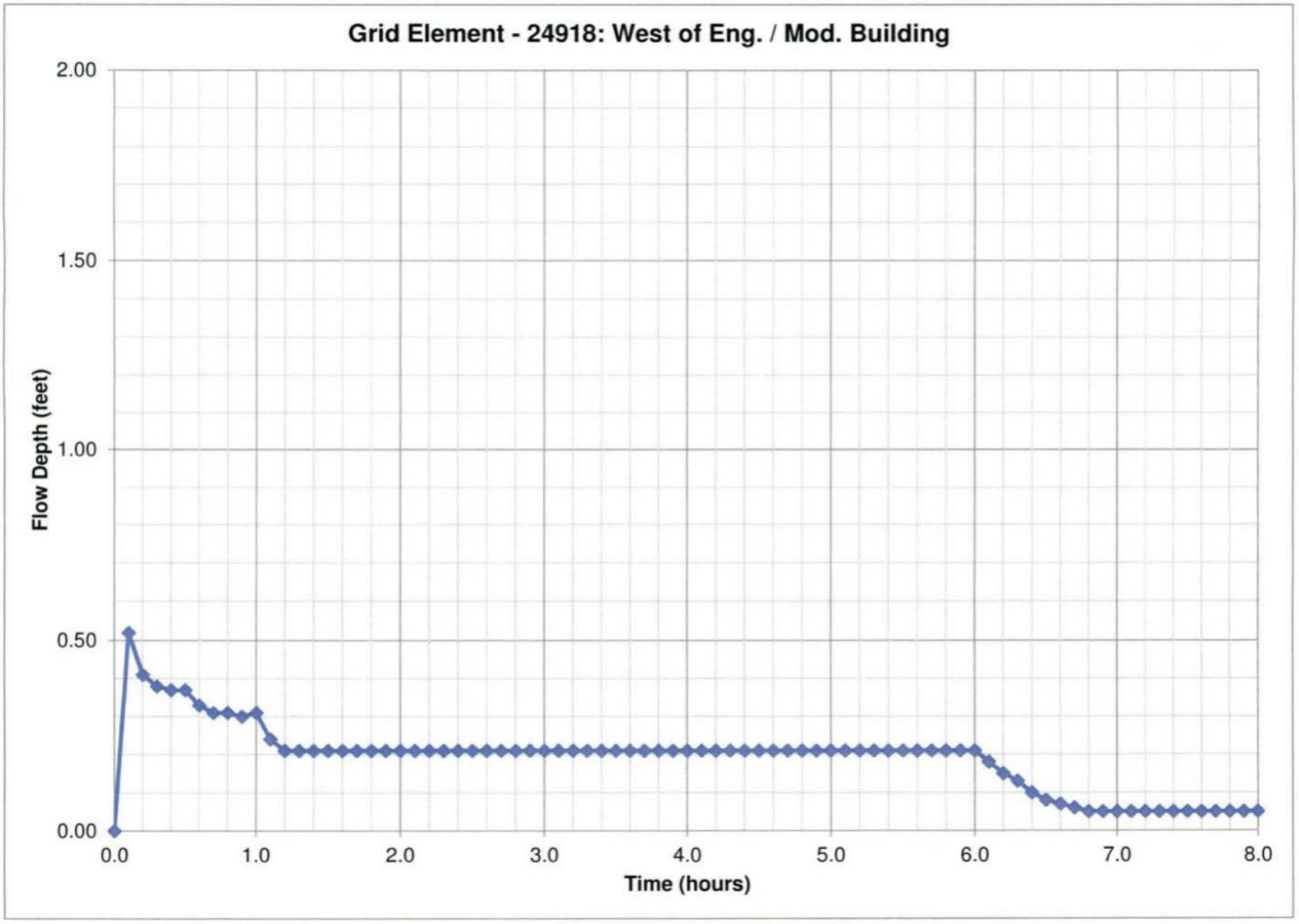
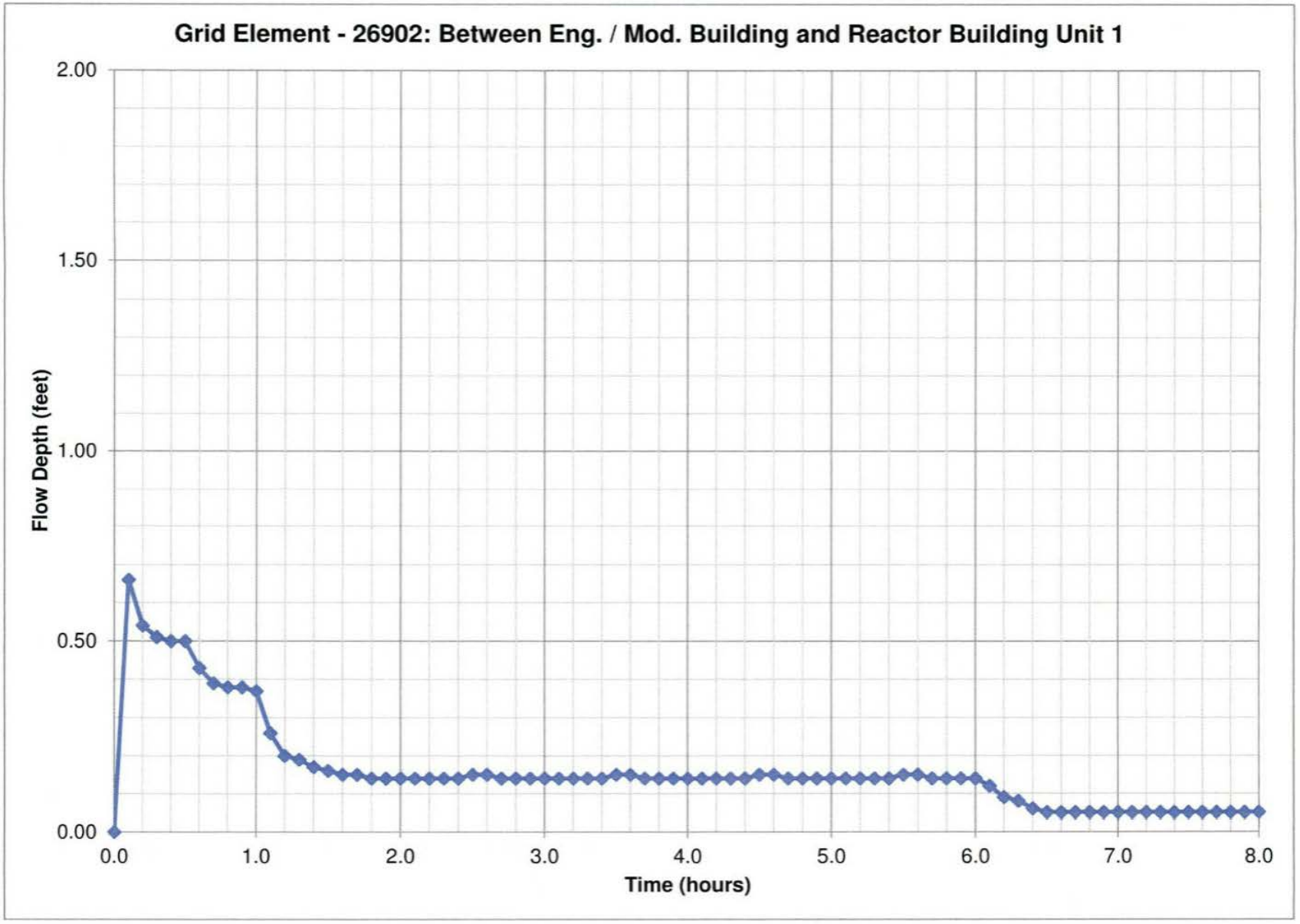
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)

Grid Element - 21787: West of Warehouse
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 23311: South of Warehouse
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 24918: West of Eng. / Mod. Building
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 26902: Between Eng. / Mod. Building and Reactor Building Unit 1
Flow Depth (feet)
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1.50
1.00
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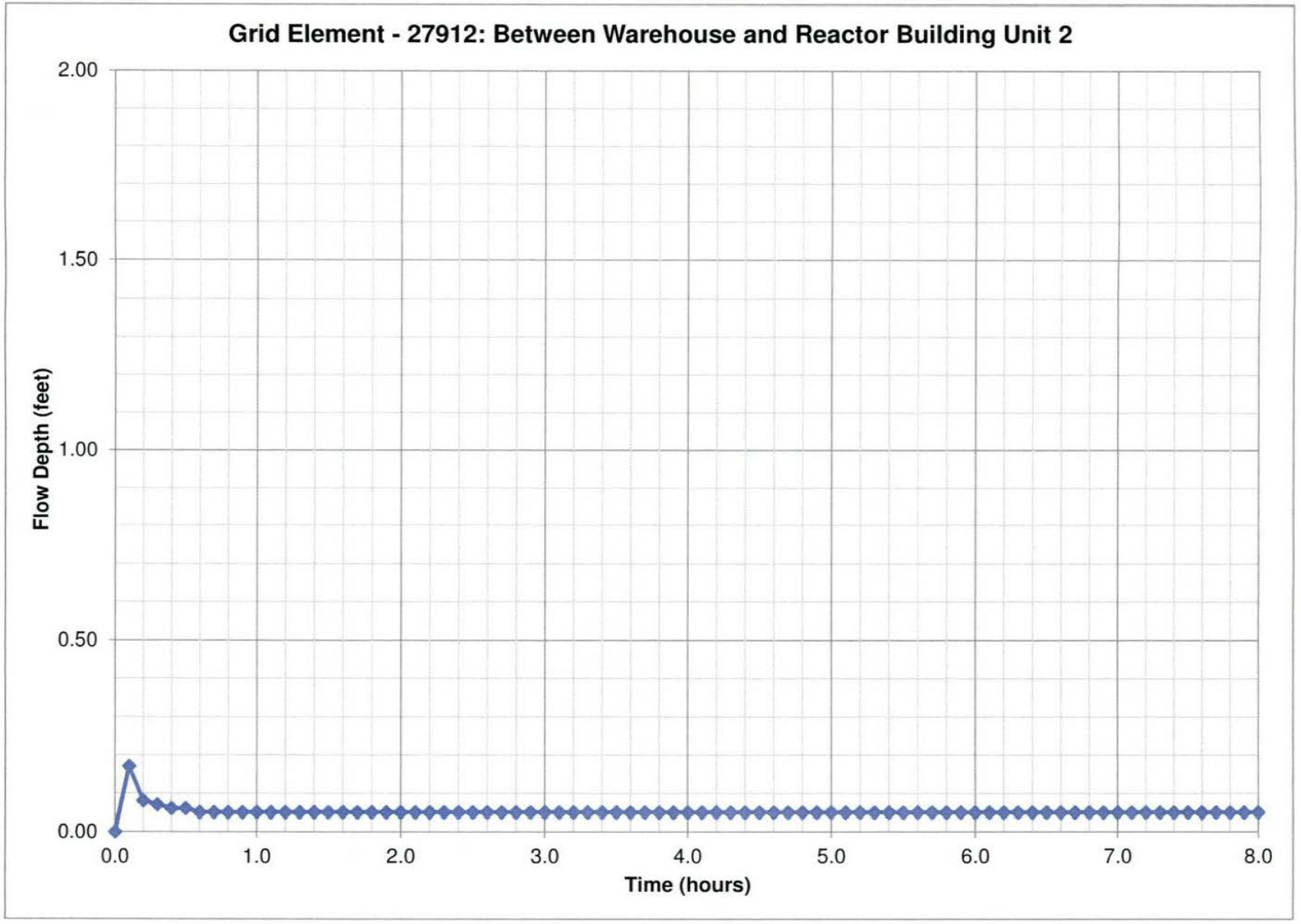
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 27912: Between Warehouse and Reactor Building Unit 2
Flow Depth (feet)
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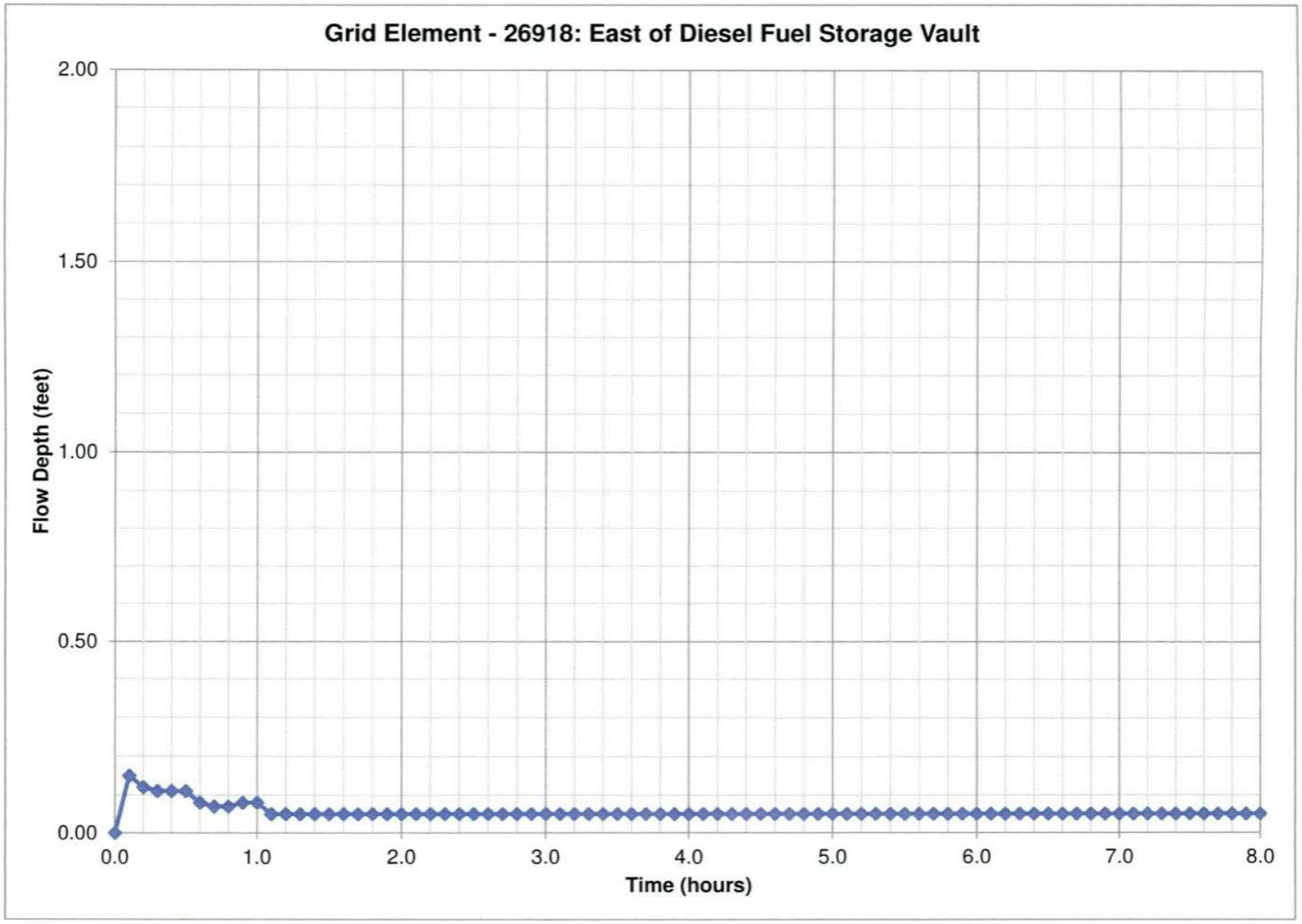
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 26918: East of Diesel Fuel Storage Vault
Flow Depth (feet)
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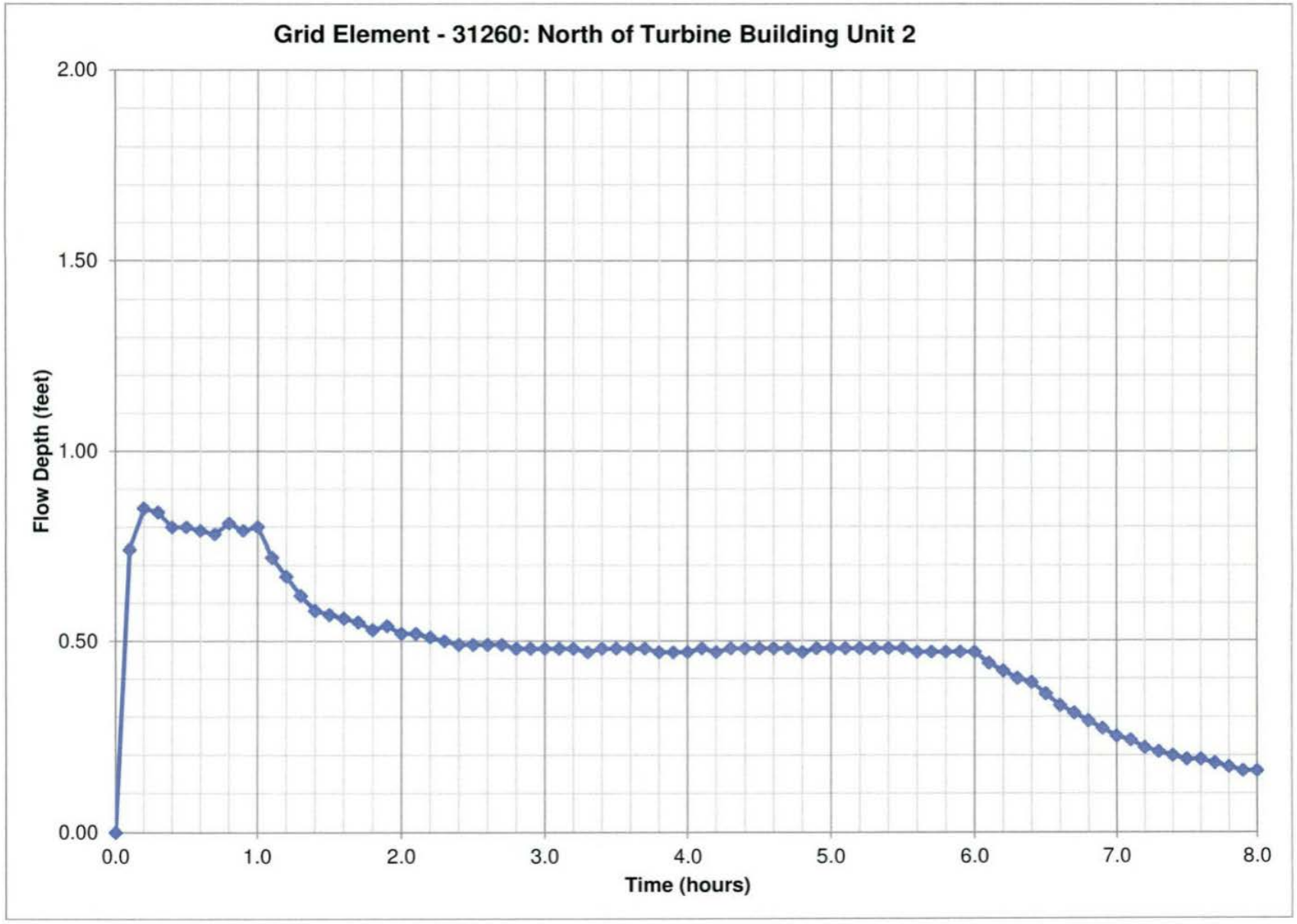
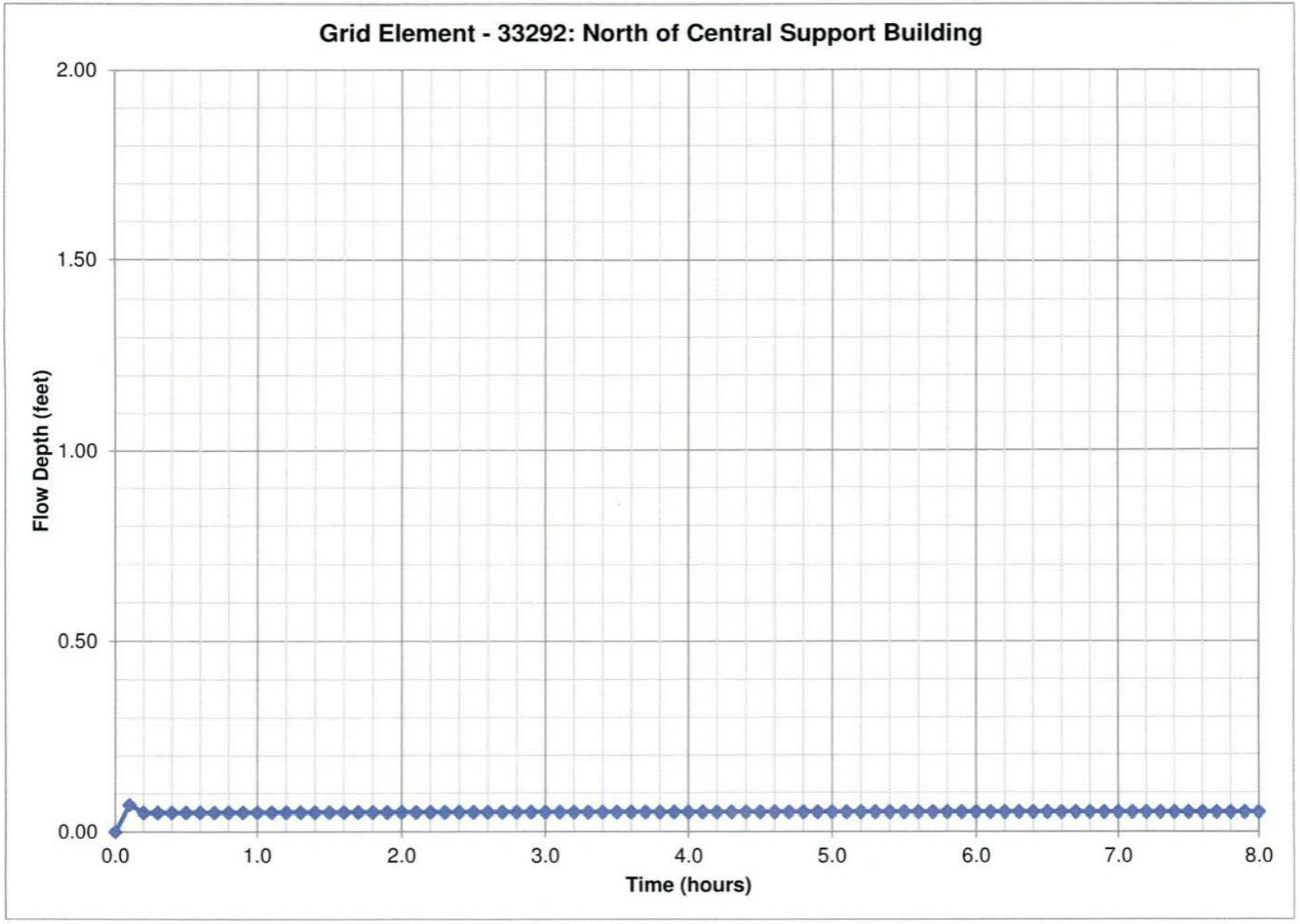
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 31260: North of Turbine Building Unit 2
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)

Grid Element - 30261: West of Maintenance Building
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 33292: North of Central Support Building
Flow Depth (feet)
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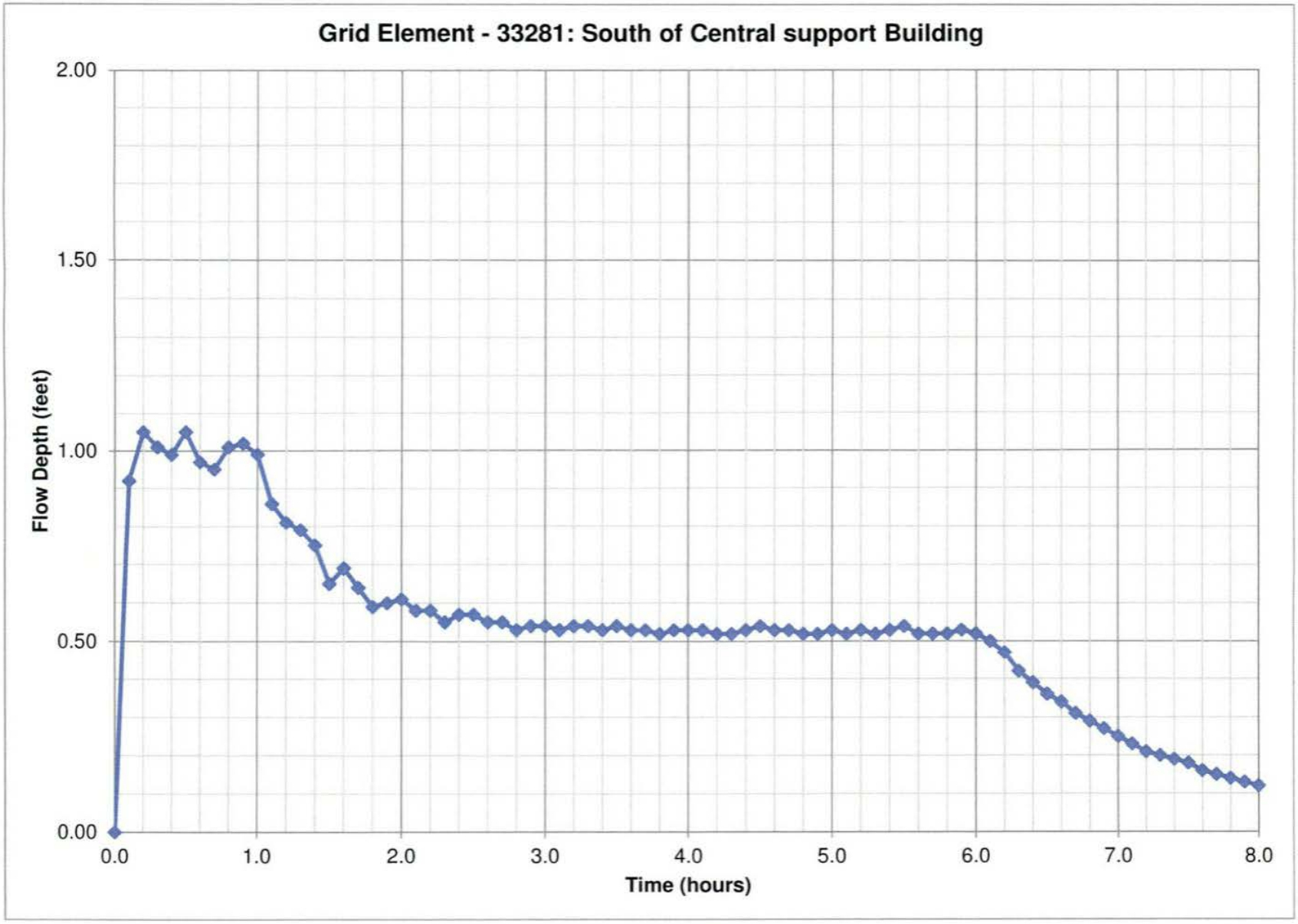
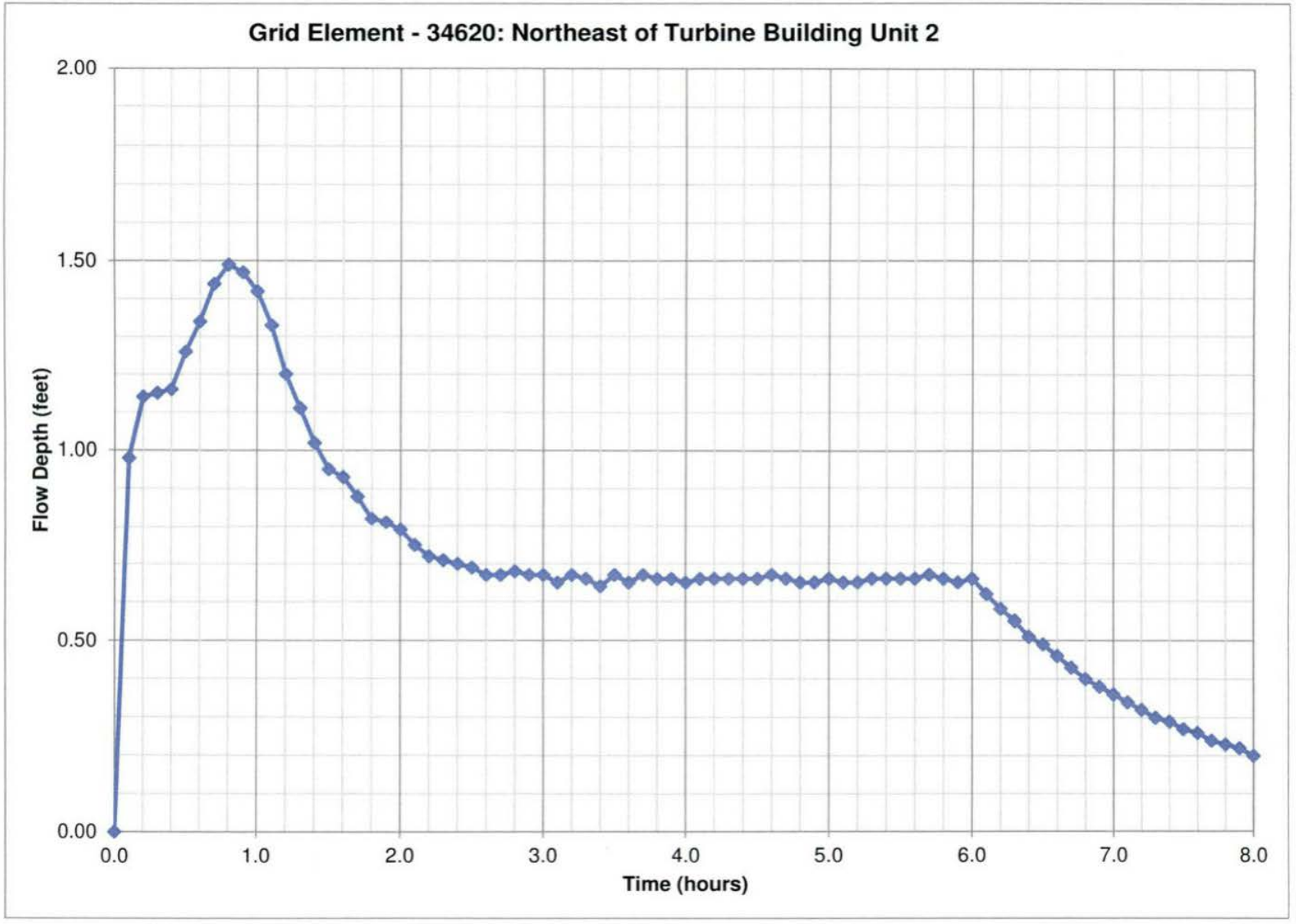
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 33281: South of Central support Building
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 34620: Northeast of Turbine Building Unit 2
Flow Depth (feet)
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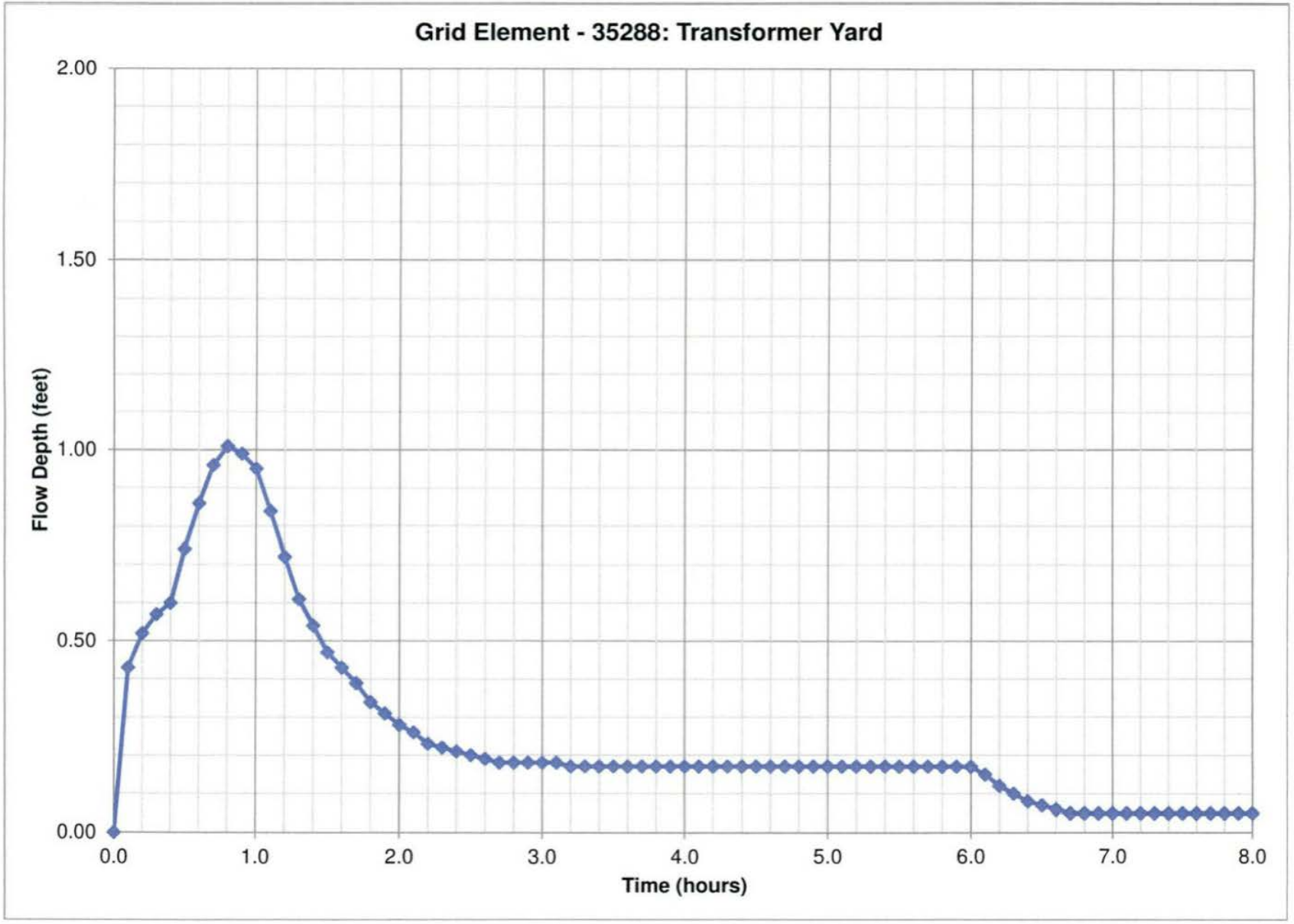
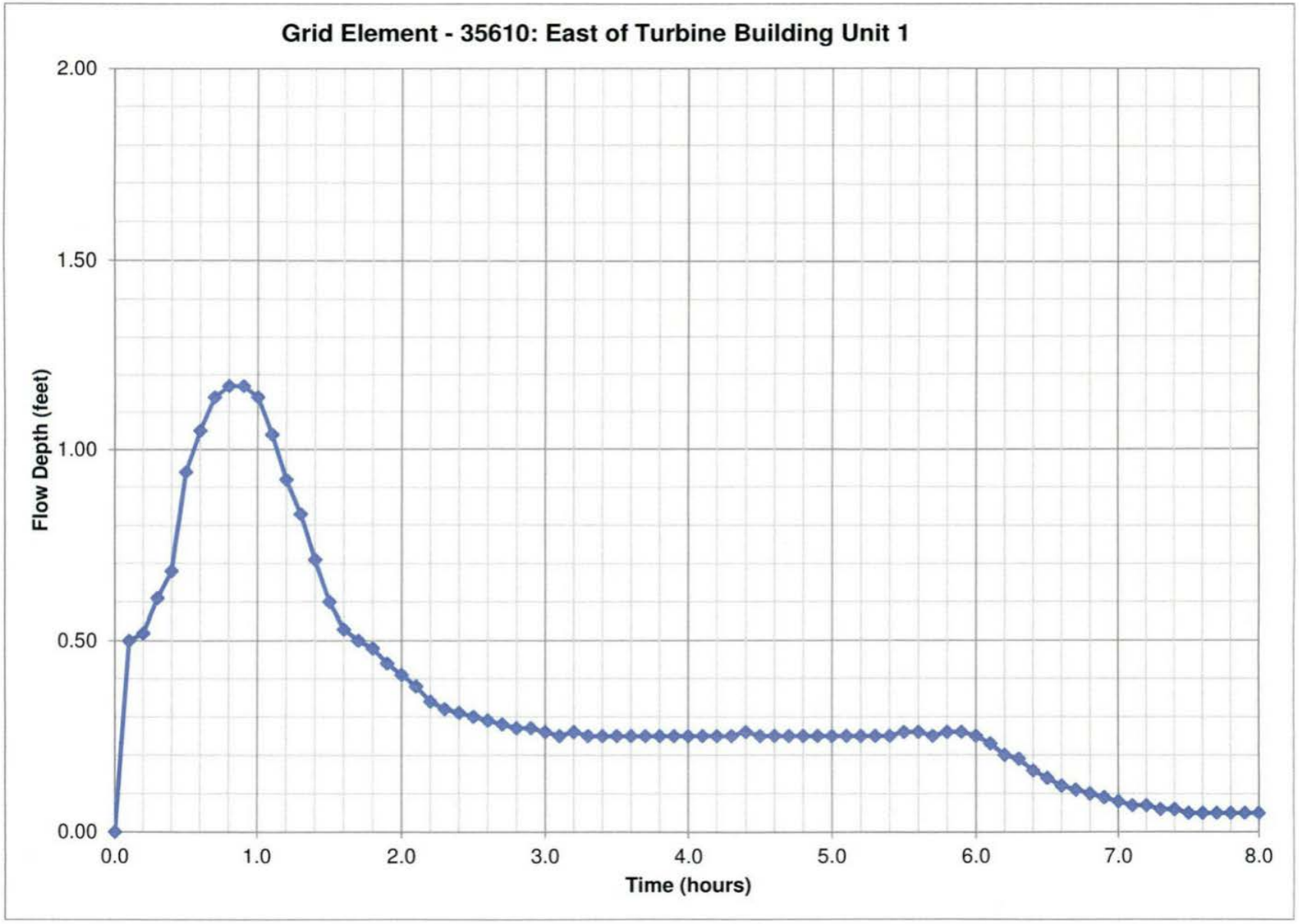
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 35288: Transformer Yard
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 35610: East of Turbine Building Unit 1
Flow Depth (feet)
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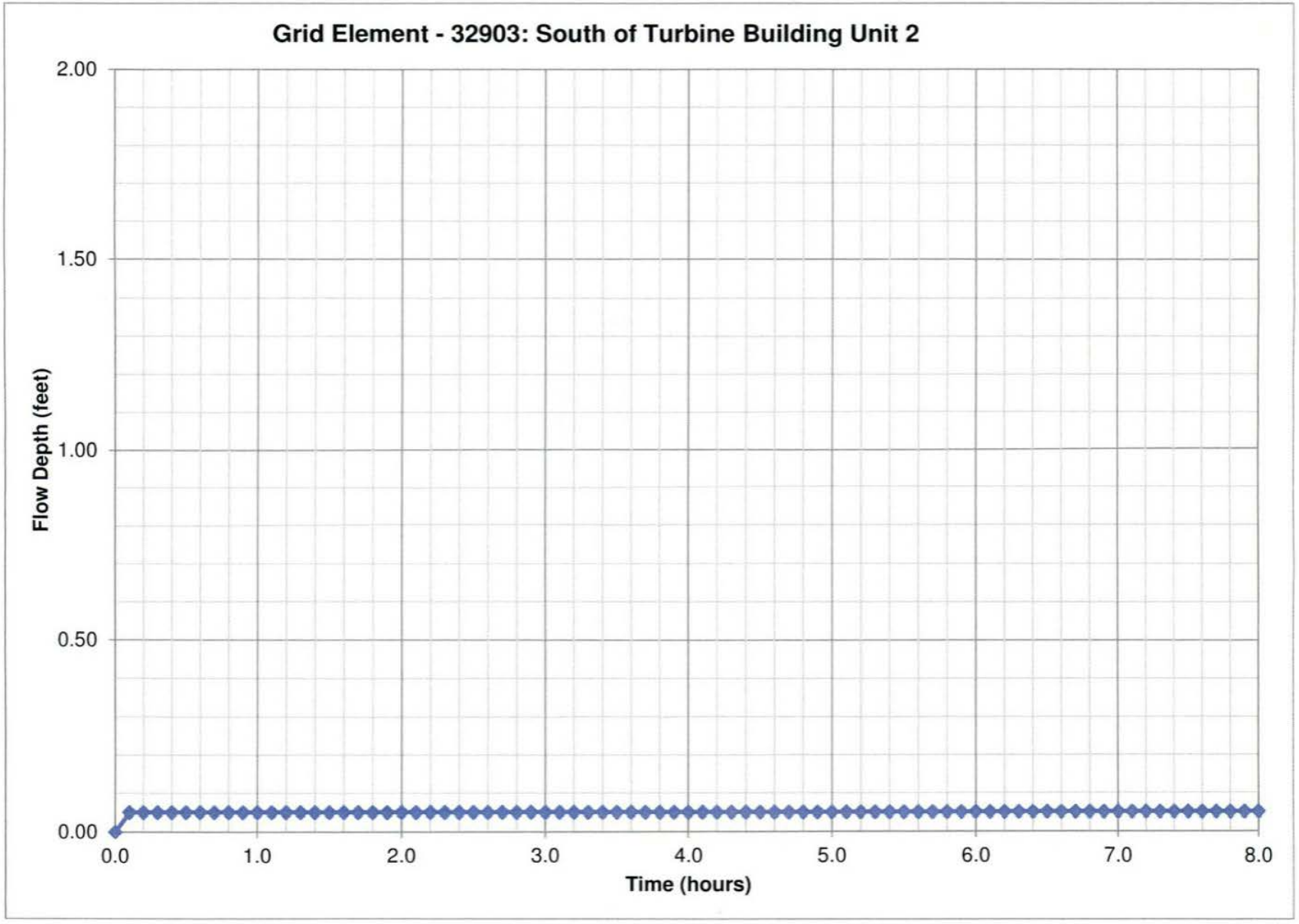
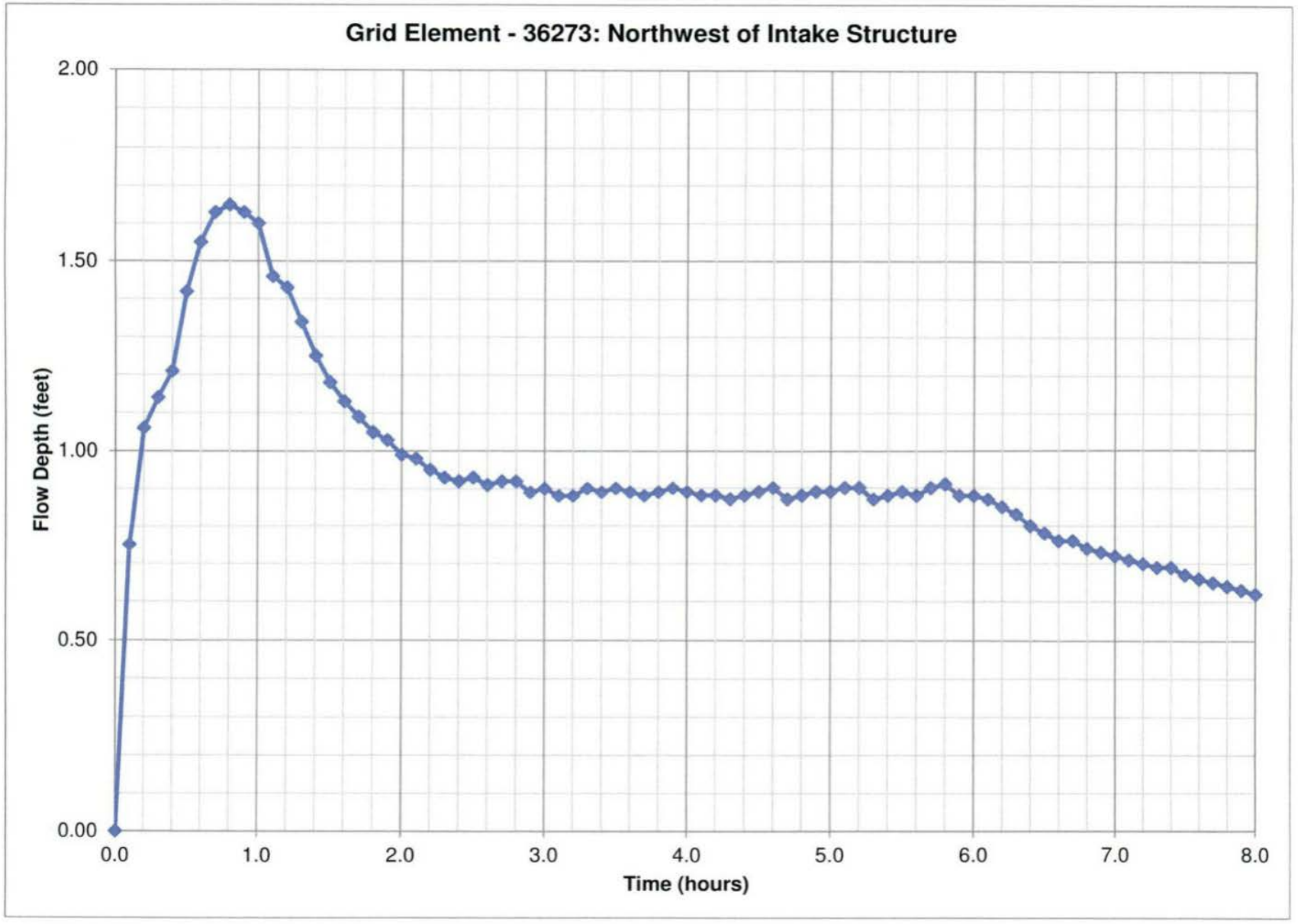
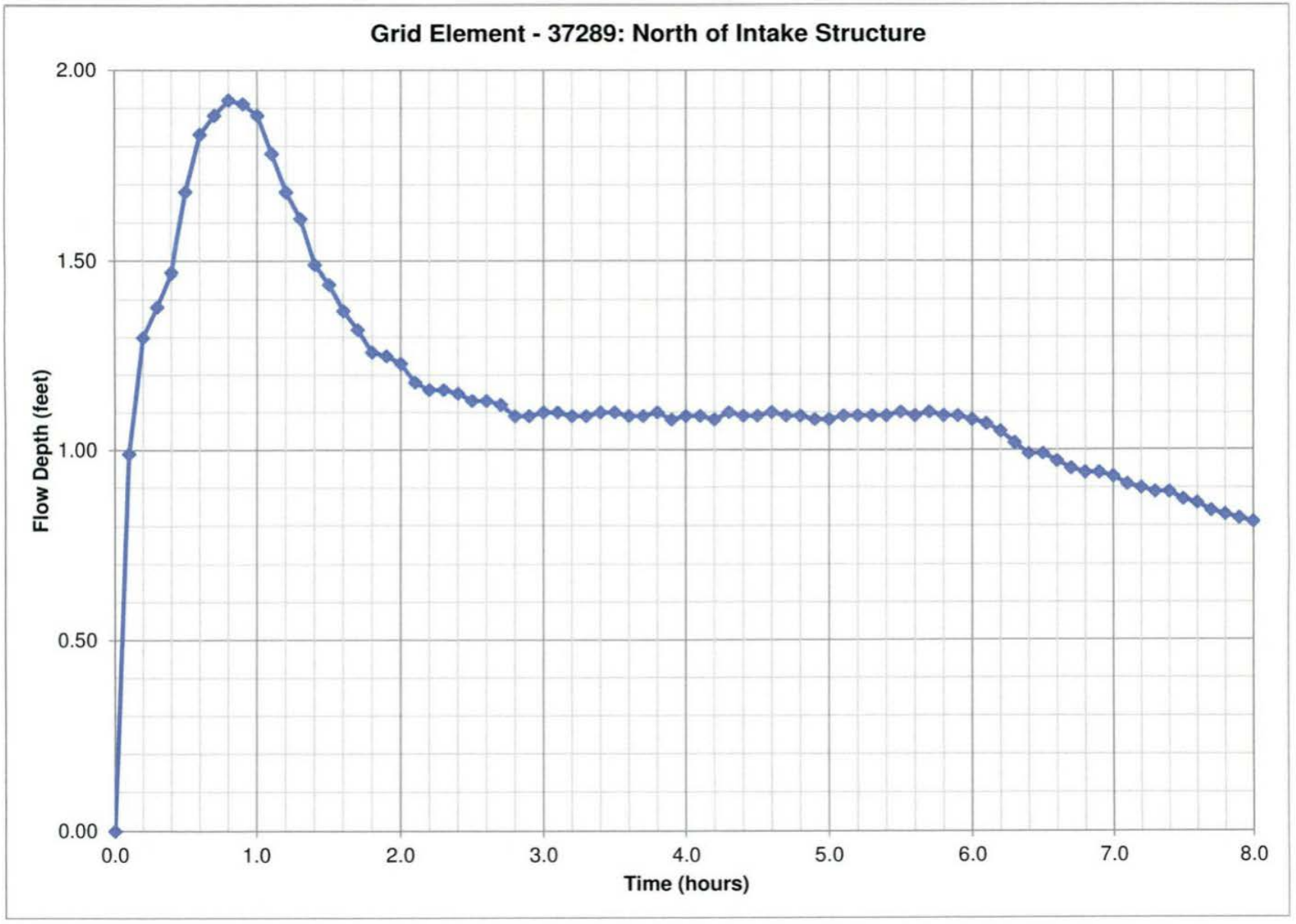
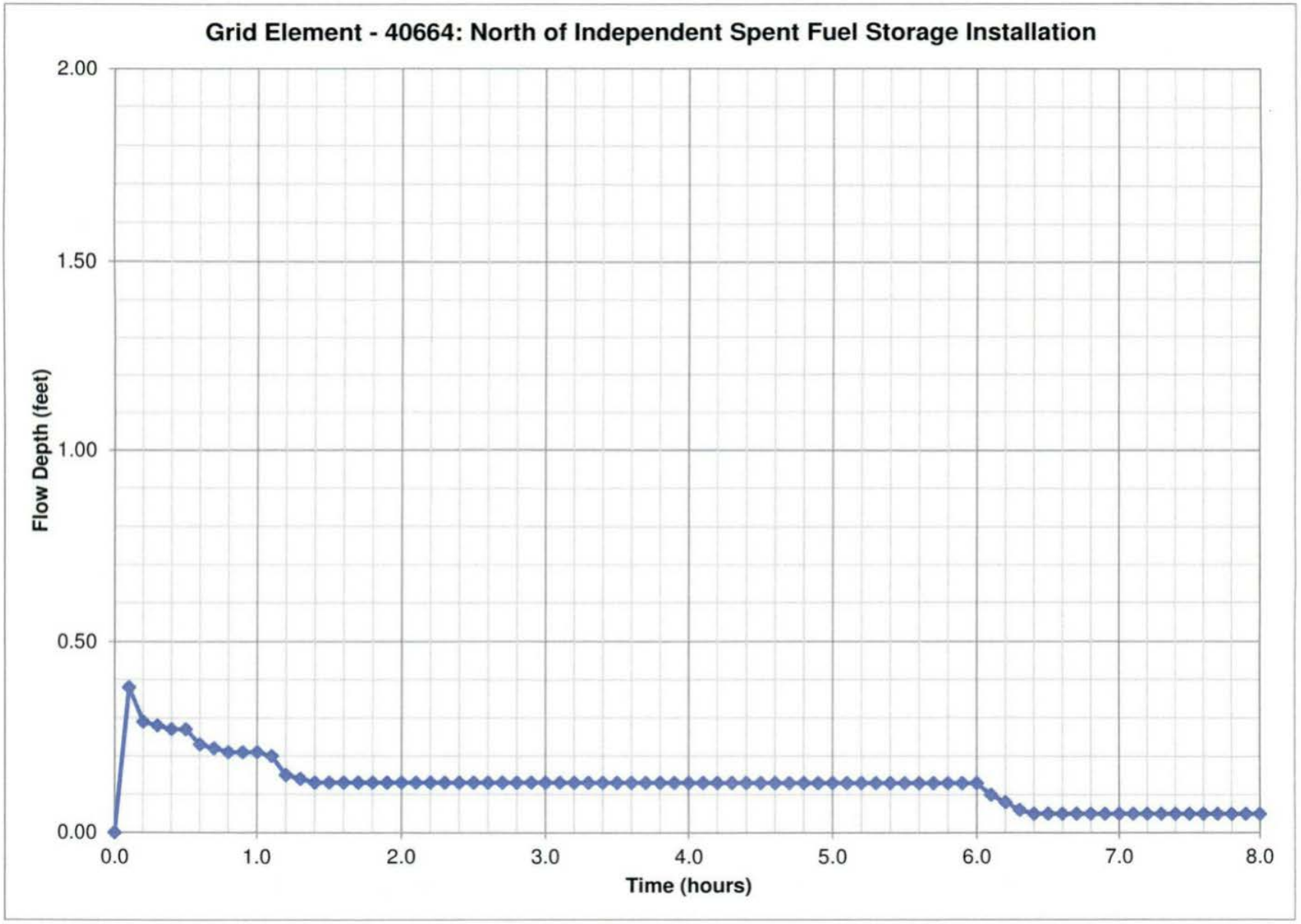
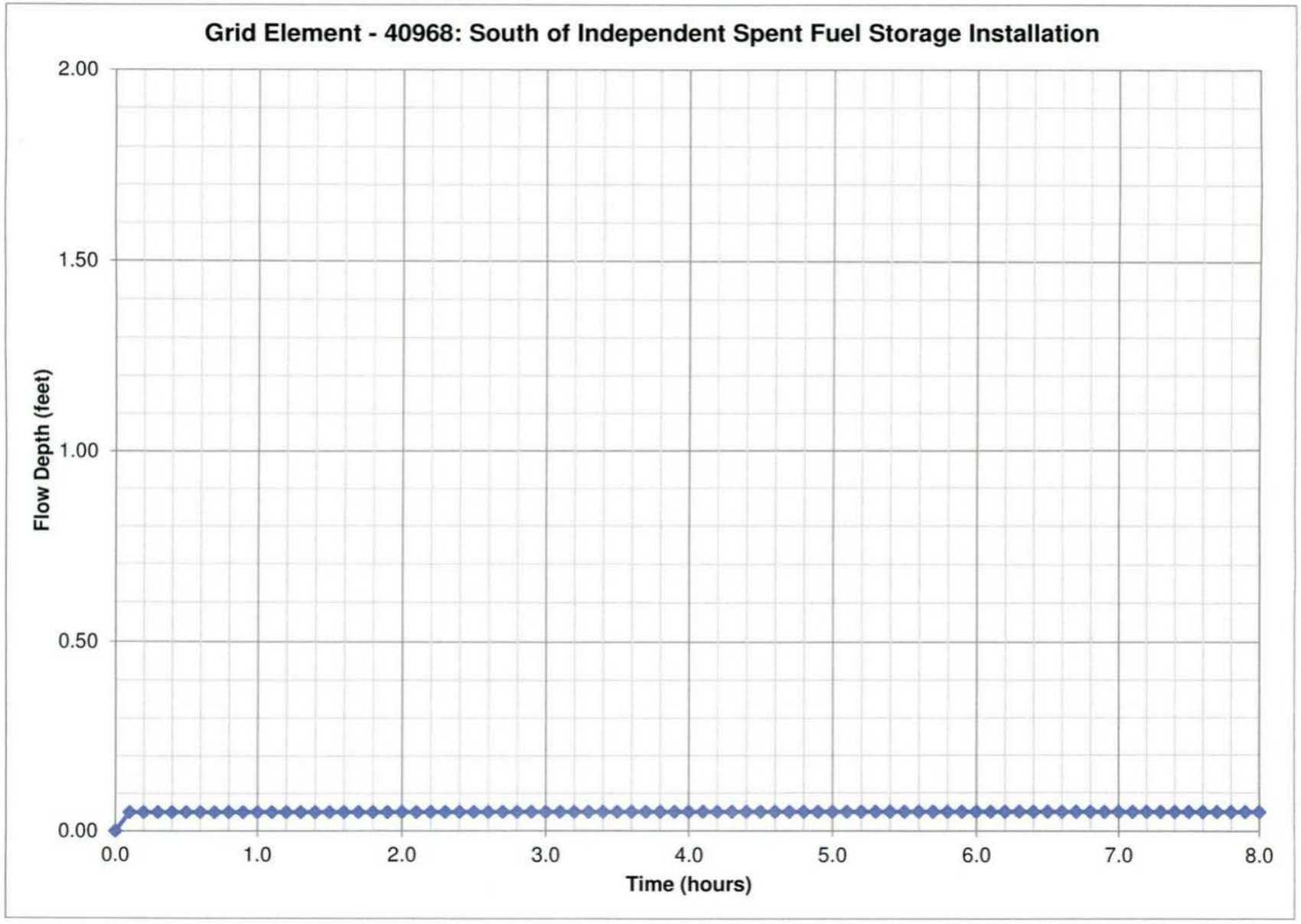
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)

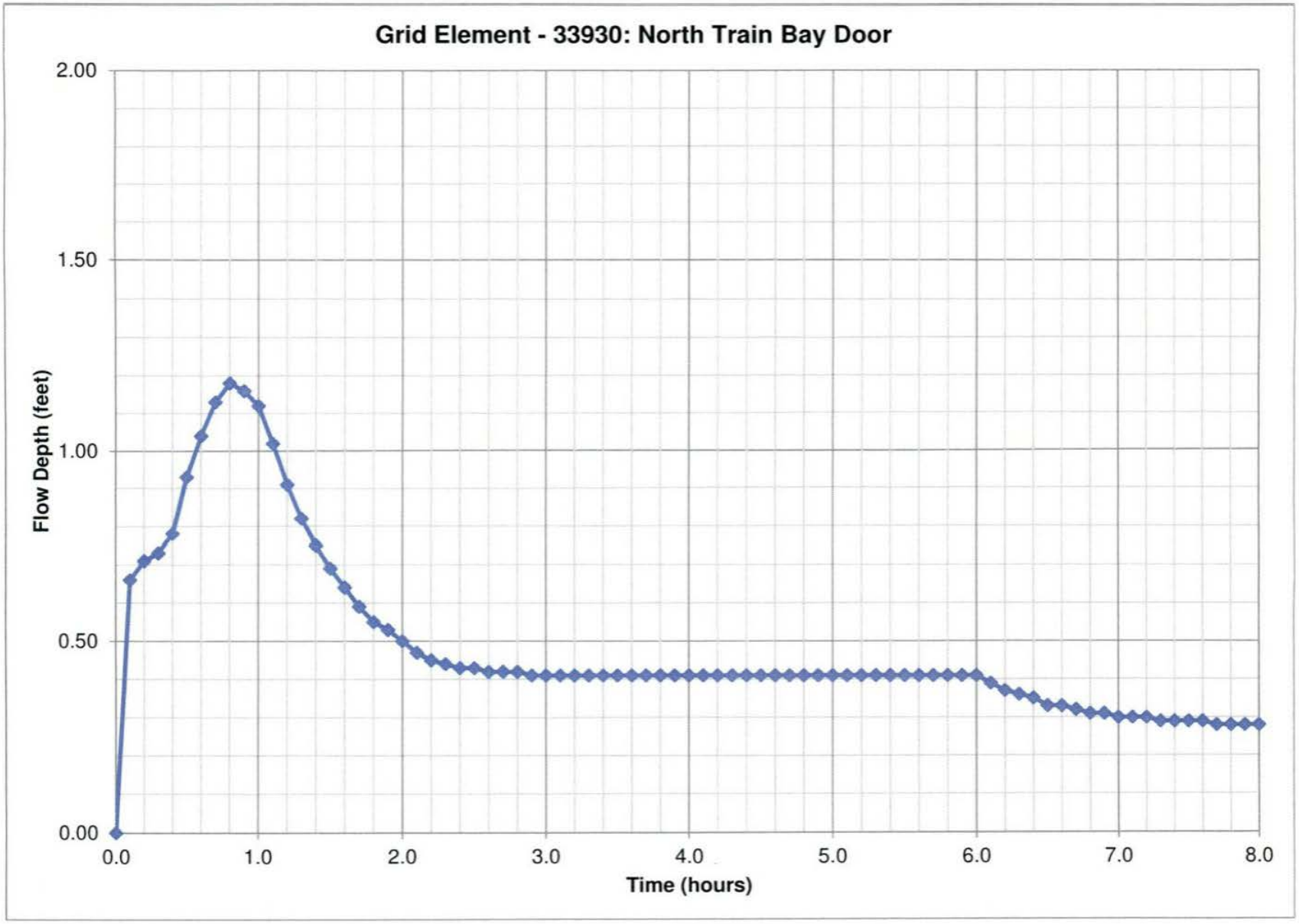

Grid Element - 32903: South of Turbine Building Unit 2
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)

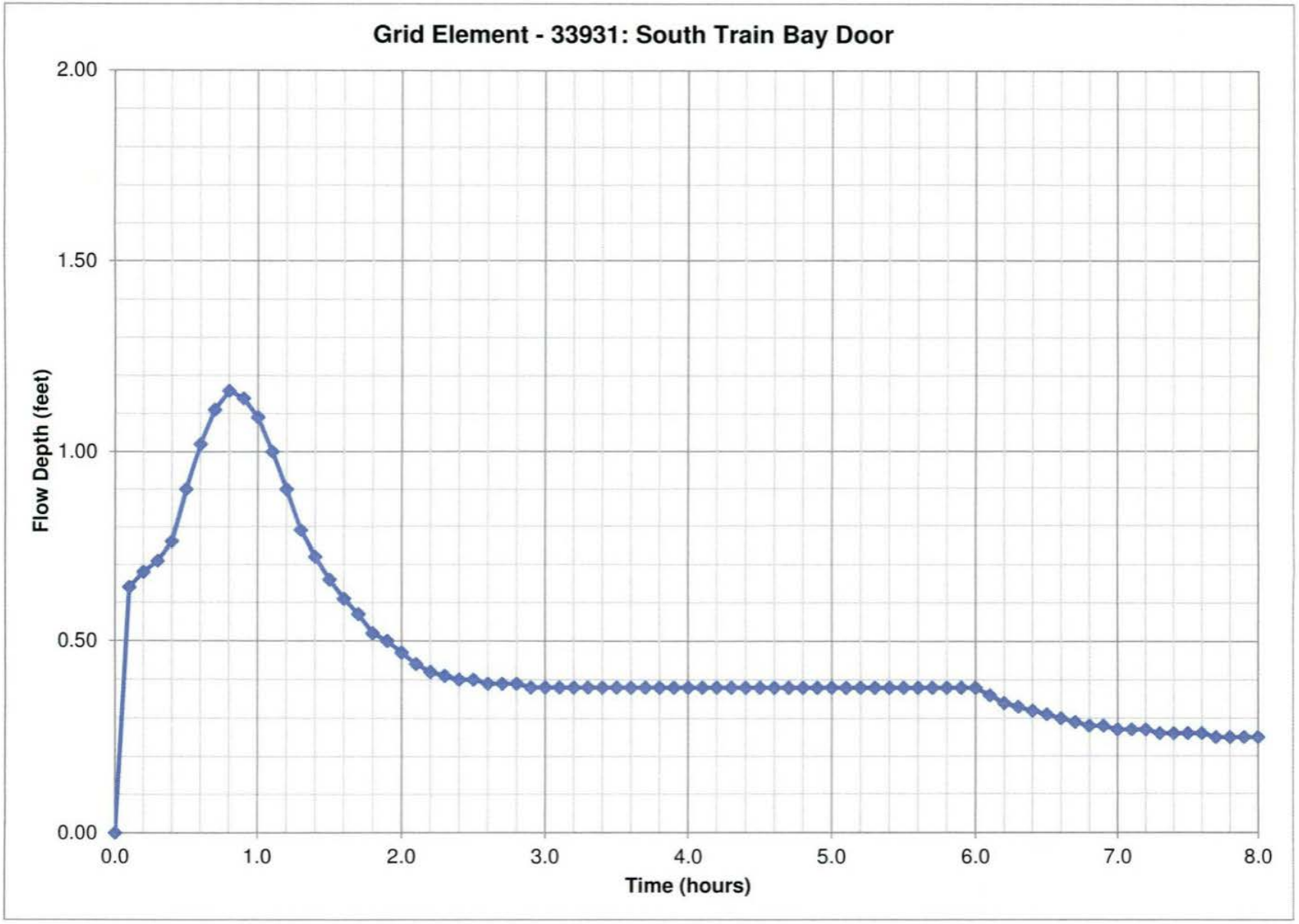

Grid Element - 36273: Northwest of Intake Structure
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 37289: North of Intake Structure
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 40664: North of Independent Spent Fuel Storage Installation
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 40968: South of Independent Spent Fuel Storage Installation
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 33930: North Train Bay Door
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)


Grid Element - 33931: South Train Bay Door
Flow Depth (feet)
2.00
1.50
1.00
0.50
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Time (hours)