UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

October 29, 2014

Mr. Fadi Diya
Senior Vice President and
Chief Nuclear Officer
Union Electric Company
P.O. Box 620
Fulton, MO 65251

SUBJECT: CALLAWAY PLANT, UNIT 1 – STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST – FLOOD-CAUSING MECHANISM REEVALUATION (TAC NO. MF1096)

Dear Mr. Diya:

By letter dated March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued a request for information pursuant to Title 10 of the *Code of Federal Regulations*, Section 50.54(f) (hereafter referred to as the 50.54(f) letter). The request was issued as part of implementing lessons-learned from the accident at the Fukushima Dai-ichi nuclear power plant. Enclosure 2 to the 50.54(f) letter requested licensees to reevaluate flood-causing mechanisms using present-day methodologies and guidance.

By letter dated March 8, 2013, Union Electric Company (doing business as Ameren Missouri) responded to this request for Callaway Plant, Unit 1. In response to NRC staff questions, this response was supplemented by letter dated February 27, 2014.

The NRC staff reviewed the information provided and, as documented in the enclosed staff assessment, determined that you provided sufficient information in response to the 50.54(f) letter. This closes out the NRC's efforts associated with TAC No MF1096.

If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

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Sincerely,

Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket No. 50-483

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report

cc w/encl: Distribution via Listserv

STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST

FLOOD-CAUSING MECHANISM REEVALUATION

BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO FLOODING HAZARD REEVALUATION REPORT

UNION ELECTRIC COMPANY

CALLAWAY PLANT, UNIT 1

DOCKET NO. 50-483

1.0 INTRODUCTION

By letter dated March 12, 2012 (NRC, 2012a), the U.S. Nuclear Regulatory Commission (NRC) issued a request for information to all power reactor licensees and holders of construction permits in active or deferred status, pursuant to Title 10 of the Code of Federal Regulations (10 CFR), Section 50.54(f) "Conditions of license" (hereafter referred to as the "50.54(f) letter"). The request was issued in connection with implementing lessons-learned from the 2011 accident at the Fukushima Dai-ichi nuclear power plant as documented in the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident (NRC, 2011b)[1]. The NRC Near-Term Task Force (NTTF) Recommendation 2.1, and subsequent Staff Requirements Memoranda (SRM) associated with Commission Papers SECY 11-0124 (NRC, 2011c) and SECY-11-0137 (NRC, 2011d), instructed the NRC staff to issue requests for information to licensees pursuant to 10 CFR 50.54(f).

Enclosure 2 to the 50.54(f) letter requested that licensees reevaluate flood hazards for their respective sites using present-day methods and regulatory guidance used by the NRC staff when reviewing applications for early site permits (ESPs) and combined licenses (COLs). The required response section of Enclosure 2 specified that NRC staff would provide a prioritization plan indicating Flooding Hazard Reevaluation Report (FHRR) deadlines for individual plants. The staff issued a letter (NRC, 2012b) providing the prioritization of the FHRRs on May 11, 2012.

Upon completion of the licensee's hazard review, if the reevaluated hazard for any flood-causing mechanism is not bounded by the current plant design-basis flood hazard, an Integrated Assessment will be necessary. The FHRR and the responses to the associated requests for additional information (RAIs) will provide the hazard input necessary to complete the Integrated Assessment report, as described in Japan Lessons-Learned Project Directorate (JLD) interim staff guidance (ISG) JLD-ISG-2012-05, "Guidance for Performing the Integrated Assessment for External Flooding" (NRC, 2012c).

[1] Issued as an enclosure to Commission Paper SECY-11-0093 (NRC, 2011a).

By letter dated March 8, 2013 (Reasoner, 2013), Union Electric Company (doing business as Ameren Missouri, the licensee) provided the FHRR for Callaway Plant (Callaway), Unit 1. The licensee did not identify any interim actions. The FHRR was supplemented by letter, including RAI responses, dated February 27, 2014 (Reasoner, 2014a).

2.0 REGULATORY BACKGROUND

2.1 Applicable Regulatory Requirements

This section describes present-day regulatory requirements that are applicable to the FHRR.

Section 50.34(a)(1), (a)(3), (a)(4), (b)(1), (b)(2), and (b)(4), of 10 CFR, describes the required content of the preliminary and final safety analysis reports, including a discussion of the facility site with a particular emphasis on the site evaluation factors identified in 10 CFR Part 100. The licensee should provide any pertinent information identified or developed since the submittal of the preliminary safety analysis report in the final safety analysis report.

Section 50.54(f) of 10 CFR states that a licensee shall at any time before expiration of its license, upon request of the Commission, submit written statements, signed under oath or affirmation, to enable the Commission to determine whether or not the license should be modified, suspended, or revoked. The 50.54(f) letter requested licensees reevaluate the flood-causing mechanisms for their respective sites using present-day methodologies and regulatory guidance used by the NRC for the ESP and COL reviews.

General Design Criterion 2 in Appendix A of Part 50 states that structures, systems, and components (SSCs) important to safety at nuclear power plants must be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without loss of capability to perform their intended safety functions. The design bases for these SSCs are to reflect appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area. The design bases are also to have sufficient margin to account for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Section 50.2 of 10 CFR defines the design-basis as the information that identifies the specific functions that an SSC of a facility must perform, and the specific values or ranges of values chosen for controlling parameters as reference bounds for design which each licensee is required to develop and maintain. These values may be (a) restraints derived from generally accepted "state of the art" practices for achieving functional goals, or (b) requirements derived from an analysis (based on calculation or experiments or both) of the effects of a postulated accident for which an SSC must meet its functional goals.

Section 54.3 of 10 CFR defines the "current licensing basis" as: "the set of NRC requirements applicable to a specific plant and a licensee's written commitments for ensuring compliance with and operation within applicable NRC requirements and the plant-specific design-basis (including all modifications and additions to such commitments over the life of the license) that are docketed and in effect." This includes 10 CFR Parts 2, 19, 20, 21, 26, 30, 40, 50, 51, 52, 54, 55, 70, 72, 73, 100 and appendices thereto; orders; license conditions; exemptions; and technical specifications as well as the plant-specific design-basis information as documented in the most recent final safety analysis report. The licensee's commitments made in docketed licensing correspondence, which remain in effect, are also considered part of the current licensing basis.

Present-day regulations for reactor site criteria (Subpart B to 10 CFR Part 100 for applications on or after January 10, 1997) state, in part, that the physical characteristics of the site must be evaluated and site parameters established such that potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site. Factors to be considered when evaluating sites include the nature and proximity of dams and other man-related hazards (10 CFR 100.20(b)) and the physical characteristics of the site, including the hydrology (10 CFR 100.21(d)).

2.2 Enclosure 2 to the 50.54(f) Letter

The 50.54(f) letter requests all power reactor licensees and construction permit holders reevaluate all external flood-causing mechanisms at each site. The reevaluation should apply present-day methods and regulatory guidance that are used by the NRC staff to conduct ESP and COL reviews. This includes current techniques, software, and methods used in present-day standard engineering practice. If the reevaluated flood-causing mechanisms are not bounded by the current plant design-basis flood hazard, an Integrated Assessment will be necessary.

2.2.1 Flood-Causing Mechanisms

Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) discusses flood-causing mechanisms for the licensee to address in the FHRR. Table 2.2-1 lists the flood-causing mechanisms the licensee should consider. Table 2.2-1 also lists the corresponding Standard Review Plan (SRP) (NRC, 2007) sections and applicable interim staff guidance containing acceptance criteria and review procedures. The licensee should incorporate and report associated effects per JLD-ISG-2012-05 (NRC, 2012c) in addition to the maximum water level associated with each flood-causing mechanism.

2.2.2 Associated Effects

In reevaluating the flood-causing mechanisms, the "flood height and associated effects" should be considered. The ISG for performing the integrated assessment for external flooding, JLD-ISG-2012-05 (NRC, 2012c), defines "flood height and associated effects" as the maximum stillwater surface elevation plus:

- wind waves and run-up effects;
- hydrodynamic loading, including debris;
- effects caused by sediment deposition and erosion;
- concurrent site conditions, including adverse weather conditions;
- groundwater ingress; and,
- other pertinent factors.

2.2.3 Combined Effects Flood

The worst flooding at a site that may result from a reasonable combination of individual flooding mechanisms is sometimes referred to as a "Combined Effects Flood." Even if some or all of these individual flood-causing mechanisms are less severe than their worst-case occurrence, their combination may still exceed the most severe flooding effects from the worst-case occurrence of any single mechanism described in the 50.54(f) letter (See SRP, Section 2.4.2, Area of Review 9 (NRC, 2007)). Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) describes the "Combined Effects Flood"[2] as defined in ANSI/ANS 2.8-1992 (ANSI/ANS, 1992) as follows:

> *For flood hazard associated with combined events, American Nuclear Society (ANS) 2.8-1992 provides guidance for combination of flood causing mechanisms for flood hazard at nuclear power reactor sites. In addition to those listed in the ANS guidance, additional plausible combined events should be considered on a site specific basis and should be based on the impacts of other flood causing mechanisms and the location of the site.*

If two less severe mechanisms are plausibly combined (per ANSI/ANS-2.8-1992 (ANSI/ANS, 1992) and SRP, Section 2.4.2, Areas of Review 9 (NRC, 2007)), then the staff will document and report the result as part of one of the hazard sections. An example of a situation where this may occur is flooding at a riverine site located where the river enters the ocean. For this site, storm surge and river flooding should be plausibly combined.

2.2.4 Flood Event Duration

Flood event duration was defined in the ISG for the integrated assessment for external flooding, JLD-ISG-2012-05 (NRC, 2012c), as the length of time during which the flood event affects the site. It begins when conditions are met for entry into a flood procedure, or with notification of an impending flood (e.g., a flood forecast or notification of dam failure), and includes preparation for the flood. It continues during the period of inundation, and ends when water recedes from the site and the plant reaches a safe and stable state that can be maintained indefinitely. Figure 2.2.4-1 illustrates flood event duration.

2.2.5 Actions Following the FHRR

For the sites where the reevaluated probable maximum flood (PMF) elevation is not bounded by the current design-basis PMF elevation for all flood-causing mechanisms, the 50.54(f) letter requests licensees and construction permit holders to:

- Submit an Interim Action Plan with the FHRR documenting actions planned or already taken to address the reevaluated hazard.
- Perform an Integrated Assessment subsequent to the FHRR to (a) evaluate the effectiveness of the current licensing basis (i.e., flood protection and mitigation systems), (b) identify plant-specific vulnerabilities, and (c) assess the effectiveness of existing or

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[2] For the purposes of this Staff Assessment, the terms "combined effects" and "combined events" are synonyms.

planned systems and procedures for protecting against and mitigating consequences of flooding for the flood event duration.

If the reevaluated PMF elevation is bounded by the current design-basis PMF elevation for all flood-causing mechanisms at the site, licensees are not required to perform an integrated assessment at this time.

3.0 TECHNICAL EVALUATION

The NRC staff reviewed the information provided for the flood hazard reevaluation of Callaway, Unit 1. The licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews. The staff's review and evaluation is provided below.

The site grade at the powerblock is elevation 840 ft (256 m)[3] mean sea level (MSL) or the North American Vertical Datum of 1988 (NAVD88), with most safety-related structures at or above elevation 840.5 ft (256.2) MSL, except for the Ultimate Heat Sink (UHS) retention pond at 840 ft (256.0 m), the base slab of the Refueling Water Storage Tank (RWST) at 835.5 ft (254.7 m) MSL, and the Essential Service Water System pipes at various elevations. Table 3.0-1 provides the summary of controlling reevaluated flood-causing mechanisms, including associated effects.

3.1 Site Information

The 50.54(f) letter includes the SSCs important to safety, and the UHS, in the scope of the hazard reevaluation. Per the 50.54(f) letter, Enclosure 2, Requested Information, Hazard Reevaluation Report, Item a, the licensee included pertinent data concerning these SSCs in its FHRR.

To provide additional information in support of the summaries and conclusions in the FHRR, the licensee made several calculation packages available to the staff via an electronic reading room. These calculation packages expand upon and clarify the information provided on the docket.

To supplement the FHRR, the staff issued RAIs (NRC, 2014a) to request additional information from the licensee. Individual RAIs, and the licensee's responses, are discussed in the appropriate sections below.

The 50.54(f) letter, Enclosure 2 (Recommendation 2.1: Flooding), Requested Information, Hazard Reevaluation Report, Item a, describes site information to be contained in the FHRR. The staff reviewed and summarized this information as follows.

3.1.1 Detailed Site Information

The licensee described in its FHRR the following site information: The Callaway site is located on a plateau that ranges in elevation from approximately 830 to 850 ft (253 to 259 m). It occupies a topographic high from which surface runoff drains radially to several small

[3] The licensee's flood hazard reevaluation studies were conducted using customary units of measure. In this report, customary measurements are followed by the equivalent measurement in metric units. Because the conversion to metric units may involve loss of precision, the measurement in customary units is definitive.

intermittent streams which are connected to the Missouri River. The licensee stated in its FHRR that the site region is tectonically and seismically stable; experiencing only infrequent and minor earthquake activities, with the closest epicenter located 38 mi (61 km) from the site (see FHRR Section 3.2.8.3). The Missouri River flows from west to east in a valley about 5 mi (8 km) south of the Callaway, Unit 1site. The river channel has been stabilized after the glacial period without changing its pre-glacial route. Figure 1-1 shows the site location in relation to regional topography.

The site grade is elevation 840 ft (256 m) MSL, which places the plant approximately 328 ft (100 m) above the Missouri River valley. The licensee stated in its FHRR that all safety-related structures are at or above 840.5 ft (256.2 m) MSL, except for the UHS retention pond at 840 ft (256.0 m) MSL, the base slab of the Refueling Water Storage Tank at 835.5 ft (254.7 m) MSL, and the Essential Service Water System (ESWS) pipes at various elevations. The ESWS piping is either protected by concrete or buried at a depth of at least 4.5 ft (1.4 m). The ESWS electrical duct banks are reinforced concrete structures and are buried at a depth of at least 3.5 ft (1.1 m).

The FHRR states that the UHS retention pond, which supplies safety-related cooling water for the Callaway site, is located approximately 400 ft (122 m) southeast of Unit 1. According to the FHRR, the pond has a surface area of about 4.1 acres (0.02 km^2) and can supply 56.03 acre-ft (69,100 m^3) of water to the UHS Cooling Tower in the case of an emergency safe shutdown. Its normal water surface elevation and normal water depth are 836 ft (254.8 m) MSL and 18 ft (5.5 m), respectively. An overflow spillway is provided to maintain the water level in the UHS retention pond below elevation 836.5 ft (255.0 m) MSL. At the pond's normal water level of 836.0 ft (254.8 m) MSL, there is at least 4 ft (1.2 m) of freeboard on the dike around the pond. The inside slope of the dike are covered with rip-rap.

Named streams receiving surface runoff from the site area include Auxvasse Creek about 2.5 mi (4 km) to the west of the site; Cow Creek approximately 5 mi (8 km) to the north and northwest of the site; Logan Creek about 2 mi (3 km) to the east of the site; and Mud Creek about 1.5 mi (2.4 km) to the south of the site. Auxvasse and Logan Creeks flow directly into the Missouri River, whereas Cow Creek and Mud Creek are tributaries to Auxvasse and Logan Creeks, respectively. Auxvasse and Logan Creeks drain a combined watershed area of about 347 square miles (899 km^2).

3.1.2 Design-Basis Flood Hazards

Ameren Missouri began operation of the Callaway, Unit 1 in 1984. The licensee submitted a combined license application for a second unit in 2008, of which the NRC review was suspended in 2009 at the licensee's request.

The licensee stated in its FHRR, Section 2.3 that the current design-basis indicates that all safety-related SSCs, except the UHS retention pond, are not subject to flooding, wave action, or wave run-up, and therefore do not require flood or wave protection. In particular, flooding and associated wind effects from the Missouri River are not applicable because the highest recorded flood level for the Missouri River near the site was approximately 290 ft (88.4 m) below all SSCs at the Callaway, Unit 1site. The UHS retention pond is the only safety-related SSC subject to wind wave activity and wave run-up. The licensee concluded that wind wave activity on the UHS retention pond is not a "major concern" because the UHS retention pond has relatively short dimensions and riprap-covered side slopes. The Callaway, Unit 1 Final Safety Analysis Report (FSAR) (Ameren Missouri, 2012a) indicates that the maximum water level in the UHS retention pond as a result of a 48-hour PMP event would be 837.7 ft (255.3 m) MSL and is mitigated by the outflow over the UHS retention pond weir. Additionally, the licensee concluded that the estimated 48-hour PMP UHS retention pond surface water level in combination with wave run-up would result in a surface water elevation of 838.3 ft (255.5 m) which is below the site grade elevation.

The Callaway, Unit 1 FSAR (Ameren Missouri, 2012a) indicates that the local PMP provides the design-basis for controlling surface runoff from safety related structures at the Callaway, Unit 1 site. The licensee determined the all-season 6-hour rainfall with an accumulation of 25.4 inches (64.5 cm) would result in ponding elevations less than 840.5 ft (256.2 m) MSL on the Callaway, Unit 1 site. The current design-basis flood levels are summarized in Table 3.1.2-1.

3.1.3 Flood-Related Changes to the Licensing Basis

Table 3-5 in the FHRR provides the current licensing basis water levels which are identical to the current design-basis flood levels presented in the FSAR (Ameren Missouri, 2012a). That is, there is no change from the design-basis flood hazards to the licensing basis flood hazards.

3.1.4 Changes to the Watershed and Local Area

The licensee noted in FHRR Section 2.5.2 that there have been only small changes to the watershed of Missouri River since Ameren Missouri began operation of the Callaway, Unit 1 plant in 1984, mostly in the form of land use changes and land development. The licensee considered detailed information regarding the current land use and soil types within each subbasin. As such, the licensee incorporated the updated condition for the watershed and local area on their flood hazard reevaluation, as applicable.

3.1.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

The licensee stated in its FHRR Section 2.6 that protection of the plant facilities from potential Missouri River flooding is not necessary because the safety-related SSCs at the site are located approximately 290 ft (88 m) above the highest expected river flood level. The licensee stated in its FHRR that the Callaway, Unit 1 site is located on the crest of a plateau which has a natural drainage system and site drainage facilities direct run-off into the natural drainage system. The FSAR (Ameren Missouri, 2012a) further states that the plant site drainage was designed to convey runoff from a 100-year storm event away from the plant area. Ice accumulation on safety-related structures was taken into consideration for the site drainage system.

3.1.6 Additional Site Details to Assess the Flood Hazard

The licensee reported in FHRR Section 2.2.1.2 that safety-related SSCs include the Reactor Building, Fuel Building, Control Building, Diesel Generator Building, Auxiliary Building, Essential Service Water System (ESWS) Pipelines, Refueling Water Storage Tank, UHS Cooling Tower, UHS retention pond, and ESWS Pump house. The licensee also stated in FHRR Section 2.2.1.3 that the source of safety-related cooling water for the Callaway site is the UHS retention pond, which supplies water to the UHS Cooling Tower in the case of an "emergency safety shutdown."

The licensee provided to the NRC an electronic copy of detailed topology data and relevant model input files used in analyzing onsite flooding in response to RAIs 1, 3, and 6 issued on January 29, 2014 (NRC, 2014a).

3.1.7 Plant Walkdown Activities

Enclosure 4 of the 50.54(f) letter requested that licensees plan and perform plant walkdown activities to verify that current flood protection systems are available, functional, and implementable. Other parts of the 50.54(f) letter (Requested Information Item 1.c, and Step 6 of Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2)) asked the licensee to report any relevant information from the results of the plant walkdown activities.

By letter dated November 27, 2012, the licensee provided the flood walkdown report for Callaway, Unit 1 (Ameren Missouri, 2012b). The walkdown report was supplemented by letter, including RAI responses, dated January 31, 2014 (Ameren Missouri, 2014).

The NRC staff prepared a staff assessment report, dated May 16, 2014 (NRC, 2014b), to document its review of the walkdown report, and concluded that the licensee's implementation of flooding walkdown methodology meets the intent of the walkdown guidance.

3.2 Local Intense Precipitation and Associated Site Drainage

The licensee stated in an RAI response (Reasoner, 2014a) that the reevaluated PMF elevation, including associated effects, for local intense precipitation (LIP) is 840.4 ft (256.15 m) MSL. This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis flood elevation for the LIP and associated site drainage hazard for the powerblock will be "less than" 840.5 ft (256.2 m) MSL as reported in the FHRR Table 3.5. The current design-basis elevation on the UHS retention pond for LIP, including wave run-up, is 838.3 ft (255.5 m) MSL.

FHRR Table 2-2 states that most safety-related structures are at or above 840.5, except the UHS Retention Pond at 840 ft (256 m) MSL elevation. Although FHRR Table 2-2 indicates that the bottom slab of the RWST is at 835.5 ft (254.7 m) MSL elevation, FSAR Section 3.8.4.1.5 describes that the RWST consists of an above-grade cylindrical steel tank founded and anchored on a 5-foot-6-inch-thick (1.68 m) reinforced concrete base slab and an associated valve house. The 5-foot-6-inch (1.68 m) thick concrete base slab raises the structure elevation above 840.5 ft (256.2 m) MSL.

To provide additional information in support of the summaries and conclusions in its FHRR, the licensee made several calculation packages available to the staff via an electronic reading room. These calculation packages expand upon and clarify the information provided on the docket.

To supplement the FHRR, the staff issued RAIs (NRC, 2014a) to request additional information from the licensee. Individual RAIs, and the licensee's responses, are discussed in the appropriate sections below.

The staff reviewed the LIP and associated site drainage, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance. The licensee adopted a hierarchical approach in evaluating the LIP flood hazard for the site.

3.2.1 Local Intense Precipitation

In the FSAR for the proposed Callaway, Unit 2 (Ameren Missouri, 2009), the licensee estimated an onsite 6-hour probable maximum precipitation (PMP) value of 25.4 in (64.5 cm) using the guidance in Hydrometeorological Report (HMR) 33 (NOAA, 1956). Table 3-4 of the FHRR lists the reevaluated onsite 6-hour PMP value of 27.5 in (699 mm) using the guidance of HMR 52 (NOAA, 1982). The response (Reasoner, 2014a) to RAI 3 states that the licensee used the 6-hour PMP scenario in its FHRR, but updated the LIP flood evaluation using up to 72-hour PMP values. Table 3.2-1 in this report summarizes the reevaluated PMP values. The staff confirmed the licensee's PMP values in Table 3.2-1 are accurate.

3.2.2 Assessing LIP Runoff Using HEC-HMS

For the purpose of reevaluating the effects of LIP flooding, the licensee delineated 51 subbasins on the Callaway site as shown on Figure 3.2-1, of which 7 subbasins (24, 30, 31, 50, 51, 56, and 58) contain safety-related SSCs. The staff noted additional information on the site and subbasin delineations was needed in order to complete the review. Accordingly, the staff issued two RAIs (NRC, 2014a), requesting a discussion of the delineation of subbasins, as well as electronic versions of digital elevation models, input and output files of the LIP model, and relevant graphical information system files that were used by the licensee in LIP modeling. In its response dated February 27, 2014 (Reasoner, 2014a), the licensee provided the requested information for the staff's review.

To assess the LIP runoff volumes on subbasins, the licensee postulated six runoff scenarios consistent with a hierarchical approach. The following summarizes the licensee's scenarios, ranging from very conservative to less conservative but more realistic:

- Scenario 1: Use the Rational Runoff Transformation Method.
- Scenario 2: Use the HEC-HMS with no runoff transformation.
- Scenario 3: Use the HEC-HMS and the Snyder Runoff Transformation Method with a peak runoff coefficient of 0.7 and a catchment shape coefficient of 0.4.
- Scenario 4: Use the HEC-HMS and the Snyder Runoff Transformation Method with a peak runoff coefficient of 0.5 and a catchment shape coefficient of 0.4.
- Scenario 5: Use the HEC-HMS model and the Snyder Runoff Transformation Method with a peak runoff coefficient of 0.4 and a catchment shape coefficient of 0.4.
- Scenario 6: Use the HEC-HMS model and the Snyder Runoff Transformation Method with a peak runoff coefficient of 0.4 and a catchment shape coefficient of 1.8.

For the first four scenarios, peak flood levels at the powerblock area exceed the critical flood level for the site. The licensee selected Scenario 5 for determining the LIP flood hazards because this scenario uses realistic runoff parameters and assumptions. Scenario 5 uses the lowest peak runoff coefficient value within the typical range described in the research reports (e.g., Viessman et al. 1977) on which the method is based. The estimated maximum peak flood elevation reported in the FHRR is 840.17 ft (256.1 m) NAVD88 based on a 6-hour PMP scenario. This value was updated using a 72-hour PMP, a conservative peak runoff coefficient of 0.7, and a transient runoff simulation scenario through the response to RAIs (Reasoner, 2014a), as discussed later. The licensee also evaluated Scenario 6, but this scenario was not adopted because its flood level is less conservative than Scenario 5. The licensee stated that their simulation approach follows the hierarchical flood hazard assessment method recommended in NUREG/CR-7046 (NRC, 2011e).

The licensee determined peak runoff values in each subbasin using the rational formula (Chow, 1964). They used a synthetic rainfall hyetograph as illustrated on Figure 3.2-2 in which the peak intensity of the one-hour PMP rainfall of 18.3 inches (465 mm) occurs at the beginning, and then decreases through the 6-hour period.

In its FHRR, the licensee did not discuss the rationale for using the 6-hr PMP value and the temporal distribution of rainfall in its evaluation of a LIP flooding event. The staff noted that a longer duration PMP scenario may result in a higher flood elevation and longer period of inundation compared to those of the 6-hour PMP scenario. Accordingly, the staff issued RAI No. 3 (NRC, 2014a) requesting the rationale for selecting the PMP scenario. In response, the licensee considered a 72-hour PMP as a bounding LIP flood scenario and revised the LIP flood parameters accordingly (Reasoner, 2014a).

The licensee implemented HEC-HMS with a PMP event in 5-minute time increments, which may not provide sufficiently fine resolution for accurate prediction of runoff generation from LIP, given the heavy rainfall and the short times of concentration associated with flood runoff in small subbasins. The staff issued RAI No. 4 requesting the licensee to consider a shorter PMP time increment in simulating the LIP flood scenarios. The licensee stated in its RAI response (Reasoner, 2014a) that the reevaluated LIP flood level with one-minute time increments of PMP values results in approximately a 0.15 ft (4.6 cm) higher LIP flood level at the reactor building (Subbasin ID 30) or a 0.05 ft (2 cm) higher level at the UHS retention pond, which is still below the floor elevations of buildings containing safety-related SSCs within the powerblock area.

Using the input files for the HEC-HMS LIP simulations provided in the RAI response (Reasoner, 2014a), the staff obtained results consistent with the licensee's reported results, thus confirming the licensee's implementation of the model.

The staff also performed a confirmatory sensitivity analysis of HEC-HMS by changing the peak flow coefficient (Cp) and time to peak coefficient (Ct). The staff's analysis found that the choice of values for these coefficients significantly affects both the magnitude of peak flows in specific subbasins and the distribution of flow between subbasins. To address this issue on modeling, the staff issued RAI 10 (NRC, 2014a). In response to this RAI, the licensee explained that the Snyder rainfall transformation method was selected primarily for its simplicity; also the licensee's sensitivity analysis found good agreement between runoff estimates determined by the Soil Conservation Service curve number method and the Snyder method with a Cp value of 0.7. In response to RAI 11, the licensee noted the HEC-HMS error messages had indicated a small problem with model convergence at a 5-minute PMP time step, but the errors did not occur with a one-minute time step.

Several of the parameter combinations considered in the licensee's sensitivity analyses resulted in water levels above the critical 840.50 ft (256.2 m) MSL elevation in at least one subbasin that contains safety-related SSCs for the quasi-steady simulation of HEC-RAS. However, the unsteady (transient) state simulation of HEC-RAS, which is more realistic than the previous one, for the revised final scenario(i.e., the Scenario C-1 as discussed below), shows that the maximum flood level is 840.4 ft, which is lower than the corresponding design-basis as shown on the response of RAI 13 (Reasoner, 2014a).

3.2.3 Estimating LIP Flood Levels Using HEC-RAS

The licensee used the HEC-RAS software package (USACE, 2010b) to determine water elevations associated with the peak subbasin flows. The choice of the HEC-RAS software for this LIP analysis is consistent with examples in NUREG/CR-7046 (NRC, 2011e). The individual subbasins were treated in HEC-RAS as reservoirs that are interconnected by "overland weirs." A total of 147 such interconnections were specified in the model. Each subbasin was assigned a constant lateral inflow rate set equal to the peak runoff rate for the subbasin, as determined by HEC-HMS. The licensee used a HEC-RAS "quasi-steady-state" option which uses peak runoff rates as upstream inflows conservatively instead of using realistic time-varying discharge rates.

For weirs connecting subbasins, a weir coefficient of 2.63 was assigned. The licensee stated this to be a conservatively low value that will result in higher flood levels. For the UHS retention pond, the analysis assumed a weir crest elevation of 836.5 ft (255.0 m) and a broad-crest weir coefficient of 2.65. The NRC staff found the choice of a different value for the weir coefficient has only a slight effect on water elevations in the critical subbasins, so this aspect of the licensee's evaluation is accepted as appropriate.

The licensee's HEC-RAS analyses were based on the NAVD 88 datum rather than the NGVD 29 datum referenced in the current design basis. The licensee clarified in its RAI response (Reasoner, 2014a) that, at the Callaway site, elevations referred to MSL are 0.02 ft (0.006 m) lower than elevations referred to NAVD88. This means that peak water level determinations generated with HEC-RAS were overestimated slightly compared to the MSL-oriented values.

For downstream reaches in HEC-RAS, the licensee set the Manning's roughness coefficient (n) at 0.035. The licensee describes this roughness coefficient as a relatively high value that will result in conservatively higher water levels within the upstream powerblock area. The staff recognized that this value is conservative for flow across smooth surfaces (Chow, 1964), but noted it may not be conservative for overland flow across a vegetated terrain. Accordingly, in RAI 8 (NRC, 2014a), the staff asked the licensee to provide descriptions of the terrain conditions of areas where overland flow would occur and the rationale for selecting a value or values for Manning's roughness coefficient for these areas. The licensee's response to this RAI (Reasoner, 2014a) explains that areas where overland flow would occur were modeled as storage areas, and because no flow is simulated in storage areas, the specified Manning's roughness values for overland plain were not sensitive in estimating maximum flood levels near the safety-related structures. This response resolves the staff's questions.

The licensee's responses (Reasoner, 2014a) to RAIs 6, 9, and 10 (NRC, 2014a) provided information on the locations of the Vehicle Barrier System (VBS) and its openings and clarified how the VBS was treated in the analysis. This information shows that the VBS is more than 200 ft (60 m) from the nearest boundary of the nearest subbasin that contains safety-related SSCs. Also, the NRC staff noted that the licensee's modeling results show that the open or closed condition of the VBS openings has little effect on water levels in those subbasins.

In its responses (Reasoner, 2014a) to RAIs 3, 4, 5, 10, 11, and 13 (NRC, 2014a), the licensee provided a consolidated description of its analytical approach and provided clarifications about details of the LIP flood analysis. The licensee also presented results of analysis of the sensitivity of the analytical results to the duration of the LIP event, rainfall distribution within the LIP event, PMP time step, and runoff transformation method. The licensee's sensitivity analyses found that a center-weighted rainfall distribution and shorter (one-minute) PMP time steps resulted in higher estimated peak water surface elevations, as did the combination of a longer storm duration, center-weighted rainfall distribution, and one-minute PMP time steps.

In its response (Reasoner, 2014a and 2014b) to RAI 13 (NRC, 2014a), the licensee presented calculations of peak LIP water levels based on runoff for 11 different scenarios: six steady-state scenarios and five transient scenarios. The licensee then selected the result of the Scenario C-1 (Table 13-1 of the RAI response) which gives a most conservative flood level among the transient simulations for subbasins ID numbers 22 (the northern excavated area) and 30 (Reactor Building). The scenario C-1 uses a 72-hour PMP with a center-weighted rainfall distribution, determined using the Snyder method of rainfall transformation with a Cp value of 0.7, and a one-minute time step. The staff noted that this combination of model input parameters is conservative for determination of runoff. The licensee reported in its response to RAI No. 4 (Reasoner, 2014a) that a "quasi-steady state" simulation, in which peak flows determined on this basis are used as constant-rate inflow to an unsteady HEC-RAS simulation, determined a maximum water surface elevation of 840.66 ft (256.23 m) MSL in a subbasin that includes safety-related SSCs, exceeding the critical elevation. However, when time-varying flows predicted by HEC-HMS were used as input to an unsteady HEC-RAS simulation, the maximum flood elevation in the powerblock area is less than 840.40 ft (256.15 m) MSL, which is less conservative but more realistic, thus they selected this scenario as a final LIP scenario. In this case, the effect of wind on LIP flood elevation was not considered because LIP flooding has relatively short fetch distance with shallow water depth. The use of time-varying inflows in a HEC-RAS unsteady flow calculation is consistent with present-day methodologies and regulatory guidance. The staff noted that this transient (unsteady) analysis is appropriately conservative.

The NRC staff noted that the licensee's highest reevaluated LIP flood elevation is for subbasin ID number 30 (Reactor Building) at 840.4 ft (256.15 m), based on information provided by the licensee in its response to RAI 13 (Reasoner, 2014a). The resulting reevaluated flood elevation, on which wind effects or other associated effects are not applicable, which is bounded by the design-basis of less than 840.5 ft (256.2 m) MSL.

3.2.4 Flooding and Wind Effects on UHS Retention Pond

FHRR Section 3.2.1.4 describes the procedure and results of analyzing LIP flooding coincident with wind effects on the UHS retention pond. The licensee noted in its FHRR that the UHS retention pond is a small body of water and is not subject to significant surges and seiches. The licensee also stated that other combined and associated effects coincident with LIP flooding are not applicable at the UHS retention pond. Figure 3.2-1 shows the location of the UHS retention pond (Subbasin ID 58).

The staff noted that the licensee's analysis for LIP Scenario 5 showed another area of ponding (HEC-RAS subbasin 22 in Figure 3.2-1) that had both a larger maximum dimension and a higher water elevation than the UHS retention pond. The staff issued RAI 12 (NRC, 2014a) to clarify this issue. In its response to RAI 12, the licensee clarified that their modeling of water surface elevations in subbasin 22 was based on a temporary situation involving an open excavation in that area, and presented HEC-RAS modeling results indicating that re-grading of the excavated area would reduce ponding, resulting in a somewhat lower peak flood elevation. Because the predicted area of ponding in subbasin 22 is at least 250 ft (76 m) away from the nearest safety-related facilities, the licensee did not consider the potential effects of wind-wave activity in subbasin 22.

Section 2.4.5 of the Callaway, Unit 1 FSAR (Ameren Missouri, 2012a) describes the UHS retention pond as having a water surface area of about 4.1 acres (16,600 m^2) and a storage capacity of about 56 acre-feet (69,000 m^3). The pond was designed to be operated at the normal surface elevation of 836 ft (254.8 m) MSL at the normal depth of 5.5 m (18 feet). An overflow spillway has a 20 ft (6 m) wide, broad-crested weir with a crest elevation of 836.5 ft (255.0 m) MSL. This spillway capacity is enough to drain the 48-hour PMP on the pond. The graded ground elevation around the UHS retention pond provides a 4 ft (1.2 m) minimum freeboard at the normal pond water level. In addition, the plant yard is graded away from the pond to prevent site runoff from entering the pond. The excavated pond slopes along the boundary are covered with riprap for protection against wave action.

Section 2.4.8.2.1 of the Callaway, Unit 1 FSAR (Ameren Missouri, 2012a) states that the calculated maximum wave runup on the riprapped dike slope is 0.6 feet (0.18 m). The resulting pond flood elevation with wind effects would be 838.3 feet (255.5 m) MSL, which is still below the plant grade. In its FHRR, the licensee reevaluated the wave setup and runup on the UHS retention pond using the methods provided in the U.S. Army Corps of Engineers (USACE) Coastal Engineering Manual (USACE, 2008), as summarized below.

The licensee determined the maximum straight-line fetch distance over the UHS retention pond to be 690 ft (210 m). The licensee considered the effect of a 2-year return period wind speed of 40.4 mph (18 m/s) based on a statistical analysis of meteorological data for Columbia, Missouri obtained from the National Climatic Data Center. The licensee stated its selection of the 2-year return period was based on guidance provided in NUREG/CR-7046 (NRC, 2011e) and ANSI/ANS 2.8 (ANSI/ANS, 1992). The licensee estimated the runup using the Equation VI-5-3 in the USACE Coastal Engineering Manual (CEM) (USACE, 2008). This equation adjusts runup using many multiplicative correction factors, including a reduction factor for influence of slope roughness. The FHRR states that the pond, having a maximum depth of 19.19 ft (5.8 m), is encompassed by a dike covered by ripraps on the inside surface with a slope of 3:1 (3 horizontal to 1 vertical) to protect the dike from wind effects. The licensee assumed that the inside slope of the dike is a smooth and impermeable surface, and selected a roughness reduction factor of 1 (no correction). Under this assumption, the licensee obtained a wave setup of 0.02 ft (0.01 m) and runup height of 0.83 ft (0.25 m).

The staff recalculated the wave setup and runup heights on the UHS retention pond with the inside slope of the dike as a riprapped surface. The staff used a realistic roughness reduction factor for the riprapped slope of 0.55 (e.g., 55 percent reduction of calculated runup height) and obtained a wave setup and runup height of 0.48 ft (0.15 m). The staff's runup estimate is still conservatively high as the runup equation is based on a bounding runup equation at the two percent exceedance probability level (USACE, 2008).

For the UHS retention pond, the licensee reported in FHRR Table 3-5 that the reevaluated LIP flood elevation with wind effects is 838.04 ft (255.4 m) MSL. This is based on a simulation of the HEC-RAS model with a 6-hour PMP scenario and an estimated wind-driven wave height of 0.85 ft (0.26 m). The licensee set up HEC-RAS so that the UHS retention pond (subbasin 58) receives PMP as inflow and drains water to subbasin 56 through a spillway having a 20 ft (6 m) width broad-crested weir with a weir coefficient of 2.65. In the response of RAI Nos. 3 and 13

(Reasoner, 2014a and 2014b), the licensee provided the result of a comprehensive sensitivity analysis of the LIP floods for different durations, incremental time steps, and temporal distributions of PMP.

The licensee did not provide in its RAI response an updated LIP flood level with wind effects on the UHS retention pond. However, the staff determined the value based on the information of the RAI response (Reasoner, 2014a and 2014b) as shown on the Table 3.0-1 of this report. That is, the staff determined, based on the review of the sensitivity analysis provided in the RAI response, that the most reasonable estimate of the UHS retention pond flood level is 837.51 ft (255.27 m) MSL as presented for subbasin 58 (UHS retention pond) on the Table 7-9 of the calculation package (Reasoner, 2014b). This flood value was estimated based on the value of 0.7 for the time of runoff concentration, PMP at 1-minute intervals, and a HEC-RAS unsteady (transient) flow simulation option that uses time-varying inflows. Using the staff-estimated wave height on the UHS retention pond of 0.48 ft (0.15 m), the staff obtained the LIP flood level with wind effects on the UHS retention pond of 838.0 ft (255.4 m) MSL, which is less than the site grade and the corresponding current design-basis flood elevation at the UHS retention pond.

3.2.5 Summary

The staff confirmed the licensee's conclusion that the reevaluated flood hazard for LIP and associated site drainage is bounded by the current design basis flood hazard.

3.3 Streams and Rivers

The licensee reported in its FHRR the reevaluated PMF elevation for site flooding from stream and rivers is 709.05 ft (216.1 m) MSL along Auxvasse Creek. This flood-causing mechanism along the Auxvasse Creek is not described in the licensee's current design-basis. The current design-basis PMF elevation for site flooding from the Missouri River near the vicinity of the site is 548 ft (167 m) MSL.

To provide additional information in support of the summaries and conclusions in its FHRR, the licensee made several calculation packages available to the staff via an electronic reading room. These calculation packages expand upon and clarify the information provided on the docket.

The staff describes its evaluation of site flooding from streams and rivers, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

3.3.1 PMF on Auxvasse Creek

The licensee reviewed nearby streams and rivers to identify sources of potential flooding hazards. The licensee screened out flooding from the Missouri River because the highest flood of record on the river was approximately 304 ft (93 m) below the site grade. Instead, the licensee identified three nearby creeks: Auxvasse Creek, Logan Creek, and Mud Creek. The "Auxvasse Creek Watershed" mentioned in the FHRR includes these three watersheds (see Figure 3.3-1).

The licensee used the HMR 52 computer program (USACE, 1987) and applied the methodology in HMR 51 (NOAA, 1978) and HMR 52 (NOAA, 1982) to develop a 72-hour PMP scenario for the watershed. The approach to develop the PMP scenario is consistent with present-day methodologies and regulatory guidance. For the watershed area, the licensee determined a critical storm area of 300 square miles (777 km^2), a storm orientation of 150 degrees, and a 72-hour PMP depth of 28.6 in (72.7 cm). Figure 3.3-2 shows the rainfall hyetograph the licensee developed for this PMP event.

The licensee used the HEC-HMS and HEC-RAS models (USACE, 2010a and 2010b, respectively) to simulate basin runoffs and flood elevations at the creek. For the HEC-HMS modeling, the watershed was divided into 60 subbasins. The model includes 15 major dams among a total of 57 dams in the watershed.

Using the HEC-HMS model, the licensee simulated five runoff scenarios, ranging from very conservative to less conservative but more realistic scenarios with progressively less conservative assumptions for Scenario 1 through Scenario 4. Scenario 5 was based on Scenario 4, but postulated to represent future watershed conditions by increasing the impervious area in each subbasin. For all scenarios, the licensee assumed no losses on the channels. The licensee also assumed that flow is steady state and one-dimensional. These assumptions result in a conservatively large runoff volume. The licensee used Manning's roughness coefficient of 0.035 for stream channels and 0.1 for overbank areas.

In Scenario 1, which is the most conservative, the licensee assumed no runoff losses in the watershed as well as no rainfall to runoff transformation or flood routing in channels. Instead, they assume that the 72-hour basin PMP event transfers to the basin outlet instantly. Then, HEC-RAS was used to determine water elevations on the creeks. The licensee's analysis determined the highest water surface elevation of 772.7 ft (235.5 m) MSL on a reach of Auxvasse Creek, which is 67.8 ft (20.7 m) below the floor elevation of safety-related SSCs.

For Scenario 2, the licensee reduced the conservatism of the HEC-HMS analysis by using the Clark Unit Hydrograph method for runoff transformation, and estimated a maximum water surface elevation of 717.47 ft (218.7 m) MSL using the HEC-RAS simulation.

For Scenario 3, flood routing was performed using the Muskingum-Cunge 8-point cross-section routing option in HEC-HMS, as was used in the FSAR for the proposed Calaway, Unit 2 (Ameren Missouri, 2009). The licensee determined a maximum water surface elevation of 709.77 ft (216.3 m) MSL for this scenario.

For Scenario 4, the licensee further reduced the conservatism of the HEC-HMS analysis by using the SCS Curve Number option in HEC-HMS to account for runoff losses in the watershed. Curve numbers were assigned on the basis of detailed analysis of soils maps and data on current land use. The HEC-HMS input files provided through the RAI response (Reasoner, 2014a) indicated that the curve numbers range from 78 (less pervious) to 92 (more pervious). This scenario results in a flood elevation of 709.09 ft (216.13 m) MSL.

For Scenario 5, the licensee added 5 percent to the impervious area of each watershed to represent potential future development conditions within the basin, and determined a maximum flood level of 709.1 ft (216.1 m) MSL.

Based on these analyses, the licensee determined that the maximum predicted water elevations from five postulated PMF scenarios on the Auxvasse Creek will not exceed the site grade. Therefore, the licensee concluded that the Auxvasse Creek PMF will not inundate the plant site. Due to the large elevation difference between the maximum PMF water surface elevation on these creeks and the plant grade, the licensee concluded that the Auxvasse Creek PMF coincident with potential associated effects including wind waves and run-up will not affect the site. Accordingly, the licensee did not analyze the associated effects, including wind wave effects, in combination with the PMF on the Auxvasse Creek.

The staff also noted that the PMF analysis submitted to the NRC in the FSAR for the proposed Callaway, Unit 2 (Ameren, 2009) indicated maximum PMF water surface elevations of 677.3 ft (206.4 m) MSL for Logan Creek, 577.6 ft (176.0 m) MSL for Mud Creek, and 704.3 ft (214.7 m) MSL for Auxvasse Creek (Ameren Missouri, 2009), which are below the plant grade.

3.3.2 Combined Events with PMF

The licensee also performed a combined event flooding analysis for the Auxvasse Creek basin using HEC-HMS and HEC-RAS. They considered the PMF coincident with mean monthly base flow, median soil moisture, antecedent rain, and wave effects. This approach is recommended by NUREG/CR-7046 (NRC, 2011e).

The licensee selected the highest of the recorded mean monthly flows at a basin outlet gaging station, then used, in HEC-HMS, the adjusted base flows to each subbasin based on the ratio of the gaged basin area to subbasin area. The licensee used 40 percent of the 72-hour, 5-minute interval PMP as an antecedent event, followed by a 72-hour PMP after 3-days without rain as a main event in HEC-HMS. The licensee ran HEC-HMS for five different scenarios that combine different loss rates, rainfall-runoff transformations, and channel routing methods. The licensee simulated flood levels near the vicinity of the site for each scenario using HEC-RAS, which uses the basin outflow hydrograph simulated by HEC-HMS as an upstream input.

The resulting flood levels for the five scenarios for combined events with wind wave effects range from 709.86 ft (216.4 m) MSL to 772.76 ft (235.5 m) MSL. The licensee selected the lowest value which is the most realistic scenario. The staff reviewed the licensee's reevaluation of combined PMF events on the Auxvasse. The staff noted that the licensee's reevaluation of the combined PMF events described in its FHRR used present-day methodologies and regulatory guidance. The staff also agreed with the licensee's position that a separate analysis of associated effects on PMF, including wind wave effects, is not necessary because the estimated Auxvasse Creek PMF elevations are below the plant grade. This combined flood-causing mechanism on the Auxvasse Creek is not described in the licensee's current design-basis.

The FHRR Table 2-3 states that the bottom of makeup water intake structure located on the Missouri River is at 525 ft (160 m) MSL elevation which is lower than the estimated Missouri River flood level. However, the FHRR Section 3.2.7 states that the intake structure is not a safety-related structure and is not required for safe shutdown of the plant. Therefore, the licensee screened out the effects of the Missouri River flooding on the intake structures, including pump and pipeline, in its flood hazard reevaluation for rivers and streams.

The staff confirmed the licensee's conclusion that the PMF from rivers and streams would not inundate the plant site.

3.4 Failure of Dams and Onsite Water Control/Storage Structures

The licensee reported in the FHRR that the reevaluated flood elevation for site flooding due to failure of dams and onsite water control/storage structures does not inundate the plant site.

This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis elevation for site flooding on the Missouri River due to failure of dams and onsite water control or storage structures is 551 ft (167.9 m) MSL (FHRR Table 3-5). In the FHRR, the licensee screened out a potential Missouri River dam failure flooding, but focused on an evaluation of Auxvasse Creek dam failure flooding, as the Auxvasse Creek has a higher elevation than that of the Missouri River.

To provide additional information in support of the summaries and conclusions in the FHRR, the licensee made several calculation packages available to the staff via an electronic reading room. These calculation packages expand upon and clarify the information provided on the docket.

The staff describes its evaluation of site flooding from failure of dams and onsite water control or storage structures, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

The licensee reevaluated potential dam flooding on the Auxvasse Creek using two simple bounding scenarios. First, the licensee investigated whether a potential dam failure flood level on the creek reaches the plant grade or not. The licensee assumed that all upstream dams fail simultaneously with a PMP-induced upstream inflow to each dam. The licensee used a 72-hr basin PMP depth of 28.6 inches (72.7 cm). Using HEC-HMS and HEC-RAS models with the PMP scenario, the licensee simulated a PMF level of less than 810 ft (247 m) MSL, which is lower than the site grade. Second, the licensee analyzed potential creek flooding based on a comparison of the potential flood volume created by failing all dams within the basin and the creek storage capacity for a hypothetical dam at the basin outlet. The licensee conservatively assumed that all dams fail simultaneously and the breach outflows from reservoirs transport to the hypothetical onsite reservoir without loss or attenuation. The total combined maximum storage of all upstream reservoirs is 16,148 ac-ft (19.91 million m^3). Assuming that the pre-existing flood level caused by any flooding mechanism is at the site grade of 840 ft (256.03 m) MSL, the licensee estimated the capacity of the hypothetical reservoir for the water levels between the site grade and the SSC elevation of 840.5 ft (256.18 m) MSL. From this analysis, the licensee identified the hypothetical storage volume is 3.6 times larger than the sum of all upstream reservoir storage volumes, indicating the flood level created by this second dam failure scenario would not reach the elevation of safety-related SSCs. Because the estimated dam failure flood level is about 30 ft (9 m) below the plant grade, the licensee did not analyze any combined events or associated effects on dam failure flooding.

Similar to the second scenario above, the staff performed a confirmatory bounding dam failure flood analysis at the Auxvasse Creek. The staff also assumed that the water level before the postulated dam failure is at the licensee-estimated PMF level. The staff identified a total of 58 upstream reservoirs from the latest National Inventory of Dams (NID) database (USACE, 2013) which provides the location and dimensions of dams and reservoirs. The storage volumes of these reservoirs range from 27 to 2,484 acre-feet (ac-ft) (0.033 to 3.06 million cubic meters (m^3). The staff calculated a total volume of 16,385 ac-ft (20.2 million m^3) which is only one percent larger than the licensee's estimate. Moreover, the estimated flood levels for both licensee's and staff's storage volumes are nearly identical and below the plant grade elevation. Therefore, the staff agrees with the licensee's conclusion that the potential dam failure flooding on the Auxvasse Creek will not inundate the plant site.

The onsite UHS pond has a slope surface that is covered by riprap and the design pond water level is lower than the plant grade, thus the staff determined that flooding from slope failure is not a plausible scenario.

In summary, the staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from dam failure does not inundate site.

3.5 Storm Surge

The licensee reported in its FHRR that the reevaluated PMF, including associated effects, for site flooding due to storm surge does not inundate the Callaway site. This flood-causing mechanism is screened out in the licensee's current design-basis.

The Callaway, Unit 1FSAR (Ameren Missouri. 2012a) states that the protection of safety-related facilities against surges and wave action is not required because the site is located on the site plateau about 325 feet (97.5 m) above the Missouri River floodplain and is not subject to flooding or other water-related phenomena associated with the Missouri River. The only body of water on the site is from the UHS retention pond that is small in volume and is not subject to significant surges.

FHRR Section 3.2.4 states that the Callaway site is not near any large bodies of water for which storm surge flooding would apply. Therefore, the licensee determined that flooding at the site due to storm surge is not expected to be a potential flooding hazard. According to the guide on ANSI/ANS-2.8 (ANSI/ANS, 1992), the region of occurrence of a hurricane surge shall be considered for United States coastline areas and areas within 100 to 200 miles (160 to 320 km) bordering the Gulf of Mexico. Because the Callaway site is located more than 620 miles (990 km) inland from the Gulf of Mexico, and at a site grade of approximately 840 ft (256 m) MSL, the staff determined hurricane surge does not present a potential flooding hazard at the site.

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from storm surge would not inundate the Callaway site.

3.6 Seiche

The licensee reported in its FHRR that the reevaluated PMF, including associated effects, for site flooding due to seiche does not affect the Callaway site because the site is located on a high inland area and far from a large body of water. This flood-causing mechanism is screened out in the licensee's current design-basis.

The Callaway, Unit 1 FSAR (Ameren Missouri, 2012a) states that the protection of safety-related facilities against seiches and wave actions is not required as the site is not subject to flooding or other water-related phenomena associated with the Missouri River. The only body of water on the site is the UHS retention pond which is small in volume and is not subject to significant seiches. FHRR Section 3.2.5 states that the site is not near to any bodies of water for which seiche flooding would apply.

The staff confirmed the reevaluated hazard for flooding from seiche is bounded by the current design-basis flood hazard.

3.7 Tsunami

The licensee reported in its FHRR that the reevaluated PMF, including associated effects, for site flooding due to tsunami does not affect the Callaway site because the site is located on a high inland area and far from a large body of water. This flood-causing mechanism is screened out in the licensee's current design-basis.

The licensee stated in its FHRR Section 3.2.6 the Callaway site is not near any large bodies of water for which tsunami flooding would apply. The Callaway site is over 620 miles (990 km) away from the Gulf of Mexico and the plant grade is approximately 840 ft (256 m) MSL above sea level. Therefore, the licensee determined tsunami flooding is not a risk to the site.

The staff confirmed the reevaluated hazard for flooding from tsunami is bounded by the current design-basis flood hazard.

3.8 Ice-Induced Flooding

The licensee reported in its FHRR that the reevaluated PMF, including associated effects, for ice-induced flooding of the site would not pose a hazard to the site. This flood-causing mechanism is discussed but screened out in the licensee's current design-basis.

To provide additional information in support of the summaries and conclusions in its FHRR, the licensee made several calculation packages available to the staff via an electronic reading room. These calculation packages expand upon and clarify the information provided on the docket.

In its FHRR, the licensee stated that, based on a review of the USACE Cold Regions Research and Engineering Laboratory Ice Jams Database (USACE, 2012), no new ice jam events have occurred during the past 40 years. The closest ice jam to the site was recorded at Boonville, Missouri on December 19, 1945. To date, ice events have not affected the operation of Callaway, Unit 1 (Ameren Missouri, 2009). The licensee also stated that streams close to the site have small drainage areas and would not pose the potential of ice flooding at the site.

The FHRR states that the maximum calculated theoretical ice thickness for the Missouri River was estimated to be approximately 4.43 in (11.3 cm). The licensee also described in its FHRR that ice and ice flooding on the Missouri River will not create a flooding problem because (1) the intake and discharge structures on the river are not safety-related structures, and (2) the warm discharge water will keep the discharge outfall open from forming any ice. The licensee also stated the potential of frazil ice accumulation blocking the Essential Service Water pump intakes is low due to the small size of the UHS retention pond, its location sheltered from the wind, and warm discharge water from the plant.

The licensee stated in its FHRR that site drainage and plant yard grading are designed to handle the runoff from local winter PMF without affecting safety-related structures, and that the licensee assumed clogging of inlets and certain size culverts by ice in their modeling of onsite flood simulations. The licensee stated a potential for ice forming on the UHS retention pond or a frazil ice blockage of the Essential Service Water pump intake are low due to the small size of the pond and warm water inflow to the pond.

The staff noted that the licensee provided sufficient information to determine that the ice formation on the rivers and streams will not create any onsite flood hazards. The staff also noted that any potential ice-formed flood on rivers and streams will be bounded by the dam failure flood postulated in the previous section because the potential inundation depth of ice jams on the rivers and streams will not exceed the height of real or hypothetical dams analyzed.

The staff confirmed the reevaluated hazard for ice-induced flooding of the site is bounded by the current design basis flood hazard.

3.9 Channel Migrations or Diversions

The licensee reported in its FHRR that the reevaluated PMF, including associated effects, for site flooding due to channel migrations or diversions does not affect the CEC site. This flood-causing mechanism is screened out in the licensee's current design-basis.

The licensee analyzed in the FHRR channel diversion flooding at the site with respect to seismic, topographical, and geological evidence in the region as follows. In the early 1940s and 1950s, the Missouri River was transformed to its current physical features through extensive man-made alterations, such as constructing a series of large reservoirs, and altering reaches, regulating upstream flows, and stabilizing banks. Specifically, channelization of the lower Missouri River eliminated sand bars, extensive depth variations, and connection with side channels and backwaters. As a result, channel diversions and migrations have become a rare occurrence along the lower Missouri River.

FHRR Section 3.2.8.3 states that, as there is no evidence of deformation zones, and capable faults, and isolated plateau with deeply incised drainage patterns near the site, potential channel diversions or migrations due to seismic activity is not expected during the lifetime of the plant. Correspondingly, the licensee concluded flooding hazards due to potential channel diversions and migrations will not pose any risk for the safe operation of the plant.

The staff reviewed the topographical and geological information provided by the licensee as well as obtained USGS topographical maps (USGS, 2013). The staff found from the review that the site is isolated and protected naturally from the Missouri River so that flooding caused by channel diversions or migrations, if any, will not inundate the plant site. The staff also found the potential of upstream diversion flooding on the Auxvasse Creek is extremely remote because (1) the channel has not changed in recent history, (2) the Auxvasse Creek in the site vicinity lies in a 250 ft (76.2 m) deep valley from the plateau on the site, and (3) there is no apparent man-made or natural event that could divert the creek.

The staff confirmed the reevaluated hazard for flooding from channel migrations or diversions is bounded by the current design-basis flood hazard.

4.0 INTEGRATED ASSESSMENT AND ASSOCIATED HAZARD DATA

The staff confirmed the reevaluated hazard results for all flood-causing mechanisms are bounded by the current design-basis flood hazard or do not inundate the plant site. Therefore, the staff concludes an Integrated Assessment is not necessary.

5.0 CONCLUSION

The NRC staff reviewed the information provided for the reevaluated flood-causing mechanisms of Callaway, Unit 1. Based on its review, the staff concludes the licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews.

Based upon the preceding analysis, the NRC staff confirmed the licensee responded appropriately to Enclosure 2, of the 50.54(f) letter, dated March 12, 2012. In reaching this determination, staff confirmed the licensee's conclusions (a) the reevaluated hazard results for each reevaluated flood-causing mechanism are bounded by the current design-basis flood hazard or do not inundate the plant site, and (b) an Integrated Assessment is not necessary. The NRC staff has no additional information needs at this time with respect to Enclosure 2.

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Table 2.2-1. Flood-Causing Mechanisms and Corresponding Guidance

Flood-Causing Mechanism	SRP Section(s) and JLD-ISG
Local Intense Precipitation and Associated Drainage	SRP 2.4.2 SRP 2.4.3
Streams and Rivers	SRP 2.4.2 SRP 2.4.3
Failure of Dams and Onsite Water Control/Storage Structures	SRP 2.4.4 JLD-ISG-2013-01
Storm Surge	SRP 2.4.5 JLD-ISG-2012-06
Seiche	SRP 2.4.5 JLD-ISG-2012-06
Tsunami	SRP 2.4.6 JLD-ISG-2012-06
Ice-Induced	SRP 2.4.7
Channel Migrations or Diversions	SRP 2.4.9

Table 3.0-1 Summary of Controlling Flood-Causing Mechanisms

Reevaluated Flood-Causing Mechanisms and Associated Effects that May Exceed the Powerblock Elevation (1)		ELEVATION (ft(m), MSL)
Local Intense Precipitation and Associated Drainage	Powerblock Area	840.4 (256.2)(2)
	UHS Retention Pond	838.0 (255.4)(3)

Notes:

(1) Flood Height and Associated Effects as defined in JLD-ISG-2012-05.
(2) From the response to RAI 13 (Reasoner, 2014a).
(3) LIP flood level of 837.51 ft (255.3 m) MSL (from RAI response Table 7-9 (Reasoner, 2014b)) plus wind effects of 0.48 ft (0.15 m) estimated by the staff (See Section 3.2.4).

Table 3.1.2-1 Current Design-Basis Flood Hazards

Flooding Mechanism	Still-Water Level (m(ft) MSL)	Associated Effects (ft (m)	Current Design Basis (CDB) Flood Elevation (ft(m) MSL)	Reference
LIP and Associated Drainage - Plant site	Not Discussed	Not Discussed	"Less than" 840.5 (256.2) from onsite runoff	FHRR 2.3 & Table 2-4
- UHS Retention Pond	837.7 (255.3)	0.6 (0.18) for wave run-up	838.3 (255.5) from UHS retention pond, including effects of wave run-up	FSAR 2.4.8.2.1
Streams and Rivers/ Missouri River	548 (167.03)	Not Discussed	548 (167.03)	FHRR Section 2.5.1 and 2.6
Failure of Dams and Onsite Water Control/Storage Structures	551 (168) including wave run-up at Missouri River	Not Discussed	551 (167.9)	FHRR Tables 2-3 and 2-4
Storm Surge	No Impact Identified	Not Discussed	No Impact Identified	FHRR Table 2-4
Seiche	No Impact Identified	Not Discussed	No Impact Identified	FHRR Table 2-4
Tsunami	No Impact Identified	Not Discussed	No Impact Identified	FHRR Table 2-4
Ice-Induced	No Impact Identified	Not Discussed	No Impact Identified	FHRR Table 2-4
Channel Migrations or Diversions	No Impact Identified	Not Discussed	No Impact Identified	FHRR Table 2-4

Table 3.2-1 Licensee's PMP values for Callaway Site local intense precipitation (from the response to RAI No. 3, Table 3-1 (Reasoner, 2014a)).

Duration	Area, mi^2(km^2)	PMP, in.(cm)
72 hour	10 (25.9)	40.20 (102.1)
48 hour	10 (25.9)	38.76 (98.5)
24 hour	10 (25.9)	34.80 (88.4)
12 hour	10 (25.9)	33.05 (93.9)
6 hour	10 (25.9)	27.64 (70.2)
1 hour	1 (2.6)	18.3 (46.5)
30 min	1(2.6)	13.83 (35.1)
15 min	1 (2.6)	9.61 (24.4)
5 min	1(2.6)	6.11 (15.5)

Figure 2.2.4-1 Flood Event Duration

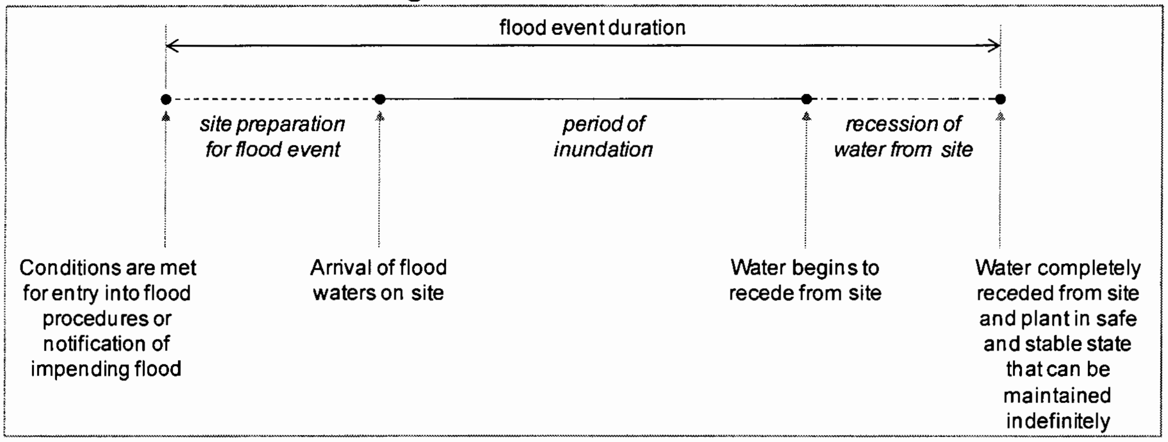


Figure 1-1 Callaway Energy Center site location map. Locations of USGS gauging stations are indicated by diamonds (modified from FHRR Figure 1-1).

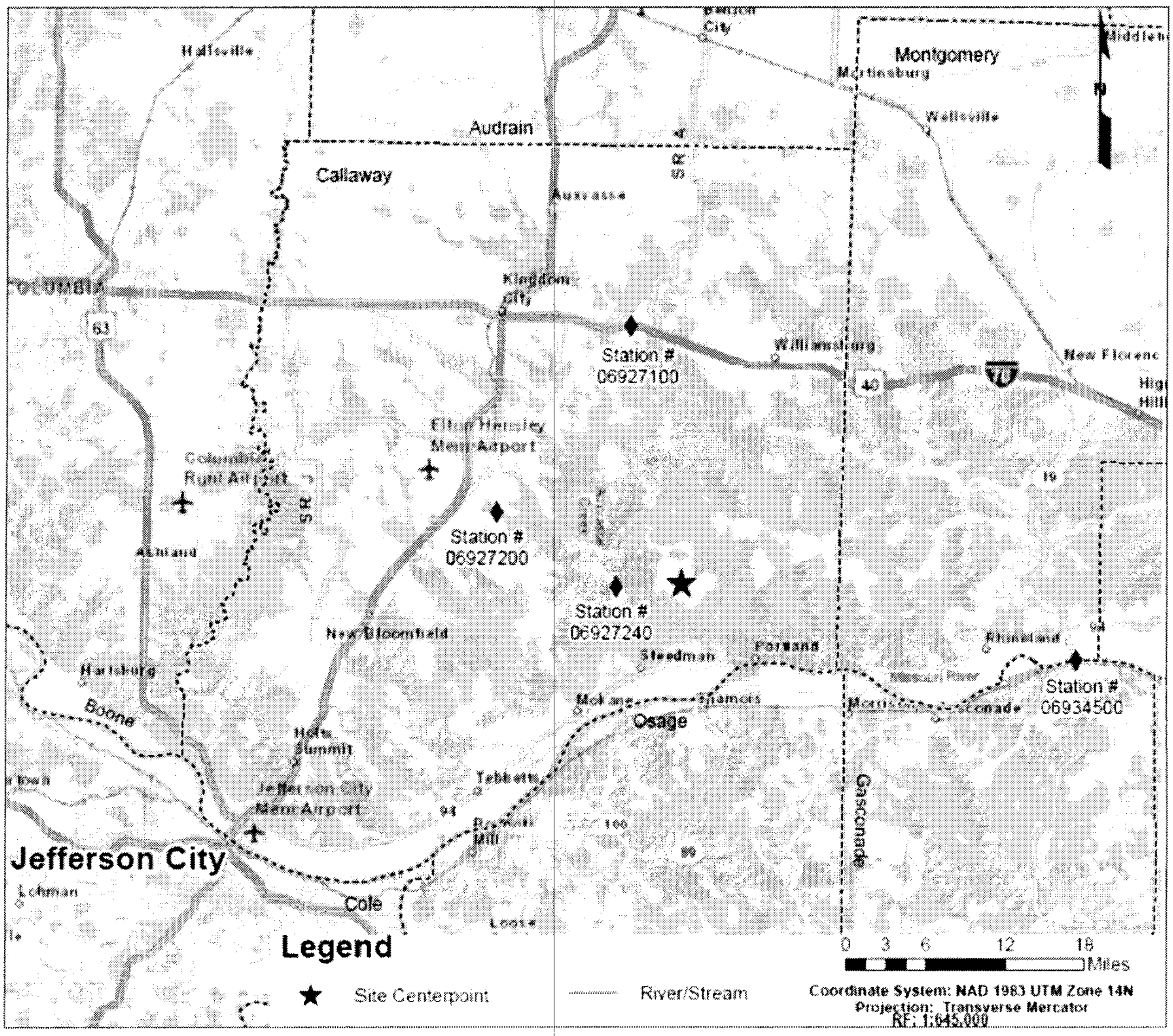


Figure 3.2-1: HEC-HMS subbasins delineated on the Callaway Energy Center site (from the RAI response (Reasoner, 2014a), Figure 2-2).

Figure 3.2-2: Synthetic hyetograph used in licensee's HEC-HMS analysis of LIP (from the FHRR Figure 3-2).

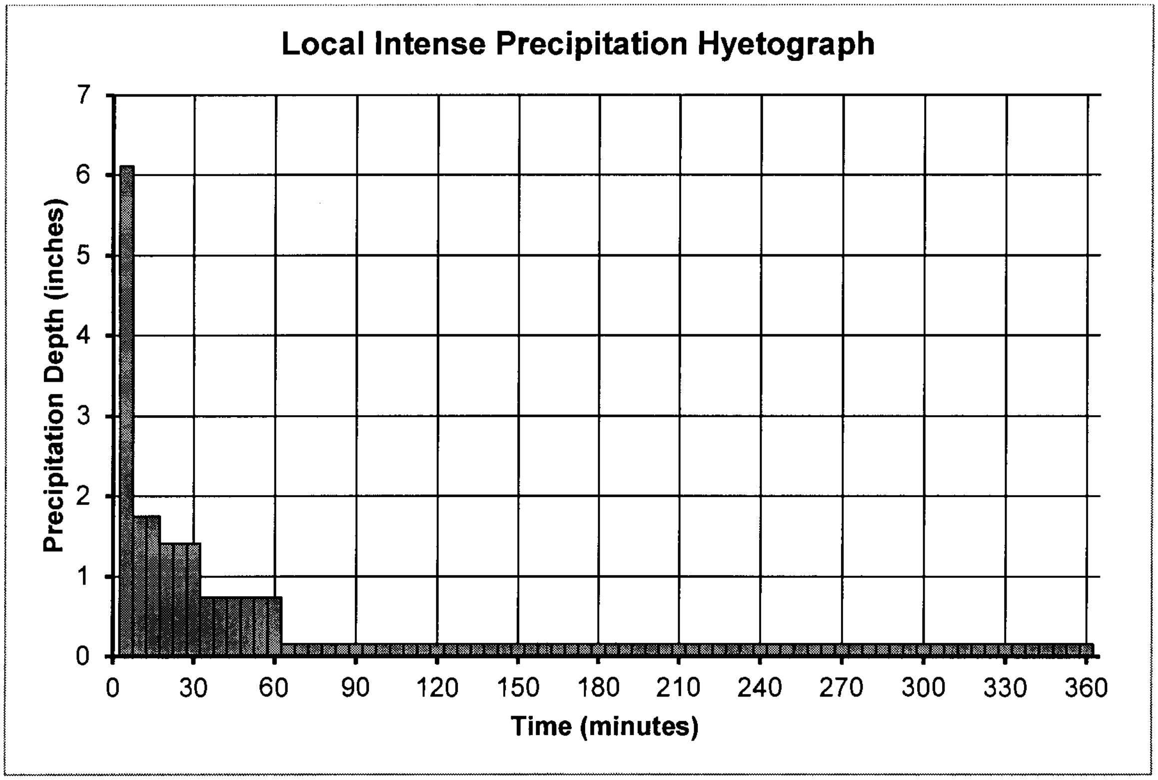


Figure 3.3-1: Map of the combined watersheds of Auxvasse Creek, Logan Creek, and Mud Creek (from the FHRR Figure 3-6).

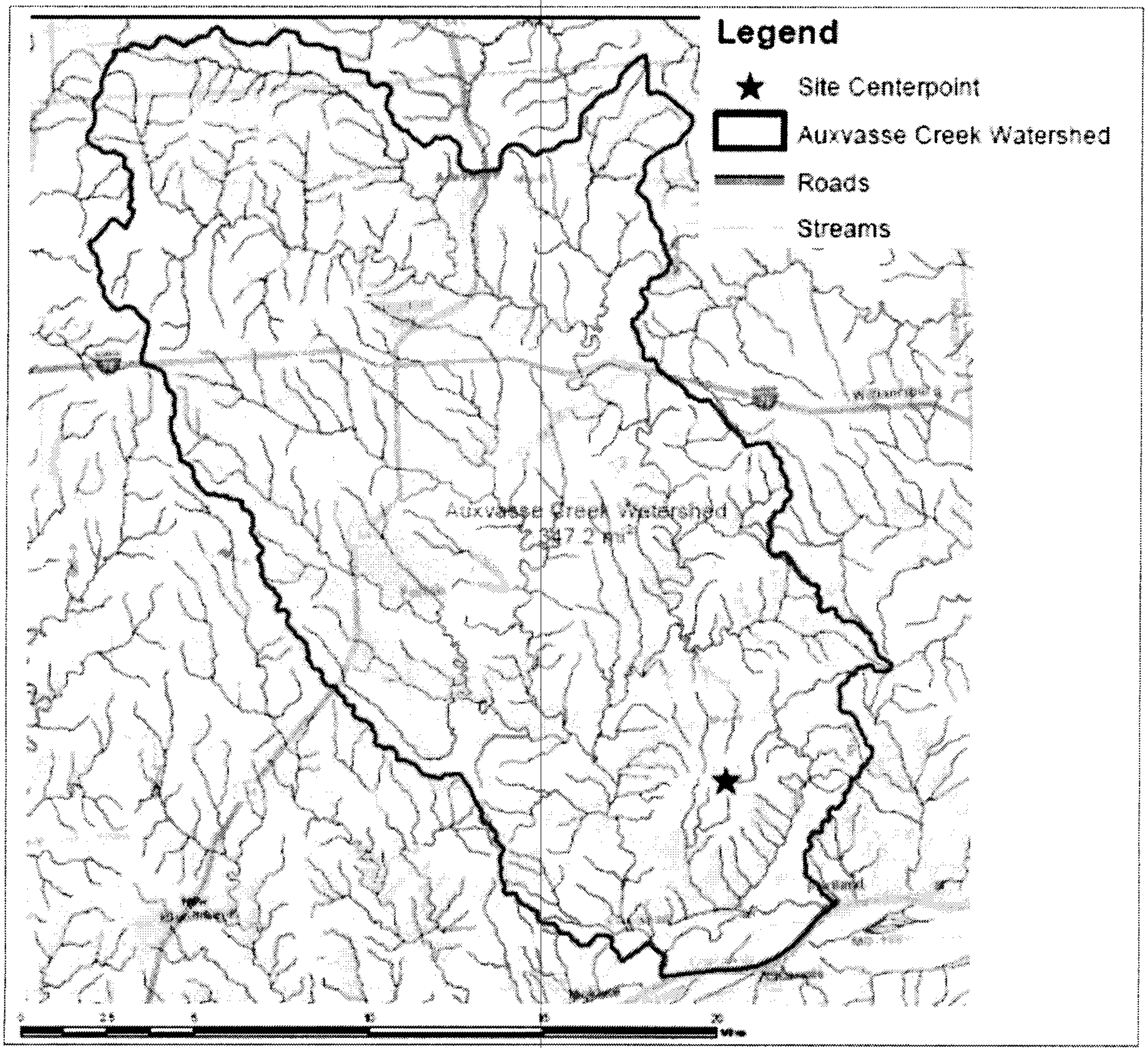
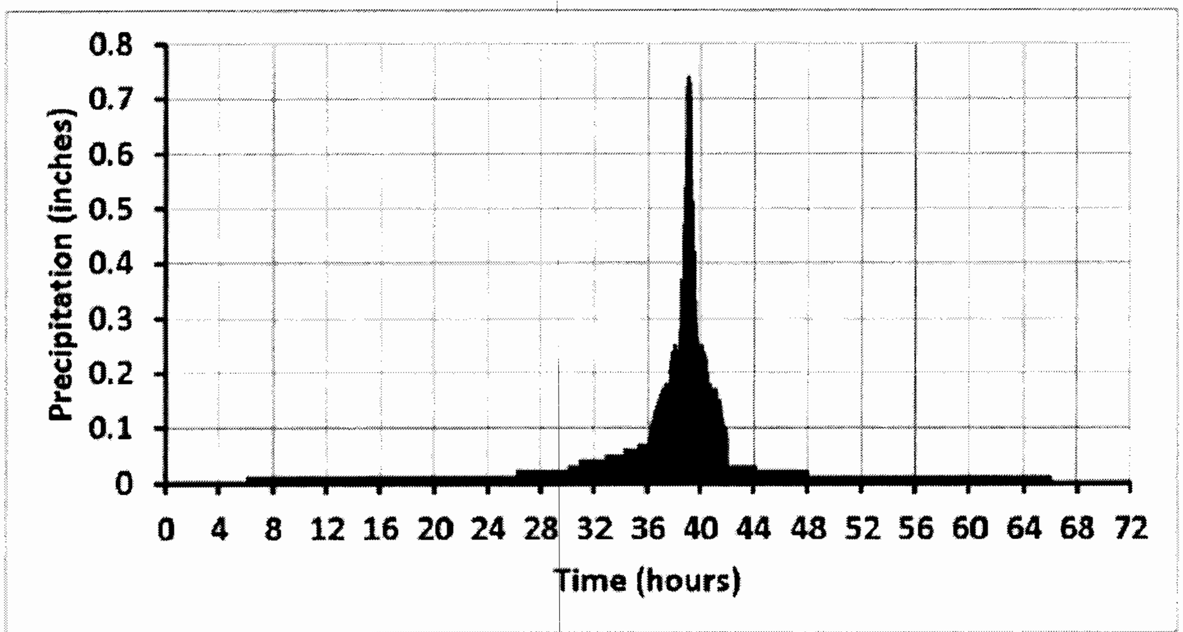


Figure 3.3-2: Rainfall hyetograph for 72-hour probable maximum storm, showing rainfall rates for 5-minute time periods (from the FHRR Figure 3-7).



PROBABLE MAXIMUM STORM FOR AUXVASSE CREEK WATERSHED

If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

Sincerely,

/RA/

Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket No. 50-483

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report

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