CALLAWAY ENERGY CENTER UNIT 1
FLOODING HAZARD REEVALUATION REPORT

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The U.S. Nuclear Regulatory Commission (NRC) issued a letter on March 12, 2012 pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f) related to the implementation of Recommendations 2.1, 2.3, and 9.3 from the Near-Term Task Force (NTTF), a portion of which calls for performing flood hazard reevaluations at all Nuclear Power Plants (NPPs) in the United States (NRC, 2012a). As part of Westinghouse Electric Company (WEC) requirements of service to address NRC Recommendation 2.1, Paul C. Rizzo Associates, Inc. (RIZZO) has prepared this Flooding Hazard Reevaluation Report for Callaway Energy Center, for submittal to WEC with due consideration to the most recent guidance and regulations. This Flooding Hazard Reevaluation Report will ultimately become part of the Callaway response submittal to NRC Recommendation 2.1.

1.2 DESCRIPTION OF STUDY AREA

The Callaway Energy Center Site is located in Callaway County, Missouri, approximately 25 miles northeast of Jefferson City. A general site location map is seen on ***Figure 1-1***. The site is positioned on a plateau which ranges from 830 to 850 feet (ft) in elevation, approximately 325 ft above the Missouri River floodplain, and approximately five miles north of the Missouri River (Mile 115). Surface drainage is intercepted by the Auxvasse Creek and one of its tributaries Cow Creek to the north and west of the site; whereas surface drainage to the east is intercepted by Logan Creek, and to the southwest by Mud Creek. The Missouri River is used as the principal source of makeup water for the cooling tower system (Ameren Missouri, 2012a).

1.3 SITE BACKGROUND AND HISTORY

The Callaway Energy Center is owned by Ameren Corporation and operated by Ameren Missouri. Callaway Energy Center, which is Missouri's only commercial nuclear unit, began commercial operation in December, 1984. In 2008, Ameren Missouri submitted an application for a potential second unit at the site, located northwest of Unit 1.

2.0 FLOOD HAZARDS AT THE SITE

Section 2.0 has been prepared in response to Requested Information Item 1.a. of NRC Recommendation 2.1 (NRC, 2012a).

2.1 HISTORIC FLOODS

Historical flooding at the Callaway Energy Center Site is of interest mainly with respect to the Missouri River, although other smaller tributaries exist near the site. The Callaway Energy Center Site is a topographic high in the area, with surface runoff draining radially into small intermittent streams (Ameren Missouri, 2012a). These streams are branches of Logan, Mud, Cow, and Auxvasse Creeks. The preceding river and streams were identified based on an initial qualitative screening of rivers, streams, and creeks in the vicinity of the Callaway Energy Center Site. This was carried out based on the distance from and location relative to the Site, as well as historical flow rates and low-flow considerations.

All elevations in this Report are given with respect to mean sea level (MSL), unless otherwise specified on a case-by-case basis.

Historical floods of record near the site have most commonly been attributed to heavy rainfall and snowmelt, leading to the flooding of the Missouri River floodplain (Ameren Missouri, 2012a). Ice formation on the Missouri River has occurred, however, no record of ice jams or ice-induced flooding exists at the Site. Records of landslides or tsunami source induced flooding events at the Site are also non-existent (Ameren Missouri, 2009). A list of the most significant historical floods with respect to the maximum water levels near the Callaway Energy Center Site is provided in ***Table 2-1***.

The flood of record near the Callaway Energy Center Site on the Missouri River occurred in 1993, and is regarded as one of the most significant and damaging natural disasters in the region. The United States Geological Survey (USGS) gauge stations nearest the Site are Boonville (06909000) and Hermann (06934500), upstream and downstream, respectively of Missouri River mile 115. The maximum recorded flow for both stations was in late July, 1993. At the Boonville gauge station, a flow of approximately 755,000 cubic feet per second (cfs) and a flood level at 602.5 ft was recorded. The Boonville gauge station lies over 50 miles west-northwest of

the Callaway Energy Site (see ***Figure 1-1***). At the Hermann gauge station, the recorded flow and flood level were 750,000 cfs and 518.53 ft, respectively (Ameren Missouri, 2009).

Also of note is the flood of 1844, the first year of continuous streamflow recording, with estimated peak streamflows of 700,000 cfs at the Hermann gauging station and 710,000 cfs at the Boonville gauging station. These flows are equivalent to a flood level of 539 ft near the Callaway Energy Center Site under present channel conditions (Ameren Missouri, 2009).

The Site is at an elevation of 840 ft on an upland plateau located north of the Missouri River. The highest flood recorded for the Missouri River near the Site was approximately 300 ft below the Site's elevation (Ameren Missouri, 2012a).

2.2 DETAILED SITE INFORMATION

Section 2.2 has been prepared in response to Requested Information Item 1.a.i. of NRC Recommendation 2.1 (NRC, 2012a). The following sections present detailed information regarding the Callaway Energy Center Site, including information regarding on-site topography, a description of relevant safety-related Structures, Systems, and Components (SSCs), and a description of the Ultimate Heat Sink (UHS).

2.2.1 Designed Site Information

Designed site information describes characteristics considered for the original licensing basis of the Callaway Energy Center Site. ***Figure 2-1*** shows the Callaway design site layout and topography. A list of the Callaway Energy Center safety-related structures, systems, and components can be found in ***Table 2-2***. A list of design parameters found in the license document (Ameren Missouri, 2012a) is included in ***Table 2-3***.

2.2.1.1 Site Topography

The Callaway Energy Center Site is part of the Ozark Plateaus Province, located atop a plateau within the Auxvasse Creek watershed, and Missouri River basin (Ameren Missouri, 2009). The plateau, approximately five miles north of the Missouri River (Mile 115), ranges from 830 to 850 ft in elevation compared to the Missouri River basin elevation of 525 ft (Ameren Missouri, 2009). The Callaway Energy Center Site itself sits at approximately 840 ft elevation, with all

Safety-Related Structures at 840.5 ft, except the UHS retention pond at 840 ft (Ameren Missouri, 2012a).

2.2.1.2 Description of Safety-Related Structures, Systems, and Components

The Callaway Energy Center Site contains several safety-related SSCs. Safety-related SSCs at the Site 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 Pumphouse (Ameren Missouri, 2012a). A list of all designed Callaway Energy Center safety-related SSCs and their elevations can be found in ***Table 2-2***, and their locations can be seen on ***Figure 2-2***.

2.2.1.3 Description of the Ultimate Heat Sink

The source of safety-related cooling water for the Callaway Energy Center Site is the UHS retention pond, which can supply 56.03 acre-ft of water to the UHS Cooling Tower in the case of an emergency safety shutdown (Ameren Missouri, 2012a). The UHS retention pond has a surface area of about 4.1 acres and is located 400 ft southeast of Callaway Energy Center Unit 1. The normal water surface elevation and normal water depth of the pond is 836 ft and 18 ft, respectively (Ameren Missouri, 2012a).

The UHS retention pond is underlain by accretion-gley and glacial till, which have extremely low permeability. Seepage losses through these materials do not significantly affect pond storage (Ameren Missouri, 2012a). Grading around the UHS retention pond is designed to prevent surface runoff from flowing into the pond. The plant yard is also graded away from the pond to prevent Site runoff from entering the pond. In addition, an overflow spillway keeps the pond water level below an elevation of 836.5 ft (Ameren Missouri, 2012a). The graded elevation around the UHS retention pond provides a 4 ft minimum free board at normal pond water level (Ameren Missouri, 2012a).

2.2.2 As-Built Site Information

As-built site information describes changes to the Callaway Energy Center Site which occurred since the current licensing basis, which may have the potential to influence the reevaluation of hydrological and flooding hazards. ***Figure 2-3*** shows the as-built site layout and topography.

2.2.2.1 Site Topography

All flooding analyses described in this Report, and described in ***Section 3.0,*** have been undertaken with consideration and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. Changes to regional and site topography since the time of license issuance have been considered within these analyses.

2.2.2.2 Description of Safety-Related Structures, Systems, and Components

As stated in the Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b), "…the walkdown visual inspection has verified that the Callaway site permanent safety-related SSCs are acceptable, not degraded, and capable of performing the design function as credited in the current licensing basis." Furthermore, "(t)here is reasonable assurance that the flood protection features are available, functional, and capable of performing their specified functions as set forth in the current licensing basis" (Ameren Missouri, 2012b). Flood related changes to the licensing basis are discussed in ***Section 2.4***. The results of the flooding hazard reevaluation, with respect to safety-related SSCs, are presented in ***Section 3.0***.

2.2.2.3 Description of the Ultimate Heat Sink

The UHS Retention Pond was completed on April 10, 1980, and from May 5 to September 26, 1980, a test was performed to determine the rate of seepage from the pond (Ameren Missouri, 2012a). The average seepage rate was found to be less than 0.5 acre-ft for a 30-day period and likely on the order of 0.3 acre-ft. A seepage loss of 0.5 acre-ft would result in an approximately 1.5 in. drop in the retention pond water surface at the normal operating level (Ameren Missouri, 2012a). If the maximum weekly seepage rate calculated from the seepage test data was projected to 30 days, the seepage loss would be slightly less than 1.0 acre-ft (Ameren Missouri, 2012a).

2.3 CURRENT DESIGN BASIS

Section 2.3 has been prepared in response to Requested Information Item 1.a.ii of NRC Recommendation 2.1 (NRC, 2012a). The current design basis, as presented in the Final Safety Analysis Report (FSAR) for the Callaway Energy Center Site, indicates that all safety-related structures, systems, and components, except the UHS Retention Pond, are not subject to flooding, wave action, or wave run-up and do not require flood or wave protection (Ameren

Missouri, 2012a). The UHS retention pond is the only safety-related structure that is subject to wind wave activity and wave run-up. The UHS retention pond has relatively short dimensions with riprap-covered side slopes. Therefore, the wind wave activity does not represent a major concern. The wave run-up is also not a major concern due to the maximum wave run-up reaching an elevation of 838.3 ft, which is below the Site grade elevation. The Site is at an elevation of 840 ft, and all safety-related structures, systems, and components are located at an elevation of 840.5 ft or above, on an upland plateau located north of the Missouri River. The highest flood recorded for the Missouri River near the Site was approximately 300 ft below the Site's elevation (Ameren Missouri, 2012a). Since snow and ice can be prevalent in the Site area, the design of the roofs for the safety-related buildings have included consideration for accumulation of snow and ice. In addition, adequate drainage for the Site is available and will prevent the safety-related structures from being flooded, and will direct flood water and storm runoff away from the Site towards the surrounding stream systems via drainage ditches and contour grading (Ameren Missouri, 2012a).

2.4 FLOOD-RELATED CHANGES TO THE LICENSING BASIS

Section 2.4 has been prepared in response to Requested Information Item 1.a.iii. of NRC Recommendation 2.1 (NRC, 2012a).

2.4.1 Description of Hydrological Changes and Flood Elevations

The design basis flood elevations for the Callaway Energy Center are summarized in ***Table 2-4***. A description of flood protection changes is provided in ***Section 2.4.2***. Reevaluated flood elevations are presented in ***Section 3.0***, and are summarized in ***Table 3-2***.

2.4.2 Description of Flood Protection Changes (Including Mitigation)

The Post Fukushima Flooding Walkdown Report for Callaway Energy Center states that, "(t)he reviews determined Callaway site flood protection features would be capable of performing their intended flood protection function if subjected to a design basis flooding hazard," and that "(n)o flooding walkdown observations were deemed to be a deficiency per Section 3.8 of NEI 12-07" (Ameren Missouri, 2012b).

2.5 CHANGES TO THE WATERSHED AND LOCAL AREA

Section 2.5 has been prepared in response to Requested Information Item 1.a.iv. of NRC Recommendation 2.1 (NRC, 2012a). Any changes to the contributing watershed within which the Callaway Energy Center Site is situated since the time of license issuance have the potential to influence potential flooding hazards. A description of the watershed at the time of license issuance and pertinent changes to the watershed since license issuance are presented in the following sections.

2.5.1 Description of Watershed and Local Area at the Time of License Issuance

The Callaway Energy Center Site is situated on a plateau approximately five miles north of the Missouri River at a site grade elevation of 840 ft within the Missouri River watershed. The Missouri River flows in an east-west valley at an elevation of approximately 328 ft below the Callaway Energy Center Site (Ameren Missouri, 2012a). The Probable Maximum Flood (PMF) on the Missouri River, near the Site, is estimated to be 548 ft, based on an estimated PMF peak discharge of 1,300,000 cfs. The PMF was estimated based on data provided by flood studies of the Missouri River by the United States Army Corps of Engineers (USACE). With the estimated PMF of 548 ft for the Missouri River, the Callaway Energy Center Site will not be affected due to the large difference in elevation between the Missouri River and the Site grade of 840 ft (Ameren Missouri, 2012a).

Natural surface runoff around the Site flows easterly into the Logan Creek drainage basin, and southwesterly into the watershed area of Mud Creek. Logan Creek drains approximately 16.7 square miles, and is located within two miles of the Site, flowing into the Missouri River. Mud Creek is located about one and a half miles south of the Site; eventually flowing to its confluence with Logan Creek. Cow Creek is located to the north of the Site and flows west to its confluence with Auxvasse Creek. Auxvasse Creek then flows into the Missouri River (Ameren Missouri, 2012a).

2.5.2 Description of any Changes to the Watershed and Local Area since License Issuance

Changes to the watershed since the time of license issuance, mostly in the form of changes in land use and land development, have occurred. All flooding analyses described in ***Section 3.0*** have been undertaken with consideration and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. For

each delineated watershed, detailed information was considered regarding the current land use and soil types within each subbasin. As such, any such changes to the watershed and local area, as represented by the existing conditions, have been considered within these analyses.

2.6 CURRENT LICENSING BASIS FLOOD PROTECTION

Section 2.6 has been prepared in response to Requested Information Item 1.a.v. of NRC Recommendation 2.1 (NRC, 2012a). The safety-related structures, systems, and components at the Callaway Energy Center Site are all located approximately 290 ft above the highest expected flood level of the Missouri River. Therefore, flood protection for the safety-related structures, systems, and components for floods from the Missouri River is not necessary (Ameren Missouri, 2012a). The current licensing basis PMF water level near the Site is estimated at 548 ft, which is well below the Site grade elevation of 840 ft. The UHS retention pond is surrounded by a sloped grading to keep storm surface water from entering the pond. To prevent the water surface in the pond from exceeding the outlet crest elevation of 836.5 ft, an outlet structure and spillway are provided to drain excess storage (Ameren Missouri, 2012a). The Site is located on the crest of a plateau which has a natural drainage system and the Site drainage facilities of the Site direct runoff into the natural drainage system. Ice accumulation on the roofs of the safety-related structures has been taken into consideration for the Site drainage system (Ameren Missouri, 2012a).

2.7 ADDITIONAL SITE DETAILS

Section 2.7 has been prepared in response to Requested Information Item 1.a.vi. of NRC Recommendation 2.1 (NRC, 2012a).

2.7.1 Recommendation 2.3 Walkdown Results

The Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b) gives results of the walkdown, including key findings and identified degraded, non-conforming, or unanalyzed conditions, and includes detailed descriptions of the actions taken or planned to address these conditions. The Post Fukushima Flooding Walkdown Report for Callaway Energy Center states that, "(t)here are no other planned or newly installed flood protection systems or flood mitigation measures to further enhance the external flood protection at Callaway site" (Ameren Missouri, 2012b).

2.7.2 Site-Specific Visit Results

The RIZZO representatives conducted a site visit from 24-26 September 2012. RIZZO personnel met with WEC and Callaway Energy Center representatives to discuss the overall scope of work for the Flood Hazard Reevaluation. RIZZO personnel also inspected the cooling water intake structure on the Missouri River. Additionally, photographs were taken of the site and site area which were later used to aid in developing the appropriate inputs to the hydrologic models such as Manning's n roughness coefficients.

3.0 FLOODING HAZARD REEVALUATION ANALYSIS

Section 3.0 has been prepared in response to Requested Information Item 1.b. of NRC Recommendation 2.1 (NRC, 2012a).

3.1 SUMMARY OF RECOMMENDATION 2.1

To respond to Phase 1 of NRC Recommendation 2.1 (NRC, 2012a) and the 2012 Appropriations Act, the NRC requested that each licensee provide a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the Site, PMF on streams and rivers, storm surges, seiches, tsunamis, and dam failures. Natural events may reduce or limit the available safety-related cooling water supply, and the reevaluation should verify that an adequate water supply exists to shut down the Plant under conditions requiring safety-related cooling. A hazard evaluation should be performed for each reactor licensed under 10 CFR Part 50, including the spent fuel pool and the various modes of reactor operation. The reevaluation should apply present-day regulatory guidance and methodologies being used for Early Site Permit (ESP) and Combined Operating License (COL) reviews. The reevaluation should employ current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The work presented in this Flood Hazard Reevaluation Report will ultimately become part of the overall response submittal to NRC Recommendation 2.1.

3.2 FLOOD CAUSING MECHANISMS

The NRC NUREG/CR-7046 (NRC, 2011) recommends using a Hierarchical Hazard Assessment (HHA) method for evaluating the safety of SSCs. The HHA method is a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of the Site with the most conservative plausible assumptions consistent with available data. The HHA process proceeds as follows (NRC, 2012a):

a) Select one flood causing mechanism to be reanalyzed.

b) Develop a conservative estimate of the site-related parameters using simplifying assumptions for a flood causing mechanism and perform the reevaluation.

c) Determine if the reevaluated flood hazard elevation (from Step b) is higher than the original design flood elevation for the selected flood causing mechanism. If not, use this flood elevation for this causal mechanism for comparison of reevaluation against the current design basis.

d) Determine if the site-related parameters can be further refined. If yes, perform reevaluation (repeat Step c). If no, use this flood elevation for this causal mechanism for comparison of reevaluation against the current design basis.

e) Determine if all flood causing mechanisms have been addressed. If yes, continue to the following. If no, select another flood causing mechanism (Step a).

For each flood causing mechanism, compare the final flood elevations from the hazard reevaluation against the current design basis flood elevations. Using this comparison, determine whether the design basis flood bounds each reevaluated hazard. If it is determined that the current design basis flood bounds all of the reevaluated hazards, the reevaluation report is submitted. If not, additional analysis may be necessary. Any potential additional analysis will be conducted in the Integrated Assessment (NRC, 2012a).

Each potential flood causing mechanism has been evaluated based on present-day methodologies and regulatory guidance. Details regarding the considerations and outcome of the analyses regarding each flood causing mechanism are presented herein.

3.2.1 Local Intense Precipitation

Section 3.2.1 addresses the effects of Probable Maximum Precipitation (PMP) on the local area of the Callaway Energy Center Site. The HHA diagram for local intense precipitation flooding can be seen on ***Figure 3-1***.

3.2.1.1 Site-Specific Probable Maximum Precipitation

The local intense PMP was evaluated to determine the Probable Maximum Storm (PMS) event capable of generating the maximum amount of direct runoff (or peak discharge) at the Callaway Energy Center Site. The local intense PMP for the site drainage basin was calculated using the Hydrometeorological Report No. 52 (HMR 52) (National Weather Service [NWS], 1982). The Callaway Energy Center Site drainage basin boundary utilized for the calculation of the local intense PMP encloses an area of 41.9 acres or 0.07 mi^2. The peak one-hour storm event acting on an area of 1 mi^2 (1-hour, 1-mi^2) was determined for utilization as input to the reevaluation of the Callaway Energy Center local site drainage HEC-HMS model. The PMP depth generated by the 1-hour 1-mi^2 storm event acting on the Callaway Energy Center Site was determined using the isohyet chart presented on Figure 24 of the HMR 52 report (NWS, 1982).

The FSAR for the Callaway Energy Center Unit 1 states that an all-season six-hour rainfall with an accumulation of 25.4 inches (in.) is the governing PMP event affecting the surface runoff aspects of the SSCs, and gives a 48 hour PMP of 35 in. (Ameren Missouri, 2012a). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-3 and 3-4***, respectively.

The calculated results of the flooding reevaluation pertaining to Callaway Energy Center utilizing HMR 52 Figure 24 give a 1-hour 1-mi^2 PMP depth of 18.3 in. Local intense precipitation hyetographs are provided on ***Figure 3-2***.

3.2.1.2 Effects of Local Intense Precipitation

The local PMF at the Callaway Energy Center Site was calculated per the HHA method (NRC, 2011) by first using the Rational Runoff Transformation Method (see description below) and second using more site-specific data in the HEC-HMS software. The site drainage basin model used to calculate the local PMF divides the total power plant site area into 51 subbasins (see ***Figure 3-3***). The area of each of the 51 subbasins is listed in ***Table 3-1***.

As per the HHA method for local intense precipitation analysis (***Figure 3-1***), six scenarios were simulated with decreasing levels of conservatism. The six scenarios are as follows:

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

The Rational Runoff Transformation Method is appropriate for small basins, especially in areas that are mostly paved. The Rational Runoff Transformation Method assumes that the maximum rate of runoff from a drainage basin occurs when all parts of the watershed contribute and that the rainfall is uniformly distributed over the catchment area. No runoff losses were assumed and the runoff coefficient was assumed to be 1.0. The 1-hour 1-mi^2 reevaluated PMP described in ***Section 3.2.1.1*** was converted to rainfall intensity with consideration of the rainfall depth and duration. The peak runoff for each subbasin was calculated based on the rainfall intensity and subbasin area, as well as the runoff coefficient. The calculated rainfall intensity and peak runoff for each subbasin is listed in ***Table 3-1***.

In accordance with the approach of the HHA method (***Figure 3-1***), a more refined and more detailed evaluation of runoff at the Callaway Energy Center Site was performed using HEC-HMS 3.5 (USACE, 2010). ***Figure 3-3*** shows the HEC-HMS model watershed routing layout for the 51 subbasins.

The plant footprint area was assumed to be impervious, which implies that the Callaway Energy Center Site is assumed to have no losses (according to the HHA method). This assumption is per NUREG/CR-7046 guidance (NRC, 2011). Time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed, was used to calculate the lag time (Natural Resources Conservation Service [NRCS], 2010). The HEC-HMS model utilizes the reevaluated PMP discussed above.

A summary of site area drainage details, including peak runoff, calculated for both the Rational Runoff Transformation Method and using the HEC-HMS model are shown in ***Table 3-1***. Utilization of the HEC-HMS model, which incorporates more site-specific data, yields less conservative results than the Rational Runoff Transformation Method. The determination of water levels due to local intense precipitation at the Callaway Energy Center Site is discussed in ***Section 3.2.1.3***.

3.2.1.3 Water Level due to Local Intense Precipitation

Following the standards of the HHA method, six scenarios were considered with each less conservative than the next. In Scenario 1, peak runoff rates estimated in HEC-HMS were used, assuming no rainfall losses and no runoff transformation. It was assumed that conveyance structures and opening along the Vehicle Barrier System (VBS) were blocked. The VBS was modeled as 42 in. high and to have 36 in. openings at specified locations along its length.

Scenario 2 is like Scenario 1, except that the VBS openings in areas of potential inundation were represented as open. Scenario 3 is like Scenario 2 except that the Snyder Runoff Transformation Method was used to reduce peak runoff rates, and the coefficient of peak runoff was set to a value of 0.7. Scenario 4 is like Scenario 3, except that peak runoff rates, estimated by HEC-HMS, were calculated using a value of 0.5 for the coefficient of peak runoff. Scenario 5 is like Scenario 4, except that peak runoff rates, estimated by HEC-HMS, were calculated using a value of 0.4 for the coefficient of peak runoff. Scenario 6 is like Scenario 5, except that the shape coefficient used to calculate lag time was increased from 0.4 to 1.8. ***Figure 3-3*** shows the HEC-HMS model corresponding to Scenario 5, and shows the model subbasins which were used to represent the conditions of Scenario 5. ***Table 3-1*** lists the peak discharge rates for each subbasin calculated with Scenario 1 using the Rational Runoff Transformation Method and in the HEC-HMS simulations associated with Scenario 5.

For the first four scenarios, water levels at the Callaway Energy Center Site exceeded 840.5 ft, which is the critical flood elevation for the Site. For Scenario 5, simulated peak water levels do not exceed 840.5 ft. The sixth and least conservative assumption was included for additional information beyond Scenario 5. A peak water level of 840.17 ft North American Vertical Datum of 1988 (NAVD88) was simulated at the Callaway Energy Center Site for Scenario 5.

Figure 3-4 illustrates the simulated peak water levels and inundation area for each subbasin modeled using Scenario 5. A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-3 and 3-4***, respectively.

3.2.1.4 Wind Waves and Run-up Coincident with Local Intense Precipitation

The maximum water level due to wind-wave activity in the UHS retention pond coincident with local intense precipitation was evaluated. The wave setup and run-up generated by a two-year return period wind speed were added to the PMF still water elevation to determine the maximum water level at the UHS retention pond. The use of a two-year return period wind speed is consistent with the approaches presented in NUREG/CR-7046 (NRC, 2011). Methodologies used to determine the wind wave activity are based on the USACE Coastal Engineering Manual (CEM) (USACE, 2008).

As recommended in the CEM (USACE, 2008), the longest possible straight line fetch over the UHS retention pond of approximately 690 ft was used. The UHS pond is represented as

subbasin 58 (see ***Table 3-1***). Wave run-up was determined for the scenarios (Scenarios 1 through 6) utilized and described for the determination of the still water level due to local intense precipitation (***Section 3.2.1.3***). The maximum water surface elevations including run-up for Scenarios 1 through 6 are 839.31, 839.36, 838.74, 838.41, 838.04, and 837.84 ft, respectively.

The only area at the Callaway Energy Center Site where wind-wave activity has the potential to affect the SSCs is at the UHS retention pond. The water levels in the UHS retention pond after wave run-up do not reach the Site grade of 840 ft for Scenarios 1 through 6. Thus, water is contained in the UHS retention pond, and the Callaway Energy Center SSCs including those in the power block area will not be flooded by wind-wave activity coincident with local intense precipitation on the UHS retention pond. Ponded areas in the vicinity of the power block are shown on ***Figure 3-4***. Fetch lengths over ponded areas near the power block are small and so run-up on SSCs besides the UHS retention pond does not apply.

This approach is consistent with the FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a), which reports the maximum water level including wave run-up in the UHS retention pond as 838.3 ft.

3.2.2 River Flooding

River flooding at the Callaway Energy Center Site is of interest with respect to the Auxvasse Creek watershed and is discussed in this section. The HHA diagram for river flooding analysis is provided on ***Figure 3-5***. Dam failure flooding is discussed in detail in ***Section 3.2.3***.

3.2.2.1 Probable Maximum Precipitation for River Flooding

The PMP was calculated for the Auxvasse Creek basin in accordance with the criteria specified in HMR 52 (NWS, 1982). Basin-average precipitation for the PMS was determined in accordance with the temporal and spatial storm patterns associated with the PMP estimates provided in Hydrometeorological Report No. 51 (HMR 51) (NWS, 1978). The current design basis determined the PMF for Auxvasse Creek by imposing the PMP as obtained from Hydrometeorological Report No. 33 (HMR 33) upon the catchment (Ameren Missouri, 2012a).

The boundaries of the Auxvasse Creek watershed were defined within the HMR 52 computer program based on the reevaluated watershed delineation. The Auxvasse Creek watershed can be

seen on ***Figure 3-6***. For the delineated watershed, detailed information was considered regarding the current land use and soil types within each subbasin.

The location of the Auxvasse Creek watershed was utilized, along with the HMR 51 PMP isohyet charts to determine generalized estimates of the all-season PMP for storm areas from 10 to 20,000 mi^2 with durations of 6, 12, 24, 48, and 72 hours. The storm center coordinates were conservatively placed at the basin centroid resulting in higher runoff. A time interval of five minutes was selected in estimating the rainfall distribution.

The critical storm area for Auxvasse Creek was found by HMR 52 software to be 300 mi^2 with a critical storm orientation of 150 degrees and a critical 72-hour storm PMP rainfall of 28.63 in. An estimation of the 72-hour PMP total is not presented in the FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a).

The FSAR for Callaway Energy Center Unit 1 gives a PMP of 25.4 in. in 6 hours on an area of 75 acres in the Callaway Energy Center vicinity (Ameren Missouri, 2012a). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-3 and 3-4***, respectively.

The PMS rainfall hyetograph developed for Auxvasse Creek was used as input when developing the Auxvasse Creek watershed HEC-HMS models to determine the maximum runoff. The watershed hyetograph is provided on ***Figure 3-7***.

3.2.2.2 Probable Maximum Flood for Auxvasse Creek Watershed

The Soil Conservation Service (SCS) Curve Number, Clark Unit Hydrograph, and Muskingum-Cunge methods were used to calculate the basin loss, transformation, and routing for Auxvasse Creek. The Clark Unit Hydrograph method was chosen for transform calculations. The Manning's n roughness coefficient for the main channels was conservatively taken as 0.033 (for a channel that is straight, full, no rifts or deep pools, but more stones and weeds). Transmission losses through the reaches were assumed to be zero for all scenarios, which is conservative because all runoff that reaches the channel will be routed through the channel instead of being absorbed. The developed land use within the Auxvasse Creek watershed was used as a basis for the percentage of impervious area. Because the Auxvasse Creek watershed is controlled, representative flow rates could not be used for calibration purposes. It is noted that the FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a) did not perform model calibration.

As per the HHA method for river flooding (***Figure 3-5***), several scenarios were simulated in HEC-HMS with decreasing levels of conservatism. Reaches and junctions for the Auxvasse Creek watershed can be seen on ***Figure 3-8***. The different scenarios considered in the HEC-HMS model for Auxvasse Creek are as follows:

- **Scenario 1**: No SCS Curve Number loss, no rainfall-to-runoff transformation, no routing through the channels.
- **Scenario 2**: No SCS Curve Number loss, Clark Unit Hydrograph transformation, no routing through the channels.
- **Scenario 3**: No SCS Curve Number loss, Clark Unit Hydrograph transformation, includes routing through the channels.
- **Scenario 4**: SCS Curve Number loss method, Clark Unit Hydrograph transformation, include routing through the channels – Existing conditions.
- **Scenario 5**: SCS Curve Number loss method, Clark Unit Hydrograph transformation, include routing through the channels, but with assumed 5 percent increase in percent impervious area for each subbasin – Projected conditions.

The reevaluated calculated flow values from Junction 20 and from Subbasin 57 near the Site (from Scenario 4 – Existing conditions) are approximately 262,124 cfs and 23,213 cfs, respectively. Combined effects are discussed separately in ***Section 3.2.9***.

The FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a) did not include a comparable PMF analysis. A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-3 and 3-4***, respectively.

The outflows from the different scenarios are utilized for the determination of the water level due to the PMF as described in ***Section 3.2.2.3***.

3.2.2.3 Water Level Due to Probable Maximum Flood

The potential for river flooding at the Callaway Energy Center Site due to the PMF was evaluated using a HEC-RAS model to determine water surface flood elevations during the PMF on the Auxvasse Creek watershed. The HEC-RAS model for the Auxvasse Creek watershed is shown on ***Figure 3-9***.

With regards to the construction of the HEC-RAS models, contraction coefficients of 0.1 and expansion coefficients of 0.3 were used for normal flowing cross sections. When an obstruction was encountered, such as a bridge, a contraction coefficient of 0.3 was used with an expansion coefficient of 0.5 for cross sections immediately upstream and downstream of the bridge. A Manning's n roughness coefficient of 0.035 was used for stream channel (used for natural stream, clean, straight, full stage, no rifts or deep pools, with stones and weeds). A Manning's n roughness coefficient of 0.1 was used for the overbanks (used for floodplains; brush; medium to dense brush). The FSAR for Callaway Energy Center Unit 1 did not model the Auxvasse Creek watershed (Ameren Missouri, 2012a). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-3 and 3-4***, respectively.

Flow was assumed to be steady and one-dimensional for this analysis and, hence, a steady state analysis was conducted, which is a conservative assumption. Since both supercritical and subcritical flows were anticipated, a mixed flow regime was assumed. The upstream boundary condition was set at normal depth, whereas the downstream boundary condition was set at critical depth for the Auxvasse Creek model and at known water surface elevation for the Logan Creek/Mud Creek model. The runoff rates resulting from the PMP events described in ***Section 3.2.2.1*** were used as input into the models.

There is only one USGS gauge located along the portion of Auxvasse Creek that is modeled in HEC-RAS. However, no USGS gauges are located along the portion of Logan Creek or Mud Creek modeled in HEC-RAS. Since there is no record of measured discharge along all of the streams modeled in HEC-RAS, the model is not calibrated.

The five different scenarios considered in the HEC-RAS model are as follows:

- **Scenario 1**: No SCS Curve Number loss, no rainfall-to-runoff transformation, no routing through the channels.

- **Scenario 2**: No SCS Curve Number loss, Clark Unit hydrograph transformation, no routing through the channels.
- **Scenario 3**: No SCS Curve Number loss, Clark Unit hydrograph transformation, includes routing through the channels.
- **Scenario 4**: SCS loss, Clark Unit hydrograph transformation, include routing through the channels – Existing conditions.
- **Scenario 5**: SCS loss, Clark Unit hydrograph transformation, include routing through the channels, but with assumed 5 percent increase in percent impervious area for each subbasin – Projected conditions.

Maximum predicted water surface elevations near the Site on Auxvasse Creek for Scenarios 1 through 5 are 772.71, 717.47, 709.77, 709.05, and 709.09 ft, respectively. Thus, it can be concluded that the maximum predicted water surface elevations for all scenarios will not exceed the site grade of 840 ft. The reevaluated PMF flood elevations are presented along with the current design basis flooding elevations in ***Table 3-5***. HEC-RAS predicted water surface profiles for Auxvasse Creek, Logan Creek, and Mud Creek in Scenario 4 are shown on ***Figure 3-9***. Flow from Cow Creek into Auxvasse Creek near the Site is accounted for in the HEC-HMS models.

3.2.2.4 Wind Waves and Run-up coincident with Probable Maximum Flood

The reevaluated flood levels due to the PMF as presented in ***Section 3.2.2.3*** indicate an elevation difference between water levels in streams in the Site vicinity and the Site grade elevation of greater than 100 ft. As such, wind waves and run-up coincident with the PMF will not affect the Site and no further analysis is necessary.

3.2.3 Dam Failure Flooding

The potential flooding of the Callaway Energy Center Site due to simultaneous failure of all dams on the Auxvasse Creek watershed was evaluated. The HHA diagram for dam failure flooding can be seen on ***Figure 3-10***. The locations of dams near the Site can be seen on ***Figure 3-11***.

The total combined maximum storage of the reservoirs is 16,148 acre-ft. Auxvasse Creek is located approximately three miles west of the Site and the discharge from all upstream dams would pass through this creek. The plant grade elevation is at 840 ft, and the flood elevation for SSCs is 840.5 ft. The potential flooding of the Site was considered in two parts. The first part

was used to address the potential for flood waters to reach the plant grade and second to ascertain the potential for flood elevations starting at the plant grade elevation of 840 ft to exceed 840.5 ft and flood the SSCs with the simultaneous failure of all upstream dams.

The most conservative assumption regarding dam failure is that all upstream inflows from the PMP event and all reservoirs upstream of the Site in Auxvasse Creek fail simultaneously. Flood waters resulting from a watershed-wide PMP event of 28.63 in. combined with the concurrent failure of all upstream dams would not reach an elevation 810 ft adjacent to the Site.

Alternatively, only that storage between the elevations of plant grade (840 ft) and the SSCs (840.5) should be considered for storage of all upstream reservoirs. Furthermore, the upstream watershed storage between the elevation of 840 and 840.5 ft is 3.6 times greater than the combined storage of all upstream reservoirs. This indicates that flood waters would not reach plant grade and even if flood waters reached plant grade, the combined storage of all upstream dams (many of which would already be under water) would not flood the Callaway Energy Center Site.

3.2.3.1 Wind Waves and Run-up Coincident with Dam Failure Flooding

The analysis regarding dam failure flooding as presented in ***Section 3.2.3*** determines that water levels in streams in the Site vicinity do not approach the Site grade elevation. As such wind waves and run-up coincident with dam failure will not affect the Site and no further analysis is necessary.

3.2.4 Storm Surge Flooding

The Callaway Energy Center Site is not near any large bodies of water for which storm surge flooding would apply; therefore, the risk to the plant from a storm surge event that could cause flooding at the Site is not expected to be a potential flooding hazard. According to ANSI/ANS-2.8-1992 guidance (ANSI/ANS, 1992), the region of occurrence of a hurricane shall be considered for United States coastline areas and areas within 100 to 200 miles bordering the Gulf of Mexico. Because the Callaway Energy Center is located greater than 620 miles inland from the Gulf of Mexico, and at a site grade of 840 ft, hurricanes do not present potential flooding hazards.

3.2.5 Seiche Flooding

The Callaway Energy Center Site is not near to any bodies of water for which seiche flooding would apply. Therefore, seiche flooding is not a risk to the Site.

3.2.6 Tsunami Flooding

The Callaway Energy Center Site is not near any large bodies of water for which tsunami flooding would apply. The Callaway Energy Center Site is over 620 miles from the Gulf of Mexico and the plant grade is approximately 840 ft above sea level. Therefore, tsunami flooding is not a risk to the Site. Furthermore, NRC (2008) indicates that if regional screening identifies that the site region is not subject to tsunamis, no further analysis for tsunami hazard is required.

3.2.7 Ice Flooding

The potential for flooding hazards due to ice jams, and ice formation and accumulation at the Callaway Energy Center Site was evaluated.

A review of the USACE Cold Regions Research & Engineering Laboratory Ice Jams Database (USACE, 2012) demonstrates that no new ice jam events have occurred since those events discussed in the FSAR for Callaway Energy Center Unit 2 (Ameren Missouri, 2009). According to the Callaway Energy Center Unit 2 FSAR, the closest ice jams to the Site were recorded at Boonville, Missouri on December 19, 1945. To date such events have not affected the operation of the Callaway Energy Center (Ameren Missouri, 2009).

The Callaway Energy Center is located on a plateau at an approximate elevation of 840 ft, which is the topographic high in the area, and surface runoff from the site vicinity drains into small intermittent streams located at lower elevations than the Callaway Energy Center Site. Streams close to the Site have small drainage areas and would not pose the potential of ice flooding at the Site (Ameren Missouri, 2009).

The historical air temperatures in the region of the Callaway Energy Center were also considered. Consistent with the FSAR for Unit 2 (Ameren Missouri, 2009), historical air temperatures recorded at the NWS observation station at the Columbia Regional Airport were examined (National Climatic Data Center [NCDC], 2012). There are no major terrain feature

differences between Columbia Regional Airport and the Callaway Energy Center Site (Ameren Missouri, 2009).

The maximum potential ice thickness that could form on the Missouri River and the UHS retention pond was estimated using historic daily air temperatures from the closest NWS meteorological station to the Site. Historical air temperatures from Columbia, Missouri for the recorded periods between 1970 through 2012 were downloaded from the NCDC (2012). According to USACE, 2004 methods, theoretical ice thickness can be estimated by analysis of Freezing Degree-Days (FDD), defined as the summation of the difference between 32 degrees Fahrenheit and average daily air temperature. For this analysis, the historic average daily air temperature data was used for the estimation of ice thickness.

The maximum calculated theoretical ice thickness for the Missouri River was estimated to be approximately 4.43 in. As discussed in the Callaway Energy Center Unit 1 FSAR (Ameren Missouri, 2012a), the water supply intake and water discharge structures on the Missouri River are not safety-related structures and are not required for safe shutdown of the plant. Ice or ice flooding will not be a problem at the discharge structure, as the warm discharge water will keep the outfall open.

The maximum calculated theoretical ice thickness on the UHS retention pond was estimated to be approximately 20.67 in. The invert elevation of the Essential Service Water System (ESWS) pumphouse and the invert of the discharge pipes are approximately 26 ft and 17.5 ft below the water surface of the UHS retention pond, respectively (Ameren Missouri, 2012a), which are approximately 24.28 ft and 15.78 ft below the maximum estimated ice sheet, respectively.

An evaluation of the potential for frazil ice blockage of the ESWS pumphouse was performed in 1996 (Daly, 1996). The analyses concluded that the probability of frazil ice accumulation blocking the Essential Service Water (ESW) pump intakes is quite low. This is due to the small size of the UHS retention pond, its location sheltered from the wind, formation of ice cover over the surface of the pond, and warm water inflow from the ESW return line to the UHS pond (Daly, 1996).

For the reasons discussed above, ice effects are not a potential threat to safety-related SSCs at the Callaway Energy Center.

3.2.8 Channel Diversion Flooding

Flooding hazards due to potential diversion or rerouting of the Missouri River and its associated tributaries located near the Callaway Energy Center Site were evaluated. Per NRC Regulatory Guide (RG) 1.206, the Site was evaluated with respect to seismic, topographical, and geologic evidence in the region (NRC, 2007). Also evaluated was the potential diversion or rerouting of the source of makeup water for the Callaway Energy Center due to channel diversions.

A qualitative assessment of the streams and rivers in the vicinity of the Callaway Energy Center Site was undertaken in conjunction with a review of information presented in the FSARs for Callaway Energy Center Unit 1 and Unit 2 (Ameren Missouri, 2012a; Ameren Missouri, 2009). The evaluation of the potential for channel diversions consisted of the following considerations:

- River and stream morphology and hydrological characteristics.
- Human induced development on the Missouri River.
- Topographical, seismological, and geological evidence for channel diversion.
- Potential for diversion or rerouting of the source of makeup water for the Site.

Each consideration is discussed separately in ***Sections 3.2.8.1*** through ***3.2.8.4***.

3.2.8.1 River and Stream Morphology and Hydrological Characteristics

The Missouri River, which is the principal source of makeup water for Callaway Energy Center Unit 1, flows over 2,300 miles in a generally east-southeast direction. From the confluence of the Gallatin, Madison, and Jefferson Rivers in Montana, the Missouri River flows through seven states before joining the Mississippi River near St. Louis, Missouri (Spooner, 2000). Today the Missouri River is divided into three subsections: the free-flowing upper section, impounded middle section, and channelized lower section (Spooner, 2000). The Callaway Energy Center Site is located approximately five miles north of the Missouri River near River Mile 115, which is located within the channelized lower section of the Missouri River (Ameren Missouri, 2009). Because the Callaway Energy Center is located on a plateau, a topographic high in the area, surface runoff drains into several intermittent streams near the Site (Ameren Missouri, 2009). The streams located nearest the Site are Logan, Mud, Cow, and Auxvasse creeks. Mud Creek is a tributary of Logan, while Cow Creek is a tributary of Auxvasse. Logan and Auxvasse Creeks are in turn tributaries of the Missouri River.

3.2.8.2 Human Induced Development on the Missouri River

Prior to the 1800's, the pre-regulated Missouri River was characterized by the migration of its channel meanders and channel avulsion, with both processes resulting in the reworking of its floodplain deposits (Spooner, 2000). Channel meanders tend to migrate due to lateral erosion and the associated deposition of sediments. Particularly on the lower Missouri River this process has led to the formation of features such as levees, terraces, abandoned channels, and oxbow lakes (Spooner, 2000). Prior to the development of large scale river control activity, this dynamic equilibrium of continuous bank erosion and deposition constantly reshaped the channel and floodplain of the Missouri River (USGS, 1998).

As settlement of the Missouri River basin increased in the early 1800s, significant efforts were made to control the Missouri River's unpredictable nature and adapt it for transportation, farming, and urban development purposes (USGS, 1998). The results of the various programs implemented since then have progressively altered the natural features of the Missouri River, transforming it into a navigation system that is regulated by reservoirs and by bank stabilization and flood control structures (Spooner, 2000). Specifically in the 1940s and 1950s, programs implemented by the USACE and the Bureau of Reclamation transformed the Missouri River into a system of main stem reservoirs and altered reaches influenced by regulated flows, self-channelization, and bank stabilization (USGS, 1998).

As a result of the extensive man-made alterations to the physical features of the Missouri River, 35 percent of the river is impounded, 32 percent is channelized, and the remaining 33 percent is unchannelized (Ameren Missouri, 2009). In the upper Missouri River, deep water reservoirs have replaced free-flowing currents. In the lower river, channelization has eliminated sandbars, extensive depth variations, and connections with side channels and backwaters (Ameren Missouri, 2009). Wing dikes and revetments have concentrated flow, and levees keep the channel in place and disconnect it from the floodplain, morphing the river into a deep single-thread channel (Ameren Missouri, 2009). As a result of man-made alteration, channel avulsion and migration have become a rare occurrence along the lower Missouri River (Spooner, 2000).

3.2.8.3 Topographical, Seismological, and Geological Evidence for Channel Diversion

The Callaway Energy Center Site lies within the Central Stable Region (***Figure 3-12***) of the United States near the border between the Dissected Till Plains and Ozark Plateaus physiographic provinces (Ameren Missouri, 2009). This region is tectonically and seismically

stable, experiencing only infrequent and minor earthquake activity, with the closest epicenter located 38 miles from the Callaway Energy Center Site (Ameren Missouri, 2009). Based on the lack of evidence of deformation zones, and capable faults, and with consideration of the location of the Site on an isolated plateau with deeply incised drainage patterns extending around the plateau and to the Missouri River valley, channel diversion due to seismic activity is not expected during the lifetime of the plant (Ameren Missouri, 2009).

The Missouri River valley between Glasgow, Missouri and its confluence with the Mississippi River near St. Louis has been characterized as a bedrock trench of Ordovician and Mississippian limestone and dolomite filled with up to 120 ft of silt and sand with intermixed gravel and boulders (Spooner, 2000). Downstream from Glasgow, including the Site Area, bedrock lithology along the river valley is dominated by more resistant limestone and dolomite (Spooner, 2000). This more erosion resistant bedrock lead to the development of a narrow river valley with steep sides, as opposed to the river valley upstream of Glasgow characterized by broad meanders and softer channel bedrock (Spooner, 2000). The narrow river valley with steep sides is less prone to a meandering channel behavior leading channel diversion or migration.

During the early to mid-Pleistocene or Pre-Illinoian stage, approximately 2.5 to 0.3 million years ago (National Atlas, 2012), glacial advance throughout North America had a profound effect upon the flow path of the Missouri River, for the most part establishing the river's present course. Evidence of channel diversion of the Missouri River during this time period has been found near Kansas City, Missouri. The Blue and Little Blue Rivers each became temporary diversion channels during glacial advance, and were abandoned during glacial retreat, allowing the Missouri River to reoccupy its pre-glacial channel (Spooner, 2000). Since glacial retreat, no evidence of channel diversion of this magnitude has been found. The Missouri River channel has since stabilized, and continues to follow its pre-glacial route.

3.2.8.4 Potential for Diversion or Rerouting of the Source of Makeup Water for the Site

The source of makeup water for the Callaway Energy Center is the makeup water intake structure located on the Missouri River approximately at Missouri River Mile 115. The intake structure is not a safety-related structure and is not required for safe shutdown of the Callaway Energy Center. No geologic or seismic hazards near the site have been identified, but the possibility of mass wasting in the Missouri River valley does exist. An event such as this would result in collapsed material deposited at the shoreline location of the event (Ameren Missouri, 2009). The result of this event would disperse sand, soil, and other materials over a large area.

This, coupled with dredging activities conducted by the USACE and the design of the makeup water intake structure, limits the possibility of a loss of makeup water supply. As discussed previously, the flow of the Missouri River is heavily regulated and controlled by the USACE, and has been progressively altered in the past. Due to this control, it is highly unlikely any made-made events or actions would result in the channel flooding or diversion of the Missouri River during the lifetime of the plant.

Based on the assessment presented in ***Sections 3.2.8.1*** through ***3.2.8.4***, it is concluded that flooding hazards due to potential channel diversions, and potential impacts on the source of cooling water due to channel diversions do not present a flooding threat to the Callaway Energy Center Site. This conclusion is consistent with the Callaway Energy Center Unit 1 and Unit 2 FSARs (Ameren Missouri, 2012a; Ameren Missouri, 2009).

3.2.9 Combined Events

The water level due to combined event precipitation flooding was determined, as recommended by Section 9.2.1.1 of ANSI/ANS-2.8-1992 (ANSI/ANS, 1992) and NUREG/CR-7046 guidance (NRC, 2011). The combined events analysis was based on precipitation flooding due to the inland location of the Callaway Energy Center Site. The aforementioned guidance recommends three different alternatives consisting of a combination of events. The Callaway Energy Center Site is located on a plateau at an elevation of 840 ft, which is approximately 325 ft above the flood plain of the Missouri River (Ameren Missouri, 2012a). Because the maximum or 100-year snowpack coincident with the PMF will still not flood the Site, the following combined events were evaluated step by step:

- Mean monthly (base) flow
- Median Soil Moisture
- Antecedent (or subsequent) rain: the lesser of 1) rainfall equal to 40 percent of the PMP, and 2) a 500-year rainfall
- PMP
- Waves induced by two-year wind speed applied along the critical direction

The FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a) did not perform a combined events analysis. The mean monthly base flow for Auxvasse Creek was obtained from USGS gauge 06927240. The highest of the average monthly flows were used as the base flow

for conservatism. The base flow measurements from the USGS gauge were adjusted to each subbasin of Auxvasse Creek based on the gauge area and subbasin area. The adjusted base flows were then utilized in the HEC-HMS model for Auxvasse Creek, which is discussed in ***Section 3.2.2.2***.

The 72-hour, 5-minute interval PMP hyetograph for Auxvasse Creek calculated using HMR 52 and discussed in ***Section 3.2.2.1*** was utilized. Antecedent rain equal to 40 percent of the PMP event for 72-hours followed by a PMP event for 72-hours (6-day, 5-minute incremental PMP) was used as input into the HEC-HMS models for Auxvasse Creek, along with the adjusted base flows. The HEC-HMS models were run for the following scenarios as per the HHA method for combined events flooding (***Figure 3-13***).

- **Scenario 1**: No SCS Curve Number loss, no rainfall-to-runoff transformation, no routing through the channels – Most conservative case. Coincides with Scenario 1 presented in ***Section 3.2.2.2***.
- **Scenario 2**: No SCS Curve Number loss, includes Clark Unit Hydrograph transformation, no routing through the channels. Coincides with Scenario 2 presented in ***Section 3.2.2.2***.
- **Scenario 3**: No SCS Curve Number loss, includes Clark Unit Hydrograph transformation, include routing through the channels. Coincides with Scenario 3 presented in ***Section 3.2.2.2***.
- **Scenario 4**: Includes loss, Clark Unit Hydrograph transformation and routing through the channels – Existing conditions. Coincides with Scenario 4 presented in ***Section 3.2.2.2***.
- **Scenario 5**: Includes loss, Clark Unit Hydrograph transformation and routing through the channels – Projected conditions with an assumed 5 percent increase in percent impervious area for each subbasin. Coincides with Scenario 5 presented in ***Section 3.2.2.2***.

The PMF values determined using the approach described above were input into the HEC-RAS models for Auxvasse Creek, with the other inputs and parameters (discussed in ***Section 3.2.2.3***) remaining the same. These scenarios also included the combined events as described earlier in this section.

The maximum predicted water surface elevations due to combined events flooding for Scenarios 1 through 5 in Auxvasse Creek, are 772.76, 717.56, 710.02, 709.86, and 709.92 ft, respectively. The FSAR for Callaway Energy Center Unit 1 (Ameren Missouri, 2012a) did not evaluate

flooding due to combined events. Thus, a comparison of the reevaluated results to the results of the FSAR for Callaway Energy Center Unit 1 is not possible.

3.2.9.1 Wind Waves and Run-up Coincident with Combined Events Flooding

The reevaluated flood levels due to combined events flooding as presented in ***Section 3.2.9***, results in an elevation difference between predicted water levels in streams in the Site vicinity and the Site grade elevation of greater than 100 ft. As such wind waves and run-up coincident with combined events will not affect the Site and no further analysis is necessary.

4.0 COMPARISON OF CURRENT AND REEVALUATED PREDICTED FLOOD LEVELS

Section 4.0 has been prepared in response to Requested Information Item 1.c. of NRC Recommendation 2.1 (NRC, 2012a).

4.1 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

The flood causing mechanisms evaluated under the current design basis are local intense precipitation, river flooding, including wave run-up at the Site and dam failure flooding, storm surge and seiche flooding, tsunami flooding, ice flooding, and channel diversion flooding (Ameren Missouri, 2012a). Flood causing mechanisms considered under the flooding reevaluation include local intense precipitation, river flooding, dam failure flooding, storm surge and seiche flooding, tsunami flooding, ice flooding, channel diversion flooding, and combined events flooding.

The primary difference between the current and reevaluated flood causing mechanisms are that water levels were not determined for combined events under the current licensing basis. Additionally, dam failure flooding under the current licensing basis reports the maximum water level only on the Missouri River and not on Auxvasse Creek near the Site (Ameren Missouri, 2012a). The potential for storm surge and seiche flooding, tsunami flooding, ice flooding, and channel diversion flooding is qualitatively dismissed under both the current design basis (Ameren Missouri, 2012a) and reevaluated analyses.

4.2 ASSESSMENT OF THE CURRENT DESIGN BASIS FLOOD ELEVATIONS TO THE REEVALUATED FLOOD ELEVATIONS

The current design basis, as presented in the FSAR for Callaway Energy Center Unit 1 indicates that all safety-related structures, except the UHS Retention Pond, are not subject to flooding, wave action, or wave run-up and do not require flood or wave protection (Ameren Missouri, 2012a). The UHS retention pond is the only safety-related structure that is subject to wind wave activity and wave run-up. The UHS retention pond has relatively short dimensions with riprap-covered side slopes, therefore, the wind wave activity does not represent a major concern. The wave run-up is also not a major concern due to the maximum wave run-up reaching an elevation of 838.3 ft, which is below the Site grade elevation. The Callaway Energy Center grade is at elevation 840 ft, while the peak water level is at 548 ft for the PMF on the Missouri River

(Ameren Missouri, 2012a). As such, design basis flooding on the Missouri River does not pose a risk to the Callaway Energy Center Site. A list of Callaway Energy Center licensed water levels and corresponding flooding mechanisms can be found in ***Table 2-4***.

All reevaluated flood levels for each individual flood causing mechanism are discussed in ***Section 3.0***. A comparison of all of the current design basis flooding elevations and each of the reevaluated flood elevations for each flood causing mechanism presented in Attachment 1 to NRC Recommendation 2.1 (NRC, 2012a) is provided in ***Table 3-5***.

Although the reevaluated flood levels for river flooding do not exceed the plant grade or that of the SSCs, they do, however, differ from the current design basis (***Table 3-5***). However, the reevaluated water level for river flooding is determined for Auxvasse Creek and is not directly comparable to the current design basis water level, which is reported on the Missouri River.

4.3 SUPPORTING DOCUMENTATION

RIZZO has prepared 18 separate calculation briefs in support of the flooding hazard reevaluation at Callaway Energy Center on which the reevaluated flood levels are based. Additionally, the Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b) provides further information regarding the design basis flood hazard levels, as well as flooding protection and mitigation features.

4.3.1 Technical Justification of the Flood Hazard Analysis

All flooding analyses described in this Report have been undertaken with consideration to and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The technical basis for, and key assumptions utilized in the determination of the reevaluated flooding levels for each flood causing mechanism, are discussed individually in ***Section 3.0***.

4.3.2 Technical Justification by the Walkdown Results

With respect to the implementation and conclusions of the flooding hazard reevaluation, there are a few primary results from the Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b) that have been taken into consideration.

"…(T)he walkdown visual inspection has verified that the Callaway Site permanent safety-related SSCs are acceptable, not degraded, and capable of performing their design function as credited in the current licensing basis" and "(t)he reviews determined that Callaway Site flood protection features would be capable of performing their intended flood protection function if subjected to a design basis flooding hazard" (Ameren Missouri, 2012b).

The Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b), also states that "(t)here is reasonable assurance that the flood protection features are available, functional, and capable of performing their specified functions as set forth in the current licensing basis." Also, "(t)he Callaway external flood protection features are effective and able to perform their intended flood protection function when subject to a design basis external flooding hazard" (Ameren Missouri, 2012b).

The reevaluated determination of runoff due to local intense PMP at the Callaway Energy Center Site, as described in ***Section 3.2.1***, assumed that all hydraulic structures were blocked, including (initially) the VBS openings. Thus, no allowance is given for water drained by these facilities which conservatively maximizes the estimation of runoff or peak discharges at the Site. However, when determining the water level due to local intense precipitation, the calculated peak discharges are routed through natural drainage paths at the Site.

4.4 CONCLUSIONS

The reevaluated water levels associated with the most refined analysis for each flood causing mechanism do not exceed the elevations of SSCs at the Callaway Energy Center Site. A comparison of all of the current design basis flooding elevations and each of the reevaluated flood elevations for each flood causing mechanism is provided in ***Table 3-5***. It has been determined that the current design basis flood levels do not bound the reevaluated hazards for river flooding. However, the reevaluated water level which is reported on Auxvasse Creek is not directly comparable to the current design basis which is reported on the Missouri River. It should be noted that some of the mechanisms considered in the reevaluation were not part of the original design basis and so direct comparisons may not be practicable in some cases (***Table 3-5***).

5.0 INTERIM EVALUATION AND ACTIONS

Section 5.0 has been prepared in response to Requested Information Item 1.d. of NRC Recommendation 2.1 (NRC, 2012a).

5.1 EVALUATION OF THE IMPACT OF THE REEVALUATED FLOOD LEVELS ON SSCS

The reevaluated flooding levels due to all potential flood causing mechanisms, as presented in ***Section 3.0***, do not exceed the elevations of SSCs at the Callaway Energy Center Site.

5.2 ACTIONS TAKEN TO ADDRESS HIGHER FLOODING HAZARDS

Requested Information Item 1.d. of NRC Recommendation 2.1 (NRC, 2012a) specifies that the flooding reevaluation contain an interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment, if necessary.

The Post Fukushima Flooding Walkdown Report for Callaway Energy Center (Ameren Missouri, 2012b) states that "…the walkdown visual inspection has verified that the Callaway Site permanent safety-related SSCs are acceptable, not degraded, and capable of performing their design function as credited in the current licensing basis." Furthermore, "(t)he reviews determined Callaway Site flood protection features would be capable of performing their intended flood protection function if subjected to a design basis flooding hazard" (Ameren Missouri, 2012b). Lastly, "(t)here are no other planned or newly installed flood protection systems or flood mitigation measures to further enhance the external flood protection at the Callaway Site."

6.0 ADDITIONAL ACTIONS

Section 6.0 has been prepared in response to Requested Information Item 1.e. of NRC Recommendation 2.1 (NRC, 2012a).

At this time, there are no additional actions beyond Requested Information Item 1.d. of NRC Recommendation 2.1 (NRC, 2012a) (***Section 5.0***) which have been taken or are planned to address flooding hazards at the Callaway Energy Center.

Per the guidance for performing the integrated assessment for external flooding (NRC, 2012b), addressees are requested to perform an integrated assessment if the current design basis flood hazard does not bound the reevaluated flood hazard for all mechanisms. Furthermore, "The integrated assessment will evaluate the total plant response to the flood hazard, considering multiple and diverse capabilities such as physical barriers, temporary protective measures, and operations procedures (NRC, 2012b)."

7.0 REFERENCES

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Nuclear Regulatory Commission (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," Washington DC, March 12, 2012.

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TABLES

TABLE 2-1
HISTORIC FLOODS NEAR THE SITE

RIVER/STREAM	DATE	GAUGING STATION	MAX WATER LEVEL (ft)	DISCHARGE (cfs)
Missouri River	July 29, 1993	06909000	602.5	755,000
Osage River	May 20, 1943	06926500	**571.73	216,000
*Gasconade River	January 6, 1897	*N/A	*N/A	120,000
Gasconade River	April 15, 1945	06933500	**685.34	101,000

LT-1

Notes:

* The 1897 flood occurred outside of the period of record, discharge was estimated near Jerome, Missouri.

** Water Levels taken from USGS Streamflow Measurements Data

References:

Ameren Missouri, 2012, "Final Safety Analysis Report (FSAR), Callaway Energy Center Unit 1," Revision OL-19a, 2012.

Ameren Missouri, 2009, "Final Safety Analysis Report (FSAR), Callaway Energy Center Unit 2," Revision 2, 2009.

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TABLE 2-2
LIST OF SAFETY-RELATED STRUCTURES, SYSTEMS, AND COMPONENTS AND THEIR ELEVATIONS

SSC	ELEVATION (ft)
Reactor Building	840.5
Control Building	840.5
Auxilliary Building	840.5
Diesel Generator Building	840.5
Fuel Building	840.5
Essential Service Water Pump House	840.5
Ultimate Heat Sink Retention Pond	840.0
Ultimate Heat Sink Cooling Tower	840.5
Refueling Water Storage Tank	*835.5
Essential Service Water System Pipes	**Variable

LT-2

Notes:

* Elevation of bottom slab is 835.5 ft.

** When not protected by concrete, all Essential Service Water System pipes are buried a minimum depth of 4.5 ft. All Essential Service Water System electrical duct banks are reinforced concrete structures which are buried a minimum depth of 3.5 ft.

Reference:

Ameren Missouri, 2012, "Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1," Revision OL-19a, 2012.

TABLE 2-3
DESIGN PARAMETERS IN THE LICENSE DOCUMENT

DESIGN PARAMETER	VALUE
Grade Level (SSCs except UHS, Refueling Water Storage Tank, and ESWS Pipes)	840.5 ft
Probable Maximum Precipitation	35 in. (48 hour PMP)
Maximum Water Level at the Site Including Wave Run-up	551 ft
Cooling Water Flow Rate	38,000 gpm
Elevation of Intake Structure	525 ft
Annual Mean Temperature	55.6 °F

LT-3

Reference:

Ameren Missouri, 2012, "Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1," Revision OL-19a, 2012.

TABLE 2-4
LICENSED WATER LEVELS DUE TO ALL FLOODING MECHANISMS

FLOODING MECHANISM	WATER LEVEL (ft)
Local Intense Precipitation	The PMP Calculations Document that Ponding Elevations will be less than 840.5
	838.3 Including Wave Run-up in the UHS retention pond
River Flooding	537 / 548 *
Dam Failure Flooding	551
Storm Surge and Seiche Flooding	N/A
Tsunami Flooding	N/A
Ice Flooding	N/A
Channel Diversion Flooding	N/A
Combined Events	N/A

LT-4

Note:

* The FSAR for Callaway Energy Center Unit 1 reports 537 ft as the Standard Project Flood (SPF) and 548 ft as the Probable Maximum Flood (PMF).

Reference:

Ameren Missouri, 2012, "Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1," Revision OL-19a, 2012.

TABLE 3-1
SITE DRAINAGE AREA DETAILS

DRAINAGE SUB BASIN	AREA (mi^2)	RAINFALL INTENSITY I (inch/hr)	PEAK RUNOFF Q (cfs) (RATIONAL METHOD)	PEAK RUNOFF Q (cfs) (HEC-HMS MODEL, SCENARIO 5*)
10	0.076	45.01	2196	1019
12	0.043	30.78	854	444
13	0.007	47.16	220	97
17	0.015	46.83	440	196
18	0.002	47.16	48	21
19	0.005	47.16	140	62
20	0.025	40.36	651	332
21	0.006	47.16	172	76
22	0.024	47.16	721	320
24	0.001	47.16	44	20
25	0.030	45.21	880	407
26	0.004	47.16	119	53
30	0.006	47.16	168	75
31	0.005	47.16	154	68
33	0.005	41.87	131	65
34	0.001	47.16	44	20
35	0.006	47.16	177	78
37	0.003	47.16	102	45
41	0.012	37.39	279	149
42	0.010	40.43	266	135
43	0.046	35.84	1059	544
47	0.004	47.16	126	56
48	0.013	44.98	368	171
49	0.004	47.16	134	59
50	0.002	47.16	52	23
51	0.002	47.16	48	21
52	0.001	47.16	16	7
53	0.003	47.16	84	37
54	0.002	47.16	48	21
55	0.001	47.16	42	19
56	0.007	45.25	216	100
58	0.009	47.16	266	118
1091	0.002	47.16	72	32
1092	0.003	47.16	89	40
1093	0.003	47.16	91	40
1094	0.029	36.65	679	355
1095	0.006	47.16	184	82

TABLE 3-1
SITE DRAINAGE AREA DETAILS
(CONTINUED)

DRAINAGE SUB BASIN	AREA (mi^2)	RAINFALL INTENSITY I (inch/hr)	PEAK RUNOFF Q (cfs) (RATIONAL METHOD)	PEAK RUNOFF Q (cfs) (HEC-HMS MODEL, SCENARIO 5*)
1098	0.008	46.47	239	107
1099	0.010	47.16	291	129
1101	0.001	47.16	43	19
1102	0.005	47.12	152	67
1112	0.022	41.39	580	288
1113	0.019	44.18	551	260
1114	0.041	40.06	1060	545
1130	0.004	47.16	109	48
1131	0.005	45.30	148	68
1132	0.001	47.16	39	17
1133	0.006	35.03	142	72
1134	0.002	47.16	68	30
1135	0.002	47.16	46	20
1136	0.003	45.53	99	46

LT-5

Note:

* Peak runoff estimated using Snyder Runoff Transformation Method and the peak runoff coefficient set to 0.4 and a shape factor of 0.4.

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TABLE 3-2
REEVALUATED FLOOD LEVELS

FLOODING MECHANISM	WATER LEVEL (ft)
Local Intense Precipitation	840.17 NAVD88 (Scenario 5)
	838.04 (Scenario 5 + Wave Run-up in the UHS retention pond)
River Flooding	709.05 (Scenario 4, Existing Conditions)
Dam Failure Flooding	N/A*
Storm Surge and Seiche Flooding	N/A
Tsunami Flooding	N/A
Ice Flooding	N/A
Channel Diversion Flooding	N/A
Combined Events Flooding	709.86 (Scenario 4, Existing Conditions)

LT-7

Note:

* The dam failure analysis determined that flood levels for the PMP and concurrent failure of all upstream dams would be more than 30 ft below plant grade. Also, it was determined that even if flood waters due to dam failure reached plant grade, the combined storage of all upstream dams would not flood the SSCs at the Site.

TABLE 3-3
COMPARISON OF CURRENT DESIGN BASIS AND FLOODING REEVALUATION ASSUMPTIONS

ASSUMPTIONS	REEVALUATED HAZARDS	CURRENT DESIGN BASIS
Probable Maximum Precipitation (PMP) Calculation	HMR 52	HMR 33
Station used for Wind Speed Analysis	Columbia, Missouri	Columbia, Missouri
PMF Estimation	Clark Unit Hydrograph Method	No PMF analysis
SCS Losses	Calculated in reevaluation	Not Modeled
Base Flow	None	Not Modeled
PMP	3 day PMP	2 day PMP
Routing	Muskingum Cunge 8-point Method	Not Modeled
Transform Method	Clark Unit Hydrograph	Not Modeled
Transformation Losses	None	Not Modeled

LT-10

Reference:

Ameren Missouri, 2012**,** "Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1," Revision OL-19a, 2012.

TABLE 3-4
COMPARISON OF CURRENT DESIGN BASIS AND FLOODING REEVALUATION ANALYTICAL INPUTS

<table>
<tr><th>ASSUMPTIONS</th><th>REEVALUATED HAZARDS</th><th>CURRENT DESIGN BASIS</th></tr>
<tr><td>6-hour PMP</td><td>27.64 inches</td><td>25.4 inches</td></tr>
<tr><td>Maximum Sustained Wind (MSW)
Return Period</td><td>MSW</td><td>MSW</td></tr>
<tr><td>2</td><td>40.41</td><td>42.92</td></tr>
<tr><td>10</td><td>52.43</td><td>54.47</td></tr>
<tr><td>50</td><td>61.01</td><td>59.77</td></tr>
<tr><td>100</td><td>64.23</td><td>69.51</td></tr>
<tr><td>Subbasin Delineation</td><td>63 subbasins</td><td>4 subbasins</td></tr>
<tr><td>Auxvasse Creek Watershed Area</td><td>347.2 square miles</td><td>371.7 square miles</td></tr>
<tr><td>72-hour PMP</td><td>28.63 inches</td><td>Not Presented</td></tr>
<tr><td>Mannings roughness coefficient for channel and overbanks in the Auxvasse Creek Watershed.</td><td>($n_{channel}$) 0.035, (n_{bank}) 0.1</td><td>Not Calculated for Auxvasse Creek</td></tr>
<tr><td>Peak outflow from PMF analysis in Auxvasse Creek Watershed</td><td>262,124.3 cfs</td><td>Not Calculated for Auxvasse Creek</td></tr>
<tr><td>Highest Average Monthly Base Flow</td><td>718.72 cfs</td><td>Not Calculated for Auxvasse Creek</td></tr>
<tr><td rowspan="3">Water surface elevations due to PMF</td><td>709.09 ft Auxvasse Creek</td><td rowspan="3">N/A, Modeled only on the Missouri River</td></tr>
<tr><td>675.02 ft Logan Creek</td></tr>
<tr><td>577.51 ft Mud Creek</td></tr>
</table>

LT-11

Reference:

Ameren Missouri, 2012, “Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1,” Revision OL-19a, 2012.

TABLE 3-5
COMPARISON OF CURRENT AND REEVALUATED FLOOD LEVELS

FLOODING MECHANISM	REEVALUATED WATER LEVEL (ft)	CURRENT LICENSING BASIS WATER LEVEL (ft)
Local Intense Precipitation	840.17 NAVD88 (Scenario 5)	The PMP Calculations Document that Ponding Elevations will be less than 840.5
	838.04 (Scenario 5 + Wave Run-up in the UHS retention pond)	838.3 Including Wave Run-up in the UHS retention pond
River Flooding	709.05 (Scenario 4, Existing Conditions)	537 / 548 *
Dam Failure Flooding	N/A**	551
Storm Surge and Seiche Flooding	N/A	N/A
Tsunami Flooding	N/A	N/A
Ice Flooding	N/A	N/A
Channel Diversion Flooding	N/A	N/A
Combined Events Flooding	709.86 (Scenario 4, Existing Conditions)	N/A

LT-8

Notes:

* The FSAR for Callaway Energy Center Unit 1 reports 537 ft as the Standard Project Flood (SPF) and 548 ft as the Probable Maximum Flood (PMF). The FSAR reports water levels on the Missouri River whereas the reevaluated water level is on Auxvasse Creek.

** The dam failure analysis determined that flood levels for the PMP and concurrent failure of all upstream dams would be more than 30 ft below plant grade. Also, it was determined that even if flood waters due to dam failure reached plant grade, the combined storage of all upstream dams would not flood the SSCs at the Site. The FSAR for Callaway Energy Center Unit 1 reports 551 ft as the water level on the Missouri River and does not present a value for Auxvasse Creek near the Site.

Reference:

Ameren Missouri, 2012, "Final Safety Analysis Report (FSAR) Callaway Energy Center Unit 1," Revision OL-19a, 2012.

FIGURES