This document is a redacted publicly available version. In September 2024, the NRC staff identified information within this document that may be Critical Electric Infrastructure Information (CEII) as defined by the Federal Energy Regulatory Commission (FERC). The original file is non-public, designated as CEII, in ADAMS ML24291A076.

NRC staff actions were taken in accordance with:

- The Memorandum of Understanding Between US NRC and Federal Energy Regulatory Commission (FERC) Regarding Treatment of Critical Energy/Electric Infrastructure Information found at: https://www.nrc.gov/reading-rm/doc-collections/memo-understanding/2024/index.html.
- The FERC definition of CEII found at: https://www.ferc.gov/ceii, and, https://www.ferc.gov/enforcement-legal/ceii/designation-incoming-dam-safety-documents.

NRC FORM 464 Part I
(12-2015)

U.S. NUCLEAR REGULATORY COMMISSION

RESPONSE TO FREEDOM OF INFORMATION ACT (FOIA) REQUEST

FOIA	RESPONSE NUMBER
2016-0451	1

RESPONSE TYPE ☐ INTERIM ☑ FINAL

REQUESTER: Paul Blanch

DATE: 08/17/2016

DESCRIPTION OF REQUESTED RECORDS:

ML15356A158

PART I. -- INFORMATION RELEASED

☐ Agency records subject to the request are already available in public ADAMS or on microfiche in the NRC Public Document Room.

☑ Agency records subject to the request are enclosed.

☐ Records subject to the request that contain information originated by or of interest to another Federal agency have been referred to that agency (see comments section) for a disclosure determination and direct response to you.

☐ We are continuing to process your request.

☑ See Comments.

PART I.A -- FEES

AMOUNT* $

*See Comments for details

☐ You will be billed by NRC for the amount listed.

☑ None. Minimum fee threshold not met.

☐ You will receive a refund for the amount listed.

☐ Fees waived.

PART I.B -- INFORMATION NOT LOCATED OR WITHHELD FROM DISCLOSURE

☐ We did not locate any agency records responsive to your request. *Note*: Agencies may treat three discrete categories of law enforcement and national security records as not subject to the FOIA ("exclusions"). 5 U.S.C. 552(c). This is a standard notification given to all requesters; it should not be taken to mean that any excluded records do, or do not, exist.

☑ We have withheld certain information pursuant to the FOIA exemptions described, and for the reasons stated, in Part II.

☐ Because this is an interim response to your request, you may not appeal at this time. We will notify you of your right to appeal any of the responses we have issued in response to your request when we issue our final determination.

☑ You may appeal this final determination within 30 calendar days of the date of this response by sending a letter or email to the FOIA Officer, at U.S. Nuclear Regulatory Commission, Washington, D.C. 20555-0001, or FOIA.Resource@nrc.gov. Please be sure to include on your letter or email that it is a "FOIA Appeal."

PART I.C COMMENTS (Use attached Comments continuation page if required)

SIGNATURE - FREEDOM OF INFORMATION ACT OFFICER

Stephanie A. Blaney Stephanie Blaney

NRC FORM 464 Part II
(12-2015)

U.S. NUCLEAR REGULATORY COMMISSION

RESPONSE TO FREEDOM OF INFORMATION ACT (FOIA) REQUEST

FOIA: 2016-0451

DATE: 08/17/2016

PART II.A -- APPLICABLE EXEMPTIONS

Records subject to the request are being withheld in their entirety or in part under the FOIA exemption(s) as indicated below (5 U.S.C. 552(b)).

- [ ] Exemption 1: The withheld information is properly classified pursuant to an Executive Order protecting national security information.
- [ ] Exemption 2: The withheld information relates solely to the internal personnel rules and practices of NRC.
- [ ] Exemption 3: The withheld information is specifically exempted from public disclosure by the statute indicated.
  - [ ] Sections 141-145 of the Atomic Energy Act, which prohibits the disclosure of Restricted Data or Formerly Restricted Data (42 U.S.C. 2161-2165).
  - [ ] Section 147 of the Atomic Energy Act, which prohibits the disclosure of Unclassified Safeguards Information (42 U.S.C. 2167).
  - [ ] 41 U.S.C. 4702(b), which prohibits the disclosure of contractor proposals, except when incorporated into the contract between the agency and the submitter of the proposal.
- [ ] Exemption 4: The withheld information is a trade secret or confidential commercial or financial information that is being withheld for the reason(s) indicated.
  - [ ] The information is considered to be proprietary because it concerns a licensee's or applicant's physical protection or material control and accounting program for special nuclear material pursuant to 10 CFR 2.390(d)(1).
  - [ ] The information is considered to be another type or confidential business (proprietary) information.
  - [ ] The information was submitted by a foreign source and received in confidence pursuant to 10 CFR 2.390(d)(2).
- [ ] Exemption 5: The withheld information consists of interagency or intraagency records that are normally privileged in civil litigation.
  - [ ] Deliberative process privilege.
  - [ ] Attorney work product privilege.
  - [ ] Attorney-client privilege.
- [ ] Exemption 6: The withheld information from a personnel, medical, or similar file, is exempted from public disclosure because its disclosure would result in a clearly unwarranted invasion of personal privacy.
- [x] Exemption 7: The withheld information consists of records compiled for law enforcement purposes and is being withheld for the reason(s) indicated.
  - [ ] (A) Disclosure could reasonably be expected to interfere with an open enforcement proceeding.
  - [ ] (C) Disclosure could reasonably be expected to constitute an unwarranted invasion of personal privacy.
  - [ ] (D) The information consists of names and other information the disclosure of which could reasonably be expected to reveal identities of confidential sources.
  - [ ] (E) Disclosure would reveal techniques and procedures for law enforcement investigations or prosecutions, or guidelines that could reasonably be expected to risk circumvention of the law.
  - [x] (F) Disclosure could reasonably be expected to endanger the life or physical safety of an individual.
  - [ ] Other

PART II.B -- DENYING OFFICIALS

In accordance with 10 CFR 9.25(g) and 9.25(h) of the U.S. Nuclear Regulatory Commission regulations, the official(s) listed below have made the determination to withhold certain information responsive to your request.

DENYING OFFICIAL	TITLE/OFFICE	RECORDS DENIED	APPELLATE OFFICIAL EDO	APPELLATE OFFICIAL SECY
Stephanie Blaney	FOIA Officer (Acting)	Security related information	✓	

Appeals must be made in writing within 30 calendar days of the date of this response by sending a letter or email to the FOIA Officer, at U.S. Nuclear Regulatory Commission, Washington, D.C. 20555-0001, or FOIA.Resource@nrc.gov. Please be sure to include on your letter or email that it is a "FOIA Appeal."

Mr. Scott Batson
Site Vice President, Oconee Nuclear Station
Duke Energy Carolinas, LLC
7800 Rochester Highway
Seneca, SC 29672-0752

SUBJECT: OCONEE NUCLEAR STATION, UNITS 1, 2, AND 3– STAFF ASSESSMENT OF RESPONSE TO REQUEST FOR INFORMATION PURSUANT TO 10 CFR 50.54(f) FLOOD-CAUSING MECHANISMS REEVALUATION (CAC NOS. MF1012, MF1013, AND MF1014) AND PATH FORWARD ON CONFIRMATORY ACTION LETTER

Dear Mr. Batson:

The purpose of this letter is to provide the results of the staff's review of the Flood Hazard Reevaluation Report and to communicate the NRC's approach to address the regulatory matters associated with the Confirmatory Action Letter related to flooding at Oconee Nuclear Station.

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 12, 2013 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML13079A227), Duke Carolinas, LLC (Duke, the licensee) submitted its flood hazard reevaluation report (FHRR) for Oconee Nuclear Station, Units 1, 2 and 3 (Oconee). Duke also provided supplemental information resulting from requests for additional information and audits. The supplemental information included the licensee's revised FHRR for Oconee and was submitted on March 6, 2015 (ADAMS Accession No. ML15072A099).

By letter dated September 24, 2015 (ADAMS Accession No. ML15239B261), the NRC staff transmitted to Duke a summary of the staff's review of the licensee's reevaluated flood-causing mechanisms. The enclosed staff assessment provides the documentation supporting the NRC staff's conclusions summarized in that letter. As stated in that letter, the 2015 reevaluated flood hazard results for local intense precipitation; rivers and streams; and dam failure, were not bounded by the current plant design-basis analyses. In order to complete its response to Enclosure 2 to the 50.54(f) letter, the licensee is expected to submit an integrated assessment or a focused evaluation, as appropriate, to address these reevaluated flood hazards, as described in the NRC's September 1, 2015, letter (ADAMS Accession No. ML15174A257). The

~~The~~ Enclosures transmitted herewith contain Security-Related Information. When ~~separated from the Enclosures, this document is decontrolled.~~

NRC staff has determined that Duke has appropriately completed steps 1 through 6 of the 10-step process described in Enclosure 2 of the 50.54(f) letter. This closes out the NRC's efforts associated with CAC Nos. MF1012, MF1013, and MF1014. New CAC numbers will be assigned for future NRC review of the new information submitted to address the revised flood hazards.

Flooding hazards at the Oconee site have been the subject of significant interactions between the NRC and Duke prior to the 10 CFR 50.54(f) letter issued in response to the Fukushima Dai-ichi accident. Enclosure 2 of this letter provides a summary comparison of Duke's 2015 flood hazards evaluation and Duke's prior flood hazards evaluation docketed in 2010, relative to a postulated Jocassee dam failure. The NRC staff has determined that the 2010 licensee evaluation reflects a bounding flood hazard analysis for Oconee based on conservative assumptions. The 2015 evaluation reflects a reasonable analysis that removes some conservatism from the 2010 analysis, and is consistent with recent Commission direction regarding licensees' flood hazard reevaluation in response to the 50.54(f) letter. Therefore, the NRC staff concluded that the licensee's estimated flood levels at the ONS are considered reasonable and satisfy the information requests for each letter. Further, the staff concludes that the revised 2015 FHRR provides an acceptable alternative flood hazard analysis for ONS for the purpose of meeting the terms of the June 22, 2010, NRC Confirmatory Action Letter (CAL). Following the completion of the associated flooding modifications by June 2016, the NRC will evaluate whether the terms of the CAL have been satisfied.

By letter dated January 8, 2016, Duke submitted supplemental information regarding the external flooding hazards for Oconee. In that letter, Duke indicated its intent to maintain in effect certain compensatory actions implemented in response to the CAL. The NRC views these actions as an important element in Duke's overall strategy to mitigate the risks associated with a potential failure of the Jocassee Dam. Also in that letter, you stated your intent to revise the Oconee Updated Final Safety Analysis Report to reflect the results of the revised FHRR and the related actions taken by Duke to resolve external flooding issues, in accordance with forthcoming NRC guidance. Going forward, the NRC staff will address ongoing external flooding issues for Oconee within the framework of the Fukushima Near-Term Task Force Recommendation 2.1 process, to ensure consistency in the staff's approach to addressing these issues for all plants.

If you have any questions, please contact Juan Uribe or Randy Hall at (301) 415-3809 and (301) 415-4032 respectively, or via e-mail at Juan.Uribe@nrc.gov and Randy.Hall@nrc.gov, respectively.

Sincerely,

Jack R. Davis, Director
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Anne Boland, Director
Division of Operating Reactor Licensing
Office of Nuclear Reactor Regulation

Docket Nos. 50-269, 50-270 and 50-287

Enclosures:
1. Staff Assessment of Flood Hazard Reevaluation Report
2. Comparison of 2010 and 2015 Jocassee Dam Failure Evaluations

cc w/o encl: Distribution via Listserv

If you have any questions, please contact Juan Uribe or Randy Hall at (301) 415-3809 and (301) 415-4032 respectively, or via e-mail at Juan.Uribe@nrc.gov and Randy.Hall@nrc.gov, respectively.

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ADAMS Accession Nos.: Pkg. ML15356A161; Letter ML15352A207 (PUBLIC) ; Enclosures 1&2 ML15356A158 (NON-PUBLIC) *via email

OFFICE	NRR/JLD/JHMB/PM*	NRR/JLD/LA	NRO/DSEA/RHM1/BC*	NRR/JLD/JHMB/BC	NRR/DRA/D
NAME	JUribe	SLent	CCook	MShams	JGiitter
DATE	3/24/16	01/7/16	03/30/16	/ /16	/ /16
OFFICE	NRR/DE/D	OGC (NLO)	NRR/DORL/D	NRR/JLD/D	
NAME	JLubinski		ABoland	JDavis	
DATE	/ /16	4/4/16	/ /16	/ /16	

~~OFFICIAL RECORD COPY~~

STAFF ASSESSMENT BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO FLOODING HAZARD REEVALUATION REPORT

NEAR-TERM TASK FORCE RECOMMENDATION 2.1

OCONEE NUCLEAR STATION, UNITS NOS. 1, 2, and 3

DOCKET NO. 50-269, 50-270, and 50-287

1.0 INTRODUCTION

By letter dated March 12, 2012 (NRC, 2012b), 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 (NTTF) Review of Insights from the Fukushima Dai-ichi Accident (NRC, 2011c)[1]. Recommendation 2.1 in that document recommended that the staff issue orders to all licensees to reevaluate seismic and flooding for their sites against current NRC requirements and guidance. Subsequent Staff Requirements Memoranda associated with Commission Papers SECY-11-0124 (NRC, 2011d) and SECY-11-0137 (NRC, 2011e), directed 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 each plant. By letter dated May 11, 2012, the NRC staff issued its prioritization of the FHRRs (NRC, 2012c).

If the reevaluated hazard for a flood-causing mechanism is not bounded by the plant's current design-basis (CDB) flood hazard, an additional assessment of plant response is necessary, as described in the 50.54(f) letter and COMSECY-15-0019, "Mitigating Strategies and Flooding Hazard Reevaluation Action Plan" (NRC, 2015a, Enclosure 1). The information provided by the licensee and the results summarized in the September 24, 2015 Interim Hazard letter provide the flood hazard input necessary to complete this additional assessment consistent with the process outlined in COMSECY-15-0019 and the associated guidance that will be subsequently issued.

By letter dated March 12, 2013, Duke Carolinas, LLC (Duke, the licensee) provided a first version of its FHRR for Oconee Units 1, 2, and 3 (Duke, 2013a). In response to NRC staff's request for further documentation of a reference cited in its FHRR, the licensee provided additional documentation related to dam breach analysis methodology by letter dated April 29, 2013 (Duke, 2013b). Also, following the licensee's submittal of its FHRR for the ONS site, the

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

Enclosure 1

NRC staff issued RAIs, comprising 16 separate questions, by letter dated March 20, 2014 (NRC, 2014a), and by email dated September 15, 2014 (NRC, 2014c).

The licensee responded to the RAIs by letters dated April 25, 2014 (Duke, 2014a), and March 6, 2015 (Duke, 2015b). The March 6, 2015, submittal also included: (1) Enclosure 1, a revised FHRR (hereafter the "Revised FHRR"), dated January 29, 2015, and (2) Attachments 1 and 2 which contained supplemental information that addressed external flooding issues related to the 50.54 (f) letters issued by the NRC in 2008 (NRC, 2008) and 2012 (NRC, 2012b). The licensee identified interim actions in Sections 4 and 5 of its Revised FHRR (Duke, 2015b).

The Revised FHRR (Duke, 2015b) superseded the first version of the FHRR. The Revised FHRR contains a revised discussion of the dam breach methodology used for analyzing failure of Jocassee Dam, and a recently completed seismic analysis specific to the Jocassee Dam. The remainder of this staff assessment generally cites the Revised FHRR, except where discussing matters particular to the first version, or where referring to FHRRs in general, as for example in discussing regulatory requirements.

The reevaluated flood hazard results for local intense precipitation (LIP), rivers and streams, and dam failure flood-causing mechanisms are not bounded by the CDB hazard. Therefore, consistent with the process outlined in COMSECY-15-0019 and associated guidance, staff anticipates that the licensee will perform and document a focused evaluation for LIP and associated site drainage that assesses the impact of the LIP hazard on the site and evaluates and implements any necessary programmatic, procedural, or plant modifications to address this hazard exceedance. Additionally, for the rivers and streams and dam failure flood-causing mechanisms, the NRC staff anticipates that the licensee will submit either a focused evaluation or an integrated assessment, as outlined in COMSECY-15-0019 and associated guidance that will be issued.

On September 24, 2015, the NRC issued an Interim Staff Response (ISR) letter to the licensee (NRC, 2015d). An objective of the ISR letter is to provide flood hazard information suitable for the assessment of mitigating strategies developed in response to Order EA-12-049. The ISR letter also made reference to this staff assessment, which documents the NRC staff basis and conclusions. The flood hazard mechanism values presented in the ISR letter's enclosures match the values in this staff assessment without change or alteration. As mentioned in the ISR letter and discussed below, the licensee is expected to develop flood event duration parameters to conduct the Mitigating Strategies Assessment (MSA), as discussed in the latest revision to NEI-12-06, Appendix G (see COMSECY-15-0019 (NRC, 2015a)). The NRC staff plans to evaluate the flood event duration parameters (including warning time and period of inundation) during its review of the MSA.

The licensee submitted a separate flooding walkdown report in response to NTTF Recommendation 2.3 dated November 27, 2012 (Duke, 2012a). The NRC staff prepared a separate staff assessment to document its review of the licensee's flooding walkdown report dated June 30, 2014 (NRC, 2014b).

2.0 REGULATORY BACKGROUND

2.1 Applicable Regulatory Requirements

As stated above, 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 ESPs and COLs. This section describes present-day regulatory requirements that are applicable to the FHRR.

Sections 50.34 (a)(1), (a)(3), (a)(4), (b)(1), (b)(2), and (b)(4), of 10 CFR, describe the required content of the preliminary and final safety analysis report, 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 (NRC, 2012b) 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.

The Oconee Units 1, 2, and 3 are pre-General Design Criteria (GDC) facilities. For the purpose of the FHRR, the difference between pre-GDC and GDC are not material. As a result, the staff evaluated the analysis provided by the licensee against current GDC standards. GDC 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, tsunamis, 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 bases as the information which 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 analysis (based on calculation, experiments, or both) of the effects of a postulated accident for which a SSC must meet its functional goals.

Section 54.3 of 10 CFR defines the "current licensing basis" (CLB) 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 CLB.

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 flooding-causing mechanisms at each site. This includes current techniques, software, and methods used in present-day standard engineering practice.

2.2.1 Flood-Causing Mechanisms

Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) (NRC, 2012b) discusses flood-causing mechanisms for the licensee to address in the FHRR. Table 2.2-1 lists the flood-causing mechanisms that the licensee should consider. Table 2.2-1 also lists the corresponding Standard Review Plan (SRP) (NRC, 2007a) section(s) and applicable interim staff guidance (ISG) documents containing acceptance criteria and review procedures. The licensee should incorporate and report associated effects per Japan Lessons-Learned Directorate (JLD) JLD-ISG-2012-05 (NRC, 2012d) 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, 2012d) defines "flood height and associated effects" as the maximum stillwater surface elevation plus:

- Wind waves and runup effects
- Hydrodynamic loading, including debris
- Effects caused by sediment deposition and erosion
- Concurrent site conditions, including adverse weather conditions
- Groundwater ingress
- 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 Effect 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, Areas of Review (NRC, 2007a)). Attachment 1 of the 50.54(f) letter describes the

"Combined Effect Flood" as defined in American National Standards Institute/American Nuclear Society (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)), then the NRC 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, 2012d), 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 2.2-1 illustrates flood event duration.

2.2.5 Actions Following the FHRR

For the sites where the reevaluated flood hazard is not bounded by the CDB flood hazard for all flood-causing mechanisms, the 50.54(f) letter (NRC, 2012b) 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(s).
- Perform an Integrated Assessment subsequent to the FHRR to (a) evaluate the effectiveness of the current design basis (i.e., flood protection and mitigation systems); (b) identify plant-specific vulnerabilities; and (c) assess the effectiveness of existing or planned systems and procedures for protecting against and mitigating consequences of flooding for the flood event duration.

If the reevaluated flood hazard is bounded by the CDB flood hazard for all flood-causing mechanism at the site, licensees are not requested to perform an Integrated Assessment at this time.

COMSECY-15-0019 (NRC, 2015a) outlines a revised process for addressing cases in which the reevaluated flood hazard is not bounded by the plant's CDB. The revised process describes an approach in which the licensees with LIP hazards exceeding their current design-basis flood will not be requested to complete an integrated assessment. These licensees will instead assess the impact of the local intense precipitation hazard on their sites and then evaluate and implement any necessary programmatic, procedural, or plant modifications to address this hazard exceedance. In addition, for all mechanisms exceeding the CDB, licensees can assess the impact of the reevaluated hazard on their sites and confirm the capability of existing or proposed flood protection to address the hazard exceedance in lieu of performing an integrated assessment (NRC, 2015a). Sites with flooding hazards other than LIP exceeding the design-basis flood and where the exceedance could not be addressed through existing or proposed flood protection will proceed to performing an integrated assessment.

3.0 TECHNICAL EVALUATION

The NRC staff has reviewed the information provided for the flood hazard reevaluation of ONS, Units 1, 2, and 3. 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 NRC staff requested additional information on March 20, 2014 (NRC, 2014a), and September 15, 2014 (NRC, 2014c), from the licensee to supplement the ONS, Units 1, 2, and 3 FHRR. The licensee provided responses to the RAIs additional information by letters dated April 25, 2014 (Duke, 2014a), and March 6, 2015 (Duke, 2015b).

To provide additional information in support of the summaries and conclusions in the ONS, Units 1, 2, and 3 FHRR, the licensee made calculation packages available to the staff via an electronic reading room (ERR). When the staff relied directly on any of these calculation packages in its review, they or portions thereof, were either docketed and cited, or referenced as part of the ONS, Units 1, 2, and 3 FHRR Audit Summary Report (NRC, 2015c). Certain other calculation packages were found only to expand upon and clarify the information provided on the docket, and so are not docketed or cited.

3.1 Site Information

The 50.54(f) letter included the SSCs important to safety and the Ultimate Heat Sink 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 the Revised FHRR (Duke, 2015b). 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 ONS, Units 1, 2, and 3 Revised FHRR (Duke, 2015b) described the site specific information related to the flood hazard reevaluation. The following is a summary of the information provided in the FHRR. Table 3.1-1 provides the summary of controlling reevaluated flood-causing mechanisms, including associated effects that the licensee computed to be higher than the powerblock elevation.

The licensee stated in its ONS, Units 1, 2, and 3 Revised FHRR (Duke, 2015b) that all elevations referenced in the FHRR are based on the National Geodetic Vertical Datum of 1929 (NGVD29). The NGVD29 and "MSL" ("mean sea level") are equivalent vertical datums at the ONS site. Unless otherwise stated, all elevations in this staff assessment are given with respect to MSL.

The licensee provided site information for ONS in Section 1.1 of its Revised FHRR (Duke, 2015b). The nominal site grade (yard grade) for the ONS site is reported to be 796 ft (242.6 m) MSL as per the original design. The elevations at the mezzanine floor level for the Turbine Building, Auxiliary Building, and Service Buildings including the exterior access to these buildings is 796.5 ft (242.77 m) MSL. Principal buildings and nearby features of the ONS site are shown in Figure 3.1-1 of this report.

ONS is located in eastern Oconee County, South Carolina, approximately 8 mi (12.9 km) northeast of Seneca, South Carolina. Three reservoirs are located on the Keowee River near the ONS site, as shown in Figure 3.1-2. Jocassee Reservoir,[2] impounded by the Jocassee Dam and owned by Duke, is the farthest upstream, and is 11 mi (18 km) north of the ONS site. Keowee Reservoir, also owned by Duke, is downstream from Jocassee Reservoir, and adjoins the ONS site on the north and west sides. Hartwell Reservoir, owned and operated by the United States Army Corps of Engineers (USACE), is south of the ONS site and farthest downstream. Both Jocassee and Keowee reservoirs are licensed and regulated by the Federal Energy Regulatory Commission (FERC).

Keowee Reservoir was created in 1971. Its main features are shown in Figure 3.1-3 of this report. Keowee Reservoir has a full pond elevation of 800.0 ft (243.84 m) MSL, surface area of 17,660 acres (71.47 km^2), shoreline (including islands) of 387.9 mi (624.3 km), volume of 869,338 acre-ft (1,072,313,000 m^3), and a drainage area of 435 mi^2 (1,127 km^2) (Duke, 2014d, Table A1-2). The reservoir water is used to run the turbines of the hydroelectric power plant and provide cooling water for ONS.

Keowee Reservoir is divided into two parts: the Keowee Arm, and the Little River Arm. The Keowee Arm is the part north of the Connecting Canal, in the Keowee River watershed. The Little River Arm is the part south of the Connecting Canal, in the Little River Watershed. The two watersheds were originally separate, but were connected by excavation of the Connecting Canal. The Connecting Canal is about 2,000 ft (600 m) long and 100 ft (30 m) deep, and is located at the narrow part of Keowee Reservoir about 0.5 mi (0.8 km) north of the ONS site (Duke, 2014d, Section A2-2).

Keowee Reservoir is impounded by eight dams or dikes:

- Keowee Main Dam impounds the Keowee River. It is on the Keowee Arm of Keowee Reservoir, and is adjacent to the north side of the ONS site. It is sometimes referred to as Keowee Dam, but in some contexts, "Keowee Dam" is used collectively for the Keowee Main Dam and the West Saddle Dam.

- West Saddle Dam is on the Keowee River, immediately west of the Keowee Main Dam and adjacent to the north side of the ONS site.

- Intake Canal Dike, located at the east end of the Intake Canal, is adjacent to the ONS site on the south.

- Little River Dam is on the Little River. It is located on the east side of the Little River Arm of Keowee Reservoir about 4.5 mi (7.2 km) south of ONS.

[2] Also known as Lake Jocassee. Similarly, Keowee Reservoir is also known as Lake Keowee.

- Saddle Dikes A, B, C, and D are four small dikes located on the east side of the Little River Arm of Keowee Reservoir from 2.7 mi (4.3 km) to 3.5 mi (5.6 km) south of the ONS site.

The licensee reports that, with one exception, all man-made dams and dikes forming the Keowee Reservoir have a minimum crest elevation of 815 ft (248.4 m) MSL (Duke, 2015b, Section 1.1). The exception is two reinforced concrete trenches that extend through the Intake Canal Dike (Duke, 2015b, Section 1.3). These are discussed in Section 3.3 of this report.

Jocassee Reservoir was created in 1973. The reservoir has a full pond elevation of 1,110.0 ft (338.33 m) MSL, surface area of 7,980 acre (32.294 km^2), volume of 1,206,798 acre-ft (1,488,563,000 m^3), and a shoreline (including islands) of 92.4 mi (148.7 km) (Duke, 2014d, Table A1-1). The water from Jocassee Reservoir is used to provide pump storage capacity for the Jocassee Hydroelectric Station. Jocassee Reservoir is impounded by Jocassee Dam on the Keowee River.

3.1.2 Design-Basis Flood Hazards

During the ONS, Units 1, 2, and 3 FHRR audit (NRC, 2015c), the CDB for the ONS site was clarified by the licensee. The CDB presented during the audit supersedes information provided in the FHRR and RAI responses. The licensee stated that the ONS site was designed as a dry site. Thus, the site grade of 796 ft (242.6 m) MSL was used as the CDB level for most flooding hazards. The exception is the Keowee reservoir adjacent to the ONS site. The licensee stated that the design-basis flood level for Keowee Reservoir is 808.0 ft (246.28 m) MSL. The CDB flood levels are summarized by flood-causing mechanism in Table 3.1-2 of this report.

3.1.3 Flood-Related Changes to the Licensing Basis

The licensee reported changes to the licensing basis related to floods and flood protection in its Revised FHRR (Duke, 2015b, Section 1.3). The changes are:

Intake Canal Dike Trenches

The licensee stated that there is an exception to the statement that all man-made dams and dikes forming the Keowee Reservoir have a minimum crest elevation of 815 ft (248.4 m) MSL. The licensee described these exceptions as two reinforced concrete trenches extending through the Intake Canal Dike with a minimum crest elevation of 810 ft (246.9 m) MSL. These exceptions occur only when the covers are removed for maintenance. The licensee stated that these trenches are protected from wave action by the Condenser Cooling Water Intake Structure and the Causeway at the west end of the Condenser Cooling Water Intake Structure.

Mitigation of Site Flooding

The licensee conducted a study in 1983 (Duke, 2015b) to determine the impacts of flooding from a postulated sunny-day failure of the Jocassee Dam. The results of the study indicated that an estimated peak flood elevation of [redacted] MSL at Keowee Dam, and a resulting ONS powerblock flood depth of [redacted] In order to reduce the risk of flooding, the licensee erected walls around the entrances to the Standby Shutdown Facility

(SSF) with average wall height of [redacted] The construction of the walls was not part of the design-basis.

Because of concerns identified by the staff, the NRC requested information in a 10 CFR 50.54(f) letter dated August 15, 2008 (NRC, 2008). The licensee provided a response dated September 26, 2008 (Duke, 2008). As a result, ONS implemented a risk reduction measure by increasing the height of the flood wall around the SSF by (b)(7)(F), resulting in a wall elevation of (b)(7)(F) MSL (Duke, 2015b, Section 1.3).

The NRC staff issued a Confirmatory Action Letter (CAL) dated June 22, 2010 (NRC, 2010), requesting the licensee to address the following:

- Submit to the NRC all documentation necessary to demonstrate that the inundation of the ONS site, from the postulated sunny-day failure of Jocassee Dam, has been bounded;

- By November 30, 2010, submit a list of all modifications necessary to mitigate the inundation; and

- Make all necessary modifications by November 30, 2011. Subsequent correspondence with the NRC has since deferred these dates for completing of the modifications.

The NRC staff also requested that the compensatory measures listed in the CAL remain in place until they can be superseded by regulatory action related to the Fukushima responses. The licensee responded to the NRC's CAL by letter dated August 2, 2010 (Duke, 2010). The NRC staff prepared a staff assessment of the licensee's response, which was transmitted by letter dated January 28, 2011 (NRC, 2011a), and found that "the documentation provided sufficient justification that the parameters chosen by the licensee and the analysis performed bound the inundation of the ONS site resulting from a potential failure of the Jocassee Dam and therefore providing reasonable assurance for the overall flooding scenario at the site."

3.1.4 Changes to the Watershed and Local Area

The licensee has reported changes to the local site topography and support buildings that have occurred since the completion of the original construction discussed in Section 1.4 of the ONS Revised FHRR (Duke, 2015b). The changes that occurred in the area include housing and support facilities that were constructed since 1971 in the immediate vicinity of Keowee Reservoir. However, the licensee states that the overall land use has not changed significantly and that protected forest lands make up most of the Jocassee watershed.

3.1.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

The licensee provided a list of flood protection, mitigation, and early warning features as part of the description of the CLB flood protection and mitigation features in Section 1.5 of the Revised FHRR (Duke, 2015b). The licensee organized these features into four groups. The first group of flood protection and mitigation features is at yard elevation 796 ft (242.6 m) MSL and is intended to mitigate probable maximum precipitation (PMP) flooding. These include the Auxiliary Building exterior subsurface walls and seals; Radwaste trench covers and seals from Radwaste Facility to Turbine building; Radwaste trench covers and seals from Radwaste Facility to Auxiliary Building; Interim Radwaste trench covers and seals; Manhole 7 Cover;

Technical Support building vault and seals; Borated Water Storage Tank trench covers; yard drainage system; CT-5 trench covers in Auxiliary Building; and the PMP rainfall event flood barrier sandbags and Gryffolyn coverings.

The second group described in the Revised FHRR includes, Keowee River Dam; Intake Canal Dike; Little River Dam; Little River Dikes A, B, C, and D; Keowee Intake; Keowee Hydro Powerhouse; and Keowee Spillway are part of the Keowee Reservoir retention items.

The third group includes features used to mitigate flood conditions that were modeled based on a postulated sunny-day failure of the Jocassee Dam (NRC, 2011a). These features include plant flood protection procedures, early flood detection warning equipment, and other emergency equipment. Features in this group include the EM 5.3 Procedure (External Flood Procedure); AP/0/A/1 700/047 Procedure (External Flood Abnormal Procedure); Jocassee flood mitigating plans and procedures; Duke Hydro generation guidance document; Dam safety inspection program; Maintained monitoring program; Keowee spillway enhancements; Jocassee forebay and tailrace alarms; Jocassee storage building with backup spillway operating equipment; portable generator and electric drive motor near the spillway; documentation of table top exercises; instrumentation and alarm at selected seepage monitoring locations; video monitoring of Jocassee dam; second set of B.5.b-like equipment; and the Jocassee Dam-ONS response drill documentation.

The fourth group includes features used to provide exterior flood protection for the SSF and include SSF flood barriers including exterior walls; SSF steel plate with C02 refill access; SSF external wall penetrations; and Feature #35 which is the last feature on the flood walkdown list and corresponds to the site elevation and topography.

3.1.6 Additional Site Details to Assess the Flood Hazard

In addition to site details discussed in the previous section, the licensee also provided information in its response to requests for additional information related to the model input and output files that were used to evaluate flooding hazards from local intense precipitation, stream flooding, and dam breach induced flooding.

3.1.7 Plant Walkdown Activities

The 50.54(f) letter[3] 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[4] asked the licensee to report any relevant information from the results of the plant walkdown activities.

By letter dated November 27, 2012, Duke (Duke, 2012a) provided the flood walkdown report for ONS, Units 1, 2 and 3. The walkdown report was supplemented by letter(s), including RAI responses, dated January 30, 2014 (Duke, 2014b). The NRC staff prepared a staff assessment report, dated June 30, 2014 (NRC, 2014b), to document its review of the walkdown report. The NRC staff concluded that the licensee's implementation of the walkdown methodology met the intent of the walkdown guidance.

3.2 Local Intense Precipitation and Associated Site Drainage

The licensee reported in its Revised FHRR (Duke, 2015b), that the reevaluated flood hazard, including associated effects, for LIP and associated site drainage is based on a stillwater-surface elevation of 800.39 ft (243.959 m) MSL.

This flood-causing mechanism is discussed in the licensee's CDB, but no flood elevation was determined. The licensee stated that the ONS site is designed to be a flood dry site (NRC, 2015c). Thus, no CDB exists for LIP at the ONS site.

3.2.1 Modeling Approach for Local Intense Precipitation

The licensee reported that precipitation (rainfall) for the ONS site LIP flooding analysis was based on the values presented in the Updated Final Safety Analysis Report (UFSAR) (Duke, 2012b, Section 2.4.2.2) as a 48-h storm yielding 26.6 in (67.6 cm) of precipitation, with a maximum 1-h rainfall intensity of approximately 3.5 in/h (8.9 cm/h) (see Figure 3.2-1). The temporal rainfall distribution for this storm was based on a normalization of rainfall mass curves from an historical August 1940 precipitation event. Rainfall mass curves for this storm are shown in Figure 3.2-2.

The Revised FHRR (Duke, 2015b) states that for purposes of assessing runoff, the areas of interest are divided into two separate sections. The first section is the ONS "onsite" including all SSCs, and the second section includes delineated "offsite" drainage subbasins (see Figure 3.2-3(a)). The subbasins delineated in the offsite areas surrounding the plant (Figure 3.2-3(b)) and hydrologic characteristics of the site soils and terrain based on the Soil Conservation Service (SCS) Curve Number (CN) method are used to determine runoff rates for these subbasins (Duke, 2015b, Section 1.2.1).

The empirically based flows as determined by the SCS curve number method are used as input to a coupled 1-D (one-dimensional) and 2-D (two-dimensional) hydraulic model used to model 2-D onsite flows and depths and 1-D roof and underground drainage flows (Duke, 2015b,

[3] Enclosure 4, Requested Actions, Item 1 and Enclosure 4, Requested Information, Item 2.

[4] Enclosure 2, Requested Information, Items 1.a.vi and 1.c; Enclosure 2, Attachment 1, Steps 1 and 6; Enclosure 4, Requested Actions, Item 5; Enclosure 4, Requested Information, Items 1.c and 2; and Enclosure 4, Required Response, Item 2.

Section 1.2.1). The coupled model used for the analysis was InfoWorks CS, Version 11.5 (IWCS), produced by Innovyze (Innovyze, 2012).

The licensee developed a hydraulic model which is used for the area designated as the "2-D Simulation area" as shown in Figure 3.2-3(a). Using surface topography obtained from the 2010 survey data and updated building layout, the model is used to simulate surface water depths on the site resulting from the two-dimensional assessment of surface yard flows in conjunction with one-dimensional roof drainage-to-yard and roof-to-sub-ground drainage amounts that collect and exit the roof. For assessing roof and site drainage, the licensee analyzed three different scenarios:

- Scenario 1: Complete site system, assuming that roof and ONS Yard drainage are fully functioning.

- Scenario 2: Yard and roof drainage, assuming the yard drainage catch basins are blocked (i.e., not functioning).

- Scenario 3: Yard drainage (surface) only, assuming both the roof inlets and yard subsurface catch basins are blocked (i.e., not functioning).

Figure 3.2-4 illustrates the roof drains that are directly connected to the yard drainage system. Figure 3.2-5 depicts those connections for which roof drainage empties directly onto the ONS yard via downspouts and directly to yard drainage. The yard catch basins are mapped in Figure 3.2-6.

The maximum water surface elevation from the licensee's LIP calculated at critical locations in the yard to be 798.17 ft (243.28 m), based on analysis of Scenario 3.

3.2.2 Reevaluation of Effects of Local Intense Precipitation

The licensee reported in its Revised FHRR (Duke, 2015b), that the reevaluated LIP flooding is based on PMP values obtained using Hydrometeorological Reports Nos. 51 and 52 (HMR51 and HMR52) National Oceanic and Atmospheric Administration (NOAA) (NOAA 1978 and NOAA 1982) and a rainfall distribution based on guidance in NUREG/CR-7046 (NRC, 2011f). In its Revised FHRR, the licensee refers to this reevaluation as the "Beyond licensing/design basis case."

The licensee provided in the ERR a LIP flooding analysis (NRC, 2015c). The licensee also provided model input files as an enclosure to the FHRR (Duke, 2014a). The licensee's analysis uses a coupled 1-D and 2-D hydraulic model, InfoWorks CS, Version 11.5 (IWCS). This model allows the hydrology and hydraulics for pipe flow and overland flow to be modeled within the same software. The licensee used the following model parameters:

- A 72-hour probable maximum precipitation event;
- Site topography including grading, drainage divides, buildings, and other site drainage features; and
- Soil and surface characteristics.

The licensee's IWCS model included the switchyard to the east and the Spent Fuel Storage Installation to the south. The sub-basin containing the Keowee Powerhouse is modeled separate from the main yard. The model's computation mesh is comprised of triangles with a defined maximum and minimum size. The main Yard has a maximum mesh size of 250 ft^2 (23.2 m^2) and a minimum mesh size of 30 ft^2 (2.79 m^2).

The licensee assumed all site drainage system components are non-functional or completely blocked, per LIP Case 3 from NUREG/CR-7046 (NRC, 2011d). Overtopping flows from the roof onto the site are treated as flow over a weir. Drainage flows from sub-basins are defined at the boundaries shared by the site which is modeled as a 2-D feature. Rainfall volumes defined at the roofs of the SSCs are routed one-dimensionally to either underground drainage or onto the two-dimensionally modeled site drainage. The licensee performed 2-D modeling in the powerblock area since there was little topographic relief (i.e., relatively flat parking lot runoff surface) with little to no channeling resulting in unconfined flow. The NRC staff concurs that 2-D modeling is appropriate for simulating flood elevations in the area.

Using the IWCS model as described in their reevaluation, the licensee determined site depth-duration relationships for precipitation events of 5, 15, 30 min for 1-mi^2 (3-km^2) area and 1, 6, 12, 24, 48, and 72 h for 10 mi^2 (26 km^2) area. In addition, a 6-h LIP was developed in 5-minutes intervals for analysis and after reviewing the 6-h and 72-h results it was determined that the longer duration event could have a potentially higher flooding impact. Based on the determination, the licensee modified the 72-h event to include the peak intensity identified in the 6-h event. Using the IWCS model, the licensee evaluated the sensitivity of flood water surface elevations on the site to the temporal distribution of rainfall within the 72-h event.

The two storms that yielded the most conservative water surface elevations were 72-hour duration storms with (1) an end-loaded temporal distribution and (2) a temporal distribution centered at two-thirds. The licensee's sensitivity analysis identified the controlling event as a modified rainfall event of 46.6 in (118.4 cm) occurring over a 72-hour duration with a temporal distribution centered at two-thirds (Figure 3.2-7). The NRC staff verified the HMR 51 and HMR 52 computations and determined that the depths are appropriate.

The LIP flooding was modeled by the licensee using three different scenarios to evaluate the roof and ground drainage systems. The scenarios identify a combination of functioning and non-functioning roof and site drainage systems. The coupled 1-D and 2-D hydraulic model (IWCS) that was used to model the flow and depth of water for the main site included the Switchyard to the east and the Spent Fuel Storage Installation to the south (Duke, 2015b, Section 1.2.1). The subbasin containing the Keowee Hydro Powerhouse is modeled separately from the main yard.

3.2.3 Drainage and Local Watershed Delineation

The licensee reported that site topography was developed from surveyed data which was processed in ESRI ArcGIS™ software to create a triangulated irregular network (TIN) file. Aerial photographs were examined and any locations under construction during the time of the survey were updated to reflect current area topography. Watershed delineation was performed on the basis of topographic divides of each contributing runoff area.

The ONS yard drainage system exits into an open channel where termination points were developed based on physical characteristics of the conveyance, and boundary conditions were set to free discharge. Regarding the modeling considerations of the coupled drainage model as the flow drains from the 2-D grid to the sub-basins, the licensee stated that flow entering from offsite areas is generally from steep terrain transitioning to flat terrain at the intersection with the 2-D mesh that was used to model flow in the powerblock.

The licensee used the SCS method to determine flow from designated watersheds surrounding the main site (offsite). Curve numbers are assigned to subbasins, based on soil type and land use, following SCS guidance, and the TR-55 method (SCS, 1986) is used to calculate travel times for the subbasins, with a minimum value of 5 min for time of concentration.

In response to RAI-3, the licensee provided information related to watershed delineation and model specifics (Duke, 2014a). The licensee indicated that details are provided in the report "ONS Local Flooding Analysis Hydraulic Modeling Report, Yard and Roof Drainage Local Flooding, Current Licensing Basis" which was provided as an attachment to the RAI response dated April 25, 2014 (Duke, 2014a). The response indicated that watershed delineation was performed on the basis of topographic divides of each contributing runoff area. The SCS method was used to model the areas. Two-dimensional (2-D) modeling was performed in the ONS Yard (yard) since there was little topographic relief (i.e., relatively flat parking lot runoff surface) with little to no channeling, resulting in unconfined flow. Therefore, 2-D modeling was appropriate to simulate model flood elevations in this area.

3.2.4 IWCS Model Results

The licensee computed the water surface elevations, maximum flooding depths, and maximum velocity using the IWCS model. The maximum flood elevation of 800.4 ft (243.96 m) MSL occurs in the powerblock at the Unit 1 Reactor Building and Administrative Building. This is 3.89 ft (1.186 m) above the safety-related SSCs elevation of 796.5 ft (242.77 m) MSL. The 2-D modeling simulated results for velocity magnitudes in the yard are low around the buildings and equipment, ranging from 0.02 ft/s (0.006 m/s) to 2.9 ft/s (0.88 m/s), since the critical buildings and equipment are located toward the interior of the 2-D mesh and away from the boundary 1-D runoff areas. Exceptions to these low values are (1) the extreme north end of the yard bordering the site access roadway and the 1-D hillside and (2) the south end of the yard between the Interim Radwaste/Shredder buildings and the toe of the Oconee Intake Dike. Maximum velocities at these two sites are 6.04 ft/s (1.841 m/s) and 5.79 ft/s (1.765 m/s).

The IWCS coupled 1-D and 2-D hydraulic model was applied to determine depths for the three scenarios, as described in Section 3.2.1 of this staff assessment. Drainage flows from the subbasins are defined at the boundaries between the subbasins and the site, which is modeled as a two-dimensional feature. The scenario with the greatest flood depth is Scenario 3, in which all drains were assumed blocked. In this case, overtopping flows from the roof onto the site are treated as flow over a weir. The resulting flow is then applied to the two-dimensional grid. The model calculates flows and depth across the yard site for the duration of the LIP.

Rainfall volumes defined on the roofs of the SSCs are routed one-dimensionally by the model, either to underground drainage, or else onto the two-dimensionally modeled site surface drainage where water depths are determined. In response to RAI-2 (Duke, 2014a), the licensee stated that the IWCS model was calibrated against known rainfall events and established elevations. Five historic rainfall events were used (7/30/1991, 8/20/1995, 9/11/1995, 9/22/2003, and 06/28/2006). The licensee's response also included details of the IWCS mass balance, model configuration, and representation of the features. The SCS curve number methodology was used to model offsite hydrology and hydraulics and to route flow to specific locations. These locations are either a 1-D hydraulic node or a loading point on the 2-D mesh, as appropriate. The roofs were modeled using three categories based on the connections to the ONS Yard. Each onsite roof uses the Storm Water Management Model (SWMM) rainfall-runoff methodology (EPA, 2015) for transforming rainfall into runoff.

Regarding flow velocities and hydrodynamic forces at the yard, the licensee stated that since the critical buildings and equipment are located toward the interior of the 2-D mesh and away from the boundary 1-D runoff areas, velocities simulated by 2-D modeling in the yard are generally low around buildings and equipment. The NRC staff agrees with the licensee's representation and analysis of the results. Higher velocities are simulated, however, in the extreme north end of the yard bordering the site access roadway and hillside of the 1-D model and at the south end of the yard between the Interim Radwaste/Shredder buildings and the toe of the Intake Canal Dike. Debris and hydrodynamic loading impacts on SSCs in the yard were not evaluated in the Revised FHRR (Duke, 2015b).

3.2.5 Conclusion

The NRC staff confirmed the licensee's conclusion that the reevaluated flood hazard for LIP and associated site drainage is not bounded by the CDB flood hazard. Therefore, the NRC staff expects that the licensee will submit a focused evaluation for LIP and associated site drainage consistent with the process and guidance discussed in COMSECY-15-0019 (NRC, 2015a).

3.3 <u>Streams and Rivers</u>

The licensee reported in its revised FHRR (Duke, 2015b), that the reevaluated flood hazard, including associated effects, for streams and rivers is based on a stillwater-surface elevation of 808.9 ft (246.55 m) MSL and 812.2 ft (247.56 m) MSL with coincident wave runup. It is important to note that the PMF elevations are located within the Keowee Reservoir and are not referenced to the Oconee site. The Keowee dam structure and its appurtenant West Saddle Dam, with a crest located at elevation 815.0 ft (248.41 m) MSL, separates the Keowee Reservoir from the Oconee site located at 796.0 ft (242.62 m) MSL (see Figure 3.1-1). This flooding mechanism does not impact the Oconee site.

The licensee stated that the ONS site is designed to be a dry site (NRC, 2015c). Thus, no CDB exist for streams and rivers at the ONS site. A flood level value of 808.0 ft (246.28 m) MSL is referenced in the licensee's Revised FHRR (Duke, 2015b) and UFSAR (Duke, 2012b) and is listed as the flooding elevation in the Keowee reservoir based on a previous analysis (see Reservoir Overview and Background). This is, however, a maximum water surface elevation in Keowee Reservoir; the reservoir does not inundate the ONS powerblock area because the West Saddle Dam, with a crest elevation of 815 ft (248.4 m) MSL, separates the ONS site from the reservoir.

3.3.1 Flooding Overview and Background

The ONS site is situated on the Keowee Reservoir adjacent to the Keowee Dam. There are no significant rivers or streams in the vicinity of ONS site except for the Keowee River downstream of the Keowee Dam. Due to the location of the ONS site, the PMF (probable maximum flood) analysis focuses on flooding of reservoirs as opposed to rivers and streams. Figure 3.1-2 shows the locations of Jocassee and Keowee Reservoirs. The two most significant reservoirs upstream of the ONS site are the Keowee and the Jocassee. The licensee has stated that the design-basis flood level for the Keowee Reservoir is 808.0 ft MSL (NRC, 2015c). Figure 3.3-1 and Figure 3.3-2 show the Keowee and Jocassee watersheds and their subbasins.

The current design-basis flood elevation is based on analysis of the temporal distribution of historical storms (August 1940 for Keowee and October 1964 for Jocassee) for a rainfall of 26.6 in (67.6 cm) over 48 h. As reported in the Revised FHRR, the original studies were conducted to evaluate effects of PMP over the entire Lake Keowee drainage area (Duke, 2015b). The studies found that the spillway capacities at Keowee and Jocassee are effective in passing the design flood with adequate freeboard, thus posing no flooding concern for the ONS site. The maximum flood level at Keowee resulting from this flood is 808.0 ft (246.28 m) MSL. In addition, the flood discharge through the Keowee Reservoir is not expected to significantly affect the tailwater level downstream of Keowee Dam above approximately 686 ft (209.1 m) MSL.

The licensee reevaluated the flood hazard from the Keowee and Jocassee reservoirs using FERC guidelines (FERC, 1993) which are consistent with present-day methodologies and regulatory guidance for assessing PMF and its effect on hydroelectric projects by utilizing HMR51 and HMR52 for PMP development and Hydrologic Engineering Center (HEC)-1 (USACE, 1998) for hydrologic and hydraulic routing.

3.3.2 Keowee Reservoir Flooding Overview

The licensee's reevaluation analysis of the PMF flooding utilizes the analysis performed for the FERC Report for Keowee Flooding (NRC, 2015c). The licensee provided printed model input and output files in the ERR for staff's review in addition to the FHRR and documentation provided in the aforementioned Keowee Reservoir flooding study (NRC, 2015c).

The licensee's analysis for the PMF flooding re-evaluation at Keowee Reservoir determined the maximum water surface elevation in the Keowee reservoir resulting from the movement of a Probable Maximum Storm (PMS) through the Jocassee and Keowee watersheds in conjunction with hydrologic routing within the watersheds. The PMS size and orientation were developed with HMR-51 and 52 procedures. The response of the watershed was based on utilization of synthetic unit hydrographs developed as part of a U.S.Geological Survey (USGS) study (Bohman, 1989) that applied the SCS Curve Number (CN) methodology. Level pool routing of the PMF through Jocassee and Keowee reservoirs is performed with HEC-1. Verification of the model's response were performed and adjustments to hydrologic parameters were made to replicate the flooding events of August 1992 and 1994.

The NRC staff reviewed the available model related documentation in conjunction with the information provided in the report and supporting documentation in the ERR. The NRC staff also performed additional calculations to determine the level of conservatism incorporated in the licensee's model.

3.3.3 Keowee Watershed Hydrologic Assessment and PMF

The Keowee watershed encompasses approximately 435 mi^2 (1,126.7 km^2) and includes the watershed upstream of Jocassee dam. The hydrologic response of the watershed was performed with HEC-1 and was based on a combination of methods for stream and overland flow response and was verified with historical storms of August 22-23, 1992 and August 13-17, 1994. Unit hydrograph methodology derived by the USGS (Bohman, 1989) for the area was used to assess stream response with respect to flow and timing after adjustments were made to allow use of the unit hydrographs at specified durations. The methodology derived by USGS (Bohman, 1989) provides regional unit hydrographs by utilizing a parametrized equation for the lag time with respect to peak discharge and watershed area, and regional parameter constants. This in addition to CN adjustments, serves as the unit hydrograph calibration for particular geographic areas.

Staff concurs with the use of regional unit hydrograph calibration methodology produced by the USGS in cooperation with the South Carolina Department of Highways and Public Transportation and the Federal Highway Administration. The staff determined that the documented methodology is constructed from well-developed approaches and utilizes a thorough amount of data (over 151 storms events at about 49 stations).

Staff also determined that the SCS CN determination used by the licensee to identify the basin runoff is appropriate and accounts for effects of soil variability. As reported in the licensee's analysis, hydrologic soil group values were determined from the weighted average of 60 m^2 soil group associations. These were used to determine the appropriate CN for each of the five land use categories in each of the discretized cells in the basin. Averages of these values were used to identify the final average CN for a basin. The staff finds this approach to be a valid method to ensure that an appropriate CN is obtained.

The licensee presented and the staff reviewed validation of the regional unit hydrograph responses for the study which was performed by comparing two cases; within the sub basins containing the particular gage stations and within the entire drainage basin as a whole using observed data at Keowee and Jocassee. The staff recognizes the importance of selection of the particular rainfall events for validation. This is reflected in the details provided in the modeling documentation while assessing hydrograph performance for these two basin scales with the respective storm and runoff data available for each. In addition, the efforts outlined in a FERC analysis demonstrate a reasonable and informed approach to adjustments made to the CN and lag-time determination (for "direct" basins providing instantaneous flow to reservoirs) in the watershed to facilitate a more accurate response of the system with respect to matching observed flow at Keowee (NRC, 2015c). Sensitivity studies performed in the FERC analysis on CN number ranges (increasing by 10 and decreasing by 10) indicate that the flooding response is (b)(7)(F) [redacted] (b)(7)(F) [redacted] Manning's roughness coefficient *n* values are based on accepted published tables by Chow (Chow, 1959) and field visit visualization. Sensitivity studies by the licensee indicate that the PMF model is insensitive to Manning's roughness coefficient *n*. Sensitivity analysis of this parameter over the full range of tabulated values that could fit the field observations resulted in water level changes of no more than 0.01 ft (0.003 m). The validation results indicate good agreement for the modeled subbasins as both the magnitude and timing of the flood wave are captured adequately at the Jocassee and Keowee reservoirs.

For the PMF, all-season PMP values obtained from HMR-51 were used to create depth-area-duration (DAD) curves for storms both larger and smaller than the watershed size with durations ranging from 6 hours to 72 hours. A 700 mi^2 (1,813.0 km^2) storm positioned at an orientation angle of 187° produced the greatest total rainfall volume for the drainage basin. Instead of placing the storm at the basin centroid, HMR-52 and HEC-1 were used to incrementally assess various locations of the storm to find the one that produces the maximum reservoir elevation. The flood is routed through the Jocassee Reservoir using level pool routing. The modeling assumed that Jocassee Dam is operated to release maximum amounts of water during the PMF event, thus increasing water levels in Keowee reservoir. This approach is conservative for assessing the maximum flood level in Keowee Reservoir. Staff determined that the methods used to determine maximum rainfall amounts are consistent with current NRC guidance. The staff finds this approach to be conservative, and concurs with the licensee's assumption that the collective storage effects of the ponds and small lakes in the Keowee drainage basin could be assumed to be negligible. This is by taking into consideration the minimal effect they would have on flood elevation at Keowee Reservoir when compared with the size of the Jocassee Reservoir and Keowee Reservoir.

The reevaluated estimated peak PMF inflow to the Keowee Reservoir is 332,721 ft^3/s (9,421.6 m^3/s) with an estimated peak discharge of 139,961 ft^3/s (3963.25 m^3/s). This results in a peak headwater elevation of approximately 808.9 ft (246.55 m) MSL or about 6.1 ft (1.86 m) below the top of the dam elevation of 815 ft (248.4 m) MSL. Figure 3.3-3 presents the Keowee PMF hydrograph.

As an additional test of the appropriateness of the licensee's analysis for the PMF, the staff considered a scenario that postulates the coincident occurrence of the postulated PMP events for both Keowee and Jocassee Reservoirs. Combining the results of the two separate cases and assuming that the Jocassee flooding peak coincides with the secondary peak in the Keowee inflow hydrograph, staff calculated that the resulting water level in Keowee would reach 814.65 ft (248.305 m) MSL. This would leave about 4 in (10.2 cm) of freeboard at the West Saddle Dam. This hypothetical combined event is more severe than the PMF flooding event and the additional confirmation that the combined event flood would not overtop Keowee Dam and its appurtenant West Saddle Dam, which separates the ONS site from the Keowee Reservoir. This provides additional level of confidence in the conservatism of the licensee's analysis.

The licensee analyzed the combined effects of the PMF with wind-driven waves for both Keowee and Jocassee reservoirs. The USBR Wind Velocity Charts in the Technical Memorandum No.2 by the United States Bureau of Reclamation (USBR) titled, "Freeboard Criteria and Guidelines for Computing Freeboard Allowances For Storage Dams" (USBR, 1981) were used to determine wave height and runup for the two reservoirs. Results using the ANS 2.8, 2-year velocity (ANSI/ANS, 1992) were also determined for comparison. The USBR method yielded the most conservative results.

For the Keowee Reservoir, the combined event of the reevaluated PMF with wind-wave runup results in a flood elevation of 812.2 ft (247.56 m) MSL, leaving 2.8 ft (0.85 m) of freeboard at the West Saddle Dam.

The staff found that the licensee's re-evaluated PMF stillwater elevation of 808.9 ft (246.55 m) MSL for Keowee Reservoir (which includes the Jocassee watershed), with wind-wave runup to a flood elevation of approximately 812.2 ft (247.56 m) MSL, bounds the effects of PMF at

Keowee Reservoir and the ONS site. This finding is based on review and a qualitative assessment of the analysis provided by the licensee.

3.3.4 Conclusion

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from streams and rivers is not bounded by the current design-basis flood hazard. Therefore, the NRC staff expects that the licensee will submit a focused evaluation confirming the capability of flood protection and available physical margin or a revised integrated assessment consistent with the process and guidance discussed in COMSECY-15-0019 (NRC, 2015a).

3.4 Failure of Dams and Onsite Water Control/Storage Structures

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for site flooding due to failure of dams and onsite water control or storage structures is based on a stillwater surface elevation of (b)(7)(F) MSL. This elevation is due to a piping failure of the Jocassee main dam followed by the cascading failures of the Keowee main dam and West Saddle Dam due to overtopping. This flood-causing mechanism is not discussed in the licensee's current design basis (Duke, 2015b, Section 1.2.3).

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

3.4.1 Design-Basis and Previous Analyses for Failure of Dams

The licensee performed a dam failure analysis of Jocassee Dam in 1983 (Duke, 2015b) which indicated a peak flood elevation of [redacted] MSL at Keowee Dam resulting in a flood water elevation of [redacted] at the ONS powerblock area (note that the man-made dikes forming the Keowee Reservoir are at elevation 815 ft (248.4 m) MSL). In response to this study, [redacted] foot walls were constructed around the SSF. The walls were later increased by an additional (b)(7)(F) in response to a 10 CFR 50.54(f) request for information dated August 15, 2008 (NRC, 2008). In response to the letter, Duke increased the height of the flood walls to elevation (b)(7)(F) MSL.

In its response to a Confirmatory Action Letter (NRC, 2010), the licensee submitted the results from dam failure modeling of Jocassee Dam, which assumed a full failure of the dam with a time-to-failure of [redacted], and resulted in a peak water level of (b)(7)(F) MSL at the SSF (Duke, 2010). The NRC Staff Assessment of that response (NRC, 2011a) discussed the conservatisms contained in that analysis.

3.4.2 Dam Failure Reevaluation

In its revised FHRR (Duke, 2015b), the licensee evaluated the potential for both hydrologic overtopping and seismic failure of Jocassee Dam and determined that neither type of failure is credible for Jocassee Dam. Accordingly, the licensee considered a sunny-day failure of Jocassee Dam as the bounding critical failure event for the site. The reevaluations of hydrologic overtopping, seismic failure, and sunny-day failure are discussed in separate subsections, below. In response to staff requests for additional information, the licensee provided a description of 1-D and 2-D approach for modeling the Jocassee-Keowee Dam Failure assessment (Duke, 2014c). The licensee responded by letter dated June 13, 2014 (Duke, 2014c) which included a partial response that described a 1-D and 2-D approach for modeling the Jocassee-Keowee Dam failure assessment. The licensee also committed to provide additional responses in a separate submittal. The additional responses were provided by the licensee (Duke, 2015b). A comparison of the 2010 and 2015 postulated Jocassee Dam failure and downstream flooding evaluations performed by Duke, as well as a comparison of the NRC 2011 (NRC, 2011a) Staff Assessment and this report has been performed and included as Enclosure 2. This Enclosure provides an expanded discussion on the NRC staff's review, key results, and conclusions resulting from both evaluations.

3.4.2.1 Hydrologic Dam Failure Evaluation

Staff reviewed the licensee's evaluation of the potential for hydrologic overtopping of Jocassee Dam to determine whether the analysis is based on present-day methods and was performed in accordance with the NRC interim staff guidance JLD-ISG-2013-01 (NRC, 2013b) and other applicable regulatory guidance documents.

The reevaluated hazard study applied HMR 51 (NOAA, 1978) and HMR 52 (NOAA, 1982) to determine the storm area, size, location, and orientation to maximize the rainfall depth over the entire basin. Hydrologic routing of flow for the Jocassee watershed was performed using HEC-1. Since there are no continuous recording rainfall or stream gages in the Jocassee watershed, the response of the watershed is based on utilization of synthetic unit hydrographs. Normal depth routing of the PMF through the Jocassee reservoir is performed with HEC-1.

The watershed upstream of Jocassee Reservoir contains four reservoirs: Bad Creek pumped storage reservoir, Fairfield Lake reservoir, Sapphire Lake reservoir, and Lake Toxaway reservoir. Five different methods for assessing the times of concentration (lag time) were investigated. The Kirpich method (Kirpich, 1940) was ultimately chosen. Staff notes that the times of concentration estimated using the Kirpich method are generally smaller than estimates based on the other methods considered, therefore, the choice of this method supports conservatism with respect to the PMF flood levels.

Staff notes that the approach followed in analyzing the hydrologic response of Jocassee watershed differs from that used for determining the hydrologic response for the Keowee watershed (see Section 3.3.3) – which includes the Jocassee watershed – in that a regional parameter estimation methodology for unit hydrographs coupled with the availability of gages was used for validation in the Keowee watershed. The regional parameter estimation for the unit hydrographs could reasonably be applied to the Jocassee watershed to denote watershed response as its contribution is captured at Keowee with historical data. However, in the absence of gauging data for the Jocassee basin, a situation that creates difficulties for calibration, reasonable engineering judgments must be made and are used in this analysis. The PMF outflow hydrograph for Jocassee Reservoir is shown in Figure 3.4-1.

The licensee estimated a single SCS CN of 55 for the entire Jocassee watershed, based on a generalized assessment of the soil classification and soil cover. The licensee noted that a sensitivity study conducted by the licensee in 1993 (Bruce, 1993) indicated that varying the CN did not result in any significant changes in the peak stage. The licensee's 1993 analysis found that increasing the CN to 60 would increase peak stillwater elevation in Jocassee Reservoir by 0.76 ft (0.232 m), which is well within the freeboard determined by the licensee's analysis. The staff reviewed the licensee's basis for the estimation of CN and confirms it is appropriately conservative, and consistent with present-day methods and guidance. Sensitivity analyses conducted by the licensee found that a 30 percent reduction in lag-time resulted in only a 0.4 percent increase in flow and 0.2 ft (0.06 m) higher elevation at the Jocassee Dam after reservoir routing. The staff reviewed the Manning's roughness coefficient *n* values used in the calculations and determined that the selected values are reasonable.

For the PMF, the licensee used all-season PMP values from HMR-51 (NOAA, 1978) to create DAD curves for storms both larger and smaller than the watershed size for durations ranging from 6 hours to 72 hours. Using HMR 52 (NOAA, 1982) and from various combinations of storm-area size, location, and orientation, the maximum rainfall volume for the drainage basin was found. The average PMS depth over the watershed is estimated to be 36.41 in (92.48 cm) in 72 hours.

The PMF is routed through the four upper-basin reservoirs contained in the Jocassee watershed. Three of the facilities safely pass the PMF with only one being overtopped due to limited storage. Jocassee is modeled to operate with all four turbines and both tainter gate spillways functioning and operational due to their high availability and good operating conditions.

The licensee reported that the resulting peak PMF inflow to the Jocassee reservoir is 522,734 ft^3/s (14,802.18 m^3/s) with an estimated peak discharge of 85,405 ft^3/s (2,418.40 m^3/s). This results in a peak stillwater elevation in the Jocassee Reservoir of approximately 1,122.0 ft (341.99 m) MSL, or about 3 ft (0.9 m) below the top of the Jocassee Dam elevation of 1125 ft (342.9 m) MSL.

The licensee analyzed the combined effect of the PMF and wind-driven waves for both Keowee and Jocassee reservoirs. The United States Bureau of Reclamation (USBR) Wind Velocity Charts (USBR, 1981) were used to determine wave height and runup for the two reservoirs. Analysis using the ANS 2.8, 2-year velocity (ANSI/ANS, 1992) was also determined for comparison and it was determined that the USBR method yielded more conservative results.

At the Jocassee Reservoir, the combined maximum stillwater elevation with wind-wave runup results in a maximum elevation of approximately 1,126.4 ft (343.33 m) MSL. The analysis indicated that some waves would lap over the 20 ft (6.1 m) wide crest of the dam at elevation 1,125 ft (342.9 m) MSL, however, large diameter riprap on the upstream and downstream sides of Jocassee Dam would protect the dam from any significant erosion and slope stability issues.

3.4.2.2 Hydrologic Dam Failure Evaluation Conclusion

The NRC staff confirmed that the methodology and data used to assess flooding in the Jocassee watershed are consistent with current standards and guidance. Sensitivity studies performed by the licensee and staff provided additional confidence that the selected model parameters are reasonable. Therefore, the NRC staff agrees with the licensee's analysis that Jocassee Dam will not overtop and hydrologic failure is not reasonable based on present-day methodologies and guidance.

3.4.2.3 Seismic Dam Failure Evaluation

The NRC staff reviewed the licensee's evaluation of the seismic performance of the Jocassee Dam to assess whether it used present-day methods and was performed in accordance with the NRC interim staff guidance JLD-ISG-2013-01 (NRC, 2013b) and other regulatory guidance.

Guidance document JLD-ISG-2013-01 specifies that the following seismic load combinations should be considered in evaluating the potential for seismically induced dam failures:

- 10^{-4} annual exceedance seismic hazard (ground motion) combined with a 25-year flood;

- Half of the 10^{-4} ground motion, combined with the lesser of the a 500-year flood or one-half the probable maximum flood (PMF).

The licensee determined that the combined loads from the 10^{-4} ground motion and the 25-year flood exceeds the combined loads from one-half the 10^{-4} ground motion and the 500-year flood (found to be less than one-half the PMF). The scope of the staff's review included the seismic and hydraulic input (consistent with aforementioned load combinations) to the seismic stability analysis of the Jocassee Dam main embankment, including slope stability, liquefaction, and other relevant seismic-failure modes. Additional details regarding the licensee evaluation and staff review is provided below. Supporting information is available in the FHRR Audit Report (NRC, 2015c) and supporting technical details are available in the Center for Nuclear Waste Regulatory Analyses (CNWRA) Final Technical Evaluation Report, Review of Seismic Stability of Jocassee Dam to Support Staff Assessment of Oconee Nuclear Station Flood Hazard Reevaluation Report (CNWRA, 2015).

3.4.2.3.1 Seismic Input

Consistent with NRC guidance (NRC, 2013b), seismic input for the licensee's evaluation was based on a probabilistic seismic hazard analysis (PSHA). The overall approach follows standard development of seismic hazards for nuclear reactors (NRC, 2007b). The steps in the approach include (Duke, 2015b):

1. Development of hard rock hazard;

2. Site response analysis;

3. Hazard deaggregation and development of response spectra; and

4. Development of acceleration time histories and inputs to dam stability analysis.

The NRC staff reviewed the information provided by the licensee and concludes that the licensee conducted the seismic hazard evaluation for the Jocassee Dam according to present-day methodologies and regulatory guidance. Specifically, the licensee developed a PSHA for the Jocassee Dam based on the same seismic source characterization and ground motion models developed and used to conduct seismic re-evaluations of nuclear power reactor sites in response to the 50.54(f) letter (NRC, 2012b). For the site response, the staff finds that the licensee developed an appropriate composite soil profile based on site-specific soil properties. In addition, the licensee developed reasonable amplification factors to convert the rock hazard to a soil hazard consistent with (EPRI , 2013) and (NRC, 2001). Finally, the staff concludes that the methodologies and data used to develop the site specific ground motion inputs, including the UHRS and the scaled acceleration time histories, followed NRC guidance documents (NRC, 2001, NRC, 2007b, NRC, 2012a, and NRC, 2013b). The resulting scaled time histories are therefore appropriate for use to conduct dynamic stability analyses for the Jocassee Dam.

3.4.2.3.2 Hydraulic Loads Used In Seismic Dam Failure Analysis

The licensee developed the 25- and 500-year floods for use in the aforementioned seismic load combinations using hydrologic and hydraulic evaluations, as summarized in NRC, 2015c. The calculated peak headwater levels under these conditions are 1,110 ft (338.3 m) MSL and 1,111.5 ft (338.79 m) MSL, for the 25- and 500-year floods, respectively (NRC, 2015c). Staff finds the licensee's hydraulic loading analysis to be reasonable. The licensee's analysis of these flooding events is consistent with the methodology implemented in the licensee's analysis of the PMF for the Jocassee Reservoir (see Section 3.4.2).

Staff compared the peak discharges with the discharge capacity of the spillway and powerhouse structures and the storage capacity of the Jocassee Reservoir. This comparison supports the licensee's conclusion that minimum increases in water surface elevation would occur within the reservoir, even under the most severe scenario considered by the licensee in conjunction with the seismic event (i.e., sensitivity study involving reduced spillway conditions and electrical grid failure). The staff finds that the licensee's use of a constant headwater level of 1,110 ft (338.3 m) MSL while the 25-year flood is discharged at Jocassee Dam is reasonable because it (1) is a reasonable assumption under a condition in which the spillway tainter gates and pump-turbines continue to function and (2) is consistent with the headwater level associated with the four discharge scenarios originally considered as part of the FHRR assessment and the additional sensitivity study involving loss of the electrical grid. The licensee used the aforementioned seismic and hydraulic input for the assessment of the seismic performance of the Jocassee Dam. Because the licensee showed that the 10^{-4} ground motion and 25-year flood imposes a larger combined load, staff finds the licensee's use of the 10^{-4} ground motion plus 25-year flood combined event for conducting seismic dam failure analyses is reasonable.

3.4.2.3.3 Material Properties

Jocassee Dam consists of five distinct materials: foundation, core, filter, rockfill shell, and random rockfill. The licensee's description (and associated properties) of the foundation core, filter, and rockfill shell materials for use in the seismic performance evaluation is summarized in NRC, 2015c. The staff finds the values used for the core, the rockfill, and the filter to be generally consistent with published values, such as those in (Terzaghi, Peck, & Mesri, 1996) and with descriptions of the dam materials and construction methods described in licensee documents (NRC, 2015c). However, the staff observed that the characterization of the random rockfill was not consistent with material descriptions or historical licensee documents. Staff noted that this (1) may have an impact on the liquefaction evaluation because the random rockfill zone on the upstream part of the dam is saturated and may need further assessment if the material is not free draining, and (2) may have an impact on the stability and deformation evaluation if the strength is lower than assumed (i.e., the angle of internal friction is lower than assumed). Additional discussion addressing these two points is provided in Section 3.4.2.3.4. as well as (NRC, 2015c).

3.4.2.3.4 Seismic Performance Evaluation

The seismic performance of the Jocassee Dam includes identification of seismic failure modes and evaluation of deformation during seismic ground motion, factors of safety for post-earthquake stability, and potential liquefaction of the dam and foundation. The failure modes considered in the seismic performance evaluation are: (i) deformation and overtopping, (ii) deformation and transverse cracking at crest, (iii) liquefaction and sliding opening gaps, and (iv) deep cracking. The seismic performance evaluation is based on static stress analysis, slope stability analysis, dynamic response analysis, Newmark-type deformation analysis, and liquefaction assessment.

Liquefaction Potential

A summary of the staff review for each of the liquefaction potential of each of the dam sections is provided below:

- *Foundation materials*: Staff finds the licensee's determination of a non-liquefiable foundation is reasonable based on the material properties identified in NRC, 2015c and based on the assumption that the construction was accomplished according to design requirements.

- *Core materials*: The Standard Maximum Proctor Density identified in NRC, 2015c is 97 percent. The licensee concluded that liquefaction triggered in the core materials is unlikely because the dilation would prevent the development of excessive pore pressure during earthquake shaking. Based on this information, staff finds that the licensee's assumption that the dam's core material is not liquefiable is reasonable.

- *Rockfill*: Based on the characterization of the rockfill zone identified in NRC, 2015c, the staff finds the licensee's conclusion that this zone is both dense and free draining and will therefore not generate excess pore pressure during an earthquake to be reasonable.

- *Filter*: Based on the characterization of the rockfill zone identified in NRC, 2015c, staff finds that it is reasonable to conclude that there is no potential for liquefaction.

- *Random rockfill*: Based on the characteristics of the downstream zone identified in NRC, 2015c, staff finds that it is reasonable to conlude that this downstream zone is not likely to be saturated and development of excess pore pressure during ground motion can be generally precluded. However, the random rockfill material was used for a significant section of the upstream dam shells where the random rockfill is saturated. Based on information contained in NRC, 2015c, including supporting calculations performed by the licensee in response to NRC staff inquiries, staff finds it reasonable to conclude that pore pressure buildup will not be significant in the upstream random rockfill during seismic loading.

Staff finds that the licensee reasonably addressed the liquefaction potential of the Jocassee Dam and its foundation as part of the licensee's seismic performance evaluation.

Deformation Analysis and Post-earthquake Stability

Using SIGMA/W software, the licensee estimated the static stress state in the dam. The licensee used the UTEXAS4 software for slope stability analysis. The dynamic response of the embankment to the selected ground motions was modeled using the QUAD4MU software. For the deformation analysis, the licensee calculated displacement of the sliding mass based on the Newmark approach using the TNMN software, which is consistent with JLD-ISG-2013-01 (NRC, 2013b) since no potentially liquefiable soils are present. Additional details are provided in NRC, 2015c. Staff finds that the displacement analysis results reasonably support the licensee's conclusion that the displacement is small for the selected seismic loading and that deformation of the dam is not significant.

The post-earthquake factors of safety were evaluated on the upstream and downstream slip surfaces. The licensee states that the calculated downstream and upstream slope stability factors of safety are above 1.5 and 1.6, respectively. Staff's review of the post-earthquake slope stability verified that factors of safety were found to exceed requirements.

In response to NRC questions regarding properties related to the friction angle assigned to the random rockfill, the licensee performed a sensitivity study considering a lower friction angle and demonstrated it did not have a significantly adverse effect on computed factors of safety for slope stability (NRC, 2015c). Staff found that the licensee's sensitivity study shows that reasonable variation in friction angle will not adversely affect conclusions regarding acceptability of computed factors of safety.

Because the deformation analysis showed small deformations, the post-earthquake slope stability factor of safety was greater than 1.5, and the structure was assessed as not being susceptible to liquefaction, the licensee concluded that development of the four seismic failure modes was not likely (NRC, 2015c). Staff find that it is reasonable to conclude that the three generic failure modes (deformation and overtopping, deformation and transverse cracking at crest; and liquefaction and sliding opening gaps), would not be of concern because each of the failure modes require liquefaction and significant induced deformation. Staff finds that it is also reasonable to conclude that the deep cracking failure mode is not likely for the Jocassee Dam because seismic deformation is expected to be small based on the licensee's seismic analysis and the empirical evidence regarding performance of similar dams subjected to seismic loads (NRC, 2015c and Duke, 2014d). Specifically, the staff notes that, because there are sources of settlement that are not captured by the Newmark-type analysis, the empirical prediction of crest settlement is expected to be more than those computed by the analysis. However, staff finds that the empirical information in available literature (Swaisgood, 2014) confirms it is reasonable to expect small deformations.

Other Seismic Failure Modes

The NRC staff concludes that the four seismic failure modes (i.e., deformation and overtopping; deformation and transverse cracking at crest; liquefaction and sliding opening gaps; and deep cracking) identified by the licensee are applicable to the Jocassee Dam and consistent with generic failure modes as described by the Bureau of Reclamation (USBR, 2012). However, JLD–ISG–2013–01 and the Bureau of Reclamation (USBR, 2012) recommend identification of failure modes based on thorough review of relevant background information including performance and monitoring information. Therefore, staff considered the potential for additional failure modes. Specifically, the staff considered:

1. rapid reservoir rise due to the gates failing closed;

2. drawdown following a seismic event due to the gates failing ope;, and

3. possible exacerbation of internal erosion in the (b)(7)(F) by seismic ground motions.

In response to NRC questions regarding rapid rise and drawdown scenarios (NRC, 2015c), the licensee provided information that the tainter gates are not designed to withstand overtopping; therefore, if the tainter gates cannot be raised after a seismic event, water levels would rise, and the gates would be overtopped. Because the tainter gates are not designed to withstand overtopping, they would fail and the headwater level would drawdown to 1,077 ft (328.3 m) MSL, the elevation of the spillway crest. Existing stability analyses (as identified in NRC, 2015c) show that the factor of safety against sliding does not decrease significantly following a rapid drawdown to 1,077 ft (328.3 m) MSL.

The staff finds that the licensee's technical basis for not considering rapid rise of headwater elevations due to the gates failing closed from seismic loads (as summarized in NRC, 2015c) is reasonable. In addition, staff assessed whether the Jocassee Reservoir could be overtopped under a condition in which no outflow is credited for the spillway or powerhouse and a 25-year, 24-h peak discharge is held constant (i.e., Jocassee is modeled as a "pool" with a constant peak inflow and no outflow). Staff analysis indicates that Jocassee Reservoir has adequate storage potential to avoid overtopping under this scenario. Staff notes that this scenario was postulated only to assess whether the dam will not be overtopped under the scenario described.

Because existing stability analyses (as identified in NRC, 2015c) show that the factor of safety against sliding does not decrease significantly following rapid drawdown to 1,077 ft (328.3 m) MSL, the staff finds that this potential failure mode does not affect conclusions regarding dam stability. Moreover, in the event that an appropriate combination of the gates controlling discharge through the power tunnels and penstocks fail in the open position, staff review (as summarized in NRC, 2015c) indicates that this failure mode will not reasonably affect conclusions regarding dam stability.

Staff identified the potential failure mode involving triggering of erosion of embankment into the foundation based on information reviewed through the audit process (NRC, 2015c). In response to NRC questions, the licensee stated that this failure mode is not credible due to the location of seepage, previous treatment, and monitoring (NRC, 2015c). However, the staff noted that this failure mode remains possible due to seismic loading. Staff noted that the sunny-day failure evaluation separately considers a similar failure mode involving seepage triggered through the (b)(7)(F) Staff notes that there is a relatively modest increase in discharge and resulting rise in reservoir level associated with the 25-year flood (defined by seismic load combinations) relative to the hydraulic conditions considered in the sunny-day dam failure evaluation. Therefore, it is not expected that failure induced by seepage through the (b)(7)(F) due to a seismic event will result in consequences significantly in excess of those estimated in conjunction with the sunny-day failure (as discussed in Section 3.4.2.3). Therefore, staff concluded that exclusion of this failure mode from further consideration is reasonable.

3.4.2.3.5 Seismic Failure Mode Conclusion

Staff reviewed the licensee's evaluation of the seismic performance of the Jocassee Dam. Staff assessed whether the licensee applied present-day methods and if the evaluation was performed in accordance with the NRC interim staff guidance JLD-ISG-2013-01 (NRC, 2013b) and other appropriate regulatory guidance. Based on the review discussed in the sections above, staff concludes that seismically-induced failure of the Jocassee Dam is not reasonable based on present-day methodologies and guidance.

3.4.2.4 Sunny-Day Dam Failure Evaluation

As outlined in the Interim Staff Guidance JLD-ISG-2013-01 (NRC, 2013b), sunny-day dam failures should not be screened out and should be the default failure mode assumed when other failure modes (e.g. hydrologic and seismic) are not reasonable. Accordingly, the licensee postulated a sunny-day failure of the Jocassee Dam due to internal piping of the dam embankment. The potential effects of cascading failures of downstream dams were included in the analysis. It should be noted that the dam failure analysis provided in the FHRR differs from the 2010 Confirmatory Action Letter response (Duke, 2010), which also evaluated sunny-day failure of the Jocassee Dam but was based on different dam failure assumptions.

3.4.2.4.1 Summary of Sunny-Day Dam Failure Analyses

In its original FHRR submittal (Duke, 2013a), the revised FHRR (Duke, 2015b), and other supporting documentation provided to NRC, the licensee described several different analyses and analytical methodologies used to explore the potential consequences of a sunny-day failure of the Jocassee Dam. It is well known that there is uncertainty in the analysis of a dam breach and the resulting reservoir-outflow hydrograph (NRC, 2013b). As such, the consideration of multiple scenarios is appropriate. The licensee's analysis included use of several different published analytical methods for the analysis of dam breaches, developed from regression analyses of historical dam failures. Physically based approaches to analysis of dam breach were also considered. Breach parameters determined by these methods were used as input to the one-dimensional hydraulic model HEC-RAS (USACE, 2010), which was used to model the breach process and resulting hydrograph, route the flow through Keowee Reservoir, model the subsequent breaching of Keowee Dam and the appurtenant West Saddle Dam, and determine flood water elevations at the ONS site. Two-dimensional hydraulic modeling was conducted to provide a more refined flow analysis and provide hydrodynamic details regarding water velocity and inundation in and around the ONS site. The following subsections focus on specific components of the sunny-day failure analysis.

3.4.2.4.2 Analysis of Jocassee Dam Breach Parameters and Breach Process

The licensee's analyses of the sunny-day breach of the Jocassee Dam assumes that a piping failure occurs while the Jocassee Reservoir is at full pool (elevation 1,110 ft (338.3 m) MSL). Piping is postulated to begin in natural geologic materials in the (b)(7)(F) of Jocassee Dam, at elevation 1,020 ft (310.9 m) MSL. This is the (b)(7)(F) that has been observed throughout the life of the dam and the licensee identified it as the (b)(7)(F) (b)(7)(F) (Duke, 2013b, Enclosure 1). Piping in a rockfill dam is considered to be unlikely, as experience indicates that a rockfill dam can sustain considerable through-flows and still maintain stability (Duke, 2013b, Enclosure 1).

The initial submittal of the FHRR (Duke, 2013a) reported results of a dam-breach analysis that used regression equations developed by Xu and Zhang (Xu and Zhang, 2009) to estimate the geometry and timing of a dam breach and the subsequent release. One major difference between the Xu and Zhang methodology and most other regression-based methods is the inclusion of dam erodibility as a parameter. Using the published methodology of Xu and Zhang and treating the Jocassee Dam as a "low-erodibility" dam, the licensee generated ranges of values for breach parameters; the "best exact" estimate of mean outflow from the breached dam was approximately [redacted] (Duke, 2013b, Enclosure 1). The breach geometry values and failure time generated from the Xu and Zhang analysis were input to HEC-RAS (River Analysis System), and the orifice coefficient and weir coefficient parameters in HEC-RAS were iteratively adjusted until HEC-RAS reproduced a breach flow that matched flow predictions from the Xu and Zhang outflow equation. To support the dam breach analysis used as part of the FHRR evaluation, the licensee submitted a report developed by Ehasz and Bowles (Ehasz and Bowles, 2014) describing the process of developing these coefficients. Ehasz and Bowles report that final values of these two coefficients were 0.1 and 2.0[5], respectively. These values are outside the ranges (0.5 to 0.6 for the orifice coefficient and 2.6 to 3.0 for the weir coefficient) that the Corps of Engineers (USACE, 2014) recommends for use in HEC-RAS analysis of "earthen sand and gravel" dams. Similarly, the Colorado Department of Natural Resources (CDNR, 2010) recommends values similar to those recommended by USACE. The licensee reported that its subsequent analysis using HEC-RAS and SRH-2D (discussed in other subsections of this section) found that flood water levels would remain below the powerblock area elevation at the ONS site.

ɔ)(7)(F)

The NRC staff requested the U.S. Bureau of Reclamation (USBR) to provide an independent technical review of the Xu and Zhang methodology, and subsequently requested both the USBR and the Federal Energy Regulatory Commission (FERC) to review and comment on the licensee's implementation of the Xu and Zhang methodology and selection of breach parameters for the Jocassee Dam. The USBR review of the published Xu and Zhang methodology (Wahl, 2014a) determined that the Xu and Zhang methodology cannot be confidently applied to low-erodibility dams or to the prediction of failure time. Reviewing the specific application of the method for Jocassee Dam, Wahl determined that the dam should be classified as medium erodibility instead of low erodibility (Wahl, 2014b). Wahl also recommended "best estimate values" of breach parameters: (a) [redacted]

(b)(7)(F)

The FERC also commented (Allerton, 2014 and Brown and Burgess, 2014) on the lack of field observed data upon which to base a model for breaching of low-erodibility dams, and noted that the Jocassee Dam and Reservoir are much larger than the dams and reservoirs in the available historic data sets used to develop the Xu and Zhang methodology and other regression equations. The FERC described a suite of sensitivity studies related to the Jocassee Dam failure that the FERC staff had performed, and recommended considering a [redacted]

(b)(7)(F)

[5]The weir coefficient of 2.0 is reported on page B-15 of Ehasz and Bowles (Ehasz and Bowles, 2014), and it is the value that staff found in the HEC-RAS input data supplied by the licensee. However, Ehasz and Bowles state on page 43 of that report that the final value of the weir coefficient was 2.2, and on page B-16 that it was 2.7.

The NRC staff issued an RAI by letter dated September 15, 2014 (NRC, 2014c), requesting the licensee to reanalyze and resubmit the dam failure analyses for the FHRR after applying alternate breach-parameter estimations than those predicted using the Xu and Zhang methodology. For the analysis presented in its revised FHRR (Duke, 2015b), the licensee estimated dam breach parameters via empirical regression methodologies and considered an array of different equations in developing these parameters (Froehlich, 1995a; Froehlich, 1995b; Froehlich, 2008, Walder and O'Connor, 1997; MacDonald and Langridge-Monopolis, 1984; and Von Thun and Gillette, 1990). These methodologies are used to estimate embankment dam breach characteristics, as well as peak discharge from the breach. The breach parameters of principal interest are those for the failure of the Jocassee Dam, but it is important to note that for its assessment of flooding impacts at the ONS site, the licensee also developed dam breach parameters for Keowee Dam, its appurtenant West Saddle Dam, the ONS Intake Canal Dike and Little River Dam.

The NRC staff noted that Jocassee Dam and the impoundment Jocassee Reservoir are substantially larger than the dams and reservoirs whose failures were evaluated for the development of empirical breach equations. Most of these regression methodologies rely on the same dataset of 108 historical dams. The dam failure dataset contains dams ranging from 12 ft (3.7 m) to 305 ft (93.0 m) high, with 75 percent of the dams being less than 49 ft (14.9 m). In addition, the largest reservoir volume in the dataset is 535,000 acre-ft (660,000 m^3), with 75% of the dams having less than 12,100 acre-ft (14,900,000 m^3) of storage (NRC, 2015c). Also, most of these historical dam failures are earthen dams rather than rockfill dams and the majority of the failures resulted from overtopping rather than piping.

Since the historical datasets used in development of empirical dam breach methodologies do not include dams as large as the Jocassee Dam and are more representative of earthen dams, the licensee relied heavily upon engineering judgment to estimate breach time, pattern, and size and considered eight different empirical methodologies in developing breach parameters (NRC, 2015c).

As indicated in several dam failure literature sources (USACE, 2014; Wahl, 2004; Wahl, 2014a; and Chauhan et al., 2004), estimation of dam failure parameters and impacts is subject to a high degree of uncertainty. As such, the consideration of multiple scenarios is appropriate.

To develop the Jocassee Dam breach parameters, the licensee used various methods, including both empirical formulas and physical and hydraulic modeling. The proposed Jocassee Dam failure is assumed to occur through piping along the (b)(7)(F) [redacted] at an elevation of 1,020 ft (310.9 m) MSL, with an initial water surface elevation of 1,110 ft (338.3 m) MSL (full reservoir elevation) (NRC, 2015c).

The licensee used the National Weather Service (NWS) physically-based mathematical model (NWS BREACH) (Fread, 1991) to perform its analysis of the breach of the Jocassee Dam as part of the initial analysis. The NWS BREACH determines breach parameters and the breach outflow hydrograph by coupled analysis of reservoir inflow, breach outflow, and the sediment transport capacity of the unsteady flow through the breach channel, considering the material properties of the dam (Fread, 1991). Other physical models of the dam-breach process were considered, but were not used for reasons of unavailability or lack of previous acceptance by the NRC (NRC, 2015c). The licensee reported that its NWS BREACH analysis found a breach
(b)(7)(F) (b)(7)(F) development time of [redacted] which the licensee considered unrealistic [redacted] for a large

embankment dam like Jocassee. However, the licensee's analysis of sensitivity to the breach initiation time indicated that peak outflow predicted by the NWS BREACH model is largely insensitive to the breach initiation time – a failure time of (b)(7) resulted in a peak outflow of (b)(7)(F) while a failure time of (b)(7) resulted in a peak outflow of (b)(7)(F) (NRC, 2015c). The licensee provided a copy of the input file for NWS BREACH in response to a staff request. The NRC staff ran the model using the licensee's input file and obtained a predicted peak flow of (b)(7)(F) somewhat less than the licensee's reported result, and a failure time of (b)(7) which is higher than the licensee's reported result. The NRC staff noted that the differences in the estimated values are acceptable for the subsequent analysis. The analysis also predicted a final breach invert elevation of (b)(7)(F) MSL, equivalent to the normal full-pool elevation of Keowee Reservoir.

The licensee stated that some of the results of the Jocassee NWS BREACH model are unrealistic; however, they used both the final breach invert elevation of (b)(7)(F) MSL and the breach initiation phase results to inform its further analysis. Brown and Burgess reported (Brown and Burgess, 2014) that the FERC staff could not get the NWS BREACH model (which considers the strength of the constructed dam) to simulate a piping breach of the main dam until they increased the width of the initial pipe to (b)(7)(F), which they consider an extremely unrealistic value. The NRC staff also found that the NWS BREACH would not simulate a piping failure of the Jocassee Dam until the initial pipe size was increased to unrealistically high values. The licensee used the NWS BREACH progression shape as a basis for further progression modifications, which include no adjustment to the (b)(7)(breach initiation phase, extension of the breach development time to (b)(7) based on the failure time equation of Von Thun and Gillette (Von Thun and Gillette, 1990), and extension of the full-failure time to (b)(7) to capture the time to fully drain the Jocassee Reservoir (Duke, 2015b).

The licensee identified the empirical breach equations of Von Thun and Gillette (Von Thun and Gillette, 1990), Froehlich (Froehlich, 1995a; Froehlich, 1995b; and Froehlich, 2008) as the best candidates for analysis of the Jocassee Dam, then tested these equations by using them to estimate breach parameters for three historic dam failures (Hell Hole Dam, Teton Dam, and Oros Dam) and comparing the results with the observed values. The licensee found that the Von Thun and Gillette methodology had the least overall prediction error, and therefore selected it for use in the breach analysis for Jocassee Dam. In addition, the licensee plotted breach widths for several historic dam failures vs reservoir volume, fitted a linear trend line to the data, and added the predicted breach widths for the Jocassee Reservoir volume to the graph. The Von Thun and Gillette prediction fell closest to the linear trend (NRC, 2015c).

As discussed above, the NRC staff notes that essentially the same set of historical dam failures was used in developing all of the regression models and that the data set used for inter-comparison of the model predictions is limited with large error bands. Wahl (Wahl, 2004) found that the regression equations for dam breach analysis have uncertainties in the range of an order of magnitude (NRC, 2013b). Wahl (Wahl, 2004) states that "uncertainty of breach parameter predictions is likely to be significantly greater than all other factors, and could thus dramatically influence the outcome." The report by Wahl (Wahl, 1998) analyzed and utilized many of the currently available equations to predict breach parameters for 108 documented case studies and provided plots of the predictions vs the observed values. The results indicated that prediction errors of ±75% were not uncommon for breach width, while prediction errors for failure time often exceeded one order of magnitude. The NRC staff notes that the linear relationship between breach width and reservoir volume that the licensee presented is a poor fit to widely diverse historical data and requires extrapolation to Jocassee's reservoir volume, which exceeds the volume of any historically observed failure. However, given the analysis provided by the licensee regarding consideration of various empirical regression methodologies and equations; the use of physical models; consideration of other models of the dam-breach processes; sensitivity analyses and the limited dataset on observed dam failures, the NRC staff finds the licensee's selection of the Von Thun and Gillette method for estimating dam breach width and failure time to be reasonable and consistent with current guidance.

For the height of the Jocassee Dam and volume of the Jocassee Reservoir, the Von Thun and Gillette equations predict an average breach width of (b)(7)(F) [redacted] and a failure time of (b)(7)(F) [redacted] (NRC, 2015c).

3.4.2.4.3 1-D Hydraulic Modeling and Peak Outflow Sensitivity Analysis of the Jocassee Dam Breach

To predict peak outflow passing through the Jocassee Dam breach, the licensee performed sensitivity analysis by varying input parameters in HEC-RAS. The HEC-RAS model attempts to simulate the breach by modeling storage, breach parameters, piping flow, and weir flow over time.

The licensee (NRC, 2015c) cited Chauhan et al. (Chauhan et al., 2004) as having stated that parameters resulting from regression equations can result in overestimates of peak outflows from breached dams when used in routing models such as HEC-RAS. The licensee stated that one reason for this is that the peak outflow is likely to occur sometime before the breach is fully formed. Chauhan et al. (Chauhan et al., 2004) recommended that fractional values of breach widths and breach formation times resulting from regression equations could be used to achieve "reasonably realistic" estimates of breach parameters instead of "conservative" estimates that would result from applying the full values. Based on this recommendation, the licensee defined a range of potential fractional reductions to the Von Thun and Gillette dam breach parameters, then evaluated the sensitivity of peak outflows from the Jocassee Dam to different combinations of these parameter values (NRC, 2015c).

The two parameters that the licensee evaluated in its analysis of sensitivities of breach outflow were breach progression and breach width. The licensee reasoned that since the postulated piping failure would occur at an (b)(7)(F) [redacted] the breach width would be reduced below the model-predicted value, which is based on a dam breach that begins at the center of a dam (NRC, 2015c).

In addition, the licensee qualitatively considered the Jocassee Dam's physical size and construction to be a limitation on how fast the failure would progress (NRC, 2015c). The licensee stated that peak outflow would occur prior to full-breach development, due to decreases in head differential and water velocities across the dam (due in large part to the rise in water level on the downstream side). A site-specific breach-progression shape was developed via sensitivity runs to match the initiation phase predicted by NWS BREACH and extended out to the full-formation time as defined by HEC-RAS, which represents 100 percent breach completion. A total of nine Von Thun and Gillette sensitivity analyses were made by the licensee to compute peak outflow by altering breach width and breach progression (Table 3.4-1). The licensee included breach widths of either 100 percent, 80 percent, or 60 percent of the Von Thun and Gillette best-fit value and breach progression of either 80 percent, 70 percent, or 60 percent breach completion through the breach initiation and development phases. The licensee used a duration of (b)(7)(F) which is the (b)(7)(F) initiation phase predicted by NWS BREACH plus the (b)(7) development phase predicted by Von Thun and Gillette (NRC, 2015c).

The resulting suite of sensitivity runs presented by the licensee (Table 3.4-1) show that peak outflow from the Jocassee Dam can range from (b)(7)(F) using a (b)(percent progression, (b)(percent width scenario to (b)(7)(F) using a (b) percent progression, (b) percent width scenario (NRC, 2015c). The licensee selected an intermediate run, which uses a (b) percent progression, (b) percent width scenario and results in a peak outflow of (b)(7)(F) The final Jocassee Dam breach parameters, as determined by the licensee, include a top width of (b)(7)(F) a bottom width of (b)(7)((b)(7)(F) a final bottom breach invert of (b)(7)(F) MSL, time-to-peak-outflow of (b)(7 a full formation time of (b)(7) a reservoir emptying time of (b)(7) and a peak outflow of (b)(7)(F) (Duke, 2015b, Table 7).

The licensee's HEC-RAS model implementation simulated the breach outflow discharge from Jocassee assuming a weir coefficient of 2.0 and a piping coefficient of 0.1, both of which appear to have been based on previous calibration to match peak outflow predicted by the Xu and Zhang method (Ehasz and Bowles, 2014). The licensee presented a comparison between the predicted peak breach flow, peak breach flows predicted by other methods, and estimates of the peak breach flows from eight historical dam failures (NRC, 2015c).

The NRC staff reviewed the licensee's rationale for assessing the Jocassee Dam breach using fractional values of the Von Thun and Gillette dam breach parameters. Although the values selected do not represent the maximum or minimum values, they are within the range of the parameter estimates. The NRC staff understands the licensee's qualitative arguments for considering a smaller breach width and for assuming that breach progression is incomplete at (b)(7)(However, the NRC staff notes that 1) regression-based techniques use unverified peak outflow values, 2) significant uncertainty exists in calculating best-fit peak outflow values from limited scattered data, and 3) there is significant uncertainty in extrapolation for dams and reservoirs at sizes much larger than represented by the historical dam failures. In order to understand the sensitivity of flood level to breach parameter uncertainty discussed above, the staff performed independent analysis which is documented in Section 3.4.2.4.6.

3.4.2.4.4 Breach Parameters and Overtopping Failure of Keowee Dam

Because Keowee Dam and its appurtenant West Saddle Dam separate and protect the ONS site, analysis of the overtopping cascading failures of Keowee Dam and the West Saddle Dam are a critical element of the analysis of the consequences of the Jocassee Dam failure. Keowee Dam is a 3,500-ft (1,066.8 m) long earthfill embankment dam consisting of two main sections – a Main Dam and a West Saddle Dam. The West Saddle Dam is approximately 2,100 ft (640.1 m) in length and is between 20 ft (6.1 m) and 50 ft (15.2 m) tall, with a minimum crest elevation of 815 ft (248.4 m) MSL.

The HEC-RAS 1-D model was applied to simulate the Jocassee-Keowee dam breach. The model includes additional details and cross-section refinements. Additional details included cross-section refinements of Lake Keowee, Little River, and the connecting canal sections of the reservoir. The model includes routing to connect the intake canal dike and Little River Dam outflow back into the Keowee River.

The 1-D HEC-RAS model could not simulate the failures of both the Main Dam and West Saddle Dam using a single failure mechanism. In order to overcome this model limitation, the licensee's analysis simulated the breach of the West Saddle Dam using a crest gate of dimensions [redacted] and simulated the Main Dam using dam breach parameters. The progressive overtopping failure of the Keowee Dam begins with the water elevation within Keowee Reservoir rising rapidly and then overtopping the dam. The licensee selected 817 ft (249.0 m) MSL as the trigger elevation (the elevation at which the crest gate opens) used to initiate the breach of Keowee, which is [redacted] above the crest of the dam. The failure progression shape in the model was sinusoidal for the Main Dam and linear for the West Saddle Dam (simulated in HEC-RAS using a constant gate opening rate). Times to failure of 45 minutes and 30 minutes were selected for the Main Dam and West Saddle Dam, respectively. These values were determined by the licensee as part of the Keowee Development Structures breach parameters and reviewed by Ehasz and Bowles (Ehasz and Bowles, 2013). Staff reviewed the report and also performed sensitivity analysis, which is documented in Section 3.4.2.4.6

ɔ)(7)(F) (b)(7)(F)

The licensee used the HEC-RAS model to obtain flow, timing, and elevation solutions for the system encompassing Jocassee, Keowee, and Hartwell Reservoirs. Manning's roughness coefficients *n* are used in the main channel and overbank areas and are representative of river site conditions. Staff reviewed the model by performing sensitivity analysis in order to determine the range of parameter values and flows, and successfully replicated the licensee's results (Run H-1 in Table 3.4-2).

The licensee provided an independent technical review of the Keowee Dam breach parameters (Duke, 2013b, Enclosure 1). Further, the independent technical review suggested that "it is anticipated that the flood waters would encompass the entire crest and the breach would be rapid and likely erode most if not all of the embankment." Staff finds the parameter selection reasonable since the licensee assumed that failure is not initiated until water level is [redacted] above the top of the Keowee Dam. The NRC staff also conducted a sensitivity analysis to review the sensitivity of results to model parameters associated with timing of the failure of Keowee Dam affects flood water elevations at the ONS site, which is discussed in subsection Section 3.4.2.4.6.

(b)(7)(F)

3.4.2.4.5 2-D Hydrodynamic Modeling of Dam Failure Flooding Impacts

The licensee conducted 2-D modeling to simulate flow patterns in Keowee Reservoir and assess inundation at the ONS site. For the analysis presented in the original FHRR, the licensee used the SRH-2D model developed by the Bureau of Reclamation (Lai, 2008 and USBR, n.d.). For the analysis presented in its revised FHRR, the licensee replaced SRH-2D with the 2-D depth-averaged TUFLOW FV (finite-volume) computer model (Build Version 2014.01.007), (BMT WBM, 2013 and BMT WBM, 2014), which has additional features and capabilities. This 2-D model was used to resolve the complexity of the flow near the Keowee Dam, through the connecting canal (located between the Keowee and the Little River arms of Lake Keowee), and in the vicinity of the ONS SSF and includes routing to connect the Intake Canal Dike and Little River Dam outflow. The 2-D TUFLOW FV model is capable of processing the wetting and drying of grid cells, steady and unsteady flows, and sub/super-critical flows for complex channel geometries (Duke, 2015b and NRC, 2015c). The Keowee Dam spillway is modeled as an internal boundary condition, with both inflow and outflow boundaries specified within the 2-D model and taken from the 1-D flow hydrograph for the spillway and powerhouse portion of the Keowee Dam inline structure (Duke, 2015b).

The upstream and downstream boundary conditions for the 2-D TUFLOW model were specified based on the 1-D HEC-RAS model results. The general extent of the 2D TUFLOW model and locations of the boundary conditions are presented in Figure 3.4-2 (NRC, 2015c). The TUFLOW FV model was informed by the Jocassee Dam 70% progression, 80% width scenario output from HEC-RAS, to assess flooding impacts at the ONS site. The HEC-RAS analysis of this scenario indicated a peak outflow of (b)(7)(F) from the Jocassee Dam. Breach initiation of Keowee Dam (i.e., water elevation reaches (b)(7)(F) MSL) occurs (b)(7)(F) after the Jocassee Dam breach and produces a maximum water surface elevation in the Keowee tailrace area of (b)(7)(F) MSL (Duke, 2015b, Section 2.3.4). The licensee's analysis predicts that neither the Oconee Intake Dike nor Little River Dam is overtopped as a result. The licensee's TUFLOW FV modeling found that a maximum water surface elevation in Lake Keowee of (b)(7)(F) MSL is reached at hour (b)((Duke, 2015b, Section 2.3.4).

The NRC staff focused its review of the dam failure modeling on reviewing the: 1) methodologies and scenarios being modeled and 2) the implementation of the modeling through review of parameters and sensitivity analyses. The NRC staff's review of the 2-D model centered on the model's performance with respect to its stability and capability to reasonably predict water surface levels on the ONS site. After reviewing the dam failure analysis provided with the original FHRR, the NRC staff issued an RAI by a letter dated March 20, 2014 (NRC, 2014a), requesting information about the location of the upstream boundary conditions, flow velocity distributions for the dam breach model, and discussions on any sensitivity runs.

By letter dated April 25, 2014, the licensee responded (Duke, 2014a). In its response, the licensee identified that the velocity distribution is independent from its initial condition. The NRC staff concludes (based on engineering judgment), that in such a case involving extreme magnitudes of flow, the dam failure progression is more responsive to the flowrate and corresponding reservoir elevation as opposed to the velocity distribution entering the forebay of the Keowee Dam. The NRC staff performed a sensitivity run for a refinement of the mesh in the region of the canal that connects the Little River arm of Keowee Reservoir to the main Keowee River arm. In the flooding analysis, this region contains high gradients of velocity, contributing to appreciable water surface slopes in the reservoir on either side of the connecting canal. The refined mesh results produced by staff indicate a slight decrease of the water surface elevation on the ONS site (Run T-7, Table 3.4-3). This indicates that the licensee's analysis is reasonable with respect to mesh refinement in the canal region.

3.4.2.4.6 NRC Staff Sensitivity Analysis of the Sunny Day Dam Failure

The NRC staff performed independent sensitivity analyses to estimate the effects of dam breach parameters and other uncertain modeling parameters on water elevations at the ONS site, as described below. Given the large uncertainty associated with dam breach parameters and other assumptions required for model application, staff conducted sensitivity of a wide range of effects in order to composite a suite of sensitivities for assessing the licensee's analysis.

The licensee suggested that Jocassee dam breach progression and breach width are the most sensitive parameters affecting peak outflow. As a result, the NRC staff used HEC-RAS to evaluate the sensitivity of ONS site water elevations to the more severe values of breach parameters postulated by the licensee (NRC, 2015c) and also used TUFLOW FV (Build Version 2014.01.003) to examine the effects on water surface elevation and velocity from a subset of the more critical analyses. As described in Section 3.4.2.3.2 the NWS BREACH model results were used to inform the HEC-RAS 1D model.

In addition, the NRC staff evaluated the impacts of adjusting the side slopes of the breach opening at the Jocassee Dam, piping and weir coefficients for the HEC-RAS analysis of the Jocassee Dam breach, Keowee Dam time-to-failure, HEC-RAS model Manning's roughness coefficients, TUFLOW FV eddy viscosity values, TUFLOW FV mesh refinement, and TUFLOW FV canal constriction from bridge piers, as well as assessing how the lack of a Keowee Dam failure would affect flood elevations within the Keowee Reservoir. These various sensitivity runs are outlined below and described in detail in a separate Technical Review Documentation (ORNL, 2015).

The suite of 1-D sensitivity runs conducted by the NRC staff using HEC-RAS is presented in Table 3.4-2. Figure 3.4-3 provides a graphical comparison of results from selected 1-D sensitivity runs. The suite of 2-D sensitivity runs conducted by staff using TUFLOW FV is presented in Table 3.4-3, and Figure 3.4-4 provides a graphical comparison of results from selected 2-D sensitivity runs. The NRC Staff analysis indicates that the results of HEC-RAS and TUFLOW FV modeling analyses were not significantly different over a range of model parameters, and were most similar for higher peak outflow scenarios. The model results demonstrate that the maximum water surface elevation at ONS is highly sensitive to several parameters, the most important of which are breach progression, weir coefficient, and breach width for the postulated failure of Jocassee Dam.

The NRC staff notes by adjusting the breach progression from 70 percent to 80 percent of the Von Thun and Gillette value, maximum water surface elevations in both the 1-D and 2-D models exhibited an increase of approximately [redacted] at the ONS site (Runs H-5 and T-2; Tables 3.4-2 and 3.4-3 and Figures 3.4-3 and 3.4-4). An additional [redacted] of water results from increasing both the breach progression and the final breach width assumed (Runs H-6 and T-3; Tables 3.4-2 and 3.4-3 and Figures 3.4-3 and 3.4-4).

)(7)(F) (b)(7)(F)

Regarding the use of piping and weir coefficients, guidance on the use of HEC-RAS for dam-breach analysis (USACE, 2014, USACE, 2010, and CDNR, 2010) indicates that a piping coefficient of 0.6 and weir coefficient of 2.6 should be used. The licensee however selected values of 0.1 and 2.0, respectively. The NRC staff's sensitivity analysis indicates that the lower piping coefficient used by the licensee may result in higher water surface elevations at the ONS powerblock (Run H-7, Table 3.4-2 and Figure 3.4-3), but the licensee's weir coefficient could produce lower water surface elevations (Run H-8, Table 3.4-2 and Figure 3.4-3). The staff notes the effects of increasing the weir coefficient dominate changes in water surface elevations and changes to the piping coefficient had less effects on water surface elevations. Changing both parameters to default values in the USACE guidance increases the predicted water surface elevation at ONS by [redacted] compared to the licensee's submitted case (Runs H-9 and T-4; Tables 3.4-2 and 3.4-3 and Figures 3.4-3 and 3.4-4).

)(7)(F)

Sensitivities of peak water surface elevations to other model parameter selections are not necessarily small, but are smaller than the variations resulting from changes to the Jocassee breach progression, Jocassee weir coefficient, and Jocassee and/or Keowee breach width model parameters. Run H-12 (Table 3.4-2 and Figure 3.4-3), confirmed that higher water surface elevations occur at the ONS site if Keowee Dam is assumed to not fail. The assumption of Keowee Dam failing by being overtopped by less than [redacted] reduced the water surface elevation on the ONS site by less than [redacted]. Changes in the breach duration (time to

(b)(7)(F))(7)(F)

failure following breach initiation) for the Keowee Dam and West Saddle Dam also increased ONS site water surface elevations by (b)(7)(F) or less (Runs H-13, H-14, H-15, T-4 and T-5; Tables 3.4-2 and 3.4-3 and Figures 3.4-3 and 3.4-4). Changes in Manning's roughness coefficient also varied ONS site water surface elevations by as much as (b)(7)(F) (Runs H-16, H-17, and H-18; Table 3.4-2).

3.4.2.4.7 Model Uncertainty

The licensee's analysis considered several scenarios of breach parameters and the final selection was based on matching the peak outflow from the Jocassee Dam with that of regression-based techniques. The regression results are derived from data that are not necessarily representative of Jocassee Dam in terms of size, construction, and mode of failure. The lack of field-observed data introduces appreciable uncertainties. As discussed in Brown and Burgess (2014), it is also known that field-observed data (and, in turn, regression-based estimates) for breach formation time or peak flow values would be more accurate. The NRC staff acknowledges that the licensee has made efforts to produce a "realistic" dam failure scenario given the significant uncertainty that exists in dam breach estimation. The NRC staff's analysis indicates that the licensee's dam failure scenario and analysis is within the predicted range of values, although staff notes that the uncertainty bands on dam failure analysis are large. This results in a large variation in simulated water surface elevations at the ONS site, and the need to rely on considerable engineering judgement.

Literature on dam breach parameter estimation and simulation is diverse and evolving. Contemporary methodologies for estimating dam breach parameters include using regression equations developed from largely unverified data obtained from widely varying case histories. Considering that the majority of case histories used in developing these regression equations are for relatively small, earthen dams (neither of which descriptors applies to Jocassee Dam), the accuracy of predicted values is largely uncertain. Similarly, these methodologies also predict peak outflow values based on limited field data and often times on peak water height, which is then used to derive the peak outflow indirectly.

The licensee's FHRR evaluation of sunny-day failure of Jocassee Dam is based on assumptions of (b) percent of the predicted Von Thun and Gillette breach width and a breach progression based on (b) percent of the dam breach occurring through the breach development phase. The analysis also assumes a breach weir coefficient of 2.0 and piping coefficient of 0.1 for Jocassee Dam, time-to-failure of (b)(7)(F) and (b)(7)(F) for Keowee Dam and West Saddle Dam, respectively, Keowee failure trigger elevation of (b)(7)(F) MSL.

The NRC staff evaluated sensitivity of increasing the breach width to 100 percent of the Von Thun and Gillette value and showed that it was less sensitive than breach progression. When combined with an 80 percent progression, the 100 percent width scenario adds another (b)(7) (b)(7)(F) of water to the maximum water surface elevation at ONS SSF. The licensee stated in the FHRR that the piping breach of Jocassee Dam is most likely to occur at the (b)(7)(F) and that, consequently, the full width of the dam failure is likely to be limited by the physical propagation of failure and construction of the dam. This is consistent with an observation by Brown and Burgess (Brown and Burgess, 2014). Field and laboratory research on dam failure processes have shown that when lateral growth of a breach is limited in one direction, erosion rates in the other direction do not significantly increase to compensate. While this provides an engineering rationale for the reduction of breach width, the final breach width of (b)(7)(F) at the dam crest is only 53% of the total crest length (NRC, 2015c). Since dams as large as

Jocassee have not historically failed and the sample population and data accuracy is limited, engineering judgement and technical literature are key factors for selecting this value. The NRC staff compared the licensee's value with the range of possible values for breach width, and determined that the value is reasonable and within the bounds of the range.

The second most sensitive input variable as identified by the NRC staff is the breach weir coefficient used to estimate breach flow during the weir phase of the breach (i.e., the period following the collapse of the breach pipe). The staff's sensitivity analysis adjusted the licensee's weir coefficient of 2.0 and piping coefficient of 0.1 to the FERC applied values of weir coefficient (b)(7)(F) [redacted] The staff's sensitivity analysis resulted in an increase in maximum water surface elevation of approximately (b)(7)(F) [redacted] and indicated that weir and piping coefficients can change the maximum dam outflow. However, based on staff's sensitivity analysis and the reviewed documents mentioned earlier, staff determined that the values used by the licensee are within the bounds of the range of engineering judgment and relevant technical literature.

Additional sensitivity analysis by NRC staff considered other model variables, which had much smaller impacts on maximum water surface elevation at the ONS site. Among the variables evaluated, Manning's roughness coefficient and Keowee Dam time-to-failure were found to be the most sensitive but were relatively minor compared to the other variables evaluated. The NRC staff conducted a sensitivity analysis to evaluate various dam breach model input parameters. The results indicated that breach progression of the Jocassee Dam was the most sensitive parameter, with a change of progression of [redacted] percent to [redacted] percent resulting in an (b)(7)(F) over (b)(7)(F) [redacted] increase in maximum water elevation at the ONS SSF. Staff compared all selected model parameters to a reasonable, equally-likely, range. The NRC staff concluded the licensee's values are within the bounds of this range and the approach is reasonable.

3.4.2.4.8 Sunny-Day Failure Mode Conclusions

The NRC staff acknowledges that there is a range of expert opinion, as demonstrated in the technical literature and various FERC and licensee documents, that sunny-day failure of the Jocassee Dam is highly uncertain.

Given the degree of uncertainty associated with dam breach parameter estimation and modeling and based on its review of the licensee's information, staff determines that there are significant uncertainties in the analysis, and a reasonable basis exists for alternative analyses that would result in substantially higher or lower predictions of water elevations at the ONS site. The NRC staff determines that the licensee estimated flood level is within the range of the uncertainties observed and the estimated flood level could be considered reasonable.

It is important to realize that there is an inherent conservative assumption with regard to the initial reservoir elevation for the sunny-day failure analysis. In addition, (b)(7)(F) [redacted] (b)(7)(F) [redacted] (b)(7)(F) [redacted] In addition, staff notes (based on Brown and Burgess, 2004) that there are no documented sunny-day failures of modern rockfill dams, and that rockfill embankments have a much higher resiliency to flow than more traditional earthfill structures.

3.4.2.5 Conclusion

The NRC staff confirmed the licensee's conclusion that the reevaluated hazard from flooding from dam failures is not bounded by the CDB flood hazard. Therefore, the NRC staff expects that the licensee will submit a focused evaluation confirming the capability of flood protection and available physical margin or an integrated assessment consistent with the process and guidance discussed in COMSECY-15-0019 (NRC, 2015b).

3.5 Storm Surge

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for storm surge does not inundate the plant site, but did not report a probable maximum flood elevation. This flood-causing mechanism is not discussed in the licensee's current design-basis.

The licensee reported in its revised FHRR (Duke, 2015b) that storm surge has been reviewed and is not considered a credible mechanism to produce maximum water levels at the site. The review was performed as part of the FERC-required evaluation of the Keowee and Jocassee Developments. The licensee did not perform a separate surge flooding analysis, but in its revised FHRR the licensee provided wind-driven wave runup and 2-yr wind velocity results for the Keowee Main Dam and Jocassee Dam under fair weather and PMF conditions. The wind velocity results were obtained using methods described in ANSI/ANS 2.8 (ANSI/ANS, 1992). The resulting wave heights are all less than the design normal freeboard at each dam, which is 15 ft (4.6 m), and this freeboard will prevent overtopping by wind-driven waves. The licensee also stated that since the completion of construction, the Keowee Reservoir has not exceeded a water elevation of 800.0 ft (243.84 m) MSL, thus leaving at all times a freeboard of at least 15 ft (4.6 m) below the minimum dam-top elevation of 815 ft (248.4 m) MSL. The Jocassee Reservoir has not exceeded a water elevation of 1,110 ft (338.3 m) MSL, thus leaving at all times a freeboard of at least 15 ft (4.6 m) below the minimum dam-top elevation of 1,125 ft (342.9 m) MSL. The licensee determined that storm surge will not affect the site and is bounded by the current design-basis flood elevation of 796 ft (242.6 m) MSL.

The NRC staff reviewed the licensee's analysis of flooding hazard from storm surge, including associated effects. The staff reviewed the wind-driven wave run-up results and the 2-yr wind velocity results provided in the revised FHRR (Duke, 2015b). The results were reviewed for both the Keowee Main Dam and Jocassee Dam under fair weather and PMF conditions.

The NRC staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from storm surge is bounded by the current design basis flood hazard elevation of 796 ft (242.6 m) MSL.

3.6 Seiche

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for seiche does not inundate the plant site, but did not report a probable maximum flood elevation. This flood-causing mechanism is not discussed in the licensee's current design-basis.

The licensee reported in its revised FHRR (Duke, 2015b) that seiche flooding has been reviewed and not considered a credible mechanism to produce maximum water levels at the site. The licensee reported that on the basis of the topography and geology around the reservoirs, a seiche caused by an earthquake or landslide is not considered credible.

The NRC staff reviewed the licensee's analysis of flooding hazard from seiche, including associated effects, and confirmed the licensee's conclusion that the reevaluated hazard for flooding from seiche is bounded by the current design basis flood hazard elevation of 796 ft (242.6 m) MSL.

3.7 Tsunami

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for tsunami does not inundate the plant site, but did not report a probable maximum flood elevation. This flood-causing mechanism is not discussed in the licensee's current design-basis.

The licensee stated in its revised FHRR (Dukey, 2015b) that ONS is not located on an open ocean coast or large body of water, and concluded that tsunami-induced flooding will not produce the maximum water level at the site.

The NRC staff reviewed the licensee's analysis of flooding hazard from tsunami, including associated effects. The NRC staff observed that the site is located inland, and there are no credible tsunami-generating sources on record.

The NRC staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from tsunami is bounded by the current design basis flood hazard elevation of 796 ft (242.6 m) MSL.

3.8 Ice-Induced Flooding

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for ice-induced flooding does not inundate the plant site, but did not report a probable maximum flood elevation. This flood-causing mechanism is not discussed in the licensee's current design basis. It is not described in the ONS UFSAR (Duke, 2012b)

The licensee reviewed historical temperature records from the South Carolina State Climatology Office for the period of 1951 to 2011. The licensee also augmented the analysis with onsite temperature data collected at ONS. The licensee reported that there has been no significant surface ice formation on Jocassee Reservoir or Keowee Reservoir. Additionally, the licensee searched the USACE' *Ice Jam Database* and found that there has not been a recorded event of ice jams in the upper reach of the Savannah River, which begins in Hartwell Reservoir, with water temperatures consistently remaining above freezing. The licensee's analysis indicated that ONS has short mild winters and long humid summers. Also, the local climatology data for Pickens County, South Carolina averaged over a period of 30 yr resulted in a mean temperature of 59.7 °F.

The NRC staff reviewed the licensee's analysis of the flooding hazard from ice-induced flooding. The NRC staff confirmed the licensee's conclusion that the reevaluated hazard for ice-induced flooding of the site is bounded by the current design basis flood hazard elevation of 796 ft (242.6 m) MSL.

3.9 Channel Migrations or Diversions

The licensee reported in its revised FHRR (Duke, 2015b) that the reevaluated hazard, including associated effects, for channel migrations or diversions does not inundate the plant site, but did not report a probable maximum flood elevation. This flood-causing mechanism is not discussed in the licensee's current design basis.

The licensee reported in its Revised FHRR (Duke, 2015b) that, due to the location of ONS on the banks of Keowee Reservoir and the upstream topography of the reservoir, channel diversion is not a credible flooding event.

The NRC staff reviewed the licensee's analysis of flooding hazard from channel migrations or diversions, including associated effects. The NRC staff noted that streams near Keowee Reservoir are incised into bedrock to depths of several hundred feet, thus severely limiting channel migration and diversion (see for example the Old Pickens, SC topographic quadrangle map (USGS, 2014)).

The NRC staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from channel migrations or diversions is bounded by the current design-basis flood hazard elevation.

4.0 REEVALUATED FLOOD HEIGHT, EVENT DURATION, AND ASSOCIATED EFFECTS FOR HAZARDS NOT BOUNDED BY THE CDB

The NRC staff confirmed that the reevaluated flood hazard results for local intense precipitation, rivers and streams, and dam failure flood-causing mechanisms are not bounded by the Oconee Nuclear Station, Units 1, 2, and 3 current design-basis hazard. Therefore, the NRC staff anticipates that the licensee will perform an additional assessment (focused evaluation and/or revised integrated assessment) of plant response.

Consistent with the process and guidance discussed in COMSECY-15-0019 (NRC, 2015a), staff expects the licensee will submit a focused evaluation for LIP and associated site drainage. For the rivers and streams and dam failure flood-causing mechanisms, staff expects the licensee will submit a focused evaluation confirming the capability of flood protection and available physical margin or a revised integrated assessment consistent with the process and guidance discussed in COMSECY-15-0019 (NRC, 2015a).

The licensee provided reevaluated flood-event duration parameters associated with mechanisms that trigger additional assessments of plant response in a letter dated June 13, 2014 (Duke, 2014c), and was revised and incorporated into the revised FHRR (Duke, 2015b). Table 4.0-1 presents flood event duration parameters for the reevaluated flood-causing mechanisms.

By a letter dated April 25, 2015 (Duke, 2014a) the licensee provided flood height and associated effects as defined in Section 9 of JLD-ISG-2012-05 (NRC, 2012d) for mechanisms that trigger an Integrated Assessment. This response was revised and incorporated into the Revised FHRR (Duke, 2015b). The reevaluated flood heights for flood-causing mechanisms are summarized in Table 4.0-2 and associated effects inputs required for the additional assessments of plant response are summarized in Table 4.0-3. Wind effects associated with Keowee Reservoir flooding is addressed in Sections 3.3 and 3.4. The NRC staff concluded that other associated effects, including the effects of hydrodynamic loading, erosion and sedimentation, and groundwater ingress are not applicable to this site, and therefore, do not need to be evaluated.

Based upon the preceding analysis, the NRC staff confirmed that the reevaluated flood hazard information defined in the sections above is appropriate input to the additional assessments of plant response as described in the 50.54(f) letter and COMSECY-15-0019, "Mitigating Strategies and Flooding Hazard Reevaluation Action Plan" (NRC, 2015a).

5.0 CONCLUSION

The NRC staff has reviewed the information provided for the reevaluated flood-causing mechanisms for Oconee Nuclear Station Units 1, 2 and 3. Based on its review, the NRC staff concludes that 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 that the licensee responded appropriately to Enclosure 2, Required Response 2, of the 50.54(f) letter, dated March 12, 2012. In reaching this determination, the NRC staff confirmed the licensee's conclusions that: (1) the reevaluated flood hazard results for local intense precipitation, streams and river flooding, and upstream dam failure flooding are not bounded by the CDB flood hazard, (2) an additional assessment or revised integrated assessment of plant response will be performed for the local intense precipitation, streams and river flooding and the dam failure flood-causing mechanisms, and (3) the reevaluated flood-causing mechanism information is appropriate input to the additional assessment of plant response as described in the 50.54(f) letter and COMSECY-15-0019, "Mitigating Strategies and Flooding Hazard Reevaluation Action Plan" (NRC, 2015a, Enclosure 1). The NRC staff has no additional information needs at this time with respect to the ONS, Units 1, 2, and 3 FHRR.

6.0 REFERENCES

Notes: (1) ADAMS Accession Nos. refers to documents available through NRC's Agencywide Documents Access and Management System (ADAMS). Publicly-available ADAMS documents may be accessed through http://www.nrc.gov/reading-rm/adams.html. (2) "n.d." indicates no date is available or relevant, for example for sources that are updated by parts. "n.d.-a", "n.d.-b" indicate multiple undated references from the same source.

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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

SRP is the Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition (NRC, 2007a)

JLD-ISG-2012-06 is the "Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment" (NRC, 2013a)

JLD-ISFG-2013-01 is the "Guidance for Assessment of Flooding Hazards Due to Dam Failure" (NRC, 2013b)

Table 3.1-1. Summary of Controlling Flood-Causing Mechanisms

Reevaluated Flood-Causing Mechanisms and Associated Effects that May Exceed the Powerblock Elevation (796 ft (242.6 m) MSL)	ELEVATION (MSL)
Local Intense Precipitation and Associated Drainage	800.4 ft (243.959 m)
Streams and Rivers (Flooding in Reservoirs - Keowee) (with wind wave)	812.2 ft (247.56 m)
Failure of Dams and Onsite Water Control/Storage Structures – Jocassee Dam Sunny-Day Failure	[redacted] (b)(7)(F)

NOTE: Flood Height and Associated Effects are as defined in JLD-ISG-2012-05 (NRC, 2012d).

Table 3.1-2. Current Design-Basis Flood Hazards for Use in the MSA[1]

Mechanism	Stillwater Elevation	Waves/ Runup	Design Basis Hazard Elevation	Reference
Local Intense Precipitation	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Streams and Rivers Flooding in Reservoirs – Keowee Reservoir (not calculated at Powerblock)	808.0 ft (246.28 m) MSL	Not Applicable	808.0 ft (246.28 m) MSL	Audit Summary Report (NRC, 2015c)
Failure of Dams and Onsite Water Control/Storage Structures	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Storm Surge	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Seiche	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Tsunami	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Ice-Induced	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)
Channel Migrations/Diversions	Not Included in DB	Not Included in DB	Not Included in DB	Audit Summary Report (NRC, 2015c)

[1] Nominal site grade is at elevation 796 ft (242.6 m) MSL

Table 3.4-1. Licensee's Analysis of Sensitivity of Peak Outflow from Jocassee Dam to Percentage Adjustments in VTG Breach Parameters

Breach Width	80% Breach Formation Pattern	70% Breach Formation Pattern	60% Breach Formation Pattern
	Outflow, ft³/s (m³/s)		
100%	(b)(7)(F)	(b)(7)(F)	(b)(7)(F)
80%			
60%			

Source: Audit Summary Report (NRC, 2015c)
The VTG model was described by Von Thun and Gillette (Von Thun and Gillette, 1990).

Table 3.4-2. Summary of One-Dimensional (HEC-RAS) Sensitivity Analyses by Staff for Keowee Dam Overtopping

Run[1]	1-D Model Sensitivity Description	Primary Sensitivity Evaluated	Key Results: Jocassee Dam – Peak Outflow, ft^3/s (m^3/s)	Key Results: Keowee Dam[2] – Peak Headwater Elevation, ft (m) MSL	Key Results: Keowee Dam[2] – Peak Tailwater Elevation, ft (m) MSL
ONS	**Licensee's Results[3]**	**None**	(b)(7)(F)	(b)(7)(F)	(b)(7)(F)
Run H-1	Confirmatory Case	None			
Run H-2	Jocassee bottom breach width of 100%	Jocassee Breach			
Run H-3	Jocassee side slopes of (b)(7)(F)	Jocassee Breach			
Run H-4	Jocassee bottom breach width of 100% & side slopes of (b)(7)(F)	Jocassee Breach			
Run H-5	Jocassee 80% breach progression	Jocassee Breach			
Run H-6	Jocassee bottom breach width of 100% & 80% breach progression	Jocassee Breach			
Run H-7	Jocassee piping coefficient of 0.6	Jocassee Flow Coefficient			
Run H-8	Jocassee weir coefficient of 2.6	Jocassee Flow Coefficient			
Run H-9	Jocassee piping coefficient to 0.6 and weir coefficient to 2.6	Jocassee Flow Coefficient			
Run H-10	No downstream breach of Keowee Dam	Keowee Breach			
Run H-11	Jocassee bottom breach width of 100% & no downstream breach of Keowee Dam	Keowee Breach			

Run[1]	1-D Model Sensitivity Description	Primary Sensitivity Evaluated	Key Results		
			Jocassee Dam	Keowee Dam[2]	
			Peak Outflow, ft^3/s (m^3/s)	Peak Headwater Elevation, ft (m) MSL	Peak Tailwater Elevation, ft (m) MSL
Run H-12	(b)(7)(F) of 815 ft (248.4 m) MSL	Keowee Breach	(b)(7)(F)	(b)(7)(F)	(b)(7)(F)
Run H-13	Increase Keowee Dam and West Saddle Dam times-to-failure by 33%	Keowee Breach			
Run H-14	Decrease Keowee Dam and West Saddle Dam times-to-failure by 33%	Keowee Breach			
Run H-15	Increase Keowee Dam time-to-failure by 33%	Keowee Breach			
Run H-16	Increase Manning's roughness coefficient[4] by 5%	Manning's n			
Run H-17	Increase Manning's roughness coefficient[4] by 10%	Manning's n			
Run H-18	Increase Manning's roughness coefficient[4] by 15%	Manning's n			

)(7)(F)

1 Run names highlighted in bold text are those presented in the associated Figure 3.4-4.

2 Headwater elevation is reservoir elevation measured at the Keowee Dam inline structure. Tailwater elevation measured at inline structure represents water elevation on the ONS site.

3 ONS maximum elevations are reported by the licensee (Duke, 2015b, Table 9). Licensee's reported analysis uses breach width of of VTG, side slopes, and a breach progression for Jocassee Dam.

(7)(F)

4 Manning's n values are increased equally in all locations for runs H-16 to H-18.

)(7)(F)

Table 3.4-3Sensitivity of Two-Dimensional (TUFLOW FV) Sensitivity Analyses Performed by Staff for Jocassee Dam Breach

Run	2-D Model Sensitivity Description	Primary Sensitivity Evaluated	Maximum Water Surface Elevation at SSF, ft (m) MSL
ONS	**Licensee's Results**	**None**	(b)(7)(F)
Run T-1	Confirmatory Case	None	
Run T-2	Jocassee 80% breach progression	Jocassee Breach	
Run T-3	Jocassee bottom breach width of 100% & 80% breach progression	Jocassee Breach	
Run T-4	Jocassee piping coefficient to 0.6 and weir coefficient to 2.6	Jocassee Flow Coefficient	
Run T-5	Increase Keowee Dam and West Saddle Dam times-to-failure by 33%	Keowee Breach	
Run T-6	Change eddy viscosity value to 0.8	2-D Model: Eddy Viscosity	
Run T-7	Increase TUFLOW mesh refinement in the connecting canal area	2-D Model: Mesh Size	
Run T-8	Adding representation of bridge piers within the connecting canal	2-D Model: Canal Obstruction	

Table 4.0-1. Flood-Event Duration for Reevaluated Flood-Causing Mechanisms

Flood-Causing Mechanism	Site Preparation for Flood Event	Period of Site Inundation	Recession of Water from Site	Reference
Local Intense Precipitation and Associated Drainage	24-h based on capability to forecast atmospheric moisture that delivers 18.95 in (48.13 cm) of rain	Water will accumulate in and around the powerblock early in the 72-h duration rainfall	1-h after the rainfall has subsided	Response to RAI-14 (Duke, 2014c) and the FHRR
Streams and Rivers - Keowee Reservoir		Event does not inundate nuclear plant site due to 815 ft (248.4 m) MSL saddle dam separating ONS site from reservoir		Revised FHRR, Sections 3.2
Failure of Dams and Onsite Water Control/Storage Structures – Jocassee Dam Sunny-Day Dam Failure	(b)(7) (F)	(b)(7) (F)	(b)(7) (F)	Response to RAI-14 and Revised FHRR

NOTE: Definitions of flood-event durations are illustrated in Figure 2.2-1.

Table 4.0-2. Reevaluated Flood Hazards for Flood Causing Mechanisms for Use in the MSA[1]

Mechanism	Stillwater Elevation	Associated Effects	Reevaluated Flood Hazard	Reference
Local Intense Precipitation	800.4 ft (243.96 m) MSL	Minimal	800.4 ft (243.96 m) MSL	Revised FHRR, Sections 3.1
Streams and Rivers Flooding in Reservoirs, Keowee	808.9 ft (246.55 m) MSL	3.3 ft (1.02 m) (wind wave effects)	812.2 ft (247.56 m) MSL[2]	Revised FHRR, Sections 3.2
Failure of Dams and Onsite Water Control/Storage Structures Jocassee Dam Breach - Sunny-Day Failure	(b)(7)(F)	Not Applicable	(b)(7)(F)	Revised FHRR Sections 3.3

[1] Reported values are rounded to the nearest one-tenth of a foot.

[2] Water surface elevation in Keowee Reservoir. ONS is not inundated because it is separated from Keowee Reservoir by the Keowee Dam, which has a crest elevation of 815 ft (248.4 m) MSL.

[3] The Jocassee sunny-day dam breach flood level of (b)(7)(F) MSL was confirmed by staff to be a reasonable estimate at the ONS site; however, this value is not the most conservative estimate for a sunny-day dam breach flood. The staff acknowledges that the value is within the uncertainty range discussed in Section 3.4.

[4] Water level at ONS resulting from sunny-day failure of Jocassee Dam. Water level in Keowee Reservoir is (b)(7)(F) MSL.

Table 4.0-3. Integrated Assessment Associated Effects Inputs

	Flooding Mechanism		
Associated Effects Factor	**Local Intense Precipitation**	**Failure of Dams and Onsite Water Control/Storage Structures** – Jocassee Dam Sunny-Day Dam Failure	**Reference**
Hydrodynamic loading at plant grade	Licensee reported maximum flow velocities are generally below 1 ft/sec with exceedance in constricted flow areas such as areas between buildings. The maximum velocity reported throughout the powerblock is 1.3 ft/s (0.40 m/s). Because of depth and velocity factors, the hydrodynamic loads are expected to be minimal.	(b)(7)(F)	Response to RAI-15 in Revisions 1 FHRR (Duke, 2015b) and sensitivity analysis by staff
Debris loading at plant grade	Generation of debris is minimal because the potential sources within the protected area are paved and surrounded by vehicle barrier systems (VBS). Debris effect is negligible.	(b)(7)(F)	Response to RAI-15 in Revisions 1 FHRR (Duke, 2015b)
Sediment loading at plant grade	Generation of sediment is minimal.	(b)(7)(F)	Response to RAI-15 in Revisions 1 FHRR (Duke, 2015b)
Concurrent conditions, including adverse weather	Squall lines, thunderstorms with capping inversion, and mesoscale convective systems are typically accompanied by hail, strong winds, and even tornadoes	(b)(7)(F)	Response to RAI-15 in Revisions 1 FHRR (Duke, 2015b)

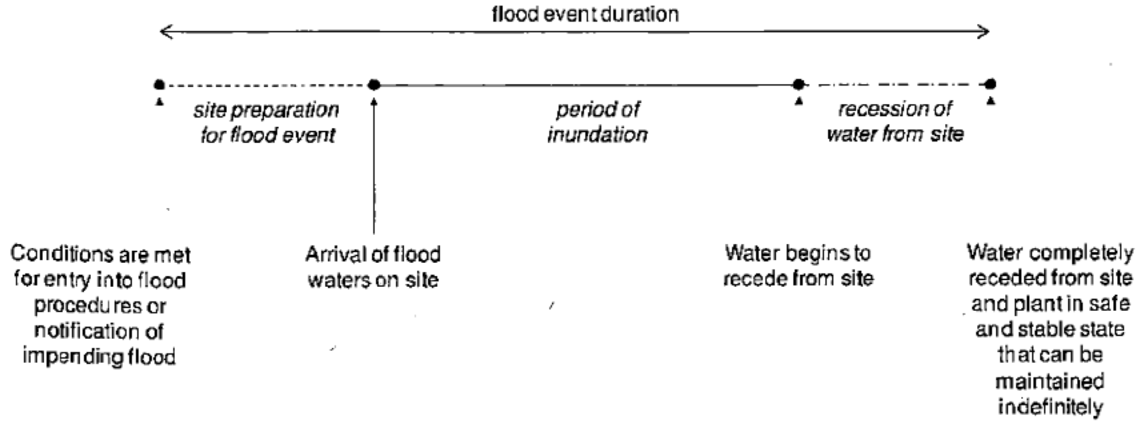
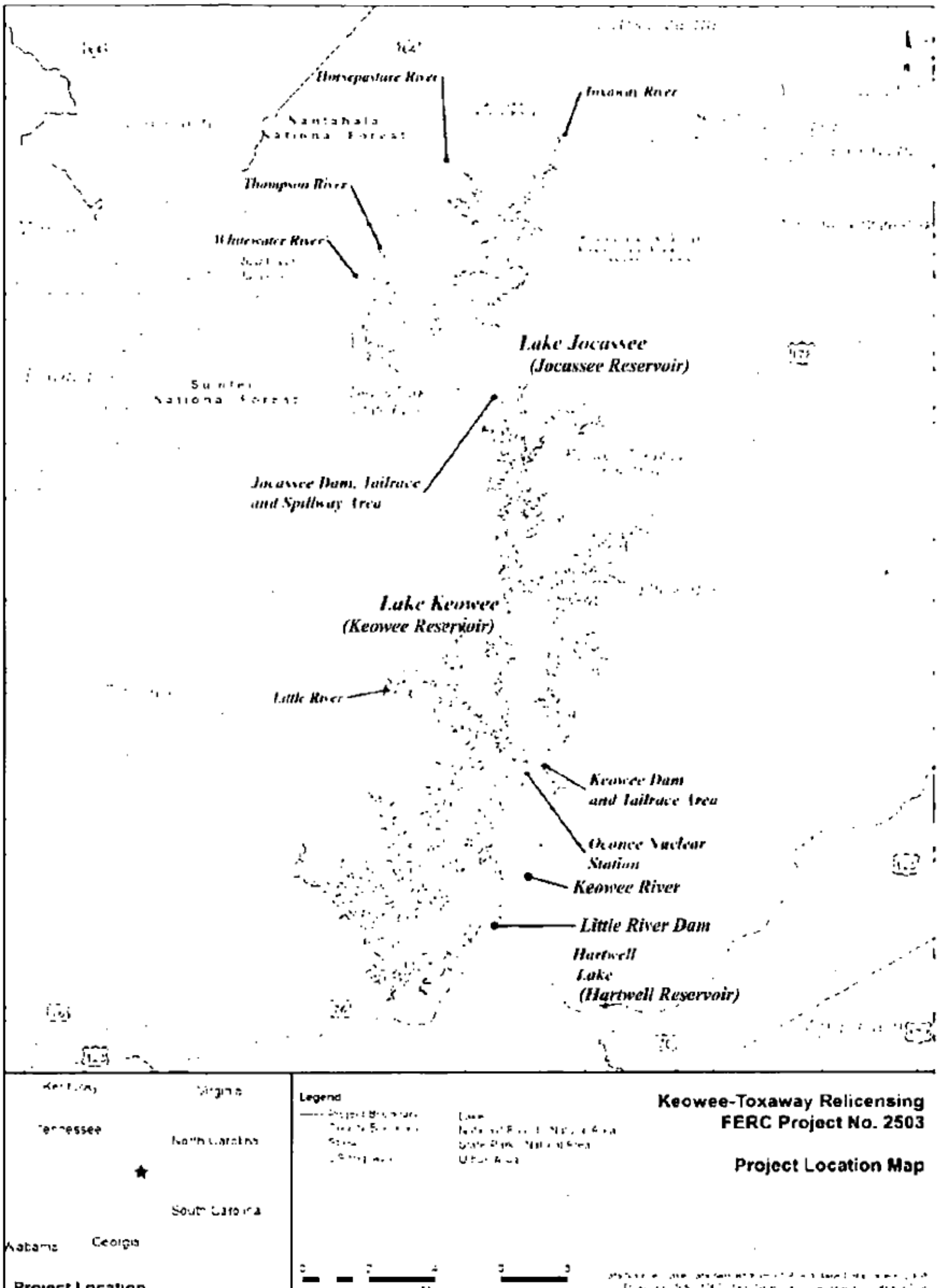


Figure 2.2-1. Flood-Event Duration

Figure 3.1-1. Oconee Nuclear Station (ONS) and Nearby Features



Source: Keowee-Toxaway Hydroelectric Project Application for New License (Duke Energy, 2014d, Figure A1-1)

Figure 3.1-2. Principal Dams and Reservoirs

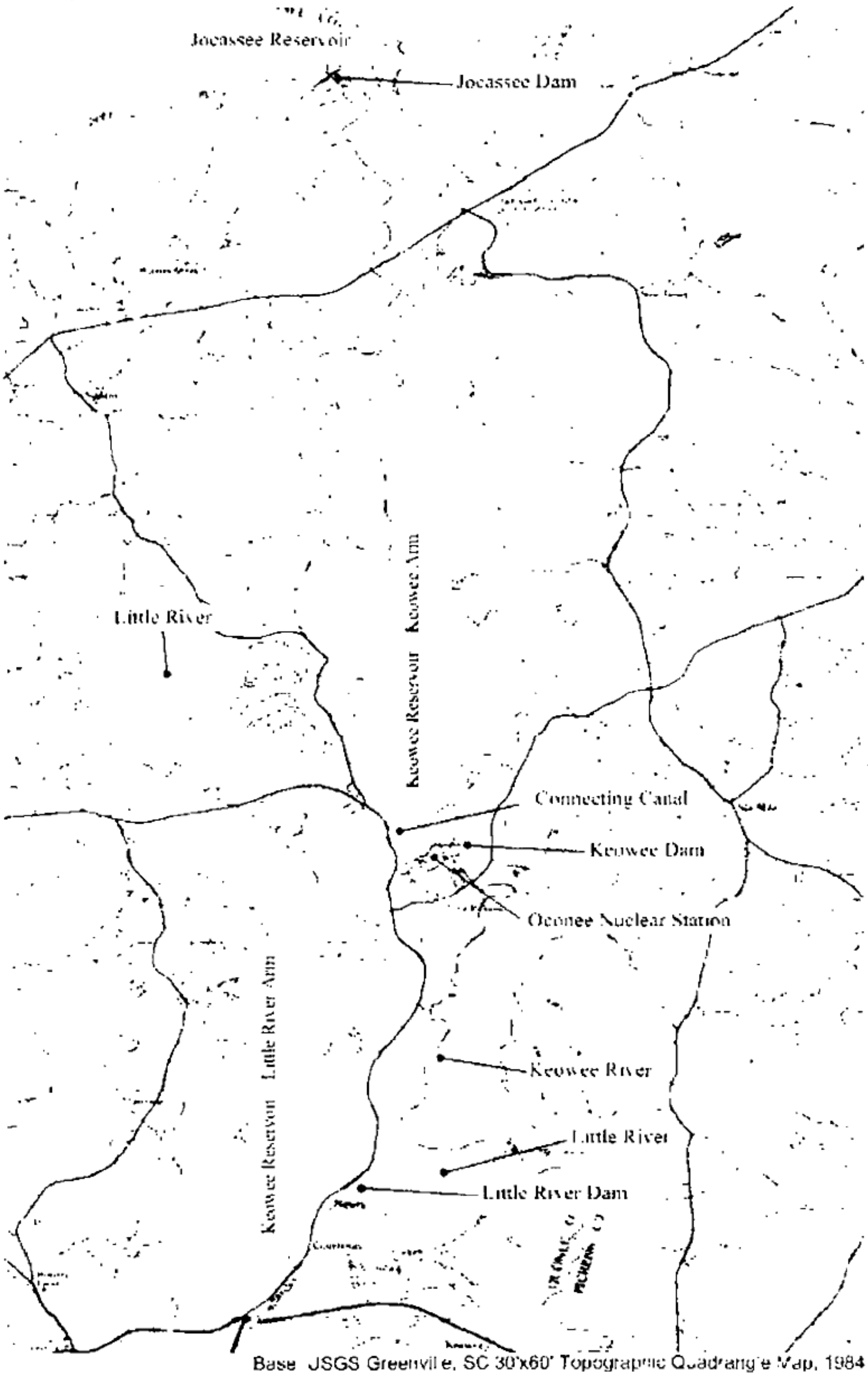


Figure 3.1-3. Keowee Reservoir

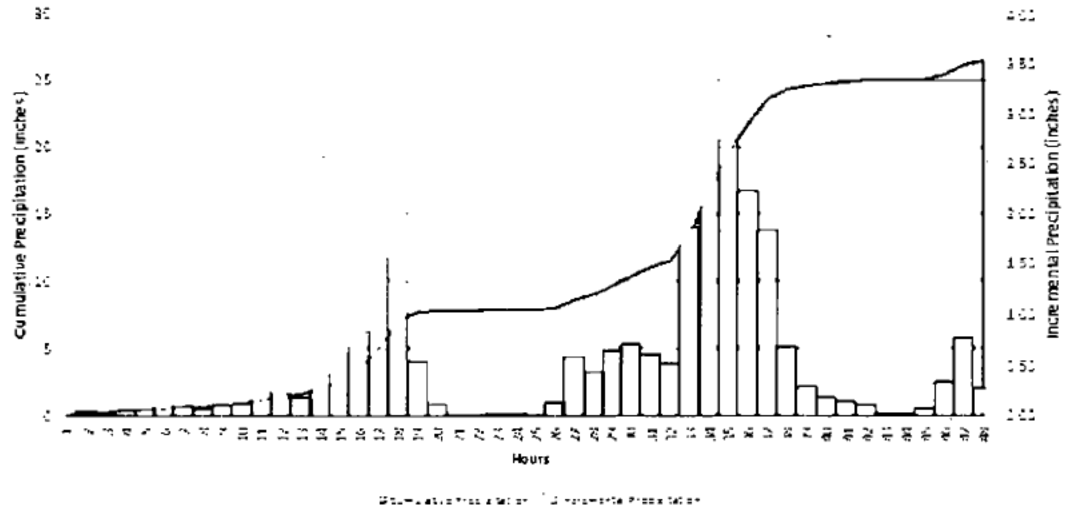


Figure 3.2-1. PMP precipitation used for original design basis - Cumulative and incremental precipitation during the 48-hur PMP for a rainfall depth of 26.6 in (67.6 cm) (adapted from the Revised FHRR (Duke, 2015b, Figure A-1)).

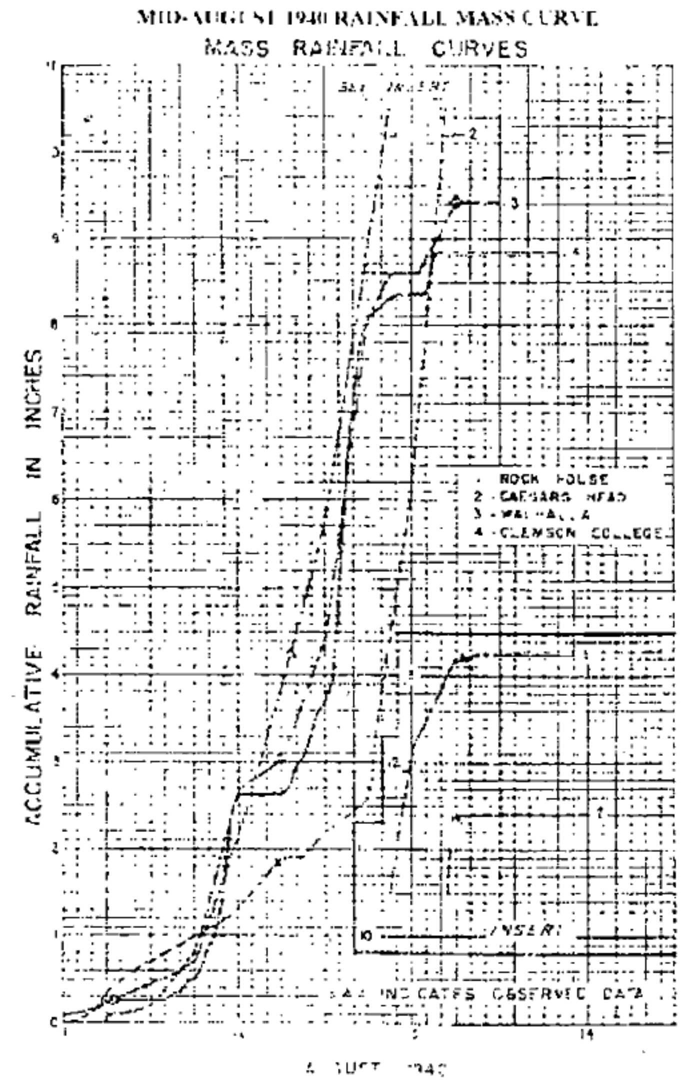


Figure 3.2-2. August 1940 Rainfall Mass Curves
(adapted from the Revised FHRR (Duke, 2015b, Figure A-2))

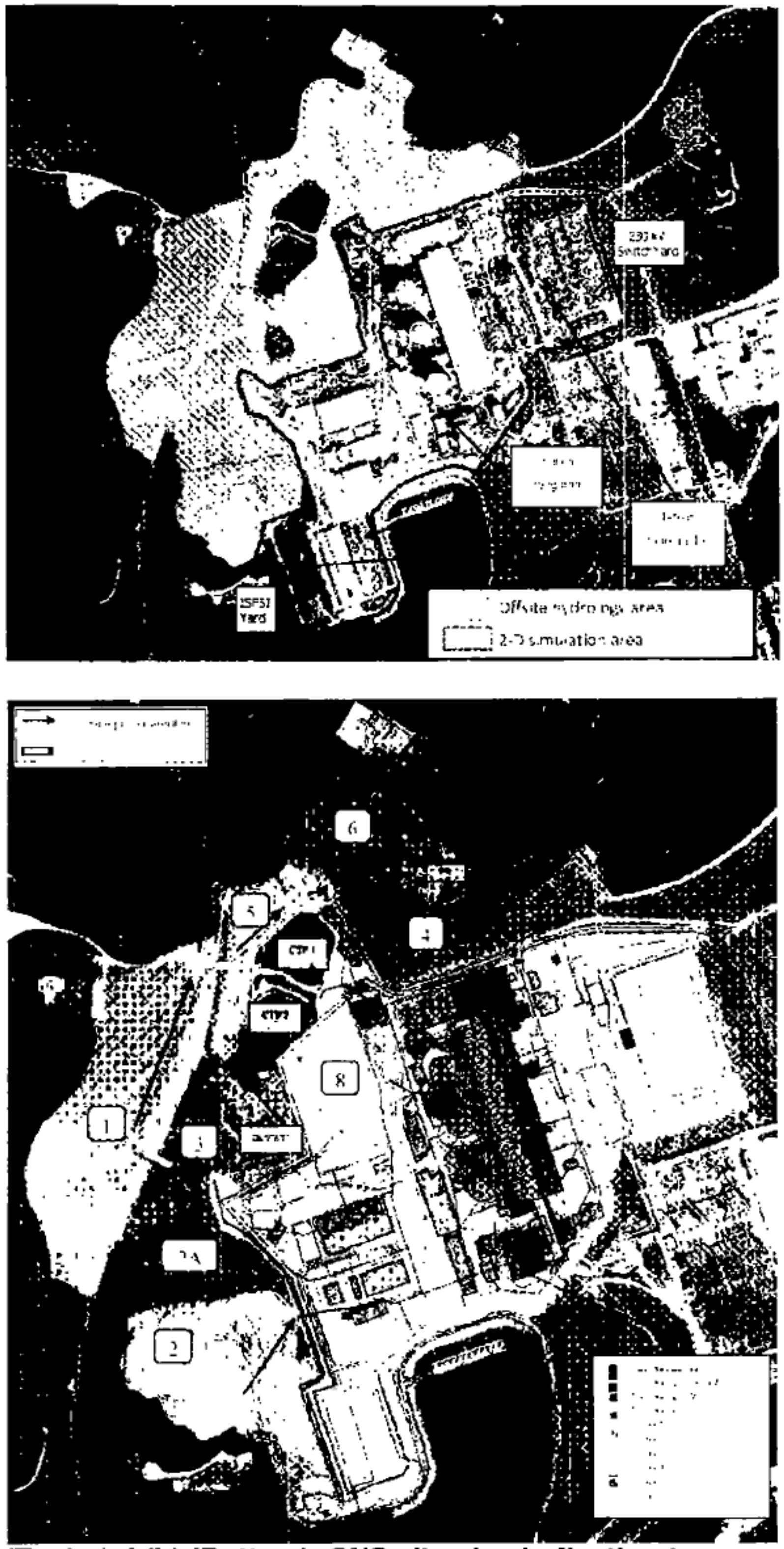


Figure 3.2-3. (a) (Top) and (b) (Bottom): ONS site plan indicating two separate areas used in assessing "offsite" and "onsite" runoff. Sources: (a) (adapted from the Revised FHRR (Duke, 2015b, Figure A-5-A)) and individual subbasins of runoff and direction (b) (adapted from the Revised FHRR (Duke, 2015b, Figure A-7-A))

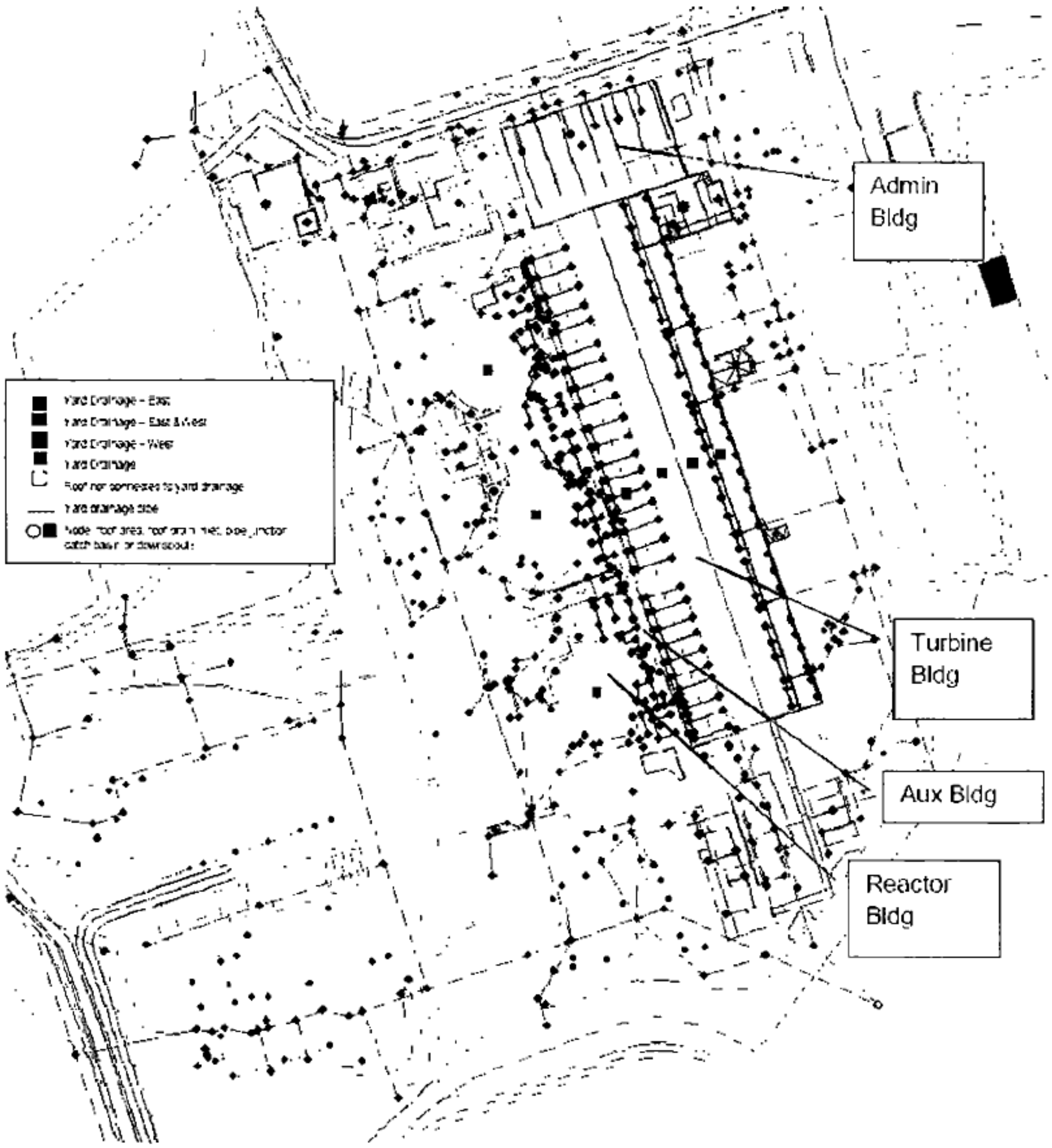


Figure 3.2-4. ONS Roof Drainage-To-Yard Drainage Connection Plan (adapted from the Revised FHRR (Duke, 2015b, Figure A-8))

Figure 3.2-5. ONS Roof Drainage-To-Yard and Yard Drainage Connection Plan (adapted from the Revised FHRR (Duke, 2015b, Figure A-9))

Figure 3.2-6. ONS Yard Catch Basins
(adapted from the Revised FHRR (Duke, 2015b, Figure A-10-A))

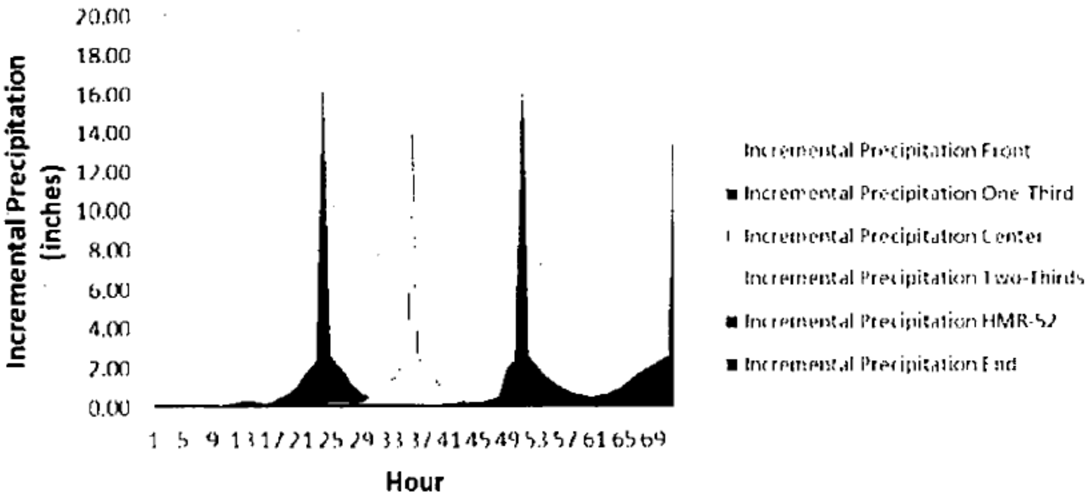


Figure 3.2-7. Incremental precipitation during the 72-h PMP for six different temporal distributions (adapted from the Revised FHRR (Duke, 2015b, Figure 3))

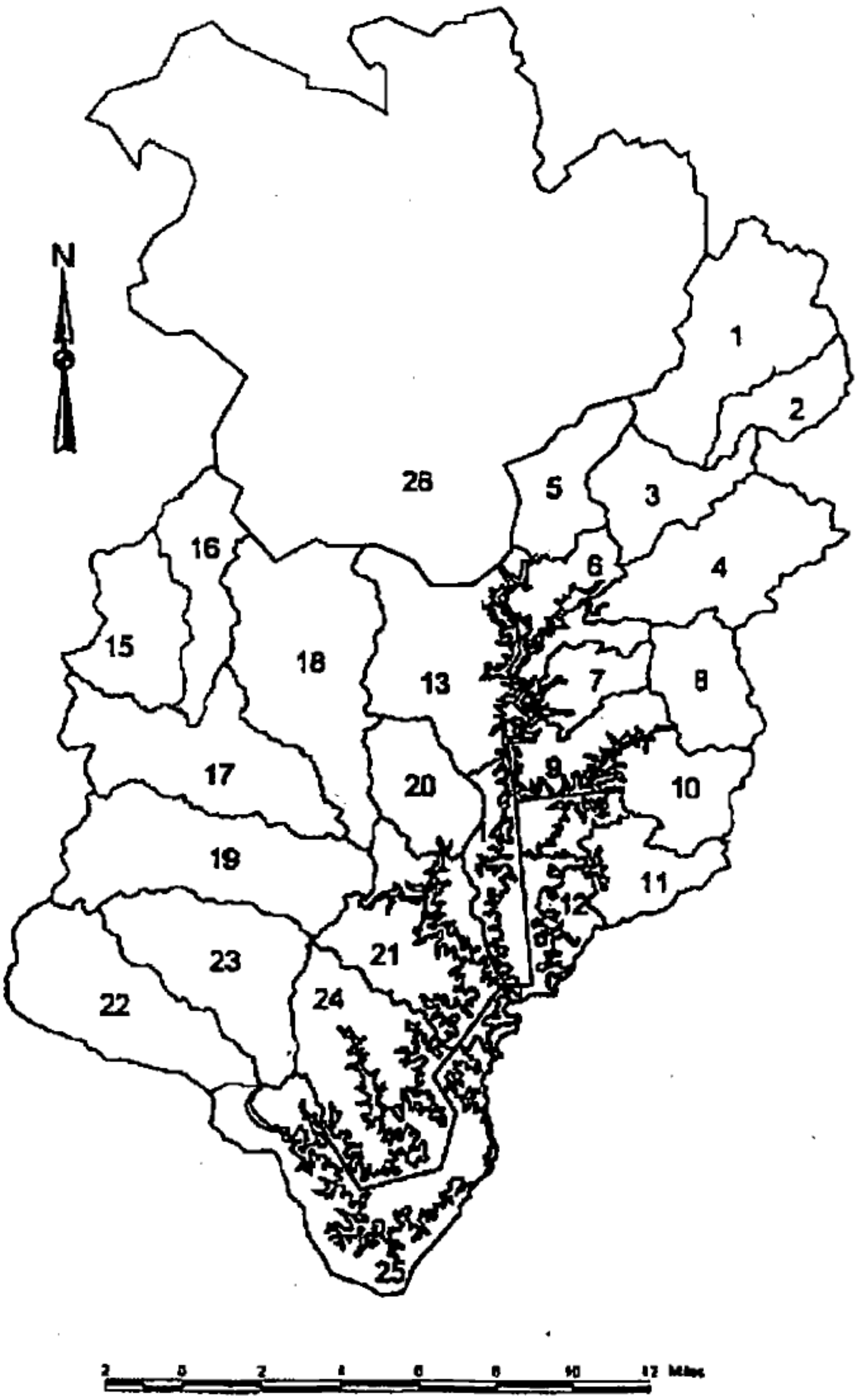


Figure 3.3-1. Keowee Watershed and Subbasins
Source: Revised FHRR (Duke, 2015b, Figure 6)

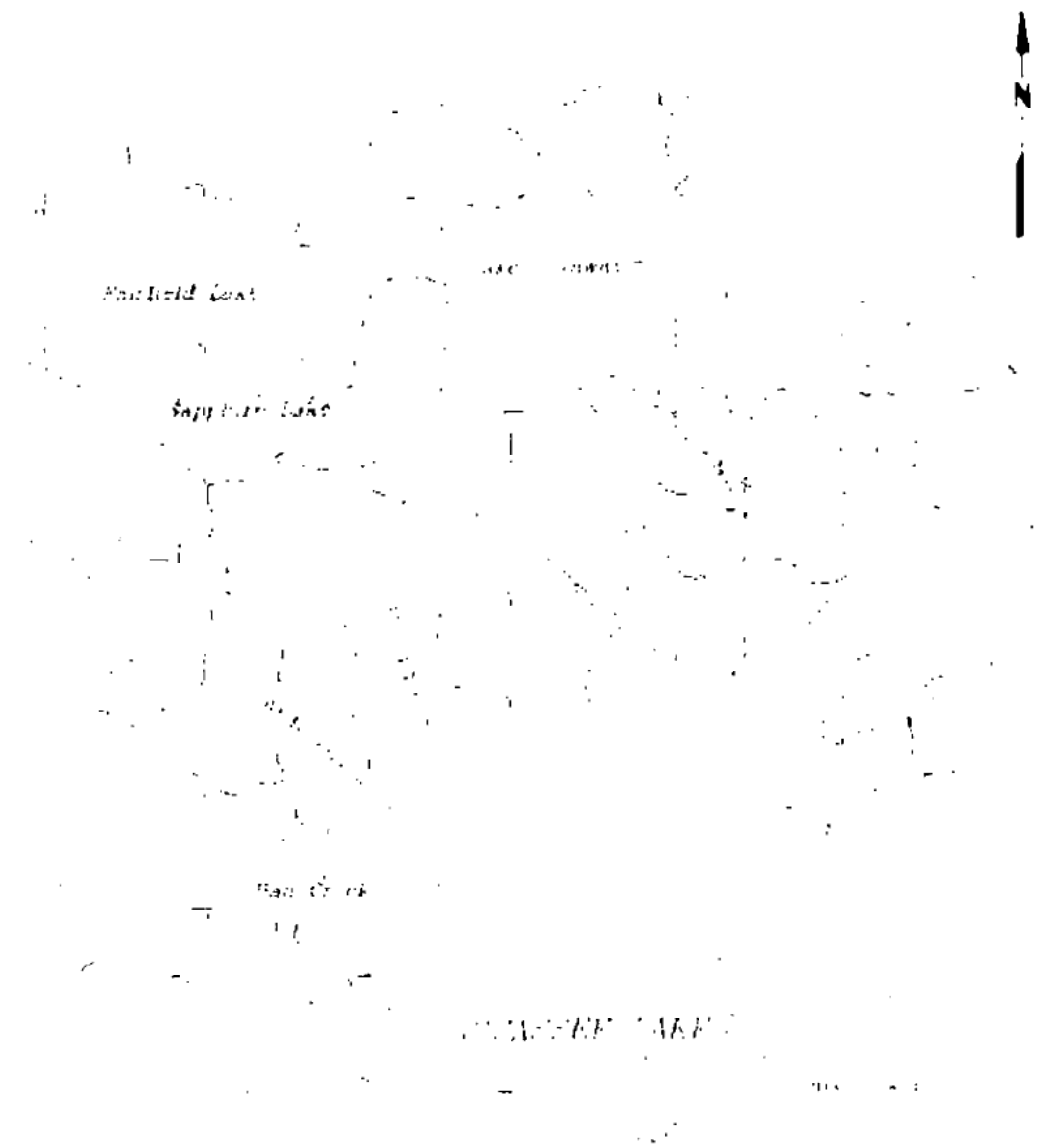


Figure 3.3-2.Jocassee Watershed and Subbasins
Source: Revised FHRR (Duke, 2015b, Figure 10)

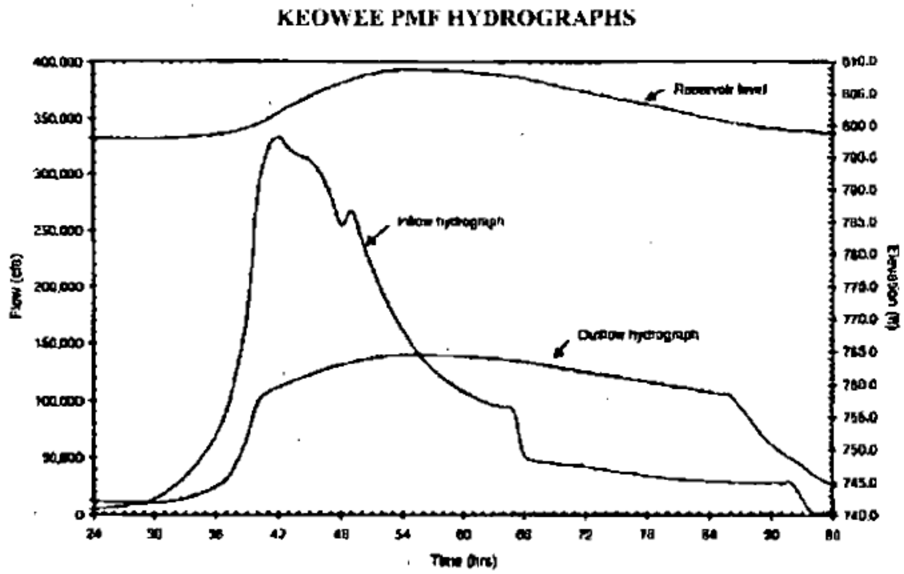


Figure 3.3-3. Keowee PMF Hydrograph
(adapted from the Revised FHRR (Duke, 2015b, Figure 8))

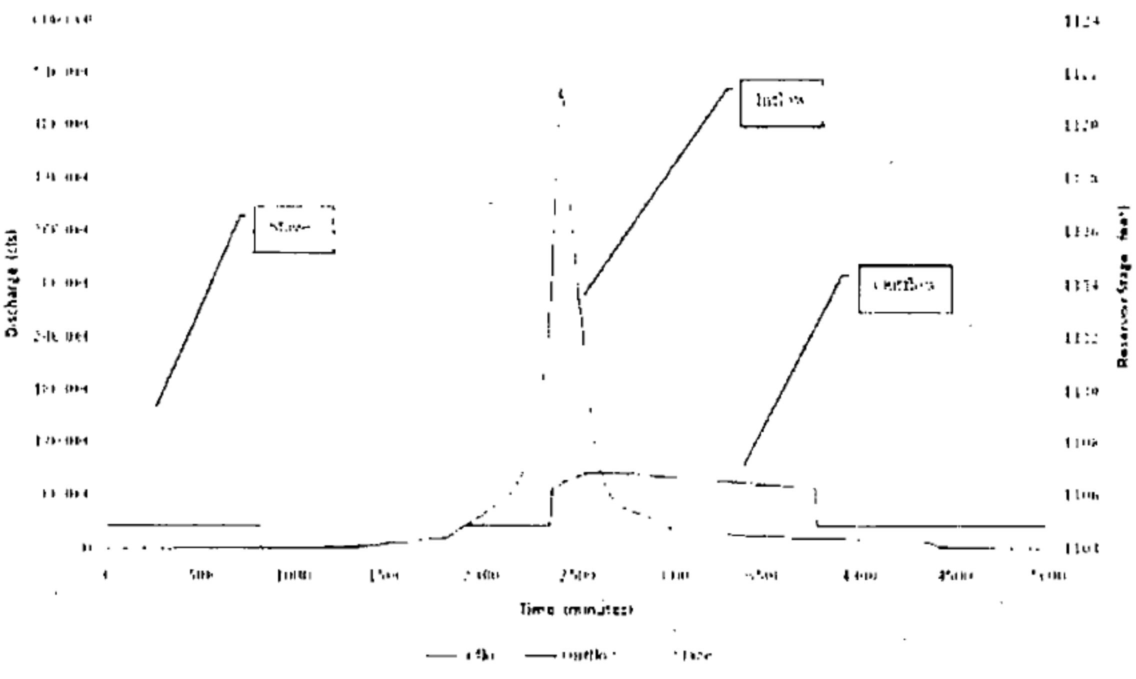


Figure 3.4-1. Jocassee PMF Hydrograph
(adapted from the Revised FHRR (Duke, 2015b, Figure 11))

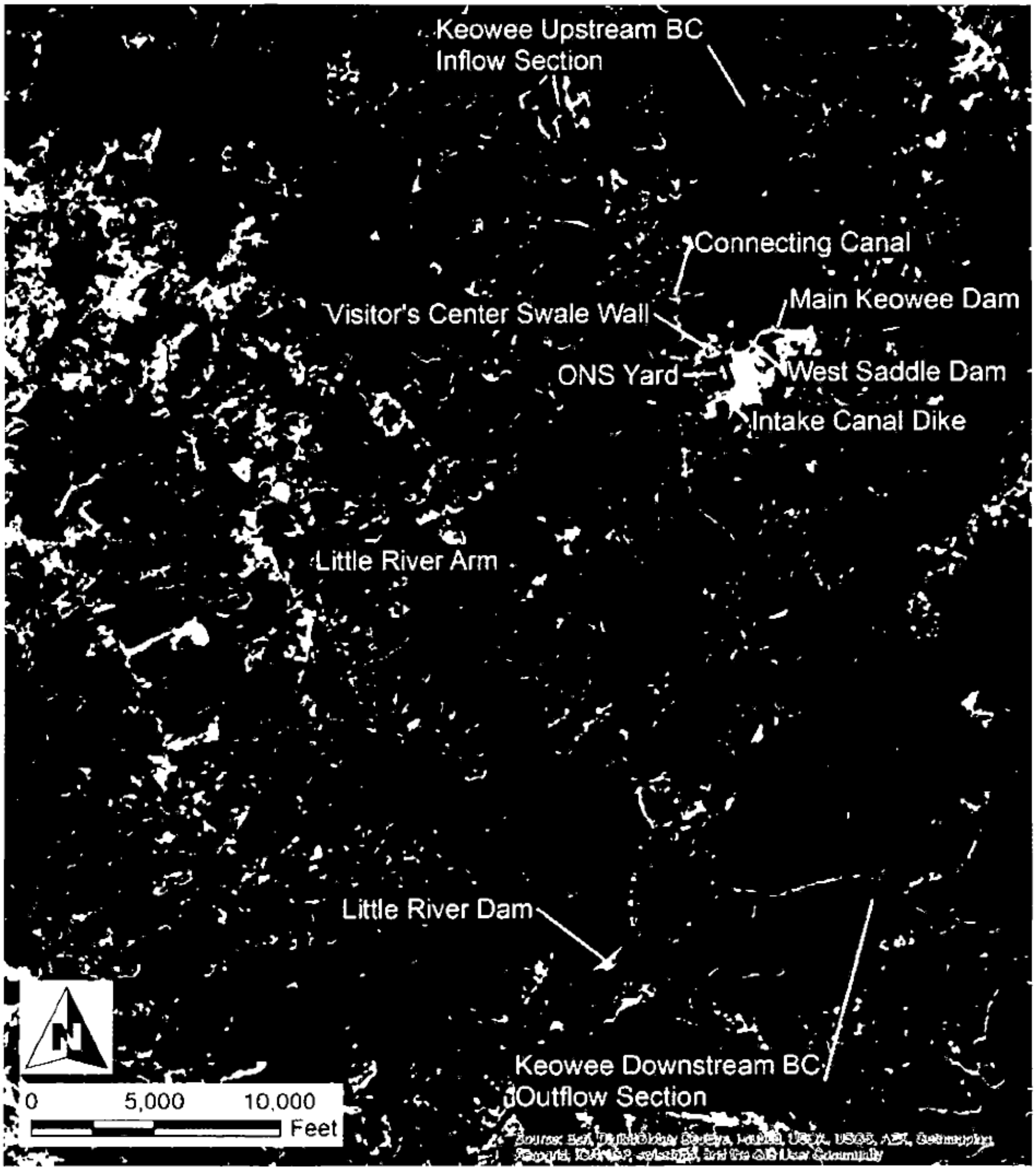


Figure 3.4-2. Jocassee-Keowee Hydraulic 2D Model (TUFLOW FV) Model Extents and Location of Boundary Conditions

(b)(7)(F)

(b)(7)(F)

ENCLOSURE 2:

ADDENDUM TO OCONEE NUCLEAR STATION UNITS 1, 2, AND 3 FHRR STAFF ASSESSMENT

Addendum to Oconee Nuclear Station Units 1, 2, and 3 FHRR Staff Assessment

Comparison of the 2010 and 2015 Postulated Jocassee Dam Failure and Downstream Flooding Evaluations by Duke Energy Carolinas

Introduction

Two separate flooding hazard evaluations related to postulated upstream dam failure of the Jocassee Dam have been performed by Duke Energy Carolinas, LLC (Duke, the licensee) for the Oconee Nuclear Station Units 1, 2, and 3 (ONS 1, 2, and 3). These two evaluations were performed as part of separate U.S. Nuclear Regulatory Commission (NRC) information requests.

The first assessment performed by the licensee was the result of a letter issued by the NRC on August 15, 2008, pursuant to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, Section 50.54(f) (Agencywide Documents Access and Management System (ADAMS) Accession No. ML081640244). This letter requested additional information regarding external flooding of the ONS, including the consequences of a potential Jocassee Dam failure. In response to this letter and subsequent interactions with the NRC, the licensee submitted their final evaluation results of a postulated sunny-day failure of Jocassee Dam by letter dated August 2, 2010 (ADAMS Accession No. ML102170006). The licensee subsequently committed to several actions to address the flood hazard at the ONS, which are documented in a Confirmatory Action Letter (CAL) issued by the NRC on June 22, 2010 (ADAMS Accession No. ML101730329).

The second evaluation performed by the licensee was the result of a 10 CFR 50.54(f) letter issued to all operating reactor licensees on March 12, 2012 (ADAMS Accession No. ML12053A340), as one part of the NRC's response to lessons-learned following events at the Fukushima Dai-ichi nuclear station in Japan (ADAMS Accession No. ML11186A950). The letter requested licensees to evaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits (ESPs) and combined license (COL) reviews. In light of this second 10 CFR 50.54(f) request and the previous 2010 licensee evaluation, the NRC requested, by letter dated September 20, 2012 (ADAMS Accession No. ML12219A163), that the licensee clarify its timeline to implement the CAL actions. The licensee responded by letter dated December 14, 2012 (ADAMS Accession No. ML12354A217), discussing the timeline and proposing that the submission and subsequent NRC approval of the licensee's Fukushima Flood Hazard Report submitted in response to the second 10 CFR 50.54(f) request would supersede the January 28, 2011, staff assessment (ADAMS Accession No. ML110280153) of the licensee's 2010 evaluation. The licensee submitted its Flood Hazard Reevaluation Report (FHRR) for ONS on March 12, 2013 (ADAMS Accession No. ML13079A228).

In a Request for Additional Information dated September 15, 2014 (ADAMS Accession No. ML14258B222), the NRC staff, with input from other Federal agencies, cited several deficiencies in the licensee's FHRR and requested Duke to submit a revised FHRR, using appropriate methods and guidance for the analysis of dam failures. In response, the licensee modified the breach methodology used to analyze the sunny-day failure for Jocassee Dam and revised the flood-hazard modeling results at the ONS. The licensee documented these changes in Revision 1 of its FHRR, and submitted the revised FHRR by letter dated March 6, 2015 (ADAMS Accession No. ML15072A099).

The purpose of this addendum to the FHRR staff assessment is to compare the reevaluation methodologies contained in the licensee's 2010 and 2015 submittals. A similar comparison by the licensee was submitted by letter dated January 8, 2016 (ADAMS Accession No. ML16019A122). A complete description of the NRC staff's review and conclusions regarding the 2010 Jocassee Dam failure evaluation is contained in the staff assessment dated January 28, 2011 (ADAMS Accession No. ML110280153). Likewise, the NRC staff's complete review and conclusions regarding the 2015 Jocassee Dam failure evaluation (i.e., Revision 1 of the FHRR) is contained in Section 3.4 of the associated staff assessment.

2010 Licensee Evaluation: Staff's Review, Key Results and Conclusions

The NRC staff evaluated the dam failure and downstream flooding results provided by Duke in a letter dated August 2, 2010 (ADAMS Accession No. ML102170006). This evaluation focused on computing peak water surface elevations at the ONS due to a random (a.k.a. sunny day) failure of the Jocassee Dam. The unmitigated Case 2 dam breach parameters were ultimately selected by the licensee as the preferred scenario results. Conservatism in the selected scenario provided the NRC staff with reasonable assurance of safety for the overall flooding scenario at the site.

The purpose of the NRC staff's review was to confirm that the licensee provided adequate justification that selected dam breach parameters and the resulting downstream flooding evaluation satisfied the NRC's 2008 10 CFR 50.54(f) letter and the terms of the June 22, 2010 CAL. The licensee considered a number of dam breach parameters and ultimately selected the unmitigated Case 2 scenario. The NRC staff assessment included a review of the dam failure methodologies and breach parameter selections, and confirmed the licensee's computed values. The NRC staff determined that the selected scenario produced flooding results at the ONS that: (a) were conservative and (b) provided reasonable assurance that flood inundation levels at the site would not exceed water surface elevations predicted by the licensee. Evaluation results predicted a maximum water surface elevation of [redacted] mean sea level (MSL), (b)(7)(F) approximately (b)(7)(F) [redacted] the ONS nominal site grade (elevation 796 ft), at the standby shutdown facility (SSF).

The NRC staff identified the following key conservatisms in the evaluation:

- Based on a comparison with the values determined from empirical models, the staff determined that the Hydrologic Engineering Center - River Analysis System (HEC-RAS) model results for peak outflow are conservative.
- The biotite gneiss which comprises the bedrock type at the base of the dam would be extremely resistant to erosion, so conservatism was noted in the licensee's evaluation when determining the breach size.
- The average width of the assumed dam breach is one of the key breach parameters. The licensee's selected value (approximately [redacted]) is larger than the average width (b)(7)(F) estimated using Froehlich's 2008 methods (approximately [redacted]). (b)(7)(F)
- The Jocassee Dam breach hypothetical failure time of 2.8 hours is short for a dam with the quality of construction, basal rock type, and degree of monitoring. Consequently, the staff noted that conservatism existed in the licensee's estimation of the maximum breach size.

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As documented in the 2011 staff assessment, the NRC staff determined that the licensee had provided the documentation necessary to compute a conservative, bounding, estimation of a postulated sunny-day Jocassee Dam failure and subsequent downstream flooding at the ONS.

2015 Licensee Evaluation: Staff's Review, Key Results and Conclusions

The purpose of the NRC staff's FHRR review was to determine whether the licensee had met the requirements set in the March 12, 2012, 50.54(f) letter. It should be noted that there are differences in regulatory guidance applied by the staff as part of its review of the 2010 and 2015 licensee evaluations. First, based on the requirements of Near-Term Task Force Recommendation 2.1, the licensee assessed all flood-causing mechanisms at the ONS site using present-day methodologies and regulatory guidance as applied for ESP and COL sites for the FHRR evaluation. In particular, the licensee's 2015 submittal (ADAMS Accession No ML15072A099) considered the potential for hydrologic, seismic, and sunny-day failures of Jocassee dam, whereas the licensee's submittal in 2010 only considered the potential for sunny-day failure[6]. Second, the NRC staff used the Japan Lessons-Learned Directorate (JLD) Interim Staff Guidance (ISG) JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards due to Dam Failure" (ADAMS Accession No. ML13151A153) as part of its review of the 2015 licensee evaluation. This document clarified methodologies acceptable to NRC staff regarding dam breach formulation, and was not available for staff use at the time staff reviewed the 2010 licensee submittal.

As documented in the FHRR staff assessment, the NRC staff concluded the licensee demonstrated that:

(1) Seismically-induced failure of the Jocassee Dam is not a reasonable mode of failure based on current information, present-day methodologies and regulatory guidance.

(2) Overtopping-induced failure of the Jocassee Dam is not reasonable model of failure based on current information, present-day methodologies and regulatory guidance.

(3) Sunny-day failure of Jocassee Dam was considered an unlikely, although reasonable, failure mode. The licensee postulated the most likely location of the breach is a section of piping in the (b)(7)(F) The NRC staff reviewed the licensee's assumptions regarding the dam breach, and concluded the licensee appropriately followed the guidance in JLD-ISG-2013-01.

The NRC staff performed independent confirmatory analyses as part of its FHRR review to determine the range of uncertainties inherent in the postulated dam breach evaluation of the Jocassee Dam. These results are documented in staff assessment Section 3.4. The NRC staff assessment concluded that the licensee conducted the hazard evaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews in an acceptable manner. Evaluation results predicted a maximum water surface elevation of (b)(7)(F), approximately (b)(7)(F) nominal site grade, at ONS due to a sunny-day failure of Jocassee Dam.

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[6] In 2010.the staff considered hydrologic and seismic failure when originally approaching the issue of reviewing the Jocassee Dam failure question, but determined those failures to have minimal impact on the site. So while the submittal and SA are silent on those two failure mechanisms they were considered during the review process of the 2010 submittal.

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Submittal and review of the FHRR satisfies the first part of the March 12, 2012, 50.54(f) letter, and establishes the appropriate flood hazards to assess the adequacy of the licensee's mitigating strategies developed in response to Order EA-12-049 (i.e., defines the mitigating strategies flood hazard information described in guidance documents currently being finalized by the industry and NRC staff). The second part of the 50.54(f) letter requests licensees whose reevaluated flood hazards exceed their current design-basis to complete an additional assessment. The FHRR for ONS identifies three flood mechanisms (local intense precipitation, stream and rivers, and dam failure) that exceed their current design bases. As a result, the second assessment discussed in the 50.54(f) letter is requested from the licensee, and the NRC staff expects the licensee will submit a focused evaluation confirming the capability of flood protection and available physical margin or a revised integrated assessment consistent with the process and guidance discussed in COMSECY-15-0019 (ADAMS Accession No. ML15153A105) for all reevaluated mechanisms not bounded by the current design basis.

Summary of the 2010 and 2015 Licensee Evaluations

The selection and application of breach parameters and hydraulic models used in both evaluations produced variations in the timing and maximum water height at the ONS. A reasonable basis exists for several alternative analyses, all of which can generate higher or lower predications of maximum water surface elevations. For both the 2010 and 2015 evaluations, the NRC staff determined that the licensee appropriately followed engineering and regulatory guidance to compute flood levels at the site within the range of the inherent uncertainties. The 2010 licensee evaluation reflects a bounding analysis and is based on several conservative assumptions including: (1) conservative breach size selection given the dam's construction and bedrock type at the dam site; (2) a hypothetical time to reach a peak outflow of 5.44 Mcfs in 2.8 hours, based on the quality of construction, basal rock type, and degree of monitoring of the Jocassee dam. The 2015 evaluation reflects a reasonable analysis that removes some conservatism in the 2010 analysis, and is consistent with recent Commission direction regarding licensees' flood hazard reevaluation in response to the 50.54(f) letter. Therefore, the NRC staff concluded that the licensee's estimated flood levels at the ONS are considered reasonable and satisfy the information requests for each letter. Further, the staff concludes that the revised 2015 FHRR provides an acceptable evaluation of a postulated sunny-day failure of the Jocassee Dam, and is appropriate to consider in assessing the need for specific actions included in the June 22, 2010 NRC Confirmatory Action Letter.

ENCLOSURE 2