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SOUTHERN COMPANY

March 6, 2014

Docket Nos.: 50-321
50-366

NL-14-0326

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D. C. 20555-0001

Edwin I. Hatch Nuclear Plant - Units 1 and 2
Recommendation 2.1 Flood Hazard Reevaluation Report
Requested by NRC Letter dated March 12, 2012

Reference: 1. NRC Letter, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Daiichi Accident*; dated March 12, 2012. (ML 12053A340)

2. NRC Letter, *Prioritization of Response Due Dates for Request for Information to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Daiichi Accident*; dated May 11, 2012. (ML 12097A509)

Ladies and Gentlemen:

On March 12, 2012, the U. S. Nuclear Regulatory Commission (NRC) issued Reference 1 to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 of Reference 1 requests licensees to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the site, probable maximum flood on stream and rivers, storm surges, seiches, tsunami, and dam failures. The NRC requested information for the following purposes:

- To gather information with respect to Near-Term Task Force Recommendation 2.1, as amended by staff requirements memoranda associated with SECY-11-0124 and SECY-11-0137, and the Consolidated Appropriations Act, for 2012 (*Pub Law* 112-74), Section 402, to reevaluate seismic and flooding hazards at operating reactor sites.
- To collect information to facilitate NRC's determination if there is a need to update the design basis and systems, structures, and components important to safety to protect against the updated hazards at operating reactor sites.
- To collect information to address Generic Issue 204 regarding flooding of nuclear power plant sites following upstream dam failures.

Enclosure 2 of Reference 1 requires each addressee to submit the Flood Hazard Reevaluation Report within one to three years from the date of the information request. Enclosure 1 of Reference 2 assigned the Edwin I. Hatch Nuclear Plant (HNP) Units 1 and 2 as a Category 2 site, meaning HNP is required to submit its flood hazard reevaluation report within two years. The Flood Hazard Reevaluation Report for HNP Units 1 and 2 is provided in the enclosure to this letter, as required by Reference 1.

This completes the flooding hazard reevaluation for HNP Units 1 and 2 as required by Reference 1. An integrated assessment as described in Enclosure 2 of Reference 1 will be completed in 2016.

This letter contains no new regulatory commitments. If you have any questions, please contact John Giddens at (205)-992-7924.

Sincerely,

C. R Pierce

C. R. Pierce
Regulatory Affairs Director

CRP/JMG/RCW

Sworn to and subscribed before me this 6th *day of* March *, 2014.*

Laura L. Crumpton
Notary Public

My commission expires: 10/8/2017

Enclosure: Flood Hazard Reevaluation Report

cc: Southern Nuclear Operating Company
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Edwin I. Hatch Nuclear Plant – Units 1 and 2
Recommendation 2.1 Flood Hazard Reevaluation Report
Requested by NRC Letter dated March 12, 2012

Enclosure

Flood Hazard Reevaluation Report

FLOOD HAZARD REEVALUATION REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING SEVERE ACCIDENT MANAGEMENT FOR FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING REEVALUATION

for the

EDWIN I. HATCH NUCLEAR PLANT
11036 HATCH PARKWAY N., BAXLEY, GEORGIA 31513
Renewed Facility Operating License No. DPR-57 & NPF-5
NRC Docket No. 50-321 & 50-366

SOUTHERN COMPANY

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Version 1.0

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Corporate Acceptance:	Terry Mitchell	SNC	Terry Mitchell	3/5/14

1. Contents

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2. LIST OF ACRONYMS

ºC	degree(s) Celsius (or Centigrade)
ºF	degree(s) Fahrenheit
ac	acre(s)
ANS	American Nuclear Society
ANSI	American National Standards Institute
CFR	Code of Federal Regulations
cfs	cubic (foot)feet per second
CLB	current licensing basis
D-A-D	depth-area-duration (curves)
DEM	Digital Elevation Model
EM	Engineer Manual
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
fps	feet per second
FSAR	Final Safety Analysis Report
FTP	file transfer protocol
GIS	Geographic Information System
GHCN	Global Historical Climatology Network
GHCND	Global Historical Climatology Network Data
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	hierarchical hazard assessment
HMR	Hydrometeorological Report
HPCI	High Pressure Coolant Injection
hr	hour(s)
HUC	Hydrologic Unit Code
in	inch
km	kilometer(s)
km^2	square kilometer(s)
LandSat	Land-Use Satellite
LIP	Local Intense Precipitation
LPCI	Low Pressure Coolant Injection
m	meter(s)
mi^2	square mile(s)
mi	mile(s)
min	minute(s)
mm	millimeter(s)
mph	miles per hour
MSL	mean sea level
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center

NED	National Elevation Dataset
NGVD29	National Geodetic Vertical Datum of 1929
NID	National Inventory of Dams
NHD	National Hydrography Dataset
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRC	United States Nuclear Regulatory Commission
NRCC	Northeast Regional Climate Center
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
OBE	operating basis earthquake
PMF	probable maximum flood
PMP	probable maximum precipitation
PMSP	probable maximum snowpack
PMS	probable maximum storm
PPT	Precipitation depth
PSW	Plant Service Water
RHRSW	Residual Heat Removal Service Water
SCS	Soil Conservation Service
SNC	Southern Nuclear Operating Company, Inc.
SSCs	structures, systems, and components
SSE	safe shutdown earthquake
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey
WSEL	Water Surface Elevation

3. PURPOSE

a. Background

In response to the nuclear fuel damage at the Fukushima Dai-ichi power plant due to the March 11, 2011 earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations, and to make recommendations to the Commission for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

On March 12, 2012, the NRC issued an information request pursuant to Title 10 of the Code of Federal Regulations, Section 50.54 (f) (10 CFR 50.54(f) or 50.54(f)) (Reference 2) which included six (6) enclosures:

1. [NTTF] Recommendation 2.1: Seismic
2. [NTTF] Recommendation 2.1: Flooding
3. [NTTF] Recommendation 2.3: Seismic

SNC PO: SNG31945-0001, Line 1, Item 764659
RCN: HAT0150.2

4. [NTTF] Recommendation 2.3: Flooding
5. [NTTF] Recommendation 9.3: EP
6. Licensees and Holders of Construction Permits

In Enclosure 2 of Reference 2, the NRC requested that licensees "reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits and combined license reviews."

On behalf of Southern Nuclear Operating Company, Inc. (SNC), this report provides the information requested in the March 12, 50.54(f) letter; specifically, the information listed under the 'Requested Information' section of Enclosure 2, paragraph 1 ("a" through "e") for the Edwin I. Hatch Nuclear Power Plant (Plant Hatch). The 'Requested Information' section of Enclosure 2, paragraph 2 ("a" through "d"), Integrated Assessment Report, will be addressed separately if the current design basis floods do not bound the reevaluated hazard for all flood causing mechanisms.

b. Requested Actions

Per Enclosure 2 of Reference 2,

> Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation (LIP) on the site, probable maximum flood (PMF) on streams and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESP and calculation reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staff's two phase process to implement Recommendation 2.1, and will be used to identify potential "vulnerabilities" (see definition below).
>
> For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.
>
> Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.
>
> *A definition of vulnerability in the context of [Enclosure 2] is as follows: Plant-specific vulnerabilities are those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended functions.*

c. Requested Information

Per Enclosure 2 of Reference 2, the final report should be provided documenting results, as well as pertinent site information and detailed analysis, and include the following:

a. Site information related to the flood hazard. Relevant structures, systems, and components (SSCs) important to safety and the UHS are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data includes the following:

i. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets;

ii. Current design basis flood elevations for all flood causing mechanisms;

iii. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance;

iv. Changes to the watershed and local area since license issuance;

v. Current licensing basis flood protection and pertinent flood mitigation features at the site;

vi. Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)

b. Evaluation of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Provide an analysis of each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.

c. Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 2 of the 50.54(f) letter (i.e., Recommendation 2.1 flood hazard reevaluations) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined.

d. Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary.

e. Additional actions beyond Requested Information item 1.d taken or planned to address flooding hazards, if any.

4. SITE INFORMATION

a. Detailed Site Information

Site Location

The Edwin I. Hatch Nuclear Plant (Plant Hatch) is located in the northwestern region of Appling County, approximately 11 miles north of Baxley, Georgia. The site is located on the south side of the Altamaha River, southeast of the intersection of the river with U.S. Hwy No.1 (Reference 13). The site location is shown in Figure 1.

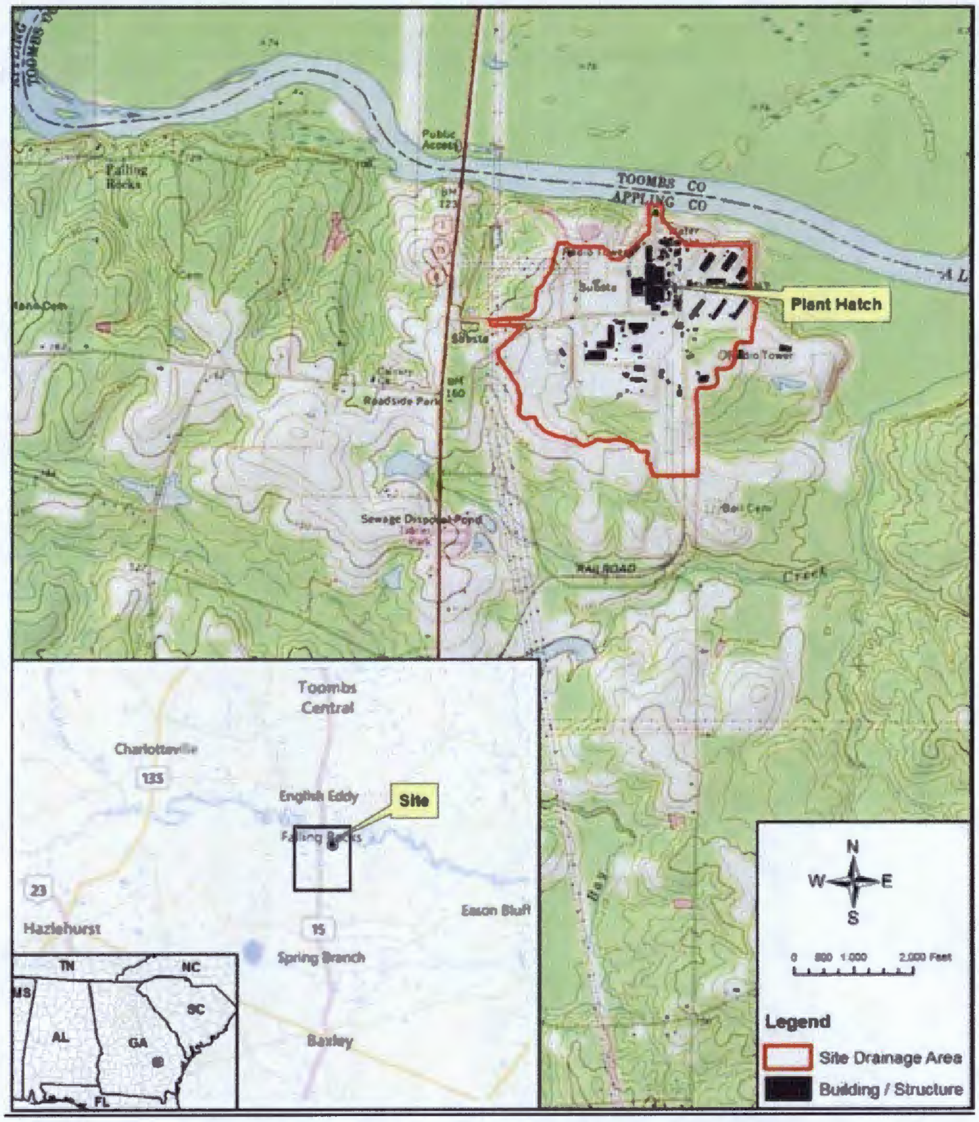

Figure 1: Site Location of Plant Hatch (Source: USGS 7.5-Minute Quadrangle Map "Baxley NE")

Site Layout and Topography

The Plant Hatch site occupies an area of approximately 2,244 acres. The power block area, which consists of the Reactor and Turbine buildings, is predominantly impervious due to buildings, asphalt/concrete roads and walkways, and gravel areas in and around the power block area, switchyards, cooling tower area, simulator/training buildings, and warehouse area. The Intake Structure is located at the northern edge of the site at river mile 116.4. The nearest USGS stream gage (02225000) is located approximately 0.5 miles upstream of Plant Hatch (Reference 13).

The natural site grade varies from approximately 175 ft* to less than 75 ft at the banks of the Altamaha River. Overall, the site slopes in the north and northeastern direction toward the Altamaha River and has a

* Unless otherwise noted, all elevations provided in this report are in National Geodetic Vertical Datum of 1929 (NGVD29)

local grade divide at the reactor building. The topography of the plant is such that the runoff is directed away from the power block by natural drainage and by a combined system of culverts, open ditches, and natural drainage channels, which drain to the river. The grade adjacent to the western side of the reactor building slopes west toward the switchyard, then slopes north toward the Altamaha River (Reference 13).

Adjacent grade elevation at the Intake Structure is 110 ft and the finished grade elevation at the control building, reactor building, turbine building and diesel building is 129.5 ft (Reference 13). The site layout and topography is shown in Figure 2 below.

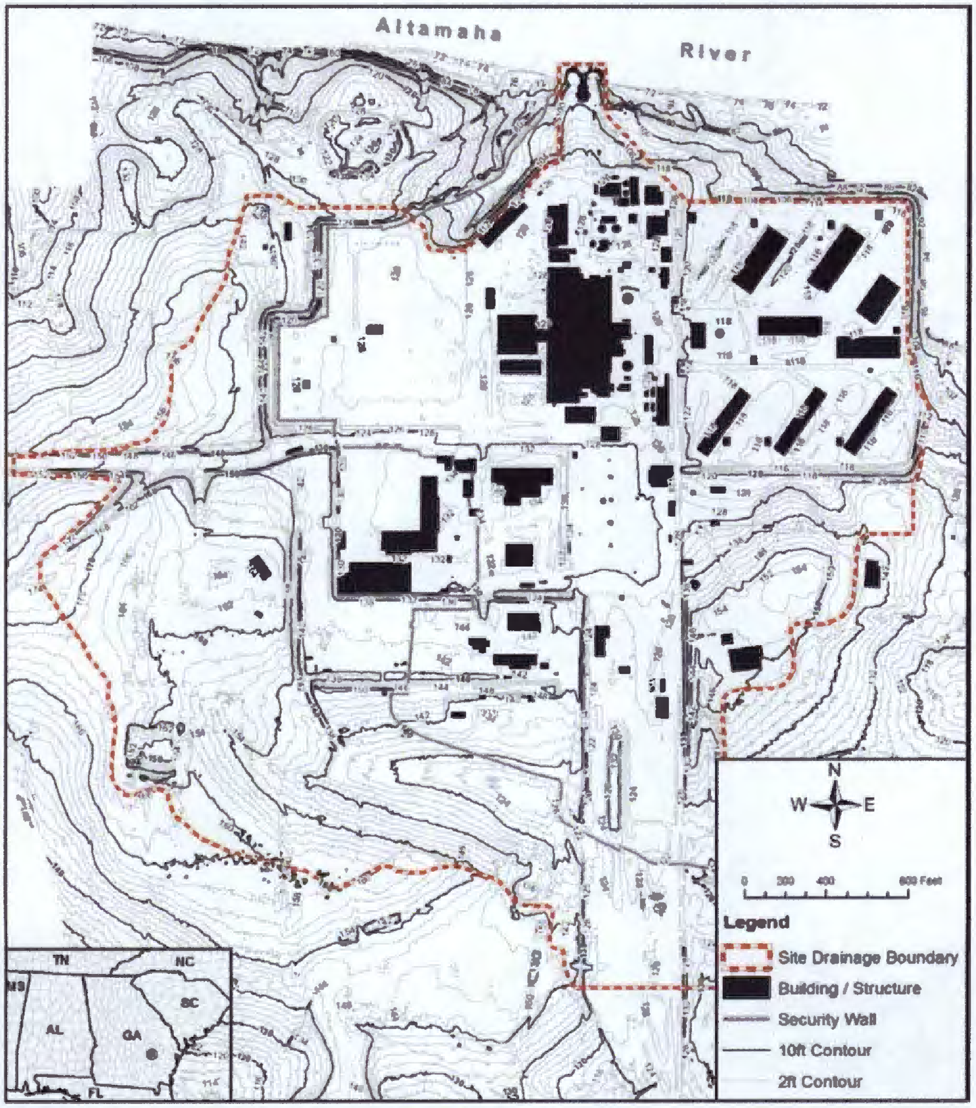

Figure 2: Site Layout and Topography of Plant Hatch

The dominant surface hydrological feature of the site's region is the Altamaha River and its contributory streams, the Oconee River and Ocmulgee River. The Oconee River and Ocmulgee River confluence to form the Altamaha River approximately 20 river miles upstream and west of the site. The contributing drainage area of the Altamaha River Watershed to Plant Hatch is approximately 11,700 square miles (Reference 13).

The average flow in the Altamaha River at the site is approximately 13,000 cfs (Reference 13). A delineation of the Altamaha River Watershed is provided in Figure 3 below.

Figure 3: Plant Hatch Watershed and Location of Major Dams

The Final Safety Analysis Report (FSAR) identifies three major dams in the Altamaha River Watershed upstream of the site. The three dams are owned and operated by Georgia Power. Sinclair Dam on the Oconee River is the largest of the dams and is located approximately 169 river miles upstream of the plant site. Wallace Dam, which is located toward the upper end of Sinclair Reservoir, is the second largest dam, followed by Lloyd Shoals Dam, which located on the Ocmulgee River approximately 268 river miles upstream of the plant site (Reference 13). In addition to the three dams identified in the FSAR, four additional dams were identified as "critical" using Method 1 from the NRC's *JLD-ISG-2013-01 Guidance for Assessment of Flooding Hazards Due to Dam Failure* (Reference 1). Table 1 provides a list of the critical dams in the Altamaha River Watershed. Further discussion on the methodology used to identify these dams as "critical" is provided in Section 4.b.

Table 1: Critical Dams Upstream of Plant Hatch

Classification	Dam Name	River	NID Height	Individual Dam Storage Volume
			ft	ac-ft
I	Sinclair Dam	Oconee River	91	490,000
II-large	Wallace Dam	Oconee River	120	400,000
I	Lake Juliette (Plant Scherer) Dam	Rum Creek	103	169,000
I	Lloyd Shoals Dam	Ocmulgee River	100	107,000
I	Lake Tobesofkee Dam	Tobesofkee Creek	54	43,054
I	Town Creek Reservoir Dam	Town Creek	110	26,855
I	Upper Towaliga Reservoir Dam	Towaliga River	45	25,700

The land use of the Altamaha River Watershed is predominantly rural with forested and agricultural land with the exception of the headwaters of the Ocmulgee River, which begins in the highly urbanized Atlanta metropolitan area. The downstream areas of the Ocmulgee River are dominated by agriculture and forested areas. The Oconee River headwaters are in a highly forested region, with an increase of agricultural land use towards the Altamaha River.

The geology of the Altamaha River Watershed plays a critical role in the watershed response due to an apparent correlation of different geologic formations in the basin. The northern area of the watershed is underlain by the Piedmont and Blue Ridge Aquifer. The center of the watershed is underlain by the Southeastern Coastal Aquifer while the southern area is underlain by the Surficial Aquifer. The general movement within the Surficial Aquifer is confined above thin clay beds, and most of the water that enters the system moves along short flow paths, and discharges back into the streams as baseflow.

SSCs Important to Safety

The following SSCs are important to safety and are located below the finished floor elevation of 130 ft:

- Low Pressure Coolant Injection (LPCI)/Decay Heat Removal pumps and valves;
- Station batteries and inverters;
- High Pressure Coolant Injection (HPCI)/makeup pumps and valves. The HPCI turbine and pump are located on floor elevation 87 ft in the reactor building;
- Plant Service Water Pumps (at Intake Structure);
- Residual Heat Removal Service Water (RHRSW) Pumps (at Intake Structure). The pumps are located inside the Intake Structure and have a centerline elevation of approximately 112 ft; and
- One 125V-DC cabinet is located in the yard.

The following SSCs are important to safety and are located at or above the finished floor elevation of 130 ft:

- Emergency electrical power diesel generators;
- Emergency electrical power distribution center;

- Vital instrument power distribution centers are located in the cabinets at elevation 130 ft of the Control Building; and
- Control rooms are located at elevation 164 ft of the Control Building.

b. Current Design Basis Flood Elevations for All Flood Causing Mechanisms

The design basis was reviewed to determine which flood-causing mechanisms are considered in the current design basis flood. Below is a summary of flood-causing mechanisms based on the design basis.

1. Local Intense Precipitation

The evaluation of LIP was based on the PMP selected from the World Record Envelop Curve (Reference 13). The equation of the curve is:

$$\text{Rainfall in inches} = 15.3 \times (\text{duration in hours})^{0.486}$$

The topography of the plant directs runoff from rainfall away from the power block area by local grade and a combination of culverts, open ditches, and natural drainage channels. However, for the LIP analysis it was assumed that the underground storm drainage system is blocked. The results of the LIP analysis showed that flooding of safety-related SSCs would not occur as a result of the LIP event (15.3 inches in one hour) (Reference 13).

2. Flooding in Streams and Rivers

The design basis was determined through detailed studies of the March 11 through 16, 1929, storm with primary center near Elba, Alabama, and the July 5 through 10, 1916 hurricane, with the center of greatest depth near Bonifay, Florida. The results of these studies determined that the 1916 storm produced the greater volume of precipitation in the Altamaha River basin above the plant site and was used to estimate the PMF (Reference 13).

The July 5 through 10, 1916 storm was positioned within meteorological limits over the basin above the plant site so as to produce the maximum volume of precipitation. The maximum position was determined by positioning the storm at several locations and finding the position for the maximum volume of precipitation by trial and error. The maximum position of the 1916 storm was with the primary storm center located approximately 8 miles northwest of Lumber City, Georgia with the storm axis rotated 20 degrees clockwise from its original bearing (Reference 13).

The transposed position of the PMP over the Altamaha River Watershed was chosen so that the amount of precipitable water is proportional to moisture charge and the storm efficiency. Due to use of the 1916 storm which resulted from a hurricane, the months considered were limited to June, July, August, September, and October. The PMP was computed for each reporting station within the basin in the transposed position, with appropriate adjustments made in the moisture charge to account for the elevation of the inflow barrier (Reference 13).

The total PMP volume over the drainage basin upstream of the plant site is computed using the Thiessen polygon methodology. The portion of the total volume within each Thiessen polygon was distributed by 6-hr periods in the same proportion as the rainfall depths at the respective precipitation station. The average total depth of storm rainfall for the 11,700 mi^2 area above the plant site was estimated to be 16.93 in (Reference 13).

The ground was assumed to be saturated at the start of the storm as the result of antecedent rainfall and, accordingly, it was assumed that there were no initial losses. A study of several historical storms, and related floods indicated that an average infiltration rate equal to 0.05 in/hr was a reasonable assumption. For each polygon the 6-hr increments of rainfall excess were obtained by deducting from the respective total precipitation volume the portions required to satisfy infiltration. The average depth of rainfall excess over the drainage basin was estimated to be 14.19 in (Reference 13).

Unit hydrographs for the plant site were developed from the floods of November and December 1948 and February and March 1961, which were found to have shorter times of concentration and higher peak discharges than other similar storms evaluated (Reference 13).

The unit hydrographs at the plant site were patterned after the unit hydrographs at the Charlotte station, located approximately 1 mile downstream of the confluence of the Oconee and Ocmulgee Rivers and approximately 19 miles upstream of the site, for the respective floods with the volumes increased in direct proportion to the drainage areas at the two locations and the peak discharges related to the square root of the drainage areas. The contributing drainage areas above the Charlotte station and the plant site were estimated to be 11,550 and 11,700 mi^2, respectively. The 6-hr unit hydrograph developed from the 1948 flood had a critical distribution and was used to obtain the probable maximum stage at the plant site (Reference 13).

The 6-hr increments of rainfall excess were applied to the adopted 6-hr unit hydrograph to obtain the hydrograph without base flow. It was assumed that the base flow would correspond to the fifth-day flow following the peak of a preceding storm runoff. The floods of record were analyzed and a base flow of 75,000 cfs was adopted. Adding the base flow to the hydrograph resulted in a peak discharge of 612,000 cfs (Reference 13).

To determine the probable maximum stillwater elevation which corresponds to the probable maximum discharge, a stage discharge curve for the site was developed by computational means from known data. However, flood stage data for this portion of the Altamaha River was very limited and the best available data was for the 1948 flood with a discharge of 79,900 cfs, corresponding to elevation 96 ft at the Charlotte gage and 83.1 ft at the Baxley gage. A straight-line hydraulic gradient was assumed between the gages at Baxley and Charlotte and projected downstream. Six valley cross-sections were surveyed in the 28-mile stretch of the river downstream of the U.S. Highway No. 1 bridge near the site. Using this stage-discharge relationship the peak discharge of 612,000 cfs was estimated to correspond to a stage of elevation 105 ft at Plant Hatch (Reference 13).

The possible maximum wave height would result from a 45-mph wind concurrent with the probable maximum discharge. The maximum fetch was estimated to be 18 miles long downstream of the site to the State Highway No. 121 bridge. The maximum sustained wind velocity with duration of more than an hour was taken as 45 mph. The significant wave height that could be developed at the site was computed to be 6.5 ft (crest to trough). This would result in a corresponding wave crest elevation of 108.3 ft at the maximum discharge. This elevation is safely below the plant grade elevation of 129 ft and below the finished floor elevation of the intake pump structure of 111.0 ft. In addition, the concrete walls of the Intake Structure are designed for impact load of 4,000 lbs at 50 mph on an area of 25 ft^2 and, therefore, wave splashing would not have an impact on the structure. The valve pit on the south side of the Intake Structure could potentially accumulate water from splashing waves but two submersible pumps can pump the water out (Reference 13).

3. Dam Breaches and Failures

A conventional routing method was used to determine the effects of dam breaches on maximum water surface elevation at Plant Hatch. Three dams (Sinclair Dam, Wallace Dam, and Lloyd Shoals Dam) were postulated to breach during their standard project flood. The breach wave was then routed downstream using the lag time and the storage coefficient based on actual experimental releases of water from both dams. The flood stage at Plant Hatch corresponding to any flood discharge was based on these two parameters in combination with a stage-discharge curve at Plant Hatch (Reference 13).

A failure of Sinclair Dam would result in a higher stage at the site than would Lloyd Shoals Dam because of its greater volume, dam length, and closer proximity to the Hatch site. Assuming instantaneous removal of the earth dike sections at Sinclair Dam, a 27-ft-high wave would be created just below the dam with a discharge of approximately 3,000,000 cfs. The lag time used to route the breach flow was determined from Sinclair releases. The instantaneous Sinclair breach flow was routed from Sinclair Dam to the site resulting in an additional 100,000 cfs and about a 4-ft increase in stage to elevation 100 ft at Plant Hatch (below the PMF stage of 105 ft). Domino failure of Sinclair Dam due to the failure of upstream Wallace Dam was also evaluated. The 29-ft breach wave from Wallace Dam would result in overtopping of Sinclair Dam by 8 ft, creating a 33-ft breach wave downstream of the dam. The breach wave from the domino failure of Wallace and Sinclair dams would result in a 5-ft increase in stage to elevation 101 ft at Plant Hatch (Reference 13).

Similarly, the breach of Lloyd Shoals Dam would result in a 24-ft high wave with a discharge of approximately 800,000 cfs resulting in a 1-ft increase in stage to elevation 97 ft at Plant Hatch (Reference 13).

Since Lloyd Shoals would be overtopped during the PMF, the effects of its failure were also considered. The Lloyd Shoals Dam failure would result in an artificial flood wave at Plant Hatch of 20,000 cfs, increasing the PMF stage by 0.3 ft to elevation 105.3 ft at the Plant Hatch (Reference 13).

4. Storm Surge

Storm surge was not identified as an applicable flood hazard (Reference 13).

5. Seiche

Seiche was not identified as an applicable flood hazard (Reference 13).

6. Tsunami

Tsunami was not identified as an applicable flood hazard (Reference 13).

7. Ice Induced Flooding

The minimum temperature of record at Doctortown, Georgia was identified as 37.4°F (3°C). Therefore, the formation of fragile ice was considered unlikely and ice-induced flooding was not identified as an applicable flood hazard (Reference 13).

8. Channel Migration or Diversion

The U.S. Highway No. 1 bridge, located approximately 0.5 mile upstream of Plant Hatch, controls the channel alignment to its present location. The river channel is relatively straight for a distance of approximately 1.5 miles downstream of the bridge and there are no meanders that could be cut across to divert flow. In addition, the Altamaha River was surveyed by the USACE at the beginning of the 20^{th} century and no major changes in channel alignment have occurred in subsequent years. The USACE estimates that an oxbow meander is cut off about once every 100 years and meanders develop very slowly. Therefore, any possible effect on water supply to the river intake from channel changes should come from extremely slow changes which can be remedied as they occur (Reference 13).

9. Combined Effects

The coincidental wind wave activity with the PMF produced the maximum water surface elevation at Plant Hatch (of the wave crest) of 108.3 ft. Dam failures were also considered but resulting water surface elevations were below that caused by the PMF concurrent with waves due to winds (Reference 13).

c. Flood Related Changes to the Licensing Basis and Any Flood Protection Changes (including mitigation) Since License Issuance

Flood-Related Changes to the Licensing Basis since License Issuance

There have been no flood related changes to the licensing basis since the last license renewal.

Flood Protection Changes since License Issuance

There have been no flood protection changes to the licensing basis since the last license renewal. In addition, the flooding walkdown (Reference 14) determined that no additional or enhanced flood protection features are warranted.

d. Changes to Watershed and Local Area since License Issuance

There have been no significant changes to the Altamaha River watershed since the last license renewal. The land use changes are minimal and generally limited to the headwaters of the Ocmulgee River (Atlanta urban area).

The main change to the local drainage area was a construction of drainage channel to the southwest of the ISFSI Pad.

e. Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

Plant Hatch is a dry site, which does not rely on flood mitigation features to maintain key safety functions, and safety-related structures are located above the maximum Current Licensing Basis (CLB) flood hazard elevation of 108.3 ft. Site topography and below-grade walls and penetrations function as flood protection features. The finished floor elevation of the lowest safety-related structure, the Intake Structure, is at 111.0 ft and the finished floor elevation of the Reactor Building and Turbine Building is at 130.0 ft. The Intake Structure external walls were constructed from reinforced concrete designed to withstand an impact load of 4,000 lb at a wind speed of 50 mph over an area of 25 ft^2; therefore, potential splashing of waves would not impact the structure. The labyrinth-type doors of the Intake Structure are not exposed to the effects of wave runup. Water accumulated in the valve pit as a result of wave runup onto the adjacent grade would be removed using two redundant, submersible sump pumps located in a small sump inside the valve pit. The valve pit is also protected by a reinforced concrete wall from rising water in the Intake Structure pump well. Below-grade safety-related areas in the power block were credited as dry and are protected from groundwater flooding. The below-grade foundation slabs and exterior walls were designed to resist upward and lateral pressures caused by the maximum flood level. Below-grade penetrations are protected with appropriate seals (Reference 13).

The air cooling vents of dry casks in the Independent Spent Fuel Storage Installation (ISFSI) pad are susceptible to flooding during the LIP event. Procedures are in place to limit the debris blockage of the cask vents (Reference 14).

f. Additional Site Details

While the Altamaha River in the immediate vicinity of the site has not exhibited a tendency to meander, sediment deposition and aggradation has been documented at the Intake Structure. In order to prevent channel diversion and loss of flow capacity, Georgia Power Company conducts continuous monitoring of the Altamaha River reach adjacent to the site. The monitoring includes bathymetric surveys, typically in late spring or early summer, followed by dredging, if required. This ensures that minimum depth of the approach channel is maintained and water is available to intake pumps during periods of low river flows (Reference 13).

Recommendation 2.3 flooding walkdowns verified that external flooding protection features were, for the most part, in place, functional, and maintained. The flooding walkdowns identified six deficiencies, of which four were scheduled to be addressed by November 27, 2013. The remaining two deficiencies are scheduled to be addressed by April 30, 2014 (Reference 14).

5. SUMMARY OF FLOOD HAZARD REEVALUATION

The following is a summary evaluation of the flood hazard at Plant Hatch for each flood causing mechanism described in NUREG/CR-7046. These evaluations are based on acceptable industry standard methodologies and regulatory guidance. An analysis to identify each flood causing mechanism that may impact the site was performed including LIP and site drainage, flooding in streams and rivers, dam breaches and failures, channel migration or diversion, and combined effects. Mechanisms that were not applicable (i.e., storm surge, seiche, and tsunami) have been screened-out, as described below.

a. Local Intense Precipitation

The LIP is a measure of the extreme precipitation (high intensity/short duration) at a given location. The duration of the event and the support area are needed to qualify an extreme precipitation event fully. Generally, the amount of extreme precipitation decreases with increasing duration and increasing area. NUREG/CR-7046 (Reference 3) specifies that the LIP should be equivalent to the 1-hr, 2.56-km^2 (1-mi^2) PMP at the plant site.

The LIP event was evaluated to determine the associated flooding elevations and velocities assuming the active and passive drainage features are non-functioning. The LIP evaluation was performed in accordance with NUREG/CR-7046 and was developed in Calculation Package SCNH-13-021 (Reference 9).

The model was created with boundaries that encompass the local site drainage. Plant Hatch is elevated from its surrounding topography and water drains north to the Altamaha River.

The runoff caused by the LIP event was estimated using FLO-2D software (Reference 15). The software uses shallow water equations to route stormwater throughout the site. FLO-2D depicts site topography using a DEM to characterize grading, slopes, drainage divides, and low areas of the site. The methodology used within the FLO-2D software included the rainfall function and the levee function to incorporate site security features which could impact the natural drainage characteristics of the site. The DEM was produced from LiDAR data and supplemented with as built drawings of site features (e.g., drainage features such as side ditches). Exterior door elevations and the surrounding areas of the safety related structures were based on a topographic survey.

The FLO-2D model uses Manning's n-values to characterize the site's surface roughness and calculate effects on flow depths and velocities. Manning's Roughness Coefficients (n-values) were based on the land cover for the site. Per NUREG/CR-7046 recommendations, runoff losses were ignored during the LIP event in order to maximize the water elevation on site from the event. Only overland flow and open channel systems were modeled and considered in the LIP flooding analysis.

The 1-hour PMP event distribution was developed using HMR 52. The total PMP depth per square mile for the 1-hr event was extrapolated from the PMP depth contour map provided in Figure 24 of HMR 52

(Reference 16). The distribution of the 1-hr PMP was developed for the 5-, 15-, and 30-minute time intervals, with the 60-minute interval being the 1-hr PMP depth. The 1-hr PMP distribution is provided in Table 2 and Figure 4 below. The 1-hr PMP was modeled in FLO-2D to calculate the subsequent site flooding.

Table 2: 1-mi^2/1-hr PMP Distribution

Time (minutes)	Percent Total PMP (%)	Cumulative Depth (inches)	Reference
0	0%	0.00	N/A
5	32.05%	6.17	Figure 36 of the HMR-52 manual
15	50.28%	9.68	Figure 37 of the HMR-52 manual
30	73.36%	14.13	Figure 38 of the HMR-52 manual
60	100%	19.26	Figure 24 of the HMR-52 manual

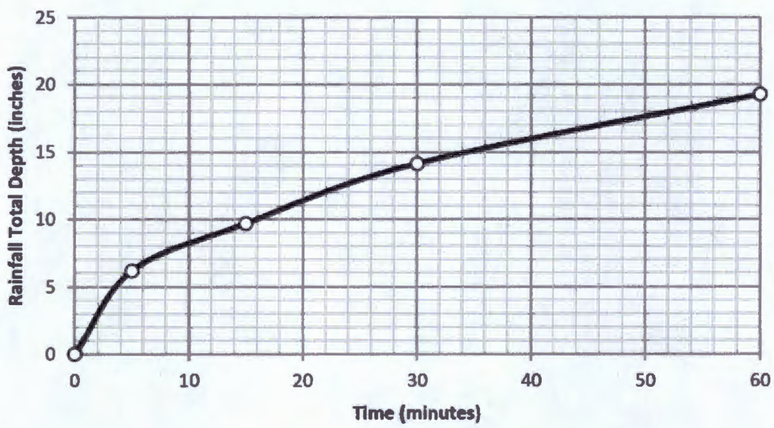

Figure 4: 1-hr PMP Distribution for Plant Hatch

To determine the flooding elevations associated with the LIP, the 1-mi^2/ 1-hr storm was applied evenly across the site, and the model was allowed to run for 24 hours to ensure that only the areas of static ponding would remain. The LIP evaluation was conducted independently of external high-water events (i.e., the LIP event was assumed to have occurred non-coincident to a river flood). Therefore, backwater or tailwater was not considered (Reference 3).

The LIP flooding evaluation (per Case 3 assumptions of NUREG/CR-7046, Section 3.2) estimated the maximum flooding depths, water surface elevations, velocities, resultant static loads, and resultant impact loads that could be expected during the LIP event, assuming the surface drainage system and storm sewer system are fully blocked (Reference 3) .

A summary of the results of the analyses is provided in Table 3.

Table 3: LIP Predicted Flooding Results at the Main Doors and Bays

Building	Door ID	Max. Water Surface Elevation	Max. Flooding Depth above Surveyed FFE	Flooding Duration above Surveyed FFE	Max. Velocity	Max. Resultant Impact Load	Max. Resultant Static Load
		ft	ft	hr	ft/sec	lb/ft	lb/ft
Intake Structure	Door D-130	110.92	0.01	< 0.1	0.91	0.72	3.96
	Door D-131	111.20	0.23	1.0	0.42	0.10	1.82
	Door D-132	111.12	0.15	0.2	0.37	0.40	53.08
Diesel Generator Building	Door D-166	130.16	0.27	1.1	0.35	0.11	4.25
	Door D-167	128.77	0.00	0.0	0.14	0.01	5.75
Turbine and Reactor Building (Unit 1 & 2)	Door R-30A	130.13	0.37	1.3	0.91	0.55	6.62
	Door R-23A	130.15	0.35	1.3	0.56	0.28	4.91
	Door T-15	130.05	0.14	1.3	1.61	1.28	1.32
	Door T-16	130.15	0.24	1.4	1.17	0.91	2.27
	Door 2T-17	130.31	0.39	1.3	0.71	0.46	6.02
	Door 2T-18	130.30	0.38	1.2	0.48	0.19	6.72
	Truck Bay Door	130.40	0.87	6.0	0.82	1.55	28.36
Control Building	Freight Elevator	131.23	1.42	1.3	3.06	35.10	78.02

b. Flooding in Streams and Rivers

The PMF in rivers and streams adjoining the site was determined by applying the PMP to the drainage basin in which the site is located. The PMF is based on a transformation of PMP rainfall on a watershed to flood flow. The PMP is a deterministic estimate of the theoretical maximum depth of precipitation that can occur at a time of year of a specified area. A rainfall-to-runoff transformation function, as well as runoff characteristics, based on the topographic and drainage system network characteristics and watershed properties are needed to appropriately develop the PMF hydrograph. The PMF hydrograph is a time history of the discharge and serves as the input parameter for the hydraulic model which develops the flow characteristics including flood flow and elevation.

The precipitation driven PMF discharge was determined from the evaluation of three combined-effect flood scenarios defined by NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3). A deterministic HEC-HMS model was developed and used to evaluate the combined-effect floods under Scenario 1. Due to the geographic location of Plant Hatch, a qualitative assessment of the Probable Maximum Snowpack (PMSP) with the snow-season PMP was performed to evaluate whether the extreme rain on snow events of Scenario 2 and Scenario 3 are bound by the Scenario 1 event. The model was based on the best available geospatial data and was calibrated to a severe storm event, which occurred on March 16, 1998. The storm event resulted in the highest recorded discharge at Plant Hatch since 1948.

The three precipitation driven combined-effect flood scenarios evaluated in this report are consistent with NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3):

- Scenario 1 – Combination of:
    - Mean monthly base flow;
    - Median soil moisture;
    - Antecedent or subsequent rain: the lesser of (1) rainfall equal to 40 percent of PMP and (2) a 500-year rainfall; and
    - PMP.

- Scenario 2 – Combination of:
  - Mean monthly base flow;
  - Probable maximum snowpack; and
  - A 100-year, snow-season rainfall.

- Scenario 3 – Combination of:
  - Mean monthly base flow;
  - A 100-year snowpack; and
  - Snow-season PMP.

The evaluation was performed consistent with the following guidance documents:

- NRC Office of Standards Development, Regulatory Guide: RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977.
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8-1992).
- NUREG/CR-7046 "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," publication date, November 2011.

Hydrologic parameters, inputs, and assumptions were selected based on recommendations provided in NUREG/CR-7046 (Reference 3). Hydrologic parameters, inputs, and assumptions based on Federal regulatory guidance from other agencies (i.e., NRCS, USGS, and USACE), previous studies, and engineering judgment were also used to develop parameters where NUREG/CR-7046 did not provide guidance.

The purpose of this evaluation was to determine the governing PMF peak discharge at Plant Hatch. The PMF scenario producing the greatest calculated peak discharge of the three scenarios evaluated (listed above) is identified as the governing PMF. For consistency with the CLB, evaluation of hydrologic dam failures is provided in the Dam Breaches and Failures Section. However, the results of the dam and reservoir screening are presented in this section due to their importance for sub-basin delineation.

The following sub sections describe the inputs, assumptions, methodology, and results of the relevant analyses.

Dam and Reservoir Screening

Section 5.5 of ANS 2.8 states: "All dams above the plant site shall be considered for potential failure, but some may be eliminated from further consideration because of low differential head, small volume, distance from plant site, and major intervening natural or reservoir detention capacity (Reference 4)." In July 2013, the NRC issued *JLD-ISG-2013-01 Guidance for Assessment of Flooding Hazards Due to Dam Failure* (Reference 1). The dams in the Altamaha River Watershed upstream of Plant Hatch were evaluated in accordance with the guidance. Dams identified to have a potential impact at the site were considered to be critical and were modeled individually in estimation of the PMF at the site.

According to the USACE National Inventory of Dams (NID) database there are approximately 1,379 dams in the Altamaha River Watershed upstream of Doctortown, GA. The latitude, longitude, maximum storage volume, and height for each dam were obtained from the NID database (Reference 7).

Structures that were not considered a dam in accordance with Section 391-3-8.02(h) of the Georgia Department of Natural Resources Environmental Protection Division Rules for Dam Safety were assumed to be inconsequential and were eliminated from the screening process. Dams that do not meet at least one of the two criteria below per Section 391-3-8.02(h) were considered inconsequential (Reference 17):

- Be twenty-five (25) feet or more in height from the natural bed of the stream or water course measured at the downstream toe or the lowest elevation of the outside limit of the barrier (whichever is lower) to the maximum water storage elevation; or
- Have an impounding capacity at maximum water storage elevation of one hundred (100) acre-feet or more.

For this analysis, a total of 292 dams in the watershed were identified as inconsequential. The remaining 1,087 dams that met the above criteria were screened using the volume method per Section 3.2 of JLD-ISG-2013-01 (Reference 1) to identify non-critical dams and potentially critical dams by estimating the potential impact from an upstream dam breach. The volume screening method assumes that the total upstream dam storage volume is simultaneously transferred to the site without attenuation. Additionally, the method assumes the only available floodplain storage is between the lowest safety-related structure and the 500-year water surface elevation.

As part of the screening process, the dams were ranked by storage volumes from the NID database. The storage volumes were cumulatively added from the lowest to the highest rank. Once the cumulative storage volume exceeded the volume between the lowest elevation of safety related equipment and the 500-year flood stage, any dams exceeding the volume and ranked higher were identified as potentially critical. The remaining dams were identified as non-critical. The potentially critical dams were modeled in HEC-HMS as individual dams. The non-critical dams were modeled as hypothetical clusters of dams in the dam breach analysis (Reference 8).

Seven (7) dams were identified as potentially critical (Table 4). These dams were incorporated into the HEC-HMS model and used in the calibration process.

Table 4: Potentially Critical Dams (Reference 7)

No	Classification	Dam Name	River	County	NID Height	Individual Dam Storage Volume Largest-to-Smallest	Cumulative Storage Volume
					ft	ac-ft	ac-ft
1	I	Sinclair Dam	Oconee River	Baldwin	91	490,000	1,844,619
2	II-large	Wallace Dam	Oconee River	Putnam	120	400,000	1,354,619
3	I	Lake Juliette Dam	Rum Creek	Monroe	103.2	169,000	954,619
4	I	Lloyd Shoals Dam	Ocmulgee River	Butts	100	107,000	785,619
5	I	Lake Tobesofkee Dam	Tobesofkee Creek	Bibb	54	43,054	678,619
6	I	Town Creek Reservoir Dam	Town Creek	Jones	110	26,855	635,565
7	I	Upper Towaliga Reservoir	Towaliga River	Spalding	45	25,700	608,710
Volume for the remaining 1,080 Dams						**583,010**	

Calibrated Hydrologic Model

The USACE Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) software, version 3.5 (Reference 18) was used to simulate the hydrologic processes of the watershed and to estimate the PMF peak flow rates during a PMP event. The HEC-HMS model was calibrated to recorded stream gage data from the March 16, 1998 storm event. The Altamaha River Watershed also experienced saturated conditions due to two significant storm events preceding the March 1998 event, which occurred in January and February of 1998, respectively. Together, these three storm events account for the top three flood events since 1948 and were considered appropriate for calibration of the HEC-HMS model (Reference 7).

Daily precipitation values from twenty-six (26) rainfall gages in and around the Altamaha River Watershed were evaluated for use in the calibration of the HEC-HMS model. Nineteen (19) of these twenty-six (26) rainfall gages were used in the analysis due to some of the gages malfunctioning or being inactive at the time of the March 1998 event.

Stream flow data from nine (9) stream gages within the Altamaha River Watershed was used in the calibration process. Location of the stream flow gages is presented in Figure 5.

Figure 5: Streamflow Gage Locations

The Altamaha River Watershed was divided into 76 sub-basins, ranging in area from approximately 2 square miles to 717 square miles. The sub-basins were delineated at each USGS stream gage, at the confluence of major tributaries, and at each of the "critical" dams, as determined during the screening process.

The initial lag time inputs for hydrograph routing were based on the NRCS Technical Release 55 (TR-55) methodology using the segmental velocity approach along the longest flow paths. The Muskingum Method was used to estimate routing in the channel reaches; the Muskingum K (representing the travel time through each reach) and X (representing the storage factor) parameters were adjusted during the calibration process to reflect the watershed response during the calibration event.

Eight (8) dams and their reservoirs (Lake Tobesofkee, Upper Towaliga Reservoir, Town Creek Reservoir, Lake Juliette, Barnett Shoals Dam, Wallace Dam, Sinclair Dam, and Lloyd Shoals Dam) were included in the calibration of the HEC-HMS model. Stage-area or stage-storage curves were used to represent the storage relationship in the HEC-HMS model based on information presented in the FSAR (Reference 13) and individual dam design documents.

The geology of the Altamaha River Watershed plays a critical role in the watershed response due to an apparent correlation of different geologic formations in the basin. The northern area of the watershed is underlain by the Piedmont and Blue Ridge Aquifer, the center of the watershed by the Southeastern Coastal Aquifer, and the southern area by the Surficial Aquifer, as shown in Figure 6. The general movement within the Surficial Aquifer is confined above thin clay beds and most of the water that enters the system moves along short flow paths, and discharges back into the streams as baseflow. To improve calibration results, sub-basins within the same geologic formation were initially assigned same loss rate and timing/area factors. These values were then manually adjusted as part of the calibration to match the watershed response during the March 1998 event. The sub-basin constant loss rates varied from 0 in/hr to 0.05 in/hr (Reference 7).

Figure 6: Geologic Aquifers in the Altamaha Watershed

The main objective of the calibration was to provide an accurate representation of watershed's response during the March 1998 event, in particular at Baxley gage (USGS Gage 02225000), which is located just upstream of Plant Hatch. Characteristics used to measure the accuracy of fit included peak discharge, volume, time to peak, and shape of the resulting hydrograph. Time to peak and volume were prioritized in terms of best fit when calibrating the model. The results of the calibration at the Baxley gage indicate that the HEC-HMS model predicts the peak flow, volume and timing accurately, as shown in Figure 7 and Table 5.

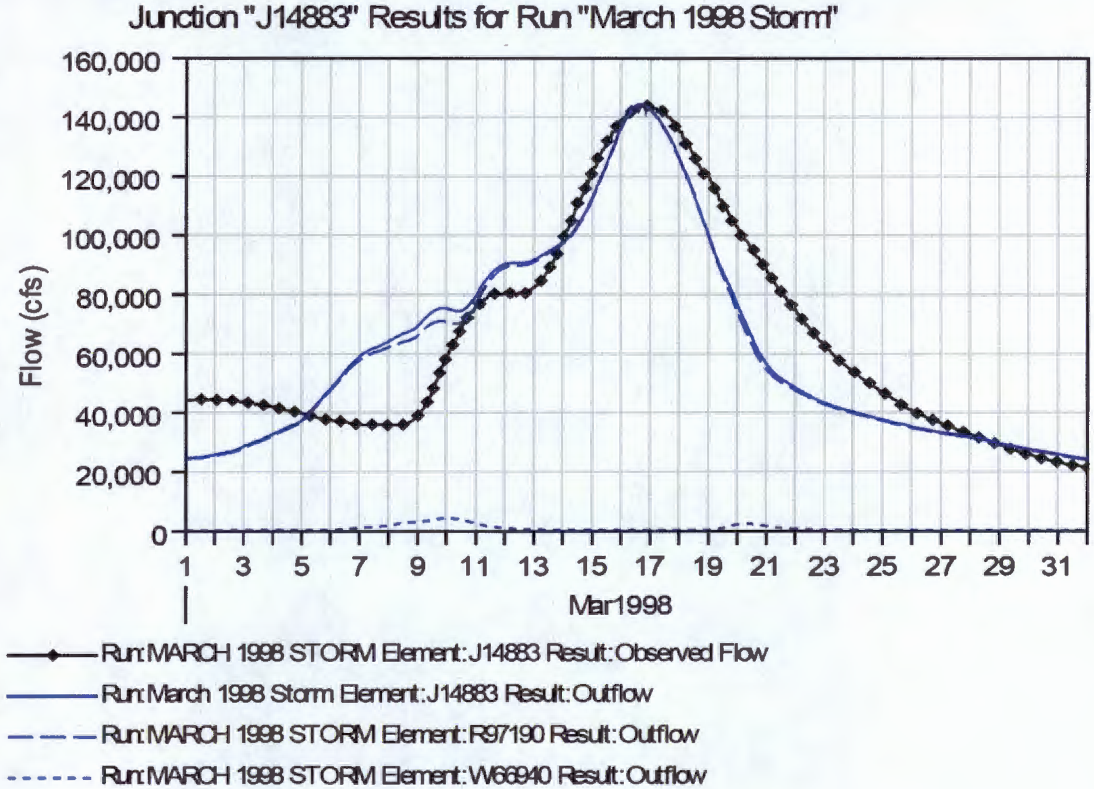

Figure 7: Calibration Hydrograph – Altamaha River at Baxley, GA (USGS Gage 02225000)

Table 5: Calibration Comparison – Altamaha River at Baxley, GA (USGS Gage 02225000)

	Peak Date	Peak Time	Peak Discharge (cfs)	Depth over Watershed (in)
Model Peak Time:	16-Mar-98	15:00	144,274	6.52
Observed Time:	16-Mar-98	16:00	144,000	6.87

At the remaining calibration points throughout the watershed, the model generally over predicts peak discharges and runoff volume, with peak discharges occurring earlier than observed. Given the watershed's unprecedented response during the March 1998 event the calibrated model provides a conservative yet realistic estimate of watershed conditions during a major storm event.

Scenario 1 PMF Evaluation

Per NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3), the Scenario 1 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- Median soil moisture;

- Antecedent or subsequent rain: the lesser of (1) rainfall equal to 40 percent of PMP and (2) a 500-year rainfall; and
- PMP.

Scenario 1 was evaluated using the calibrated hydrologic model and derived precipitation inputs to reflect the combination of mean monthly base flow, antecedent rain, and the PMP event, as well as median soil moisture conditions. Scenario 1 was evaluated in Calculation Package SCNH-13-018 (Reference 7).

Mean monthly baseflow used in the Scenario 1 calculation of the PMF for the Altamaha River Watershed was obtained from the Baxley gage (USGS Gage 02225000). The monthly averages were then divided by the watershed area of 11,600 square miles in order to calculate the baseflow per square mile. This value was applied monthly in the HEC-HMS model by multiplying the baseflow per square mile times the area of each sub-basin.

Table 6: Average Monthly Baseflow per Square Area – Altamaha River at Baxley, GA (USGS Gage 02225000)

Month	*Jan*	*Feb*	*March*	*April*	*May*	*June*	*July*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*
Mean Monthly Discharge (cfs)	14,700	20,500	23,000	18,400	8,990	6,690	5,970	5,500	4,650	5,530	5,810	10,100
Years of Record	43	43	43	43	43	43	43	43	43	44	44	44
Average Flow per Watershed Area (cfs/mi^2)	1.27	1.77	1.98	1.59	0.78	0.58	0.51	0.47	0.40	0.48	0.50	0.87
Watershed Area @ Baxley GA (sq miles)											**11, 600**	
Note: Mean monthly discharge and years of record were obtained from the USGS website on 4/26/2013												

The 72-hour, all-season PMP for the Altamaha River Watershed to the Doctortown stream gage were obtained from HMR 51, then spatially and temporally distributed using USACE HMR 52 software with the interface modified by AMEC for use in ESRI ArcGIS.

NOAA Atlas 14 (Reference 19) was used to determine a spatially averaged 500-year/3-day rainfall value for the Altamaha River Watershed for comparison with the 40% PMP. The 40% PMP for each sub-basin was computed by multiplying the 3-day total PMP depth of 19.2 inches by 0.40 resulting in 7.68-inch basin-wide 40% PMP. The 500-year 3-day rainfall depth for the Altamaha River Watershed is 12.48 inches. The comparison between the 500-year NOAA Atlas 14 rainfall at the site and the 40% PMP shows that the 40% PMP is the lesser of the two storms. Both antecedent and subsequent storm events were considered per NUREG/CR-7046 (Reference 3).

HMR 52 software with an ESRI ArcGIS interface was used to evaluate storm configurations producing the highest basin-averaged rainfall over the watershed. The following storm centers were identified for the analysis: (1) Basin Centroid representing the geometric center of the Altamaha River Watershed; (2) Full Basin Centroid situated so the storm covers the entire Altamaha River Watershed; and (3) Lower Basin situated closer to Plant Hatch and representing the design basis storm center. The locations of the three storm centers are provided in Figure 8.

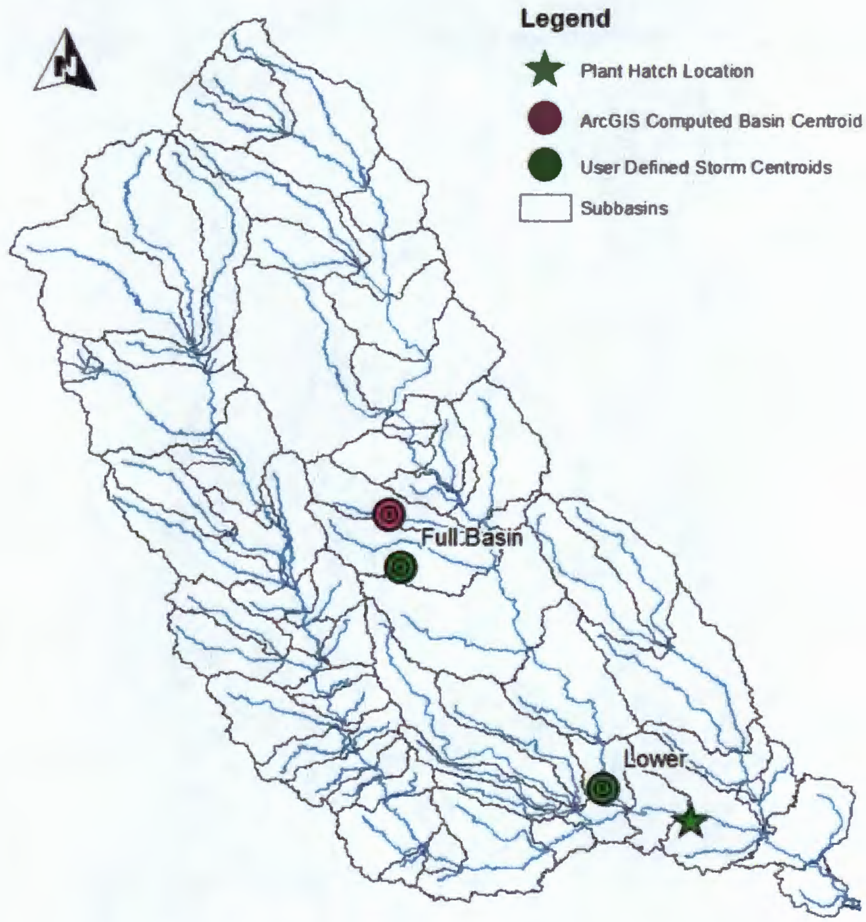

Figure 8: Location of Storm Centers

The time interval for temporal distribution was set at 120 minutes. The position of the maximum 6-hour interval was adjusted depending on the desired temporal distribution. The 1:6 hour ratio for temporal distribution, the preferred maximum storm orientation from HMR 52, and the depth-area-duration data from HMR 51 were determined based on the storm center location (Reference 7).

Various storm orientations and sizes for each storm center were evaluated using HMR 52 to determine the maximum rainfall depth over the watershed. Optimal combinations of storm orientation and size for input into HEC-HMS were determined by identifying the maximum average 72-hour depth and maximum rainfall depth at the storm peak 6-hour interval (Reference 7).

The calibrated HEC-HMS model was then used to determine the runoff at Plant Hatch resulting from the various orientations and sizes of the PMP event. The PMP event centered over the Basin Centroid, with orientation of 172 degrees and basin-averaged rainfall of 19.2 inches, generated the highest discharge at Plant Hatch (883,214 cfs). The hyetographs of antecedent and subsequent storms (40% PMP) were added and modeled in HEC-HMS. A five-day dry period between the antecedent storm and the PMP or the PMP and the subsequent storm was provided to account for the long lag time in the watershed. Various temporal distributions of the PMP event, with the peak rainfall increment occurring 36 hours, 42 hours, 48 hours, 54 hours, 60 hours and 66 hours following the start of the PMP precipitation, were evaluated using the HEC-HMS model (Reference 7). The temporal distributions for the critical PMP event are presented in Figure 9.

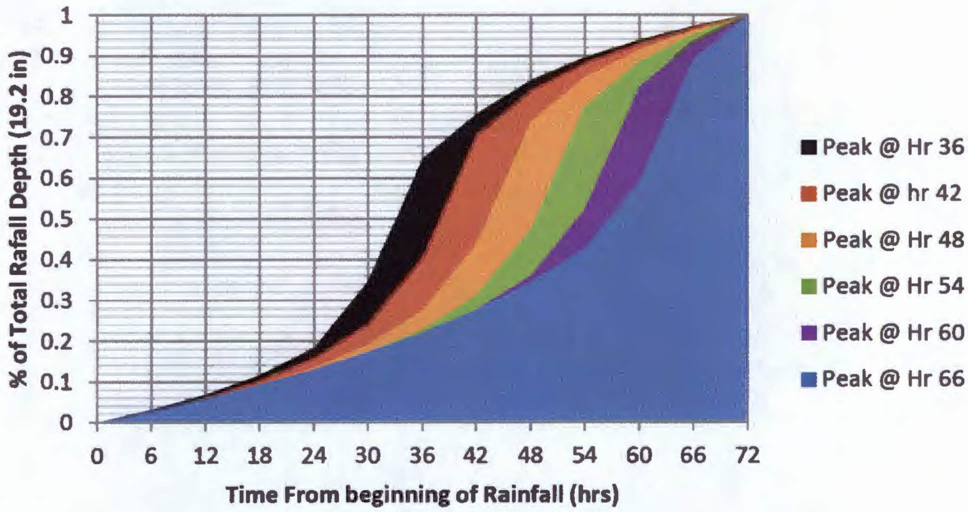

Figure 9: Temporal Distributions of the Critical PMP Event

The critical Scenario 1 PMF peak discharge is a result of a 40% antecedent PMP followed by a PMP with a peak positioned 234 hours after the beginning of rainfall. The critical Scenario 1 PMF results in a peak discharge of 899,636 cfs at Plant Hatch (Reference 7). The results of the various model runs are provided in Table 7.

Table 7: Scenario 1 PMF Discharge results

Antecedent Storm			
	Antecedent Temporal Loading	**PMP Temporal Loading**	**Peak Flow (cfs)**
Front-1/3 (Position 6)	Peak @ hour 42	Peak @ hour 228	898,296
Center (Position 7)	Peak @ hour 42	Peak @ hour 234	899,636
Center (Position 8)	Peak @ hour 42	Peak @ hour 240	897,391
2/3 (Position 9)	Peak @ hour 42	Peak @ hour 246	885,743
End (Position 10)	Peak @ hour 42	Peak @ hour 252	878,214
End (Position 11)	Peak @ hour 42	Peak @ hour 258	864,959
Max Antecedent	***Peak @ hour 42***	***Peak @ hour 234***	**899,636**

The individual sub-basin hydrographs for the critical Scenario 1 PMF were used as inflow hydrographs in the unsteady flow analysis to determine the governing PMF discharge and stage at Plant Hatch, as discussed in the "Governing Riverine PMF Discharge and Stage" and "Dam Breaches and Failures" sections.

Scenario 2 and Scenario 3

Per NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3), the Scenario 2 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- Probable maximum snowpack; and
- A 100-year, snow-season rainfall.

The Scenario 3 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- A 100-year snowpack; and
- Snow-season PMP.

A qualitative assessment was used to evaluate whether a combination of PMSP and snow-season PMP is bound by Scenario 1 combined-effect precipitation events.

The snow-season PMP was estimated based on the comparison of the 72-hr, 10-square-mile all-season PMP reported in HMR 51 to the 72-hr, 10-square-mile January/ February PMP reported in HMR 53 (Reference 7). The PMP depths derived from HMR 51 and HMR 53 are presented in Table 8.

Table 8: Inputs for Scenarios 2 and 3 from HMR 51 and HMR 53

HMR 53 Snow-Season 10-mi^2, 72-hr PMP (January/February)	**31.31 in**
HMR 51 10-mi^2, 72-hr PMP (All-Season)	51.80 in
Snow-Season PMP/All Season PMP Ratio	0.60
HMR 52 Basin-Averaged, 72-hr PMP (All-Season)	19.20 in

The greatest daily snow depth of 10 inches recorded in the State of Georgia and obtained from the NOAA NCDC United States Snow Climatology website was conservatively applied to the entire watershed.

The basin-averaged snow-season PMP for the Altamaha River Watershed was estimated by applying the snow-season/all-season PMP ratio (0.60) to the HMR 52 basin-averaged all-season PMP (19.20 inches), resulting in a basin-averaged snow-season PMP of 11.61 inches. The snow water equivalent of the PMSP was calculated using the following equation:

$$Snow\ Water\ Equivalent = \frac{\rho_s}{\rho_w} d_s = 5\ inches$$

Where:
ρ_s = Desity of snowpack (conservatively assumed 0.5 g/cm^3);
ρ_w = Density of water (1 g/cm^3); and
d_s = Depth of snowpack (conservatively assumed 10 inches).

The combined snow-season PMP (11.61 inches) and snow water equivalent of the PMSP (5 inches) was calculated to be 16.61 inches, which is less than the all-season PMP of 19.20 inches. The all-season PMP is significantly greater than a conservative estimate of the combined PMSP and snow-season PMP and, therefore, the all season PMP would produce greater peak discharges than an unlikely rain on snow event.

Further evaluation of Scenario 2 and Scenario 3 was not performed since these events are bounded by the Scenario 1 event.

Governing Riverine PMF Peak Discharge and Stage

An unsteady flow hydraulic HEC-RAS model was developed to calculate maximum flood elevations, maximum overbank velocities, and flood duration resulting from the Scenario 1 PMF discharge presented above. The same model was also used for estimation of maximum flood elevations due to dam breaches and failures presented in Section 4.c. The unsteady flow model's extent was approximately from 15 miles downstream of Plant Hatch to Lloyd Shoals Dam.

The channel and overbank geometry of the HEC-RAS model was based on USGS NED data, LiDAR and bathymetric survey data. Flow hydrographs developed in the HEC-HMS model were used for the respective model runs (i.e., March 1998 calibration run, Scenario 1 run, and dam breaches/failures runs). Rating curves obtained from multiple USGS stream gage locations were used to calibrate the model to the March 1998 event (Reference 12). The list of USGS stream gages used for calibration is provided in Table 9 below.

Table 9: USGS Stream Gages Used for Calibration

Name	USGS Gage ID	USGS Field Measurement Period of Record	Maximum Rating Curve Flow (cfs)	Reference HEC-RAS River Station
Altamaha River near Baxley, GA	02225000	1948-2013	162,000	101,365.4
Ocmulgee River at Charlotteville, GA	02224940	1978-2013	163,000	179,574.8
Ocmulgee River at Lumber City, GA	02215500	1937-2013	97,000	252,368.5
Ocmulgee River at Hawkinsville, GA	02215000	1929-2013	100,000	853,805.6
Ocmulgee River at Macon, GA	02213000	1928-2013	270,000	1,253,730.0
Ocmulgee River near Jackson, GA	02210500	1906-2013	73,000	1,484,484.0

The ground surface DEM was developed by combining available USGS NED, LiDAR, and bathymetric data to reflect current topography of the stream channel and reach overbanks. Cross section invert elevations for reaches where bathymetric survey data was not available were estimated by interpolating between the USGS stream gages datums (upstream of Plant Hatch) or by extrapolating the elevation of the downstream end of the bathymetric survey with the water surface elevation slope from the USGS DEM (downstream of Plant Hatch) (Reference 12).

Manning's n-values were calibrated by adjusting the factors within a range for natural streams and overbank conditions (Reference 12) as shown in Table 10.

Table 10: Manning's n-value Ranges

	Starting Manning's n-Value	Minimum Calibration Range	Maximum Calibration Range
Channel	0.045	0.025	0.055
Overbanks	0.100	0.060	0.160

To account for changes in the Manning's n-value as the stage and flow vary, the HEC-RAS model was calibrated using flow roughness factors and the rating curves for each USGS gage (Reference 12). The calibrated Manning's n-values are presented in Figure 10.

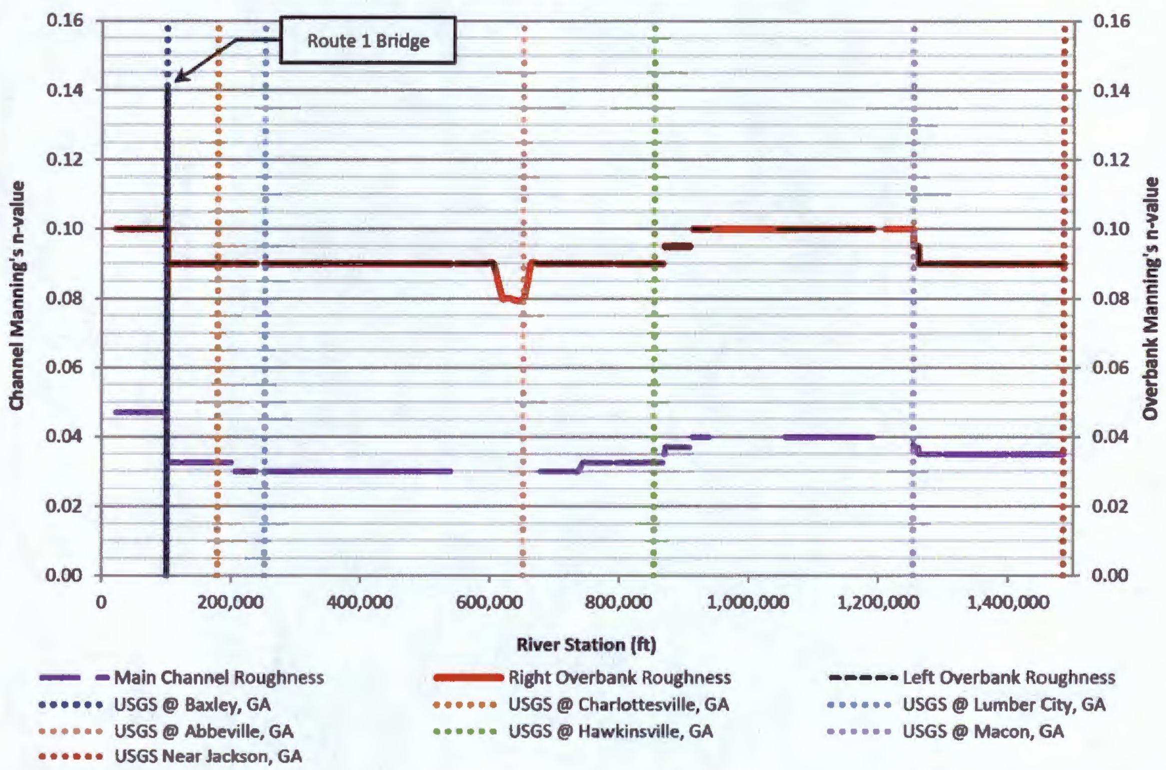

Figure 10: Change in Manning's n-values in the Calibrated HEC-RAS Model

Tributaries to the main reach provide additional floodplain storage and attenuation of the flood wave. The additional floodplain storage was modeled by extending the main river cross section to include the tributary and at some locations by utilizing a lateral weir connected to a storage area.

The results of the calibration are provided in Table 11. The model was calibrated within ±0.5 ft of the published USGS rating curves, which is within ±1.0 ft of measured stage data accuracy (Reference 12).

Table 11: HEC-RAS Calibration Results

HEC-RAS Cross Section River Station	USGS Gage Name	HEC-RAS Peak Flow (cfs)	Difference in Max. WSEL (ft)
101365.4	Altamaha River near Baxley, GA	136,201	**0.13**
179574.8	Ocmulgee River at Charlotteville, GA	136,437	**0.10**
252368.5	Ocmulgee River at Lumber City, GA	66,210	**-0.36**
853805.6	Ocmulgee River at Hawkinsville, GA	59,550	**-0.45**
1253730	Ocmulgee River at Macon, GA	61,897	**0.40**
1484484	Ocmulgee River near Jackson, GA	40,700	**0.04**

The peak flow rate for Scenario 1 precipitation event was calculated to be 726,911 cfs, which corresponds to maximum water surface elevation of 109.58 ft. The maximum right overbank velocity was calculated to be 1.34 fps (Reference 12).

c. Dam Breaches and Failures

Breach or failure of artificial barriers used to impound water for multiple possible functions, including flood control (attenuation), recreation, water supply, hydroelectric, sediment storage, aquatic habitat, stormwater (quantity/quality) management, or a combination thereof, located within the watershed of an adjacent stream/river or upslope of SSCs important to safety were evaluated. Flood waves resulting from the breach of upstream dams considered domino-type or cascading dam failures.

Upstream dam breaches and failures were evaluated for sunny day, seismic, and overtopping breach events in accordance with NUREG/CR-7046, Section 3.9 and Appendix H.2 (Reference 3). The dam failure analysis was performed using the calibrated HEC-HMS model developed in Calculation SCNH-13-018 (Reference 7).

Combined Storage Volume of Small Dams within the Watershed

The USACE NID reports that there are 1,379 dams in the Altamaha River Basin upstream of Plant Hatch. As previously discussed, approximately 1,087 dams are considered potentially non-critical or critical. The deterministic HEC-HMS model developed in Calculation SCNH-013-018 (Reference 7) was used to estimate the governing dam breach PMF discharge. The model included detailed information (e.g., stage-discharge and stage-storage relationships) for eight (8) dams, of which seven (7) were determined to be critical and one (1) non-critical (Barnett Shoals Dam) through the dam screening processes (Reference 7).

The storage volume associated with the remaining dams identified as non-critical based on the dam screening results were clustered and represented in the HEC-HMS model as hypothetical dams. A total of 31 hypothetical dams were added to the HEC-HMS model and modeled during the analysis (Reference 7). The drainage areas for each hypothetical dam are presented in Figure 11 below.

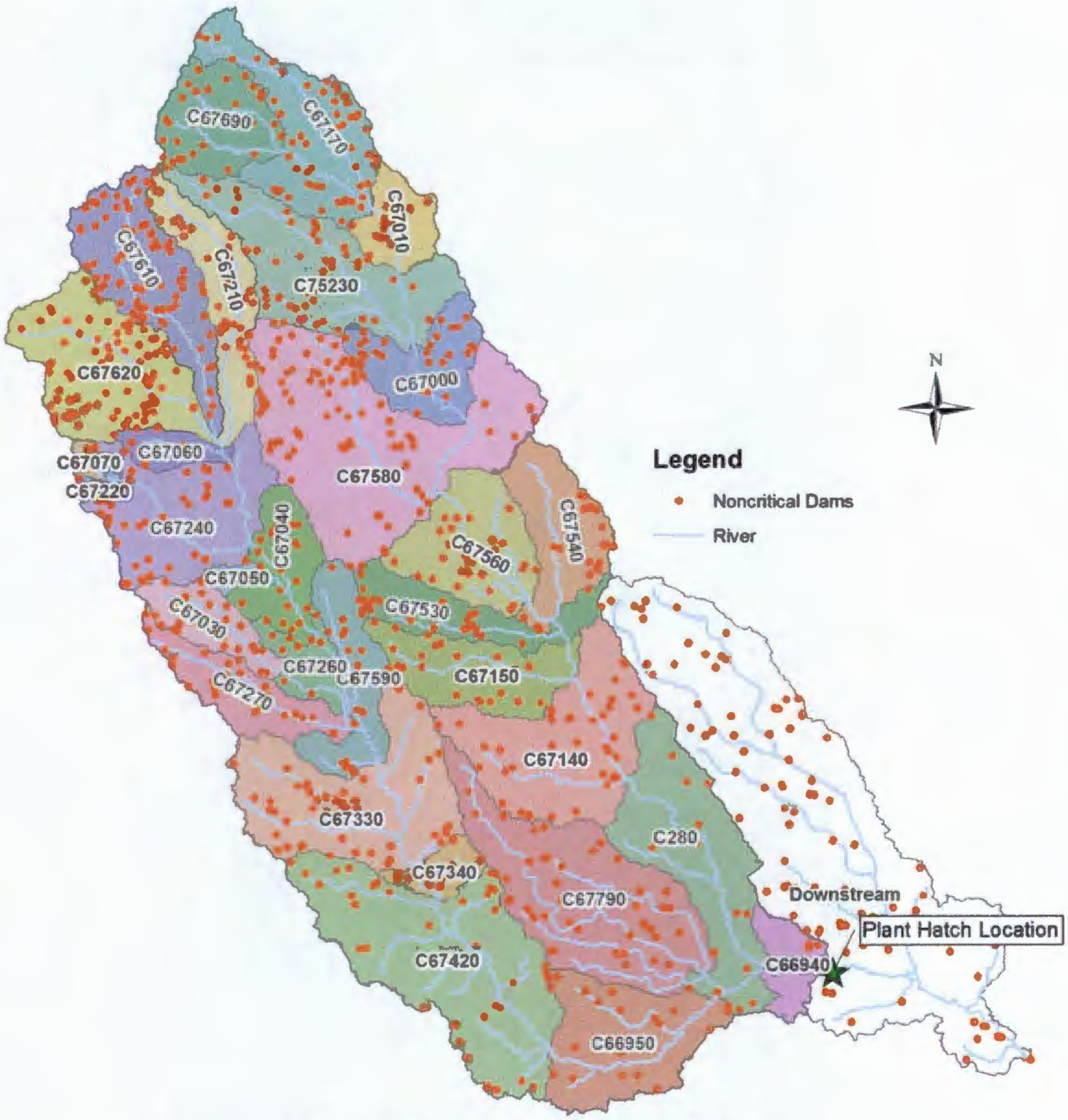

Figure 11: Drainage Areas for Hypothetical Dams

The dimensions of each hypothetical dam were based on the height of the largest individual dam in each cluster. The stage-storage relationship for each hypothetical dam was based on a simplified linear relationship. The breach parameters and timing were estimated using the Xu & Zhang methodology (Reference 5). The starting water surface elevation for each hypothetical dam was assumed to be top of dam. Each hypothetical dam was located at the downstream end of the subbasin associated with each cluster.

Sunny Day Dam Failure

In the sunny day dam breach analysis the subbasins in the HEC-HMS model were updated to remove any baseflow. The starting water surface elevations for the critical dams identified through the dam screening processes were set to the maximum normal pool elevation (i.e., crest of the auxiliary spillway). The dam breach parameters for dams owned by Georgia Power Company (Lloyd Shoals Dam, Lake Juliette Dam, Wallace Dam, and Sinclair Dam) were based on the dimensions and elevations specified in their respective Emergency Action Plans.

The dam height and storage volume from the NID database were used as input values to estimate the dam breach parameters for the hypothetical dams and for the other critical dams. The breach parameters for the critical dams not operated by Georgia Power Company were estimated using methodologies by Froehlich (Reference 20) and Xu & Zhang (Reference 5), and were applied on a dam by dam basis, depending on which methodology produced the most conservative response. The dam breach parameters for hypothetical dams were estimated using the Xu & Zhang methodology (Reference 5). To account for the progression of breach wave from the furthest dam in each hypothetical dam subbasin, the total time of breach formation was estimated by adding the time of concentration to the time of breach formation. The failure modes for all dams were assumed to be a piping breach with a piping elevation occurring at 1/3 of the height of the dam (from the base of the dam). The average breach depth was assumed to occur at ½ the breach depth.

The Xu & Zhang dam breach parameters were based on an empirical formula and were based on widely accepted equations with empirical data to close the gap between idealized parameters used in the breach analysis with actual recorded breach events. The Xu & Zhang study used 75 failure cases that had sufficient information to develop regression equations and subdivided breaching parameters into geometric and hydrographic groups. The Xu & Zhang equations for a variety of erodibility conditions are presented in Calculation SCNH-13-019 (Reference 8).

Similar to the Xu & Zhang method, the Froehlich method is dependent on the height of the dam and the storage volume of the reservoir. This method distinguishes between piping and overtopping failures, using a variable coefficient, the Failure Mode Factor (K_o). This method does not consider dam geometry or the type of soil used to construct the dam.

The resulting HEC-HMS hydrographs for the sunny day dam failure scenario were used as inputs for the unsteady flow HEC-RAS model to estimate the PMF discharge and stage at Plant Hatch (Reference 12).

The peak flow rate for sunny-day dam breach scenario was calculated to be 340,439 cfs, which corresponds to maximum water surface elevation of 92.37 ft. The maximum right overbank velocity was calculated to be 1.92 fps (Reference 12).

Seismically-Induced Dam Failure

Appendix H.2 of NUREG/CR-7046 (Reference 3) and Section 9.2.1.2 of ANS-2.8 (Reference 4) provide the following two (2) alternative combinations for seismically-induced dam failure:

1. 25-year flood, dam failure caused by the safe shutdown earthquake (SSE) coincident with the peak flood, and 2-year wind speed applied in the critical direction.
2. ½ PMF or 500-year flood, whichever is less, dam failure caused by the operating basis earthquake (OBE) coincident with the peak flood, and 2-year wind speed applied in the critical direction.

The best available interpretation of the above requirements is that seismically-induced dam failure occurs coincident with the peak pool levels for the 25-year flood, 500-year flood, or ½ PMF entering the dam; implying that runoff downstream of the dam to the site is not included.

This approach is reasonable for a single upstream dam but seems inappropriate for multiple dams. It would not be reasonable to assume that peak flood pool levels for all upstream dams occur coincident with an earthquake. Developing floods for multiple upstream dams seemed best accomplished by using watershed-wide precipitation events, not upstream flood-flow, to represent the above combinations. Therefore, Plant

Hatch's seismically-induced dam failure analysis is based on the following precipitation-based combinations:

1. 25-year precipitation throughout the site's watershed, dam failure caused by SSE, and 2-year wind speed applied in the critical direction.
2. ½ PMP or 500-year precipitation, whichever is less, throughout the site's watershed, dam failure caused by the OBE, and 2-year wind speed applied in the critical direction.

This would be consistent with NRC expectations that, as stated in NUREG/CR-7046 and ANS-2.8, combinations are thought to have a probability-of-exceedance of less than 1×10^{-6}. Also, using watershed-wide precipitation events would be conservative because it would include runoff downstream of the dams to the site. Since all dams are assumed to fail during the lower-magnitude (OBE) earthquake, only combination #2 was included in the analysis.

Precipitation depths for each basin for the ½ PMP events were calculated using the spatial distribution of the PMP as determined in Calculation SCNH-013-018 (Reference 7), adhering to HMR 51/52 methodologies. To determine the ½ PMP, the rainfall values derived in Calculation SCNH-013-018 for the all season PMP were divided in half. The 500-year precipitation was derived as the antecedent rainfall in Scenario 1 from Calculation SCNH-013-018 (Reference 7). However, the comparison between the 500-year precipitation and the 50% PMP showed that the 50% PMP is the lesser event.

Note that all dams are assumed to fail when the earthquake occurs (the timing of which is established based on optimal impact to the site), which may not result in a particular dam failing at its peak water level.

Dam breach parameters including breach time formation, side slope factor, top width, bottom width, average width, and average breach depth were estimated using the same inputs, assumptions, and methodologies as the sunny day breach analysis. The calibrated HEC-HMS model used to model the sunny day breach was modified as needed to estimate the discharge associated with a seismic dam breach. The mean monthly baseflow was added to each subbasin as described in Calculation SCNH-013-018 (Reference 7).

In order to determine the critical peak flow based on a seismic dam failure, seven (7) different breach times were evaluated with all dams failing simultaneously at various time-steps beginning 24 hours prior to the rainfall up to 36 hrs after the start of the rainfall.

The resulting HEC-HMS hydrographs for the seismic dam failure scenario were used as inputs for the unsteady flow HEC-RAS model to estimate the PMF discharge and stage at Plant Hatch.

Based on the results of the seismic dam failure analysis, simultaneous breach of all dams 33 hours after the start of rainfall produces the critical flow rate of 631,328 cfs, which corresponds to maximum water surface elevation of 106.24 ft. The maximum right overbank velocity was calculated to be 1.46 fps (Reference 12).

Overtopping Dam Breach

For the overtopping dam breach analysis, the HEC-HMS model used to estimate the peak discharge resulting from a seismic dam breach was modified with breach parameters specific to an overtopping breach event and with the critical precipitation scenario determined in Calculation SCNH-013-018 (Reference 7), consisting of a 40% PMP antecedent event followed by a PMP event.

The starting water surface elevations for the critical dams were set to the maximum normal pool elevation (i.e., crest of the auxiliary spillway). The starting water surface elevations for hypothetical dams were assumed to be at the top of the dam.

The following Georgia Power Company operated dams were not breached as part of the analysis: Lloyd Shoals Dam, Wallace Dam, Sinclair Dam, and Lake Juliette (Plant Scherer) Dam. These dams are either not overtopped during their individual PMF and Plant Hatch PMF or stability analysis have been performed to ensure that the dams will not fail during the overtopping (Lloyd Shoals Dam).

The dam height and storage volume from the NID database were used as input values to estimate the dam breach parameters for the hypothetical dams and for the other critical dams. The breach parameters for the critical dams not operated by Georgia Power Company were estimated using methodologies by Froehlich (Reference 20) and Xu & Zhang (Reference 5), and were applied on a dam by dam basis, depending on which methodology produced the most conservative response. The dam breach parameters for hypothetical dams were estimated using the Xu & Zhang methodology (Reference 5). To account for the progression of breach wave from the furthest dam in each hypothetical dam subbasin, the total time of breach formation was estimated by adding the time of concentration to the time of breach formation. Dams were set to breach at either the top of dam elevation or at the maximum water surface elevation estimated by HEC-HMS (for dams that were not overtopped during the PMF). Dams in series were set to produce a cascading failure.

Per NRC's *JLD-ISG-2013-01 Guidance for Assessment of Flooding Hazards Due to Dam Failure* (Reference 1), concrete dams should be evaluated for potential hydrologic failure modes including but not limited to:

- overtopping of the main dam, and overtopping erosion of a dam abutment or foundation;
- erosion of an unlined tunnel or spillway chute;
- erosion of a channel downstream from a stilling basin due to flow in excess of capacity;
- erosion of the spillway foundation where floor slabs have been damaged or lost overstressing of the dam, foundation, or abutments; and
- cavitation damage to spillway and outlet flow surfaces

Embankment dams should be analyzed for conditions leading to, and the effects of:

- overtopping; and
- increases in internal seepage pressures

The Georgia Power Company operated dams that were not breached as part of the analysis are maintained and inspected on a regular basis. In addition, Lloyd Shoals, Wallace, and Sinclair dams are FERC-licensed dams that meet the standards outlined in the "FERC Engineering Guidelines for the Evaluation of Hydropower Projects" guidance document. The respective FERC licensing documents, stability calculations, and the results of Calculation SCNH-13-018 (Reference 7) support the determination that the dams would not fail during both the individual and Plant Hatch PMF due to hydrologic causes. The following sections provide a summary of justification for not breaching the above-mentioned dams due to overtopping or other hydrologic causes.

Lloyd Shoals Dam

<u>Probable Maximum Flood and Reservoir Capacity</u>

Lloyd Shoals is a gravity concrete FERC-licensed dam (FERC No. 2336) consisting of a 530-ft long earth dike and a 1,570-ft long concrete structure with non-overflow walls, a powerhouse, an intake, and a spillway. The spillway crest is at elevations 528 ft and 525 ft, respectively. The 2-ft and 5-ft Obermeyer gates are used to maintain the reservoir at elevation 530 ft. In addition to the gates, Lloyd Shoals Dam is equipped with 500 ft of 10-ft high emergency flood boards that are designed to fail when overtopped at pool elevation 536 ft. The dam has a top elevation of 540 ft along the west embankment, and a top elevation

544.5 ft along the east embankment of the dam. The dam also contains a 19-ft by 12-ft bottom hinged trash gate.

The Lloyd Shoals Dam PMF was restudied in 1982 and then again in 1989 by Georgia Power Company with the probable maximum storm being based on the transposition of the March 11-16, 1929 storm centered near Elba, Alabama to the Sinclair Basin. The rainfall volume was increased by 119.5% to ensure the upstream watershed was fully enveloped.

The 1,400-mi^2 basin upstream of Lloyd Shoals Dam was routed through the reservoir by applying the effective rainfall depths in 6-hr increments to the unit hydrograph developed in the previous PMF study. The effective rainfall depth was determined by assuming that all initial infiltration losses had been satisfied due to antecedent rains with an average infiltration rate of 0.05 in/hr. The reservoir starting water surface elevation was assumed to be at full pond (530 ft) at the start of the storm. The outflow through the spillway and pool elevation was estimated by Southern Company Services through the use of a spillway rating curve and a reservoir volume curve.

The gate system allows large debris to pass over the spillway in high flows, and the lake is managed to facilitate boating and recreational activity on the reservoir. Floating debris is not expected to become a problem that could significantly impact the effectiveness of the spillway and therefore was not considered (Reference 8).

<u>Dam Stability Analysis</u>

The Lloyd Shoals Dam was evaluated by Southern Company Services for the loadings expected during the Plant Hatch PMF elevation. The intake section, spillway, and east and west non-overflow sections were estimated to be stable and the overtopping velocities will not cause erosion that would lead to failure (Reference 8).

Lake Juliette Dam

<u>Probable Maximum Flood and Reservoir Capacity</u>

Lake Juliette is classified as a Category I "very large dam" by the State of Georgia, and was designed to pass the PMF with a minimum freeboard of 3 ft. The dam has a top elevation of 450 ft, and a 100-ft free overflow spillway at elevation 437.5 ft.

The drainage area for Lake Juliette is 37.27 square miles. The PMP developed for the design of the Juliet Dam assumed that there was no reduction in rainfall from the 10-mi^2 PMP (31 inches) as allowed by HMR 52. Since the drainage area is greater than 10 square miles, the basin-averaged 6-hr PMP would be less than 31 inches. Furthermore, the PMF study conservatively assumed no losses in the watershed.

The PMF unit hydrograph for the Lake Juliette Dam was estimated following the guidelines described by the USGS for Georgia streams having less than 500-square-mile watersheds. The reservoir starting water surface elevation was assumed to be at full pond (437.5 ft) at the beginning of the storm. The outflow through the spillway and pool elevation was estimated by Southern Company Services through the use of a spillway rating curve, and a reservoir volume curve using the hydrologic modeling software, PondPack (Reference 8).

<u>Dam Stability Analysis</u>

The Lake Juliette Dam was evaluated by Southern Company Services for the loadings expected for a flood elevation of 444.1 ft. The dam was estimated to be stable during its PMF (Reference 8).

Wallace Dam

Probable Maximum Flood and Reservoir Capacity

Wallace Dam is a FERC-licensed dam (Project No. 2413) that is 1,323.33 ft long, with a top elevation of 445 ft. The spillway is controlled with five (5) 42-ft long and 44-ft high gates. The spillway and the dam have been designed to pass the PMF with adequate freeboard and structural stability.

The Wallace Dam PMF was restudied in 1990 by Southern Company Services with the probable maximum storm based on the transposition of the March 11-16, 1929 storm centered near Elba, Alabama to the Wallace Basin. The rainfall volume was increased by 117% to ensure the upstream watershed was fully enveloped. The temporal distribution for probable maximum storm (PMS) was arranged to form the critical sequence following the guidance described in HMR 52. The Wallace Dam PMS is shown in Figure 12 below.

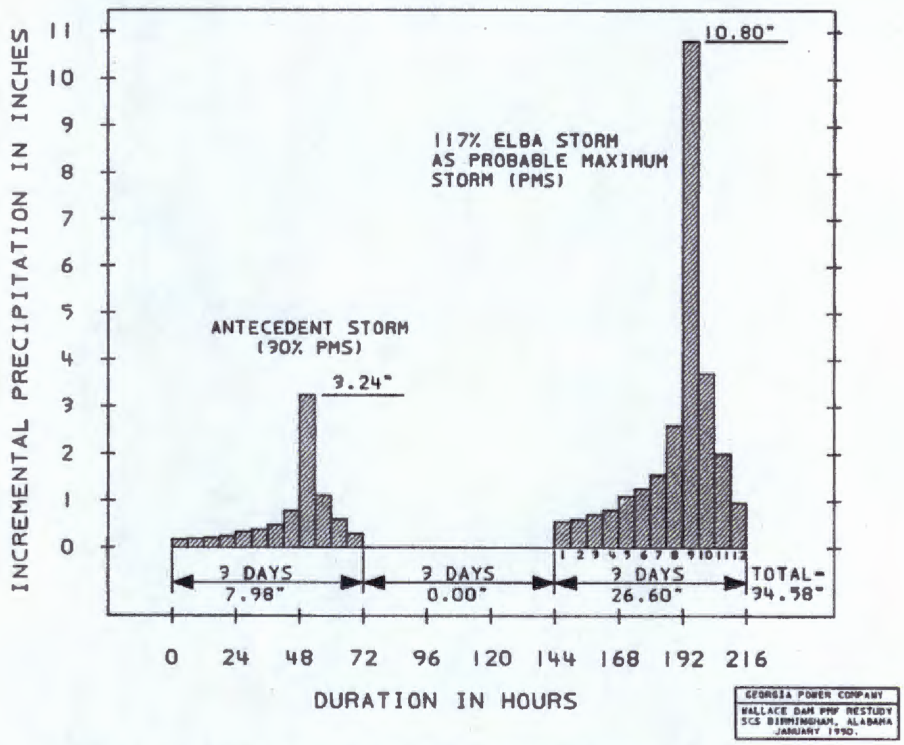

FIGURE 10 6-HOUR INCREMENTAL PRECIPITATION DISTRIBUTION PATTERN OF ANTECEDENT STORM AND PROBABLE MAXIMUM STORM FOR WALLACE BASIN (1830 SQ. MILES)

Figure 12: Wallace Dam PMS Distribution (1990 Wallace Dam Restudy)

The 1,830-square-mile basin upstream of Wallace Dam was divided into five (5) subbasins, and used as an input into a USACE HEC-1 hydrologic model to route the PMS. The infiltration loss rate of 0.05 in/hr was assumed for each subbasin. A mean annual flow was assumed to be the baseflow for the antecedent storm with a user specified recession decay rate between 1.03 and 1.21 depending on the subbasin. The PMF Inflow through the Wallace Dam reservoir was estimated by routing the inflow to the reservoir from the five upstream subbasins using the Modified Puls method. The outflow through the spillway and pool elevation was estimated by Southern Company Services through the use of a spillway rating curve and a stage storage curve.

The results of the 1990 Wallace Dam restudy showed that the dam could safely pass its PMF inflow of 345,300 cfs with a maximum pool elevation 440.2 ft, which is 4.8 ft below the top of the dam. Figure 13 shows the relevant hydrographs from the above-mentioned restudy.

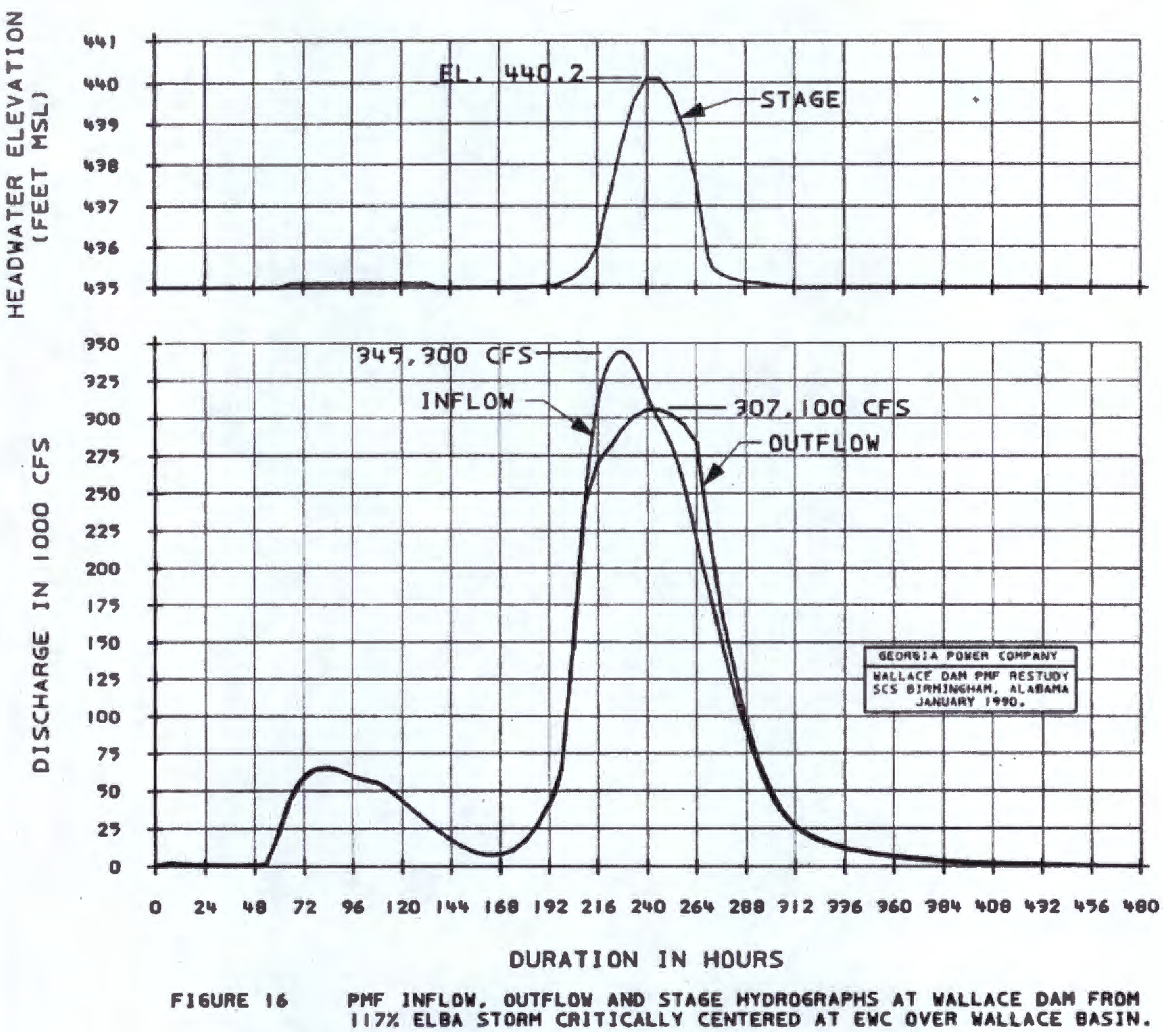

Figure 13: HEC-1 Hydrograph at Wallace Dam (1990 Wallace Dam Restudy)

Lake Oconee has not experienced significant floating debris field problems over more than 30 years since impoundment. The lake is managed to facilitate boating and recreational activity on the reservoir. Therefore, floating debris is not expected to impact the effectiveness of the five spillway gates (Reference 8).

Dam Stability Analysis

The Wallace Dam was evaluated by Southern Company Services for the loadings expected for a flood elevation of 441.0 ft (0.8 ft above the estimated PMF maximum pool elevation). The intake unit, bulkhead, spillway, and east and west non-overflow sections were assessed to be stable during the dam's individual PMF and meet the FERC stability requirements (Reference 8).

Sinclair Dam

Probable Maximum Flood and Reservoir Capacity

Sinclair Dam is a FERC-licensed dam (Project No. 1951) that consists of 1,596 ft of earth dike and 1,392 ft of concrete structure consisting of non-overflow walls, powerhouse, intake, and spillway. The dam has a top elevation of 355 ft. The spillway is controlled with twenty-four (24) 30 ft long by 21 ft high gates. The spillway and dam have been designed to pass the PMF with adequate freeboard and structural stability.

The Sinclair Dam PMF was restudied in 1985 and then again in 1987 by Georgia Power Company. The PMS in the restudy was based on the transposition of the March 11-16, 1929 storm centered near Elba, Alabama to the Sinclair Basin. The rainfall volume was increased by 124% to ensure the upstream watershed was fully enveloped.

The 2,910-square-mile basin upstream of Sinclair Dam was divided into seven (7) subbasins, which were modeled using the USACE HEC-1 hydrologic model. The infiltration loss rate of 0.08 in/hr was assumed for each subbasin. Subbasins 1 through 6 were combined and routed through the dam using the Modified Puls method. The outflow from the dam was combined with the outflow with subbasin 7 and routed through Lake Sinclair to the dam using the Modified Puls method. The reservoir starting water surface elevation was assumed to be at full pond (340 ft) at the start of the storm. The outflow through the spillway and pool elevation was estimated by Southern Company Services through the use of a spillway rating curve, and a reservoir volume curve.

The results of the 1987 Sinclair Dam restudy showed that Sinclair Dam could safely pass its PMF inflow of 516,566 cfs with a maximum pool elevation 349.3 ft, which is 5.7 ft below the top of the dam. The freeboard provides additional capacity for contingencies.

Lake Sinclair has not experienced significant floating debris field problems over more than 50 years since impoundment. The lake is managed to facilitate boating and recreational activity on the reservoir. Therefore, floating debris is not expected to become a problem that could significantly impact the effectiveness of the twenty four spillway gates (Reference 8).

Dam Stability Analysis

The Sinclair Dam was evaluated by Southern Company Services for the loadings expected for a flood elevation of 349.3 ft. The Intake section, the spillway, and the east and west non-overflow sections were estimate to be stable during its PMF, and meet the FERC stability requirements (Reference 8).

Hypothetical Dams

To assess the sensitivity of including hypothetical dams in the model and to assess the impact that they have on the peak breach discharge at Plant Hatch, the HEC-HMS model containing hypothetical dams was compared to the governing value presented in the Calculation SCNH-13-018 (Reference 7).

Table 12: Results of Sensitivity Analysis at Baxley Gage

Overtopping Dam Failure Scenario	Peak Date	Peak Time	Peak Discharge (cfs)	Depth over Watershed (in)
HEC-HMS model updated with hypothetical dams	18-Mar-00	11:00	877,482	25.5
Calculation SCNH-013-018 Governing Peak Flow (no hypothetical dams included)	18-Mar-00	12:00	899,636	25.7

The results indicate that the hypothetical dams result in flow attenuation and decrease in peak flow by 22,154 cfs. In addition, various scenarios were modeled as part of the analysis to determine the critical dam breach PMF (Reference 8). Figure 14 shows the peak discharges associated with the various overtopping dam breach scenarios.

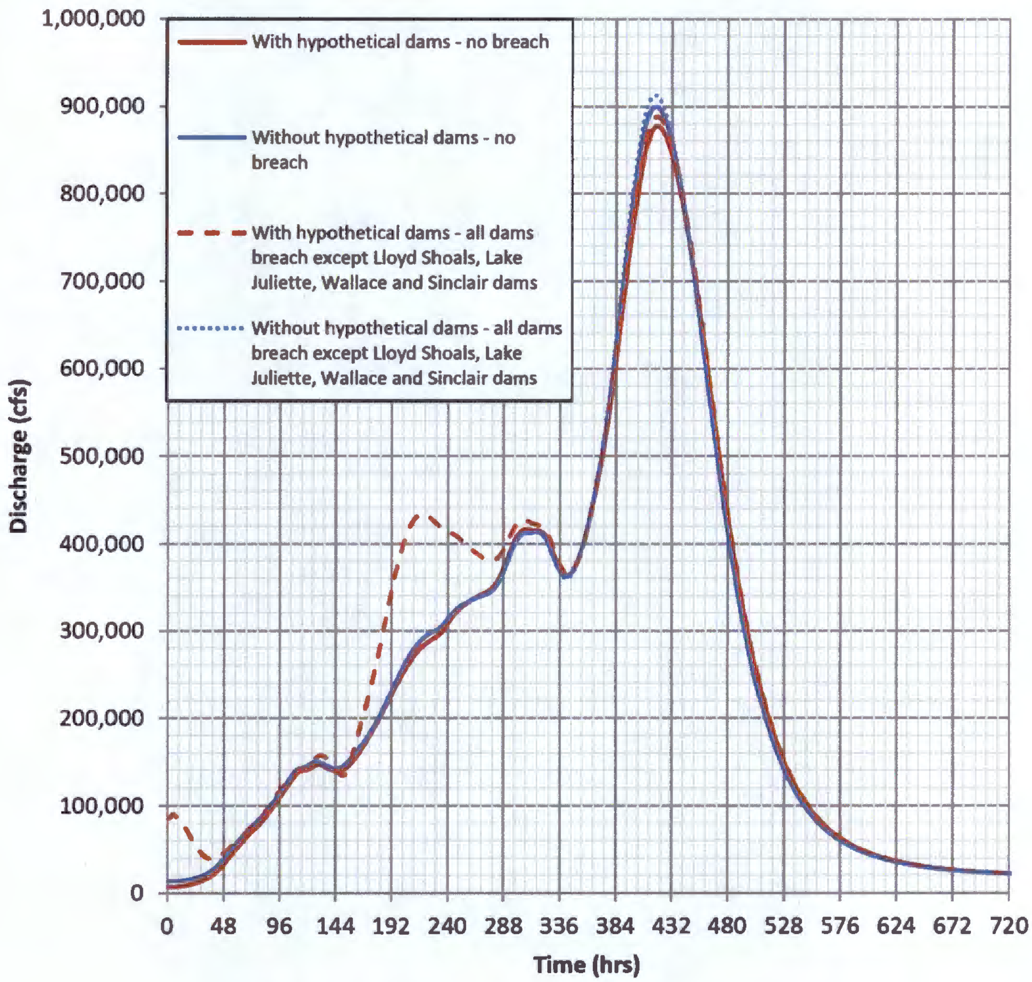

Figure 14: Overtopping Dam Breach Hydrographs at Baxley Gage

The results of the sensitivity analysis indicate that the inclusion of hypothetical dams in the HEC-HMS model to account for flood storage associated with smaller dams in the watershed causes a decrease in the peak discharge. This unintended decrease is caused by attenuation of flood flows during the routing through the reservoirs of hypothetical dams. As such, the critical overtopping dam breach scenario did not include modeling of hypothetical dams (Reference 8).

The resulting HEC-HMS hydrographs for the governing overtopping dam failure scenario were used as inputs for the unsteady flow HEC-RAS model to estimate the PMF discharge and stage at Plant Hatch (Reference 7).

The peak flow rate for the overtopping dam breach scenario was calculated to be 732,155 cfs, which corresponds to maximum water surface elevation of 109.72 ft. The maximum right overbank velocity was calculated to be 1.34 fps (Reference 12).

The time vs. discharge-stage hydrograph for the governing PMF with dam breach at the Intake Structure is presented in Figure 15.

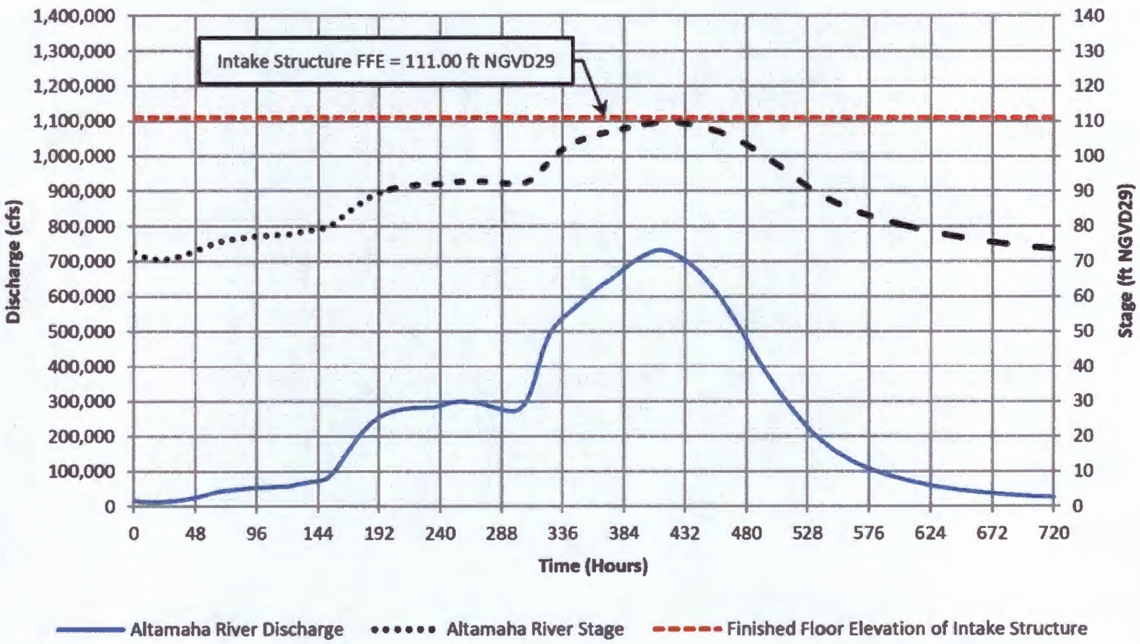

Figure 15: Time vs. Discharge-Stage Relation for Governing PMF with Dam Break at Intake Structure

d. Storm Surge

Storm surge is the rise of offshore water elevation caused principally by the shear force of the hurricane or tropical depression winds acting on the water surface. Plant Hatch is located approximately 82 miles upstream of tidal influence (which extends approximately 35 miles upstream of the mouth of the river) and, therefore, is not susceptible to storm surge flooding.

e. Seiche

Seiche is an oscillation of the water surface in an enclosed or semi-enclosed water body initiated by an external cause. The Altamaha River is not a semi-enclosed body of water and, therefore, is not susceptible to seiche flooding.

f. Tsunami

Tsunami is a series of water waves generated by a rapid, large scale disturbance of a water body due to seismic, landslide or volcanic tsunamigenic sources. Plant Hatch is located approximately 82 miles upstream of tidal influence (which extends approximately 35 miles upstream of the mouth of the river) and, therefore, is not susceptible to storm surge flooding.

g. Ice Induced Flooding

Ice jams and ice dams can cause flooding by impounding water upstream of a site and subsequently collapsing or downstream of a site impounding and backing up water. There is no method to assess a probable maximum ice jam or ice dam. Therefore, historical records are generally used to determine the most severe historical event in the vicinity of the site. Based on the river temperature data for the Altamaha River at Doctortown, Georgia, the minimum temperature of record is 37.4°F and is safely above the freezing temperature (Reference 13). Therefore, the formation of fragile ice is unlikely and ice induced flooding or blockage of the Intake Structure is not considered possible.

h. Channel Migration or Diversion

The flood hazard associated with channel diversion is due to the possible migration either toward the site or away from it. For natural channels adjacent to the site, historical and geomorphic processes should be reviewed for possible tendency to meander. For man-made channels, canals or diversions used for the conveyance of water located at a site, possible failure of these structures should be considered.

As indicated in NUREG/CR-7046 Section 3.8 (Reference 3), there are no well-established predictive models for channel diversions and, therefore, it is not possible to postulate a probable maximum channel diversion event. A qualitative evaluation of the tendency of the Altamaha River to meander was performed based on a review of available historic data. The channel alignment from the U.S. Highway Route 1 bridge to Plant Hatch appears stable and has not shown a historic tendency to meander. Channel meandering and diversion of the Altamaha River away from the site could be a possibility due to potential sedimentation; however, observations of river patterns showed that meanders develop very slowly and any potential impacts to the Intake Structure and water supply could be remedied as they occur (Reference 11).

In addition, a quantitative evaluation of sedimentation volumes was calculated based on available bathymetric survey provided by Georgia Power Company. The potential for sedimentation appears to increase with increased mean stream flow as sediment is transported from upstream and then deposited downstream of the U.S. Highway Route 1 bridge as the channel widens and velocities decrease. Continuous monitoring of the channel bottom conditions on a yearly basis by Georgia Power Company and dredging on an as needed basis reduces the likelihood of channel migration away from the site (Reference 11).

i. Combined Effect Flood

The combined effect flood was evaluated for the critical combination of flood causing mechanisms, which was determined to be Scenario 1 of NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3). Scenario 1 included a combination of mean monthly base flow; antecedent rainfall equal to 40% PMP; and the PMP. As indicated in the Dam Breaches and Failures section, Scenario 1 results in overtopping and breach of several upstream dams with a discharge of 732,155 cfs at Plant Hatch and a maximum water surface elevation of 109.72 ft at the Intake Structure (Reference 12). Therefore, wind generated waves caused by the 2-year wind speed in the critical direction were applied to the flood stage associated with the overtopping dam breach to determine the resultant water surface elevations during the combined effects flood.

Wind-Generated Waves

Per NUREG/CR-7046, the 2-year wind speed was applied in the critical direction on top of highest stillwater elevation produced by the governing flooding scenario. A fetch length of 26.17 miles was determined to be the longest possible open water path to the Intake Structure coincident with the stillwater elevation estimated in Calculation No. SCNH-13-082 (Reference 12).

The 2-year wind speed of 50 mph at 30 ft above ground was obtained from Figure 1 of ANSI/ANS-2.8-1992 (Reference 4). The topographic data was assembled using LiDAR data, bathymetric data, and USGS DEM Data (Reference 12).

The analysis was performed using equations from the USACE Coastal Engineering Manual (CEM) EM 1110-2-1100. The floodplain associated with the stillwater elevation was extended and approximated beyond the extents of the HEC-RAS model to capture the entire fetch length.

Wind-generated wave heights during the PMF were computed using the simplified wave prediction method in EM 1110-2-1100. The maximum wave height was determined in accordance with section 7.4.3 of ANSI/ANS 2.8 (Reference 4).

The classification of the wave conditions along the critical fetch was estimated following the equations described in EM-1110-2-1100 (Reference 6). The wave classification was estimated to be transitional based on the ratio of average depth to the wavelength per EM-1110-2-1100 (Reference 6).

The wind set-up is the estimation of the "piling up" of water on the leeward end of a fetch. The wind set up was estimated using equation provided in the United States Bureau of Reclamation (USBR) *Guidelines for computing Freeboard Allowances for Storage Dams*.

The characteristic wave height associated with a fetch limited wave period of 5.0 seconds and a wind velocity of 50.65 mph to the Intake Structure was estimated to be 9.2 ft crest to trough, with a maximum (1%) wave height of 15.4 ft crest to trough. The wind setup was estimated to be approximately 1.2 ft. The reinforced concrete walls of the Intake Structure will not be impacted by wave runup and the labyrinth design of the Intake Structure doors protects them from the direct effects of wave runup. As such, the interior components of the Intake Structure would only be subject to the maximum stillwater elevation (109.72 ft) rising through the pump well (Reference 10).

The resulting combined effect maximum water surface elevation at the Intake Structure, including the maximum wave height and wind setup, is 118.6 ft. This value represents the elevation associated with a 2-year wind coincident with an overtopping breach of upstream dams during the PMF (Reference 10).

Hydrodynamic Load

The PMF combined with upstream overtopping dam failures maximum water surface elevation is estimated to be 109.72 ft, and is below the finished floor elevation of the lowest safety-related structure (Intake Structure is at elevation 111.0 ft). Based on this evaluation, the Plant Hatch site is not flooded during the combined event and, therefore, is not susceptible to hydrodynamic loads from flooding.

Debris Load

The PMF combined with upstream overtopping dam failures maximum water surface elevation is estimated to be 109.72 ft, and is below the finished floor elevation of the lowest safety-related structure (Intake Structure is at elevation 111.0 ft). Based on this evaluation, the Plant Hatch site is not flooded during the combined event and, therefore, is not susceptible to debris loads from flooding.

6. COMPARISON WITH CURRENT BASIS FLOOD HAZARD

The current and reevaluated flood causing mechanisms at the site were compared to assess whether the reevaluated flood hazard is bounded by the current design basis flood elevation. The comparison is provided in Table 13.

Table 13: Summary Comparison with Current Licensing Basis Flood Hazard

Flood Causing Mechanism	Current Licensing Basis Flood Hazard Elevation	Flood Hazard Reevaluation Elevation	Licensing Basis Bounds Reevaluation Flood Hazard?
Local Intense Precipitation	Below Plant Finished Floor Elevation (water surface elevation not provided)	Varies between 130.05 ft and 131.23 ft in the power block and between 110.92 ft and 111.20 ft around the Intake Structure	Not Bounded
Flooding in Streams and Rivers	105 ft	109.58 ft	Not Bounded
Dam Breaches and Failures – Overtopping Failure	105.3 ft	109.72 ft	Not Bounded
Dam Breaches and Failures – Seismic Failure	Not addressed by design basis	106.24 ft	Not Bounded
Storm Surge	Not an applicable flood causing mechanism	Not an applicable flood causing mechanism	Not Applicable
Seiche	Not an applicable flood causing mechanism	Not an applicable flood causing mechanism	Not Applicable
Tsunami	Not an applicable flood causing mechanism	Not an applicable flood causing mechanism	Not Applicable
Ice Induced Flooding	Not an applicable flood causing mechanism	Not an applicable flood causing mechanism	Not Applicable
Channel Migration or Diversion	Screened Out	Screened Out	Not Applicable
Combined Effect Flood (PMF with overtopping dam failure with wind-induced waves)	108.3[†] ft	118.6 ft	Not Bounded

[†] The design basis combined effect flood was a result of the PMF discharge and a coincident wave activity and did not include overtopping dam failure.

7. INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED TO ADDRESS ANY HIGHER FLOODING HAZARDS RELATIVE TO THE DESIGN BASIS

The NRC 10 CFR 50.54(f) Request for Information letter dated March 12, 2012 (Reference 2) provides that flood hazard reevaluations are performed using updated flooding hazard information and present-day regulatory guidance and methodologies. Vulnerabilities identified during the flood hazard reevaluations were entered into the corrective action process and will be dispositioned accordingly.

Plant-specific vulnerabilities based on updated hazard assessments resulted in LIP and PMF levels that exceeded the existing Design Basis flood levels listed in the FSAR (Reference 13) and are conditions beyond the CLB. These do not call into question operability, and need not be reported to the NRC pursuant to 10 CFR 50.72 or 10 CFR 50.9. NRC notification of the flooding reevaluation results and any actions taken in response to them will be reported in the reevaluation report and the integrated assessment, if necessary, required by the 50.54(f) letter.

Re-evaluated LIP values resulted in water surface elevations at several doors exceeding the finished floor elevation of the Reactor Building, Turbine Building, Control Building, Diesel Building, as well as potential accumulation of water in the Intake Valve Pit. The LIP event also resulted in flooding of the ISFSI facility and covering the lower cask vents for approximately up to 21 hours. The re-evaluated PMF remains below the plant grade and the finished floor elevation of the Intake Structure (111 ft). However, the maximum crest of wind-generated waves during the PMF is at elevation 118.6 ft. The reinforced concrete walls of the Intake Structure will not be impacted by the waves and the wave runup. In addition, water inside the pump well could only rise up to the stillwater elevation of 109.72 ft, which is below the elevation of the pump motors. Wave splashing and wave refraction could potentially result in accumulation of water in the Intake Valve Pit.

a. Interim Corrective Actions to mitigate the Beyond Design Basis LIP values are as follows:

   Revise station procedures to include the following:

   - Add a table of the affected doors and areas contained in the LIP evaluation.
   - Provide guidance for sandbagging items contained in the table prior to the event.
   - Develop Administrative Controls for maintaining an adequate supply of sand for items listed in the table.

   The existing procedure guidance to address ISFSI facility flooding is adequate since the re-evaluated flood duration does not exceed 64 hours. No further revision is required for ISFSI flooding.

b. Interim Corrective Actions to mitigate the Beyond Design Basis PMF values are as follows:

   Revise station procedures to address the following:

   - A Beyond Design Basis flood, its predictability, and the site's ability to perform necessary actions prior to flood arrival.
   - The length of time before arrival of a Beyond Design Basis flood, duration of event, and when to secure Plant Service Water (PSW) pumps based on increasing river level.
   - When to shutdown the reactor and cooldown the reactor to Cold Shutdown.
   - Available alternative injection sources as necessary to provide Core Cooling and SFP Make-up/Cooling.
   - Pre-staging of portable injection pumps when necessary to provide Core Cooling and SPF Make-Up/Cooling.

- When and How to pump down the Intake Valve Pit.
- Referring the Operator to the appropriate procedure if an Extended Loss of All AC Power (ELAP) occurs as a result of the Beyond Design Basis flood
- Referring the Operator to the appropriate procedure for Shutting down PSW system when required

The intake FLEX Header used in Hatch's Primary FLEX Core Cooling Strategy may be submerged during the wave run-up associated with the Beyond Design Basis flooding event; therefore, use of the existing OIP Alternate FLEX Strategy for Core Cooling and SPF Cooling will be necessary. Upcoming FLEX procedures will be developed to address the potential use of the OIP Alternate Strategy during Beyond Design Basis floods.

c. Existing procedures are to be revised within 60 days to incorporate required changes.

8. REFERENCES

1. U.S. Nuclear Regulatory Commission. *Guidance for Assessment of Flooding Hazards Due to Dam Failure*. JLD-ISG-2013-01. Interim Staff Guidance, Revision 0. July 29, 2013.

2. U.S. Nuclear Regulatory Commission. Letter to Licensees. *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*. March 12, 2012.

3. U.S. Nuclear Regulatory Commission. NUREG/CR-7046, PNNL-20091, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America.* ML11321A195, November 2011.

4. American Nuclear Society (ANS). 1992. *American National Standard for Determining Design Basis Flooding at Power Reactor Sites*. Prepared by the American Nuclear Society Standards Committee Working Group ANS-2.8, La Grange Park, Illinois.

5. Xu, Y. and Zang, L. M., "Breaching Parameters for Earth and Rockfill Dams", Journal of Geotechnical and Geoenvironmental Engineering, December 2009.

6. United States Army Corps of Engineers (2008). Engineering Publication EM 1110-2-1100.

7. Southern Nuclear Operating Company Calculation Package SCNH-13-018, "Hydrologic Evaluation for Plant Hatch - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 2.0, January 2013.

8. Southern Nuclear Operating Company Calculation Package SCNH-13-019, "Evaluation of Plant Hatch Dam Breaches and Failures - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 1.0, December 2013.

9. Southern Nuclear Operating Company Calculation Package SCNH-13-021, "Evaluation of Plant Hatch Local Intense Precipitation - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 1.0, November 2013.

10. Southern Nuclear Operating Company Calculation Package SCNH-13-022, "Plant Hatch Wind-Generated Wave Analysis - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 1.0, January 2013.

11. Southern Nuclear Operating Company Calculation Package SCNH-13-023, "Plant Hatch Channel Migration or Diversion Evaluation - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 1.0, August 2013.

12. Southern Nuclear Operating Company Calculation Package SCNH-13-082, "Hatch Probable Maximum Flood Hydraulics - Severe Accident Management (SAM) for Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 Flooding Re-evaluation." Version 1.0, January 2013.

13. Southern Nuclear Operating Company. "Edwin I. Hatch Nuclear Plant, Units 1 & 2 Final Safety Analysis Report (FSAR) Rev 21." 2003.

14. Southern Nuclear Operating Company. "Hatch Units 1 and 2 Flooding Walkdown Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding." 2013

15. FLO-2D Reference Manual, Version 2009.

16. U.S. Weather Bureau (1982), "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian," Hydrometeorological Report No. 52.

17. Georgia Department of Natural Resources-Environmental Protection Division (2001). Georgia Stormwater Management Manual, Volume 2-Technichal Handbook, Appendix H.

18. U.S. Army Corps of Engineers, Hydrologic Modeling System HEC-HMS computer software. Version 3.5. Hydrologic Engineering Center, Davis, CA. August 2010.

19. National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Precipitation Frequency Estimates in GIS Compatible Format. http://dipper.nws.noaa.gov/hdsc/pfds/pfds_gis.html.

20. Froehlich, D. C. "Embankment Dam Breach Parameters and Their Uncertainties," Journal of Hydraulic Engineering, Vol. 134, No. 12, May, pgs 1708-1720. 2008.