2.4 Hydrologic Engineering

Section 2.4 describes the hydrological characteristics of the Lee Nuclear Site. The site location and description are provided in Section 2.4 of this report in sufficient detail to support the safety analysis. This section discusses characteristics and natural phenomena that have the potential to affect the design basis for the Westinghouse AP1000 reactor (AP1000) units. The section is divided into the following 14 subsections:

- 2.4.1 Hydrologic Description.
- 2.4.2 Floods.
- 2.4.3 Probable Maximum Flood on Streams and Rivers.
- 2.4.4 Potential Dam Failures.
- 2.4.5 Probable Maximum Surge and Seiche Flooding.
- 2.4.6 Probable Maximum Tsunami Flooding.
- 2.4.7 Ice Effects.
- 2.4.8 Cooling Water Canals and Reservoirs.
- 2.4.9 Channel Diversions.
- 2.4.10 Flood Protection Requirements.
- 2.4.11 Low Water Considerations.
- 2.4.12 Groundwater.
- 2.4.13 Accidental Releases of Liquid Effluents in Ground and Surface Waters.
- 2.4.14 Technical Specifications and Emergency Operation Requirements.

2.4.1 Hydrologic Description

Information provided in this subsection includes descriptions of the site and its features, hydrosphere, hydrologic characteristics, drainage, dams and reservoirs, water management changes, and surface water uses.

2.4.1.1 Site and Facilities

The 1900-acre (ac.) Lee Nuclear Site is located south and west of the Broad River in eastern Cherokee County, South Carolina (Figure 2.2-201). The nuclear island for the Lee Nuclear Station is located south and west of the Ninety-Nine Islands Reservoir portion of the Broad River, approximately 1 mile (mi.) due northwest of the Ninety-Nine Islands Dam. In addition to the Broad River and several tributaries, the Ninety-Nine Islands Reservoir, Make-Up Pond B, Make-Up Pond A, and Hold-Up Pond A (Figure 2.4.1-201) make up the majority of the surface water features in the

vicinity of the site. Make-Up Pond C is an off-site facility, located on a tributary of the Broad River, west of the Lee Nuclear Station (Figure 2.4.1-213).

2.4.1.1.1 Previous Construction Activities

The Lee Nuclear Site, formerly known as Cherokee Nuclear Station, was evaluated for and received a construction permit from the U.S. Nuclear Regulatory Commission to construct three Combustion Engineering System 80+ nuclear units. Approximately 750 ac. of ground were disturbed during the 1977-1982 construction activities, which resulted in extensive alteration of the site. This alteration included vegetation clearing; establishment of on-site construction roads; establishment of a railroad spur to the site; extensive excavation and grading with heavy equipment; building of on-site warehouses, shops, and construction support facilities; and construction of power unit buildings (portion of one power block building and about half of its associated cylindrical reactor containment/ shield building). About 25 ac. were excavated into underlying bedrock for construction of the reactor units.

The site currently consists of open, partially-developed industrial land with low groundcover vegetation and scattered areas of sparse tree growth. However, the terrestrial environment surrounding the site consists primarily of deciduous hardwood forest and farms. The aquatic environs are dominated by the Broad River and the Ninety-Nine Islands Reservoir.

The Lee Nuclear Station is planned within the large, open, contiguous area of land that was cleared for previous construction activities on the site. The partially built reactor containment building is to be razed prior to new construction. The base mat slab and several warehouses will be kept. Construction of the intake structure is planned on the Broad River, and the blowdown discharge sparger is planned on upstream side of Ninety-Nine Islands Dam.

2.4.1.1.2 Plant Design

Duke Energy selected the AP1000 certified plant design for the Lee Nuclear Station combined operating license application. The AP1000 units (Units 1 and 2) are planned to be in the vicinity of the previously proposed Cherokee Units 1 and 3. The AP1000 is rated at 3400 megawatts thermal (MWt) with a minimum electrical output of 1000 megawatts electrical (MWe). Each unit uses two mechanical draft towers for circulating water system cooling with the intake system providing all raw water requirements. During normal flow conditions raw water is pumped from Broad River raw water intake structure to Make-Up Pond A through the raw water discharge structure. During low-flow conditions raw water from Make-Up Pond B is pumped from the Make-Up Pond B intake structure to Make-Up Pond A through the raw water discharge structure. If Make-Up Pond B usable storage is not sufficient to meet plant needs, Make-Up Pond C is then used to supply supplemental water. Water is pumped from the Make-Up Pond C intake structure to a discharge structure in Make-Up Pond B and then is pumped from Make-Up Pond B to Make-Up Pond A, as previously described. The ultimate heat sink for the Lee Nuclear Station is the atmosphere.

2.4.1.1.3 Safety-Related Structures

The plant arrangement is comprised of five principal structures (as described in Subsection 1.2.1.6): nuclear island, turbine building, annex building, diesel generator building, and radioactive waste building. Of the five principal structures, only the nuclear island is designed to Category I seismic requirements, and it contains all safety-related equipment for accident mitigation. The nuclear island

consists of a free-standing steel containment building, a concrete shield building, and an auxiliary building. The foundation for the nuclear island is an integral basemat that supports these buildings.

The AP1000 reference floor elevation of 100 ft. corresponds to the nuclear island finished floor elevation set at 593 ft. above msl. Therefore, the nuclear island basemat elevation is 553.5 ft. above msl. Yard grade elevation is 592 ft. above msl, which keeps water from pooling in areas of safety related structures (Subsection 2.4.2.3). An extensive site stormwater drainage system is planned and is slated for implementation before the construction commences on Units 1 and 2. The elevations of safety-related components are presented on Table 2.4.1-201.

2.4.1.1.4 Plant Water Systems

Plant water consumption and water treatment for the Lee Nuclear Station are determined from the AP1000 Design Control Document, site characteristics, and engineering evaluations. The raw water system supplies water to Make-Up Pond A for plant use, including make-up to the circulating water system (CWS) cooling towers, to makeup for water consumed as a result of evaporation, drift and blowdown. The raw water intake structure is located on the west bank of the Broad River, north-northeast of Unit 2 (Figure 2.4.1-201). The raw water discharge structure is located at the north end of Make-Up Pond A near the Unit 2 cooling towers. Water withdrawn from the Broad River is pumped into Make-Up Pond A and from there enters the make-up water intake structure. Raw water is also processed through the clarifier and used in plant water systems including the service water system, the demineralized water treatment system and the fire protection system. Effluent from the Lee Nuclear Station is to be diffused into the river at the upstream face of the Ninety-Nine Islands Dam near the intakes for the hydroelectric station (Reference 256), avoiding recirculation of the plant effluent to the intake structure located approximately 1.25 river miles upstream (Figure 2.4.1-201).

<u>Intake System</u>

The intake system provides all raw water requirements for the plant. During normal flow conditions, raw water is pumped from the Broad River raw water intake structure to Make-Up Pond A through the raw water discharge structure. During low flow conditions, raw water from Make-Up Pond B is pumped from the Make-Up Pond B intake structure to Make-Up Pond A through the raw water discharge structure. If Make-Up Pond B usable storage is not sufficient to meet plant needs, Make-Up Pond C is then used to supply supplemental water. Water is pumped from the Make-Up Pond C intake structure to a discharge structure in Make-Up Pond B and then is pumped from Make-Up Pond B to Make-Up Pond A, as previously described.

After low flow conditions have ceased, Make-Up Pond B is replenished using water from the Broad River which is pumped into Make-Up Pond A and subsequently into Make-Up Pond B. Raw water is pumped from the Make-Up Pond A intake structure to Make-Up Pond B using the same piping to supply Make-Up Pond A with water from Make-Up Pond B. Water is discharged into Make-Up Pond B using the Make-Up Pond B intake structure. An alternative refill path is to use the refill pumps on the river intake structure that pump directly to Make-Up Pond B.

Make-Up Pond C is normally refilled directly from the river using the same refill pumps on the river intake structure that pump directly to Make-Up Pond B. The section of pipe between Make-Up Pond B and Make-Up Pond C is used to both supply Make-Up Pond B from Make-Up Pond C and to refill Make-Up Pond C from the river. Water is discharged into Make-Up Pond C using the Make-Up Pond C intake structure. An alternative refill path for Make-Up Pond C is to pump from the Broad River into Make-Up Pond A, then pump from Make-Up Pond A to Make-Up Pond B, and then pump

from Make-Up Pond B to Make-Up Pond C using a dedicated line only for refilling Make-Up Pond C. The intake, discharge, and pump structures for Make-Up Ponds A and B are shown in Figure 2.4.1-201. Make-Up Pond C is an off-site facility, located west of the Lee Nuclear Station, as shown in Figure 2.4.1-213.

The river intake structure serves as a platform to support trash racks, traveling screens, pumps, motors, and other equipment. Intake water taken from the Broad River passes through bar screens and traveling screens designed to minimize uptake of aquatic biota and debris. Each traveling screen has fish collection and return capability. Return of impinged fish is to a location downstream of the intake. Debris collected by the trash racks and traveling screens is collected and disposed of as solid waste (Reference 256).

The raw water requirements vary depending on the operating mode, therefore the flow rates and intake velocities also vary. During the first four modes of operation, which include power operation, startup, hot standby, and safe shutdown, both the CWS and the service water system (SWS) require makeup water. The raw water system (RWS) supplies an average of 35,030 gallons per minute (gpm) (60,001 gpm maximum) raw water flow as makeup to the CWS, the SWS, and the demineralized water treatment system for the two units. Flow to the fire protection system (FPS) and the waste water system (WWS) is intermittent. The screens are sized so that the average through-screen velocity is in accordance with the Section 316 (b) of the Clean Water Act. The intake velocity is less than 0.5 fps. For the remaining two modes of operation, cold shutdown and refueling, the flow rate and the intake velocity is less as only the SWS requires makeup water from the raw water intake. For these final two modes of operation, the flow rate is 650 gpm per unit and the intake velocity is negligible.

Discharge System

The primary purpose of the discharge system is to disperse cooling tower blowdown into the Broad River along with other wastewater streams to limit the concentration of dissolved solids in the heat rejection system. Any additives in the discharge are as approved by the U.S. Environmental Protection Agency (EPA) as safe for humans and the environment. The volume and concentration of the constituents discharged to the environment will meet the requirements established in the South Carolina Department of Health and Environmental Control (SCDHEC) administered National Pollution Discharge Elimination System (NPDES) permit.

Effluent from the Lee Nuclear Station is to be diffused into the river at the upstream face of the Ninety-Nine Islands Dam near the intakes for the hydroelectric generating units. This discharge includes non-radioactive process waste (including cooling tower blowdown) and low level liquid radioactive waste (at an average rate of 4 gpm within regulatory limits).

The discharge structure consists of a submerged pipe that is perforated for the last portion of its length, diffusing the effluent into the hydroelectric station intakes. The effluent discharge rate to the Broad River during normal operations is approximately 8216 gpm with a maximum plant water discharge rate of 28,778 gpm (for two units).

2.4.1.2 Hydrosphere

The location of the Lee Nuclear Station, as described in Subsection 2.4.1.1, falls within the Broad River basin. The Broad River and Ninety-Nine Islands Reservoir are the main hydrologic features that may affect or be affected by construction activities in the immediate vicinity of the Lee Nuclear

Site. Ninety-Nine Islands Reservoir is the nearest major body of surface water to the Lee Nuclear Site. This reservoir is an impoundment of the Broad River by Ninety-Nine Islands Dam. The Lee Nuclear Site is located adjacent to the reservoir, which surrounds the site to the north and east. Land along the south boundary of the site is private property. Current surface water features at the site include Make-Up Pond B, Make-Up Pond A, and Hold-Up Pond A. Make-Up Pond C is an off-site facility, located on a tributary of the Broad River, west of the Lee Nuclear Station. A brief description of local groundwater conditions is also provided in this subsection.

2.4.1.2.1 Physiography and Topography

The Lee Nuclear Site is located within the Piedmont physiographic province, a southwest to northeast-oriented province of the Appalachian Mountain System (Figure 2.4.1-203). The Piedmont province is 80 – 120 mi. wide, and it is situated between the Blue Ridge province, a mountainous region to the northwest, and the Atlantic Coastal Plain province to the southeast. The province is a seaward-sloping plateau, dominated by a monotonous topography of low rounded ridges with gentle slopes and ravines largely underlain by saprolite developed on crystalline rock.

The principal drainageway in the region of the Lee Nuclear Site is the Broad River. Near the larger streams, tributaries cut through deep and steep valleys that (when traced headward) become wide, shallow, and of gentle gradient. The regional southeastward drainage of the Broad River basin is reflected in the trend of the Broad River (Reference 220). The Piedmont region of the Broad River basin is a plateau of forested, rolling hills with tight, dissected river valleys that generally contain small floodplains. The tributaries of the Broad River generally follow a dendritic pattern before draining to the Broad River and eventually the Atlantic Ocean.

Construction activities at the former Cherokee Nuclear Site altered local topography to cut and fill the site to a yard-grade elevation of 588 ft. above msl. Following excavation in the power block area, site topography changed from hills and valleys to a relatively flat upland setting punctuated by a massive excavation to competent rock, which over time filled with water from both groundwater seepage and precipitation. Subsection 2.4.12.2.3 describes the dewatering of this excavation in support of exploration activities for the Lee Nuclear Station. Numerous springs and seeps identified during the 1973 investigation (Reference 214) were disturbed during the 1975 – 1982 construction activities for the Cherokee Nuclear Station. Those springs and seeps were located within valley draws and natural drainage ways. Surface conditions around these springs appear to have been altered so that no flow-through discharge occurs. The undisturbed topography remaining at the Lee Nuclear Site is generally characterized by rounded hilltops and narrow valleys with elevations ranging from 511 ft. at the Broad River to around 810 ft. along the ridgeline of McKowns Mountain, located west of the power block area and south of Make-Up Pond B.

2.4.1.2.2 Upper Broad River Watershed

The Broad River basin region, the Broad River, and the majority of its tributaries originate in the Blue Ridge Mountains of North Carolina and extend toward the foothills before entering the Piedmont ecoregion, all within the larger Santee River basin, U.S. Geological Survey (USGS) (six-digit Hydrological Unit Code [HUC] 030501) (Figure 2.4.1-204) (Reference 290).

The USGS divides the Broad River basin into the Upper Broad (HUC 03050105) and Lower Broad (HUC 03050106) River basins with the Lee Nuclear Site positioned within the Upper Broad River basin (Figure 2.4.1-204). The Upper Broad River basin is located in both North and South Carolina. The Broad River drainage basin above Ninety-Nine Islands Dam is located within the Upper Broad

River basin and includes the Green River, First Broad River, Second Broad River, and Buffalo Creek as major tributaries (Figure 2.4.1-205) (Reference 231). The drainage area of the Upper Broad River basin is approximately 2500 sq. mi. (Table 2.4.1-202) and is situated over the North Carolina-South Carolina state border. The drainage area of the Upper Broad River basin to Ninety-Nine Islands Dam (one-half river mile downstream from the site) is approximately 1550 sq. mi. (Reference 216).

Watershed elevations range from about 1200 ft. above msl at the headwaters of the First Broad River in the mountains of North Carolina to 620 ft. above msl when the Broad River crosses the North Carolina/South Carolina border. Watershed elevations along the Broad River continue to decrease southward to 511 ft. above msl upstream of Ninety-Nine Islands Dam, and 440 ft. below Ninety-Nine Islands Dam. At the confluence of the Broad River with the Saluda River in Columbia, South Carolina the elevation is 140 ft. above msl. The slope percentage of the Broad River is 0.55, and it has a gradient of 28.9 ft/mi (Reference 290).

The Broad River starts in Buncombe County, flows through Henderson, Rutherford and Cleveland counties in North Carolina and then into Cherokee County, South Carolina. In North Carolina, the basin encompasses most of Cleveland, Polk, and Rutherford counties and small portions of Buncombe, Henderson, Lincoln, Gaston, Burke and McDowell counties (Figure 2.4.1-206). Larger municipalities within the basin include the towns of Forest City, Kings Mountain, Lake Lure, Rutherfordton, Shelby, and Spindale. Approximately one-half of the basin is covered in forest; however, agriculture is still widespread (Reference 231).

2.4.1.2.2.1 Local Watersheds

The Broad River accepts drainage from Ross Creek (Sarratt Creek), Mikes Creek, Bowens River (Wylies Creek), the Buffalo Creek watershed, and the Cherokee watershed (Figure 2.4.1-207). Further downstream, Peoples Creek (Furnace Creek, Toms Branch) drains into the Broad River near the city of Gaffney. Doolittle Creek enters the river near the town of Blacksburg, followed by London Creek (which feeds Lake Cherokee and Make-Up Pond C, and has the Little London Creek as a tributary), Bear Creek, McKowns Creek (which feeds Make-Up Pond B at the site), Dry Branch, the Kings watershed, and Quinton Branch. Mud Creek enters the Broad River next, downstream from Mud Islands, followed by Guyonmbore Creek, Mountain Branch, Abington Creek (Wolf Branch, Service Branch, and Jenkins Branch), the Thicketty Creek watershed, Beaverdam Creek (McDaniel Branch), the Bullock Creek watershed, and Dry Creek (Nelson Creek). There are numerous ponds and lakes located off-site (totaling 246 ac., not including the approximately 620 ac. Make-Up Pond C) in this watershed (03050105-090) and all 133 stream mi. are classified as fresh water (Reference 268).

The Lee Nuclear Site is located in USGS Hydrologic Unit 03050105-090 of Cherokee and York counties, South Carolina, and this unit consists primarily of the Broad River and its tributaries from the North Carolina border to the Pacolet River (Figure 2.4.1-205). Land use/land cover in the watershed includes: 67.8 percent forested, 18.8 percent agriculture land, 5 percent scrub/shrub land, 4.5 percent urban land, 2.8 percent water, and 1.1 percent barren (Reference 268).

2.4.1.2.2.2 Broad River Description

The Broad River has a length of about 185 river mi. The drainage area of the Upper Broad River basin is approximately 2500 sq. mi. (North Carolina and South Carolina). The drainage area of the Upper Broad River basin to Ninety-Nine Islands Dam, one-half river mile downstream from the site, is approximately 1550 sq. mi. The Broad River drainage basin above Ninety-Nine Islands Dam includes

these major tributaries: Green River, First Broad River, Second Broad River, and Buffalo Creek (Reference 232).

The Broad River originates upstream of Lake Lure. Lake Lure Dam is located on the east side of Lake Lure, and the majority of the lake water is provided by the Broad River (also known as the Rocky Broad River).

The middle and lower portions of the Broad River in North Carolina cover about 40 river mi. from Lake Lure to the confluence of the Second Broad River near the Cleveland-Rutherford county line. Major tributaries in this section include the Green and Second Broad Rivers. The headwaters of these tributaries begin in the Mountains and then flow into the Piedmont ecoregion. Smaller tributary catchments of the Broad River include Mountain and Cleghorn creeks (Reference 232). The headwater reaches of the Green River are located in Henderson County, North Carolina.

Discharge Characteristics

The nature of flow in the Broad River was characterized by USGS gauging stations described in Table 2.4.1-203. The 2005 annual mean flows are also provided in Table 2.4.1-203 to illustrate the Broad River's gaining stream characteristics. USGS gauging stations are shown on Figure 2.4.1-205.

Broad River discharge recorded at the USGS Station No. 02153551 located just below Ninety-Nine Islands Dam ranged from 138 cubic feet per second (cfs) on September 14, 2002, to over 60,000 cfs in September 2004. Additionally, the Gaffney USGS Station (No. 02133500) located approximately 8 mi. north of the Lee Nuclear Site and having about 60 sq. mi. less drainage area than Ninety-Nine Islands Reservoir, detected the highest recorded flow on record of 119,100 cfs, recorded on August 14, 1940 (Reference 214).

Based on an 83-year period of record (1926 – 2008) for the Broad River at the Gaffney Station, an average annual flow of the Broad River was determined to be approximately 2500 cfs. The 83-year period of record was derived using three USGS stream gauges located on the Broad River. The Broad River gauge near Gaffney, SC (USGS 02153500) is located just upstream of the Lee Nuclear Site and has available data from 1938-1971 and 1986-1990. The Gaffney gauge data was used without correction for drainage area size and applied to the site.

The Broad River gauge near Blacksburg, SC (USGS 02153200) is located upstream from the Gaffney gauge and has available data from 1997-2008. The Blacksburg gauge data was corrected by a ratio of drainage areas for the Gaffney gauge to the Blacksburg gauge and then applied to the site. The Broad River gauge near Boiling Springs, NC (USGS 02151500) is located upstream from the Blacksburg gauge and has available data from 1926-2008. Only data from the absent years of the Gaffney and Blacksburg gauges were corrected by a ratio of drainage areas for the Gaffney gauge to the Boiling Springs gauge and then applied to the site. The overlapping data from the Boiling Springs gauge were not utilized.

Low-flow conditions on the Broad River are a function of natural flow in the rivers and streams, available storage capacity of upstream reservoirs, and regulated discharge flow from upstream dams. Low-flow conditions are generally defined as the lowest consecutive 7-day stream flow that is likely to occur every 10 years (7Q10). The 7Q10 was calculated with the same database described above to be 439 cfs using Log-Pearson Type III distribution (Subsection 2.4.11.5).

The South Carolina climate is subject to periodic droughts. Since 1900, severe droughts have occurred statewide in 1925, 1933, 1954, 1977, 1983, 1986, 1990, 1993, 1998, 2002, 2007, and 2008. The drought that officially began in June 1998 abated in the late summer of 2002 with the onset of the hurricane season. The effects of these droughts are reflected in the Broad River discharge characteristics. Low-flow conditions are further discussed in Subsection 2.4.11.

In September 2006, during a bathymetry study (Reference 298), water velocities were characterized in the vicinity of the intake structure. Station No. 02153551 (located below the Ninety-Nine Islands Dam) measured Broad River discharge ranging from 1960–3090 cfs at the time of this assessment. Bathymetry at the intake structure shows a narrow linear feature (i.e., scour hole) aligned along the direction of flow and appears to be approximately 30-ft. deep (elevation 480 ft. msl). This linear feature is located in a section of the Broad River channel that is approximately 240 ft. across. Water velocities were measured at seven stations along a transect crossing the Broad River perpendicular to the intake at channel depths of 1, 5, 10, and 15 ft. Water velocity around the intake structure had an average flow rate of 0.32 feet per second (fps) with a standard deviation of 0.04 fps. No water velocity measurements were obtained near the dam and the proposed plant discharge location due to access restrictions and safety considerations related to hydroelectric operations.

To supplement characterization of the Broad River as a heat sink for the discharge of cooling water blowdown, temperature data from USGS Station No. 02156500 (located near Carlisle, South Carolina, in Union County) were compiled and are presented in Table 2.4.1-204. For the period 1996 to 2006, the monthly water temperatures ranged from 40.8°F to 85.3°F (4.9°C to 29.6°C).

Generally, the Broad River flow conditions and discharge characteristics are consistent with those observed in the 1970's. As such, the bedforms and sediment transport observations presented in the Cherokee Nuclear Station Construction Permit Environmental Report (ER) are relevant today and are discussed below.

Bedforms

The bottom of the Broad River is influenced by the formation of bedforms. Bedforms are likely to be (1) scoured in bedrock, (2) formed from sand resulting in migrating dunes, (3) created from alluvial bed material of mixed sizes forming pools and riffles, or (4) produced by a combination of the above. Pools and riffles are the most common bedforms. At low flow, riffles are essentially flow-resistant dams forming each upstream pool. Water velocity over the riffles at low flow is considerably greater than that in adjacent pools. Therefore, fine sediment such as sand or silt is found on riffles.

At high flow, the stepped water surface characteristic of pools and riffles at low flow tends to disappear, and bedform conditions may be greatly altered from that found at low flow. At high flow, pools become areas of greater scour and thus may have similar water velocity as that found in the adjacent riffle areas. Although pools are quiet environments similar to impoundments during low flow, they generally have a high water velocity at the center of the river and the outside bends of the river. During high river flows the riparian vegetation and inside bends of the river provide the low velocity regions typically provided by the pools at low flow. The boundary between a pool and the adjacent riffles is primarily a function of discharge. The basic morphology of these forms does not change through exposure to a variety of flow levels. The most distinct break is between a riffle and an upstream pool; the deepest part of the pool is likely to be fairly close to the adjacent downstream riffle (Reference 214).

Bedform surveys for areas on the Broad River upstream and downstream of the Lee Nuclear Station were conducted in the 1970s. Between the Gaston Shoals impoundment and U.S. Highway 29

(U.S. 29), the Broad River channel was characterized by pools and riffles. The riffles were bedrock ridges cut into felsic schist. The bed material in pools and moving through riffles was entirely composed of uniform sand. Between U.S. 29 and Cherokee Falls, a resistant outcrop of felsic gneiss formed a long, continuous area of shallow riffles in which no pools had developed. From Cherokee Falls to Ninety-Nine Islands Reservoir, the stream was again characterized by bedrock highs (riffles) formed from schist, alternating with deeper pools in which the substrate material was nearly all sand. Below the reservoir another resistant gneiss bedrock outcrop created a long, continuous shallow riffle area that gave way downstream to more pools and riffles. Below the Irene Bridge, the pools became larger and much longer while the riffles became smaller and less conspicuous. This dominance of pools was accompanied by steeper river banks, a diminution of sand beds, and the introduction of silt and mud substrates in the pools (Reference 214).

In summary, alternating pools and riffles cut in bedrock are the dominant bedforms of the Broad River above and below the Lee Nuclear Station. Where bands of resistant gneiss cross the course of the river, they create anomalous shallow riffles. The bedload is mostly coarse sand, making scoured rock outcrops and sand beds the two common substrate types (Reference 214).

Sediment Transport

The Broad River is generally wide and fairly shallow (Figure 2.4.1-208), and it normally carries a high bedload composed mainly of sands with some coarse gravels and cobbles. Water samples were collected in the early 1970s to estimate the suspended sediment load in the river for the Cherokee Nuclear Station Construction Permit Environmental Report (ER). Samples were collected from October 1973 through September 1974.

Sample results from Station 8, located just above the proposed site (Figure 2.4.1-206), ranged from 20 to 282 mg/L and an average sediment concentration of 73.9 mg/L, with a standard deviation of 63.3 mg/L (Reference 214). In a study conducted in 1989 – 1990 for the Ninety-Nine Islands Dam license renewal, the Broad River exhibited a mean TSS of 41 mg/L, ranging from 6 to 243 mg/L (Reference 216). Suspended solids concentrations can vary widely as a function of stream flow.

Analytical results from samples collected quarterly in 2006 show a mean TSS concentration of 11.5 mg/L. TSS concentrations ranged from 1 to 62 mg/L with a standard deviation of 12.4 mg/L. The waters within the main channel of the Broad River near the intake structure exhibited a mean TSS concentration of 10.2 mg/L. Additional sampling of the Ninety-Nine Islands Reservoir, conducted in 2007, reported a TSS range of less than 4 to 204 mg/L. Particle size analyses of suspended solids revealed a range from 0.00035 (clay) to 0.35355 millimeters (mm) (medium grade sand). From the five water samples collected and analyzed, the average of their median particle sizes was 0.0171 mm (medium silt) with a settling velocity calculated to be 0.0001 feet per second (fps).

The values used for the design basis are an average TSS concentration of 20 mg/L and a maximum TSS concentration of 300 mg/L, based on current Broad River data from Duke’s surrounding power plants.

Modeling studies conducted for the water intake structure of the former Cherokee Nuclear Station demonstrated that local flows near the intake are expected to deter significant sediment accumulation in the local scour hole near the intake structure. However, this same study noted some bedload sediment deposits in the intake structure as a result of pump operations and high-flow events, which will require annual maintenance dredging. Dredging would usually be limited to approximately 150 cubic yards (cu. yd.) annually.

2.4.1.2.2.3 Major Tributaries

The four major tributaries of the Broad River above the Lee Nuclear Site include the First Broad River, Second Broad River, Green River, and Buffalo Creek (Figure 2.4.1-206) (Reference 233).

First Broad River

The First Broad River originates in Rutherford County and flows into the Broad River in Cleveland County, North Carolina, just above the South Carolina border (Figure 2.4.1-205). The entire First Broad River and its tributaries are located in USGS Hydrologic Subbasin 030804. Tributaries of the First Broad River include Brier Creek and North Fork First Broad Creek, Brushy, Hinton, Knob, and Wards creeks (Reference 231, Figure 2.4.1-206).

Approximately two-thirds of the 426 sq mi. (Table 2.4.1-202) of the First Broad River subbasin are forested and one-third is in pasture. The largest urbanized areas in this subbasin are the towns of Shelby and Boiling Springs. These municipalities are restricted to the southern third of the subbasin and are concentrated along the U.S. 74 corridor. There are 11 permitted dischargers in the subbasin, including the towns of Shelby and Boiling Springs, wastewater treatment plants, and PPG Industries (Reference 232). The First Broad River has a slope of 0.33 percent and a gradient of 17.4 ft./mi., based on analysis of a USGS topographic map (Reference 290).

Second Broad River

The Second Broad River originates in McDowell County and flows into the Broad River near the Rutherford and Cleveland counties border (Figures 2.4.1-205 and 2.4.1-206). The Second Broad River and its tributaries lie within USGS Hydrologic Subbasin 030802; it has a drainage area of approximately 513 sq mi. (Table 2.4.1-202). Tributaries of the Second Broad River include Catheys, Hollands, and Roberson creeks (Figure 2.4.1-206). The largest urbanized areas are the towns of Spindale and Forest City. There are three permitted dischargers in this subbasin that release greater than 0.5 million gallons per day (Mgd) of effluent to the Second Broad River watershed. These are the wastewater plants for the towns of Spindale, Forest City, and Cone Denim LLC (Reference 232). The Second Broad River has a slope of 0.37 percent and a gradient of 19.7 ft./mi. (Reference 289).

Green River

The Green River has been impounded at two locations to form Lake Summit and Lake Adger (Figure 2.4.1-205). Both reservoirs are used to produce hydroelectric power. The Green River and its tributaries lie within USGS Hydrologic Subbasins 030802 and 030803 (Figures 2.4.1-205 and 2.4.1-206) and comprise a drainage area of approximately 137 sq. mi. (Table 2.4.1-202). This drainage area is mostly undeveloped with more than 90 percent of the surface area forested. Tributaries of the Green River include the Hungry River and Brights Creek (Figure 2.4.1-206). R.J.G. Inc.'s Six Oaks Complex has the only permit to discharge on the Green River (above Summit Dam). The Bright's Creek Golf Club development has a temporary construction discharge permit; however, once the facility is operational, it is expected to have a nondischarge permit (Reference 232). The Green River has a slope of 0.69 percent and a gradient of 36.5 ft./mi. (Reference 289).

Buffalo Creek

Buffalo Creek drains eastern Cleveland, southwestern Lincoln, and northwestern Gaston counties in North Carolina (Figure 2.4.1-206), and this creek and its tributaries flow south through USGS Hydrologic Subbasins 030805 and 100 (Figure 2.4.1-206). The Buffalo Creek drainage area is approximately 181 sq. mi. (Table 2.4.1-202) in North Carolina and 16 sq. mi. in South Carolina. Approximately 40 percent of the surface area is pasture land, and almost 50 percent continues to be forested. Tributaries of Buffalo Creek include Muddy Fork and Beason Creek (Figure 2.4.1-206). Buffalo Creek is impounded approximately 16 river mi. northeast of the Lee Nuclear Site to form Kings Mountain Reservoir in North Carolina. The creek discharges into the Broad River approximately 7 river mi. north of Ninety-Nine Islands Dam (Reference 232). Buffalo Creek has a slope percentage of 0.29 and a gradient of 15.1 ft/mi (Reference 289).

2.4.1.2.2.4 Local Tributaries

In addition to the Broad River and its major tributaries, there are several smaller streams in the vicinity of the Lee Nuclear Site (above Ninety-Nine Islands Dam), including Cherokee Creek, Doolittle Creek, London Creek, and McKowns Creek. In addition, an intermittent stream flows into Make-Up Pond A (Figure 2.4.1-206).

The most significant of these features is McKowns Creek, which is dammed at the Lee Nuclear Site to form Make-Up Pond B (see Subsection 2.4.1.2.2.6). McKowns Creek’s drainage area is estimated to be 1633 ac., including a small impoundment feeding the creek. The small impoundment has a drainage area of approximately 181 ac. (Reference 254). The intermittent stream mentioned in the previous paragraph features a drainage area of approximately 385 ac.

There are a number of other creeks and impoundments within a 6-mi. radius of the Lee Nuclear Site. Most of these features are hydraulically insignificant (i.e., small storage, low hazard structures, or outside drainage) with the exception of Make-Up Pond C. The largest of these features within this radius is Make-Up Pond C located on London Creek, as shown in Figure 2.4.1-213, which has a maximum storage of approximately 22,000 acre-feet (ac.-ft.). Details of Make-Up Pond C are provided in Subsection 2.4.1.2.3.1. Lake Cherokee (also known as Wildlife Dam and Reservoir) is located on London Creek just upstream of Make-Up Pond C. Lake Cherokee has a maximum storage of 720 ac.-ft. and is hydraulically insignificant.

2.4.1.2.2.5 Ninety-Nine Islands Reservoir

Ninety-Nine Islands Dam is located on the Broad River approximately 1 linear mi. southeast of the Lee Nuclear Station. The reservoir backs up to Cherokee Falls Dam, approximately 3 mi. to the north. The Ninety-Nine Islands Dam and associated hydroelectric plant were constructed in 1910, and the dam structure is a concrete gravity dam 62 ft. in height and 1568 ft. in length (References 216 and 217).

The Federal Energy Regulatory Commission (FERC) operating license for Ninety-Nine Islands Hydroelectric Station limits reservoir drawdown to 1 ft. below full pond (511 ft. above msl) from March through May and 2 ft. below full pond elevation from June through February. In addition, the minimum flows to be maintained below the dam are: 966 cfs January through April; 725 cfs May, June, and December; and 483 cfs July through November (Reference 216). When river flow drops below 483 CFS and the elevations drop to the maximum drawdown limit, the Ninety-Nine Islands Hydroelectric Station must discharge accumulated inflow on an hourly basis.

Reservoir Characteristics

Ninety-Nine Islands Dam impounds a 433-ac. mainstem “run-of-the-river” reservoir[a] with a normal water level at 511 ft. above msl and a shoreline of approximately 14 mi. (Reference 216). Flow through Ninety-Nine Islands Reservoir is dominated by the flow of the river channel, which divides the reservoir into two backwater regions. The two backwater regions exhibit very little circulation during nonflood periods. Therefore, the average transit time through the reservoir is conservatively estimated from the volume of the reservoir along the main channel excluding the backwater areas. Based on a storage volume of 570 ac-ft along the main channel to a point about 0.7 river mi. upstream from the dam and an average annual flow of the Broad River of approximately 2500 cfs, the average transit time for water flow through the reservoir is approximately 3 hours. During low flow conditions the transit time slows to around 14 hours (Subsection 2.4.11).

From October 1998 to 2006, the USGS recorded a minimum pool elevation in the Ninety-Nine Islands Reservoir of 508.20 ft. on February 14, 2005 (Reference 293). Duke Power data from 1964 to 1973 indicate that the minimum pool elevation was 505.6 ft. during May 1965 (Reference 214). Low water considerations are discussed in Subsection 2.4.11. The maximum water surface elevation for the Broad River at the site is discussed in Subsections 2.4.2, 2.4.3 and 2.4.4. Based on the flood frequency curve generated from analysis of the USGS Gaffney gauge, the projected 100-yr flow is 97,900 cfs and the projected 500-year flow is 127,000 cfs. The corresponding elevations based on interpolation of the rating curve for Ninety-Nine Islands Dam and assuming flashboard failure are 520.95 ft. and 522.63 ft. for the 100-year and 500-year events, respectively.

Because the Ninety-Nine Islands Reservoir is a “run-of-the-river” reservoir, evaporation and seepage have little effect on the water budget of the reservoir. The aspects of annual yield and dependability as they relate to the construction or operation of Lee Nuclear Station are discussed above in terms of discharge and low-flow characteristics of the Broad River.

Morphology

Ninety-Nine Islands Reservoir is characterized by three hydrographic areas, the main river channel and two backwater areas, that have developed because of sedimentation patterns since impoundment of the reservoir. The reservoir is a dynamic system that is constantly changing, due to the effects of floods, low flow, sedimentation, and scouring. In its present state the reservoir is a combination of two large backwater areas separated by the river channel and its associated sediment bars, spits, banks, and coves. A bathymetry study of the reservoir was conducted in the fall of 1973 by Duke Power Company (Reference 214). In the fall of 2006, additional bathymetry of the reservoir and the Broad River was conducted. This impoundment exhibited a maximum depth of 35.2 ft. (Figure 2.4.1-209, Sheet 1) and a mean depth of 9.2 ft. The impoundment is relatively shallow and relatively minor fluctuations in reservoir levels can result in significant changes in surface area. The estimated volume of storage is 1691 ac.-ft. based on the limited 233-ac. survey area. The U.S. Army Corps of Engineers (USACE) National Inventory of Dams (NID) reports the storage volume as 2300 ac.-ft. Deltaic sedimentation associated with creeks was evident in the backwater areas and limited the aerial extent of the survey.

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a. The mainstem refers to the main channel of the river in a river basin, as opposed to the streams and smaller rivers that feed into it. A “run-of-the-river” dam is a dam without a large reservoir and, therefore, with only a limited capacity for water storage.

The backwater areas can be divided into two hydrographic sections: one paralleling the river-influenced channel areas (being separated from them by an area of sediment deposition) and the other located at the lower end of each backwater area perpendicular to the main stream flow. Shallow backwater sections parallel to the main channel areas contain large deposits of river-borne sediments deposited during flooding conditions. The areas of backwater perpendicular to the river flow are less influenced by the main channel sediment transport. These sections exhibit relatively deeper waters with shoreline and bathymetric profiles more reflective of local topography and original reservoir characteristics (Reference 214).

The main channel area is characterized by a shallow sand and gravel bed extending through the center of the reservoir area and between the two major backwater areas. Unlike the previously described backwater areas, the main channel portion of the reservoir has a strong current when the hydroelectric station is operating and has relatively homogeneous physiochemical characteristics.

River-borne sedimentation has greatly altered the reservoir from its original condition. Dredging in the dam area has been performed periodically to ensure efficient hydroelectric generating operations. Dredging activities include keeping the hydroelectric intakes clear of sediment, which is a routine maintenance issue for most hydroelectric projects in this area. Large areas of the stream bed in the original reservoir have been filled completely and stabilized by heavy vegetation growth. During the 1973 study, backwater areas that were not already completely filled, exhibited changes in some water depths in the first 6-month sampling period, thus illustrating the influence of heavy sedimentation (Reference 214).

Circulation and Mixing

Ninety-Nine Islands Reservoir circulation and mixing characteristics are influenced primarily by discharge. The central channel is almost completely dominated by river discharge and accounts for the primary circulation pattern of the reservoir during nonflood periods. Currents through the Ninety-Nine Islands Reservoir are much stronger than expected for an impoundment, although less than currents in the upstream and downstream river. Based on data from the 1975 Cherokee Nuclear Station Construction Permit ER, temperature and chemical constituents were homogeneous at all depths due to thorough turbulent mixing. Sampling performed in 2006 confirms the thorough mixing (Reference 214).

Backwater areas exhibit a very different flow regime because of the lack of circulation in these waters, especially during nonflood periods. Stagnation is common during low-flow periods. The backwater areas are influenced by temperature and tend to slightly stratify during periods of warm weather.

Wind apparently has little effect upon circulation in these backwater areas because they are protected by topographic relief and heavy vegetation, especially in the limited floodplain areas. Lower than normal dissolved oxygen concentrations result from decomposition of organic materials and poor circulation.

Flooding conditions greatly alter the normal hydrologic setting. Washover from the river channel portion of the reservoir during high flow tends to flush waters from the upper backwaters toward the lower portion of the reservoir. During these periods, extremely turbid conditions prevail throughout the impoundment due to the import of river-borne sediments and the resuspension of lake sediments (Reference 214).

2.4.1.2.2.6 Surface Water Impoundments

The Lee Nuclear Site has three manmade impoundments: (1) Make-Up Pond B, including the Upper Arm feature, (2) Make-Up Pond A, and (3) Hold-Up Pond A. These features, along with the constructed earthen dams and site structures, are shown in Figure 2.4.1-201. New retention ponds are constructed or existing ponds are used, if necessary, to accommodate surface water runoff and allow sediment-laden water from dewatering activities to pass through the impoundments prior to discharge at a NPDES permitted outfall. Make-Up Pond C is an off-site facility, located on a tributary of the Broad River, west of the Lee Nuclear Station. Details of Make-Up Pond C are provided in Subsection 2.4.1.2.3.1.

Make-Up Pond B

Make-Up Pond B was formed by constructing an earthen dam that impounds McKowns Creek west of Lee Nuclear Station. This reservoir was constructed in the 1970s in the initial construction phase of the Cherokee Nuclear Station. A cofferdam within Make-Up Pond B was utilized to support the original construction of the Make-Up Pond B dam. Upon filling of the pond, the cofferdam was submerged creating a bathymetric division of the pond. Very little to no sediment accumulation is observed within this impoundment.

The cofferdam is apparent on the bathymetric map, Figure 2.4.1-209 (Sheet 2 of 4), as two approximately parallel 540 ft. contours midway between McKowns Mountain and the Make-Up Pond B dam. This cofferdam will be breached to allow full communication between the two bathymetric divisions within Make-Up Pond B.

Make-Up Pond B dam crest elevation is 590 ft. Make-Up Pond B has a normal full pond elevation of 570 ft. above msl (spillway elevation) and occupies approximately 11 percent of the total drainage area of McKowns Creek. Bathymetry exhibited a maximum depth of 59.3 ft., a mean depth of 31.4 ft., total storage capacity of approximately 4000 ac.-ft. and the surface area at full pond is approximately 150 ac. (Figure 2.4.1-209, Sheet 2). The useable storage is approximately 3200 ac.-ft.

During 2006 – 2007, water levels in Make-Up Pond B varied 0.49 ft., representing approximately 73 ac-ft or approximately 1.8 percent of the total storage volume. It should be noted that Make-Up Pond B was receiving waters from dewatering activities, thus affecting the water balance. These activities were conducted to remove water from the original excavation for Cherokee Nuclear Station which was full of water prior to site characterization activities in 2006. All of this water was pumped to Make-Up Pond B. Inflow from rainfall and runoff contribute approximately 1271 gpm to the impoundment. Site observations and aerial photographs indicate that Make-Up Pond B retains water to near full pond level under natural conditions.

Make-Up Pond B includes an adequately sized outlet structure and is not located on a sizeable river or stream. Therefore, the potential for significant debris to be picked up by a rise in the water level and then transported to the outlet structure where it could collect as an obstruction is minimal which eliminates the need for clear cutting around the perimeter of the pond. Floating debris has not been a problem historically and no clogging of the overflow spillway has been recorded.

To ensure no debris blockage of the spillway, a shoreline management program is established along the banks of Make-Up Pond B. The shoreline management program consists of annually inspecting the shoreline around Make-Up Pond B and removing any trees that show distress of falling into the pond and removing any trees that may be down on the ground. In addition, Duke Energy will inspect

the spillway after any rain event greater than 3 inches per hour to ensure that the spillway remains clear of any debris.

Even though the shoreline management program is considered to be adequate for preventing debris blockage of the spillway, as a secondary measure a debris barrier system will be installed approximately 350 feet away from the spillway as shown on Figure 2.4.1-214. The debris barrier is designed to rise and fall with fluctuations in the pond water level. The debris barrier system is considered non-safety related.

The maximum flood level of surface water features at the Lee Nuclear Station is elevation 589.10 ft. msl. This elevation would result from a Probable Maximum Flood (PMF) event on Make-Up Pond B watershed with the added effects of coincident wind wave activity as described in Subsection 2.4.4. The Lee Nuclear Station safety-related structures have a grade elevation of 593 ft. msl.

An access road spanning across the Upper Arm Dam embankment was constructed in the late 1970's during Cherokee Nuclear Station construction. The result of this construction created a separate impoundment of Make-Up Pond B that takes surface water runoff from the east slope of McKowns Mountain, and from the west slope of ridge to east of Upper Arm. A 54 in. culvert pipe was placed to allow for positive drainage between the Upper Arm and Make-Up Pond B. The location of this dam is shown on Figure 2.4.1-209, Sheet 2.

The Upper Arm Dam has a design crest elevation of 590 ft. located at the access road. The normal pool elevation of the Upper Arm is 575 ft and the Upper Arm Pond surface area at full pond conditions is approximately 5 percent of the total drainage area of the Upper Arm watershed. Bathymetry exhibited a maximum depth of 32.2 ft., a mean depth of 31.4 ft., total storage capacity of approximately 101 ac.-ft. and the surface area at full pond is approximately 9.1 ac. (Figure 2.4.1-209, Sheet 2).

Make-Up Pond A

Make-Up Pond A was also constructed in the 1970s during the initial construction phase of the Cherokee Nuclear Station. The basin is situated east of the proposed Lee Nuclear Station reactor locations and was formed by constructing an earthen dam across a backwater arm of Ninety-Nine Islands Reservoir. Very little to no sediment accumulation is observed within this impoundment.

Make-Up Pond A crest elevation varies from 557.5 ft. to a low point of 555 ft. above msl (Reference 254). At the time of the survey, the impoundment elevation was approximately 546.1 ft. above msl with full pond elevation at 547 ft. This is a relatively small surface water impoundment with a full pond surface area of approximately 62 ac. Bathymetry exhibited a maximum depth of 59.6 ft., a mean depth of 26.1 ft., and an estimated volume storage of 1425 ac.-ft. (Figure 2.4.1-209, Sheet 3). The useable storage is approximately 1200 ac.-ft.

During 2006 – 2007, water levels in Make-Up Pond A varied 0.89 ft., representing approximately 53 ac-ft or 3.7 percent of the total storage volume. Rainfall and runoff contribute on average 396 gpm to the impoundment. Based on site observations and review of available historical aerial photographs, Make-Up Pond A retains water to near full pond level under natural conditions.

Hold-Up Pond A

Hold-Up Pond A is a small impoundment located north of the proposed reactor locations (Figure 2.4.1-209, Sheet 4). Two dams were built in the 1970s to form this impoundment. The crest elevation of the dam is approximately 539.7 ft. above msl, and it has a current normal pond elevation of approximately 536 ft. above msl (Reference 254). Very little to no sediment accumulation was observed in this impoundment. The surface area at full pond is 4.4 ac. and the total storage volume at full pond is 56.4 ac-ft. Rainfall and runoff contribute on average 18 gpm to the pond. Based on site observation and review of available historical aerial photographs, Hold-Up Pond A retains water to near full pond level under natural conditions.

2.4.1.2.2.7 Local Wetlands

Wetlands are areas that are inundated or saturated by surface water or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. At the Lee Nuclear Site, wetlands occupy a total of 46.4 ac. or 2.4 percent of the site. They are currently represented by Alluvial Wetlands, Non-alluvial Wetlands, and Non-jurisdictional Wetlands that total 3.2 ac. (0.2 percent), 10.8 ac. (0.6 percent), and 32.4 ac. (1.7 percent) of the total site area, respectively. No appreciable seasonal variations of wetland settings were documented during 2006.

2.4.1.2.3 Dams and Reservoirs

There have been dams in the Upper Broad River drainage basin since the construction of Cherokee Falls Dam in 1826. The primary functions of the larger storage reservoirs are water supply and hydroelectric power. Table 2.4.1-205 presents information for the six major reservoirs in the Upper Broad River Basin including drainage areas, elevation-storage relationships, and short term (maximum storage) and long term (normal storage) storage allocations. Ninety-Nine Islands Dam, Cherokee Falls Dam, and Gaston Shoals Dam are in the vicinity of the Lee Nuclear Site, and all are used for hydroelectric power. Most of the dams within the Upper Broad River basin were not constructed for flood control.

There are approximately 132 dams (five recreational dams are listed as breached) upstream from the Lee Nuclear Site (Reference 276). Six large dams (see Subsection 2.4.1.2.3.1 below) are upstream from the site and represent approximately 88 percent of the total storage capacity for the Broad River basin. There are two additional smaller dams (Cherokee Falls and Gaston Shoals) immediately upstream of the site on the Broad River; however, they possess less than 2 percent of the total storage capacity for the basin. Both of these dams are essentially run-of-river structures used for hydroelectric power and not flood control. Currently, Cherokee Falls Dam is not operating and is a low-head structure without much volume/storage.

In addition, according to the *Federal Register* (Reference 224), USACE and the Cleveland County Sanitary District are proposing to construct an upstream dam and reservoir on the First Broad River (a tributary of the Broad River) approximately 1 mi. north of Lawndale, North Carolina (about 22 mi. north of the Lee Nuclear Site). Additional information on this dam is presented in Subsection 2.4.1.2.3.3.

2.4.1.2.3.1 Upstream Dams and Reservoirs

Make-Up Pond C, shown in Figure 2.4.1-213, is located approximately 2 mi. west of the Lee Nuclear Station on London Creek in Cherokee County, South Carolina. Make-Up Pond C is formed by construction of an earthen dam and saddle dikes that impound London Creek just upstream of the confluence with Little London Creek. The Make-Up Pond C dam crest elevation is 660 ft. above msl. A labyrinth spillway sets the normal pool elevation at 650 ft. above msl. Make-Up Pond C has a drainage area of 2479 ac. At normal pool elevation, bathymetry exhibits a maximum depth of 116 ft., a total storage capacity of approximately 22,000 ac.-ft., and a surface area of approximately 620 ac. Make-Up Pond C water is used to supplement the Lee Nuclear Station during low flow conditions. The useable storage is approximately 17,500 ac.-ft.

Lake Whelchel is located approximately 8 mi. northwest of the Lee Nuclear Site on Cherokee Creek in Cherokee County, South Carolina. This Lake Whelchel dam is an earthen design that was constructed in 1964 and modified in 1989. The dam height is 61 ft. and the length is 2100 ft. The dam creates a reservoir that is owned by and used as a water supply source for Gaffney, South Carolina. The dam and associated reservoir are owned and operated by the city of Gaffney. The normal pool elevation of the reservoir is 670 ft. above msl (Table 2.4.1-205). The reservoir has a surface area of approximately 177 ac. and a normal storage of approximately 2438 ac.-ft. The maximum storage of Lake Whelchel at the dam crest elevation of 685 ft. is approximately 5698 ac.-ft. No hydroelectric power plant is associated with this dam.

Kings Mountain Reservoir (Moss Lake Dam) is located in Cleveland County, North Carolina, approximately 16 mi. northeast of the Lee Nuclear Site. Discharge waters from this dam are released to Buffalo Creek. The dam was constructed in 1973 and created Kings Mountain Reservoir, which is owned by the city of Kings Mountain and used as a water supply source for the city of Shelby, North Carolina, as well as several smaller communities. In addition, the reservoir is used for recreational activities such as boating and fishing. Moss Lake Dam is an earthen structure that is 840 ft. long and 99 ft. in height. The normal pool elevation of Kings Mountain Reservoir is 736 ft. above msl (Table 2.4.1-205). The reservoir has a surface area of approximately 1329 ac. and a normal storage of 44,400 ac.-ft. and a maximum storage capacity of 53,280 ac.-ft. No hydroelectric power plant is associated with this dam.

Lake Adger (also Turner Shoals) is located on the Green River approximately 44 mi. northwest of the Lee Nuclear Site in Polk County, North Carolina. The Lake Adger dam and associated hydroelectric plant were constructed in 1925 and are owned and operated by Hydro LLC. In addition, the reservoir (Lake Adger) is used for recreational activities such as boating and fishing. Lake Adger Dam is a concrete multiple-arch design that is 689 ft. in length and 90 ft. in height. The normal pool elevation of Lake Adger is 912 ft. above msl (Table 2.4.1-205). The lake has a surface area of approximately 460 ac. and an estimated normal storage of 11,700 ac.-ft. The maximum storage is 16,760 ac.-ft.

Lake Lure is located on the Broad River in Rutherford County, North Carolina, approximately 46 mi. northwest of the Lee Nuclear Site. The Lake Lure dam and associated hydroelectric plant were constructed in 1927 and are owned and operated by the town of Lake Lure. In addition, the reservoir is used for recreational activities such as boating and fishing. Lake Lure Dam is a concrete multiple-arch design that is 480 ft. in length and 124 ft. in height. The normal pool elevation of Lake Lure is 991 ft. above msl (Table 2.4.1-205). The lake has a surface area of approximately 740 ac., a normal storage of 32,295 ac.-ft. and a maximum capacity of 44,914 ac.-ft.

Lake Summit Dam is located on the Green River in Henderson County, North Carolina, approximately 52 mi. northwest of the Lee Nuclear Site. The dam and associated hydroelectric plant were constructed in 1920 and are owned and operated by Duke Energy. In addition, the reservoir is used for recreational activities such as boating and fishing. Lake Summit Dam is a single-concrete-arch design with a concrete buttress structure that is 254 ft. in length and 130 ft. in height. The normal pool elevation of Lake Summit is 2012.6 ft. above msl (Table 2.4.1-205). The lake has a surface area of approximately 276 ac. and a normal storage of 9300 ac.-ft. and maximum storage of 15,840 ac.-ft. The maximum drawdown is 20 ft., yielding a useable storage of 4134 ac-ft.

2.4.1.2.3.2 Downstream Dams and Reservoirs

There are two significant reservoirs located downstream from the Lee Nuclear Site: Ninety-Nine Islands Reservoir and the Lockhart Reservoir. Similar to the Cherokee Falls and Gaston Shoals dams, Ninety-Nine Islands and Lockhart dams are run-of-river structures and are not used for flood control. Dams located further downstream include Neal Shoals Dam (approximately 50 mi.) and Parr Shoals Dam (approximately 52 mi.).

As shown on Figure 2.4.1-205, Lockhart Dam is located in Union County, South Carolina, on the Broad River, 3 mi. south of the confluence with the Pacolet River and approximately 19 mi. south to southeast of the Lee Nuclear Site. The normal pool elevation of the Lockhart Reservoir is around 395 ft. above msl with a surface area of approximately 300 ac. and a normal storage of 2400 ac-ft. The Lockhart Dam and its associated hydroelectric power plant were constructed in 1921 and are currently owned and operated by Lockhart Power Company of Lockhart, South Carolina.

Completed in 1905, the Neal Shoals Dam is located in Chester and Union Counties. The normal pool elevation of Neal Shoals reservoir is around 325 ft. above msl. with a surface area of approximately 550 ac. and a normal storage of 1350 ac-ft.

2.4.1.2.3.3 Water Management Changes

As mentioned in Subsection 2.4.1.2.3, USACE and the Cleveland County Sanitary District (CCSD) are proposing to construct a dam on the First Broad River (upstream and hydraulically connected to the Broad River) approximately 1 mi. north of Lawndale, North Carolina. This is about 26 mi. north of the Lee Nuclear Site. The USACE permit application (Section 404 of the Clean Water Act) states that the dam affects approximately 24 mi. of river and stream habitat and approximately 1 ac. of wetlands. Initial feasibility estimates state that an earth-filled dam across the First Broad River may be approximately 83 ft. high and 1245 ft. wide at the base. The associated emergency spillway, located south of the dam, is approximately 1000 ft. wide. The dam creates a reservoir with a surface area of approximately 2245 ac., impounding those areas below 860 ft. above msl. A 100-ft. buffer zone would likely surround the reservoir (Reference 224).

The CCSD is proposing this dam to increase the water supply for the region. Based on current rates of growth, CCSD projects that water needs for its customers would double by 2050 (Reference 224). The reservoir is also projected to lessen the occurrence of water shortages during drought conditions.

2.4.1.2.4 Regional Hydrogeology

The Piedmont aquifer system is basically a two-layered slope-aquifer system. The shallow water table aquifer is comprised of the saprolite and residual soil, which is typically low-yielding. The

underlying bedrock aquifer consists of weathered and unweathered crystalline igneous and metamorphic rocks that store and transmit water through fractures. The shallow aquifer is unconfined, meaning that the upper surface of the saturated zone is not effectively separated from the ground surface by a low-permeability clay layer. The bedrock fracture system is a network of discontinuities that increases in prevalence upward through the crystalline rock as it transitions into saprolite (Figure 2.4.1-210). Because of the permeability of the transition zone, the bedrock aquifer is also considered unconfined and not effectively isolated. Thus, the saprolite and bedrock zones function as one interconnected aquifer system (Reference 266).

Groundwater occurs almost everywhere throughout the Piedmont region; however, it is not in a single, widespread aquifer. It occurs in various local aquifer systems and compartments that have similar characteristics and are hydraulically connected. Groundwater recharge in this area is derived entirely from infiltration by local precipitation. Groundwater flow within this combined system can be complex. The fractures, relic rock textures, and directional differences in permeability or ease of groundwater movement may significantly affect the local groundwater flow direction. Recharging of the groundwater in the Piedmont occurs by the addition of rainwater, first to the shallow saprolite aquifer and then to the uppermost fracture zone. Recharge occurs mostly on upland topographic highs or at least above the slopes of stream valleys.

The average annual rainfall in the region is about 50 inches. The annual pan evaporation rate is 51.8 inches for the region. Pan evaporation rates are higher than actual lake evaporation due to radiation and heat exchange effects. The pan coefficients range from 0.64 and 0.81, with an average of 0.7 used for the United States. Therefore, the annual evaporation rate is 36.26 inches. Groundwater is contained in the pores that occur in the weathered material (residual soil, saprolite) above the relatively unweathered rock and within the fractures in the igneous and metamorphic rock. The depth to the water table depends on climate, topography, rock type, and rock weathering. The water table varies from ground surface elevation in valleys to more than 100 ft. below the surface on sharply rising hills. Although the precipitation in the Piedmont is relatively evenly distributed throughout the year, the water table fluctuates noticeably, typically declining during the late spring and summer due to evapotranspiration and rising in the late fall and winter when the evaporation potential is reduced (Reference 297).

A detailed discussion of regional and local groundwater characteristics is presented in Subsection 2.4.12. A detailed discussion of regional and local geology and soil properties is presented in Section 2.5.

2.4.1.2.5 Water Use

This subsection describes surface water and groundwater in the vicinity of the Lee Nuclear Site that could affect or be affected by the construction and operation of two AP1000 units. The information provided in this subsection includes descriptions of surface water and groundwater uses that could affect or be affected by construction or operation of the Lee Nuclear Station, including transmission corridors and off-site facilities. In addition, a detailed assessment of water use within the vicinity of the facility, types of consumptive and nonconsumptive water uses, identification of their locations, and qualification of water withdrawals and returns are discussed in this subsection.

2.4.1.2.5.1 Surface Water Use

The Lee Nuclear Site is located on the west bank of the Broad River approximately 3 mi. south-southeast (downstream) of Cherokee Falls and 1 mi. north-northwest (upstream) of the Ninety-Nine

Islands Dam and Hydroelectric Station. Surface water in the vicinity of the Lee Nuclear Site consists of the Broad River, three on-site man-made impoundments, and one off-site man-made impoundment. These features are discussed in detail in Subsections 2.4.1.2.2.6 and 2.4.1.2.3.1.

According to available SCDHEC information on water use for 2005 (Reference 267), total water usage in Cherokee County was 8.4 Mgd. This information is presented in Table 2.4.1-206. Total 2005 water withdrawals from Cherokee, Chester, Greenville, Spartanburg, Union, and York counties, South Carolina, are presented in Table 2.4.1-207 (Reference 267).

No surface water usage in Cherokee County was reported for domestic self-supplied systems, aquaculture, golf courses, irrigation, livestock, mining, or thermoelectric power uses. According to SCDHEC, water use for hydroelectric power was 1116 Mgd in 2005 for Cherokee County (Reference 267). The USGS 2000 data did not reference hydroelectric power water use; however, these data were included in the 1995 data set. According to the U.S. Army Corps of Engineers National Inventory of Dams, there have been no hydroelectric dams constructed in the watershed since 1995. Therefore, the USGS 1995 data remains unchanged. According to the USGS, there were 2037.1 Mgd of instream water use for hydroelectric power in 1995 for Cherokee County. Surface water-use details for the Broad River watershed within 60 mi. of the Lee Nuclear Site are presented in Tables 2.4.1-207 and 2.4.1-208.

Nineteen permitted surface water intakes at sixteen separate facilities are located in the Upper Broad River basin upstream from the Lee Nuclear Site (Table 2.4.1-209, Figure 2.4.1-211). The closest surface water intake is the Gaffney Board of Public Works intake about 8 mi. upstream on the Broad River. In addition to the existing intakes, Duke Energy anticipates modernizing and expanding the Cliffside Steam Station (located 19 mi. upstream from the site in Cleveland County, North Carolina), which will use the existing surface water intake from the Broad River. Cliffside Steam Station expansion is discussed in Subsections 2.2.2.1.4 and 2.4.11.4.

Three permitted surface water intakes for public water supply are located downstream from the Lee Nuclear Site (Figure 2.4.1-211). The closest of these is the city of Union, which withdraws water from the Broad River about 21 mi. downstream from the site and has a maximum withdrawal rate of 23.8 Mgd. The second and third are the Carlisle Cone Mills (approximately 30 miles downstream; maximum capacity 8.1 Mgd) and the V.C. Summer Nuclear Station (approximately 52 miles downstream; maximum capacity 3.1 Mgd) (Table 2.4.1-209). Two additional AP1000 units are planned for the V.C. Summer Nuclear Station. Details are not currently available. Additional surface water uses not included in the table are located within 20 – 50 mi. of the site. These additional intakes are relatively insignificant because they are located outside the watershed or on tributaries that join the Broad River downstream from the site.

The plant water use is discussed in Subsection 2.4.1.1.4. Table 2.4.1-210 and Table 2.4.1-211 present raw water use and effluent discharge as a percentage of Broad River flow rates. The maximum consumption rate of Broad River water, predominantly resulting from evaporation during plant operations, is expected to be 63 cfs, approximately 3 percent of the average annual mean discharge of the Broad River (approximately 2500 cfs).

2.4.1.2.5.2 Groundwater Use

Groundwater produced for water supply in counties located in the Piedmont aquifer system is reported to be approximately 79 Mgd (122.5 cfs). This can be compared to some Upper Coastal Plain counties that withdraw up to several thousand Mgd of groundwater (Reference 293).

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A reported 1.02 million gal. of groundwater were used for thermoelectric power generation in Cherokee County (Reference 267). No groundwater usage in Cherokee County for domestic self-supplied systems, aquaculture, golf courses, irrigation, livestock, mining, or thermoelectric power was reported in the 2005 SCDHEC data (Reference 267). According to a private well report from SCDHEC, based on data from January 1985 to June 2006, the number of reported private wells in Cherokee County was 1076 (Reference 261). The USGS and state water-use data were reviewed, and groundwater withdrawals are presented in Tables 2.4.1-207 and 2.4.1-208. Groundwater withdrawals for Cherokee and surrounding counties in South Carolina (Table 2.4.1-207) only account for 4.7 Mgd, and the majority (85 percent) of that volume is pumped from Spartanburg County, approximately 25 mi. west of the Lee Nuclear Site.

[Based on information received from the USGS, SCDHEC, and local agencies, as well as field reconnaissance, local groundwater use in the vicinity of the Lee Nuclear Site is predominantly from domestic wells. The majority of the residents within a 2-mi. radius of the site have their own water wells. A review of the addresses associated with these well reports, coupled with a site reconnaissance, indicates approximately 177 residential water wells exist within a 2-mi. radius of the Lee Nuclear Site, the closest of these being approximately 5000 ft. south of the center of the site near McKowns Mountain Road (Table 2.4.1-212, Figure 2.4.1-212). The Cherokee Nuclear Station Construction Permit ER identified 50 domestic water wells and provided construction details including well diameter, well depth, and depth to water. Construction details for the 50 wells are anticipated to be relatively unchanged. According to the Draytonville Water District Board Chairman, municipal water supply lines are present in the area surrounding the Lee Nuclear Site, but some nearby residents continue to use their wells for potable water. The future use of self-supplied groundwater is expected to decline as residents increase their dependence on the municipal water supply.]^{SRI}

The Lee Nuclear Site is not expected to use groundwater as a source of water for any purpose. Water for temporary fire protection, concrete batching, and other construction uses will be obtained from the Draytonville Water District. Groundwater is not used as a safety-related source of water or as a primary water supply resource for any purpose. Further discussion regarding groundwater characteristics and use is provided in Subsection 2.4.12.

2.4.2 Floods

2.4.2.1 Flood History

Floods on the Broad River occur primarily as a result of precipitation runoff over the watershed. There have been dams in the Upper Broad River drainage basin since the construction of Cherokee Falls Dam in 1826. However, the majority are not flood control dams. The primary function of the larger storage reservoirs are water supply and hydroelectric power. Ninety-Nine Islands Dam, Cherokee Falls Dam, and Gaston Shoals Dam, in the vicinity of the Lee Units 1 and 2, are all used for hydroelectric power (Reference 276). The hydroelectric facility at Cherokee Falls Dam is not currently operating. Although these structures affect the low flow conditions of the Broad River, peak flow conditions at Lee Units 1 and 2 are generally not affected by regulation. The limited flood control dams in the basin are small storage structures and would have limited effect on peak flows.

The Gaffney stream gauge station (USGS No. 02153500) is located about 5 river miles upstream of Lee Units 1 and 2 between Gaston Shoals Dam and Cherokee Falls Dam. Figure 2.4.2-201 shows

the location of area gauges. The Gaffney gauge has a drainage area of 1490 sq. mi., about 96 percent of the drainage area at the site. The drainage area of the Broad River at the Lee Nuclear Station is about 1550 sq. mi. Table 2.4.2-201 summarizes peak flow for the broken period of record from 1939 to 1990 (Reference 290). The flood of record for the Gaffney gauge, 119,000 cfs on August 14, 1940, corresponds to a Ninety-Nine Islands reservoir elevation of about 522.5 ft. at Lee Nuclear Station.

The Ninety-Nine Islands reservoir elevation is maintained by Duke Energy and recorded on a daily basis from 1999 to 2005 by USGS gauge No. 02153550. The USGS gauge record high is 107.29 ft. on September 8, 2004 for a reservoir elevation of 518.75 ft. (Reference 208). The gauge datum is 411.46 ft. above the National Geodetic Vertical Datum of 1929 (NGVD 1929). Earlier reservoir records from 1964 to 1973 indicate that the highest elevation was about 513.6 ft. during May 1972.

The stream gauge station below Ninety-Nine Islands Dam (USGS No. 02153551, listed in Table 2.4.1-203 as Broad River below Cherokee Falls, SC) is located in the tailrace and has a drainage area of 1550 sq. mi. Peak flow records are limited for this station and the gauge is not calibrated for large flows. Table 2.4.2-202 summarizes peak flow and gauge height for the period of record from 1999 to 2005 (Reference 290).

No historical data exists regarding flooding due to surges, seiches, tsunamis, dam failures, or landslides. Surge and seiches are discussed in Subsection 2.4.5. Tsunamis are discussed in Subsection 2.4.6. Dam failures are discussed in Subsection 2.4.4. Channel diversions are discussed in Subsection 2.4.9. Historical information related to icing and ice jams is provided in Subsection 2.4.7.

2.4.2.2 Flood Design Considerations

The AP1000 is designed for a normal groundwater elevation up to plant elevation 98’ and for a flood level up to plant elevation 100’. For structural analysis purposes, grade elevation is also established as plant elevation 100’. Actual grade will be a few inches lower to prevent surface water from entering doorways.

For a portion of the annex building the site grade will be 107 feet to permit truck access at the elevation of the floor in the annex building and inside containment. Subsection 3.4.1 describes design provisions for groundwater and flooding.

The Lee Nuclear Station conforms to Regulatory Position 1 of Regulatory Guide 1.59. There are no safety-related structures that could be affected by floods and flood waves.

Adverse effects of flooding due to high water or ice effects do not have to be considered for site-specific nonsafety-related structures and water sources outside the scope of the certified design. Flooding of water intake structures, cooling canals, or reservoirs or channel diversions would not prevent safe operation of the plant.

The type of events evaluated to determine the worst potential flood include (1) probable maximum precipitation (PMP) on the total watershed and critical sub-watersheds including seasonal variations and potential consequent dam failures, as discussed in Subsection 2.4.3, (2) dam failures, as discussed in Subsection 2.4.4, including in a postulated safe shutdown earthquake with a coincident

25-year flood or operating basis earthquake with a coincident one-half probable maximum flood (PMF), (3) local intense precipitation, and (4) two year coincident wind waves as discussed in Subsection 2.4.3. Local intense precipitation is discussed below. Both static and dynamic assumed hypothetical conditions to determine the design flood protection level are evaluated in Subsections 2.4.3 and 2.4.4.

Specific analysis of Broad River flood levels resulting from ocean front surges, seiches, and tsunamis is not required because of the inland location and elevation characteristics of the Lee Nuclear Station. Additional details are provided in Subsection 2.4.5 and Subsection 2.4.6. Snowmelt and ice effect considerations are unnecessary because of the temperate zone location of the Lee Nuclear Station. Additional details are provided in Subsection 2.4.7. Flood waves from landslides into upstream reservoirs required no specific analysis, in part because of the absence of major elevation relief. In addition, elevation characteristics of the vicinity relative to the Broad River, combined with the limited storage volume availability of nearby upstream reservoirs, prohibit significant landslide induced flood waves. Additional details are provided in Subsection 2.4.9.

The maximum flood level at the Lee Nuclear Station is established as the maximum of calculated results from flooding events analyzed in Section 2.4. That maximum flood level is elevation 592.56 ft. msl. This elevation would result from a PMP event on the Lee Nuclear Station site (local intense precipitation) as described in Subsection 2.4.2.3. The Lee Nuclear Station safety-related plant elevation is 593 ft. msl. This maximum flood level is identified as a site characteristic in Table 2.0-201.

2.4.2.3 Effects of Local Intense Precipitation

The Lee Nuclear Station drainage system was evaluated for a storm producing the PMP on the local area. For the purpose of the evaluation all subsurface drainage features (i.e., culverts, inlets, etc.) including the vehicle barrier system trench are assumed non-functional and all precipitation is assumed to be transformed to runoff.

The site is generally defined by wide flat areas. However, the site is graded such that runoff will drain away from safety-related structures either to Make-Up Pond B, Make-Up Pond A, or directly to the Broad River. Runoff from a specific power block area flows through four graded channels per unit as described in the discussion below and then flows across the site to the receiving water body. Computed water surface elevations in the vicinity of safety-related structures are below plant elevation 593 ft. The site grading and drainage plan is shown in Figure 2.4.2-202.

The site is graded to drain runoff away from the power blocks. The finished floor elevation of the safety related structures for each unit is 593 ft. The areas immediately adjacent to the power blocks range in elevation from 592 ft. to 590 ft. The adjacent area is generally bounded by a roadway surrounding the power blocks. The power block area bounded by the roadway is either paved or gravel surfaced. Areas beyond the roadway are generally maintained grass surfaces. Further from the power blocks, the site is flat from the roadway to the plant side of the vehicle barrier system at elevation 590 ft. The opposite bank of the vehicle barrier system is at elevation 588 ft. Beyond the vehicle barrier system, the site is generally flat at elevation 588 ft. before encountering the steeper slopes into the adjacent, downstream water bodies.

The effects of local intense precipitation are analyzed using a series of models, each establishing boundary conditions for additional modeling. The overall site, generally described by the flat areas at elevation 588 ft., is idealized as a dry reservoir and modeled using level-pool storage routing with

U.S. Army Corps of Engineers HEC-HMS 3.5 computer software (Reference 302) for the site drainage area shown in Figure 2.4.2-202. The area of the site upstream of the vehicle barrier system, generally described by the flat areas at elevation 590 ft. are also idealized as a dry reservoir and modeled using level-pool storage routing with HEC-HMS 3.5 computer software.

The idealized reservoir for the overall site is defined by an elevation-discharge-storage relationship. Storage is based on an elevation-area relationship and is developed using the available storage areas across the site within the drainage area. Storage routing does not incorporate the entire area of the power block bounded by the vehicle barrier system and a sloped area that transitions from elevation 590 ft. to 588 ft., located north of Unit 2. In addition, all other site structures and the switchyard area are assumed to provide no storage.

The discharge relationship for this idealized reservoir is determined using broad crested weir flow. The 588 ft. contour along the banks of the steeper slopes into adjacent, downstream water bodies is used to develop the length of the weir. The total length was reduced to account for ineffective areas where adjacent slopes may not be as steep as areas where structures could obstruct flow discharging from the site. The downstream water bodies are used to establish boundary conditions and determine any tailwater effects. Although tailwater effects are not determined to affect weir flow, a conservative estimate of 2.0 is used for the weir flow coefficient.

The local intense PMP is defined by Hydrometeorological Report (HMR) Nos. 51 and 52. PMP values for durations from 6-hr. to 72-hr. are determined using the procedures as described in HMR No. 51 for areas of 10-sq. mi. (Reference 255). Using the Lee Nuclear Station location, the rainfall depth is read from the HMR No. 51 PMP charts for each duration.

The 1-sq. mi. PMP values for durations of 1-hour and less are determined using the procedures as described in HMR No. 52 (Reference 225). Using the Lee Nuclear Station location, the rainfall depth is read from the HMR No. 52 PMP charts for each duration. A smooth curve is fitted to the points. The derived PMP curve is detailed in Table 2.4.2-203. The corresponding PMP depth duration curve is shown in Figure 2.4.2-203.

HMR 52 guidance indicates that PMP rates for 10-sq. mi. areas are the same as point rainfall. Also indicated in HMR 52, the 1-sq. mi. PMP rates may also be considered the point rainfall for areas less than 1-sq. mi. Therefore, intensities for any drainage areas with durations longer than 1-hr. are derived from the PMP rates for 10-sq. mi. areas. Intensities for drainage areas with durations equal to or less than 1-hr. are derived from the PMP rates for 1-sq. mi. areas.

The AP1000 plant design is based on a PMP of 20.7 in/hr as provided in Table 2.0-201. As shown in Figure 2.4.2-203, the site is within the plant design limits for PMP. The PMP is identified as a precipitation site characteristic in Table 2.0-201. Roofs are sloped to preclude ponding of water.

Two storms are modeled on the basis of the PMP curve detailed in Table 2.4.2-203 and Figure 2.4.2-203. A 72-hr. duration storm with a 1-hr. precipitation interval is examined along with a 6-hr. duration storm with a 5-min. precipitation interval to capture the effect of the short-term, high intensity on the peak flow. The local intense PMP is converted to runoff at each increment by multiplying the drainage area by the intensity of each increment and converting the units to cubic feet per second. This approach is essentially equivalent to the Rational Method (Reference 201) using a runoff coefficient of one. Therefore, all rainfall is converted to runoff instantaneously and no runoff losses are included.

Runoff is applied to the site reservoir model in HEC-HMS and level-pool storage routing is used to determine the resulting water surface elevation. Several time distributions are examined for both modeled storm events. For the 72-hr. duration storm, several temporal distributions produce the highest water surface elevation for the site. For reference, the tail end peaking hyetograph is provided in Figure 2.4.3-236.

As a conservative approach, the results from the 72-hr. duration storm are used to establish the starting elevation for the 6-hr. duration storm. For the 6-hr. duration storm, a tail end peaking storm event is found to result in the highest water surface elevation for the site. The corresponding hyetograph is provided in Figure 2.4.3-235. Based on a combination of the two storms the maximum water surface elevation determined using HEC-HMS is 588.82 ft. This elevation is applied to the overall site and used as the downstream boundary condition for the analysis of the area upstream of the vehicle barrier system.

Similar to the previous discussion, the idealized reservoir for the area upstream of the vehicle barrier system is defined by an elevation-discharge-storage relationship. Storage is based on an elevation-area relationship and is developed using the available storage areas within the drainage area. Storage routing does not incorporate the entire area of the power block bounded by the elevation 590 ft. contour adjacent to the road looping around the power block. In addition, all other structures in the area are assumed to provide no storage.

The discharge relationship for this idealized reservoir is determined using broad crested weir flow. The upstream, higher side of the vehicle barrier system 590 ft. contour is used to develop the length of the weir. The total length does not include the sloped transition area north of Unit 2 and was reduced to account for ineffective areas where structures could obstruct flow discharging from the area. The result for the downstream area is less than the bank elevation of 590 ft. Therefore, there are no tailwater effects. As a conservative estimate, a weir flow coefficient of 2.0 is used.

Two storms are modeled as previously identified for the downstream area. The local intense PMP is converted to runoff instantaneously and no runoff losses are included. Runoff is applied to the idealized reservoir model in HEC-HMS and level-pool storage routing is used to determine the resulting water surface elevation. Several time distributions are examined for both modeled storm events. For the 72-hr. duration storm, all temporal distributions produce the same water surface elevation for the area.

As a conservative approach, the results from the 72-hr. duration storm are used to establish the starting elevation for the 6-hr. duration storm. For the 6-hr. duration storm, several temporal distributions produce the highest water surface elevation for the area. Based on a combination of the two storms the maximum water surface elevation determined using HEC-HMS is 590.56 ft. This elevation is applied to the area upstream of the vehicle barrier system and used as the downstream boundary condition for the analysis of the power block area.

As shown in Figure 2.4.2-204, runoff is directed away from the power block units to lower lying areas via four discharge channels. Under the assumption that all subsurface drainage features are non-functional, runoff would flow over roadways or other topographical features as the flow exits the areas immediately adjacent to the power block units.

For each power block area shown in Figure 2.4.2-204, the peak runoff is determined using the maximum PMP intensity of 6.2 in/5 min from Table 2.4.2-203. The peak runoff is determined by multiplying the drainage area by the intensity and converting the units to cubic feet per second. This

approach is essentially equivalent to the Rational Method using a runoff coefficient of one. Therefore, all rainfall is converted to runoff instantaneously and no runoff losses are included.

The power block drainage areas, shown in Figure 2.4.2-204, are evaluated using the maximum water surface elevation for the idealized reservoir as the downstream boundary condition. Therefore, the HEC-HMS modeling for the idealized reservoir becomes the downstream boundary condition for the power block areas' channel flow evaluation. The four discharge channels for the Unit 1 power block area and the four discharge channels for the Unit 2 power block area are evaluated by steady state, open channel flow, backwater analysis, modeled using HEC-RAS version 4.1.0 software.

Cross sections for each of the four discharge channels (A1, B1, C1, and D1), which discharge from the Unit 1 power block area, are determined based on the grading and drainage plan. Cross sections for each of the four Unit 2 related discharge channels (A2, B2, C2, and D2), are determined in the same manner. Site structures are modeled to obstruct flow and are assumed to provide no storage. A Manning's roughness coefficient of n = 0.026 is used for all of the power block cross sections, which bounds the ground cover used for site conditions (i.e., gravel lined channels). HEC-RAS modeling was performed using steady state analysis to establish a maximum water surface elevation at the upstream cross section.

The resulting water surface elevations are provided in Table 2.4.2-204. The maximum water surface elevation determined is 592.56 ft. and occurs at drainage area B1 of the Unit 1 power block area and at drainage area B2 of the Unit 2 power block area. These drainage areas, B1 and B2, are located on the west side of each, respective, power block area between the Annex Building, north storage tanks and ramp, and the Transformer Area. All Lee Nuclear Station safety-related structures are located above the effects of local intense precipitation at plant elevation 593 ft.

Due to the temperate climate and relatively light snowfall, significant icing is not expected. Based on the site layout and grading, any potential ice accumulation on site facilities is not expected to affect flooding conditions or damage safety-related facilities. Ice effects are discussed in Subsection 2.4.7.

2.4.3 Probable Maximum Flood on Streams and Rivers

The guidance in Appendix A of U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 was followed in determining the Probable Maximum Flood (PMF) by applying the guidance of ANSI/ANS-2.8-1992 (Reference 202). ANSI/ANS-2.8-1992 was issued to supersede ANSI N170-1976, which is referred to by Regulatory Guide 1.59. Although ANSI/ANS-2.8-1992 has been withdrawn, there has been no replacement standard issued. The NRC NUREG-0800 retains both Regulatory Guide 1.59 and ANSI/ANS-2.8-1992 as historical technical references.

<u>Broad River</u>

The PMF for the Broad River above the Lee Nuclear Station is determined from the probable maximum precipitation (PMP) for the watershed of Ninety-Nine Islands Dam. Lee Nuclear Station is located about 1 mi. upstream from the dam and adjacent to the Broad River. The 1550-sq. mi. Broad River drainage basin at Ninety-Nine Islands Dam is shown in Figure 2.4.1-205.

McKowns Creek/Make-Up Pond B

The PMF for McKowns Creek and Make-Up Pond B is determined from the PMP for the 2.190-sq. mi. drainage basin of Make-Up Pond B and the 0.294 sq. mi drainage basin of the Upper Arm. The Make-Up Pond B drainage basin, including the Upper Arm, is shown in Figure 2.4.3-201.

Intermittent Stream/Make-Up Pond A

The PMF for the intermittent stream and Make-Up Pond A are determined from the PMP for the 0.619-sq. mi. drainage basin of Make-Up Pond A. Make-Up Pond A drainage basin is shown in Figure 2.4.3-201.

London Creek/Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, but is not adjacent to the Lee Nuclear Station. However, the PMF for London Creek and Make-Up Pond C is determined for combination with dam failure permutations as discussed in Subsection 2.4.4.1. The PMF is determined from the PMP for the 3.87-sq. mi. drainage basin of Make-Up Pond C. The Make-Up Pond C drainage basin is shown in Figure 2.4.3-239.

2.4.3.1 Probable Maximum Precipitation

Broad River

The PMP for the watershed above the Lee Nuclear Station is defined by Hydrometeorological Report (HMR) Nos. 51 and 52 (References 255 and 225). The PMP is based on an existing study for Ninety-Nine Islands Dam (Reference 217) and modified to include antecedent storm conditions, as specified by Appendix A of Regulatory Guide 1.59.

Using the location of the drainage basin, HMR-51 PMP charts are used to determine generalized estimates of the all-season PMP for drainage areas from 10 to 20,000 sq. mi. for durations from 6 to 72 hrs. The resulting depth-area-duration (DAD) values are shown in Table 2.4.3-201.

HMR-52 is used to determine spatial and temporal distribution of PMP estimates derived from HMR-51. The recommended elliptical isohyetal pattern from HMR-52, shown in Figure 2.4.3-202, is used for the watershed. The watershed model contains 20 subbasins and is shown in Figure 2.4.3-203. The Lake Whelchel and Make-Up Pond C subbasins were not included in the existing study for Ninety-Nine Islands Dam. Both the Lake Whelchel and Make-Up Pond C watersheds are contained within the original subbasin labeled BR-15. Therefore, appropriate modifications were made to subbasin BR-15 to accommodate subbasins for Lake Whelchel and Make-Up Pond C.

HMR-52 computer software (Reference 271), developed by the U.S. Army Corps of Engineers (USACE), is used to determine the optimum storm size and orientation to produce the greatest PMP over the entire basin using the HMR-51 derived DAD table. The HMR-52 recommended temporal distribution is also used and provided by the HMR-52 computer software. Several storm centers were examined and the critical storm center was found to be near the centroid of the watershed for Gaston Shoals Dam, located upstream of Ninety-Nine Islands Dam based on the runoff model discussed in Subsection 2.4.3.3. The critical storm area was found to be 1000 sq. mi., corresponding to isohyet I

in Figure 2.4.3-202. The critical storm orientation was found to be 270 degrees. Refer to Figure 2.4.1-205 for structure locations and watershed.

The critical 72-hr. storm PMP rainfall total is 25.48 in. for the entire watershed. The corresponding temporal arrangement of 6-hr. precipitation increments is provided in Table 2.4.3-202. The hourly temporal distribution of the 72-hr. PMP rainfall of each of the 20 subbasins is provided in Table 2.4.3-203.

In accordance with Appendix A of NRC Regulatory Guide 1.59, the 72-hr. PMP storm is combined with an antecedent storm equal to 40 percent of the PMP. Therefore, the complete sequential storm considered includes a 3-day, 40 percent PMP event followed by a 3-day dry period, which is followed by the 3-day full PMP event.

The PMP estimates are associated with the summer months. HMR 53 (Reference 260) provides estimates for maximum seasonal precipitation. Although HMR 53 applies to 10 sq. mi. drainage areas, it is used as a basis for the larger Broad River watershed. HMR 53 winter precipitation estimates for December through February are less than 57 percent of the all-season PMP estimates identified in Table 2.4.3-201 for the 10 sq. mi. drainage area. The 57 percent ratio is applied to the all-season PMP for the Broad River watershed identified in Table 2.4.3-202 to determine the maximum winter precipitation estimates.

According to guidance (Reference 202) the winter precipitation is evaluated coincident with the 100-yr. snowpack. The water equivalent of the 100-yr. snowpack identified in Subsection 2.3.1.2.7.1 is approximately13 percent of the 72-hr. PMP for the Broad River watershed identified in Table 2.4.3-202. It is assumed that the 100-yr. snowpack is distributed across the entire watershed and completely melts during a winter precipitation event. The combined result of winter precipitation and 100-yr. snowpack is approximately 70 percent of the PMP. Therefore, snowmelt is not considered to be a factor in modeling the PMF event.

McKowns Creek /Make-Up Pond B

The PMP for McKowns Creek, Make-Up Pond B, and the Upper Arm, is defined in Subsection 2.4.2.3. Two storms were modeled on the basis of the PMP curve detailed in Table 2.4.2-203 and Figure 2.4.2-203. The total PMP depth of the 72-hr. duration storm is 46.8 in. A 6-hr. storm with a 5-min. precipitation interval was examined to capture the effect of the short-term, high intensity on the peak flow. In addition, a 72-hr. storm with a 1-hr. precipitation interval was examined to identify the total runoff volume of a PMP event.

Several time distributions were examined for both modeled events. For Make-Up Pond B, for a 72-hr. storm, a tail end peaking storm event was found to provide the greatest runoff and the peak water surface elevation. For the 6-hr. storm, a two-thirds peaking storm event was found to provide the greatest runoff and peak water surface elevation for the short term event.

For the Upper Arm to Make-Up Pond B, for a 72-hr. storm, a tail end peaking storm event was found to provide the greatest runoff and the peak water surface elevation. For the 6-hr. storm, the one-third, two-thirds and center peaking storms were found to provide the greatest runoff. However, the tail-end peaking storm provides the peak water surface elevation. The 6-hr and 72-hr. storm events are discussed in Subsection 2.4.3.5. Hyetographs are provided in Figure 2.4.3-204 and Figure 2.4.3-205 for the two-thirds peaking storm events. Hyetographs are provided in Figure 2.4.3-235 and Figure 2.4.3-236 for the tail end peaking storm events.

Intermittent Stream/Make-Up Pond A

The PMP for the intermittent stream and Make-Up Pond A is defined in Subsection 2.4.2.3. Two storms were modeled on the basis of the PMP curve detailed in Table 2.4.2-203 and Figure 2.4.2-203. The total PMP depth of the 72-hr. duration storm is 46.8 in. A 6-hr. storm with a 5-min. precipitation interval was examined to capture the effect of the short-term, high intensity on the peak flow. In addition, a 72-hr. storm with a 1-hr. precipitation interval was examined to identify the total runoff volume of a PMP event.

Several time distributions were examined for both modeled events. For the 72-hr. storm, a tail end peaking storm event was found to provide the greatest runoff and peak water surface elevation. The corresponding hyetograph is provided in Figure 2.4.3-236. For the 6-hr. storm, multiple peaking distributions, including the two-thirds peaking distribution provided the maximum runoff and peak water surface elevation. For reference, the two-thirds peaking hyetograph is provided in Figure 2.4.3-204.

London Creek/Make-Up Pond C

The PMP for London Creek and Make-Up Pond C is defined in Subsection 2.4.2.3. The storm is modeled on the basis of the 72-hr. PMP curve detailed in Table 2.4.2-203 and Figure 2.4.2-203. The total PMP depth of the 72-hr. duration storm is 46.8 in.

The 72-hr. PMP storm is combined with an antecedent storm equal to 40 percent of the PMP. Therefore, the complete sequential storm considered includes a 3-day, 40 percent PMP event followed by a 3-day dry period, which is followed by the 3-day full PMP event.

Several time distributions were examined for the PMP event using a 1-hr. precipitation interval. A tail end peaking storm event was found to provide the greatest discharge and water surface elevation at Make-Up Pond C. The hyetograph is provided in Figure 2.4.3-240.

2.4.3.2 Precipitation Losses

Broad River

Precipitation losses are based on an existing study (Reference 217) using the U.S. Department of Agriculture (USDA), Soil Conservation Service (SCS) (now the Natural Resources Conservation Service [NRCS]) curve number method. The initial study used geographic information systems (GIS) and the NRCS state soil geographic database (STATSGO) to determine hydrologic soil group values. The GIS and U.S. Geological Survey (USGS) information were also used to determine land-use and impervious cover. An average antecedent moisture condition (AMC II) was then used to compute a weighted curve number for each subbasin.

The SCS Curve Number method was also used to determine precipitation losses for the Lake Whelchel subbasin and the Make-Up Pond C subbasin. The NRCS Web Soil Survey (Reference 300 and Reference 301) was used to determine hydrologic soil group values. Aerials and USGS information were used to determine land-use and impervious cover. An average antecedent moisture condition (AMC II) was also used to compute a weighted curve number for the subbasin.

Precipitation losses are incorporated into the USACE HEC-HMS model discussed in Subsection 2.4.3.3. Initial losses of the SCS Curve Number loss model are developed using the initial abstraction formula.

$I_a = 0.2 * S$

where I_a = initial abstraction (in.)

S = maximum potential storage of the watershed (in.)

where S = 1000 / CN – 10 and CN = average curve number for the watershed

Initial losses for each subbasin are provided in Table 2.4.3-204.

The SCS Curve Number loss model collectively includes interception, infiltration, storage, evaporation, and transpiration. Precipitation losses are derived from the equation for precipitation excess.

$P_e = (P – I_a)^2 / (P – I_a + S)$

where P_e = accumulated precipitation excess at time t (in.)

P = accumulated rainfall depth at time t (in.)

I_a = initial abstraction (in.)

S = maximum potential storage of the watershed (in.)

where S = 1000 / CN – 10 and CN = average curve number for the watershed

The precipitation loss rate is variable and decreases as cumulative rainfall increases during the storm. The total precipitation depth, losses, and excess for each subbasin are provided in Table 2.4.3-204. Antecedent precipitation is 40 percent of the PMP, preceding the main storm for 3 days, with a 3 day dry period between. During the antecedent storm, precipitation losses account for between 37 and 74 percent of the total rainfall with an average of 53 percent. During the main storm, precipitation losses only account for between 3 to 22 percent with an average of 9 percent.

As discussed in Subsection 2.4.3.3, the existing study used three significant storm events occurring in October 1964, June 1972, and October 1976 to verify the subbasin unit hydrographs. As part of the verification process, loss rates were verified by comparison with back calculated curve numbers from the three historical extreme storm events.

McKowns Creek/Make-Up Pond B

No precipitation losses were assumed for evaluation of Make-Up Pond B watershed. All rainfall was assumed to be transformed to runoff.

Intermittent Stream/Make-Up Pond A

No precipitation losses were assumed for evaluation of Make-Up Pond A watershed. All rainfall was assumed to be transformed to runoff.

London Creek/Make-Up Pond C

Precipitation losses are incorporated into the USACE HEC-HMS model, as discussed in Subsection 2.4.3.3, using the SCS Curve Number method as previously described for the Broad River. The NRCS Web Soil Survey (Reference 300) was used to determine hydrologic soil group values. Aerials and USGS information were used to determine land-use and impervious cover. An average antecedent moisture condition (AMC II) was then used to compute a weighted curve number for the watershed.

The SCS Curve Number loss model collectively includes interception, infiltration, storage, evaporation, and transpiration. Initial losses and precipitation losses are derived as previously described for the Broad River. The precipitation loss rate is variable and decreases as cumulative rainfall increases during the storm. Most losses occur during the antecedent precipitation as identified in the hyetograph, Figure 2.4.3-240. The total precipitation depth is 65.52 in., including the antecedent storm. Precipitation losses account for 4.57 in. resulting in 60.95 in. of precipitation converted to runoff.

2.4.3.3 Runoff and Stream Course Models

Broad River

The Broad River runoff and stream course model is based on an existing HEC-1 study (Reference 217) and modified to include the antecedent rainfall conditions. The watershed in Figure 2.4.1-205 was divided into 20 subbasins as shown in Figure 2.4.3-203. The watershed is predominately identified as Piedmont, as discussed in Subsection 2.4.1.2.1. Referencing Figure 2.4.3-203, subbasins labeled LS-1, LA-2, LL-4, CC-16, 2BR-19, and USS-18A correspond to mountainous areas and foothills of the Blue Ridge Mountains. Topographic characteristics of the Broad River watershed are also discussed in Subsection 2.4.1.2.2. The USACE HEC-HMS, Version 3.0.1 (Reference 272), modeling software was used for rainfall runoff and routing calculations. Figure 2.4.3-206 shows the HEC-HMS model watershed routing layout.

Unit hydrographs for all subbasins except Make-Up Pond C were derived from the techniques described in the regional unit hydrograph study for South Carolina, which was performed by the USGS (Reference 203). The USGS study uses a multiple regression analysis to describe regional unit hydrographs with an adjusted lag time, based on each region of the study. For the HEC-1 study, the unit hydrographs were subsequently converted to 1-hr. durations.

Methods adopted to account for nonlinear basin response at high rainfall rates include increasing the peak of each unit hydrograph by 20 percent and reducing the time to peak by approximately 33 percent. The remaining ordinates of the modified unit hydrographs were adjusted to maintain smooth unit hydrographs with the standard characteristic of 1 in. of runoff. To accommodate the Lake Whelchel subbasin and the Make-Up Pond C subbasin, the BR-15 subbasin unit hydrograph was also modified based on the decrease in drainage area. The resulting unit hydrographs for 19 of the subbasins except Make-Up Pond C are presented in Figure 2.4.3-207, Figure 2.4.3-208, and Figure 2.4.3-209 and tabulated in Table 2.4.3-205.

For the Make-Up Pond C subbasin, the SCS unit hydrograph method was used as a basis for a modified unit hydrograph to transform rainfall to runoff and account for nonlinear basin response. An equivalent SCS unit hydrograph was first determined using the equations and ratios of the SCS dimensionless unit hydrograph. The equivalent SCS unit hydrograph was then modified by increasing the peak of the unit hydrograph by 20 percent and reducing the time to peak by approximately 33 percent. The remaining ordinates of the modified unit hydrograph were adjusted to maintain a smooth unit hydrograph with the standard characteristic of 1 in. of runoff.

The best calibration of the modified SCS unit hydrograph with the initial SCS unit hydrograph was found using a 10-min. computational time step in the HEC-HMS modeling software. Therefore, the time step used to define the ordinates of the modified SCS unit hydrograph is also 10 min. The Make-Up Pond C subbasin has a lag time of 77 min. The initial SCS unit hydrograph and modified unit hydrograph to account for the effects of nonlinear basin response are provided in Figure 2.4.3-241. The modified SCS unit hydrograph is tabulated in Table 2.4.3-207.

The Muskingum-Cunge 8-point cross section method was used for the river routing reaches, except for the Green River reach between Lake Summit and Lake Adger. Because of the Lake Adger backwater effects on the reach, the Modified Puls storage routing method was used. Channel slope, length, and cross section data were developed using USGS quadrangles. Cross sections were field-verified as part of the existing study and modified as necessary. Manning's roughness coefficients were estimated on the basis of accepted published tables by Chow (Reference 206).

The existing study (Reference 217) contained discharge rating curves for the Tuxedo, Turner Shoals, Gaston Shoals and Ninety-Nine Islands dams. These curves were developed from Duke Power Company project file data. The rating curves for Lake Lure, Kings Mountain Reservoir, Cherokee Falls, and Lockhart dams were estimated in the existing study by using drawings obtained from the dam owners and the North Carolina State Dam Safety Engineer's office. Reservoirs were modeled using full-pond starting elevations and no turbine discharges were assumed. The flashboards at Gaston Shoals and Ninety-Nine Islands dams were assumed to fail due to overtopping and were incorporated into the rating curves. Additionally, the gates at Lake Lure were assumed to be closed. Reservoir rating curves are presented in Figure 2.4.3-210, Figure 2.4.3-211, Figure 2.4.3-212, Figure 2.4.3-213, Figure 2.4.3-214, Figure 2.4.3-215, Figure 2.4.3-216, and Figure 2.4.3-217.

The Lake Whelchel discharge rating curve is based on a riser with outlet pipe and spillway configuration. The riser maintains the normal pool elevation of 670 ft. The outlet pipe through the dam is a 48 in. concrete pipe. The spillway elevation varies from 680 ft. to 683 ft. The Lake Whelchel rating curve is presented in Figure 2.4.3-238. Lake Whelchel was modeled using a full-pond starting elevation.

The Make-Up Pond C discharge rating curve is based on the designed 4-cycle labyrinth spillway rating curve. Each cycle has a lateral width of 20 ft. The spillway crest elevation is 650 ft. Sensitivity analyses were performed based on a 10 percent increase and decrease of the designed labyrinth spillway rating curve. The Make-Up Pond C rating curve is presented in Figure 2.4.3-242. Make-Up Pond C was modeled using a full-pond starting elevation.

The entire watershed and individual subbasin unit hydrographs of the existing HEC-1 study were verified using three significant storm events occurring in October 1964, June 1972, and October 1976. Base-flow separation was estimated by evaluating semilog plots of each storm event and confirmed with historical daily mean flows at USGS gauging locations. Several USGS gauges are located throughout the watershed. Subbasin input parameters, including the modified

BR-15 subbasin, Lake Whelchel subbasin, and Make-Up Pond C subbasin, are listed in Table 2.4.3-206. The exponential recession method is used to model baseflow. The Lake Whelchel subbasin and the Make-Up Pond C subbasin use the same baseflow characteristics as the BR-15 subbasin with an adjusted recession threshold based on the ratio of drainage areas for the two subbasins. Snowmelt is not considered to be a factor in modeling the PMF event, as described in Subsection 2.4.3.1.

To assure HEC-HMS model calibration with the existing study, the HEC-HMS model was first examined using the existing HEC-1 model inputs without antecedent conditions or the modifications for the addition of the Lake Whelchel subbasin and the Make-Up Pond C subbasin. The results were satisfactorily comparable. The HEC-HMS model was then examined using the modifications for the addition of the Lake Whelchel subbasin and the Make-Up Pond C subbasin and the PMP with antecedent rainfall conditions.

Because of large magnitude flows and backwater effects at Gaston Shoals, Cherokee Falls, and Ninety-Nine Islands dams, a standard step method, unsteady-flow hydraulic analysis was performed to more accurately determine the water surface elevation at the Lee Nuclear Station. The USACE HEC-RAS, Version 3.1.3 (Reference 273), modeling software was used to route hydrographs from above Gaston Shoals Dam to Lockhart Dam.

Cross sections were estimated using the existing study, USGS quadrangles, and the USACE NID database. Cross section interpolations were done as necessary to provide a stabilized HEC-RAS model. Manning's roughness coefficients range from 0.03 to 0.08. Contraction and expansion coefficients are based on gradual transitions. Reservoir cross sections were created to approximate the volumes associated with each reservoir. Rating curves were approximated using modeled inline structures. The HEC-RAS model uses a 5-min. computation interval.

The HEC-RAS model is based on the existing study's NWS DAMBRK model. To assure HEC-RAS model calibration, the HEC-RAS model was examined using the DAMBRK input and without antecedent conditions. The results were satisfactorily comparable. Hydrographs from the HEC-HMS analysis, including antecedent rainfall and accounting for nonlinear basin response, were then used as inflow to the HEC-RAS model. Lateral inflows representing local flow between Gaston Shoals Dam and Ninety-Nine Islands Dam were also included in the model. Input hydrographs are shown in Figure 2.4.3-218, Figure 2.4.3-219, Figure 2.4.3-220, Figure 2.4.3-221, Figure 2.4.3-243, and Figure 2.4.3-245.

McKowns Creek/Make-Up Pond B

For McKowns Creek and Make-Up Pond B and the Upper Arm, HEC-HMS modeling software was used for rainfall runoff and storage routing calculations. The watershed is shown in Figure 2.4.3-201. Methods adopted to account for nonlinear basin response at high rainfall rates include increasing the peak of the unit hydrograph by 20 percent and reducing the time to peak by approximately 33 percent. Topographic characteristics of the site and watershed are described in Subsection 2.4.1.2.1.

The Soil Conservation Service (SCS) unit hydrograph method was used as a basis for a modified unit hydrograph to transform rainfall to runoff. An equivalent SCS unit hydrograph was first determined using the equations and ratios of the SCS dimensionless unit hydrograph. The equivalent SCS unit hydrograph was then modified by increasing the peak of the unit hydrograph by 20 percent and reducing the time to peak by approximately 33 percent. The remaining ordinates of the modified unit

hydrograph were adjusted to maintain a smooth unit hydrograph with the standard characteristic of 1 in. of runoff.

The best calibration of the modified SCS unit hydrograph with the initial SCS unit hydrograph was found using a 10-min. computational time step in Make-Up Pond B in the HEC-HMS modeling software. Therefore, the time step used to define the ordinates of the modified SCS unit hydrograph is also 10 min. The Make-Up Pond B subbasin has a lag time of 76.8 min. The initial SCS unit hydrograph and modified unit hydrograph to account for the effects of nonlinear basin response are provided in Figure 2.4.3-237. The modified SCS unit hydrograph is tabulated in Table 2.4.3-208.

The best calibration of the modified SCS unit hydrograph with the initial SCS unit hydrograph was found using a 2-min. computational time step in the Upper Arm watershed in the HEC-HMS modeling software. Therefore, the time step used to define the ordinates of the modified SCS unit hydrograph is also 2 min. The Upper Arm subbasin has a lag time of 16.2 min. The initial SCS unit hydrograph and modified unit hydrograph to account for the effects of nonlinear basin response are provided in Figure 2.4.3-246. The modified SCS unit hydrograph is tabulated in Table 2.4.3-209.

The drainage area, length of watercourse, and average slope of the Make-Up Pond B and Upper Arm watershed was determined from aerial topography created for the area. The lag time was determined using the standard SCS curve number regression equation:

$$T_{lag} = (L^{0.8} * (S+1)^{0.7}) / (1900 * Y^{0.5})$$

where

T_{lag}	=	lag time (hr.)
L	=	hydraulic length of the watershed (ft.)
S	=	maximum potential storage of the watershed (in.); where S = 1000/CN -10 and CN = average curve number for the watershed
Y	=	average watershed land slope (percent)

The resulting characteristic parameters for the Make-Up Pond B watershed are as follows:

Drainage Area (sq. mi.)	L (ft.)	CN	S (in.)	Y (%)	T_{lag} (hr.)
2.190	10,320	87	1.49	1.60	1.28

The resulting characteristic parameters for the Upper Arm watershed are as follows:

Drainage Area (sq. mi.)	L (ft.)	CN	S (in.)	Y (%)	T_{lag} (hr.)
0.294	3194	86	1.63	6.03	0.27

The curve number is used to determine the lag time only. During rainfall routing, the model does not use the curve number loss method, under the conservative assumption that precipitation losses do not occur. The curve number was developed using the NRCS Web Soil Survey (Reference 278) to determine the soil types in the watershed. About 95 percent of the soil belongs to Hydrologic Soil Group B, and the remaining 5 percent to Hydrologic Soil Group C. The land use is predominately wooded. Make-Up Pond B and the Upper Arm watersheds are modeled as impervious cover. Wet antecedent moisture conditions (AMC III) were also assumed.

Base flow was determined using the minimum average monthly flow of the Gaffney and Ninety-Nine Island gauges (USGS No. 02153500 and 02153551). The flow was then corrected on the basis of a ratio of drainage basin areas. Base flow was estimated to be 1.77 cfs for the Make-Up Pond B watershed and 0.24 cfs for the Upper Arm watershed. Baseflow is applied to the model as a constant rate.

Make-Up Pond B outflow structure rating curve was developed using standard weir and orifice flow equations with coefficients of 3.5 and 0.8 respectively. The structure is a 35 ft. wide concrete ogee spillway with a crest elevation of 570 ft. The road along Make-Up Pond B crest restricts the opening of the structure to a height of 13.5 ft. The outlet empties into backwaters of the Broad River. The Make-Up Pond B rating curve is provided in Figure 2.4.3-222. Available storage was determined based on aerial topography. Figure 2.4.3-223 provides the storage capacity curve. Full pond elevation of 570 ft. was assumed for antecedent conditions.

The Upper Arm Dam outlet structures consist of a 54 in. steel pipe with headwalls at both the upstream and downstream inverts. The upstream invert within the Upper Arm Dam is placed at an elevation of 575.0 ft., which is the normal full pond elevation. The downstream invert emptying into Make-Up Pond B is placed at an elevation of 570.0 ft. Figure 2.4.3-249 shows a schematic of the Upper Arm culvert structure. The Upper Arm culvert is evaluated considering full flow capacity and also no flow.

The access road separating the Upper Arm Dam from Make-Up Pond B is at elevation 590.0 ft. and acts as a broad-crested weir with a crest length of 390 ft. with a crest breadth of 8 ft. The maximum height of the dam is 15 ft. from the normal full pond elevation of 575 ft. up to the crest embankment. Water volume below 575 ft. is not considered due to nearly equivalent hydrostatic forces on both sides of the dam embankment during the PMF event. Overtopping of the Upper Arm dam crest is evaluated using the standard weir flow equation with a coefficient of 2.6. The Upper Arm Dam overtopping discharge rating curve is provided in Figure 2.4.3-247. Available storage was determined based on aerial topography. Figure 2.4.3-248 provides the storage capacity curve. Antecedent conditions for the normal full pond elevation were assumed to be 575 ft. based on historical observation.

Intermittent Stream/Make-Up Pond A

For the intermittent stream and Make-Up Pond A, HEC-HMS modeling software was used for rainfall runoff calculations. The watershed is shown in Figure 2.4.3-201. The following analysis for Make-Up Pond A does not account for nonlinear basin response at high rainfall rates. During severe flooding events, Make-Up Pond A is inundated by backwaters of the Broad River. Broad River flooding coincident with dam failures, as discussed in Subsection 2.4.4, exceeds the maximum flooding elevation for Make-Up Pond A. Therefore, coincident wind wave activity for Make-Up Pond A is based on flooding from the Broad River. By incorporating the Broad River analysis to determine the maximum water surface elevation, the Make-Up Pond A coincident wind wave evaluation accounts

for nonlinear basin response at high rainfall rates as discussed above. Topographic characteristics of the site and watershed are described in Subsection 2.4.1.2.1.

The SCS unit hydrograph method was used to transform rainfall to runoff. The drainage area, length of watercourse, and average slope of the watershed were determined from aerial topography created for the area. The lag time was determined using the standard SCS curve number regression equation:

$$T_{lag} = (L^{0.8} * (S+1)^{0.7}) / (1900 * Y^{0.5})$$

where

T_{lag}	=	lag time (hr.)
L	=	hydraulic length of the watershed (ft.)
S	=	maximum potential storage of the watershed (in.); where S = 1000/CN -10 and CN = average curve number for the watershed
Y	=	average watershed land slope (percent)

The resulting characteristic parameters for the watershed are as follows:

Drainage Area (sq. mi.)	L (ft.)	CN	S (in.)	Y (%)	T_{lag} (hr.)
0.619	3340	92	0.87	3.48	0.29

The curve number is used to determine the lag time only. During rainfall routing, the model does not use the curve number loss method, under the conservative assumption that precipitation losses do not occur. The curve number was developed using the NRCS Web Soil Survey (Reference 278) to determine the soil types in the watershed. About 95 percent of the soil belongs to Hydrologic Soil Group B, and the remaining 5 percent to Hydrologic Soil Group C. The land use is predominately industrial. Make-Up Pond A is modeled as impervious cover. Wet antecedent moisture conditions (AMC III) were also assumed.

Base flow was determined using the minimum average monthly flow of the Gaffney and Ninety-Nine Island gauges (USGS No. 02153500 and 02153551). The flow was then corrected on the basis of a ratio of drainage basin areas. Base flow was estimated to be 0.50 cfs and applied to the model as a constant rate.

Although the full pond elevation is 547 ft., the crest elevation low point of 555.1 ft. was assumed for water surface elevation antecedent conditions. Make-Up Pond A overtopping flows empty into backwaters of the Broad River. The outflow rating curve was developed using the standard weir flow equation with a 2.6 discharge coefficient. The embankment crest is approximately 1500 ft. long and has an irregular shape. The rating curve is provided in Figure 2.4.3-224. Available storage was determined based on aerial topography. Figure 2.4.3-225 provides the storage capacity curve.

London Creek/Make-Up Pond C

For London Creek and Make-Up Pond C, HEC-HMS modeling software was used for rainfall runoff calculations. The watershed is shown in Figure 2.4.3-239. The SCS unit hydrograph method was used as a basis for a modified unit hydrograph to transform rainfall to runoff and account for nonlinear basin response. As discussed above for the Make-Up Pond C subbasin in the Broad River watershed, an equivalent SCS unit hydrograph was first determined using the equations and ratios of the SCS dimensionless unit hydrograph. The equivalent SCS unit hydrograph was then modified by increasing the peak of the unit hydrograph by 20 percent and reducing the time to peak by approximately 33 percent. The remaining ordinates of the modified unit hydrograph were adjusted to maintain a smooth unit hydrograph with the standard characteristic of 1 in. of runoff.

The best calibration of the modified SCS unit hydrograph with the initial SCS unit hydrograph was found using a 10-min. computational time step in the HEC-HMS modeling software. Therefore, the time step used to define the ordinates of the modified SCS unit hydrograph is also 10 min. The initial SCS unit hydrograph and modified unit hydrograph to account for the effects of nonlinear basin response are provided in Figure 2.4.3-241. The modified SCS unit hydrograph is tabulated in Table 2.4.3-207.

The drainage area, length of watercourse, and average slope of the watershed were determined from aerial topography created for the area. The lag time was determined using the standard SCS curve number regression equation:

$$T_{lag} = (L^{0.8} * (S+1)^{0.7}) / (1900 * Y^{0.5})$$

Where:

T_{lag}	=	lag time (hr.)
L	=	hydraulic length of the watershed (ft.)
S	=	maximum potential storage of the watershed (in.); where S = 1000/CN -10 and CN = average curve number for the watershed
Y	=	average watershed land slope (percent)

The resulting characteristic parameters for the watershed are as follows:

Drainage Area (sq. mi.)	L (ft.)	CN	S (in.)	Y (%)	T_{lag} (min.)
3.87	5393	63.9	5.65	2.23	77

The curve number was developed using the NRCS Web Soil Survey (Reference 300) to determine the soil types in the watershed. About 87.4 percent of the soil belongs to Hydrologic Soil Group B, 10.4 percent belonging to Hydrologic Soil Group C, and the remaining 2.2 percent to Hydrologic Soil Group C/D and D. The land use is predominately wooded, grassland, and large lot residential. The watershed contains approximately 27.8 percent impervious cover, including Make-Up Pond C and

Lake Cherokee. Average antecedent moisture conditions (AMC II) were used, along with the 40 percent PMP antecedent rainfall.

Base flow was determined based on the Broad River watershed BR-15 subbasin. The recession baseflow method was used with an initial discharge per area of 1.63 cfs/sq. mi. and a recession constant of 0.4919. The recession threshold was calculated to be 23 cfs based on a ratio of the Make-Up Pond C and BR-15 subbasin drainage areas.

The Make-Up Pond C discharge rating curve is based on the designed 4-cycle labyrinth spillway rating curve. Each cycle has a lateral width of 20 ft. The spillway crest elevation is 650 ft. Sensitivity analyses were performed based on a 10 percent increase and decrease of the designed labyrinth spillway rating curve. The Make-Up Pond C rating curve is presented in Figure 2.4.3-242. Available storage was determined based on aerial topography. Figure 2.4.3-244 provides the storage capacity curve. A full pond elevation of 650 ft. msl was assumed for antecedent conditions.

2.4.3.4 Probable Maximum Flood Flow

Broad River

Applying the precipitation, described in Subsection 2.4.3.1, and the precipitation losses, described in Subsection 2.4.3.2, to the runoff model, described in Subsection 2.4.3.3, the peak PMF discharge at the Lee Nuclear Station was determined to be 823,212 cfs resulting from the 1000-sq. mi. storm centered near the centroid of the Gaston Shoals Dam drainage basin. The resulting flow hydrograph at the Lee Nuclear Station is shown in Figure 2.4.3-226. Temporal distribution of the PMP and storm location is discussed in Subsection 2.4.3.1. Inclusion of upstream and downstream river structures is discussed in Subsection 2.4.3.3. Dam failures are discussed in Subsection 2.4.4. No credit is taken for the lowering of flood levels at the site due to downstream dam failure.

McKowns Creek/Make-Up Pond B

The precipitation, described in Subsection 2.4.3.1, with no precipitation losses, described in Subsection 2.4.3.2 is applied to the runoff model, described in Subsection 2.4.3.3. Assuming the Upper Arm Dam culvert is not functional produces the maximum conditions. The McKowns Creek and Make-Up Pond B peak PMF runoff was determined to be 20,039 cfs resulting from the 6-hr. two-thirds peaking storm event. The routed peak discharge is 6471 cfs.

However, the 72-hr. tail end peaking storm event resulting in a peak PMF runoff of 18,937 cfs and a routed discharge of 8386 cfs provided the controlling water surface elevation. The peak runoff in the Upper Arm Dam during the 72-hr. tail end peaking storm event will be 3577 cfs with a peak discharge of 3549 cfs. The resulting Make-Up Pond B flow hydrograph for the 72-hr. tail end peaking storm event is shown in Figure 2.4.3-227. Temporal distribution of the PMP is discussed in Subsection 2.4.3.1.

Because the Make-Up Pond B and Upper Arm Dam watersheds are small, the position of the PMP is considered point rainfall affecting the entire watershed equally. With the exception of the Upper Arm Dam, there are no upstream structures. Failure of the Upper Arm Dam is discussed in Subsection 2.4.4. No credit is taken for the lowering of flood levels at the site due to downstream dam failure.

Intermittent Stream/Make-Up Pond A

Applying the precipitation, described in Subsection 2.4.3.1, with no precipitation losses, described in Subsection 2.4.3.2, to the runoff model, described in Subsection 2.4.3.3, the intermittent stream and Make-Up Pond A peak PMF runoff was determined to be 11,644 cfs resulting from the 6-hr. storm event. The routed peak discharge is 9847 cfs. The resulting flow hydrograph is shown in Figure 2.4.3-228. Temporal distribution of the PMP is discussed in Subsection 2.4.3.1. Because the watershed is small, the position of the PMP is considered point rainfall affecting the entire watershed equally. There are no upstream structures. No credit is taken for the lowering of flood levels at the site due to downstream dam failure.

London Creek/Make-Up Pond C

Applying the precipitation, described in Subsection 2.4.3.1, and the precipitation losses, described in Subsection 2.4.3.2, to the runoff model, described in Subsection 2.4.3.3, the London Creek and Make-Up Pond C peak PMF runoff providing the highest water surface elevation from the 72-hr. tail end peaking storm event was determined to be 29,167 cfs. The routed peak discharge is 10,577 cfs. Temporal distribution of the PMP is discussed in Subsection 2.4.3.1. Because the watershed is small, the position of the PMP is considered point rainfall affecting the entire watershed equally. The upstream Lake Cherokee watershed was incorporated into the Make-Up Pond C watershed. Therefore, Lake Cherokee was assumed to pass runoff flow without any detention. No credit is taken for the lowering of flood levels at the Lee Nuclear Station due to downstream dam failure.

2.4.3.5 Water Level Determinations

Broad River

Subsection 2.4.4.3 addresses coincident wind wave activity for the Broad River. The maximum Lee Nuclear Station flood elevation is 551.49 ft. resulting from the 1000-sq. mi. storm centered near the centroid of the Gaston Shoals Dam drainage basin. Subsection 2.4.3.3 describes the models used to translate the PMP discharge to the elevation hydrograph. The resulting elevation hydrograph at the Lee Nuclear Station is shown in Figure 2.4.3-229. The maximum flood elevation is well below the station’s safety-related plant elevation of 593 ft.

McKowns Creek/Make-Up Pond B

Subsection 2.4.4.3 addresses coincident wind wave activity for Make-Up Pond B. The maximum water surface elevation of Make-Up Pond B without considering Upper Arm Dam failure, resulting from the 6-hr. two-thirds peaking storm event modeled with a 1-min. time step, was found to be 583.29 ft. The elevation hydrograph is provided in Figure 2.4.3-230. The maximum water surface elevation of Make-Up Pond B resulting from the 72-hr. tail end peaking storm event modeled with a 1-min. time step was found to be 584.40 ft. The maximum is produced by the condition that the Upper Arm Dam culvert is not functional, but does include overtopping flows. The peak water surface elevation in the Upper Arm Dam for the 72-hr. tail end, peaking storm will be 592.28 ft. The ridge on the east side of the Upper Arm Dam separates the Upper Arm and the site, as illustrated in Figure 2.4.3-201. At elevations above 590.0 ft., discharge across the dam embankment flows directly into Make-Up Pond B. Nevertheless, peak water surface elevations for the Upper Arm are below the station’s safety-related plant elevation of 593 ft. The elevation hydrograph for Make-Up Pond B is provided in Figure 2.4.3-231.

Make-Up Pond B includes an adequately sized outlet structure and is not located on a sizeable river or stream. Therefore, the potential for significant debris to be picked up by a rise in the water level and then transported to the outlet structure where it could collect as an obstruction is minimal. Blockage of the outlet structure was not considered in the analysis and debris blockage of the outlet structure is not considered to be a credible event due to Duke Energy's shoreline management program and debris barrier system discussed in Subsection 2.4.1.2.2.6.

Intermittent Stream/Make-Up Pond A

Subsection 2.4.4.3 addresses coincident wind wave activity for Make-Up Pond A. The maximum water surface elevation of Make-Up Pond A, resulting from the 6-hr. storm, two-thirds peaking distribution, modeled with a 1-min. time step, was found to be 558.15 ft. The elevation hydrograph is provided in Figure 2.4.3-233. Subsection 2.4.3.3 describes the models used to translate the PMP discharge to elevation.

London Creek/Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, but is not adjacent to the Lee Nuclear Station. However, the PMF for London Creek and Make-Up Pond C is determined for the purpose of combination with dam failure permutations as discussed in Subsection 2.4.4.1. Because the PMF discharge flow from Make-Up Pond C is bounded by the Broad River watershed PMF, spillover from Make-Up Pond C during a PMF event is not a limiting event for flooding at the Lee Nuclear Station when taken as an isolated event. For reference to the dam failure permutations, the maximum water surface elevation of Make-Up Pond C, resulting from the 72-hr storm modeled with a 10 min. time step, was found to be 656.68 ft. Subsection 2.4.3.3 describes the models used to translate the PMP discharge to elevation.

2.4.3.6 Coincident Wind Wave Activity

Coincident wind wave activity is evaluated for the Broad River, Make-Up Pond A and Make-Up Pond B. Fetch lengths are determined using the longest straight line fetch based on U.S. Geological Survey quadrangles and the site grading and drainage plan. Wave height, setup, and runup are estimated using U.S. Army Corps of Engineers guidance (Reference 295). A coincident 2-year annual extreme mile wind speed of 50 mph is estimated based on ANSI/ANS-2.8-1992 (Reference 202). Wind setup is estimated using additional U.S. Army Corps of Engineers guidance (Reference 269).

Broad River

Coincident wind wave activity for the Broad River is addressed in Subsection 2.4.4.3.

Intermittent Stream/Make-Up Pond A

Coincident wind wave activity for Make-Up Pond A is addressed in Subsection 2.4.4.3.

McKowns Creek/Make-Up Pond B

Coincident wind wave activity for Make-Up Pond B is addressed in Subsection 2.4.4.3.

London Creek/Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, such that wind wave activity has no consequence to the Lee Nuclear Station. However, a postulated failure of the Make-Up Pond C dam would release water to the Broad River prior to reaching the Lee Nuclear Station. A failure of the Make-Up Pond C dam coincident with the PMF is discussed in Subsection 2.4.4.1, and flooding effects as a result of wind wave activity are bounded by that discussion.

2.4.4 Potential Dam Failures

The guidance in Appendix A of NRC Regulatory Guide 1.59, Rev. 2, *Design Basis Floods for Nuclear Power Plants*, was followed in evaluating potential dam failures, by applying the guidance of American National Standards Institute/American Nuclear Society-2.8-1992, *Determining Design Basis Flooding at Power Reactor Sites* (Reference 202).

The Upper Broad River drainage basin upstream of Ninety-Nine Islands Dam derives water from several tributaries that contain a considerable number of dams. According to the U.S. Army Corps of Engineers (USACE), National Inventory of Dams, there are approximately 131 upstream dams, not including Make-Up Pond C, and five of those have been breached (Reference 276). Most of the dams in the drainage basin have small to insignificant storage capacity. The six largest reservoirs in the basin represent about 88 percent of the total storage capacity for the basin. Two additional dams, Cherokee Falls and Gaston Shoals, located immediately upstream from the Lee Nuclear Station, possess less than 2 percent of the total storage capacity for the basin.

Make-Up Pond B and Make-Up Pond A are located at elevations much lower than the Lee Nuclear Station's safety-related facilities. Failure of these water features would result in a discharge to smaller ponds and then directly to the Broad River. The respective volumes are small compared to the available capacity of the Broad River and the freeboard available at the site. Failure of the on-site reservoirs would not affect the safety-related facilities.

The Upper Arm Dam is located upstream of Make-Up Pond B southwest of the nuclear island. Failure of this dam would result in discharges directly to Make-Up Pond B. The resulting rapid increase of water volume would increase the peak water surface levels and discharge rates in Make-Up Pond B. The volume of discharge from the Upper Arm Dam is small compared to the volume of Make-Up Pond B. Failure of this reservoir will not affect the safety-related facilities.

Make-Up Pond C is located on a tributary of the Broad River, west of the Lee Nuclear Station. As described below, the critical dam failure evaluation coincident with the PMF for the Broad River watershed includes the assumed overtopping failure of Make-Up Pond C. Assumed overtopping dam failure coincident with the PMF for the Make-Up Pond C watershed has also been evaluated, but does not exceed the maximum flood elevation associated with the Broad River critical dam failure event and, thus, is bounded by the critical dam failure event. Therefore, there are no safety-related structures that could be affected by flooding due to a Make-Up Pond C dam failure.

The described studies have been made solely to ensure the safety-related facilities of the Lee Nuclear Station are protected against floods caused by the assumed failure of dams. The postulated dam failure events do not infer or concede that the dams are unsafe.

The critical dam failure event is the assumed overtopping failures of Lake Lure Dam, Tuxedo Dam, Turner Shoals Dam, Lake Whelchel Dam, Kings Mountain Reservoir Dam, and Make-Up Pond C Dam, including the dam at Lake Cherokee, coincident with the probable maximum flood (PMF). The resulting flow rate and water surface elevation at the station is provided in the discussion below. There are no safety-related structures that could be affected by flooding due to dam failure. All elevations provided in this subsection are above mean sea level.

2.4.4.1 Dam Failure Permutations

According to guidance (Reference 202), seismic dam failure is to be examined using the safe shutdown earthquake coincident with the peak of the 25-year flood and operating basis earthquake coincident with the peak of one-half PMF or the 500-year flood. Dam failure permutations were first examined assuming hydrologic failure of dams coincident with the PMF. Many of the upstream structures are designed to withstand overtopping. However, structural analysis of each structure has not been performed. The PMF is a more extreme event than the listed hydrologic events coincident with seismic dam failure. Seismic dam failure coincident with lesser flooding would result in lower flood elevations and has not been examined. Therefore, the evaluations described below comply with Regulatory Guide 1.59.

Broad River

The considered upstream structures are described below. Reservoirs were modeled using normal water surface elevations with no turbine discharges. Additionally, the gates at Lake Lure were assumed to be closed. Antecedent conditions are discussed in Subsection 2.4.3.

Failure of the downstream structure, Ninety-Nine Islands Dam, would result in lowering the water surface elevation at the Lee Nuclear Station to some degree. Conservatively, Ninety-Nine Islands Dam has not been considered to fail during any of the dam failure scenarios. However, failure of the flashboards has been incorporated into the rating curve.

Cherokee Falls and Gaston Shoals

Cherokee Falls Dam is approximately 4.5 river mi. upstream of Ninety-Nine Islands Dam on the Broad River in Cherokee County, South Carolina. The dam, built in 1826, is a concrete gravity structure approximately 1700 ft. long and 16 ft. high. It has an ogee spillway elevation of 531.5 ft. with 4-ft. flashboards raising the operating pond level to 535.5 ft. The impounded reservoir has an estimated storage capacity of 200 ac.-ft. at normal water surface elevation. Flashboard failure is incorporated into the discharge rating curve used for the structure.

Gaston Shoals Dam is approximately 11.5 river mi. upstream of Ninety-Nine Islands Dam on the Broad River in Cherokee County, South Carolina. The dam, built in 1908, is a series of three gravity structures. The upper masonry gravity spillway is about 707 ft. long with an overflow spillway crest elevation of 599.40 ft. and 6-ft. flashboards that raise the operating pond level to 605.4 ft. The middle concrete gravity section was built in 1917 and is about 381 ft. long. The overflow spillway crest elevation is 601.2 ft. with 4-ft. flashboards up to 605.2 ft. The masonry gravity bulkhead section is about 472 ft. long with a crest elevation of 613.4 ft. The impounded reservoir has an estimated storage capacity of 2500 ac.-ft. at normal water surface elevation. Flashboard failure is incorporated into the discharge rating curve used for the structure.

Both dams are significantly overtopped for a lengthy duration during PMF conditions. Dam failure has been conservatively assumed to occur coincident with the PMF peak flood wave in order to maximize water surface elevations. The breach characteristics for Cherokee Falls assume complete failure of the full height and length of the structure to occur in 0.5 hours (hr.). The breach characteristics for Gaston Shoals assume failure of the full height and length of the middle spillway structure to occur in 0.5 hr., along with failure of the embankment abutments separating the three structures.

An overtopping breach of Gaston Shoals, coincident with the PMF, results in a flow of 824,000 cfs and a water surface elevation of 551.52 ft. at the Lee Nuclear Station. Overtopping breaches of both Gaston Shoals and Cherokee Falls, coincident with the PMF, result in the same flow and water surface elevation. Because of the small reservoir volumes and large PMF discharge, the dam failures have little effect on the resulting flow and water surface elevations.

Major Upstream Structures

Lake Lure is about 47 mi. northwest of the Lee Nuclear Station on the Broad River in Rutherford County, North Carolina. The dam, built in 1927, is a concrete, multiple-arch structure approximately 480 ft. long and 124 ft. high, with a full pond elevation at 991 ft. There are gated spillways and the arches are set at various elevations, providing additional discharge capacity. The discharge rating curve, used for modeling purposes, conservatively assumes the gates are in the closed position. The impounded reservoir has an estimated storage capacity of about 32,295 ac.-ft. at normal water surface elevation.

Tuxedo Dam, impounding Lake Summit, is about 52 mi. northwest of the Lee Nuclear Station on the Green River in Henderson County, North Carolina. The dam, built in 1920, is a concrete-arch structure approximately 254 ft. long and 130 ft. high, with a full pond elevation at 2012.6 ft. The impounded reservoir has an estimated storage capacity of about 9300 ac.-ft. at normal water surface elevation.

Turner Shoals Dam, impounding Lake Adger, is about 43 mi. northwest of the Lee Nuclear Station, downstream of Tuxedo Dam on the Green River in Polk County, North Carolina. The dam, built in 1925, is a concrete, multiple-arch structure approximately 689 ft. long and 90 ft. high, with a full pond elevation at 911.6 ft. The impounded reservoir has an estimated storage capacity of about 11,700 ac.-ft. at normal water surface elevation.

Kings Mountain Reservoir Dam, also referred to as Moss Lake Dam, is about 17 mi. northeast of the Lee Nuclear Station on Buffalo Creek in Cleveland County, North Carolina. The dam, built in 1973, is a compacted earth-fill structure approximately 840 ft. long and 99 ft. high. The top of the dam is at an elevation of 750 ft. The spillway is located at the right abutment and consists of a 350 ft. long concrete ogee section with a crest elevation of 736 ft. The impounded reservoir has an estimated storage capacity of 44,400 ac.-ft. at normal water surface elevation.

Lake Whelchel is located approximately 8 mi. northwest of the Lee Nuclear Station on Cherokee Creek in Cherokee County, South Carolina. The dam at Lake Whelchel, built in 1964, is a compacted earth-fill structure approximately 2100 ft. long and 61 ft. high. The dam crest elevation is 685 ft. A riser and 48 in. concrete pipe outlet works sets the normal pool elevation at 670 ft. The spillway is 565 ft. long and varies in elevation from 680 ft. to 683 ft. Lake Whelchel has an estimated storage capacity of approximately 2438 ac.-ft. at normal water surface elevation.

Make-Up Pond C is located approximately 2 mi. west of the Lee Nuclear Station on London Creek in Cherokee County, South Carolina. Make-Up Pond C is formed by construction of an earthen dam and saddle dikes that impound London Creek just upstream of the confluence with Little London Creek. The Make-Up Pond C dam crest elevation is 660 ft. A labyrinth spillway sets the normal pool elevation at 650 ft. The designed 4-cycle labyrinth spillway has a lateral width of 20 ft. per cycle. The dam is 132 ft. high. The impounded reservoir has an estimated storage capacity of approximately 22,000 ac.-ft. at normal water surface elevation.

Lake Lure Dam, Tuxedo Dam, and Turner Shoals Dam are designed to withstand overtopping. However, the structural integrity of the dams and foundations has not been examined. The degree and duration of overtopping each dam is capable of withstanding is not considered in this evaluation. Therefore, overtopping dam failure has been calibrated to occur coincident with the PMF peak flood wave in order to maximize water surface elevations for Lake Lure Dam and Tuxedo Dam. Lake Summit and Lake Adger are located in series on the Green River. Overtopping failure of the Turner Shoals Dam was calibrated to coincide with the resulting peak flood wave of the Tuxedo Dam failure. Breach parameters assume failure of the complete structures to occur in 0.1 hr.

Kings Mountain Reservoir Dam, the Lake Whelchel Dam, and the Make-Up Pond C Dam are not expected to be overtopped based on the PMF analysis with antecedent storm conditions. However, overtopping failure is postulated for this analysis, and dam failures have been calibrated to occur coincident with the PMF peak flood wave in order to maximize water surface elevations.

Lake Cherokee is located just upstream of Make-Up Pond C on a tributary of London Creek in Cherokee County, South Carolina. The dam is a compacted earth-fill structure approximately 940 ft. long, 40 ft. high and has an estimated maximum storage capacity of 720 ac.-ft. The dam at Lake Cherokee is assumed to fail by overtopping based on the full height of the structure. The peak failure flow is derived using the HEC-HMS dam failure equation identified below. No tailwater elevation was assumed, maximizing the head difference and breach outflow. The peak outflow is added to the PMF peak flood wave for the Make-Up Pond C watershed to maximize the Make-Up Pond C dam failure.

$$Q_{max} = 3.09 * W_b * h^{1.5} + 2.48 * S * h^{2.5}$$

Where:

Q_{max} = peak outflow (cfs)

W_b = width of breach (ft.)

h = smaller of the head difference between the reservoir interior water surface elevation and the tailwater surface elevation, or head difference between the reservoir interior water surface elevation and the breach bottom invert elevation (ft.)

S = side slope of the breach

Embankment breach characteristics are based on the USACE RD-13 (Reference 250). Failure development time for embankment sections is estimated to occur from 0.5 to 4 hr. Breach width for embankment sections is estimated to be from 0.5 to 3 times the dam height. Side slopes for an embankment breach are estimated to be from 0:1 to 1:1 (horizontal:vertical). To maximize the peak

outflow, a breach width of 3 times the dam height was used along with 1:1 side slopes and the shortest failure development time of 0.5 hr.

Sensitivity was also performed based on the time of failure for the various structures. Additionally, several failure times were examined based on the peak outflow time at Ninety-Nine Islands Dam. Using the same breach parameters as discussed above, all structures were assumed to fail simultaneously, rather than individually based on the peak flood wave at each dam. It was determined that the critical dam failure scenario occurred when all dams failed simultaneously with a failure time near to the peak PMF outflow at Ninety-Nine Islands Dam.

The multiple failures due to overtopping, coincident with the PMF, result in a peak flow of approximately 1,850,000 cfs. The peak flow is determined using the HEC-HMS model discussed in Subsection 2.4.4.2.

McKowns Creek/Make-Up Pond B

As described earlier in Subsection 2.4.4, the failure of the Upper Arm Dam would directly impact Make-Up Pond B. The dam crest is at 590 ft. Simulation of dam failure was performed in HEC-HMS. Embankment breach parameters were selected based on the USACE RD-13 (Reference 250) document. Failure development time for embankment sections is estimated to occur at 0.5 hr. from the onset of dam breach. Breach width for embankment sections is estimated to be 3 times the height of the Upper Arm Dam as described in Subsection 2.4.3.3. Side slopes for the embankment breach facing the Make-Up Pond B are set at 1:1. Dam breach parameters were selected to maximize the peak outflow.

The maximum peak PMF runoff from Make-Up Pond B, considering Upper Arm Dam failure, resulting from the 6-hr. tail end peaking storm event modeled with a 1-minute time step, was found to be 23,726 cfs. However, the controlling water surface elevation resulted from the 72-hr. tail end peaking storm event modeled with a 1-minute time step. The peak elevation is produced by the condition that the Upper Arm Dam culvert is not functional. The peak PMF runoff from the 72-hr. tail end peaking storm into Make-Up Pond B was found to be 23,515 cfs. The peak runoff hydrograph is provided in Figure 2.4.4-203. The peak runoff in the Upper Arm Dam resulting from the 72-hr. tail end peaking storm is 3577 cfs with a dam failure peak discharge of 6785 cfs.

Make-Up Pond C Dam

Assumed overtopping dam failure of the Make-Up Pond C Dam has also been evaluated coincident with a more intense PMF confined to the smaller Make-Up Pond C watershed as described in Subsection 2.4.3. As previously discussed, failure of the dam at Lake Cherokee was also included to maximize the peak dam failure outflow from Make-Up Pond C.

The Make-Up Pond C peak dam failure outflow was combined with the maximum historical flow recorded on the Broad River at Gaffney, identified in Table 2.4.2-201, to account for any coincidental flow in the Broad River. However, the resulting combined peak outflow of 1,336,000 cfs does not exceed the critical dam failure event for the Broad River watershed previously described. Therefore, even if routed to the Lee Nuclear Station without attenuation, the resulting water surface elevation would not exceed the elevation determined from the critical multiple dam failure scenario coincident with the Broad River watershed PMF.

Cleveland County Sanitary District

According to the Federal Register (Reference 226), a notice of intent was filed on July 11, 2006, for a Draft Environmental Impact Statement (DEIS) to be prepared for a proposed reservoir on the First Broad River in Cleveland County, North Carolina. The Cleveland County Sanitary District applied for a permit to construct a water supply reservoir about 1 mi. north of Lawndale, North Carolina. The DEIS is currently in preparation. Lawndale is about 26 mi. northeast of the Lee Nuclear Station.

The proposed embankment dam may be about 83 ft. high and 1245 ft. long, impounding a surface area of 2245 ac. and inundating areas lower than 860 ft. Using USGS quadrangle contours, volume calculations estimate the storage to be about 47,500 ac.-ft. The embankment is approximately the same size as Kings Mountain Reservoir. The reservoir contains approximately the same storage with twice the surface area of the Kings Mountain Reservoir. It is assumed that the dam is designed to prevent failure during a PMF event. Based on the distance from the Lee Nuclear Station and available freeboard, it is also assumed that any failure effects due to seismic activity coincident with lesser flood events would be no worse than those estimated for the Kings Mountain Reservoir. The above evaluation includes failure of the Kings Mountain Reservoir Dam during the PMF. Therefore, any failure effects from the proposed embankment dam would be less than those provided.

Other Considerations

There are no safety-related facilities that could be affected by loss of water supply due to dam failure. This is addressed further in Subsection 2.4.11. Additionally, there are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during dam failure induced flooding. There are no onsite water control or storage structures located above site grade that may induce flooding. As discussed in Subsection 2.4.4.3, the Lee Nuclear Station's safety-related facilities are located above the resulting water surface elevation. Therefore, no safety-related structures could be affected by waterborne missiles.

2.4.4.2 Unsteady-Flow Analysis of Potential Dam Failures

The failures for the dams immediately upstream of the Lee Nuclear Station, Cherokee Falls and Gaston Shoals, were examined using the same HEC-RAS unsteady flow model from the PMF calculation described in Subsection 2.4.3. The model was modified by including the HEC-RAS breach feature and adjusting the computation interval to 30 sec. Unsteady state flow is computer solved using the principles of the continuity and momentum equations.

The failures of the additional dams upstream of the Lee Nuclear Station, were first examined using the same HEC-HMS quasi-unsteady flow model from the PMF calculation described in Subsection 2.4.3. The dam breach feature in the HEC-HMS model was used to determine the resulting flow of the Broad River at Ninety-Nine Islands Dam. HEC-HMS employs finite difference methods approximating the continuity and momentum equations.

The HEC-HMS peak flow determined at Ninety-Nine Islands Dam was then used as input to the HEC-RAS model from the PMF calculation described in Subsection 2.4.3. Steady state analysis was performed to determine the water surface elevation at the Lee Nuclear Station. Steady state flow is computer solved using the principles of the continuity and energy equations.

Verification of the models is discussed in Subsection 2.4.3. However, verifying the models with actual data approaching the magnitude of the PMF is not possible. The resulting extreme flows determined

using HEC-HMS, HEC-RAS unsteady state flow, and HEC-RAS steady state flow are discussed below. The comparative results indicate the models are appropriate for artificially large floods.

The HEC-RAS models are used to route the flood flow through downstream Ninety-Nine Islands Dam and Lockhart Dam. Coefficients, antecedent conditions, and coincident flow are discussed above and in Subsection 2.4.3. Domino-type failure is discussed above. As discussed in Subsection 2.4.4.3, the Lee Nuclear Station's safety-related facilities are located above the resulting water surface elevation. Therefore, no safety-related structures could be affected by flood waves.

2.4.4.3 Water Level at the Plant Site

The methods and models used to determine the resulting water surface elevation are described above and in Subsection 2.4.3. Model verification and reliability is also discussed above and in Subsection 2.4.3. The HEC-RAS model, as described above, was used to model a resulting steady state flow of 1,850,000 cfs to determine the water surface elevation at the station.

The resulting water surface elevation at the Lee Nuclear Station is 576.50 ft. The maximum flood elevation is well below the station's safety-related plant elevation of 593 ft. The resulting water surface elevation of the dam failure analysis using HEC-HMS and HEC-RAS was compared with the resulting water surface elevations of the PMF analysis using HEC-HMS and HEC-RAS. The comparison is provided in Table 2.4.4-201. Given the significant freeboard remaining at the site, a full unsteady-flow analysis to determine dam breach flows and resulting water surface elevations with greater precision was determined to be unnecessary.

McKowns Creek/Make-Up Pond B

Using the HEC-HMS model, the maximum water surface elevation of Make-Up Pond B, considering Upper Arm Dam failure, resulting from the 72-hr. tail end peaking storm event modeled with a 1-min. time step was found to be 585.06 ft. The maximum is produced by the condition that the Upper Arm Dam culvert is not functional. The elevation hydrograph is provided in Figure 2.4.4-205. The peak water surface in the Upper Arm Dam resulting from the 72-hr. tail end peaking storm is 592.28 ft. The ridge on the east side of the Upper Arm separates the Upper Arm and the site, as illustrated in Figure 2.4.3-201. At elevations above 590.0 ft., discharge across the dam embankment flows directly into Make-Up Pond B. Nevertheless, peak water surface elevations for the Upper Arm are below the station’s safety-related plant elevation of 593 ft.

Coincident Wind Wave Activity

Coincident wind wave activity is evaluated for the Broad River, Make-Up Pond A and Make-Up Pond B. Fetch lengths are estimated using the longest straight line fetch based on U.S. Geological Survey quadrangles and the site grading and drainage plan. Wave height, setup, and runup are estimated using U.S. Army Corps of Engineers guidance (Reference 295). A coincident 2-year annual extreme mile wind speed of 50 mph is estimated based on ANSI/ANS-2.8-1992 (Reference 202). Wind setup is estimated using additional U.S. Army Corps of Engineers guidance (Reference 269).

Broad River

Wind wave activity on the Broad River is evaluated coincident with the maximum water surface elevation of the PMF including the effects of dam failures as discussed above. The determined fetch

length of 2.77 mi., shown in Figure 2.4.4-201, has a runup slope of 40 percent. The PMF including effects of dam failures and the coincident wind wave activity results in a flood elevation of 584.79 ft. msl. The Lee Nuclear Station safety-related plant elevation is 593 ft. msl and is unaffected by flood conditions and coincident wind wave activity. A more critical wind wave activity result was determined considering a fetch length through Make-Up Pond A, which becomes inundated by backwaters of the Broad River during severe flooding events. Therefore, the critical wind wave activity for the Broad River is equal to the wind wave activity for Make-Up Pond A, as discussed below.

Intermittent Stream/Make-Up Pond A

During severe flooding events, Make-Up Pond A is inundated by backwaters of flooding of the Broad River. Therefore, wind wave activity on Make-Up Pond A is evaluated coincident with the maximum water surface elevation of the PMF on the Broad River including the effects of dam failures as discussed above. The determined critical fetch length of 2.69 mi. is shown in Figure 2.4.4-202. The 2-year annual extreme mile wind speed is adjusted based on the factors of fetch length, level overland or over water, critical duration, and stability. The critical duration is approximately 53 min. The adjusted wind speed is 49.9 mph.

Significant wave height (average height of the maximum 33-1/3 percent of waves) is estimated to be 2.76 ft., crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 4.59 ft., crest to trough. The corresponding wave period is 2.6 sec.

The 47 percent slopes along the banks of Make-Up Pond A adjacent to the site are used to determine the wave setup and runup. The maximum runup, including wave setup, is estimated to be 8.79 ft. The maximum wind setup is estimated to be 0.07 ft. Therefore, the total wind wave activity is estimated to be 8.86 ft. The PMF including effects of dam failures and the coincident wind wave activity results in a flood elevation of 585.36 ft. msl for Make-Up Pond A and the Broad River. The Lee Nuclear Station safety-related plant elevation is 593 ft. msl and is unaffected by flood conditions and coincident wind wave activity.

McKowns Creek/Make-Up Pond B

Wind wave activity on Make-Up Pond B is evaluated coincident with the maximum water surface elevation of the PMF including the effects of dam failure, as discussed above. The determined critical fetch length of 1.39 mi. is shown in Figure 2.4.3-234. The 2-year annual extreme mile wind speed is adjusted based on the factors of fetch length, level overland or over water, critical duration, and stability. The critical duration is approximately 35 min. The adjusted wind speed is 50.33 mph.

Significant wave height (average height of the maximum one-third of waves) is estimated to be 2.00 ft., crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 3.35 ft., crest to trough. The corresponding wave period is 2.1 sec.

The slopes approaching the units are not constant. The slopes above the PMF elevation are steep up to elevation 588 ft., then level out to a flat area. To represent a conservative approach, runup is calculated assuming the runup slope continues above elevation 588 ft. A conservative estimate of 25 percent is determined for the runup slope based on finished grade contours. The maximum runup, including wave setup, is estimated to be 3.97 ft. The maximum wind setup is estimated to be 0.07 ft. Therefore, the total wind wave activity is estimated to be 4.04 ft. The PMF and the coincident wind wave activity results in a flood elevation of 589.10 ft. msl. The Lee Nuclear Station safety-related plant elevation is 593 ft. msl and is unaffected by flood conditions and coincident wind wave activity.

London Creek/Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, such that a postulated failure of the Make-Up Pond C dam would release water to the Broad River prior to reaching the Lee Nuclear Station. Failure of the Make-Up Pond C dam coincident with the PMF for the Make-Up Pond C watershed is discussed in Subsection 2.4.4.1. Flooding effects as a result of dam failure due to wind wave activity are bounded by that discussion.

Other Smaller Upstream Dams

Numerous other ponds and small lakes with dam structures are located in the Ninety-Nine Islands watershed. However, these numerous features have negligible storage capacity. A breach would have no measurable effect on the water surface elevations determined by the PMF analysis.

2.4.5 Probable Maximum Surge and Seiche Flooding

Regulatory guidance prescribed by Regulatory Guide 1.59 describes the probable maximum surge and seiche flooding based on a probable maximum hurricane (PMH), probable maximum windstorm, or moving squall line. The region of occurrence for a PMH is along U.S. coastline areas (Reference 202). The probable maximum windstorm region of occurrence is along coastline areas and large bodies of water such as the Great Lakes. A moving squall line is considered for the Great Lakes region.

The U.S. Army Corps of Engineers guideline procedures for geologic hazard evaluations consider seiche waves greater than 7 ft. to be rare (Reference 281). According to U.S. Army Corps of Engineers guidance, the seiche hazard can be screened out for sites located more than 7 ft. above the adjacent water body.

Regulatory guidance prescribed by Regulatory Guide 1.59 indicates consideration of a PMH for areas within 200 miles of coastal areas. The Lee Nuclear Station is located approximately 175 miles inland from the Atlantic Coast. The safety-related plant elevation is 593 ft. The normal maximum water surface elevation of the Broad River is 511.1 ft., the spillway flashboard elevation at Ninety-Nine Islands Dam (Reference 217).

The Broad River is a tributary of the Santee River which flows to the Atlantic Ocean. The mouth of the Santee River is about 45 mi. northeast of Charleston, South Carolina. The Santee River is also diverted by a series of lakes, Lake Marion and Lake Moultrie, to the Cooper River. The Cooper River flows into the Atlantic Ocean at Charleston, South Carolina.

According to Regulatory Guide 1.59, the probable maximum surge estimate for Folly Island, located at Charleston, South Carolina, is 28.2 ft. above mean low water. The surge estimate includes wind setup of 17.15 ft., pressure setup of 3.23 ft., initial water level of 1.0 ft., and 10 percent exceedance high tide of 6.80 ft. Mean sea level is 2.7 ft. higher than mean low water at Charleston, South. Carolina (Reference 202). The maximum surge estimate is 25.5 ft. above mean sea level. A sea level anomaly of 1.0 ft. has been known to occur for the predicted astronomical tides at Charleston, South Carolina (Reference 202). Therefore, the probable maximum surge estimate is 26.5 ft. above mean sea level.

Regulatory Guide 1.59 only contains surge data up to 1975. The maximum storm surge along the Atlantic Coast after 1975 occurred as a result of hurricane Hugo. Storm surge from hurricane Hugo inundated the South Carolina coast from Charleston to Myrtle Beach in 1989. Maximum storm tides of 20 ft. were observed. Although the site is within 200 miles of the coastline, surge due to a PMH event would not cause flooding at the site. Transposition of the probable maximum surge, without any type of reduction for distance or instream structures, is nearly three times less than the 81.9-ft. difference in elevation between the station and the adjacent river.

There are no known documented surge or seiche occurrences on the Broad River near the Lee Nuclear Station. Seismically induced seiche are discussed in Subsection 2.4.3. Based on data provided above, and site location and elevation characteristics, the station's safety-related facilities are not considered at risk from surge and seiche flooding. Resonance wave phenomena including oscillations of waves at natural periodicity, lake reflection, and harbor resonance are traditionally characteristics of harbors, estuaries, and large lakes and not associated with river settings. Any effects on the Broad River produced by similar phenomena would not affect the Lee Nuclear site. Coincident wind-generated wave activity is discussed in Subsection 2.4.3.6. Additionally, there are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during storm surge or seiching.

Surge flooding is evaluated for Make-Up Pond A and Make-Up Pond B using the maximum wind speed identified in Subsection 2.3.1.2.8. This is consistent with the maximum wind speeds identified in U.S. Army Corps of Engineers guidance (Reference 295). Fetch lengths are estimated using the longest straight line fetch directed toward the site for each water body. Wave height, setup, and runup are estimated using U.S. Army Corps of Engineers guidance (Reference 295). Wind setup is estimated using additional U.S. Army Corps of Engineers guidance (Reference 269).

Estimates for surge flooding are made coincident with 100-yr. flood levels of Make-Up Pond A and Make-Up Pond B. Resulting 100-yr. runoff rates for the watersheds are determined using USGS regression equations for small watersheds in South Carolina (Reference 296). The overflow rating curves for the respective ponds, discussed in Subsection 2.4.3.3, are used to determine the resulting coincident water surface elevations.

Make-Up Pond A

Make-Up Pond A surge flooding is evaluated coincident with the 100-yr. water surface elevation of 556.08 ft. The critical fetch length is 0.39 mi. as shown in Figure 2.4.5-201. The wind speed is adjusted based on the factors of fetch length, level overland or over water, critical duration, and stability using U.S. Army Corps of Engineers guidance (Reference 295). The critical duration is 11 min. The adjusted wind speed is 92.7 mph.

Significant wave height (average height of the maximum 33-1/3 percent of waves) is estimated to be 2.30 ft., crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 3.84 ft., crest to trough. The corresponding wave period is 1.8 sec.

The slopes along the banks of Make-Up Pond A adjacent to the site area are approximately 42 percent at most and are used to determine the wave setup and runup. The maximum runup, including wave setup, is estimated to be 5.48 ft. The maximum wind setup is estimated to be 0.12 ft. Therefore, the total water surface elevation increase due to high speed wind wave activity is estimated to be 5.60 ft. The resulting flood elevation is 561.68 ft. The Lee Nuclear Station

safety-related plant elevation is 593 ft. and is unaffected by high speed wind wave activity flooding conditions.

Make-Up Pond B

Make-Up Pond B surge flooding is evaluated coincident with the 100-yr. water surface elevation of 576.18 ft. The critical fetch length is 1.38 mi. as shown in Figure 2.4.5-202. The wind speed is adjusted based on the factors of fetch length, level overland or over water, critical duration, and stability using U.S. Army Corps of Engineers guidance (Reference 295). The critical duration is 28 min. The adjusted wind speed is 89.9 mph.

Significant wave height (average height of the maximum 33-1/3 percent of waves) is estimated to be 4.10 ft., crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 6.86 ft., crest to trough. The corresponding wave period is 2.7 sec.

The slopes along the banks of Make-Up Pond B adjacent to the site area are approximately 25 percent and are used to determine the wave setup and runup. The maximum runup, including wave setup, is estimated to be 7.48 ft. The maximum wind setup is estimated to be 0.28 ft. Therefore, the total water surface elevation increase due to high speed wind wave activity is estimated to be 7.76 ft. The resulting flood elevation is 583.94 ft. The Lee Nuclear Station safety-related plant elevation is 593 ft. and is unaffected by high speed wind wave flooding conditions.

Seiche evaluation is based on the natural fundamental period for Make-Up Pond A and Make-Up Pond B. The natural fundamental period of both water bodies is determined using Merian's formula (Reference 295).

$$T = 2 * L / (g * h)^{0.5}$$

where;

T = natural oscillation period at the fundamental mode (sec.)

L = fetch length (ft.)

g = gravitational acceleration (ft/sec^2)

h = depth of water (ft.)

Based on bathymetry mapping, an average depth of 20.10 ft. is determined for Make-Up Pond A and used as the depth of water. The resulting natural fundamental period is 2.7 min. The Make-Up Pond B average depth is 28.59 ft. The resulting natural fundamental period is 8.0 min. The wave periods determined above (1.8 sec. and 2.7 sec.) are much shorter than the natural fundamental period for both water bodies (2.7 min. and 8.0 min.). Furthermore, natural fundamental periods are significantly shorter than meteorologically induced wave periods (e.g., synoptic storm pattern frequency and dramatic reversals in steady wind direction necessary for wind setup). Since the natural periods of Make-Up Pond A and Make-Up Pond B are significantly different than the period of the excitations, they are not susceptible to meteorologically induced seiche waves. Seismically induced waves are discussed in Subsection 2.4.6.

Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, such that a postulated failure of the Make-Up Pond C dam would release water to the Broad River prior to reaching the Lee Nuclear Station. Failure of the Make-Up Pond C dam coincident with the PMF for the Make-Up Pond C watershed is discussed in Subsection 2.4.4.1. Flooding effects as a result of dam failure due to surge and seiche are bounded by that discussion.

2.4.6 Probable Maximum Tsunami

Tsunamis affecting the Atlantic Coast have not been extensively studied due to the lack of significant trigger mechanisms. No specific tsunami hazard maps are available for the East Coast of the United States. The U.S. Army Corps of Engineers has developed a general tsunami risk map (Figure 2.4.6-201) (Reference 281). The East Coast is located in Zone 1, which corresponds to a wave height of 5 ft.

According to the National Oceanic & Atmospheric Administration (NOAA) tsunami database (Reference 228), the maximum recorded tsunami wave height along the East Coast is about 20 ft. This was recorded at Daytona Beach, Florida, on July 3, 1992. The database notes that the wave was probably meteorologically induced.

The Lee Nuclear Station is located approximately 175 mi. inland from the Atlantic Coast. The safety-related plant elevation is 593 ft. Based on data provided above, and site location and elevation characteristics, the station’s safety-related facilities are not considered at risk from tsunami flooding.

Significant landslide generated waves triggered by hill slope failure are not plausible for the on-site Ponds A and B. No irregular weathering conditions or natural landslide hazards are noted in field investigations, as discussed in Subsection 2.5.1.1. There is no documented evidence that landslides of sufficient magnitude (e.g., size and velocity) at the site or adjacent to the ponds would occur. Potential slope failures that could occur would be of limited size and characterized as shallow soil or fill 'popouts'. Landslides of this type are considered minor, contain an insufficient volume of material, and are of low velocity so that potential landslide-induced waves would be insignificant.

Slopes surrounding Make-up Ponds A and B are either natural slopes that have existed for a long period of time (through most or all of the Holocene; natural slopes), or cut and fill slopes developed as part of the Cherokee Nuclear Station construction in the early 1980's. These slopes exhibit acceptable stability without visual evidence of groundwater seepage, past failure, incipient movement, or major creep, as discussed in Subsection 2.5.5.1.

Seismic induced waves resulting from surface fault rupture in the site vicinity are also not plausible. As discussed in Subsection 2.5.3, there are no capable tectonic sources within the Lee Nuclear Site vicinity (25 mi. radius), and there is negligible potential for tectonic fault rupture at the site and within the site vicinity. The only identified occurrence of a seismic induced seiche on the Broad River was measured approximately 64 miles downstream of the Lee Nuclear Station. A 0.08 ft. seiche was induced by the Alaska earthquake of 1964. Any seismic event that could occur would generate potential waves that would be insignificant compared to the available freeboard of the on-site make-up ponds or the Broad River.

As shown in Figure 2.4.1-209, Make-Up Pond A and Make-Up Pond B have normal pool elevations of 547 ft. msl and 570 ft. msl, respectively. Safety-related facilities are located at an elevation of 593 ft. Therefore, Make-Up Pond A has an available freeboard of 46 ft. and Make-Up Pond B has an available freeboard 23 ft. The geology and seismology and geotechnical engineering characteristics of the Lee Nuclear Station are presented in Section 2.5.

Make-Up Pond C

The Make-Up Pond C reservoir is located on a tributary of the Broad River, west of the Lee Nuclear Station, as shown in Figure 2.4.1-213, such that a postulated failure of the Make-Up Pond C dam would release water to the Broad River prior to reaching the Lee Nuclear Station. Failure of the Make-Up Pond C dam coincident with the PMF for the Make-Up Pond C watershed is discussed in Subsection 2.4.4.1. Flooding effects as a result of dam failure due to seismic- or landslide-induced waves are bounded by that discussion.

2.4.7 Ice Effects

There are 10 U.S. Geological Survey (USGS) gauging stations, located upstream of the Lee Nuclear Site on the Broad River and its tributaries, that recorded water temperatures for different periods between 1962 and 1981 (Reference 290). Figure 2.4.2-201 identifies the location of area gauges. The lowest recorded water temperatures during winter periods range from 32°F to 48.2°F. The lowest was recorded on the Broad River near Earl, North Carolina (USGS No. 02152622), located about 14 river mi. upstream of the site. The lowest was also recorded on Buffalo Creek near Grover, North Carolina (USGS No. 02153456), located about 14 river mi. upstream of the site.

The USGS gauging station on the Broad River east of Gaffney (USGS No. 02153500), located about 5 river mi. upstream of the site at the U.S. Highway 29 bridge crossing, is most representative of water temperatures near the site. The gauge recorded water temperatures from 1969 to 1973. The lowest recorded water temperature was 41.9°F. The recordings are summarized in Table 2.4.7-201. The longest record from a gauge near the site is located about 40 river mi. downstream of the site on the Broad River near Carlisle, South Carolina (USGS No. 02156500). The gauge recorded water temperatures from 1962 to 1975. The lowest recorded water temperature near Carlisle was 38.3°F.

According to the EPA STORET database (Reference 284), four stations located on the Broad River near the site recorded water temperatures between 1959 and 2004. The lowest water temperature recorded was 35.6°F near Gaffney, South Carolina (Station B-042). This gauge is located about 8 river mi. upstream of the site. A second station also recorded a water temperature of 35.6°F (Station B-044). This station is located about 9 river mi. downstream from the site.

The North Carolina Department of Environmental and Natural Resources collected temperature data from 1995 to 2000 at nine gauging stations in North Carolina on the Broad River and its tributaries (Reference 231). Minimum temperatures vary from 33.8°F to 39.2°F. The nine gauging stations are in the vicinity of 10 USGS gauging stations discussed above. The resulting minimum temperatures are also within the range measured by USGS gauges. Historical and more recent measurements consistently indicate that Broad River water temperatures remain above freezing.

According to the U.S. Army Corps of Engineers (Reference 275), ice jams occur in 36 states, primarily in the northern tier of the United States (Figure 2.4.7-201). Neither South Carolina nor North Carolina is included in this coverage. The U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory historical ice jam database was consulted for the Broad River

(Reference 274). There are no recorded ice jams for the Broad River. A query for ice jams in South Carolina also yielded no historical occurrence of an ice jam. However, one ice jam was recorded in North Carolina on the Neuse River at Kinston from January 26 to January 29, 1940. The maximum stage of the Neuse River resulting from the ice jam was well below flood stage. Kinston is located about 220 mi. east of the site. There are no known documented ice sheet or ice ridge occurrences on the Broad River.

The Lee Nuclear Station's safety-related plant elevation is 593 ft. The normal maximum water surface elevation for the Broad River adjacent to the Lee Nuclear Station is 511.1 ft., due to operation of the Ninety-Nine Islands Dam and hydropower plant (Reference 217). The maximum water surface elevation during a probable maximum flood event is more than 40 ft. below the site (Subsection 2.4.3). The possibility of inundating the site due to an ice jam is remote.

According to the U.S. Army Corps of Engineers, frazil ice forms in supercooled, turbulent water in rivers and lakes (Reference 275). Anchor ice is defined as frazil ice attached to the river bottom, irrespective of the nature of its formation. Although the potential for freezing (i.e., frazil or anchor ice) and subsequent ice jams on the Broad River is remote, the numerous pond and lake features adjacent to the site may be susceptible to some degree of freezing. However, there are no safety-related water storage bodies. Additionally, sustained periods of subfreezing water temperatures are not characteristic of the region. The climate and operation of Ninety-Nine Islands Reservoir prevent any significant icing on the Broad River. There are no safety-related facilities that could be affected by ice-induced low flow of the Broad River or reduction in capacity of water storage facilities.

2.4.8 Cooling Water Canals and Reservoirs

There are no current or proposed safety-related cooling water canals or reservoirs required for the Lee Nuclear Station. The atmosphere provides the ultimate heat sink (UHS) with the containment vessel and passive containment cooling system (PCS) providing the heat transfer mechanism. Additional details are provided in Subsection 2.4.11.

2.4.9 Channel Diversions

There is no evidence to suggest historical diversions or realignments of the Broad River. Several shoals are located in the Broad River near the Lee Nuclear Site. However, these features are confined within the natural banks of the river. The topography does not suggest potential diversions or landslides. The streams and rivers in the region are characterized by traditional shaped valleys with no steep, unstable side slopes that could contribute to landslide cutoffs or diversions. There is no evidence of ice-induced channel diversion.

Several instream dams are located upstream and downstream of the Lee Nuclear Station (References 217 and 276). Cherokee Falls Dam was completed in 1826. Gaston Shoals Dam was completed in 1908. Both are located immediately upstream of the site and are run-of-river hydroelectric power plants. Ninety-Nine Islands Dam, completed in 1910, is located immediately downstream and is also a run-of-river hydroelectric power plant.

The greatest potential for geothermal energy exists in areas of above average heat flow, generally the result of recent volcanic activity or active tectonics. The eastern United States has below average to average geothermal heat flow and is characterized as low temperature (Reference 251). The eastern United States is relatively tectonically stable (Reference 252). No thermal anomalies in the

eastern United States are attributed to young-to-contemporary volcanic or other igneous activity (Reference 291). Therefore, channel diversion because of geothermal activity is not expected.

The atmosphere provides the UHS with the containment vessel and PCS providing the heat transfer mechanism. The UHS does not directly rely on the Broad River intake. Therefore, channel diversion cannot adversely affect safety-related structures or systems. Additional details are provided in Subsection 2.4.11. Geologic and seismic characteristics of the region are discussed in Section 2.5.

2.4.10 Flooding Protection Requirements

All safety-related facilities are located at an elevation above the maximum flood levels resulting from all types of flooding as described in Subsection 2.4.2. The critical flooding event is identified and discussed in detail in Subsection 2.4.2. Based on the design information provided above, flood protection measures and emergency procedures to address flood protection are not required.

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in Rivers and Streams

The headwaters of the Broad River and its major tributaries originate in the higher elevations of the Appalachian Mountains of North Carolina before descending into the foothills and Piedmont region of North Carolina (Reference 231). The Broad River continues its course though the gently rolling hills and narrow stream valleys of the Piedmont region in South Carolina (Reference 259). The Lee Nuclear Station is located on this section of the river, just upstream of Ninety-Nine Islands Dam.

The Upper Broad River drainage basin above the Ninety-Nine Islands Dam derives water from several smaller tributaries that contain a considerable number of dams. According to the U.S. Army Corps of Engineers National Inventory of Dams, there are approximately 132 upstream dams of which five dams have been breached (Reference 276). Therefore, the water volume available during low-flow conditions on the Broad River is a function of natural flow in contributing rivers and streams, available storage capacity of upstream reservoirs, and regulated discharge flow from upstream dams.

Dam failure could affect normal operation during low-flow conditions. Failure of Ninety-Nine Islands Dam would drain the associated reservoir. In this portion of the Broad River, flow would resemble a function of natural flow. However, there are no safety-related facilities that could be affected by low-flow or drought conditions, since the passive cooling system does not rely on the Broad River as a source of water. If necessary, the make-up ponds can be used to supplement natural flow to support continued operations for additional periods of time. Non-safety related water supply during drought is addressed in Subsection 2.4.11.5.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

There are no safety-related facilities that could be affected by low water. The site is not at risk to low water resulting from surge, seiche, or tsunami effects, due to the inland location on a run-of-river reservoir with limited storage capacity. See Subsections 2.4.5 and 2.4.6 for additional details.

Flooding due to ice jams has not been recorded at the site. It is unlikely that an ice jam would occur based on the historical water temperatures of the Broad River. Therefore, low flow due to or

exaggerated by ice effects is not expected to occur at the site. See Subsection 2.4.7 for additional details.

2.4.11.3 Historical Low Water

Low-flow conditions at the site were analyzed based on stream flow records at USGS gauging stations on the Broad River (Reference 290). Low-flow conditions typically exist during the months of July through November. The six largest reservoirs in the basin, Lake Lure, Lake Summit, Lake Adger, Kings Mountain Reservoir, Lake Whelchel, and Make-Up Pond C represent about 88 percent of the total storage capacity for the basin. Two additional dams, Cherokee Falls and Gaston Shoals, immediately upstream from the Lee Nuclear Site, possess less than 2 percent of the total storage capacity for the basin.

The Gaston Shoals Dam has affected the drainage basin upstream of the site since 1908. Ninety-Nine Islands Dam downstream of the site was completed in 1910. Cherokee Falls Dam, located upstream of the site between Gaston Shoals Dam and Ninety-Nine Islands Dam, was completed in 1826.

Gaston Shoals Dam is part of a hydropower facility owned and operated by Duke Energy. The facility is regulated by the Federal Energy Regulatory Commission (FERC). During the months of July through November, license requirements maintain a release of at least 434 cfs or the natural flow in the Broad River, whichever is less. Should natural flow in the Broad River become less than 434 cfs, the FERC license provides measures for flow to be stored and released on an hourly basis (Reference 222).

Cherokee Falls Dam is part of a hydropower facility owned and operated by the Broad River Electric Cooperative. However, the hydroelectric facility at Cherokee Falls Dam is not currently operating. The dam is essentially a run-of-river facility with spillway flow at high-flow conditions and low-level outlets to provide constant flow under low-flow conditions (Reference 204).

The Ninety-Nine Islands Dam is part of a hydropower facility owned and operated by Duke Energy. The facility is regulated by the FERC. During the months of July through November, license requirements maintain a release of at least 483 cfs or the natural flow in the Broad River, whichever is less. Should natural flow in the Broad River become less than 483 cfs, the FERC license provides measures for the reservoir to be drawn down, at most 2 ft. below the full pool elevation of 511.1 ft., depending on the time of year. Release of accumulated flow is then made on an hourly basis (Reference 223).

There is a USGS gauging station (USGS No. 02153551) located about 2 river mi. downstream from the site in the tailrace below Ninety-Nine Islands Dam. The drainage area associated with this gauge is 1550 sq. mi. This is essentially the same drainage basin for the Broad River adjacent to the site. The annual minimum daily flows for the period of record (1998 to 2006) are presented in Table 2.4.11-201. The minimum flow observed during the period of record is 138 cfs on September 14, 2002. While these data are insufficient to determine the frequency of low-flow occurrences or to determine the lowest recorded flow, they are instructive in that this flow occurred during a period of severe drought. The flow gauge is located downstream of the Ninety-Nine Islands dam and does not measure the flow passing in front of the plant intake, although it is representative of river conditions.

The USGS gauging station (USGS No. 02153500), located about 5 river mi. upstream from the site on the Broad River near Gaffney, South Carolina, has a drainage area of 1490 sq. mi. This gauge is located downstream from Gaston Shoals Dam and upstream from Cherokee Falls Dam. The annual minimum daily flows for the period of record (1938 to 1990) are presented in Table 2.4.11-202. The gauge was discontinued in 1990 by the USGS. The minimum flow observed during the period of record is 224 cfs on October 24, 1954.

Low-flow frequency analysis was performed in accordance with USGS Bulletin 17B using the Log-Pearson Type III distribution method (Reference 253, Reference 270, and Reference 287). Due to the importance of the more recent drought years, not included in the period of record for the Gaffney gauge, the Ninety-Nine Islands gauge data were combined with the Gaffney gauge data to determine low-flow frequencies. The results provide more conservative flow estimates than if only the Gaffney gauge had been used in the analysis.

Table 2.4.11-203 provides 100-yr. drought flow rates at different durations. The 30-day 100-yr. drought flow rate is 346 cfs. A 100-yr. return period is defined as a 1 percent chance the event will occur during any one year. Therefore, the 30-day 100-yr. drought flow rate has a 1 percent chance each year that the flow rate or less will occur for at least 30 consecutive days.

Historical flow data at Gaffney, South Carolina (USGS No. 02153500) indicate that 30-day 100-yr. drought flow rates or less have been achieved on 12 days from 1938 to 1990. Historical flow data from the gauging station just below Ninety-Nine Islands Dam (USGS No. 02153551) indicate that 30-day 100-yr. drought flow rates or less have been achieved 79 days from 1998 to 2002. During this time, there were 26 consecutive days with less than 30-day 100-yr. drought flow rates. Additionally, there were 54 days of 30-day 100-yr. drought flows concentrated over a 61-day period.

Since 1900, according to the South Carolina Department of Health and Environmental Control, severe droughts have occurred statewide in 1925, 1933, 1954, 1977, 1983, 1986, 1990, 1993, and 1998 (Reference 267). USGS reports indicate more recent droughts occurred from 1998 to 2002 in areas of North Carolina belonging to the headwaters of the Broad River and in South Carolina (Reference 294). Most of the drainage area for the Broad River adjacent to the site is in North Carolina. The Gaffney gauge period of record includes the 1954, 1977, 1983, 1986 and 1990 drought years, while the gauge at Ninety-Nine Islands Dam includes the more recent years. Mid-to-late 2007 weather pattens indicate a potential for Broad River flows to drop to levels characteristic of drought conditions. While 2007 data are continuing to be collected, an analysis of these data was not available in time to be included with the application. An analysis will be conducted upon termination of current drought conditions and provided to the NRC.

The normal full pool elevation of the Ninety-Nine Islands reservoir is 511.1 ft. (Reference 217). Provisions are made to draw the reservoir down by at most 2 ft. below normal full pool during periods of low flow. Due to maintenance operations, the pool has dropped below the 2 ft. drawdown limit for short periods. The following historical lows were due to maintenance. According to USGS water year reports, the historical minimum pool elevation was 508.2 ft. on February 14, 2005 (Reference 208). The water year reports have a period of record from October 1998 to the present. Additional historical data from 1964 to 1973 indicate the minimum pool elevation was about 505.7 ft. during May 1965.

The U.S. Army Corps of Engineers historical database of ice jams on the Broad River was reviewed (Reference 274). See Subsection 2.4.7 for additional discussion. Ice effects are not a concern for low water considerations, due to the climate and reservoir operations.

2.4.11.4 Future Controls

The majority of the Broad River drainage basin upstream of the site is in North Carolina. According to the North Carolina State Water Supply Plan, public supply water use in the Broad River watershed is projected to increase by about 56 percent from 2000 to 2020 (Reference 233). This includes both surface water and groundwater use. Available supply is noted as the withdrawal capacity in Table 2.4.1-209.

According to the North Carolina Local Water Supply Plans, none of the upstream surface water public supply systems require more than 80 percent of their maximum use rate before 2030. Of the five surface water public supply systems, only the Cleveland County Sanitary District indicated exceeding 80 percent of their maximum use rate before 2050. Demand is expected to increase 238 percent by 2050, for a total demand of 28.6 million gpd.

According to the Federal Register, a Notice of Intent was filed on July 11, 2006 for a Draft Environmental Impact Statement (DEIS) to be prepared for a proposed reservoir on the First Broad River in Cleveland County, North Carolina (Reference 226). The Cleveland County Sanitary District applied for a permit to construct a water supply reservoir with a surface area of approximately 2245 ac., about 1 mi. north of Lawndale, North Carolina. The DEIS is currently in preparation. Lawndale is about 26 miles north of the Lee Nuclear site.

The USGS maintained a gauge (USGS No. 02152500) about 2.5 mi. southeast of Lawndale on the First Broad River from 1940 to 1971. During this period, the average monthly flow at Lawndale represented about 11 percent of the flow in the Broad River at Gaffney. The drainage area at the Lawndale gauge is 200 sq. mi., or roughly 13 percent of the drainage area at the USGS gauge near Gaffney.

Duke Energy is planning to expand the Cliffside Steam Station by as early as 2010. The incremental additional consumptive use withdrawal from the Broad River upstream from the station is estimated to be 17 cfs. However, four of the five existing units at Cliffside will be retired. Additional intake sources not represented by the USGS stream gauges include an expansion of the Shelby, North Carolina, water system. Shelby has constructed an intake on the Broad River, and it may withdraw up to 10 million gpd on a temporary emergency basis (Reference 207).

The North Carolina General Statutes require registration for interbasin transfers of 100,000 gpd or more (Reference 249). The North Carolina Division of Water Resources does not require a transfer certificate unless the transfer is 2 million gpd or more. Total known interbasin transfers include about 1.47 million gpd out of the basin and about 0.15 million gpd into the basin (Reference 230). North Carolina also requires registration of withdrawals of 100,000 gpd or more.

State regulations for South Carolina currently require registration of withdrawals of surface water in excess of 3,000,000 gallons per month (Reference 258). This is essentially 100,000 gpd. The construction of the embayment and intake structure requires coordination with the U.S. Army Corps of Engineers. The design and placement of the embayment and intake structure are done in accordance with the appropriate federal and state regulations. There are no safety-related facilities that could be adversely affected by any increase in water use or drought conditions.

2.4.11.5 Plant Requirements

Raw water needs, including makeup to the normal heat sink cooling towers, are supplied by the intake as described in Subsection 2.4.1.1.4. The intake structure includes necessary intake screens, pumps, etc. to convey the river water to Make-Up Pond A. Use of raw water from Make-Up Pond A is described in Subsection 2.4.1.1.4. Intake screen locations consider the Broad River minimum level. There are no safety-related plant requirements provided by the Broad River.

The normal river intake flow rate for the station is approximately 35,000 gpm. The maximum expected river intake flow is approximately 60,000 gpm. Institutional restraints on water use are imposed by Federal and State agencies as discussed. Title 40 Code of Federal Regulations Part 125 Section 84 requires that for cooling water intake structures located in a freshwater river or stream, the total design intake flow must be no greater than five percent of the source water annual mean flow. Water use and annual mean flow are discussed in Subsection 2.4.1.2.5.1 and Subsection 2.4.1.2.2.2. The South Carolina Code of Laws Title 49 Chapter 23 Part 40 identifies that during a drought declaration, the use of water from a managed watershed impoundment shall not be restricted as long as minimum streamflow or flow equal to the 7Q10 is maintained, whichever is less. Make-Up Pond B and Make-Up Pond C are expected to be used to supplement flow during periods of low flow.

The 7Q10 for the Gaffney gauge was determined to be 439 cfs using the USGS recommended Log-Pearson Type III distribution. However, because the 7Q10 is less than the Ninety-Nine Islands Dam FERC license minimum flow requirement of 483 cfs for July through November (Subsection 2.4.11.3), the FERC license minimum flow was used as a constraint in evaluating operation during low flow conditions. Furthermore, FERC license minimum flow requirements are more restrictive than the 100-year drought flow rates described in Subsection 2.4.11.3 and Table 2.4.11-203. Therefore, the following low flow analysis applies to the discussion of nonsafety related water supply during a 100-year drought.

A low flow analysis was performed based on the FERC licensed 483 cfs minimum flow requirements at Ninety-Nine Islands Dam and the Lee Nuclear Station consumptive use requirements. Consumptive use is estimated to be approximately 55 cfs. When flows in the Broad River drop below 538 cfs, combined FERC licensed 483 cfs minimum flow plus 55 cfs consumptive use, makeup water to the station is supplemented by on-site water storage, Make-Up Pond B and off-site Make-Up Pond C. When flows in the Broad River drop below 483 cfs, the station relies only on Make-Up Pond B and Make-Up Pond C storage for consumptive uses of the station.

Detailed bathymetry mapping of the on-site Make-Up Pond B (Figure 2.4.1-209 Sheet 2) and Make-Up Pond A (Figure 2.4.1-209 Sheet 3) was performed in September 2006. Make-Up Pond A is designed for a normal full pond elevation of 547 ft. Based on current topography Make-Up Pond A retains a volume of 1425 ac.-ft. The usable storage is approximately 1200 ac.-ft.

Make-Up Pond B is designed for a normal full pond elevation of 570 ft. Based on current topography, Make-Up Pond B retains a volume of approximately 4000 ac.-ft. The usable storage is approximately 3200 ac.-ft.

Make-Up Pond C is designed for a normal full pond elevation of 650 ft. Based on the bathymetry shown in Figure 2.4.1-213, Make-Up Pond C retains a volume of approximately 22,000 ac.-ft. The usable off-site storage capacity is approximately 17,500 ac.-ft. The total usable storage capacity of Make-Up Pond B and Make-Up Pond C is approximately 20,700 ac.-ft. Make-Up Pond C has

sufficient capacity to support full power operation for approximately 160 days. Make-Up Pond B has sufficient capacity to support full power operations for approximately 30 days.

There are no safety-related water requirements for normal plant shutdown associated with the AP1000. Make-Up Pond A nominally provides for approximately 1200 ac.-ft. of usable water storage. Make-Up Pond A has sufficient capacity to conduct a normal plant shutdown and to maintain shutdown conditions for both units. Make-Up Pond A can be replenished with water from the Broad River, from Make-Up Pond B, and from Make-Up Pond C via Make-Up Pond B.

The circulating water system for the station is a closed-cycle type system coupled with mechanical draft, wet cooling towers. For each unit the circulating water system flow rate is estimated at 600,000 gpm (Subsection 10.4.5). Figure 10.4-201 presents the circulating water system. Make-Up Ponds B and C are used to supplement flow during periods of low flow. Emergency cooling is discussed in Subsection 2.4.11.6.

Effluent from the new facility discharges into the river at the upstream face of the Ninety-Nine Islands Dam near the intakes for the hydroelectric generating units. This configuration ensures no recirculation to the embayment area and intake screens of the new facility.

2.4.11.6 Heat Sink Dependability Requirements

The atmosphere is the UHS. A continuous natural circulation flow of air removes heat from the containment vessel. The steel containment vessel and PCS provides the heat transfer mechanism, as described in Section 6.2. A separate gravity-drained, passive containment cooling water storage tank provides containment wetting. The PCS is not reliant on the source of water from the river intake. Makeup to the passive containment cooling water storage tank is provided by demineralized water from the passive containment cooling ancillary water storage tank. Therefore, no warning of impending low flow from the river water makeup system is required. Low river water conditions would not affect the ability of the emergency cooling water systems and the UHS to provide the required cooling for emergency conditions.

The passive containment cooling water storage tank has a volume capacity for 72 hours of containment wetting. The passive containment cooling ancillary water storage tank has a volume capacity to maintain containment wetting for an additional 4 days. Makeup for long-term containment wetting can be supplied to the PCS by Make-Up Pond A or alternate external resources, through multiple system paths. Site-related events and natural phenomena would not affect the atmosphere functioning as the UHS. As described in Subsection 2.4.3, the station is capable of withstanding the PMF. Seismic design is addressed in Section 3.7.

2.4.12 Groundwater

2.4.12.1 Description and On-Site Use

2.4.12.1.1 Regional Aquifers, Formations, Sources, and Sinks

The Lee Nuclear Site is located within the Piedmont physiographic province, a southwest-northeast-oriented province of the Appalachian Mountain System (Figure 2.4.1-203). The Piedmont province is 80 - 120 mile (mi.) wide and situated between the Blue Ridge province, a mountainous region to the northwest, and the Atlantic Coastal Plain province to the southeast. The majority of rocks in the Piedmont are medium-to high-grade metamorphic rocks. These rocks are generally stratified and

compositionally layered with distinct foliation. In addition, lineaments and fault systems are common in the region, and several major thrust sheets are present in the basin. Numerous granitic plutons and stocks have intruded older metamorphic rocks, and are often marked by areas of higher topography; a result of the massive, resistant nature of these intrusive rocks. The Lee Nuclear Site is located within the Kings Mountain Belt of the Piedmont province, which contains a complex series of deformed rocks consisting of felsic and mafic schists and gneisses, quartzites, conglomerates, and marble, generally considered to be of Precambrian and early Paleozoic age (Subsection 2.5.1).

Throughout the Piedmont region, bedrock is overlain by a mantle of unconsolidated material known as regolith. The regolith includes, where present, the soil zone, a zone of weathered and decomposed bedrock known as saprolite, and alluvium. Saprolite, the product of chemical and mechanical weathering of underlying bedrock, is typically composed of clay and coarser granular material that may reflect the texture of the rock from which it was formed. Typically, the formation of soils is attributed to the in-place weathering of the underlying rock and the deposition of material transported by water and laid down as clay, silt, sand, or large rock fragments (Reference 285). Crystalline rocks are commonly weathered in the Piedmont region because of the warm, humid conditions. Iron oxide-stained kaolinite and other aluminosilicate clay minerals are the dominant constituents of upland soils in many areas. Modern fluvial sediments generally occupy only the active beds and small floodplains of local streams and rivers.

The Piedmont aquifer system is basically a two layered slope-aquifer system (Figure 2.4.1-210). The shallow water table aquifer is composed of the saprolite and residual soil, which is typically low yielding. The shallow water table aquifer is unconfined, meaning that the upper surface of the saturated zone is not effectively separated from the ground surface by a low-permeability clay layer. The underlying bedrock aquifer consists of weathered and unweathered crystalline igneous and metamorphic rocks that store and transmit water through fractures. The fracture system in the bedrock is a network of discontinuities that increase in prevalence upward through the crystalline rock as it transitions into saprolite. Because of the permeability of the transition zone, the bedrock aquifer is also considered unconfined and not effectively isolated. Thus, the saprolite and bedrock zones function as one interconnected aquifer system (Reference 266). While confined settings can occur in fracture bedrock, none were indicated in this study. The rocks typically yield small amounts of water to domestic users, small cities, and low-water-demanding industries.

Groundwater occurs almost everywhere throughout the Piedmont province; however, it is not a single, widespread aquifer. Groundwater occurs in various local aquifer systems and compartments that have similar characteristics and are hydraulically connected. Groundwater recharge in this area is derived from infiltration by local precipitation or infiltration from nearby surface water. Additionally, with the construction of the on-site impoundments, recharge also occurs from these surface waters.

The on-site impoundments consist of Make-Up Pond B, Make-Up Pond A, and Hold-Up Pond A. Details of these impoundments are discussed in Subsection 2.4.1.2.2.6. Surface water in these impoundments is in direct communication with groundwater and the water levels represent the water table.

The water table varies from ground surface elevation in valleys to more than 100 ft. below the surface on sharply rising hills. The groundwater levels in the Piedmont typically decline during the late spring and summer due to evapotranspiration and rise in the late fall and winter when the evaporation potential is reduced (Reference 297).

The fractures, relic rock textures, and directional differences in permeability or ease of groundwater movement may significantly affect the direction of local groundwater flow. Recharging of the groundwater in the Piedmont is by the addition of precipitation water, first to the shallow soil/saprolite aquifer (referred to as the water table aquifer in the regional discussion) and then to the uppermost fracture zone (transition zone). Recharge mostly occurs on upland topographic highs or at least above the slopes of stream valleys. Water does not generally move to great depths, but it is directed almost laterally by reduced permeabilities of crystalline rock with lower fracture density.

Cross-sections of the Lee Nuclear Site are presented in Figure 2.4.12-205, Sheets 1 to 4, and depict the relationship between groundwater beneath the site and the surface water bodies surrounding the site.

2.4.12.1.2 Local Aquifers, Formations, Sources, and Sinks

The Lee Nuclear Site overlies rocks of the Battleground Formation with the exception of later diabase dikes (Figures 2.5.1-218a, 2.5.1-219a, and 2.5.1-220). The Battleground Formation comprises rocks primarily felsic to intermediate in composition (dacite to andesite protoliths), volcaniclastic sequences with intrusions of similar composition (meta granodiorite to metatonalite, metadiorite, and meta gabbro), and interfingered, marine-influenced metasedimentary sequences. Petrographic examination of thin sections obtained from the Lee Nuclear Station site revealed the following rock types: Mica Schist, Meta Quartz Diorite, Meta Dacite Porphyry, and Meta Basalt (Section 2.5). Geologic maps show the distribution of rock types, which tend to have locally erratic outcrop and subsurface distribution patterns, but regionally trend northeast to southwest.

Subsurface investigations performed at the Lee Nuclear Site in 1973 for the former Cherokee Nuclear Station and in March 2006 reveal that geologic and hydrologic conditions at the Lee Nuclear Site are similar to the regional conditions described above in Subsection 2.4.12.1.1. The first occurrence of groundwater beneath the Lee Nuclear Site is within the surficial hydrogeologic unit. The groundwater flows under unconfined conditions in the surficial hydrogeologic unit, which is generally composed of three different media beneath the site: (1) fill material placed in valley lows during site grading using on-site borrow materials, (2) the soil and saprolite zone that overlies the bedrock, and (3) partially weathered rock. The shallow groundwater beneath the site is mostly affected by the excavated area and the current dewatering activities (effects of the dewatering are discussed in Subsection 2.4.12.2).

2.4.12.2 Sources

The AP1000 reactor design has no safety-related heat sink that relies on groundwater supplies. The Lee Nuclear Site is not expected to use groundwater as a source of water for any purpose. Additional information related to local and on-site groundwater use is presented in Subsection 2.4.1.2.5.2.

2.4.12.2.1 Regional and Local Groundwater Uses

Groundwater supplies in the Piedmont physiographic province of South Carolina occur in three types of hydrogeologic environments. These are the unweathered and fractured crystalline rocks, overlying saprolite and residuum, and to a lesser extent, alluvial valley-fill deposits. Most public water supply wells are completed in fractured igneous and metamorphic rocks, often referred to as “crystalline bedrock,” while some private wells are simply dug or bored into the overlying saprolite. Yields of 4 – 170 gpm have been recorded from 30 South Carolina ambient groundwater quality network wells in the Piedmont bedrock (Reference 257). Regional groundwater studies consulted during the

Cherokee Nuclear Station site investigation indicated that most domestic wells are not drilled to develop maximum yield, are generally less than 150 ft. deep, and have flow rates ranging from 3 to 150 gpm with a median flow rate of 7 gpm (Reference 214).

According to South Carolina Department of Health and Environmental Control (SCDHEC) water-use data for 2005, 1.02 million gallons (gal.) of groundwater were used for thermoelectric power generation in Cherokee County. No groundwater use in Cherokee County for domestic self-supplied systems, aquaculture, golf courses, industry, irrigation, livestock, mining, or hydroelectric power was reported in the 2005 SCDHEC data (Reference 267). According to a private well report from SCDHEC, based on data from January 1985 to June 2006, the number of domestic wells completed in Cherokee County was 1076 (Reference 261). The USGS and state water-use data were reviewed, and groundwater withdrawals for counties located in the Upper Broad River watershed are presented in Table 2.4.1-208. Groundwater withdrawals for Cherokee and surrounding counties in South Carolina (Table 2.4.1-207) account for only 4.7 million gal. per day (Mgd), and the majority (85 percent) of that volume is pumped from Spartanburg County, outside the watershed for the Lee Nuclear Site.

Local groundwater use in the vicinity of Lee Nuclear Station is predominantly from domestic wells, and is described in Subsection 2.4.1.2.5.2.

2.4.12.2.2 Historical On-Site Conditions

Site hydrologic data were gathered prior to construction activities (through the early 1970s) and during the construction activities (late 1970s to the early 1980s). Surface and groundwater conditions at the Lee Nuclear Site have changed because of the excavation and site grading conducted as part of the construction activities for the former Cherokee Nuclear Station. No significant changes to the Lee Nuclear site have occurred since those construction activities.

Prior to the construction activities for the Cherokee Nuclear Station, a subsurface investigation was conducted, and water level measurements were obtained to develop an understanding of the groundwater setting. A groundwater table elevation map was developed to represent site conditions at that time and is presented in Figure 2.4.12-201. Initial potentiometric surface data collected from July, August, and September 1973 indicated site-specific groundwater flow directions were primarily toward the north and east from the reactor area, which generally mimicked the preconstruction site topography. A north-south-trending groundwater divide was apparent west of the reactor area and east of the Nuclear Service Water Reservoir, now known as Make-Up Pond B.

According to the former Cherokee Nuclear site groundwater investigation, measured depths to groundwater beneath ridges ranged from about 40 to 80 ft. below ground surface. The groundwater table was reportedly at or near the surface in valleys and draws, as was evidenced by observed springs. Near the proposed locations of the reactor buildings, the groundwater table varied between depths of 10 – 60 ft. below ground surface with potentiometric surface elevations ranging from 570 to around 605 ft. above msl (Reference 220).

Construction activities for the Cherokee Nuclear Station began in the late 1970s, resulting in significant alterations to on-site topography. Because of the relationship between topography and depth to water, changes to the potentiometric surface were monitored with a network of observation wells across the site. A review of historical data identified groundwater levels in observation wells prior to and during the construction. Based on water level data, construction dewatering from the site excavation was indicated around January 1977. Between November 1977 and March 1978,

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approximately 5.74 million gal. of water were reportedly pumped from the water table aquifer through dewatering wells over the 5-month period. These wells were pumped at average rates ranging from 38 to 65 gpm with well depths from 200 to 280 ft. below ground surface. The effect of construction dewatering was assessed on the basis of historical groundwater measurements collected across the site and in the nearest residential well during construction dewatering activities. The apparent drawdown in the observation wells, caused by the cumulative dewatering activities, is shown on Figure 2.4.12-202. The dewatering activities did not affect observation wells outside the area shown. In addition, the nearest residential well, the [Mullinax]^{SRI} well, was not affected by construction dewatering activities (References 215 and 218). The [Mullinax]^{SRI} well is completed in the Piedmont Aquifer and is located approximately 5000 ft. south of the center of the excavation on the north side of McKowns Mountain Road. Several wells located adjacent to the excavation, around the site, and at a nearby residence (the [Mullinax]^{SRI} well) were gauged on a monthly basis between 1976 and 1985, providing limited-term historical water level data. Only observation wells nearest the excavation, as shown in Figure 2.4.12-202, appeared to be affected by the Cherokee site dewatering activities.

2.4.12.2.3 On-Site Conditions in 2006 to 2007 and Projected Post-Construction On-Site Conditions

In March 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for the Lee Nuclear Site. The dewatering of the existing excavation preceded the subsurface investigation, thus returning the site to hydrogeologic conditions similar to those of the previous construction phase. Approximately 740 million gal. of water were removed from the excavation from December 19, 2005, through September 7, 2006. Following the initial dewatering, an apparent 5-foot thick interval of staining was observed on the existing Cherokee concrete structures, the top of which was surveyed at an elevation of 578.72 ft. msl. The staining observed between elevations 574 and 579 ft. msl is indicative of the range that water level fluctuated in the open excavation since termination of Cherokee era construction activities. A comparison of the apparent water levels in this impoundment, as shown on the February 1994 and February 2005 aerial photographs, with the topographic survey conducted in 2006 indicated a similar range of water levels in the excavation area (574 ft. msl in 1994 to 579 ft. msl in 2005). Precipitation data for the period preceding these observations indicated near normal conditions, confirming the aerial images captured typical impoundment water levels. Ongoing maintenance dewatering activities are expected to end following construction activities.

As part of the 2006-2007 groundwater investigation, fifteen borings were drilled into the crystalline bedrock, and monitoring wells were installed in partially weathered rock intervals. In July 2006, nine additional monitoring wells were installed to evaluate shallow groundwater conditions across the site. Details regarding well construction are presented in Table 2.4.12-201.

Following well development, water levels were measured monthly from April 2006 to April 2007 (Table 2.4.12-202) to characterize seasonal trends in groundwater levels and to identify flow pathways surrounding the Lee Nuclear Site. The hydrograph for this groundwater data is presented on Figure 2.4.12-203. Surface waters at four locations were also gauged as part of the monitoring program. These locations included Make-Up Pond B, a water retention impoundment below Make-Up Pond B, Make-Up Pond A, and Hold-Up Pond A. Based on this year of study, groundwater levels were observed to fluctuate, with the highest groundwater elevations observed between January and April 2007 and the lowest groundwater elevations between September and November

2006. This trend correlates with both the river flow and rainfall patterns and confirms that both groundwater levels and river flow are governed by local precipitation (Section 2.3).

Potentiometric surface maps developed from water level data showed that during the 2006 construction dewatering and site investigation, groundwater was drawn toward the excavation (Figure 2.4.12-204, Sheets 1 - 7). During the dewatering activities, continuous decline of water levels in areas downgradient of the excavation was observed, as recharge entering the power block area from the south was intercepted by the excavation, pumped and discharged to Make-Up Pond B. Following the completion of construction dewatering, the potentiometric surface beneath the reactor buildings is expected to rebound to equilibrium conditions.

Under natural conditions the topography of the water table within the Piedmont mimics the topography of the land surface, but has less relief. Cross-sections of the Lee Nuclear Site are presented in Figure 2.4.12-205, Sheets 1 - 4. These figures depict the relationship between groundwater beneath the site and the surface water bodies surrounding the site. Groundwater flow in the Piedmont province is typically restricted to the topographic area underlying the slope that extends from a divide to an adjacent stream.

Both regionally and locally, surface topography plays a dominant role in groundwater occurrence. Post-Cherokee plant construction topography was observed to affect groundwater conditions such that cuts in topography induce a lowered water table and fill induces a raised water table. Field evidence for this is based on comparison between the Cherokee water table map (Figure 2.4.12-201) and the maps developed from the Lee Nuclear Site investigation (Figure 2.4.12-204, Sheets 1-7). For example, MW-1204, located on the Unit 2 Cooling Tower Pad, is where construction fill was placed during Cherokee construction, resulting in a significantly higher land surface elevation (approximately 610 ft. msl compared to its pre-grading elevation of around 560 ft. msl). Consequently, the water table elevation is higher in MW-1204: groundwater elevation of approximately 570 ft. msl compared with the former groundwater elevation of less than 550 ft. msl. Another example includes MW-1200, located west-northwest of Unit 1, where construction cuts resulted in a significantly lower land surface elevation (approximately 590 ft. msl compared to its pre-grading elevation of approximately 670 ft. msl). Consequently, the water table elevation has lowered (groundwater elevation of 565 ft. msl compared with the former groundwater elevation of more than 585 ft. msl).

Following construction of the Lee Nuclear Station and return to equilibrium conditions, the water table is expected to mimic land surface elevation contours, consistent with slope-aquifer conditions of the Piedmont physiographic province. The potentiometric surface elevation near Lee Units 1 and 2 is expected to rebound between 574 and 579 ft. msl, consistent with concrete stain observations discussed previously. Allowing for moderate frequency short-term fluctuations in water table level above this range that may not be evident in concrete stain observations, groundwater level near Lee Units 1 and 2 may occur between 574 ft. and 584 ft. msl.

The projected post-dewatering water table conditions following the construction of the Lee Nuclear Station are illustrated in Figure 2.4.12-204, Sheet 8. The potentiometric conditions shown in Figure 2.4.12-204, Sheet 8 affect the directions of groundwater flow surrounding the Lee Nuclear Station. Each of the ponds serves as a constant head flow boundary. Ultimately, groundwater flow discharges to the Broad River, which is the groundwater sink for the site and the surrounding area.

Based on site observations, a network of storm drains and buried piping was partially installed during the Cherokee project to manage surface water runoff. While no as-built drawings for the existing storm drain system for the former Cherokee Nuclear Station exist, a review of stormwater plans was

conducted to assess the drain system's potential effect on groundwater movement. Storm drains located more than 500 ft. upgradient (south) of the power block could potentially intercept the water table and allow shallow groundwater movement towards Make-Up Pond A; these drains do not affect groundwater movement in the power block area. Other storm drains appear to be above the water table and would not affect the movement of groundwater. One exception is a storm drain originally designed to transfer stormwater from the Cherokee power block area to Hold-Up Pond A. The depth of this storm drain pipe appears to be below the projected water table. Therefore, if left in place, this conduit could potentially cause a preferential groundwater pathway from the power block area downgradient to Hold-Up Pond A once groundwater recovers from the construction dewatering activities. The existing storm drain and bedding materials will be removed by overexcavation. The remaining void will then be plugged with low-permeability backfill material, and compacted to density sufficient to assure no short-circuiting can occur.

Stormwater controls at the Lee Nuclear Station include a combination of surface grading to facilitate surface water flow, construction of a storm drain system (DRS), and construction of a roof drain and collection system. The Lee Nuclear Station DRS is designed to facilitate and control the runoff of precipitation along surface water flow paths, diverting surface runoff away from the power block area and reducing the potential for flooding. The site grading and drainage plan is shown in Figure 2.4.2-202. As discussed in Subsection 2.4.2.3, portions of the site are relatively flat; however, the site is graded such that overall runoff will drain away from safety-related structures to Make-Up Pond B, Make-Up Pond A, or directly to the Broad River. Precipitation falling on buildings is captured by a roof drain and collection system, channeled through drainage downspouts, and directed to the DRS. The DRS is not expected to directly affect groundwater flow system of the limiting groundwater flow pathway.

2.4.12.2.3.1 Maximum Post-Construction Groundwater Analysis

An analysis of maximum post-construction groundwater elevation in the area of the Units 1 and 2 power block areas was performed. The analysis utilized MODFLOW numerical method model (Reference 306). The following summarizes the analysis approach.

- The analysis considered planned post-construction surface cover treatment, as illustrated in Figure 2.4.12-209.

- The model domain covered a 3,000 ft. by 3,000 ft. area that includes both Unit 1 and Unit 2 power block areas and extends to include much of the area encompassed by the vehicle barrier system. However, no credit was taken in this analysis for vehicle barrier system drainage capacity. MODFLOW observation points were defined and located to provide estimated groundwater elevations over the duration of the simulation run. The model domain and location of observation points, relative to the power block areas, are shown in Figure 2.4.12-210. Figure 2.4.12-211 provides a hydrograph of groundwater elevations at each observation point over the duration of the modeled storm event.

- The model reflects the fill, soil/saprolite, and PWR uppermost aquifer unit of the Lee site. Placement of granular fill and general fill was also included in the model construction. Hydraulic conductivity and specific yield values were derived from site investigations and expected properties of granular fill materials to be used during plant construction.

- Starting groundwater elevations for the analysis were based on hydraulic heads from the projected potentiometric surface map in Figure 2.4.12-204, Sheet 8.

- Precipitation input was developed from the 1995 Tropical Storm Jerry which exhibited the maximum monthly precipitation and maximum 24-hr precipitation at the regional Greenville/ Spartanburg station near Greer, South Carolina. This storm is considered the most severe historically recorded precipitation event for the site and surrounding area. The storm duration, based on gage data from the Greer station is presented in Table 2.4.12-205. To maximize saturation of soils and associated groundwater mounding, the storm event definition included an antecedent storm (40% of the Table 2.4.12-205 distribution values), a 72-hr dry-out period, and followed by the full 100% precipitation, using the Table 2.4.12-205 distribution.

- Infiltration is assumed to occur instantaneously (with no time lag as water travels through the vadose zone). Infiltration occurs at a constant rate determined by the runoff coefficient of the surface material and does not consider a decrease in actual soil absorption capacity during the precipitation event.

- Modeled surface runoff from impervious surfaces is considered as additional water directed onto grass covered areas. This additional water is added to precipitation that falls directly on the down-slope grass surface.

The analysis concluded that the maximum post-construction groundwater elevation remained below 584 ft. msl; therefore, satisfying the site parameter for maximum groundwater elevation of less than 591 ft. msl (Table 2.0-201).

2.4.12.2.4 Aquifer Characteristics

2.4.12.2.4.1 Porosity

Site-specific subsurface materials in the area surrounding the power block include fill, residual soil, saprolite, and partially weathered rock (PWR). Based on the results of the geotechnical investigation, representative engineering properties of the soils were determined according to methods described in Subsection 2.5.4.2. Characterization of porosity and effective porosity were made using the data provided in Table 2.5.4-211.

Fill materials are located in former drainageways, which were built up to existing elevations during Cherokee construction. Based on the specific gravity (particle density, 2.71 grams per cubic centimeter, g/cc) and dry unit weight (101 pounds per cubic foot, pcf) provided for fill material, a mean total porosity of 40 percent was determined. The effective porosity is assumed to be equivalent to specific yield, and was estimated using grain size distribution described within Water Supply Paper 1662-D (Reference 299). This technique indicates effective porosity was estimated to be 9 percent. Fill materials have been cut from other areas of the site, and they are typically comprised of undifferentiated materials (residual soils, saprolite, and/or PWR) similar to native materials.

The residual soils have undergone relatively complete weathering and lack the relict features found in the saprolite zone. Saprolite is the isovolumetrically weathered zone which does not reflect the characteristics of surficial soil development processes, but does reflect some of the physical properties of the underlying parent rock from which it was formed. According to the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), surficial soils in the vicinity of the power block area consisted predominantly of Tatum silty clay loam and Tatum very fine sandy loam with variable slope and erosion (Figure 2.4.12-206). Tatum soils are well-drained (not seasonally saturated) and are typically derived from sericite schist, phyllite, and/or other related metamorphic rocks of the Piedmont. Tatum soils are typically composed of a surficial 0 - 8 in. silty

clay loam or very fine sandy loam (CL, CL-ML, ML). These soil horizons grade subsoils of clay, silty clay, and/or silty clay loam (CH, MH). Clay content in the subsoil stratum of Tatum soils ranges from 12 to 60 percent. Tatum soils transition at depths of 45-60 inches to saprolite materials reflecting the characteristics of the underlying parent rock. The saturated hydraulic conductivity of Tatum soils is reported by the NRCS to be moderately permeable: 4 to 14 micrometers per second (mm/s) (4 to 14 x 10^{-4} centimeters per second [cm/s]). Tatum soils are not prone to flooding and exhibit erosion factors (Kf) that range from 0.32 to 0.43. The soils are highly corrosive to both concrete and steel (Reference 278). Based on geotechnical analyses of both the residual soil and saprolite, a mean total porosity of 45 percent was estimated for these materials. The effective porosity was estimated to be approximately 20 percent. The native soils in the immediate area of the power block were essentially completely removed or mixed with deeper saprolite materials to become site fill materials during Cherokee-era activities. Regardless, knowledge of the natural properties of these surface soil materials is useful in understanding characteristics of site soils, and conditions in the undisturbed portions of the site.

PWR is a transitional weathering zone between the saprolite and the hard, competent, underlying bedrock. The PWR materials are similar to the overlying saprolite zone, but include more fragments of less weathered and less porous rock. PWR was conservatively estimated to have an effective porosity of 8 percent. This value is based on the free drainage (specific yield) represented by the difference between saturated unit weight (140 pcf) and the wet unit weight (135 pcf). The total porosity of PWR, based on saturated unit weight, is estimated to be 27 percent.

2.4.12.2.4.2 Permeability

The permeability of a material is a measure of its ability to transmit water. Generally within the Piedmont region, the soil/saprolite zone has a low permeability. Also, fractures within the competent bedrock become sparse and poorly connected at increasing depths, thus limiting crystalline bedrock permeability. Fracture permeability consistently occurs in the transition zone, including the uppermost part of bedrock; therefore, this zone often exhibits the highest consistent permeability.

During the Cherokee investigation in the 1970’s, 135 field and laboratory tests were conducted to characterize soil and rock permeability. Fifty-five packer tests were conducted in soil and rock intervals in 17 soil borings across the site. An additional 42 field and 38 laboratory tests were performed to evaluate soil permeability. The recent investigation supplements the above investigation with the performance of an additional 11 packer tests in bedrock materials, 16 slug-out tests across the site, and one multi-well aquifer pump test performed within the limiting groundwater flow path (i.e., the flow path with the shortest time-of-travel) from the nuclear island area toward the Broad River to the north.

Based on results from the Cherokee investigation, packer tests, multiwell pumping tests, geotechnical laboratory analyses, and field tests (combined with the results of the 2006 slug tests, packer tests, and multiwell pumping tests), the following conclusions are made regarding aquifer permeability at the Lee Nuclear Site, noting that maintenance dewatering is ongoing and may have affected the recent aquifer test results:

- Reported vertical soil hydraulic conductivities (Kv) of soil and saprolite ranges from 2.45 x 10^{-8} cm/s to a maximum value of 2.55 x 10^{-4} cm/s with a median of 2.10 x 10^{-6} cm/s. For samples exceeding the median hydraulic conductivity of the data set, the geometric mean (4.4 x 10^{-5} cm/s) represents a conservative vertical hydraulic conductivity value for the residuum. For the purpose of permeability analysis, a conservative value is one that

increases the rate of water movement. Vertical hydraulic conductivity generally increases with depth.

- Reported horizontal hydraulic conductivities (Kh) of soil and saprolite ranges from 9.67×10^{-7} cm/s (i.e., the lower limit of the test range) to a maximum value of 2.26×10^{-3} cm/s with a median of 1.14×10^{-4} cm/s. For samples exceeding the median hydraulic conductivity of the data set, the geometric mean (4.5×10^{-4} cm/s) represents a conservative hydraulic conductivity value for the residuum.

- Reported hydraulic conductivities measured in the partially weathered rock (PWR), or transition zone, range from approximately 9.67×10^{-7} cm/s to a maximum value of 9.89×10^{-3} cm/s with a median of 1.53×10^{-4} cm/s. For samples exceeding the median hydraulic conductivity of the data set, the geometric mean (1.0×10^{-3} cm/s) represents a conservative hydraulic conductivity value for the PWR transition zone across the site. Based on its thorough review of the properties of the PWR zone, Duke asserts that a value of 1.4×10^{-3} cm/s is a scientifically-sound, conservative, and representative hydraulic conductivity value for PWR materials at the Lee site. This is the value obtained from an aquifer test in 2006 for an area believed to best represent the limiting groundwater flow path, and is used as the representative value of hydraulic conductivity for PWR. Figure 2.4.12-207 includes three PWR samples that were subsequently excavated in the area of the reactors.

- Values of hydraulic conductivity reported in the Cherokee-era studies represent the upper 100 ft. of the saturated interval. This undifferentiated aquifer zone is comprised of residual soil, saprolite, and partially weathered rock. The resultant hydraulic conductivity values range from 2.21×10^{-4} cm/s to 3.90×10^{-3} cm/s. These results are consistent with and support the recent findings of the Lee-era site investigation. These more recent studies determined the hydraulic conductivity of PWR, the most hydraulically conductive aquifer material, to be 1.4×10^{-3} cm/s.

- Fill materials placed in former valleys during site grading are currently groundwater aquifer materials in some areas. Slug tests conducted in 2006 and 2007 characterized these materials to have hydraulic conductivities ranging from 1.81×10^{-5} cm/s to 7.44×10^{-5} cm/s. The median hydraulic conductivity for the fill material is 5.39×10^{-5} cm/s. For samples equal to and greater than the median hydraulic conductivity of the data set, the geometric mean (7.0×10^{-5} cm/s) represents a conservative hydraulic conductivity value for the fill materials.

A summary of the various test results is presented in Table 2.4.12-204. Figure 2.4.12-207 depicts the distribution of hydraulic conductivities with depth. This figure shows the wide variability of hydraulic conductivities observed across the site during both the Cherokee and Lee site investigations. Hydraulic conductivities generally decrease with depth as partially weathered rock transitions to continuous rock. Figure 2.4.12-207 includes the results for partially weathered rock samples that were subsequently removed during excavation for the Cherokee Nuclear Station reactor buildings.

2.4.12.3 Groundwater Movement

2.4.12.3.1 Groundwater Pathways

The nature and depth of groundwater circulation in the Piedmont is predictably variable. This variability is a function of the singular aquifer system being comprised of weathered saprolite, partially weathered rock, and fractured bedrock, and the degree of interconnection of pores and

fractures between these materials. Typical of the Piedmont, groundwater flow is from topographic positions (recharge areas) to the regional drainage features (discharge areas). Groundwater flow at this site likewise generally mirrors the surface topography, with strong gradients and flow paths from the power block area, northward to the Broad River.

The projected groundwater movement in the vicinity of the Lee Nuclear Station power block was assessed to evaluate contaminant migration for the postulated release scenario (Subsection 2.4.13). For the release scenario, radwaste contaminant sources include the Units 1 and 2 radwaste storage tanks, located below plant grade at elevation 559.5 ft. msl. This elevation is 32.5 ft. below plant grade. For the assessment of alternative pathways, four locations were assumed to be plausible points of exposure (i.e., locations at which groundwater would be discharged to the surface to allow human contact or to facilitate transport). The pathways evaluated are:

- Pathway 1: Unit 2 to Hold-Up Pond A
- Pathway 2: Unit 2 to the Broad River
- Pathway 3: Unit 2 to Make-Up Pond A
- Pathway 4: Unit 1 to Make-Up Pond B

The impacts of construction and operation of Make-Pond Up C within the London Creek watershed were evaluated and determined not to affect groundwater conditions beyond Little London Creek drainage way. Consequently, Make-Up Pond C does not affect the groundwater flow regime at the Lee Nuclear Station, including the evaluation of hydrostatic loading (Subsection 2.4.12.5) or analyses of accidental releases of radioactive liquid effluents (Subsection 2.4.13).

2.4.12.3.2 Groundwater Velocity

The rate of flow (i.e., the velocity) of groundwater depends on (1) the permeability and effective porosity of the medium through which it is moving and (2) the hydraulic gradient. Average interstitial groundwater flow velocity within the water table aquifer was determined using a form of the Darcy equation as follows:

$$V = K\,(dh/dl)/n_e$$

Where: V = average groundwater velocity (ft/yr)

K = hydraulic conductivity (cm/s converted to ft/yr)

dh/dl = groundwater gradient (ft/ft)

n_e = effective porosity (%)

After construction dewatering and the return to static conditions, the potentiometric surface in the area of the reactor buildings is expected to rebound to a maximum elevation of approximately 584 ft. msl. These conditions reflect the maximum anticipated groundwater level during operations.

Travel distances for contaminants from postulated release points at the reactors to downgradient receptors were estimated from site information for each of four possible flow paths. Although the

aquifer is comprised principally of saprolite and PWR, the more conservative PWR values for hydraulic conductivity and effective porosity were used in the analysis of groundwater velocities. Estimated travel times for the four groundwater flow paths are as follows:

- Pathway 1: Groundwater travels from Unit 2 to Hold-Up Pond A in approximately 1.6 years.
- Pathway 2: From Unit 2 to the Broad River in approximately 2.6 years.
- Pathway 3: From Unit 2 to Make-Up Pond A in approximately 4.0 years.
- Pathway 4: From Unit 1 to Make-Up Pond B in approximately 5.5 years.

These flow paths are represented on Figure 2.4.12-208. This analysis indicates the limiting flow path for the evaluated postulated release to be from the Unit 2 radwaste storage tank to Hold-Up Pond A (Pathway 1, Figure 2.4.12-205, Sheet 3).

Soil distribution characteristics for radiological isotopes (i.e., Co-60, Cs-137, Fe-55, I-129, Ni-63, Pu-242, Tc-99, U-235) were determined from soil and water samples collected along the preferred groundwater flow path. This data is presented in Subsection 2.4.13 to assist in the development of calculations for fate and transport analyses in the event of accidental releases of effluents to groundwater.

2.4.12.3.3 Effects of Local Area Pumping

While the groundwater is not intended to be used at the Lee Nuclear Site, consideration is given to the movement of groundwater beneath the site in response to potential pumping associated with dewatering or domestic well use. Based on permeability characteristics beneath the site and an understanding of typical wells in the vicinity, a radius of influence can be estimated. For unconfined aquifers, such as those encountered in the Piedmont province, the radius of influence can be determined using the following equation provided by the Departments of the Army, the Navy, and the Air Force in Publication TM5-818-5:

$$R = 3\Delta H \, (K \times 10^4)^{1/2}$$

Where: R = the radius of influence of a pumping well (ft.)

ΔH = the drawdown within the well (ft.)

K = the hydraulic conductivity of the aquifer (cm/s)

Most domestic wells in the vicinity of the Lee Nuclear Site are completed as either shallow bored wells, or deeper drilled wells. Shallow bored wells are usually completed in the saprolite zone, typically no deeper than 75 ft. Deeper drilled wells are installed in the PWR and fractured bedrock zones. Both types of wells generally have yields of 5-10 gpm, or less. Using these conditions provides a conservative estimate of the potential reach of a typical domestic well producing at full capacity. Assuming the hydraulic conductivities are consistent with partially weathered rock, as listed in Table 2.4.12-204, the radius of influence is approximately 1700 ft. (0.32 mi.) from these wells. The lateral area of influence of the dewatered excavation is approximately 500 ft. (0.095 mi.).

Based on site reconnaissance of the area, the closest domestic water supply well is located approximately 5000 ft. (0.95 mi.) south of the nuclear island. The influence of the surrounding impoundments (i.e., Make-Up Pond B and Make-Up Pond A) would further buffer the potential draw created from off-site pumping or on-site pumping, if needed. No off-site wells are considered capable of reversing groundwater flow beneath the site, or vice versa, based on the geographic positions of these wells (i.e., the distance of the domestic wells) and the character of these wells (i.e., the typical low-flow rates and the relatively shallow completion depths).

The Cherokee Nuclear Station Construction Permit ER identified 50 domestic water wells and provided construction details for these wells, including well diameter, well depth, and depth to water (see Table 2.4.1-212 and Figure 2.4.1-212). Only three of these 50 wells have total depths of 150 ft. or greater. Since 1985, 19 wells have been installed within a 1-mi. radius of the Lee Nuclear Site property boundary and to a depth greater than 150 ft. (Reference 261). However, according to information provided by the Draytonville Water District, public water supply lines were installed in the late 1990s and continue to be added in the area surrounding the Lee Nuclear Site. As of 2007, since public water supply lines were installed in the area, approximately 55 percent of residents within a 2-mi. radius of the reactor buildings have converted from self-supplied groundwater systems to public water supplies. Furthermore, with the addition of water-supply lines planned for completion in 2009, the public water is expected to be available to approximately 83 percent of those residents within the 2-mi. radius of the plant. The projected use of self-supplied groundwater systems is expected to continue to decline as public water supply lines are built into rural areas and residents increase their dependence on the public water supply.

2.4.12.4 Monitoring or Safeguard Requirements

There are two potential sources for radiological impacts to groundwater: (1) leaks from radioactive waste tanks and (2) leaks from the spent fuel pool. To minimize the potential for contact of radioactive material with groundwater, the Lee Nuclear Site is equipped with a water barrier around the building foundation up to 1 ft. above grade. The water barrier is installed to prevent water from seeping into the auxiliary building that holds the liquid radioactive waste tanks. In addition, groundwater monitoring will be conducted at the Lee Nuclear Site. The groundwater monitoring program will be consistent with the guidance in "Generic FSAR Template Guidance for Life Cycle Minimization of Contamination" (NEI 08-08). The groundwater monitoring program will include a network of wells for early detection (near-field wells) and for verification of no off-site migration (far-field wells). Wells will be installed in proximity to plant systems that may be a source of radiological releases, and/or in nearby plausible down-gradient flow direction from such sources. Both shallow and deep wells will be utilized as needed to monitor the location closest to the potential release area. The laboratory analyses of groundwater samples will include gamma isotopes and tritium.

The groundwater monitoring program is described in Subsection 12AA.5.4.14. Accident effects are discussed in Subsection 2.4.13. Additionally, analysis of the relationship of the Lee Nuclear Site groundwater to seismicity and the potential for related soil liquefaction and the potential for undermining of safety-related structures is discussed in Section 2.5.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

The AP1000 design maximum groundwater elevation is 2 ft. below plant elevation. The Lee Nuclear Station plant elevation is 593 ft. above msl and the yard grade is 592 ft. above msl; therefore, the design maximum groundwater elevation for the Lee Site is 591 ft. above msl. A maximum groundwater elevation, considering the most severe historically recorded natural phenomena for the

Lee site is estimated to be approximately 584 ft. msl, as discussed in Subsection 2.4.12.2.3.1. The hydrostatic loading is not expected to exceed design criteria. An unsaturated zone of at least 8 ft. below plant grade elevation will be maintained during operations. The installation and operation of a permanent dewatering system is not a facility design requirement.

2.4.13 Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters

2.4.13.1 Groundwater

This section provides a conservative analysis of a postulated accidental liquid effluent release to the environment at the Lee Nuclear Site. The following sections describe the scenario and conceptual model used to evaluate the transport pathways to the nearest potable water supply in an unrestricted area. RESRAD-OFFSITE Version 2.0 is used to model the transport and provide resulting radionuclide concentration values in the potable water receptor body.

Acceptable results are those that are less than the effluent concentrations listed in 10 CFR 20 Appendix B, Table 2, Column 2. Individual radionuclide concentration results and the sum of fractions value are compared against these limits. The sum of fractions (i.e., unity value) is a comparison of the ratio of known radionuclides to their limit. This unity value may not exceed "1". As applied through Branch Technical Position 11-6, these criteria apply to the nearest potable water supply in an unrestricted area.

Historical and projected groundwater flow paths were evaluated in Subsection 2.4.12 to characterize groundwater movement from the nuclear island area to a point of exposure. Groundwater at the Lee Site exists as a single, undifferentiated aquifer, comprised of soil, saprolite, partially weathered rock (PWR), competent bedrock, and, to a limited extent, fill soils. Although the projected groundwater flow paths travel through zones with saprolite, fill, and PWR, the more conservative hydrogeologic characteristics of PWR were used in both the determination of the limiting groundwater flow path and as inputs, where appropriate, into the RESRAD-OFFSITE model. Using the PWR characteristics for hydraulic conductivity, bulk density, and effective porosity, the flow path from the Unit 2 effluent hold-up tank to Hold-Up Pond A is assumed to be the limiting pathway of radionuclide migration, with the shortest (i.e., most rapid) travel time to a surface water body. For purposes of this analysis, because the spillway and dam of Hold-Up Pond A are proximal to the Broad River, entry concentrations at Hold-Up Pond A are assumed to be entry concentrations at the Broad River. This direct conveyance to the Broad River thus provides for no additional retardation, hold-up, or restrictions to transport between Hold-Up Pond A and the Broad River. Figures 2.4.12-204, Sheet 8 and 2.4.12-205, Sheet 3 depict subsurface conditions that control the movement of groundwater beneath the Lee Nuclear Station.

While groundwater functions as the transport media for fugitive radionuclides, interaction of individual radionuclides with the soil matrix can potentially delay their movement. The solid/liquid distribution coefficient, K_d, is, by definition, an equilibrium constant that describes the process wherein a species (e.g., a radionuclide) is partitioned between a solid phase (soil, by adsorption or precipitation) and a liquid phase (groundwater, by dissolution). Soil properties affecting the distribution coefficient include the texture of soils (sand, loam, clay, or organic soils), the organic matter content of the soils, pH values, the soil solution ratio, the solution or pore water concentration, and the presence of competing cations and complexing agents. Because of its dependence on many soil properties, the value of the distribution coefficient for a specific radionuclide in soils can range over several orders of magnitude under different conditions. The measurement of distribution coefficients of radionuclides

within the limiting groundwater pathway allows further characterization of the rate of movement of fugitive radionuclides in groundwater.

Soil and groundwater samples were collected from Monitoring Wells MW-1208 and MW-1210 located on the north and south sides of the nuclear island (Figure 2.4.12-205, Sheet 1). Three soil samples were collected from the saturated zone at depths ranging from 45 to 73 ft. below ground level. The samples were submitted for laboratory analysis of soil distribution characteristics for specific radiological isotopes (Co-60, Cs-137, Fe-55, I-129, Ni-63, Pu-242, Sr-90, Tc-99, U-235). Results of these analyses are presented in Table 2.4.13-201, along with default K_d values found in literature, for comparison. For conservatism, those radionuclides which had been evaluated for site-specific distribution coefficients used the lowest measured Kd values in the evaluation, regardless of the media from which the samples were collected. The values are adjusted to the low limit of their reporting range (e.g., for a reported Cs-137 value of 1156±163 cm3/g, a value of 993 cm3/g was used in the analysis). All other radionuclides use the most conservative Kd value of 0.

2.4.13.2 Accident Scenario

The limiting postulated failure of a Unit 2 effluent holdup tank, located in the Unit 2 auxiliary building, is analyzed to estimate the resulting concentration of radioactive contaminants entering Hold-Up Pond A via groundwater flow. Contaminant concentrations at this point are then assumed to represent entry concentrations to the surface water receptor, the Broad River, which is located proximal to Hold-Up Pond A.

The event is defined as an unexpected and uncontrolled release of radioactive water produced by plant operations from a tank rupture. The AP1000 tanks which normally contain radioactive liquid are listed in Table 2.4.13-202. The contents from the effluent holdup tank are conservatively assumed to enter the environment instantaneously, allowing radionuclides to be transported in the direction of groundwater flow. The flow path from Unit 2 to Hold-Up Pond A is determined to be the limiting pathway based on travel time.

It is noted that no outdoor tanks contain radioactivity. In particular, the AP1000 does not require boron changes for load follow and so does not recycle boric acid or water; therefore, the boric acid tank is not radioactive.

The spent resin tanks are excluded from consideration, because most of their activity is bound to the spent resins; they have minimal free water that would be capable of migrating from the tank in the event of a tank failure. Tanks inside the containment building were not considered because the containment building, a seismic Category I structure, is a freestanding cylindrical steel containment vessel (Subsection 1.2.4.1). Credit is taken for the steel liner to mitigate the effect of a postulated tank failure.

The Liquid Radwaste System (WLS) monitor tanks located in the radwaste building extension are considered because of their location in a non-seismic building. These tanks have a maximum capacity of 15,000 gallons each. They receive fluid that has been processed and must be monitored prior to discharge. The radwaste building has a well sealed, contiguous basemat with integral curbing that can hold the maximum liquid inventory of any tank. Floor drains in the area lead to the liquid radwaste system. The foundation for the entire building is a reinforced concrete mat on grade. Liquid spilled due to failure of any one of these tanks would be contained within the building, and would involve low activity liquids being held for discharge. Any release to the environment would be leakage

through cracks in the concrete. The radiological consequences of such leakage are bounded by the analysis for the effluent holdup tanks. Therefore, these monitor tanks are not the limiting fault.

The remaining four tank applications were considered - the effluent holdup tanks, waste holdup tanks, monitor tanks (located in the auxiliary building), and chemical waste tank. Of these tanks, the effluent holdup tanks have both the highest potential radioactive isotope concentrations and the largest volume. The effluent holdup tanks are also located on the lowest level of the auxiliary building, which is a limiting location relative to an uncontrolled release from the auxiliary building via the groundwater pathway. Therefore, an effluent holdup tank is limiting for the purpose of calculating the effects of the failure of a radioactive liquid-containing tank.

The effluent holdup tanks are located in an unlined room on the lowest level of the auxiliary building. This level is 32 feet 6 inches below the existing surface grade elevation of the plant. Each unit has two effluent holdup tanks, one of which is postulated to fail.

The analysis considers the tank liquid level, decay of the tank contents, potential paths of spilled liquid to the environment, and other pertinent factors.

The total volume of each effluent holdup tank is 28,000 gallons. Since credit can not be taken for liquid retention by unlined building foundations; a conservative analysis assumes that the tank content (80 percent of capacity, or 22,400 gallons) is immediately released through cracks in the auxiliary building walls and floor into the surrounding sub-surface soil. These assumptions follow the position in Branch Technical Position 11-6, March 2007.

2.4.13.3 Source Term

The radioactive source term is:

- Tritium source term concentration is 1.0 microcuries per gram taken from Table 11.1-8;
- Corrosion product source terms Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58, and Co-60 taken from Table 11.1-2;
- Other isotope source terms taken from Table 11.1-2 multiplied by 0.12/0.25 to adjust the radionuclide concentrations to the required 0.12 percent failed fuel fraction outlined in Branch Technical Position 11-6, March, 2007; and
- Gaseous state nuclides and nuclides with short half-lives not included in the RESRAD default library are removed from consideration as they have no impact on the evaluation. These radionuclides include:

Ba-137m	Br-83	Br-85	I-131
I-133	Kr-83m	Kr-85	Kr-85m
Kr-87	Kr-88	Kr-89	Rh-106
Te-131	Te-131m	Xe-131m	Xe-133
Xe-133m	Xe-135	Xe-137	Xe-138

Analysis of failure of the effluent holdup tank of Unit 2 rather than Unit 1 is conservative in that the pathway from the Unit 2 effluent holdup tank to Hold-Up Pond A has the shortest (i.e., most rapid) travel duration, assuming conservative PWR characteristics along the entire flow path.

The impacts of construction and operation of Make-Up Pond C within the London Creek watershed were evaluated and determined not to affect groundwater conditions beyond Little London Creek drainage way. Consequently, Make-Up Pond C does not affect the groundwater flow regime at the Lee Nuclear Station and therefore has no impact on the transport paths and accidental release analyses discussed in this subsection.

As discussed in Subsection 2.4.12, dewatering activities are currently occurring at the site. After construction is complete, dewatering activities will end.

The conceptual model of radionuclide transport through groundwater, from Unit 2 to Hold-Up Pond A, is shown in Figure 2.4.12-205 (Sheet 3). As stated in Subsection 2.4.13.1, a direct conveyance between Hold-Up Pond A and the Broad River is assumed. With the failure of the effluent holdup tank and subsequent liquid release to the environment, radionuclides enter the subgrade soils at an elevation of 32 feet 6 inches below the surrounding grade. The contaminated zone is, therefore, a volume of contaminated soil for which the effective porosity is saturated with contaminated water released from the liquid effluent holdup tank. The contaminated zone soil is assumed to exhibit PWR characteristics. Because RESRAD-OFFSITE considers soil at the source of the contamination, the liquid initial source term concentrations were converted to an equivalent concentration on a soil mass basis.

Currently, the overburden soils continually receive the average annual onsite precipitation. In general, the precipitation that does not run off or is not lost to evapotranspiration infiltrates the overlying unsaturated zone and contributes to groundwater as recharge. However, as an additional conservative measure in the model, runoff was assumed to be zero, and precipitation not lost to evapotranspiration was treated by RESRAD-OFFSITE as recharge.

2.4.13.4 Conceptual Model

The conceptual model assumes that one of the liquid effluent tanks, located at the lowest level of the auxiliary building, ruptures while containing 80 percent of its total capacity. The liquid is assumed to be released in accordance with Branch Technical Position 11-6 of NUREG-0800. The liquid from the ruptured tank would flood the tank room and proceed to the auxiliary building radiologically controlled area sump by way of the floor drains. The sump pumps are assumed to be inoperable to create a bounding case. The liquid then enters the environment outside the auxiliary building. The consequence is a release of 22,400 gallons of contaminated liquid into the soil. The liquid is transported via groundwater flow to the surface water receptor, the Broad River. Because Hold-Up Pond A is the surface water body with the shortest (i.e., most rapid) groundwater transport time, assuming PWR characteristics, the model calculates radionuclide concentrations in a hypothetical well at the edge of this pond. The dam and spillway of Hold-Up Pond A are proximal to the Broad River. This model then assumes that concentrations in Hold-Up Pond A are immediately conveyed to the Broad River, without any additional intermediate retardation, hold up, or transport restrictions between Hold-Up Pond A and the Broad River. The conceptual model then assumes the liquid is diluted in the Broad River reservoir upstream of the Ninety-Nine Islands Dam. This is conservative because the nearest potable water supply using the Broad River surface water is located approximately 21 miles downstream from the postulated release point, at the City of Union public water supply. Concentrations are modeled for an evaluation period of 1,000 years.

The conceptual model is conservative because it provides for the shortest (i.e., most rapid) travel time to a surface water body, even though that surface water body is not the receptor body, and it also includes faulting the limiting tank. The analysis uses conservative estimates for parameters that

are not developed from site-specific data. In addition, site-specific inputs to the model are also conservative, including the use of the lowest Kd values and the assumption that all groundwater pathways traveled through geo-media with the porosity and conductivity properties of PWR. Values used as inputs in the model are shown in Table 2.4.13-203. The straight-line flow path is used, which is also conservative as actual groundwater pathways are more tortuous, have longer transport times, and lower hydraulic conductivities for the fractures and joints.

Radionuclide concentrations in the hypothetical well at the edge of Hold-Up Pond A and in the Broad River at the Ninety-Nine Islands Dam are modeled using RESRAD-OFFSITE (Reference 212). The model considers the effects of different transport rates for radionuclides and progeny nuclides, while allowing radioactive decay during the transport process. The concentration of each radionuclide transmitted to the Broad River is determined by the transport through the groundwater system, dilution by groundwater and infiltrating surface water from the overburden soils, adsorption, and radioactive decay.

Radionuclide decay during transport by groundwater occurs and is considered in the analysis. Radionuclide transport by groundwater is assumed to be affected by adsorption by the surrounding soils. As discussed in Subsection 2.4.12, the soils surrounding the auxiliary building at the elevation of the liquid release are modeled as having the porosity and hydraulic conductivity characteristics of PWR.

The saturated zone dispersion values are set to mimic infusion, rather than injection, of the contaminated liquid into the groundwater flow by assigning a value to the longitudinal dispersivity equal to one-hundredth of the length of the transport distance (contaminated zone). The horizontal dispersivity is one tenth of the longitudinal dispersivity and the vertical dispersivity is one hundredth of the longitudinal dispersivity distance. Table 2.4.13-203 indicates the values used in the analysis for these parameters. These settings allow the contamination to move with the natural groundwater flow rather than be pushed through the groundwater and arrive over a longer time frame in a more dilute state.

2.4.13.5 Sensitive Parameters

Sensitivity analyses were performed on a number of input parameters to evaluate the sensitivity of the RESRAD-OFFSITE model to a range of values for specific input factors. A parameter is considered sensitive if the resulting effect on the evaluated radionuclide concentration varied by more than 10 percent. Input parameters evaluated in the sensitivity analyses include:

- Hydraulic gradient of the saturated zone (varied by a factor of 2);
- Well pump intake depth (varied by a factor of 2);
- Volume of the surface water receptor (varied by a factor of 2); and
- Kd values in the saturated zone for site-specific (non-zero) radionuclides (varied by a factor of 10).

Overall, the sensitivity analyses indicate that variations in the single parameters analyzed have no significant impact on the resulting concentrations; in no case do the resulting concentrations exceed 10 CFR 20 Appendix B, Table 2, Column 2 limits or a sum of fractions calculation. Of particular note:

- When the surface water volume is reduced by a factor of 2, concentrations doubled, but the sum of fractions remained in the E-05 range. This expected outcome confirmed that even with a significant reduction in available volume, the sum of fractions remained below the unity value of one.

- Even with a relatively high hydraulic gradient (0.06 ft/ft considered not plausible for this site), increases in radionuclide concentrations varied by less than 10 percent, and the sum of fractions remained below 10 CFR 20 Appendix B, Table 2, Column 2 limits and unity standard.

2.4.13.6 Regulatory Compliance

10 CFR 20 Appendix B states, "The columns in Table 2 of this appendix captioned "Effluents," "Air," and "Water," are applicable to the assessment and control of dose to the public, particularly in the implementation of the provisions of §20.1302. The concentration values given in Columns 1 and 2 of Table 2 are equivalent to the radionuclide concentrations which, if inhaled or ingested continuously over the course of a year, would produce a total effective dose equivalent of 0.05 rem (50 millirem or 0.5 millisieverts)." Thus, meeting the concentration limits of 10 CFR 20 Appendix B, Table 2 Column 2 results in a dose of less than 0.05 rem and therefore demonstrates that the requirements of 10 CFR 20.1301 and 10 CFR 20.1302 are met.

The radiological consequence of a postulated failure of the Unit 2 effluent holdup tank as the limiting fault is evaluated and determined not to exceed 10 CFR 20 Appendix B, Table 2, Column 2 limits at the nearest waters adjoining the Lee site (Broad River). The analysis demonstrates that radionuclide concentrations in both the hypothetical well located at the edge of Hold-Up Pond A and in the Broad River at the Ninety-Nine Islands Dam are below 10 CFR 20 Appendix B, Table 2, Column 2 limits. Further, the nearest potable water supply located in an unrestricted area using the Broad River surface water is the City of Union public water supply located approximately 21 miles downstream of the Ninety-Nine Islands Dam.

The maximum radionuclide concentration for each isotope sum of fractions of 10 CFR 20 Appendix B, Table 2, Column 2 limits calculated for both the hypothetical well at the edge of Hold-Up Pond A and in the receptor body, the Broad River, during the 1,000-year period, is below a value of 1. Table 2.4.13-204 provides the fraction of effluent concentration for the significant radionuclide.

2.4.14 Technical Specifications and Emergency Operation Requirements

The maximum flood level at the Lee Nuclear Station is established as the maximum of calculated results from flooding events analyzed in Section 2.4. That maximum flood level is elevation 592.56 ft. msl. This elevation would result from a PMP event on the Lee Nuclear Station site (local intense precipitation) as described in Subsection 2.4.2.3. The Lee Nuclear Station safety-related structures have a plant elevation of 593 ft. msl. This maximum flood level is identified as a site characteristic in Table 2.0-201. Also, Subsection 2.4.12.5 describes plant elevation relative to the maximum anticipated groundwater level. The hydrostatic loading is not expected to exceed design criteria.

There are no safety-related facilities that could be affected by low-flow or drought conditions of the Broad River. At low flow conditions, water is drawn from Make-Up Ponds B and C (Subsection 2.4.11.5). Full power plant operations could be sustained for approximately 190 days with water from Make-Up Ponds B and C, with sufficient water remaining in Make-Up Pond A to shutdown the plant and maintain safe shutdown conditions.

Based on site-specific conditions of the Lee Nuclear Station, there are no emergency protective measures designed to minimize the impact of adverse hydrology-related events on safety-related facilities.

2.4.15 Combined License Information

2.4.15.1 Hydrological Description

Major hydrologic features on or in the vicinity of the site are addressed in Subsection 2.4.1.

2.4.15.2 Floods

Site-specific information on historical flooding and potential flooding factors is addressed in Subsections 2.4.2, 2.4.3, 2.4.4, 2.4.5, 2.4.6, 2.4.7, and 2.4.10.

2.4.15.3 Cooling Water Supply

The water supply sources to provide makeup water to the service water system cooling tower are addressed in Subsections 2.4.8, 2.4.9, and 2.4.11.5.

2.4.15.4 Groundwater

Site-specific information on groundwater is addressed in Subsections 2.4.12.1, 2.4.12.2, 2.4.12.3, and 2.4.12.5.

2.4.15.5 Accidental Release of Liquid Effluents into Ground and Surface Water

Site-specific information on the ability of the ground and surface water to disperse, dilute, or concentrate accidental releases of liquid effluents and the effects of these releases on existing and known future use of surface water resources are addressed in Subsections 2.4.12.2.3, 2.4.12.2.4, 2.4.12.3, 2.4.12.4, and 2.4.13.

2.4.15.6 Emergency Operation Requirement

Flood protection emergency procedures required to meet the site parameter for flood level are addressed in Subsection 2.4.14.

2.4.16 References

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Table 2.4.1-201 (Sheet 1 of 2)
Site Features and Elevations

Site Feature	Elevation (ft. msl)
Nuclear Island	593
Railcar Bay/Filter Storage Area door	593
Bottom of Basemat (Units 1 and 2)	553.5
Annex Building	593
Temporary Electric Power Supply Room door	593
Door to SO3 Stairs	593
Door to SO4 Stairs	593
Men’s Change Room door	593
Corridor 40321 door	593
Corridor door 40311	593
Access Area 40300 doors	593
Containment Access Corridor Hatch and Door	600.1
Diesel Generator Building	593
Diesel Generator Room A doors	593
Diesel Generator Room B doors	593
Combustion Air Cleaner Area A plenum	593
Combustion Air Cleaner Area B plenum	593
Radwaste Building	593
Mobile Systems Facility doors	593
HVAC Equipment Room door	593
Electrical/Mechanical Equipment Room door	593
Turbine Building	593
Mobile Systems Facility doors	593
Door to SO2 Stairs	593
Aux Boiler Room door	593
Motor Driven Fire Pump Room door	593
Door to SO1 Stairs	593
Turbine Building Grade Deck Room 20300	593

Table 2.4.1-201 (Sheet 2 of 2)
Site Features and Elevations

Site Feature	Elevation (ft. msl)
Other Features	
Heavy Haul Road	590
Raw Water Intake Pumping Station (base)	497.3
Raw Water Intake Pumping Station (entry)	508
Heavy Lift Derrick - Crane	589.5
Low Level Waste Storage Area	588
Wastewater Treatment Area	588
Ninety-Nine Islands Dam Crest	511
Broad River above Ninety-Nine Islands Dam	511
Broad River below Ninety-Nine Islands Dam	440
Make-Up Pond A	547
Make-Up Pond B	570
Hold-Up Pond A	536
Make-Up Pond C	650
Cooling Tower	588

ft. - feet

msl - mean sea level

Source: Section 1.2.

Table 2.4.1-202 (Sheet 1 of 2)
Description of Upper Broad River Watersheds

Watershed Name	Basin	Subbasin	Drainage Area (sq. mi.)	Drainage Area Above Ninety-Nine Islands Dam (sq. mi.)
Upper Broad River Basin (03050105) of North Carolina				
Upper Broad River and Lake Lure	03050105	030801	184	184
Second Broad River and tributaries	03050105	030802	513	513
Green River	03050105	030803	137	137
First Broad River	03050105	030804	426	426
Buffalo Creek	03050105	030805	181	163
North Palocet	03050105	030806	73	0
Upper Broad River Basin (03050105) of South Carolina				
Broad River	03050105	050	26	26
Broad River	03050105	090	129	65
Buffalo Creek	03050105	100	16	16
Cherokee Creek	03050105	110	23	23
Kings Creek	03050105	120	52	0
Thicketty Creek	03050105	130	157	0
Bullock Creek	03050105	140	118	0
North Pacolet River	03050105	150	49	0
South Pacolet River	03050105	160	91	0

Table 2.4.1-202 (Sheet 2 of 2)
Description of Upper Broad River Watersheds

Watershed Name	Basin	Subbasin	Drainage Area (sq. mi.)	Drainage Area Above Ninety-Nine Islands Dam (sq. mi.)
Pacolet River	03050105	170	115	0
Lawsons Fork Creek	03050105	180	85	0
Pacolet River	03050105	190	102	0
Totals			2477	1553

Source (SC): Reference 268

Source (NC): Reference 230

sq. mi. - square miles

Table 2.4.1-203 (Sheet 1 of 2)
USGS Gauging Stations on the Broad River

Station Name	Station Number	Location	Drainage Area (sq. mi.)	2005 Water Year Annual Mean Flow (cfs)
Broad River near Boiling Springs, NC	02151500	Lat. 35°12’39”, Long. 81°41’51”, on right bank half mi. upstream from Sandy Creek, 3 mi. downstream from Second Broad River, and 3½ mi. SW of Boiling Springs, Cleveland County.	864	NIA
Broad River near Blacksburg, SC	02153200	Lat 35°07'26", Long 81°35'17", at upstream side of bridge on SC Highway 18, 1.2 mi upstream of Buffalo Creek, 1.2 mi downstream of Gaston Shoals Reservoir, 3.2 mi west of Blacksburg, and at mile 275.2.	1290	1802
Broad River near Gaffney, SC	02153500	Water-stage recorder, Lat. 35°05’20”, Long. 81°34’20”, at a bridge on US Hwy. 29, 0.3 mi. upstream from Cherokee Creek, 4.4 mi. downstream from Gaston Shoals Dam, and 4.5 mi. ENE of Gaffney, Cherokee County.	1490	NIA
Broad River below Cherokee Falls, SC	02153551	Water-stage recorder, Lat. 35°01’52”, Long. 81°29’34”, at left bank of tailrace below Ninety-Nine Islands Reservoir, 3.1 mi. downstream of Cherokee Falls, and 0.3 mi. upstream of Kings Creek.	1550	2532

Table 2.4.1-203 (Sheet 2 of 2)
USGS Gauging Stations on the Broad River

Station Name	Station Number	Location	Drainage Area (sq. mi.)	2005 Water Year Annual Mean Flow (cfs)
Broad River near Carlisle, SC	02156500	Water-stage recorder, Lat. 34°35’46”, Long. 81°25’20”, on right bank at downstream side of bridge on State Highway 72, 1.3 mi upstream from Sandy River, 2.0 mi downstream from Seaboard Coast Line Railroad bridge, 2.5 mi east of Carlisle, 5.0 mi downstream from Neal Shoals Dam, and at mile 226.0., Union County.	2790	3892

Source: References 214, 290, and 293.

See Figure 2.4.1-205

NIA = No Information Available

mi. - miles

sq. mi. - square miles

Table 2.4.1-204 (Sheet 1 of 2)
Broad River Monthly Discharge and Temperature Variability

DISCHARGE VARIABILITY

	Monthly Mean Stream Flow Recorded in Cubic Feet Per Second (cfs)											
Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1998											1098	1253
1999	2021	2040	1812	1851	1422	964	796	517	538	925	1137	1338
2000	1619	1840	2142	1997	1301	713	591	518	678	669	1129	890
2001	865	985	1727	1318	793	801	1020	589	764	574	630	843
2002	1336	1139	1473	1104	835	560	377	242	505	865	1592	3312
2003	1441	2747	6686	8733	7433	5608	5051	4983	1838	1619	2094	2727
2004	1744	3100	1637	2104	1439	2626	1503	1219	8764	2219	3541	4710
2005	2615	2229	3930	3162	1926	2489	5418	1998	1356	2658	997	2031
2006	2659	1773	1516	1382	1100	1394	982	1254	2054	1245	1828	2143
Mean of Monthly Discharge	1852	2102	2779	2935	2202	2085	2194	1583	2285	1493	1655	2323
Max:	2659	3100	6686	8733	7433	5608	5418	4983	8764	2658	3541	4710
Min:	865	985	1473	1104	793	560	377	242	393	574	630	843

Notes:

Average annual flow: Approximately 2500 cfs (1926-2008)
Maximum monthly flow: 8764 cfs (1926-2006)
Minimum monthly flow: 242 cfs (1998-2006)
cfs - cubic feet per second
Source:
USGS 02153551 Broad River Below Ninety Nine Islands Reservoir, SC
(1998 to 2006)

Cherokee County, South Carolina
Hydrologic Unit Code 03050105
Latitude 35°01'52", Longitude 81°29'34" NAD27
Drainage area 1550 square miles
Gauge datum 412.20 feet above sea level NGVD29
Missing data - No information available from USGS

Maximum and Minimum Monthly Average Flows
1998 - 2006

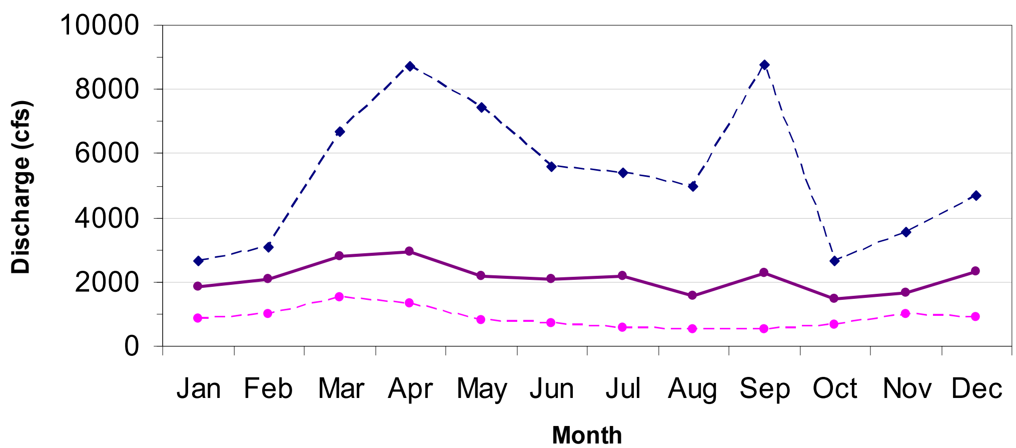


Table 2.4.1-204 (Sheet 2 of 2)
Broad River Monthly Discharge and Temperature Variability

TEMPERATURE VARIABILITY

	Monthly Mean Water Temperature (deg. C)											
YEAR	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1996										16.9	11.3	7.55
1997	7.89	9.30	14.2	15.8	19.5	22.5	27.2	26.6	23.6	18.0	9.77	6.60
1998	7.40	8.77	11.3					27.2	25.4	19.1	13.4	9.81
1999	7.29	9.38	11.1	18.6	21.3	25.3	28.3	29.1	24.3	18.1	13.3	8.42
2000	6.87	8.33	14.0	16.5	23.7			27.9	23.6	18.4	11.9	
2001	4.92	9.86	11.7	18.3	23.3	26.4	27.0	28.3	23.6	17.3	12.7	10.6
2002	6.07	9.57	12.8	20.9	22.8	28.1	29.6	28.3	25.5	20.0		
2003		8.02	13.1	15.5	19.6	23.5	25.9	25.5		18.1	14.8	7.37
2004	6.83	6.83	13.4	17.5	24.4	26.0		26.4			14.0	7.54
2005	8.05	8.33	11.1	16.6	21.0				25.7	19.6	12.4	6.67
2006	8.42	8.51	13.0	19.8	22.2			28.5	24.3			
Mean of Monthly Temp.	7.10	8.70	12.6	17.7	22.0	25.4	27.7	27.5	24.5	18.4	12.6	8.10
Max:	8.4	9.9	14.2	20.9	24.4	28.1	29.6	29.1	25.7	20.0	14.8	10.6
Min:	4.9	6.8	11.1	15.5	19.5	22.5	25.9	25.5	23.6	16.9	9.8	6.6

Notes:

Average monthly temperature: 17.7°C
Average monthly maximum temperature: 19.6°C
Average monthly minimum temperature: 15.7°C
Maximum monthly temperature: 29.6°C
Minimum monthly temperature: 4.9°C
Degree Celsius - °C
Source:
USGS 02156500 Broad River Near Carlisle, SC (1996 to 2006)
No incomplete Data is used for Statistical Calculation

Union County, South Carolina
Hydrologic Unit Code 03050106
Latitude 34°35'46", Longitude 81°25'20" NAD27
Drainage area 2790 square miles
Gauge datum 290.79 feet above sea level NGVD29
Missing data - No information available

Maximum and Minimum Monthly Temperatures
1996 - 2006

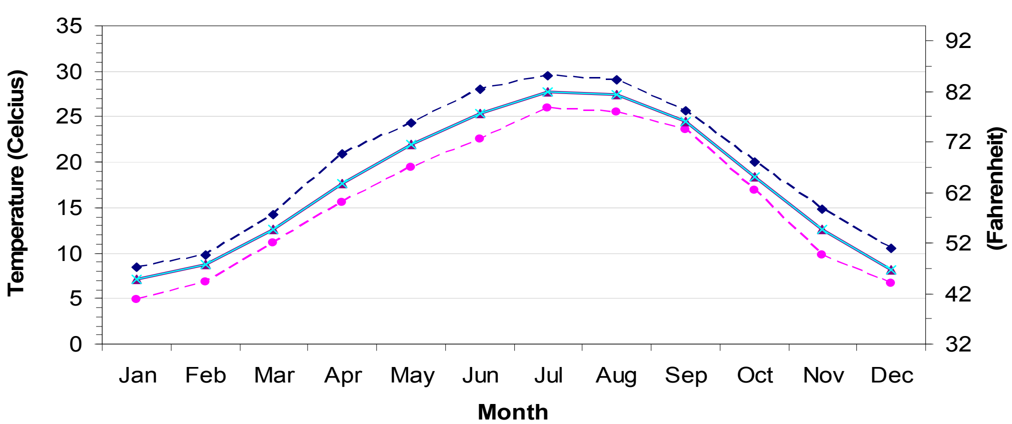


Table 2.4.1-205
Major Reservoirs Located in the Upper Broad River Basin

Name	In Service Date	Owner	Type	Drainage Area (sq. mi.)	Water Surface Area (ac.)	Dam Height (ft.)	Dam Length (ft.)	Spillway Width (ft.)	Normal Storage (ac.-ft.)	Flood Storage[a] (ac.-ft.)	Normal Pool Elevation (ft. msl)
Ninety-Nine Islands	1910	DPC	CNPG	1550	433	62	1568	891[2]	2300	2300	511[2]
King Mountain Reservoir (Moss Lake)	1973	City of King Mtn.	RE	68[1]	1329	99	840	[b]	44,400	53,280	736[2]
Lake Lure	1927	Town of Lake Lure	CNVA	95	740	124	480	[b]	32,295	44,914	991[2]
Lake Adger (Turner Shoals)	1925	Hydro, LLC	CNVA	138[1]	460[1]	90	689	[b]	11,700[1]	16,760	912[2]
Lake Summit	1920[2]	Duke Energy	CNSA	43[1]	276	130[2]	254[2]	[b]	9300	15,840	2012.6[2]
Lake Whelchel	1964	City of Gaffney	RE	14.7	177	61	2100	565	2438	5698	670[4]
Make-Up Pond C	[c]	Duke Energy	RE	3.87	620	132	2370	80	22,000	28,764	650

ac. - acre
ac.-ft. - acre-foot
sq. mi. - square miles
ft. - feet
msl - mean sea level
DPC - Duke Power Company

RE – earth filled
CN – concrete
CNPG – concrete gravity arch
CNVA - Concrete mult-arch
CNSA - Concrete single-arch

a) Dams and reservoirs on the Broad River and its major tributaries are utilized for thermoelectric power, water supply, and recreation and not for significant flood control.
b) No seismic design or spillway design criteria available for review.
c) Under development.

Sources:
1) Reference 230; North Carolina Department of Environmental and Natural Resources, "Broad River Basinwide Water Quality Plan", March 2003.
2) Reference 217; Duke Power Company, "Ninety-Nine Islands Hydro Project, FERC Project No. 2331, Determination of Probable Maximum Flood", Duke Engineering Services, Charlotte, North Carolina, November 7, 1997."
3) Reference 276; U.S. Army Corps of Engineers, "National Inventory of Dams, Website, http:/crunch/tec.army.mil/nid/webpages/nid.cfm, accessed June 2006
4) USGS Quadrangle, Blacksburg South, South Carolina

Table 2.4.1-206
SCDHEC 2005 Water Usage for Cherokee County, South Carolina

Usage	Quantity		
	Million Gallons	Mgd$^{(1)}$	cfs
Public Supply	2561.1	7.02	10.9
Industrial	504.13	1.38	2.14
TOTAL	3065.23	8.4	13.02

Source: Reference 267

Mgd - Million gallons per day

cfs - cubic feet per second

SCDHEC - South Carolina Department of Health and Environmental Control

(1) Quantity reported by SCDHEC in Million Gallons. Mgd was estimated by dividing SCDHEC's reported 2005 water use by 365 days.

Table 2.4.1-207
SCDHEC 2005 Water Usage for Cherokee, Chester, Greenville, Spartanburg, Union, and York Counties, South Carolina

County Name	Total Withdrawals					
	Groundwater		Surface Water		Total	
	Mgd	cfs	Mgd	cfs	Mgd	cfs
Cherokee, SC	0.003	0.005	8.39	13.0	8.40	13.0
Chester, SC	0.07	0.11	3.55	5.50	3.62	5.61
Greenville, SC	0.34	0.53	66.6	103	67.0	104
Spartanburg, SC	4.01	6.22	41.6	64.5	45.6	70.7
Union, SC	0.008	0.012	4.88	7.56	4.89	7.58
York, SC	0.27	0.42	93.1	144	93.4	145

Note: Withdrawal totals excluded hydroelectric power usage

Source: Reference 267

Mgd - Million gallons per day

cfs - cubic feet per second

Table 2.4.1-208
2000 Water Use Totals by County in the Upper Broad River Watershed

	Total Withdrawals					
	Groundwater		Surface Water		Total	
County Name	Mgd	cfs	Mgd	cfs	Mgd	cfs
Cherokee, SC	0.44	0.68	15.4	23.9	15.9	24.6
Chester, SC	1.80	2.79	4.6	7.1	6.4	9.9
Greenville, SC	3.03	4.70	53.3	82.6	56.3	87.1
Spartanburg, SC	4.01	6.22	57.0	88.3	61.0	94.4
Union, SC	0.25	0.39	8.2	12.7	8.5	13.2
York, SC	8.52	13.2	209	324	217	335.7
Buncombe, NC	8.77	13.6	33.7	52.3	42.5	65.8
Burke, NC	3.09	4.79	21.0	32.5	24.1	37.3
Catawba, NC	6.18	9.58	1182	1832	1188	1838
Cleveland, NC	2.51	3.89	189	293	192	297
Gaston, NC	7.67	11.9	965	1495	972	1504
Henderson, NC	3.7	5.74	13.4	20.8	17.1	26.5
Lincoln, NC	3.77	5.84	6.15	9.53	9.9	15.3
McDowell, NC	4.39	6.80	4.09	6.34	8.5	13.2
Polk, NC	1.24	1.92	1.48	2.29	2.7	4.2
Transylvania, NC	1.88	2.91	22.1	34.2	23.9	36.9

NOTES:

1. Greenville, Union, and York Counties within the Broad River Watershed are not part of the drainage basin for the Broad River adjacent to the site.
2. Cherokee, Cleveland, Polk, and Rutherford Counties compose the majority of the area in the Broad River Watershed above the site.
3. Total withdrawals for aquaculture and mining were 0 Mgd for all counties.
4. Hydroelectric water use not included.

Mgd - Million gallons per day

cfs - cubic feet per day

Source: Reference 286

Table 2.4.1-209 (Sheet 1 of 3)
Area Surface Water Intakes in and Downstream from the Upper Broad River Watershed

Facility	County, State	Distance		Source	Withdrawal Capacity		Consumptive Use(f)		Use Type
		mi.[a]	Direction		Mgd	cfs	Mgd	cfs	
Gaffney BPW	Cherokee, SC	8	Upstream	Lake Whelchel	12	18.6	NIA	NIA	Public Supply
Gaffney BPW	Cherokee, SC	9	Upstream	Broad River	(b)	(b)	NIA	NIA	Public Supply
CNA Holdings, Inc. – Ticona-Shelby	Cleveland, NC	12	Upstream	Buffalo Creek	1.15	1.78	0.290	0.45	Industrial
Shelby	Cleveland, NC	13	Upstream	Broad River	10 [c]	15.5	0	0	Public Supply
Northbrook Carolina Hydro, LLC – Stice Shoals Plant	Cleveland, NC	14	Upstream	First Broad River	(e)	(e)	(e)	(e)	Instream Hydro
Martin Marietta Materials, Inc (Kings Mountain Quarry)	Cleveland, NC	16	Upstream	Storm Water Quarry	0.23	0.36	0	0	Industrial
Kings Mountain	Cleveland, NC	17	Upstream	Moss Lake	37.6	58.3	1.611	2.50	Public Supply
Cleveland County Country Club	Cleveland, NC	18	Upstream	Lake/Pond	1.15	1.79	0.047	0.07	Golf Course
Shelby	Cleveland, NC	19	Upstream	First Broad River	18	28	2.424	4	Public Supply
Duke Energy Corp. – Cliffside Steam Station	Cleveland, NC	19	Upstream	Broad River	288	446	75	116	Industrial
Duke Energy Corp. – Cliffside Steam Station (planned)[d]	Cleveland, NC	19	Upstream	Broad River	32	50	20.645	32	Industrial
Cleveland-Caroknit	Cleveland, NC	25	Upstream	First Broad River	1	1.55	0.017	0.03	Industrial

Table 2.4.1-209 (Sheet 2 of 3)
Area Surface Water Intakes in and Downstream from the Upper Broad River Watershed

Facility	County, State	Distance		Source	Withdrawal Capacity		Consumptive Use(f)		Use Type
		mi.[(a)]	Direction		Mgd	cfs	Mgd	cfs	
Mako Marine International (formerly ITG/Burlington Industries – J.C. Cowan Plant)	Rutherford, NC	26	Upstream	Second Broad River	3	4.65	0.07	0.11	Industrial
Cleveland County Sanitary District	Cleveland, NC	27	Upstream	First Broad River	6	9.3	3.364	5.21	Public Supply
Cleveland County Sanitary District (planned)	Cleveland, NC	27	Upstream	Knob Creek	6	9.3	3.445	5.3	Public Supply
Forest City	Rutherford, NC	31	Upstream	Second Broad River	12	18.60	1.483	2.30	Public Supply
Broad River Water Authority (formerly Rutherfordton-Spindale)	Rutherford, NC	33	Upstream	Broad River	13	20.15	4.733	7.34	Public Supply
Northbrook Carolina Hydro, LLC – Turner Shoals Plant	Polk, NC	43	Upstream	Green River	(e)	(e)	(e)	(e)	Instream Hydro
Duke Energy Corp. – Tuxedo Hydro	Henderson, NC	52	Upstream	Lake Summit	(e)	(e)	(e)	(e)	Instream Hydro
Kenmure Country Club	Henderson, NC	54	Upstream	King Creek	0.82	1.26	0.97	1.50	Golf Course
City of Union	Union, SC	21	Downstream	Broad River	23.80	36.89	NIA	NIA	Public Supply
Carlisle Cone Mills	Union, SC	30	Downstream	Broad River	8.10	12.56	NIA	NIA	Public Supply
V.C. Summer Nuclear Station	Fairfield, SC	52	Downstream	Lake Monticello	3.1	4.81	NIA	NIA	Industrial

Table 2.4.1-209 (Sheet 3 of 3)
Area Surface Water Intakes in and Downstream from the Upper Broad River Watershed

Facility	County, State	Distance		Source	Withdrawal Capacity		Consumptive Use(f)		Use Type
		mi.[(a)]	Direction		Mgd	cfs	Mgd	cfs	
V.C. Summer Nuclear Station (planned)	Fairfield, SC	52	Downstream	Lake Monticello	NIA	NIA	NIA	NIA	Industrial

Notes:

a) Distance provided is a linear distance and not river miles.

b) The Gaffney BPW (Board of Public Works) system is authorized 18 Mgd and uses Lake Whelchel for storage.

c) The Shelby Broad River intake is used as a temporary emergency supply intake.

d) Additional Cliffside Steam Plant use rate is based on anticipated expansion of 1 unit. “Planned” figures include the consumption of the existing Cliffside Unit 5 (15 cfs), and the planned expansion unit (17 cfs)

e) Instream hydro facilities maximum use rate not reported. Instream water use indicates water is returned directly to source. Additional hydro facilities are present within watershed, but no withdrawal permits exist.

f) Consumptive Use based on reported withdrawals and returns from 1999 registration and 2002 LWSP reports.

Source: Reference 268, References 234 - 248, References 262 - 265.

See Figure 2.4.1-211

NIA - No Information Available

Mgd - Million gallons per day

cfs - cubic feet per second

Table 2.4.1-210
Estimated Surface Water Withdrawal and Consumption for Lee Nuclear Station Operations

Broad River Flow Rates[a]		Average Withdrawal		Maximum Withdrawal	
cfs	gpm	gpm	cfs	gpm	cfs
Mean Annual Flow Approximately 2500 cfs (1926-2008)	1,122,000	35,030	78	60,001	134
Regulatory Low Flow[b] (FERC) 483 cfs	216,867	35,030	78	NA	NA

Broad River Flow Rates[a]		Average Consumption		Maximum Consumption	
cfs	gpm	gpm	cfs	gpm	cfs
Mean Annual Flow Approximately 2500 cfs (1926-2008)	1,122,000	24,813	55	28,274	63
Regulatory Low Flow[b] (FERC) 483 cfs	216,867	24,813	55	NA	NA

gpm - gallons per minute

cfs - cubic feet per second

NA - not applicable

Notes:

a) Broad River flow rates were compiled from USGS measurements recorded at the Gaffney Gauge (USGS Gauge #02153500), the Blacksburg Gauge (#02153200) and Boiling Springs Gauge (#02151500) for average annual flow.

b) The 7Q10 for the Gaffney gauge was determined to be 439 cfs using the USGS recommended Log-Pearson Type III distribution. However, because the 7Q10 is less than the Ninety-Nine Islands Dam FERC license minimum flow requirement of 483 cfs for July through November, the FERC license minimum flow was used as a constraint in evaluating operation during low flow conditions.

Table 2.4.1-211
Estimated Discharge Volume From Station Operations

Broad River Flow Rates[a]		Average Discharge		Maximum Discharge	
cfs	gpm	gpm	cfs	gpm	cfs
Mean Annual Flow Approximately 2500 cfs (1926-2008)	1,122,000	8,216	18	28,778	64
Regulatory Low Flow[b] (FERC) 483 cfs	216,867	8,216	18	28,778	64

Notes:

a) Broad River flow rates were compiled from USGS measurements recorded at the Gaffney Gauge (USGS Gauge #02153500), the Blacksburg Gauge (#02153200) and Boiling Springs Gauge (#02151500) for average annual flow.

b) The 7Q10 for the Gaffney gauge was determined to be 439 cfs using the USGS recommended Log-Pearson Type III distribution. However, because the 7Q10 is less than the Ninety-Nine Islands Dam FERC license minimum flow requirement of 483 cfs for July through November, the FERC license minimum flow was used as a constraint in evaluating operation during low flow conditions.

Withhold from Public Disclosure Under 10 CFR 2.390(a)(9)

Table 2.4.1-212 (Sheet 1 of 3)
Historical Domestic Wells in Vicinity of Site

Well Number	Former Owner or Description	Diameter (in.)	Total Depth (ft.)	Depth to Water (ft.)	Flow Rate		Surface Elevation (ft.)	Remarks-Driller
					gpm	cfs		
1	Smith Davis	24	48	34	(a)	(a)	691	Hand Dug
2	Wayne E. Stepp	24	47	34	(a)	(a)	688	Hand Dug
3	R.B. Peterson	(a)	(a)	(a)	(a)	(a)	680	
4	Danny Cain	24	65	38	(a)	(a)	650	Arnold Drill Company
5	William T. Mullinax	24	48	28	(a)	(a)	661	
6	Al Mullinax	24	47	40	(a)	(a)	661	Hand Dug
7	Pat Mckown	24	36	25	(a)	(a)	681	Rented House-Hand Dug
8	Jack Orr	8	145	45	4	0.0089	650	Arnold Drill Company
9	Pearl Peterson	(a)	(a)	(a)	(a)	(a)	671	No Well, Water Supplied by Spring
10	David Peterson	24	100	67	(a)	(a)	688	Spangler of Shelby, NC
11	Brent Upchurch	24	80	58	(a)	(a)	689	Man Owens
12	J.D. Upchurch	24	47	37	(a)	(a)	691	Falkner of Kings Creek, SC
13	Boyce Sellars	24	49	42	(a)	(a)	685	
14	Larry Hughey	24	63	39	(a)	(a)	672	
15	Larry Hughey	26	33	25	(a)	(a)	670	Abandoned House
16	McKowns Mt. Church	24	65	41	(a)	(a)	661	
17	Ralph Mosley	24	55	40	(a)	(a)	662	
18	Lamar Mullinax	24	75	55	(a)	(a)	655	Lower Capacity Well
19	Lewis F. Moss	24	75	55	(a)	(a)	653	Higher Capacity Well

Withhold from Public Disclosure Under 10 CFR 2.390(a)(9)

Table 2.4.1-212 (Sheet 2 of 3)
Historical Domestic Wells in Vicinity of Site

Well Number	Former Owner or Description	Diameter (in.)	Total Depth (ft.)	Depth to Water (ft.)	Flow Rate gpm	Flow Rate cfs	Surface Elevation (ft.)	Remarks-Driller
20	Vacant House	24	51	51	(a)	(a)	665	
21	Dean Upchurch	(a)	(a)	(a)	(a)	(a)	538	No Well, Water Supplied by Spring
22	Bonnie Sellars	24	33	25	(a)	(a)	680	
23	Dobson Mullinax	24	29	14	(a)	(a)	660	
24	Margaret Peeler	24	76	33	(a)	(a)	652	
25	Marvin Mullinax	24	60	32	(a)	(a)	668	
26	J. Cash	24	65	35	(a)	(a)	646	
27	Wilton Stroupe	6.5	149	15	25	0.056	648	House Removed, Well Filled with Rocks
28	Minnie Stroupe	6.5	150	17	25	0.056	660	
29	Noal Lawson	24	59	41	(a)	(a)	651	
30	Milles Mullinax	8	55	38	(a)	(a)	635	
31	Stroupe	24	61	37	(a)	(a)	645	
32	Rose Spencer	24	62	36	(a)	(a)	639	
33	Sam Erwin	24	56	37	(a)	(a)	635	House Removed
34	J.R. Stroupe	6	250	110	7–10	0.022	640	Falkner of Kings Creek, SC
35	Nancy Huey Owens	24	52	41	(a)	(a)	620	
36	Lamar Mullinax	24	96	72	(a)	(a)	630	
37	Owner-Junior Erwin	24	79	57	(a)	(a)	605	
38	Junior Erwin	24	74	64	(a)	(a)	600	House Removed
39	Owner-Erwin Resident-Mackabe	24	44	34	(a)	(a)	545	

Withhold from Public Disclosure Under 10 CFR 2.390(a)(9)

Table 2.4.1-212 (Sheet 3 of 3)
Historical Domestic Wells in Vicinity of Site

Well Number	Former Owner or Description	Diameter (in.)	Total Depth (ft.)	Depth to Water (ft.)	Flow Rate gpm	Flow Rate cfs	Surface Elevation (ft.)	Remarks-Driller
40	Jesse Peterson	8	147	65	16	0.036	(a)	
41	Bob Upchurch	6	65	(a)	7	0.016	539	
42	J.C. Upchurch	6	115	31	12	0.027	571	
43	Albert Hughey	24	40	30	(a)	(a)	633	
44	Nesbitt Stroupe	18	65	49	(a)	(a)	654	White Scale in Well
45	Boyd Childers	(a)	10	(a)	(a)	(a)	520	Spring Dug Out
46	Roy Parker	24	73	32	(a)	(a)	520	Roebucks
47	C.R. Davis	(a)	44	32	(a)	(a)	520	Old Well
		24	61	32	(a)	(a)	520	Well in Use
48	Wilton Stroupe	6.5	165	28	12	0.027	520	
49	Emma Upchurch	24	83	75	(a)	(a)	715	
50	Effie Mullinax	24	51	39	(a)	(a)	709	

Notes:

a) No Data

Table 2.4.2-201 (Sheet 1 of 2)
Peak Streamflow of the Broad River Near Gaffney, South Carolina
(USGS Station 02153500) 1939-1990

Water Year[(a)]	Date	Discharge (cfs)
1939	8/18/1939	21,000
1940	8/14/1940	119,000
1941	7/17/1941	26,000
1942	2/17/1942	21,800
1943	1/28/1943	38,400
1944	3/20/1944	21,700
1945	9/18/1945	61,600
1946	1/7/1946	43,400
1947	6/15/1947	27,800
1948	8/4/1948	25,600
1949	11/29/1948	35,700
1950	10/7/1949	31,000
1951	12/8/1950	23,900
1952	3/4/1952	44,200
1953	2/21/1953	21,900
1954	1/23/1954	41,000
1955	2/7/1955	14,700
1956	4/16/1956	22,400
1957	4/6/1957	23,400
1958	4/28/1958	37,900
1959	9/30/1959	38,600
1960	2/6/1960	37,200
1961	6/22/1961	26,600
1962	12/13/1961	28,400
1963	3/13/1963	41,800
1964	4/8/1964	31,100
1965	10/6/1964	67,100
1966	3/4/1966	32,600

Table 2.4.2-201 (Sheet 2 of 2)
Peak Streamflow of the Broad River Near Gaffney, South Carolina
(USGS Station 02153500) 1939-1990

Water Year[a]	Date	Discharge (cfs)
1967	8/24/1967	33,800
1968	3/13/1968	25,900
1969	4/19/1969	25,400
1970	8/10/1970	47,500
1971	10/31/1970	14,300
1972	10/16/1971	46,900
1973	3/17/1973	42,900
1974	4/5/1974	34,400
1975	3/15/1975	55,300
1976	10/18/1975	32,100
1977	10/10/1976	84,900
1978	11/7/1977	38,100
1980	7/21/1980	37,600
1981	10/1/1980	10,500
1982	1/4/1982	33,900
1983	2/3/1983	21,900
1984	2/14/1984	39,900
1985	8/18/1985	26,600
1986	8/18/1986	10,900
1987	3/1/1987	65,800
1988	1/20/1988	8,700
1989	2/28/1989	12,500
1990	10/2/1989	38,800

a) Water Year = October 1 to September 30

Note: Peak streamflow for water year 1979 not available.

(Reference 290)

Table 2.4.2-202
Peak Gauge Height of the Broad River below Ninety-Nine Islands Reservoir, South Carolina (USGS Station 02153551) 1999-2005

Water Year[a]	Date	Gauge Height[b] (feet)	Discharge (cfs)
1999	4/2/1999	30.77	4,350
2000	3/21/2000	32.91	(c)
2001	3/30/2001	31.37	(c)
2002	1/24/2002	30.01	4,490
2003	3/20/2003	38.22	(c)
2004	9/9/2004	40.43	(c)
2005	12/10/2004	35.19	(c)

a) Water Year = October 1 to September 30

b) Datum = 412.2 feet above NGVD29

c) not recorded

(Reference 290)

Table 2.4.2-203
Local Intense Probable Maximum Precipitation for the Lee Nuclear Site

	Duration								
	5-min.	15-min.	30-min.	1-hr.	6-hr.	12-hr.	24-hr.	48-hr.	72-hr.
PMP(in.)	6.2	9.7	14	18.9	29.9	35.5	40.4	44.3	46.8

Note: Durations from 5-min. to 1-hr. derived from HMR No. 52, 1-sq. mi. point rainfall. Durations from 6-hr. to 72-hr. derived from HMR No. 51, 10-sq. mi. point rainfall.

Table 2.4.2-204
Site Drainage Areas Details

Drainage Area	Area Acres (ac)	Flow Rate (cfs)	Maximum Velocity (fps)	Maximum Depth of Flow (ft.)	Maximum Water Surface Elevation (ft.)
A1	1.62	121	3.51	0.43	592.43
B1	5.19	389	3.44	0.76	592.56
C1	2.01	151	1.39	0.53	592.03
D1	7.93	595	2.05	0.35	592.35
A2	1.62	121	3.51	0.43	592.43
B2	5.19	389	3.44	0.76	592.56
C2	2.01	151	1.39	0.53	592.03
D2	7.44	558	1.97	0.32	592.32

Table 2.4.3-201
Broad River Watershed PMP (in.) Depth-Area-Duration Relationship

Area	Duration (hr.)				
(sq. mi.)	6	12	24	48	72
10	29.7	35.3	40	43.5	46
200	21.5	25.8	30.1	33.5	36
1000	15.9	20.7	24.8	28.2	30.1
5000	9.3	13.1	16.9	20.9	23
10,000	7.1	10.4	13.9	17.7	19.7
20,000	5.1	8.4	11.2	14.8	16.8

Note: Values derived from the all-season PMP charts published in HMR-51 (Reference 255).

Table 2.4.3-202
Broad River Watershed 6-hr. Incremental PMP Estimates

Duration (hr.)	Incremental PMP (in.)
6	0.38
12	0.46
18	0.59
24	0.83
30	1.38
36	4.30
42	12.80
48	2.09
54	1.03
60	0.69
66	0.52
72	0.41
Total	25.48

Note: Values derived from HMR-51 (Reference 255), HMR-52 (Reference 225) and the use of the USACE HMR-52 computer software (Reference 271). Critical storm was determined to be 1000 sq. mi. with a 270 degree orientation centered near the centroid of the watershed for Gaston Shoals Dam.

Table 2.4.3-203 (Sheet 1 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09	WLCHL
Day 1 0100	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0200	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0300	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0400	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0500	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0600	0.045	0.067	0.069	0.064	0.069	0.066	0.069	0.064	0.067	0.039
0700	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
0800	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
0900	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
1000	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
1100	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
1200	0.055	0.082	0.084	0.078	0.084	0.080	0.084	0.078	0.082	0.047
1300	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1400	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1500	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1600	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1700	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1800	0.070	0.106	0.108	0.100	0.108	0.103	0.108	0.101	0.105	0.061
1900	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085
2000	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085
2100	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085
2200	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085
2300	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085
2400	0.098	0.148	0.151	0.140	0.151	0.144	0.151	0.141	0.147	0.085

Table 2.4.3-203 (Sheet 2 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09	WLCHL
Day 2 0100	0.143	0.215	0.220	0.204	0.220	0.210	0.220	0.205	0.215	0.124
0200	0.149	0.225	0.230	0.213	0.229	0.219	0.229	0.214	0.224	0.129
0300	0.157	0.237	0.242	0.224	0.242	0.231	0.242	0.225	0.236	0.136
0400	0.167	0.251	0.257	0.238	0.256	0.244	0.256	0.239	0.250	0.144
0500	0.178	0.267	0.274	0.253	0.274	0.261	0.274	0.255	0.266	0.154
0600	0.190	0.286	0.293	0.271	0.294	0.279	0.294	0.273	0.285	0.165
0700	0.357	0.542	0.563	0.511	0.576	0.530	0.574	0.520	0.541	0.311
0800	0.411	0.628	0.655	0.591	0.674	0.614	0.670	0.602	0.628	0.361
0900	0.465	0.715	0.750	0.672	0.778	0.700	0.773	0.687	0.715	0.407
1000	0.517	0.803	0.849	0.753	0.889	0.788	0.881	0.775	0.804	0.450
1100	0.567	0.891	0.950	0.834	1.007	0.878	0.995	0.865	0.894	0.489
1200	0.616	0.980	1.054	0.916	1.131	0.971	1.115	0.958	0.985	0.526
1300	0.774	1.272	1.415	1.185	1.594	1.281	1.551	1.275	1.288	0.641
1400	1.040	1.804	2.067	1.667	2.418	1.837	2.332	1.839	1.836	0.838
1500	1.261	2.429	2.821	2.211	3.324	2.462	3.202	2.459	2.473	1.002
1600	1.309	3.994	4.838	3.430	5.757	3.954	5.539	3.908	4.069	1.040
1700	1.182	2.167	2.512	1.987	2.970	2.208	2.858	2.210	2.209	0.942
1800	0.964	1.637	1.865	1.518	2.170	1.665	2.095	1.666	1.665	0.782
1900	0.311	0.471	0.487	0.445	0.497	0.461	0.495	0.452	0.470	0.269
2000	0.279	0.419	0.432	0.397	0.439	0.410	0.438	0.402	0.418	0.242
2100	0.252	0.377	0.387	0.357	0.391	0.368	0.391	0.361	0.376	0.218
2200	0.230	0.343	0.351	0.325	0.354	0.335	0.354	0.328	0.342	0.199
2300	0.213	0.318	0.325	0.302	0.327	0.311	0.327	0.304	0.317	0.185
2400	0.201	0.302	0.309	0.286	0.310	0.295	0.310	0.288	0.301	0.174

Table 2.4.3-203 (Sheet 3 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09	WLCHL
Day 3 0100	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0200	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0300	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0400	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0500	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0600	0.123	0.185	0.189	0.175	0.189	0.180	0.189	0.176	0.184	0.106
0700	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
0800	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
0900	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
1000	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
1100	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
1200	0.082	0.123	0.126	0.117	0.126	0.120	0.126	0.117	0.123	0.071
1300	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1400	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1500	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1600	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1700	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1800	0.061	0.092	0.095	0.088	0.095	0.090	0.095	0.088	0.092	0.053
1900	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
2000	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
2100	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
2200	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
2300	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
2400	0.049	0.074	0.076	0.070	0.076	0.072	0.076	0.071	0.074	0.043
Total	15.44	26.84	29.54	24.79	32.51	26.64	31.81	26.33	27.05	12.96

Table 2.4.3-203 (Sheet 4 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	MUPC
Day 1 0100	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0200	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0300	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0400	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0500	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0600	0.064	0.046	0.045	0.065	0.052	0.031	0.064	0.064	0.065	0.024
0700	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
0800	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
0900	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
1000	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
1100	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
1200	0.078	0.057	0.055	0.079	0.063	0.038	0.078	0.078	0.080	0.029
1300	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1400	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1500	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1600	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1700	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1800	0.100	0.073	0.070	0.102	0.081	0.049	0.101	0.100	0.102	0.038
1900	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053
2000	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053
2100	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053
2200	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053
2300	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053
2400	0.140	0.102	0.098	0.142	0.114	0.069	0.141	0.140	0.143	0.053

Table 2.4.3-203 (Sheet 5 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	MUPC
Day 2 0100	0.204	0.148	0.143	0.207	0.165	0.100	0.206	0.204	0.208	0.077
0200	0.213	0.155	0.149	0.216	0.173	0.105	0.215	0.213	0.217	0.080
0300	0.224	0.163	0.157	0.228	0.182	0.110	0.226	0.224	0.229	0.085
0400	0.238	0.173	0.166	0.241	0.193	0.117	0.239	0.238	0.243	0.090
0500	0.254	0.184	0.178	0.257	0.205	0.124	0.255	0.253	0.259	0.096
0600	0.271	0.198	0.190	0.276	0.220	0.133	0.273	0.271	0.277	0.103
0700	0.511	0.370	0.357	0.516	0.410	0.247	0.514	0.510	0.527	0.181
0800	0.590	0.426	0.411	0.596	0.471	0.285	0.595	0.590	0.611	0.205
0900	0.670	0.481	0.465	0.676	0.533	0.319	0.675	0.670	0.697	0.225
1000	0.750	0.536	0.517	0.756	0.594	0.349	0.757	0.750	0.785	0.243
1100	0.831	0.589	0.567	0.835	0.656	0.376	0.839	0.832	0.876	0.258
1200	0.913	0.641	0.616	0.914	0.718	0.398	0.923	0.913	0.968	0.271
1300	1.180	0.811	0.774	1.171	0.918	0.468	1.196	1.182	1.282	0.309
1400	1.657	1.097	1.040	1.632	1.261	0.589	1.686	1.663	1.843	0.373
1500	2.191	1.335	1.261	2.159	1.572	0.690	2.233	2.198	2.470	0.429
1600	3.354	1.386	1.309	3.310	1.829	0.717	3.439	3.358	3.967	0.445
1700	1.972	1.251	1.182	1.939	1.456	0.652	2.009	1.979	2.216	0.407
1800	1.510	1.017	0.964	1.488	1.162	0.554	1.534	1.515	1.670	0.355
1900	0.445	0.323	0.311	0.450	0.359	0.216	0.448	0.445	0.458	0.162
2000	0.397	0.289	0.279	0.402	0.322	0.195	0.400	0.397	0.408	0.149
2100	0.357	0.261	0.252	0.363	0.291	0.177	0.360	0.357	0.366	0.137
2200	0.326	0.238	0.230	0.331	0.266	0.162	0.328	0.326	0.333	0.126
2300	0.302	0.221	0.213	0.307	0.246	0.150	0.304	0.302	0.308	0.117
2400	0.286	0.209	0.201	0.291	0.233	0.142	0.288	0.286	0.293	0.110

Table 2.4.3-203 (Sheet 6 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)									
Time (hr.)	FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	MUPC
Day 3 0100	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0200	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0300	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0400	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0500	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0600	0.175	0.127	0.123	0.178	0.142	0.086	0.176	0.175	0.179	0.066
0700	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
0800	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
0900	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
1000	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
1100	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
1200	0.117	0.085	0.082	0.119	0.095	0.057	0.118	0.117	0.119	0.044
1300	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1400	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1500	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1600	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1700	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1800	0.088	0.064	0.061	0.089	0.071	0.043	0.088	0.088	0.089	0.033
1900	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
2000	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
2100	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
2200	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
2300	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
2400	0.070	0.051	0.049	0.071	0.057	0.034	0.071	0.070	0.072	0.026
Total	24.64	16.14	15.44	24.63	18.48	9.82	24.96	24.67	26.60	6.91

Table 2.4.3-203 (Sheet 7 of 7)
Broad River Watershed Subbasin Hourly Incremental PMP Estimates

	Subbasin Hourly Incremental PMP (in.)

Notes:

Reference Figure 2.4.3-203 for subbasin locations
LS-1, Lake Summit/Tuxedo Hydro
LA-2, Lake Adger/Turner Shoals
GD-3, Green River (Turner Shoals to Broad R.)
LL-4, Lake Lure/hydro
BR-5, Broad River (Lake Lure to Green R.)
BD3-6, Broad River (Green R. to Second Broad R.)
2BR-7, Second Broad River
BD2-8, Broad River (Second Broad R. to First Broad R.)
SS-09, Stice Shoals
FB-10, First Broad River (Stice Shoals to Broad R.)
GS-11, Broad River (First Broad to Gaston Shoals)
BD1-12, Broad River (Gaston Shoals to Buffalo Creek)
KMR-13, Kings Mountain Reservoir (Buffalo Cr.)
BC-14, Buffalo Creek (Kings Mountain Reservoir to Broad R.)
BR-15, Broad River (Buffalo Cr. to Ninety-Nine Islands)
CC-16, Cove Creek (Broad R. near Lake Lure)
USS-18A, Upper First Broad River
2BR-19, Upper Second Broad River
MUPC, Make-Up Pond C
WLCHL, Lake Whelchel

Table 2.4.3-204 (Sheet 1 of 2)
Broad River Watershed Subbasin Precipitation Losses

			Antecedent Precipitation			PMP Precipitation		
Subbasin	CN	Initial Losses (in.)	Depth (in.)	Losses (in.)	Excess (in.)	Depth (in.)	Losses (in.)	Excess (in.)
LS-1	55	1.64	6.18	4.51	1.66	15.44	2.86	12.57
LA-2	56	1.57	10.73	5.76	4.97	26.84	2.21	24.63
GD-3	60	1.33	11.81	5.41	6.40	29.54	1.64	27.89
LL-4	56	1.57	9.92	5.55	4.37	24.79	2.28	22.50
BR-5	58	1.45	13.01	5.90	7.11	32.51	1.77	30.74
BD3-6	64	1.13	10.66	4.67	5.99	16.64	1.33	25.31
2BR-7	60	1.33	12.72	5.54	7.18	31.81	1.57	30.23
BD2-8	66	1.03	10.53	4.35	6.17	26.33	1.16	25.16
SS-09	68	0.94	10.82	4.13	6.69	27.05	0.99	26.06
FB-10	71	0.82	9.86	3.63	6.23	24.64	0.83	23.81
GS-11	65	1.08	6.46	3.70	2.76	16.14	1.58	14.54
BD1-12	67	0.99	6.18	3.45	2.73	15.44	1.42	14.01
KMR-13	68	0.94	9.85	3.95	5.90	24.63	1.03	23.60
BC-14	67	0.99	7.39	3.70	3.69	18.48	1.30	17.18
BR-15	65	1.08	3.93	2.89	1.03	9.82	1.88	7.93
CC-16	56	1.57	9.99	5.63	4.35	24.96	2.30	22.67
USS-18A	56	1.57	9.87	5.61	4.26	24.67	2.31	22.36
2BR-19	56	1.57	10.64	5.78	4.86	26.60	2.23	24.37
MUPC	63.9	1.13	2.76	1.74	1.03	6.91	1.54	5.37
WLCHL	63.7	1.14	5.18	3.42	1.77	12.96	1.86	11.10

Table 2.4.3-204 (Sheet 2 of 2)
Broad River Watershed Subbasin Precipitation Losses

Notes:

Reference Figure 2.4.3-203 for subbasin locations
LS-1, Lake Summit/Tuxedo Hydro
LA-2, Lake Adger/Turner Shoals
GD-3, Green River (Turner Shoals to Broad R.)
LL-4, Lake Lure/hydro
BR-5, Broad River (Lake Lure to Green R.)
BD3-6, Broad River (Green R. to Second Broad R.)
2BR-7, Second Broad River
BD2-8, Broad River (Second Broad R. to First Broad R.)
SS-09, Stice Shoals
FB-10, First Broad River (Stice Shoals to Broad R.)
GS-11, Broad River (First Broad to Gaston Shoals)
BD1-12, Broad River (Gaston Shoals to Buffalo Creek)
KMR-13, Kings Mountain Reservoir (Buffalo Cr.)
BC-14, Buffalo Creek (Kings Mountain Reservoir to Broad R.)
BR-15, Broad River (Buffalo Cr. to Ninety-Nine Islands)
CC-16, Cove Creek (Broad R. near Lake Lure)
USS-18A, Upper First Broad River
2BR-19, Upper Second Broad River
MUPC, Make-Up Pond C
WLCHL, Lake Whelchel

Table 2.4.3-205 (Sheet 1 of 7)
Broad River Watershed Subbasin Unit Hydrographs

Time (hr.)	Subbasin Unit Hydrograph Discharge (cfs) LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09
1	389	377	2	402	2	1	1	1	1
2	1597	2058	33	2055	33	22	24	32	12
3	2660	3601	144	3596	144	134	108	140	56
4	2883	4806	353	4799	352	380	270	364	156
5	2700	5306	660	5296	660	768	581	719	329
6	2400	4940	1051	5000	1050	1217	943	1140	589
7	2126	4490	1501	4500	1500	1729	1374	1712	941
8	1881	4138	1982	4086	1981	2272	1852	2254	1304
9	1652	3795	2470	3745	2469	2818	2354	2800	1705
10	1453	3437	2940	3390	2939	3163	2860	3147	2130
11	1271	3092	3276	3084	3275	3295	3350	3314	2565
12	1096	2805	3364	2798	3366	3200	3598	3250	3000
13	934	2515	3300	2508	3300	3080	3693	3110	3422
14	788	2232	3200	2225	3200	2950	3610	2950	3822
15	659	1964	3080	1958	3080	2800	3500	2825	4193
16	547	1715	2930	1709	2930	2670	3350	2700	4282
17	451	1488	2780	1483	2780	2560	3200	2600	4200
18	370	1283	2640	1279	2650	2439	3050	2457	4090
19	302	1101	2504	1097	2506	2327	2920	2349	3960
20	245	941	2379	937	2353	2197	2800	2220	3830
21	198	800	2238	797	2213	2079	2693	2105	3700
22	159	678	2087	675	2089	1976	2567	2003	3560
23	128	573	1954	570	1981	1864	2455	1893	3440
24	102	482	1816	480	1866	1747	2332	1777	3330
25	82	405	1700	403	1747	1627	2201	1657	3196
26	65	339	1603	337	1626	1507	2091	1537	3093
27	52	283	1504	282	1506	1389	1977	1438	2976
28	41	236	1404	235	1407	1291	1884	1338	2849
29	32	196	1306	195	1308	1194	1788	1240	2747
30	26	163	1209	162	1211	1101	1690	1145	2638
31	20	135	1116	134	1118	1011	1591	1053	2523

Table 2.4.3-205 (Sheet 2 of 7)
Broad River Watershed Subbasin Unit Hydrographs

Time (hr.)	Subbasin Unit Hydrograph Discharge (cfs) LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09
32	16	112	1026	111	1028	925	1493	965	2404
33	12	92	940	92	942	843	1396	881	2312
34	10	76	859	76	860	766	1301	802	2217
35	8	63	782	62	784	695	1209	728	2119
36	6	52	710	51	712	628	1121	659	2020
37	5	42	643	42	645	566	1036	595	1921
38	4	35	581	35	575	509	956	536	1822
39	3	28	524	28	519	457	880	482	1724
40	2	23	472	23	467	409	807	432	1627
41	2	19	423	19	419	365	740	387	1533
42	1	16	379	15	375	326	676	345	1442
43	1	13	339	13	336	290	617	308	1353
44	1	10	303	10	300	258	562	274	1268
45	1	8	270	8	267	228	511	243	1185
46	0	7	240	7	238	202	464	216	1107
47	0	6	213	6	211	179	421	191	1032
48	0	5	189	5	187	158	381	169	960
49	0	4	167	4	166	139	344	149	892
50	0	3	148	3	147	122	311	131	828
51	0	2	131	2	130	108	280	116	768
52	0	2	115	2	114	94	252	102	711
53	0	2	102	2	101	83	227	89	657
54	0	1	89	1	89	72	203	78	607
55	0	1	79	1	78	63	182	68	560
56	0	1	69	1	68	55	163	60	516
57	0	1	60	1	60	48	146	52	475
58	0	1	53	1	53	42	131	46	436
59	0	0	46	0	46	37	117	40	401
60	0	0	41	0	40	32	104	35	368
61	0	0	35	0	35	28	93	30	337
62	0	0	31	0	31	24	83	26	309

Table 2.4.3-205 (Sheet 3 of 7)
Broad River Watershed Subbasin Unit Hydrographs

Time (hr.)	Subbasin Unit Hydrograph Discharge (cfs)								
	LS-1	LA-2	GD-3	LL-4	BR-5	BD3-6	2BR-7	BD2-8	SS-09
63	0	0	27	0	27	21	74	23	283
64	0	0	23	0	23	18	65	20	258
65	0	0	20	0	20	16	58	17	236
66	0	0	18	0	18	14	52	15	216
67	0	0	15	0	15	12	46	13	197
68	0	0	13	0	13	10	41	11	179
69	0	0	12	0	12	9	36	10	163
70	0	0	10	0	10	8	32	8	149
71	0	0	9	0	9	7	28	7	135
72	0	0	8	0	8	6	25	6	123
73	0	0	7	0	7	5	22	5	112
74	0	0	6	0	6	4	19	5	101
75	0	0	5	0	5	4	17	4	92
76	0	0	4	0	4	3	15	3	83
77	0	0	4	0	4	3	13	3	76
78	0	0	3	0	3	2	12	3	68
79	0	0	3	0	3	2	10	2	62
80	0	0	2	0	2	2	9	2	56
81	0	0	2	0	2	1	8	2	51
82	0	0	2	0	2	1	7	1	46
83	0	0	2	0	1	1	6	1	41
84	0	0	1	0	1	1	5	1	37
85	0	0	1	0	1	1	5	1	34
86	0	0	1	0	1	1	4	1	30
87	0	0	1	0	1	1	4	1	27
88	0	0	1	0	1	1	3	1	25
89	0	0	1	0	1	0	3	0	22
90	0	0	0	0	0	0	0	0	0

Table 2.4.3-205 (Sheet 4 of 7)
Broad River Watershed Subbasin Unit Hydrographs

Time (hr.)	Subbasin Unit Hydrograph Discharge (cfs) FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	WLCHL
1	5	12	18	2	2	7	339	392	358	55
2	103	154	224	45	34	94	1754	1862	1723	566
3	366	521	701	203	142	346	3273	3826	3509	1125
4	772	1041	1290	536	347	800	4323	5139	4670	1366
5	1243	1679	1481	1008	651	1409	4625	5785	5112	1230
6	1814	1823	1400	1606	1037	2104	4360	5530	4840	1050
7	2070	1695	1220	2175	1483	2270	3950	5080	4430	886
8	1970	1540	1039	2581	2021	2170	3568	4684	4059	736
9	1810	1390	852	2747	2522	2020	3203	4237	3715	606
10	1640	1248	698	2710	3008	1870	2870	3865	3321	482
11	1496	1108	549	2580	3301	1730	2554	3500	2979	376
12	1366	965	430	2420	3392	1595	2292	3159	2634	284
13	1245	829	338	2260	3320	1489	2034	2851	2354	213
14	1123	707	261	2110	3180	1359	1786	2547	2084	156
15	996	600	197	1974	3020	1249	1555	2255	1828	112
16	879	501	147	1838	2870	1129	1344	1982	1592	80
17	766	412	108	1705	2730	1007	1153	1731	1377	56
18	660	336	78	1581	2580	899	984	1503	1185	38
19	562	270	56	1469	2444	794	836	1298	1014	26
20	474	215	40	1351	2324	695	707	1116	863	18
21	396	170	28	1232	2214	603	595	956	732	12
22	329	134	20	1129	2092	519	499	815	619	8
23	271	104	14	1028	1987	443	417	693	521	5
24	222	80	9	930	1875	376	347	587	438	3
25	181	62	6	836	1781	318	288	496	366	2
26	146	47	4	747	1682	267	239	418	306	1
27	118	36	3	665	1581	223	198	352	255	1
28	94	27	2	589	1479	185	163	295	212	1
29	75	21	1	519	1378	153	134	247	176	0
30	60	15	1	456	1278	126	110	206	145	0
31	47	12	1	399	1181	104	90	172	120	0

Table 2.4.3-205 (Sheet 5 of 7)
Broad River Watershed Subbasin Unit Hydrographs

	Subbasin Unit Hydrograph Discharge (cfs)									
Time (hr.)	FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	WLCHL
32	37	9	0	347	1088	85	74	143	99	0
33	29	6	0	302	999	69	60	119	82	0
34	23	5	0	261	914	56	49	99	67	0
35	18	4	0	226	834	46	40	82	55	0
36	14	3	0	194	759	37	33	68	45	0
37	11	2	0	167	689	30	27	56	37	0
38	8	1	0	143	623	24	22	46	30	0
39	7	1	0	122	563	19	17	38	25	0
40	5	1	0	104	507	15	14	31	20	0
41	4	1	0	89	456	12	11	26	16	0
42	3	0	0	75	410	10	9	21	13	0
43	2	0	0	64	367	8	7	18	11	0
44	2	0	0	54	328	6	6	14	9	0
45	1	0	0	46	293	5	5	12	7	0
46	1	0	0	39	261	4	4	10	6	0
47	1	0	0	33	232	3	3	8	5	0
48	1	0	0	27	206	2	3	6	4	0
49	0	0	0	23	183	2	2	5	3	0
50	0	0	0	19	162	2	2	4	3	0
51	0	0	0	16	143	1	1	4	2	0
52	0	0	0	13	127	1	1	3	2	0
53	0	0	0	11	112	1	1	2	1	0
54	0	0	0	9	99	1	1	2	1	0
55	0	0	0	8	87	0	1	2	1	0
56	0	0	0	7	76	0	0	1	1	0
57	0	0	0	5	67	0	0	1	1	0
58	0	0	0	4	59	0	0	1	0	0
59	0	0	0	4	52	0	0	1	0	0
60	0	0	0	3	45	0	0	1	0	0
61	0	0	0	3	40	0	0	0	0	0
62	0	0	0	2	35	0	0	0	0	0

Table 2.4.3-205 (Sheet 6 of 7)
Broad River Watershed Subbasin Unit Hydrographs

	Subbasin Unit Hydrograph Discharge (cfs)									
Time (hr.)	FB-10	GS-11	BD1-12	KMR-13	BC-14	BR-15	CC-16	USS-18A	2BR-19	WLCHL
63	0	0	0	2	30	0	0	0	0	0
64	0	0	0	1	26	0	0	0	0	0
65	0	0	0	1	23	0	0	0	0	0
66	0	0	0	1	20	0	0	0	0	0
67	0	0	0	1	17	0	0	0	0	0
68	0	0	0	1	15	0	0	0	0	0
69	0	0	0	1	13	0	0	0	0	0
70	0	0	0	0	11	0	0	0	0	0
71	0	0	0	0	10	0	0	0	0	0
72	0	0	0	0	9	0	0	0	0	0
73	0	0	0	0	7	0	0	0	0	0
74	0	0	0	0	6	0	0	0	0	0
75	0	0	0	0	6	0	0	0	0	0
76	0	0	0	0	5	0	0	0	0	0
77	0	0	0	0	4	0	0	0	0	0
78	0	0	0	0	4	0	0	0	0	0
79	0	0	0	0	3	0	0	0	0	0
80	0	0	0	0	3	0	0	0	0	0
81	0	0	0	0	2	0	0	0	0	0
82	0	e	0	0	2	0	0	0	0	0
83	0	0	0	0	2	0	0	0	0	0
84	0	0	0	0	2	0	0	0	0	0
85	0	0	0	0	1	0	0	0	0	0
86	0	0	0	0	1	0	0	0	0	0
87	0	0	0	0	1	0	0	0	e	0
88	0	0	0	0	1	0	0	0	0	0
89	0	0	0	0	1	0	0	0	0	0
90	0	0	0	0	0	0	0	0	0	0

Table 2.4.3-205 (Sheet 7 of 7)
Broad River Watershed Subbasin Unit Hydrographs

Notes:

Reference Figure 2.4.3-203 for subbasin locations
LS-1, Lake Summit/Tuxedo Hydro
LA-2, Lake Adger/Turner Shoals
GD-3, Green River (Turner Shoals to Broad R.)
LL-4, Lake Lure/hydro
BR-5, Broad River (Lake Lure to Green R.)
BD3-6, Broad River (Green R. to Second Broad R.)
2BR-7, Second Broad River
BD2-8, Broad River (Second Broad R. to First Broad R.)
SS-09, Stice Shoals
FB-10, First Broad River (Stice Shoals to Broad R.)
GS-11, Broad River (First Broad to Gaston Shoals)
BD1-12, Broad River (Gaston Shoals to Buffalo Creek)
KMR-13, Kings Mountain Reservoir (Buffalo Cr.)
BC-14, Buffalo Creek (Kings Mountain Reservoir to Broad R.)
BR-15, Broad River (Buffalo Cr. to Ninety-Nine Islands)
CC-16, Cove Creek (Broad R. near Lake Lure)
USS-18A, Upper First Broad River
2BR-19, Upper Second Broad River
WLCHL, Lake Whelchel

Table 2.4.3-206 (Sheet 1 of 2)
Broad River Watershed Subbasin Input Parameters

Subbasin	Area (sq. mi.)	Base Flow (Recession Method), Recession Constant, k = 0.4919		Loss Rates (SCS Method)	
		Initial Discharge per Area (cfs / sq. mi.)	Recession Threshold (cfs)	Curve Number CN	% Impervious Area
LS-1	42.4	1.62	254	55	0.91
LA-2	94.5	1.64	567	56	0.63
GD-3	106.6	1.64	640	60	0.01
LL-4	94.3	1.64	566	56	1.18
BR-5	106.7	1.64	640	58	0
BD3-6	101.8	1.64	611	64	0
2BR-7	131	1.65	786	60	0
BD2-8	103.3	1.64	620	66	0.45
SS-09	182	1.66	1092	68	0
FB-10	36.4	1.61	218	71	0
GS-11	27.6	1.61	166	65	1.7
BD1-12	17.38	1.59	102	67	1.7
KMR-13	67.97	1.63	408	68	1.7
BC-14	108.44	1.64	648	67	1.7
BR-15	44.61	1.63	378	65	1.7
CC-16	79	1.64	474	56	0
USS-18A	106	1.64	636	56	0
2BR-19	90	1.64	540	56	0
MUPC	3.87	1.63	23	63.9	27.8
WLCHL	14.71	1.63	88	63.7	2.5

Table 2.4.3-206 (Sheet 2 of 2)
Broad River Watershed Subbasin Input Parameters

Notes:

Reference Figure 2.4.3-203 for subbasin locations
LS-1, Lake Summit/Tuxedo Hydro
LA-2, Lake Adger/Turner Shoals
GD-3, Green River (Turner Shoals to Broad R.)
LL-4, Lake Lure/hydro
BR-5, Broad River (Lake Lure to Green R.)
BD3-6, Broad River (Green R. to Second Broad R.)
2BR-7, Second Broad River
BD2-8, Broad River (Second Broad R. to First Broad R.)
SS-09, Stice Shoals
FB-10, First Broad River (Stice Shoals to Broad R.)
GS-11, Broad River (First Broad to Gaston Shoals)
BD1-12, Broad River (Gaston Shoals to Buffalo Creek)
KMR-13, Kings Mountain Reservoir (Buffalo Cr.)
BC-14, Buffalo Creek (Kings Mountain Reservoir to Broad R.)
BR-15, Broad River (Buffalo Cr. to Ninety-Nine Islands)
CC-16, Cove Creek (Broad R. near Lake Lure)
USS-18A, Upper First Broad River
2BR-19, Upper Second Broad River
MUPC, Make-Up Pond C
WLCHL, Lake Whelchel

Table 2.4.3-207
Make-Up Pond C Subbasin Unit Hydrograph

Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)
10	124	150	333	290	21
20	380	160	273	300	17
30	843	170	227	310	14
40	1472	180	187	320	12
50	1641	190	154	330	10
60	1590	200	125	340	8
70	1410	210	103	350	7
80	1240	220	85	360	6
90	1083	230	69	370	4
100	942	240	57	380	3
110	794	250	46	390	2
120	660	260	38	400	1
130	533	270	31	410	0
140	419	280	26	420	0

Notes:
Reference Figure 2.4.3-203 for subbasin locations
MUPC, Make-Up Pond C

Table 2.4.3-208
Make-Up Pond B Subbasin Unit Hydrograph

Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)
10	71.40	150	185.95	290	10.68
20	219.10	160	151.78	300	8.75
30	486.11	170	126.44	310	7.03
40	814.45	180	103.97	320	5.88
50	935.26	190	85.35	330	4.90
60	915.00	200	69.31	340	4.21
70	820.00	210	56.89	350	3.52
80	715.00	220	46.90	360	2.36
90	616.17	230	37.97	370	1.82
100	533.18	240	31.14	380	1.34
110	448.23	250	23.48	390	0.86
120	370.44	260	19.19	400	0.38
130	296.71	270	15.91	410	0.00
140	234.48	280	12.97		

Table 2.4.3-209
Upper Arm Subbasin Unit Hydrograph

Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)	Time (min.)	Discharge (cfs)
2	36.65	32	120.53	62	7.39
4	115.29	34	99.59	64	6.13
6	221.30	36	83.78	66	5.00
8	368.06	38	69.99	68	4.22
10	555.70	40	58.29	70	3.52
12	588.82	42	47.42	72	3.08
14	570.00	44	39.87	74	2.62
16	520.00	46	33.02	76	2.16
18	456.33	48	27.36	78	1.71
20	395.86	50	22.66	80	1.32
22	334.32	52	18.49	82	0.94
24	277.50	54	15.53	84	0.57
26	228.85	56	12.82	86	0.19
28	183.74	58	10.74	88	0.00
30	147.85	60	8.90		

Table 2.4.4-201
Peak Flows and Resulting Water Surface Elevations

Event	Model	Peak Flow (cfs)	Lee Nuclear Station Water Surface Elevation (ft.)	Ninety-Nine Islands Dam Water Surface Elevation (ft.)
PMF (no breach)	HEC-HMS	802,000	(a)	542.78
PMF (no breach)	HEC-RAS (unsteady state)	823,000	551.49	546.06
PMF (no breach)	HEC-RAS (steady state)	823,000	552.61	546.06
Gaston Shoals Dam failure coincident with the PMF	HEC-RAS (unsteady state)	824,000	551.52	546.09
Gaston Shoals Dam and Cherokee Falls Dam failures coincident with the PMF	HEC-RAS (unsteady state)	824,000	551.52	546.09
Major upstream structures failures coincident with the PMF(b)	HEC-HMS	1,850,000	(a)	560.10
Major upstream structures failures coincident with the PMF(b)	HEC-RAS (steady state)	1,850,000	576.50	564.93

a) Not calculated. Resulting hydrographs or peak flow used as input to the HEC-RAS model to determine the water surface elevations at the Lee Nuclear Station.

b) Upstream failures include overtopping failure of Lake Lure Dam, Tuxedo Dam, Turner Shoals Dam, Kings Mountain Reservoir Dam, Lake Whelchel, Lake Cherokee, and Make-Up Pond C. All failures occur simultaneously with a failure time near to the peak PMF outflow at Ninety-Nine Islands Dam.

Table 2.4.7-201 (Sheet 1 of 2)
Water Temperature Data for the Broad River Near Gaffney, South Carolina
(USGS Station 02153500)

SAMPLE DATE	°F
8/26/1969	75.0
9/24/1969	68.7
10/22/1969	65.8
11/17/1969	44.6
12/15/1969	44.1
1/11/1970	43.7
1/20/1970	42.6
2/19/1970	52.2
3/20/1970	53.4
4/27/1970	65.1
5/21/1970	76.1
6/16/1970	77.4
7/7/1970	83.1
8/18/1970	78.4
9/15/1970	80.6
10/15/1970	73.0
11/20/1970	52.7
12/21/1970	48.2
1/11/1971	43.7
2/22/1971	54.5
3/23/1971	53.6
4/19/1971	66.2
5/10/1971	69.8
6/14/1971	77.9
7/8/1971	76.1
8/24/1971	78.8
9/13/1971	76.1
10/4/1971	75.2

Table 2.4.7-201 (Sheet 2 of 2)
Water Temperature Data for the Broad River Near Gaffney, South Carolina (USGS Station 02153500)

SAMPLE DATE	°F
11/22/1971	45.5
12/9/1971	49.1
1/19/1972	42.8
2/9/1972	41.9
3/24/1972	50
4/20/1972	68.9
5/22/1972	67.1
6/13/1972	72.5
7/25/1972	83.3
8/24/1972	79.7
10/2/1972	64.4
10/25/1972	62.6
11/16/1972	51.8
12/29/1972	46.4
1/23/1973	50.9
2/8/1973	48.2
3/20/1973	53.6
4/25/1973	64.4
5/30/1973	69.8
6/21/1973	71.6
Min T, 2/9/1972	41.9
Max T, 7/25/1972	83.3

(Reference 290)

Table 2.4.11-201
Minimum Daily Streamflow Observed on the Broad River
Below Ninety-Nine Islands Dam, South Carolina, (USGS Station 02153551)
1998-2006

Climatic Year[a]	Date	Minimum Flow, cfs
1998[b]	11/2/1998	805
1999	9/18/1999	233
2000	9/16/2000	342
2001	9/14/2001	224
2002	9/14/2002	138
2003	9/21/2003	1,230
2004	8/18/2004	605
2005	11/7/2005	851
2006[b]	7/12/2006	534

a) Climatic Year – April 1 to March 31

b) Year 1998 incomplete, available data 10/30/1998 – 3/31/1999
Year 2006 incomplete, available data 4/1/2006 – 9/30/2006

(Reference 290)

Table 2.4.11-202 (Sheet 1 of 2)
Minimum Daily Streamflow Observed on the Broad River Near Gaffney, South Carolina, (USGS Station 02153500) 1938-1990

Climatic Year[a]	Date	Minimum Flow, cfs
1938[b]	12/24/1938	985
1939	10/22/1939	586
1940	7/2/1940	443
1941	10/14/1941	466
1942	7/21/1942	659
1943	9/27/1943	699
1944	9/10/1944	730
1945	9/3/1945	743
1946	10/7/1946	811
1947	9/22/1947	657
1948	7/6/1948	845
1949	7/5/1949	1,260
1950	10/16/1950	991
1951	10/21/1951	598
1952	7/29/1952	746
1953	8/30/1953	466
1954	10/24/1954	224
1955	9/20/1955	444
1956	9/3/1956	300
1957	9/8/1957	381
1958	12/7/1958	867
1959	8/24/1959	986
1960	9/26/1960	1,050
1961	10/10/1961	908
1962	9/3/1962	947
1963	8/19/1963	651
1964	9/28/1964	942

Table 2.4.11-202 (Sheet 2 of 2)
Minimum Daily Streamflow Observed on the Broad River Near Gaffney, South Carolina, (USGS Station 02153500) 1938-1990

Climatic Year[a]	Date	Minimum Flow, cfs
1965	9/19/1965	916
1966	9/11/1966	682
1967	8/20/1967	874
1968	9/4/1968	468
1969	7/21/1969	1,140
1970	7/21/1970	836
1971[b]	7/19/1971 & 9/16/1971	1,270
1986[b]	7/15/1986	261
1987	10/11/1987	560
1988	7/29/1988	300
1989	8/11/1989	656
1990[b]	9/28/1990	1,030

a) Climatic Year – April 1 to March 31

b) Year 1938 incomplete, available data 12/1/1938 - 3/31/1939

Year 1971 incomplete, available data 4/1/1971 - 9/30/1971

No data available from 9/30/1971 - 6/9/1986

Year 1986 incomplete, available data 6/9/1986 - 3/31/1987

Year 1990 incomplete, available data 4/1/1990 - 9/30/1990

(Reference 280)

Table 2.4.11-203
100-Yr. Return Period Low Flow Rates[a]

	Duration, days		
	1	7	30
Flow Rate, cfs	172	269	346

a) Low flow based on statistical analysis of combined data for USGS gauges on the Broad River near Gaffney, South Carolina (USGS No. 02153500 climatic years from 1938 to 1990) and below Ninety-Nine Islands Dam (USGS No. 02153551 climatic years from 1998 to 2002).

Table 2.4.12-201 (Sheet 1 of 4)
Well Construction and Water Table Elevations (ft above msl)

	Reference Elevations		Well Construction Depths							Additional Info	
Well I.D.	GL Elev	TOC Elev	Boring Depth	TD from TOC	B/Screen	T/Screen	T/Sand	T/Seal	Material	DTW WD	Date Plugged
MW-1200	591.93	593.99	41	41.93	40	25	23	20	2-inch PVC Sch40	23.0	NA
MW-1201	589.91	592.12	102.5	103.81	101.5	86.5	84.5	82.5	2-inch PVC Sch40	37.0	NA
MW-1201A	590.07	592.11	48	49.78	47	37	36	34	2-inch PVC Sch40	37.0	NA
MW-1202	587.47	589.68	78.5	79.82	77.5	62.5	58	55	2-inch PVC Sch40	20.6	NA
MW-1203	589.51	591.87	77	77.67	75	60	58	55	2-inch PVC Sch40	22.5	NA
MW-1204	609.92	612.42	115	116.59	114	99	97	95	2-inch PVC Sch40	37.1	NA
MW-1204A	609.93	612.42	50	51.82	49	39	37	35	2-inch PVC Sch40	37.1	NA
MW-1205	609.99	612.59	124	125.33	123	108	106	104	2-inch PVC Sch40	43.9	NA
MW-1206	589.66	591.51	68.5	69.89	67.5	52.5	50	47.5	2-inch PVC Sch40	31.7	NA
MW-1206A	589.75	591.43	43	44.09	42	32	31	29	2-inch PVC Sch40	31.7	NA
MW-1207	589.03	591.39	108	110.02	107	92	90	88	2-inch PVC Sch40	29.2	NA
MW-1207A	588.91	591.05	43	44.68	42	32	31	29	2-inch PVC Sch40	29.2	NA
MW-1208	587.77	590.00	79	78.92	76.5	61.5	59	56	2-inch PVC Sch40	47.0	NA
MW-1209	586.91	588.91	106	106.28	104	89	87	84.6	2-inch PVC Sch40	16.3	NA
MW-1209A	586.93	589.03	28	29.45	27	17	16	14	2-inch PVC Sch40	16.3	NA
MW-1210	589.78	592.27	101.5	103.10	101.5	86.5	84.5	82.5	2-inch PVC Sch40	16.5	NA
MW-1210A	589.42	591.66	30	32.06	29	19	18	16	2-inch PVC Sch40	16.5	NA
MW-1211	589.88	591.63	39	39.94	37.5	22.5	20.5	18	2-inch PVC Sch40	21.5	NA
MW-1212	610.24	612.29	47.5	48.88	46.5	31.5	29.5	26.5	2-inch PVC Sch40	31.0	NA
MW-1213	NA	NA	78.30	NA	NA	NA	NA	NA	NA	18.0	4/11/06
MW-1214	605.00	606.51	44.5	44.74	43	28	26	23	2-inch PVC Sch40	14.0	NA

Table 2.4.12-201 (Sheet 2 of 4)
Well Construction and Water Table Elevations (ft above msl)

	Reference Elevations		Well Construction Depths							Additional Info	
Well I.D.	GL Elev	TOC Elev	Boring Depth	TD from TOC	B/Screen	T/Screen	T/Sand	T/Seal	Material	DTW WD	Date Plugged
MW-1215	590.22	592.13	101.5	101.20	100	40	38	35.5	6-inch PVC	35.0	NA
MW-1216	588.01	590.69	29.0	31.31	28.0	18	17	15	2-inch PVC Sch40	18.0	NA
MW-1217	587.64	590.10	24.0	24.85	22.3	12	11	9	2-inch PVC Sch40	10.5	NA
MW-1218	588.12	590.18	16.0	18.31	15.0	5	4	2	2-inch PVC Sch40	17.5	NA
DW2	588.94	589.67	NIA	~150	NIA	NIA	NIA	NIA	6-inch Metal	NIA	NA
DW3	590.56	591.34	NIA	~107.5	NIA	NIA	NIA	NIA	6-inch PVC	NIA	NA
DW4	591.22	591.51	NIA	~130	NIA	NIA	NIA	NIA	6-inch PVC	NIA	NA
DW5	587.73	589.20	NIA	>201	NIA	NIA	NIA	NIA	6-inch Metal	NIA	NA

Table 2.4.12-201 (Sheet 3 of 4)
Well Construction and Water Table Elevations (ft above msl)

	Location Information				Reference Elevations		Well Construction Elevations				
Well I.D.	Latitude	Longitude	Northing	Easting	GL Elev	TOC Elev	T/Sand Elev.	T/Screen Elev.	B/Screen Elev.	Boring Depth Elev.	Date Completed
MW-1200	35.03776	-81.51582	1166348.442	1845571.069	591.93	593.99	568.93	566.93	551.93	550.93	4/10/06
MW-1201	35.03872	-81.51247	1166689.304	1846578.824	589.91	592.12	505.41	503.41	488.41	487.41	4/14/06
MW-1201A	NM	NM	1166693.529	1846576.539	590.07	592.11	554.07	553.07	543.07	542.07	7/18/06
MW-1202	35.03962	-81.50948	1167018.978	1847472.030	587.47	589.68	529.47	524.97	509.97	508.97	4/14/06
MW-1203	35.03874	-81.50824	1166702.120	1847838.422	589.51	591.87	531.51	529.51	514.51	512.51	4/11/06
MW-1204	35.03719	-81.50761	1166141.154	1848033.400	609.92	612.42	512.92	510.92	495.92	494.92	4/14/06
MW-1204A	NM	NM	1166133.724	1848034.258	609.93	612.42	572.93	570.93	560.93	559.93	7/17/06
MW-1205	35.03582	-81.50665	1165631.431	1848304.849	609.99	612.59	503.99	501.99	486.99	485.99	4/15/06
MW-1206	35.03862	-81.50948	1166655.908	1846689.086	589.66	591.51	539.66	537.16	522.16	521.16	4/18/06
MW-1206A	NM	-81.50948	1166656.288	1846693.299	589.75	591.43	558.75	557.75	547.75	546.75	7/17/06
MW-1207	35.03912	-81.51216	1166849.173	1846668.764	589.03	591.39	499.03	497.03	482.03	481.03	4/24/06
MW-1207A	NM	NM	1166846.232	1846673.410	588.91	591.05	557.91	556.91	546.91	545.91	7/18/06
MW-1208	35.04006	-81.51243	1167188.532	1846583.513	587.77	590.00	528.77	526.27	511.27	508.77	4/13/06
MW-1209	35.03431	-81.50742	1165084.761	1848071.547	586.91	588.91	499.91	497.91	482.91	480.91	4/18/06
MW-1209A	NM	NM	1165076.658	1848072.885	586.93	589.03	570.93	569.93	559.93	558.93	7/17/06
MW-1210	35.03496	-81.50956	1165321.305	1847439.208	589.78	592.27	505.28	503.28	488.28	488.28	4/16/06
MW-1210A	NM	NM	1165312.832	1847436.803	589.42	591.66	571.42	570.42	560.42	559.42	7/17/06
MW-1211	35.03460	-81.51307	1165197.583	1846406.261	589.88	591.63	569.38	567.38	552.38	550.88	4/11/06
MW-1212	35.03508	-81.51621	1165365.927	1845452.195	610.24	612.29	580.74	578.74	563.74	562.74	4/10/06
MW-1213	35.03876	-81.51229	NM	NM	NA	NA	NA	NA	NA	NA	NA
MW-1214	35.03181	-81.51050	1164177.882	1847153.830	605.00	606.51	579.00	577.00	562.00	560.50	4/11/06

Table 2.4.12-201 (Sheet 4 of 4)
Well Construction and Water Table Elevations (ft above msl)

	Location Information				Reference Elevations		Well Construction Elevations				
Well I.D.	Latitude	Longitude	Northing	Easting	GL Elev	TOC Elev	T/Sand Elev.	T/Screen Elev.	B/Screen Elev.	Boring Depth Elev.	Date Completed
MW-1215	35.03876	-81.51230	1166710.545	1846624.819	590.22	592.13	552.22	550.22	490.22	488.72	4/17/06
MW-1216	35.03452	-81.51129	1165171.882	1846927.273	588.01	590.69	571.01	570.01	560.01	559.01	7/19/06
MW-1217	35.03419	-81.51109	1165042.463	1846983.878	587.64	590.10	574.64	573.64	563.64	563.64	7/19/06
MW-1218	35.03368	-81.51059	1164859.672	1847139.635	588.12	590.18	584.12	583.12	573.12	572.12	7/18/06
DW2	35.03489	-81.51162	1165319.974	1846821.466	588.94	589.67	NIA	NIA	NIA	NIA	~1977
DW3	35.03521	-81.51028	1165408.943	1847234.503	590.56	591.34	NIA	NIA	NIA	NIA	~1977
DW4	35.03412	-81.51358	1165035.485	1846277.086	591.22	591.51	NIA	NIA	NIA	NIA	~1977
DW5	NM	NM	1167933.393	1847896.940	587.73	589.20	NIA	NIA	NIA	NIA	~1977

TOC Elev. = top of casing elevations obtained from professional surveyors (McKim & Creed)

GL Elev. = ground level elevations obtained from professional surveyors (McKim & Creed)

Latitude, Longitude: Obtained using hand-held Garmin Rino 120 GPS unit

Northing/Easting: Obtained from professional surveyors (McKim & Creed)

Wells designated "A" wells are the shallow cluster wells located around 5 feet from the cluster twin well.

Location 1213 was completed as a boring only. MW-1215 is the aquifer test pumping well.

Units are ft.

DTW WD = Depth to water while drillng

NIA = No Information Available

NA = Not Applicable

NM = Not Measured

DW Wells completed during Cherokee activities, records not available, possibly used for dewatering

Table 2.4.12-202 (Sheet 1 of 8)
Water Table Elevations

	Reference Elev.		4/18/2006		5/14/2006		5/23/2006		5/29/2006		6/6/2006	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1200	593.99	591.93	31.80	562.19	32.51	561.48	32.77	561.2	32.90	561.1	33.13	560.9
MW-1201	592.12	589.91			34.69	557.43	35.17	557.0	35.35	556.8	35.60	556.5
MW-1201A	592.11	590.07										
MW-1202	589.68	587.47	23.90	565.78	24.49	565.19	24.76	564.9	24.86	564.8	24.99	564.7
MW-1203	591.87	589.51	20.60	571.27	21.05	570.82	21.40	570.5	21.51	570.4	21.65	570.2
MW-1204	612.42	609.92	39.80	572.62	39.87	572.55	40.25	572.2	40.33	572.1	40.36	572.1
MW-1204A	612.42	609.93										
MW-1205	612.59	609.99	46.90	565.69	46.89	565.70	47.28	565.3	47.33	565.3	47.20	565.4
MW-1206	591.51	589.66			32.98	558.53	33.43	558.1	33.63	557.9	33.89	557.6
MW-1206A	591.43	589.75										
MW-1207	591.39	589.03			33.31	558.08	33.74	557.6	33.93	557.5	34.17	557.2
MW-1207A	591.05	588.91										
MW-1208	590.00	587.77	41.30	548.70	41.84	548.16	42.25	547.8	42.37	547.6	42.46	547.5
MW-1209	588.91	586.91			19.21	569.70	19.55	569.4	19.62	569.3	19.57	569.3
MW-1209A	589.03	586.93										
MW-1210	592.27	589.78	19.50	572.77	20.08	572.19	20.17	572.1	20.51	571.8	20.64	571.6
MW-1210A	591.66	589.42										
MW-1211	591.63	589.88	27.50	564.13	27.93	563.70	27.99	563.6	28.11	563.5	28.21	563.4
MW-1212	612.29	610.24	35.45	576.84	36.26	576.03	36.62	575.7	36.81	575.5	37.17	575.1
MW-1214	606.51	605.00	16.80	589.71	17.60	588.91	18.01	588.5	18.25	588.3	18.61	587.9
MW-1215	592.13	590.22			34.65	557.48	35.14	557.0	35.34	556.8	35.56	556.6
MW-1216	590.69	588.01										

Table 2.4.12-202 (Sheet 2 of 8)
Water Table Elevations

	Reference Elev.		4/18/2006		5/14/2006		5/23/2006		5/29/2006		6/6/2006	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1217	590.10	587.64										
MW-1218	590.18	588.12										
DW2	589.67	588.94	33.80	555.87	37.11	552.56	37.11	552.56	37.56	552.11	37.75	551.92
DW3	591.34	590.56	22.50	568.84	23.59	567.75	23.59	567.75	24.65	566.69	24.63	566.71
DW4	591.51	591.22										
DW5	589.20	587.73										
SG-1		568.23			0.98	569.21						
SG-2		547.81			1.40	546.41						
SG-3		536.09			2.40	533.69						
SG-4		525.64			1.40	524.24						

Table 2.4.12-202 (Sheet 3 of 8)
Water Table Elevations

	Reference Elev.		6/12/2006		7/15/2006		7/21/2006		8/15/2006		9/11/2006	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1200	593.99	591.93	33.29	560.7	34.13	559.9	34.31	559.68	34.95	559.0	36.64	557.3
MW-1201	592.12	589.91	35.80	556.3	36.80	555.3	36.97	555.15	37.55	554.6	38.19	553.9
MW-1201A	592.11	590.07					38.60	553.51	36.69	555.4	37.10	555.0
MW-1202	589.68	587.47	25.10	564.6	25.73	563.9	25.82	563.86	26.28	563.4	26.81	562.9
MW-1203	591.87	589.51	21.78	570.1	22.51	569.4	22.65	569.22	23.14	568.7	24.70	567.2
MW-1204	612.42	609.92	40.45	572.0	41.06	571.4	41.17	571.25	41.58	570.8	42.14	570.3
MW-1204A	612.42	609.93					33.54	578.88	33.06	579.4	33.44	579.0
MW-1205	612.59	609.99	47.25	565.3	47.66	564.9	47.75	564.84	47.98	564.6	48.50	564.1
MW-1206	591.51	589.66	34.10	557.4	35.10	556.4	35.29	556.22	35.89	555.6	36.51	555.0
MW-1206A	591.43	589.75					35.31	556.12	35.92	555.5	36.54	554.9
MW-1207	591.39	589.03	34.39	557.0	35.39	556.0	35.54	555.85	36.21	555.2	36.84	554.5
MW-1207A	591.05	588.91					34.77	556.28	35.39	555.7	36.03	555.0
MW-1208	590.00	587.77	42.62	547.4	43.18	546.8	43.38	546.62	43.69	546.3	44.20	545.8
MW-1209	588.91	586.91	19.62	569.3	20.10	568.8	20.20	568.71	20.51	568.4		
MW-1209A	589.03	586.93					17.72	571.31	17.78	571.3	18.27	570.8
MW-1210	592.27	589.78	20.95	571.3	21.67	570.6	21.91	570.36	22.26	570.0	22.61	569.7
MW-1210A	591.66	589.42					20.42	571.24	20.81	570.8	21.25	570.4
MW-1211	591.63	589.88	28.33	563.3	28.62	563.0	28.80	562.83	28.85	562.8	28.73	562.9
MW-1212	612.29	610.24	37.42	574.9	38.69	573.6	38.90	573.39	39.62	572.7	40.14	572.2
MW-1214	606.51	605.00	18.91	587.6	20.31	586.2	20.62	585.89	21.38	585.1	22.04	584.5
MW-1215	592.13	590.22	35.75	556.4	36.73	555.4	36.91	555.22	37.50	554.6	38.15	554.0
MW-1216	590.69	588.01					25.00	565.69	25.96	564.7	26.92	563.8
MW-1217	590.10	587.64					22.19	567.91	23.33	566.8	24.41	565.7

Table 2.4.12-202 (Sheet 4 of 8)
Water Table Elevations

	Reference Elev.		6/12/2006		7/15/2006		7/21/2006		8/15/2006		9/11/2006	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1218	590.18	588.12					16.63	573.55	17.22	573.0	17.77	572.4
DW2	589.67	588.94	37.73	551.94	38.90	550.77	39.41	550.26	40.03	549.64	40.42	549.25
DW3	591.34	590.56	25.24	566.10	26.24	565.10	26.88	564.46	26.90	564.44	27.33	564.01
DW4	591.51	591.22			23.82	567.69	23.91	567.60	23.94	567.57		
DW5	589.20	587.73							58.35	530.85	58.72	
SG-1		568.23			0.84	569.07					1.02	569.25
SG-2		547.81			1.70	546.11					1.6	546.21
SG-3		536.09			2.48	533.61					1.7	534.39
SG-4		525.64			1.20	524.44					1.38	524.26

Table 2.4.12-202 (Sheet 5 of 8)
Water Table Elevations

Location	Reference Elev.		9/14/2006		10/10/2006		11/14/2006		12/20/2006		1/17/2006	
	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1200	593.99	591.93	35.67	558.3	35.99	558.00	36.44	557.55	35.03	558.96	32.20	561.79
MW-1201	592.12	589.91			38.88	553.24	39.44	552.68	40.35	551.77	40.74	551.38
MW-1201A	592.11	590.07			38.12	553.99	37.90	554.21	39.04	553.07	39.64	552.47
MW-1202	589.68	587.47	26.82	562.9	27.19	562.49	27.67	562.01	28.02	561.66	28.06	561.62
MW-1203	591.87	589.51	23.64	568.2	23.93	567.94	24.17	567.70	23.97	567.90	23.59	568.28
MW-1204	612.42	609.92	41.95	570.5	42.37	570.05	42.68	569.74	42.95	569.47	42.81	569.61
MW-1204A	612.42	609.93	33.17	579.2	33.58	578.84	33.71	578.71	34.75	577.67	35.16	577.26
MW-1205	612.59	609.99	48.23	564.4	48.61	563.98	48.76	563.83	49.20	563.39	49.22	563.37
MW-1206	591.51	589.66			37.27	554.24	37.83	553.68	38.60	552.91	38.96	552.55
MW-1206A	591.43	589.75			37.31	554.12	37.85	553.58	38.62	552.81	38.98	552.45
MW-1207	591.39	589.03			36.88	554.51	38.16	553.23	38.90	552.49	39.25	552.14
MW-1207A	591.05	588.91			37.64	553.41	37.38	553.67	38.10	552.95	38.44	552.61
MW-1208	590.00	587.77			44.73	545.27	45.02	544.98	45.73	544.27	45.89	544.11
MW-1209	588.91	586.91	20.85	568.1	21.22	567.69	21.44	567.47	21.75	567.16	21.67	567.24
MW-1209A	589.03	586.93	18.01	571.0	18.46	570.57	18.80	570.23	20.02	569.01	20.21	568.82
MW-1210	592.27	589.78	22.18	570.1	23.06	569.21	22.54	569.73	22.67	569.60	21.66	570.61
MW-1210A	591.66	589.42	21.11	570.5	21.64	570.02	21.49	570.17	21.55	570.11	20.74	570.92
MW-1211	591.63	589.88	28.12	563.5	28.70	562.93	28.21	563.42	27.86	563.77	26.83	564.80
MW-1212	612.29	610.24	40.15	572.1	40.25	572.04	40.03	572.26	37.78	574.51	33.44	578.85
MW-1214	606.51	605.00	22.02	584.5	22.40	584.11	22.35	584.16	21.05	585.46	20.01	586.50
MW-1215	592.13	590.22			38.89	553.24	39.43	552.70	40.28	551.85	40.65	551.48
MW-1216	590.69	588.01	26.91	563.8	27.49	563.20	27.89	562.80	26.92	563.77	25.75	564.94
MW-1217	590.10	587.64	24.33	565.8	24.47	565.63	24.49	565.61	24.14	565.96	22.47	567.63

Table 2.4.12-202 (Sheet 6 of 8)
Water Table Elevations

	Reference Elev.		9/14/2006		10/10/2006		11/14/2006		12/20/2006		1/17/2006	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1218	590.18	588.12	8.60	581.6	17.88	572.30	17.77	572.41	16.63	573.55	15.10	575.08
DW2	589.67	588.94	40.12	549.55	40.64	549.03	40.44	549.23	40.11	549.56	38.99	550.68
DW3	591.34	590.56	25.92	565.42	27.88	563.46	26.50	564.84	26.54	564.80	24.57	566.77
DW4	591.51	591.22	23.32	568.19	23.88	567.63	23.51	568.00	23.05	568.46	21.93	569.58
DW5	589.20	587.73	58.62	530.58	58.84	530.36	58.92	530.28	59.12	530.08	59.08	530.12
SG-1		568.23			0.68	568.91	0.97	569.20	0.95	569.18	1.00	569.23
SG-2		547.81			1.95	545.86	1.87	545.94	1.47	546.34	1.25	546.56
SG-3		536.09			2.34	533.75	1.74	534.35	1.37	534.73	1.78	534.31
SG-4		525.64			1.47	524.17	1.38	524.26	0.00	525.64	1.38	524.27

Table 2.4.12-202 (Sheet 7 of 8)
Water Table Elevations

	Reference Elev.		2/19/07		3/13/07		4/19/07	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
MW-1200	593.99	591.93	32.00	561.99	28.88	565.11	31.26	562.73
MW-1201	592.12	589.91	40.91	551.21	41.14	550.98	41.46	550.66
MW-1201A	592.11	590.07	39.69	552.42	40.04	552.07	40.36	551.75
MW-1202	589.68	587.47	27.82	561.86	27.80	561.88	28.00	561.68
MW-1203	591.87	589.51	23.00	568.87	22.79	569.08	23.20	568.67
MW-1204	612.42	609.92	42.14	570.28	41.85	570.57	41.96	570.46
MW-1204A	612.42	609.93	34.71	577.71	35.06	577.36	35.00	577.42
MW-1205	612.59	609.99	48.59	564.00	48.56	564.03	48.39	564.20
MW-1206	591.51	589.66	39.22	552.29	39.46	552.05	39.82	551.69
MW-1206A	591.43	589.75	39.25	552.18	39.50	551.93	39.85	551.58
MW-1207	591.39	589.03	39.50	551.89	39.72	551.67	40.08	551.31
MW-1207A	591.05	588.91	38.71	552.34	38.92	552.13	39.29	551.76
MW-1208	590.00	587.77	45.77	544.23	45.89	544.11	45.92	544.08
MW-1209	588.91	586.91	20.92	567.99	20.79	568.12	20.61	568.30
MW-1209A	589.03	586.93	18.72	570.31	18.70	570.33	18.15	570.88
MW-1210	592.27	589.78	21.33	570.94	20.85	571.42	20.94	571.33
MW-1210A	591.66	589.42	20.24	571.42	19.83	571.83	19.93	571.73
MW-1211	591.63	589.88	27.06	564.57	26.53	565.10	26.83	564.80
MW-1212	612.29	610.24	34.08	578.21	31.21	581.08	33.91	578.38
MW-1214	606.51	605.00	18.68	587.83	17.72	588.79	17.32	589.19
MW-1215	592.13	590.22	40.84	551.29	41.06	551.07	41.40	550.73
MW-1216	590.69	588.01	24.66	566.03	23.91	566.78	24.24	566.45
MW-1217	590.10	587.64	21.46	568.64	20.33	569.77	20.97	569.13
MW-1218	590.18	588.12	14.76	575.42	13.69	576.49	14.19	575.99

Table 2.4.12-202 (Sheet 8 of 8)
Water Table Elevations

	Reference Elev.		2/19/07		3/13/07		4/19/07	
Location	TOC	GL	DTW	WT Elev	DTW	WT Elev	DTW	WT Elev
DW2	589.67	588.94	38.94	550.73	37.62	552.05	38.17	551.50
DW3	591.34	590.56	24.77	566.57	23.14	568.20	23.26	568.08
DW4	591.51	591.22	22.66	568.85	21.72	569.79	18.19	573.32
DW5	589.20	587.73	58.95	530.25	58.65	530.55	58.49	530.71
SG-1		568.23	0.98	569.21	1.00	569.23	1.17	569.40
SG-2		547.81	1.23	546.58	1.23	546.58	1.06	546.75
SG-3		536.09	1.86	534.23	1.81	534.28	1.70	534.39
SG-4		525.64	1.38	524.27	1.50	524.14	1.34	524.30

TOC = top of casing elevation
GL = ground level elevation
SG-1 = DTW value is height above reference elevation
All values expressed as feet above msl, except DTW, expressed in feet.

DTW = depth to water
WT Elev = water table elevation
BLANK - no data

Table 2.4.12-203
Deleted

Table 2.4.12-204
Aquifer Characteristics

Material	Hydraulic Conductivity (K) Minimum	Median	Conservative Estimate	Maximum	Source
Saprolite/Soil K_v	2.45×10^{-8}	2.10×10^{-6}	4.4×10^{-5}	2.55×10^{-4}	1973 investigation laboratory analyses.
Saprolite/Soil K_h	9.67×10^{-7}	1.14×10^{-4}	4.5×10^{-4}	2.26×10^{-3}	1973 investigation field tests and 2006 slug tests.
Bedrock - PWR K_h	9.67×10^{-7}	1.53×10^{-4}	1.4×10^{-3}	9.89×10^{-3}	1973 investigation packer tests and 2006 slug, aquifer pumping, and packer tests.
Undifferentiated Material	2.21×10^{-4}	4.0×10^{-4}	1.5×10^{-3}	3.90×10^{-3}	1977 aquifer pumping tests.
Fill Material	1.81×10^{-5}	5.39×10^{-5}	7.0×10^{-5}	7.44×10^{-5}	2006 slug tests.

Units are in centimeters per second (cm/sec).
PWR - Partially weathered rock.
K_v - Vertical hydraulic conductivity.
K_h - Horizontal hydraulic conductivity.

Conservative Estimate - The geometric mean of samples exceeding the median (applicable to Saprolite/Soil K_v, K_h and Fill Material).
Conservative Estimate for Bedrock K_h was obtained from results of 2006 pumping test and was used to calculate the groundwater velocity. The Bedrock K_h of 1.4E-03 cm/s bounded the geometric mean of samples exceeding the median (i.e., 1E-03 cm/s).
Undifferentiated Material -Identification used for 1977 data where well screens bracketed the entire saturated zone, and did not differentiate between the fill material, soil, saprolite, or partially weathered rock. Conservative estimate of Undifferentiated Material K_h is presented for comparison purposes only and is based on an average of results from 1977 pumping tests.

Table 2.4.12-205 (Sheet 1 of 3)
Maximum Historically-Recorded Rainfall Distribution
(Tropical Storm Jerry)

Date	Time of Day (hr. : min.)	Rainfall (in.)	Cumulative Rainfall Duration (hr.)
25-Aug-95	1:00	0.00	
	2:00	0.00	
	3:00	0.00	
	4:00	0.00	
	5:00	0.00	
	6:00	0.00	
	7:00	0.00	
	8:00	0.00	
	9:00	0.00	
	10:00	0.00	
	11:00	0.00	
	12:00	0.00	
	13:00	0.00	
	14:00	0.00	
	15:00	0.00	
	16:00	0.00	
	17:00	0.30	1
	18:00	0.10	2
	19:00	0.00	3
	20:00	0.00	4
	21:00	0.00	5
	22:00	0.00	6
	23:00	0.00	7
	0:00	0.00	8

Table 2.4.12-205 (Sheet 2 of 3)
Maximum Historically-Recorded Rainfall Distribution
(Tropical Storm Jerry)

Date	Time of Day (hr. : min.)	Rainfall (in.)	Cumulative Rainfall Duration (hr.)
	1:00	0.00	9
	2:00	0.01	10
	3:00	0.32	11
	4:00	0.10	12
	5:00	0.11	13
	6:00	0.10	14
	7:00	0.14	15
	8:00	0.14	16
	9:00	0.11	17
	10:00	0.11	18
	11:00	0.14	19
	12:00	0.11	20
26-Aug-95	13:00	0.26	21
	14:00	0.10[a]	22
	15:00	0.30[a]	23
	16:00	0.11[a]	24
	17:00	0.33[a]	25
	18:00	0.23[a]	26
	19:00	0.70[a]	27
	20:00	0.81[a]	28
	21:00	0.54[a]	29
	22:00	0.42[a]	30
	23:00	1.51[a]	31
	0:00	2.62[a]	32

Table 2.4.12-205 (Sheet 3 of 3)
Maximum Historically-Recorded Rainfall Distribution
(Tropical Storm Jerry)

Date	Time of Day (hr. : min.)	Rainfall (in.)	Cumulative Rainfall Duration (hr.)
27-Aug-95	1:00	1.74[a]	33
	2:00	1.20[a]	34
	3:00	0.17[a]	35
	4:00	0.04[a]	36
	5:00	0.06[a]	37
	6:00	0.06[a]	38
	7:00	0.03[a]	39
	8:00	0.02[a]	40
	9:00	0.01[a]	41
	10:00	0.10[a]	42
	11:00	0.18[a]	43
	12:00	0.57[a]	44
	13:00	0.47[a]	45
	14:00	0.07	46
	15:00	0.03	47
	16:00	0.00	
	17:00	0.00	
	18:00	0.00	
	19:00	0.00	
	20:00	0.00	
	21:00	0.00	
	22:00	0.00	
	23:00	0.00	
	0:00	0.00	
Maximum 24-hr Rainfall (in.)		12.32	
Total 47-hr Storm Rainfall (in.)		14.47	

Note:
Data collected at Greenville-Spartanburg Airport, Greer, South Carolina
GSP Station, Gage ID No. 383747 (Reference 305)

a) Rainfall measurements during the 24-hour maximum period.

Table 2.4.13-201
Distribution Coefficients (K_d)

	K_d Analytical Results per Argonne National Laboratory			Default K_d Values Used by RESRAD and Values From Other Sources[a]			
Sample Loc.	MW-1208	MW-1208	MW-1210	Sheppard & Thibault	IAEA	NUREG/ CR-5512 Kennedy & Strenge (1992)	RESRAD (v. 5.62 & later)
Sample Depth ft bgs[b]	45-46	58.5-59	69-73				
Sample Zone	Soil/Saprolite	Soil/Saprolite	Soil/Saprolite				
Soil Sample Texture	Sand, silty (SM)	Sand, silty (SM)	Silt, sandy (ML)	Loam	Loam	Sand	NIA[c]
Element	cm^3/g	cm^3/g	cm^3/g	cm^3/g	cm^3/g	cm^3/g	cm^3/g
Co	1103 ± 118	1971 ± 214	>7714	1300	1300	60	1000
Cs	3704 ± 524	2117 ± 299	1156 ± 163	4600	4400	270	1000
Fe	1689 ± 239	5478 ± 775	3628 ± 513	800	810	160	1000
I	1.4 ± 0.2	0.07 ± 0.01	2.5 ± 0.4	5	5	1	0.1
Ni	269 ± 38	167 ± 24	152 ± 22	300	300	400	1000
Pu-242	89 ± 13	>1921	987 ± 140	1200	1200	550	2000
Sr	739 ± 82	262 ± 33	73 ± 9	20	810	-	30
Tc-99	0.28 ± 0.04	0.04 ± 0.01	0.42 ± 0.06	0.1	-	0.1	0
U-235	>3159	1702 ± 241	>3636	15	-	15	50

a) References 209 and 210

b) Below ground surface

c) No information available

Table 2.4.13-202 (Sheet 1 of 2)
AP1000 Tanks Containing Radioactive Liquid

Tank	Location[(a)]	Nominal Tank Volume	Radioisotope Contents	Considerations/Features to Mitigate Release
PXS Tanks (IRWST and CMT's)	Inside Containment	NA	NA	Inside Containment; release need not be considered.
Spent Fuel Pool	Auxiliary Building	NA	NA	Not a tank, per se. Fully lined and safety related. Located entirely inside Auxiliary Building; does not have any potential for foundation cracks to allow leakage directly to environment. Leakage would be to another room of Auxiliary Building.
WLS Reactor coolant drain tank	Inside Containment	NA	NA	Inside containment; release need not be considered.
WLS Containment sump	Inside Containment	NA	NA	Inside containment; release need not be Considered.
WLS Effluent Holdup Tanks	Auxiliary building El. 66'-6"	28,000 gal	Essentially reactor coolant	Located in unlined room at lowest portion of Auxiliary Building
WLS Waste Holdup Tanks	Auxiliary Building El. 66'-6"	15,000 gal	Less than reactor coolant	Located in unlined room at lowest portion of Auxiliary Building
WLS Monitor Tanks A, B, C	Auxiliary Building El. 66'-6", 92'-6" and 107'- 2"	15,000 gal	Effluent prepared for environmental discharge - much less than reactor coolant	Located in unlined room at lowest portion of Auxiliary Building

Table 2.4.13-202 (Sheet 2 of 2)
AP1000 Tanks Containing Radioactive Liquid

Tank	Location[a]	Nominal Tank Volume	Radioisotope Contents	Considerations/Features to Mitigate Release
WLS Monitor Tanks D, E, F	Radwaste Building	15,000 gal	Effluent prepared for environmental discharge - much less than reactor coolant	Located in unlined room at grade level in curbed, non-seismic building
WLS Chemical Waste Tank	Auxiliary Building El. 66'-6"	8,900 gal	Less than reactor coolant	Located in unlined room at lowest portion of Auxiliary Building
WSS Spent Resin Storage Tanks	Auxiliary Building El. 100'[a]	300 ft^3 (liquid volume will be much less)	Approx. reactor coolant	Located entirely inside Auxiliary Building; does not have any potential for foundation cracks to allow leakage directly to environment. Leakage would be to another room of aux. building.

a) Floor elevations are based on design plant grade of 100 ft.

Table 2.4.13-203 (Sheet 1 of 5)
Listing of Lee Nuclear Station Data and Modeling Parameters Supporting the Effluent Holdup Tank Failure

Soil Parameter	Parameter Description	Parameter Value [(a)] [(b)]	Parameter Justification
Silver Transport K_d Coefficient (cm^3/g)[(b)]	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Barium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Bromine Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Cerium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Cobalt Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	985	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Chromium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Cesium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	993	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Iron Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	1,450	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Tritium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Iodine Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0.06	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Lanthanum Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Manganese Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Molybdenum Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.

Table 2.4.13-203 (Sheet 2 of 5)
Listing of Lee Nuclear Station Data and Modeling Parameters Supporting the Effluent Holdup Tank Failure

Soil Parameter	Parameter Description	Parameter Value [(a)] [(b)]	Parameter Justification
Niobium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Promethium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Rubidium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Rhodium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Ruthenium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Strontium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	64	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Technetium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0.03	Radionuclide-specific K_d values are measured by Argonne National Laboratory using Lee soil. Lowest value of the laboratory reporting range is used.
Tellurium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Yttrium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Zirconium Transport K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 assumes no retardation.
Precipitation (meters per year)	Average quantity of precipitation annually	1.27	Based on the 50 inches per year typical annual precipitation for Cherokee County.
Area of contaminated zone (square meters)	Area containing liquids released by the tank failure	~104	This is the area of a cube required to contain 80% of the effluent tank total capacity, distributed into that portion of the soil voids represented by the effective porosity (for PWR).
Thickness of contaminated zone (meters)	Describes the thickness of the area considered to be the contaminated zone	~10.2	The volume is assumed to be a cube. The area required to contain a volume with 80% of the liquid effluent tank (22,400 gallons), accounting for effective porosity of the contaminated zone.

Table 2.4.13-203 (Sheet 3 of 5)
Listing of Lee Nuclear Station Data and Modeling Parameters Supporting the Effluent Holdup Tank Failure

Soil Parameter	Parameter Description	Parameter Value (a) (b)	Parameter Justification
Length of Primary Contamination in X direction (meters)	Describes the X-axis length of the primary contamination	~10.2	The width of the area of soil saturated with water from the effluent tank failure. The shape is assumed to be a cube.
Length of Primary Contamination in Y direction (meters)	Describes the Y-axis length of the primary contamination	~10.2	The length of the area of soil saturated with water from the effluent tank failure. The shape is assumed to be a cube.
Evapotranspiration coefficient	Describes the fraction of precipitation and irrigation water penetrating the topsoil that is lost to evaporation and by transpiration by vegetation	0.64	This is a parameter used by RESRAD-OFFSITE to determine the amount of available water obtained from either precipitation or irrigation that infiltrates to the saturated zone. The value, when used in conjunction with precipitation and runoff, creates a recharge rate of ~18 inches/yr. This value is suggested by a study of regional data and is conservative when considering conditions likely present following construction.
Runoff coefficient (unitless)	Coefficient (fraction) of precipitation that runs off the surface and does not infiltrate into the soil	0	This is a parameter used by RESRAD-OFFSITE to determine the amount of available water obtained from either precipitation or irrigation that infiltrates to the saturated zone. The value, when used in conjunction with precipitation and evapotranspiration, creates a recharge rate of ~18 inches/yr. This value is suggested by a study of regional data and is conservative when considering conditions likely present following construction.
Contaminated zone total porosity (unitless)	Total porosity of the contaminated sample, which is the ratio of the soil pore volume to the total volume	2.7E-01	On-site data collected at Lee. A value representative of partially weathered rock is used for conservatism.
Density of contaminated zone (g/cm^3)	Density of the contaminated soil impacted by the liquid tank failure	1.8E+00	On-site data collected at Lee. A value representative of partially weathered rock is used for conservatism.
Contaminated zone hydraulic conductivity (meters per year)	Flow velocity of groundwater through the contaminated zone under a hydraulic gradient	~4.42E+02	The hydraulic conductivity was calculated from on-site data collected at Lee. Based on a value representative of 1.40E-03 cm/s for partially weathered rock is used for conservatism, converted to m/y.
Density of saturated zone (g/cm^3)	Density of the saturated zone soil that transmits groundwater	1.98E+00	On-site data was collected at Lee. A value representative of partially weathered rock is used for conservatism.

Table 2.4.13-203 (Sheet 4 of 5)
Listing of Lee Nuclear Station Data and Modeling Parameters Supporting the Effluent Holdup Tank Failure

Soil Parameter	Parameter Description	Parameter Value [(a)] [(b)]	Parameter Justification
Saturated zone total porosity (unitless)	Total porosity of the saturated zone soil, which is the ratio of the pore volume to the total volume	2.7E-01	On-site data was collected at Lee. A value representative of partially weathered rock is used for conservatism.
Saturated zone effective porosity (unitless)	Ratio of the part of the pore volume where water can circulate to the total volume of a representative sample	8.0E-02	On-site data was collected at Lee. A value representative of partially weathered rock is used for conservatism.
Saturated zone hydraulic gradient to surface water body (unitless)	Change in groundwater elevation per unit of distance in the direction of groundwater flow to a surface water body	4.7E-02	The site-specific hydraulic gradient, representative of partially weathered rock, for the pathway having shortest (i.e., most rapid) travel time to the nearest off-site surface water body. Assumed to be nearest on-site surface water body (Hold-Up Pond A) for conservatism.
Longitudinal dispersivity to surface water body (meters)	Describes the ratio between the longitudinal dispersion coefficient and the pore water velocity, The parameter depends on the length of the saturated zone	3.77E+00	Follows recommendations in the RESRAD-OFFSITE User Manual.
Lateral (horizontal) dispersivity to surface water body (meters)	Describes the ratio between the horizontal lateral dispersion coefficient and the pore water velocity	3.77E-01	Follows recommendations in the RESRAD-OFFSITE User Manual.
Lateral (vertical) dispersivity to the surface water body (meters)	Describes the vertical dispersion. The user may either model (a) vertical dispersion in the saturated zone and ignore the effects of clean infiltration along the length of the saturated zone or (b) ignore vertical dispersion in the saturated and model the effects of clean infiltration along the length of the saturated zone.	3.77E-02	Follows recommendations in the RESRAD-OFFSITE User Manual.

Table 2.4.13-203 (Sheet 5 of 5)
Listing of Lee Nuclear Station Data and Modeling Parameters Supporting the Effluent Holdup Tank Failure

Soil Parameter	Parameter Description	Parameter Value [(a)] [(b)]	Parameter Justification
Distance to the nearest surface water body (meters)	Distance to the nearest off-site surface water body that contributes to a potable drinking water source	376.9	Site-specific value corresponding to the distance from the Unit 2 auxiliary building to the “hypothetical” well location, i.e., the nearest edge of Hold-Up Pond A minus the length of the contaminated zone.
Volume of the surface water body (m^3)	Describes the size of the surface water body	856,036	Site-specific value corresponding to the volume of the Broad River reservoir from the postulated release point downstream to the Ninety-Nine Islands Dam.
Residence time (yrs)	The average time that water spends in the surface water body	0.00397	Site-specific value obtained by dividing the volume of the surface water body by the volume of water that is extracted annually from it.

a) Parameter values are provided in metric units as used with RESRAD-OFFSITE.

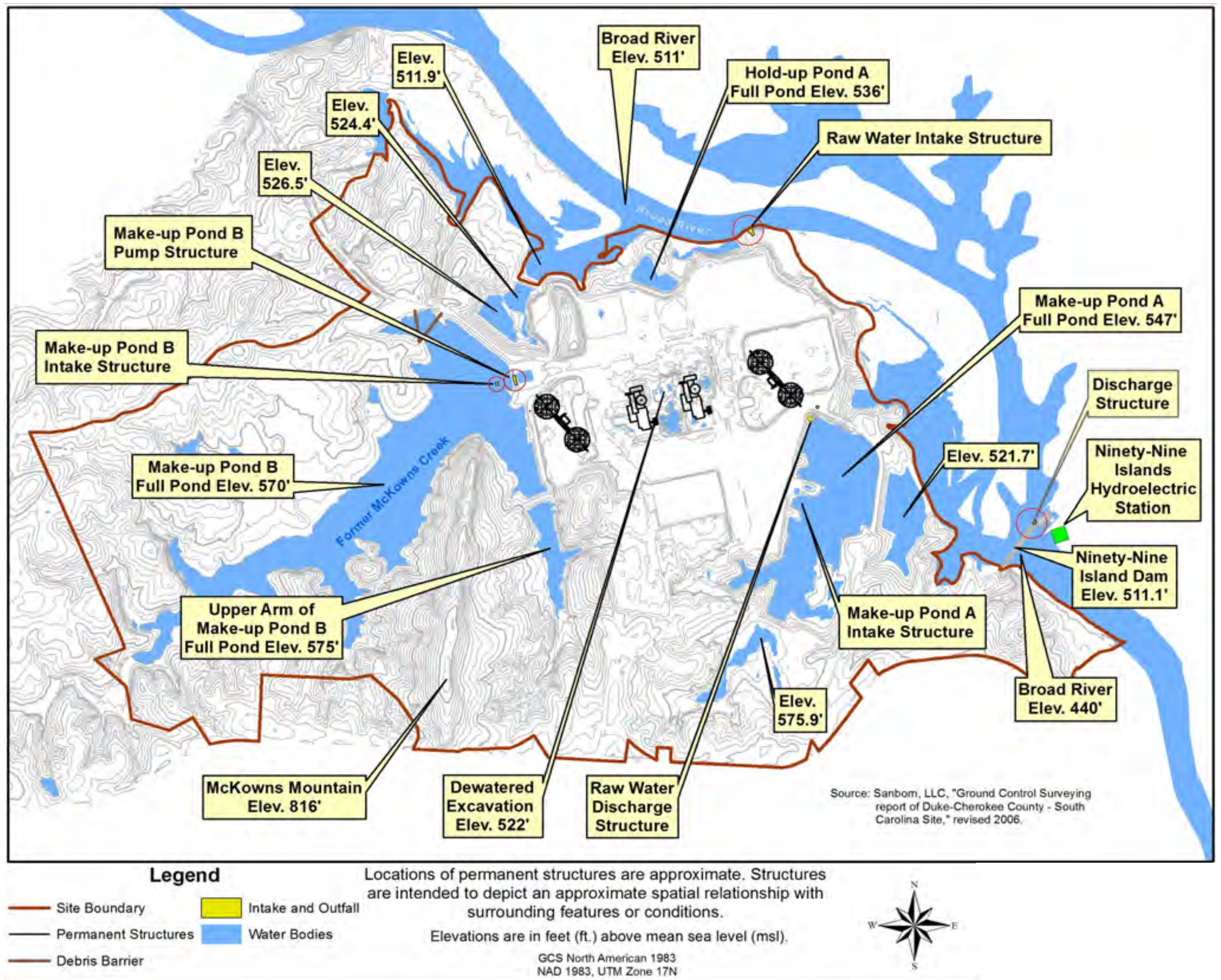
b) K_d values reported in the laboratory analysis for nickel, plutonium, and uranium are not included in the liquid effluent source term and, therefore, are not listed in this RESRAD-OFFSITE input table.

Table 2.4.13-204
Radionuclide Concentration at Nearest Drinking Water Source in an Unrestricted Area Due to Effluent Holdup Tank Failure

Detected Radionuclide	Radionuclide Concentration microcuries/ml	10 CFR 20 Appendix B Table 2 Column 2 microcuries/ml	Sum of Fractions Contribution[a]
H-3	3.47E-08	1.00E-03	3.47E-05
			Sum of Fractions[b]
			3.50E-05

a) Those radionuclides with Sum of Fractions Contribution less than 1.0E-5 are negligible and not included in the table.

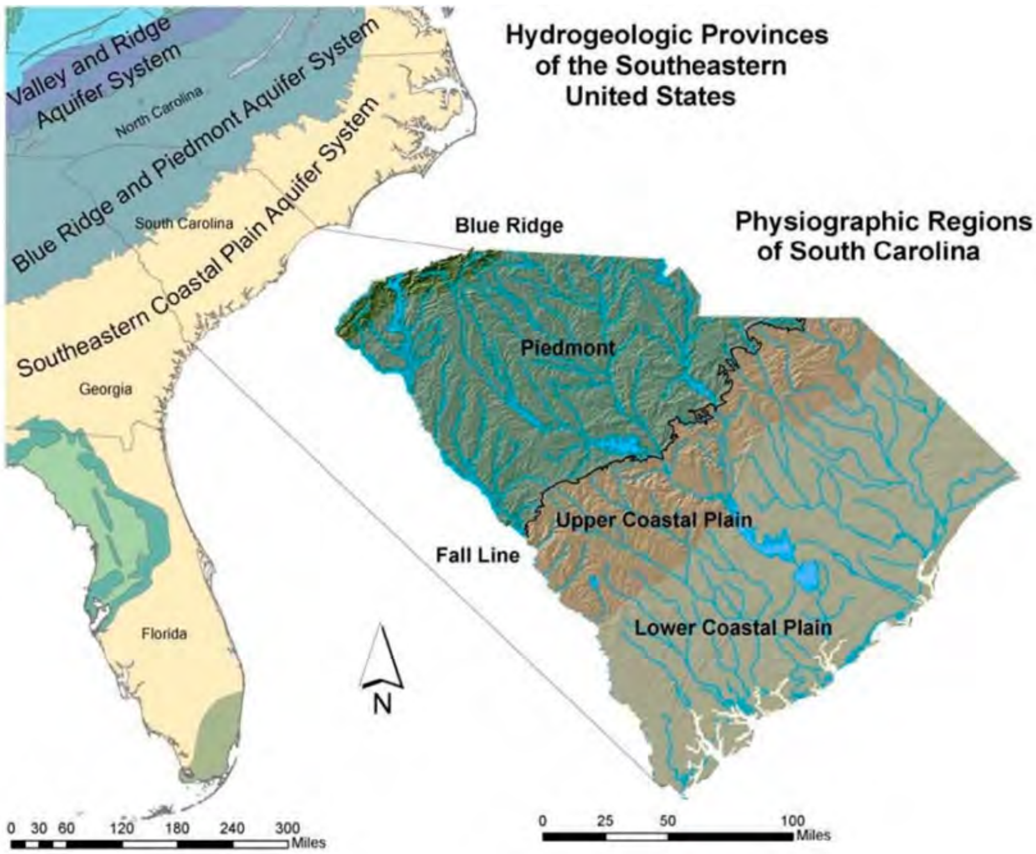
b) Total for all detected radionuclides.

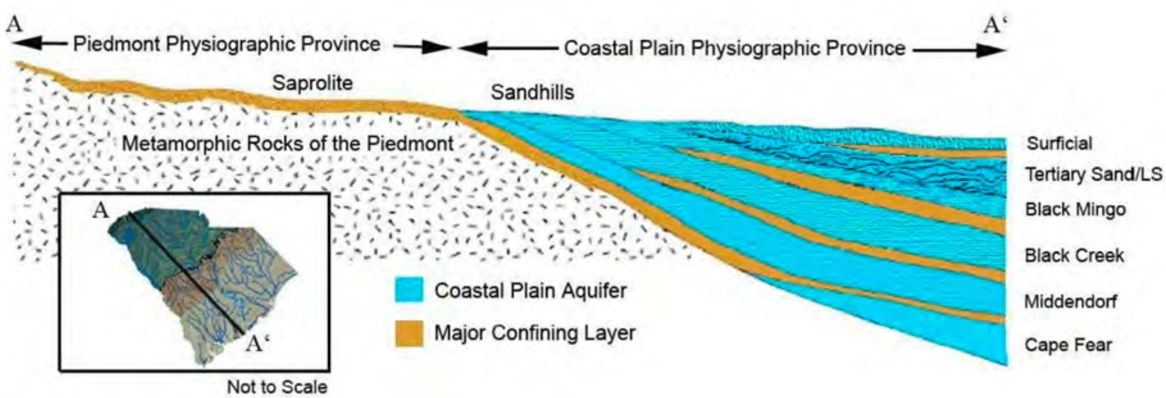


Datum: South Carolina State Plane Coordinate System
NAD 83, NAVD 88, UTM Zone 17N

Figure 2.4.1-201
Site Surface Water Features

Figure 2.4.1-202
Not Used





Generalized Hydrogeologic Cross-Section from the Blue Ridge through the Lower Coastal Plain in South Carolina
Source: Reference 267

Figure 2.4.1-203
Physiographic and Hydrogeologic Provinces of South Carolina

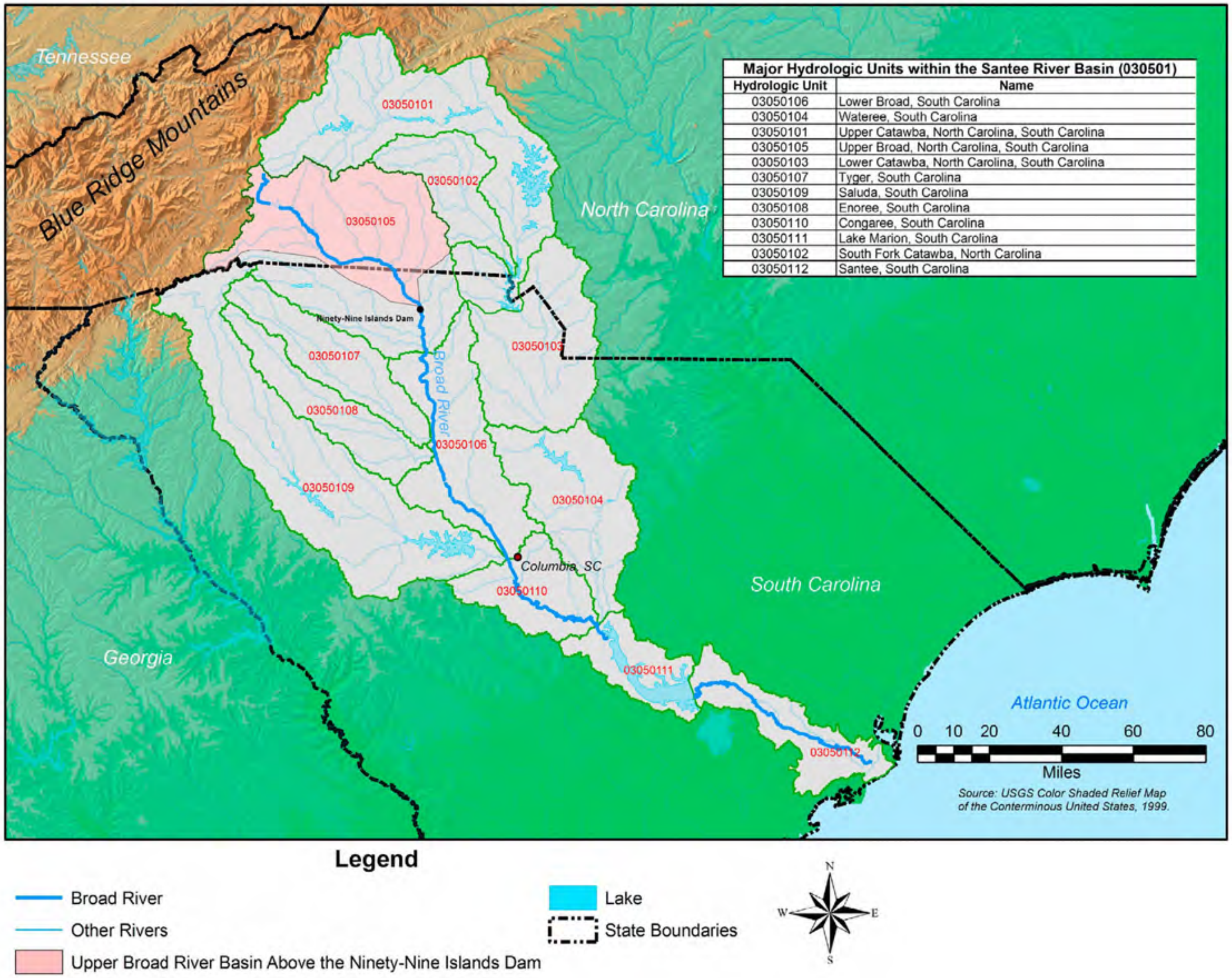


Figure 2.4.1-204
The Broad River Within the Santee River Basin

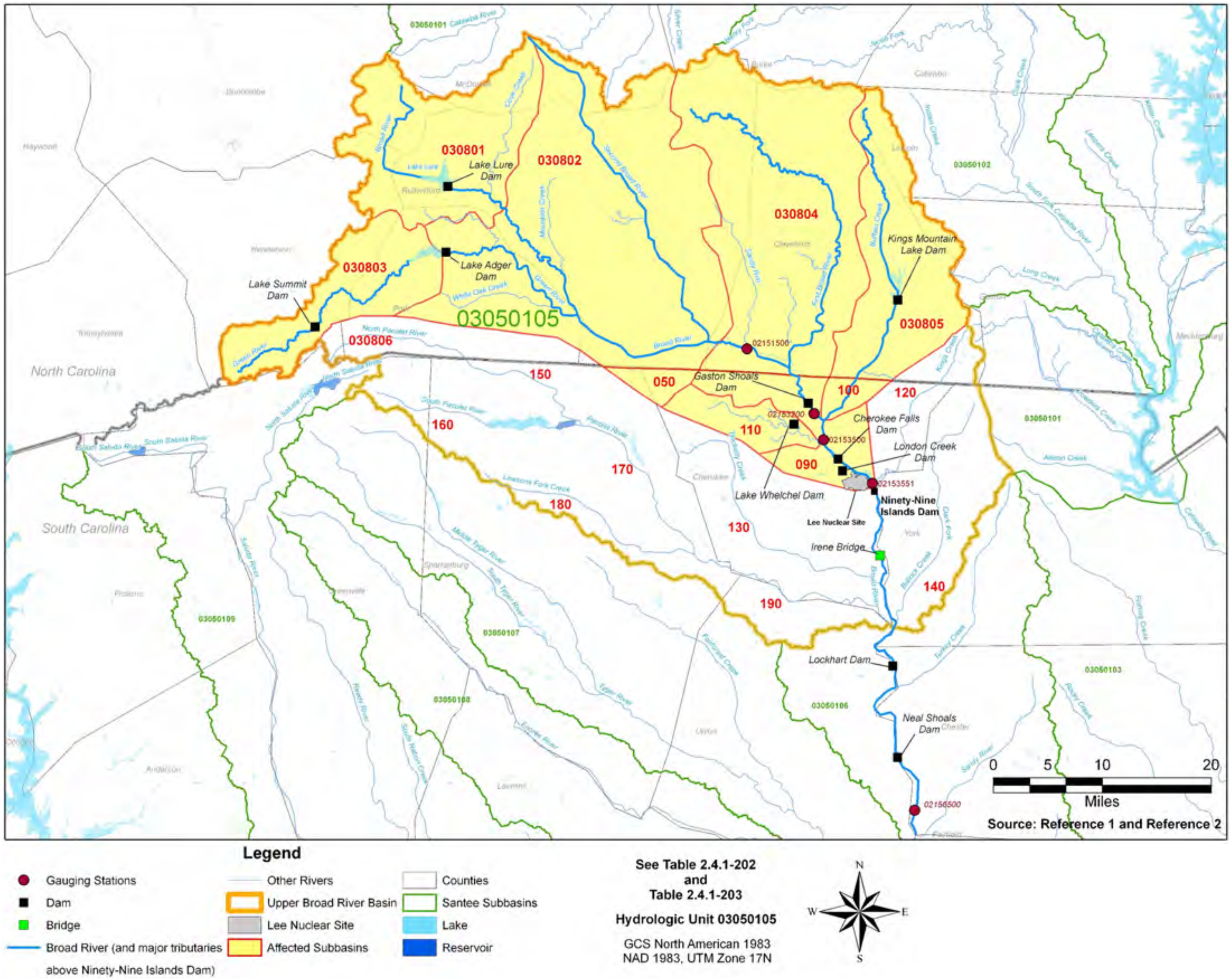


Figure 2.4.1-205
Upper Broad River Basin and Subbasins

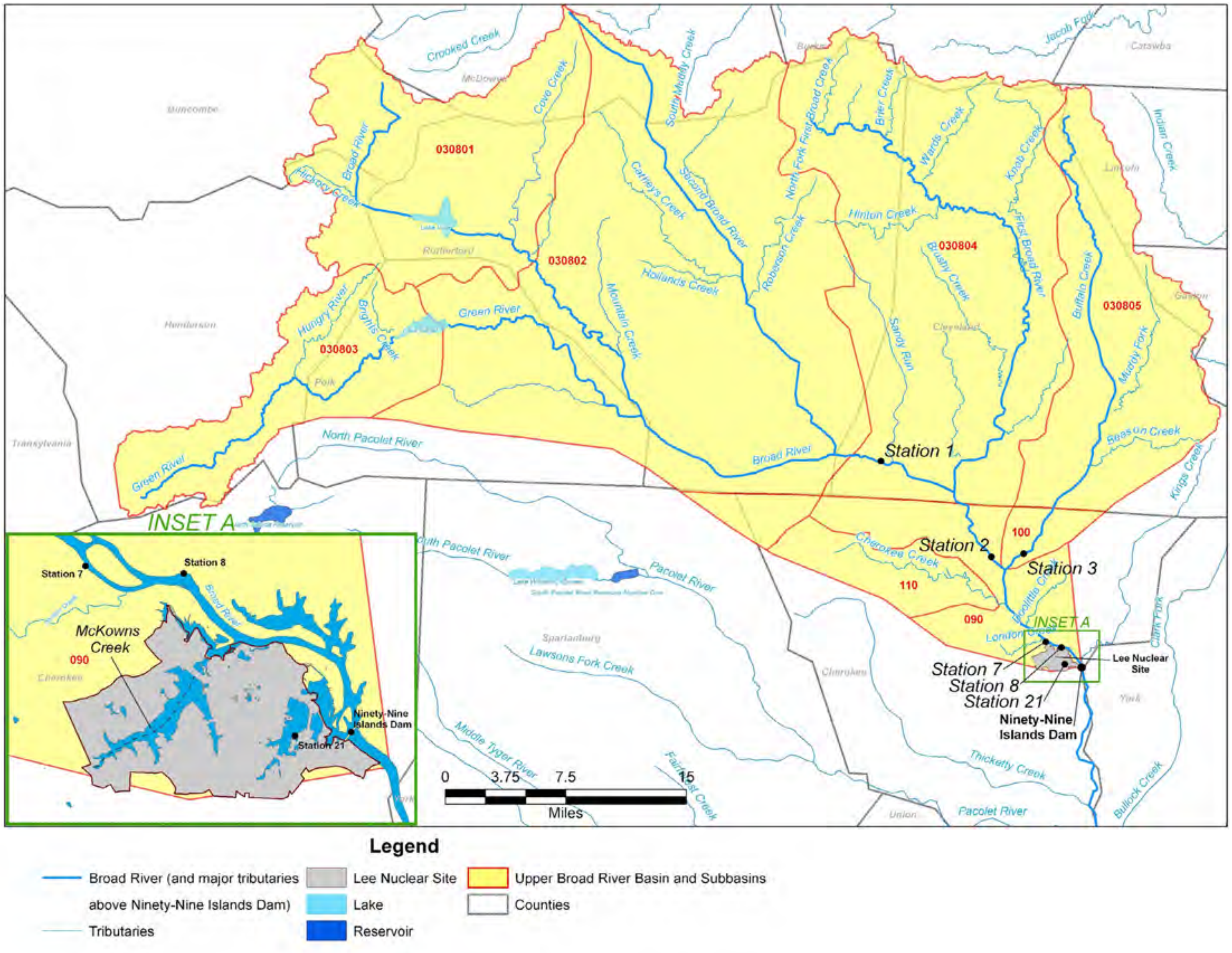
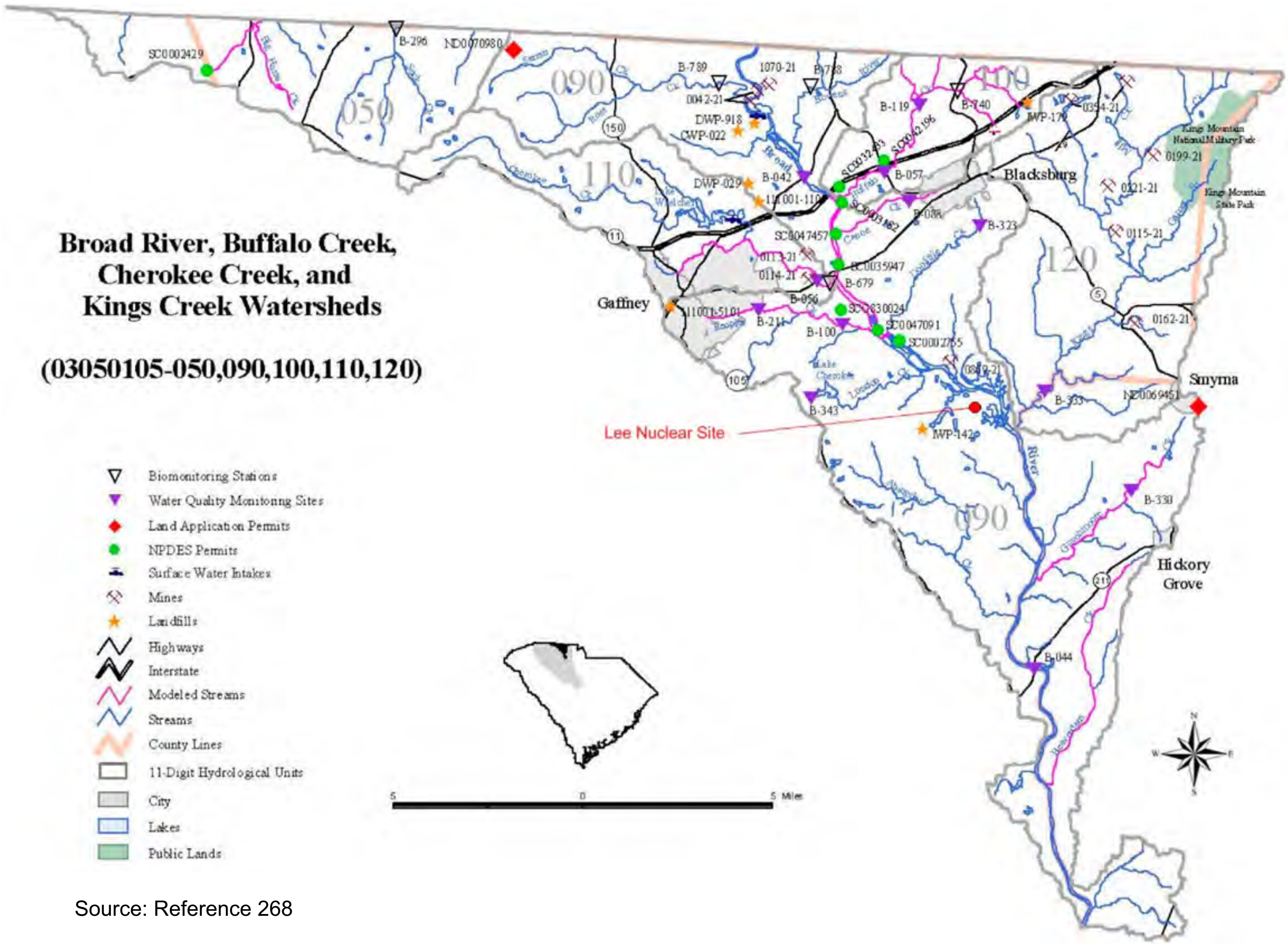
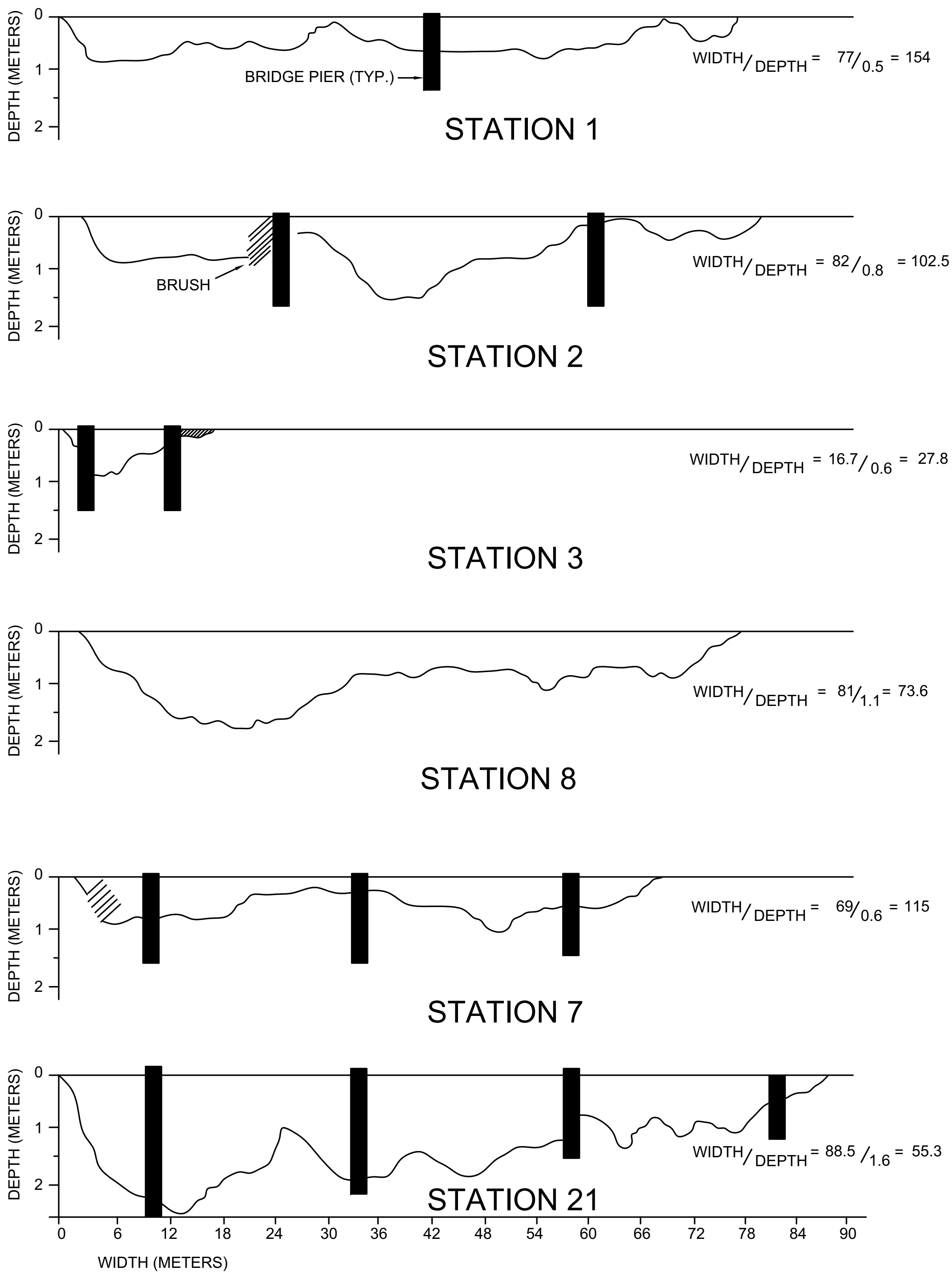


Figure 2.4.1-206
The Broad River and its Tributaries Above Ninety-Nine Islands Dam



Source: Reference 268

Figure 2.4.1-207
Cherokee County Watershed: Select Facilities and Monitoring Stations



Station Locations shown on Figure 2.4.1-206
Source: Reference 214

Figure 2.4.1-208
Broad River Width and Depth Data

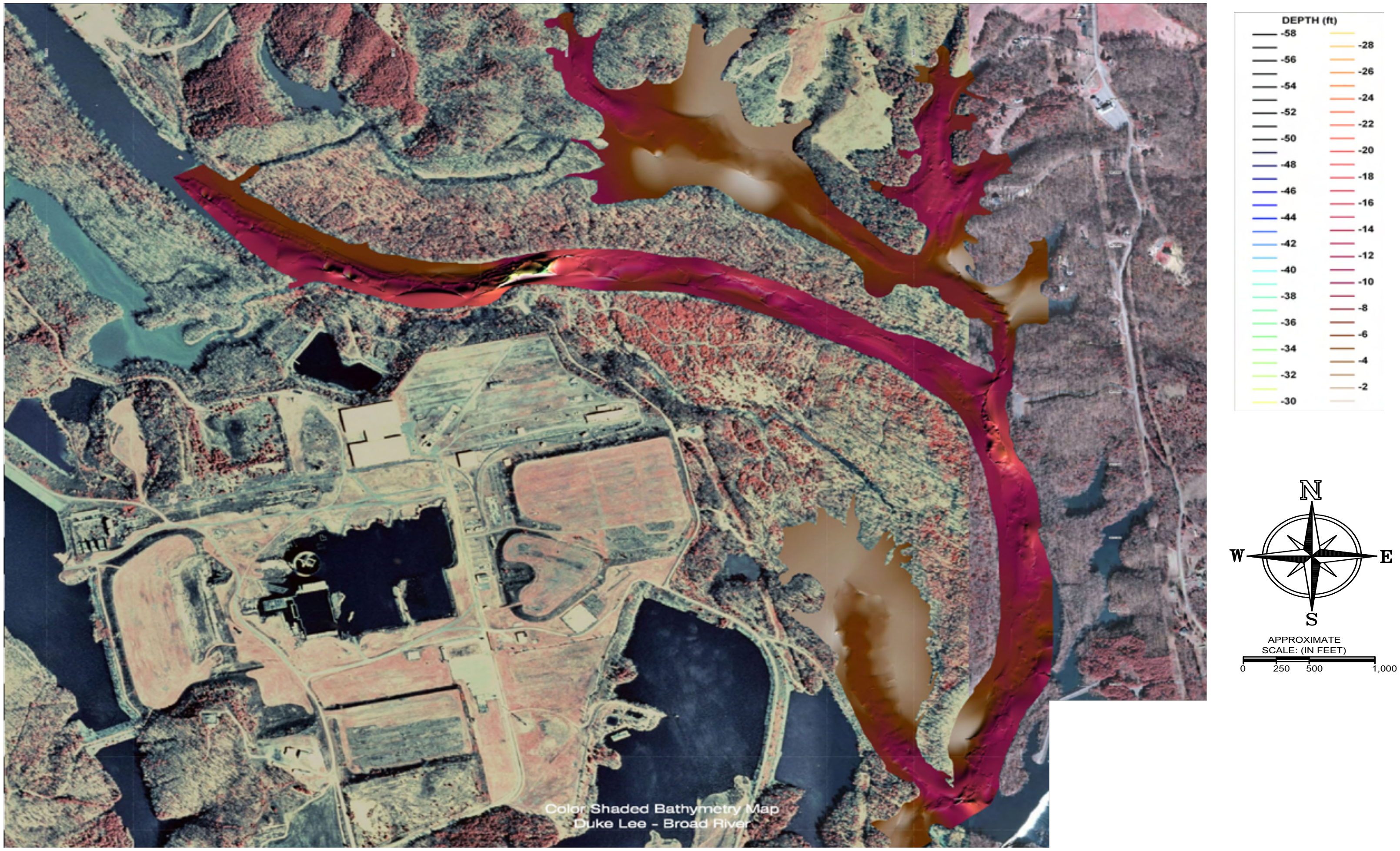


Figure 2.4.1-209 (Sheet 1 of 4)
Bathymetry Map: Ninety-Nine Island Reservoir

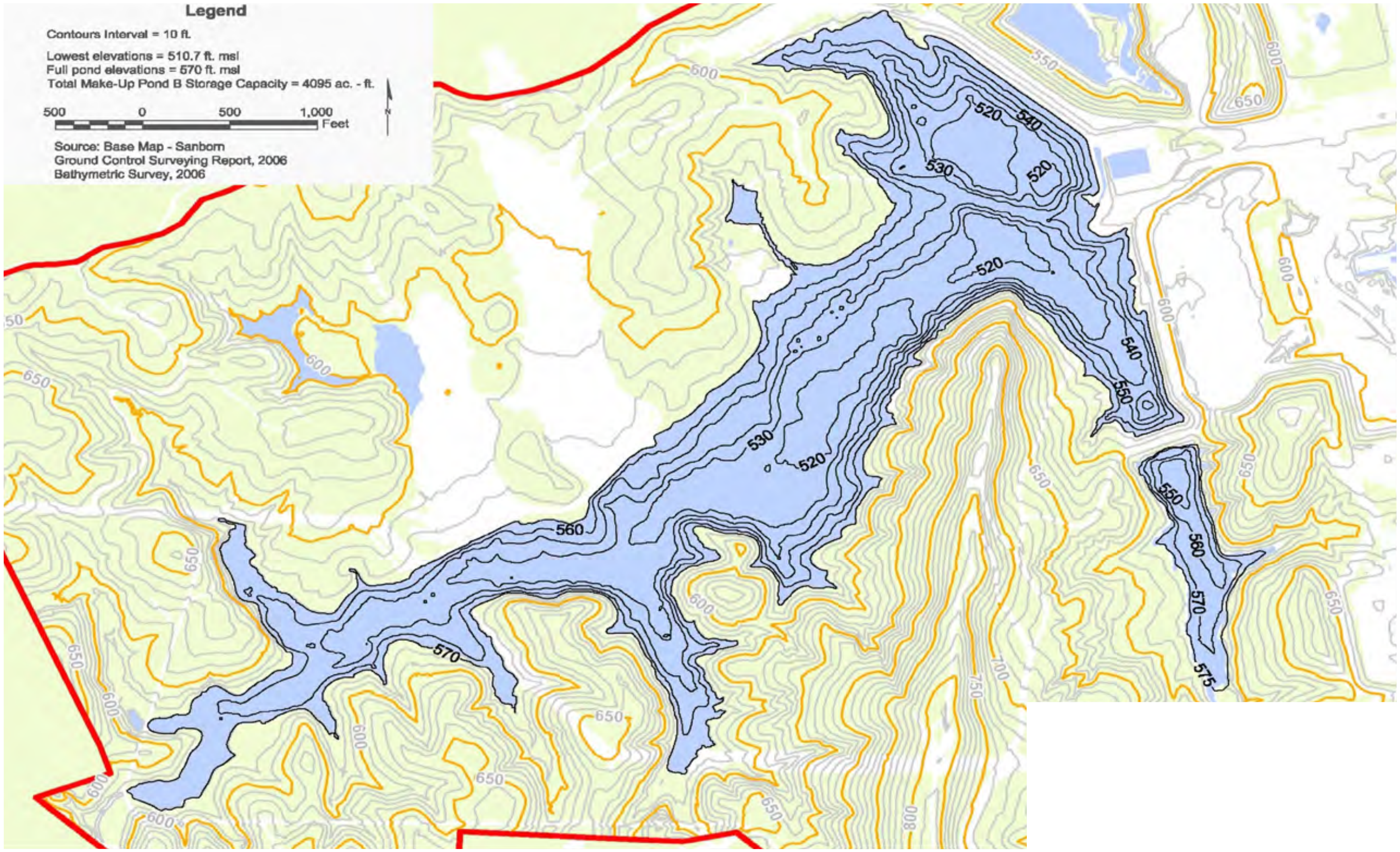


Figure 2.4.1-209 (Sheet 2 of 4)
Bathymetric Map: Make-Up Pond B

Figure 2.4.1-209 (Sheet 3 of 4)
Bathymetric Map: Make-Up Pond A

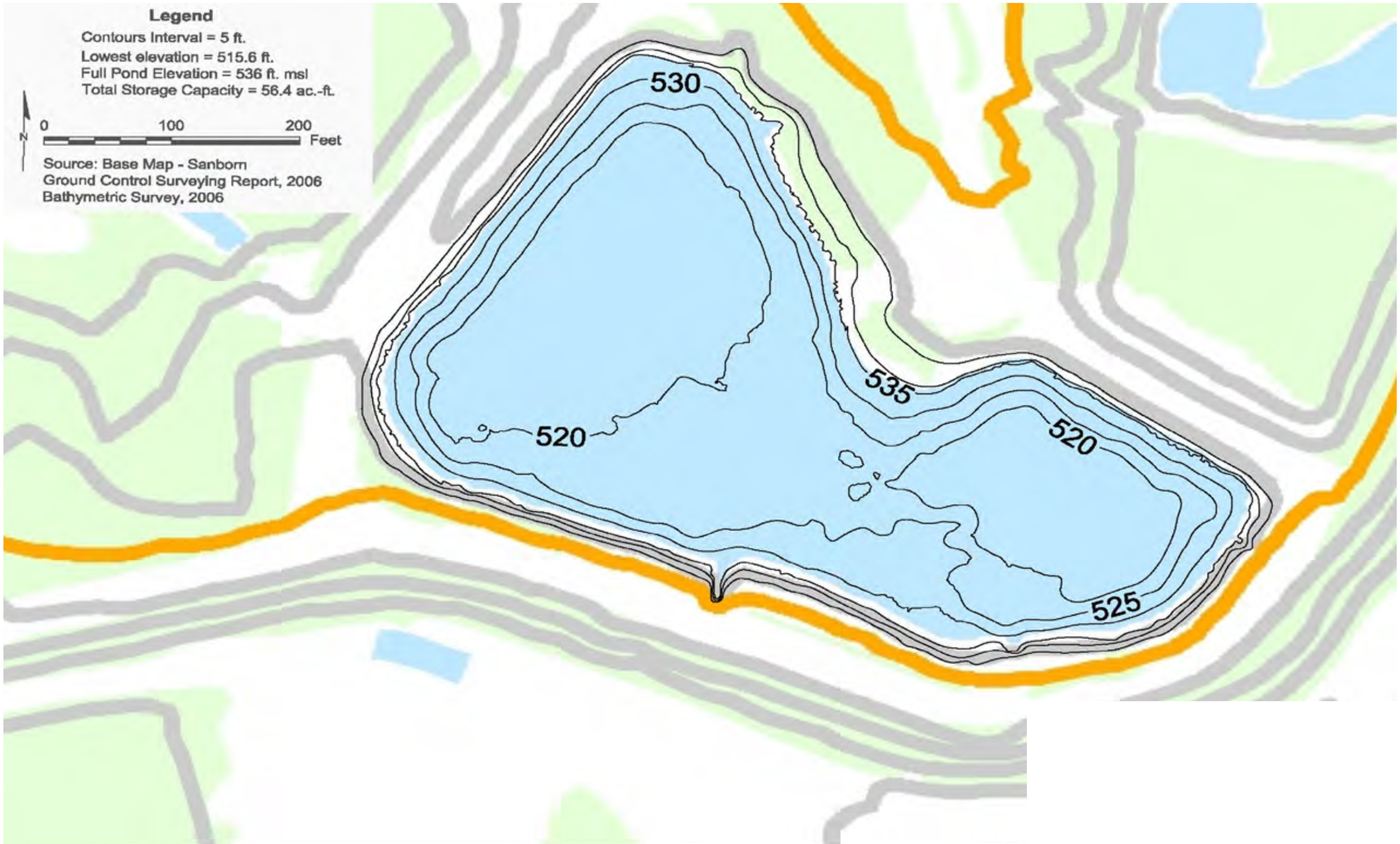
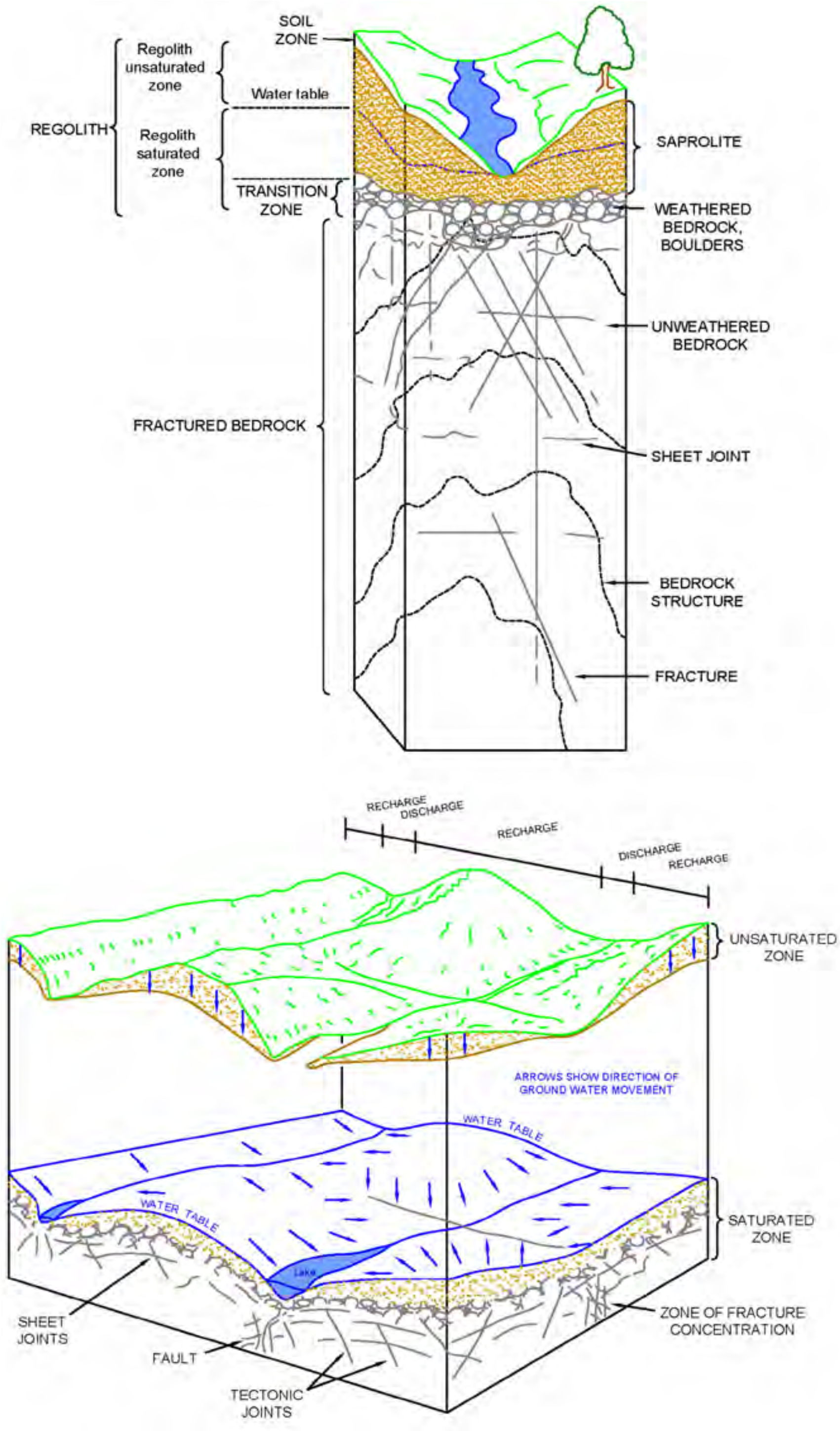


Figure 2.4.1-209 (Sheet 4 of 4)
Bathymetric Map: Hold-Up Pond A



Source: Reference 285

Figure 2.4.1-210
The Piedmont Aquifer System

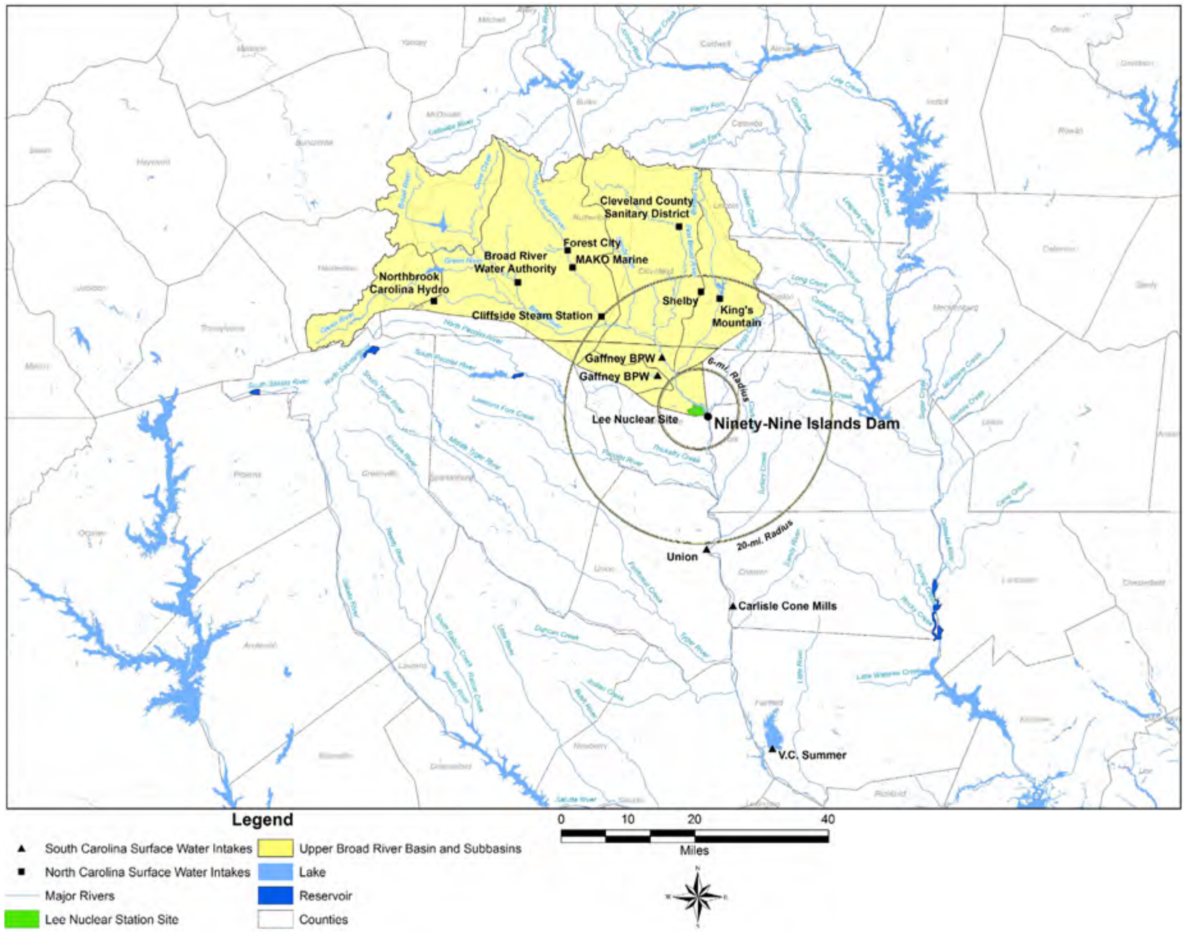
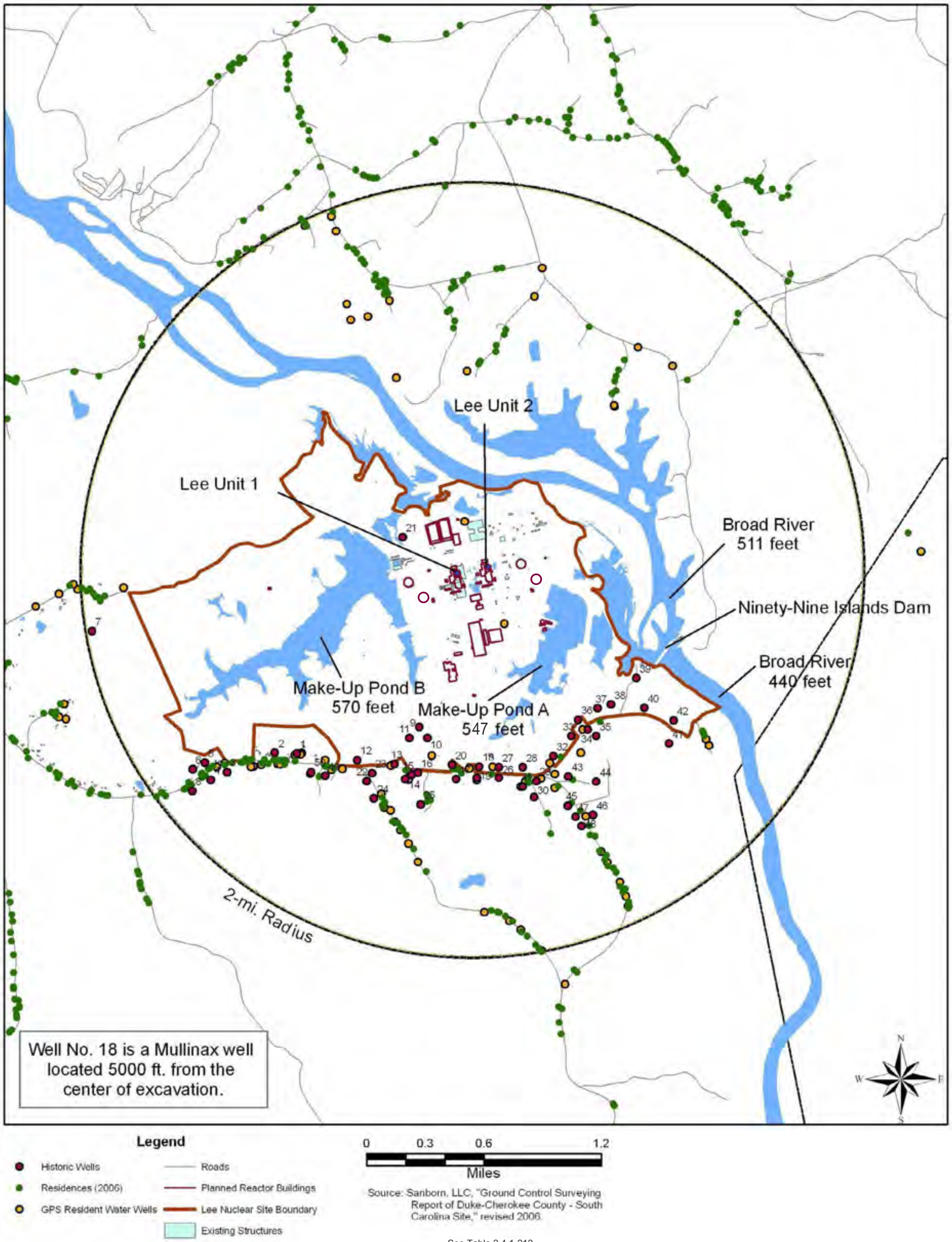


Figure 2.4.1-211
Relevant Surface Water Intakes on the Broad River

Withhold From Public Disclosure Under 10 CFR 2.390(a)(9)



See Table 2.4.1-212

Figure 2.4.1-212
Groundwater Supply Wells Surrounding the Lee Nuclear Site

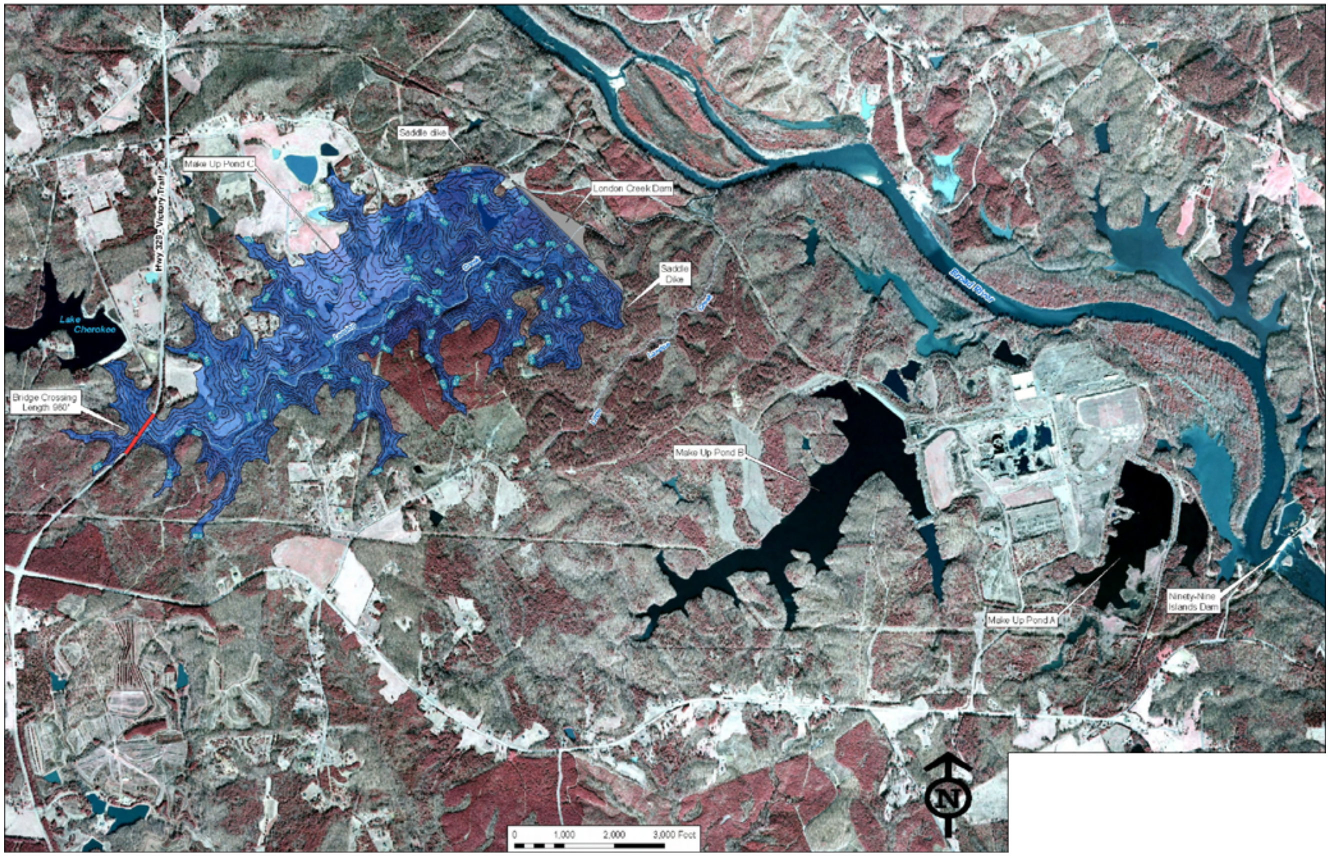


Figure 2.4.1-213
Make-Up Pond C Location and Bathymetry

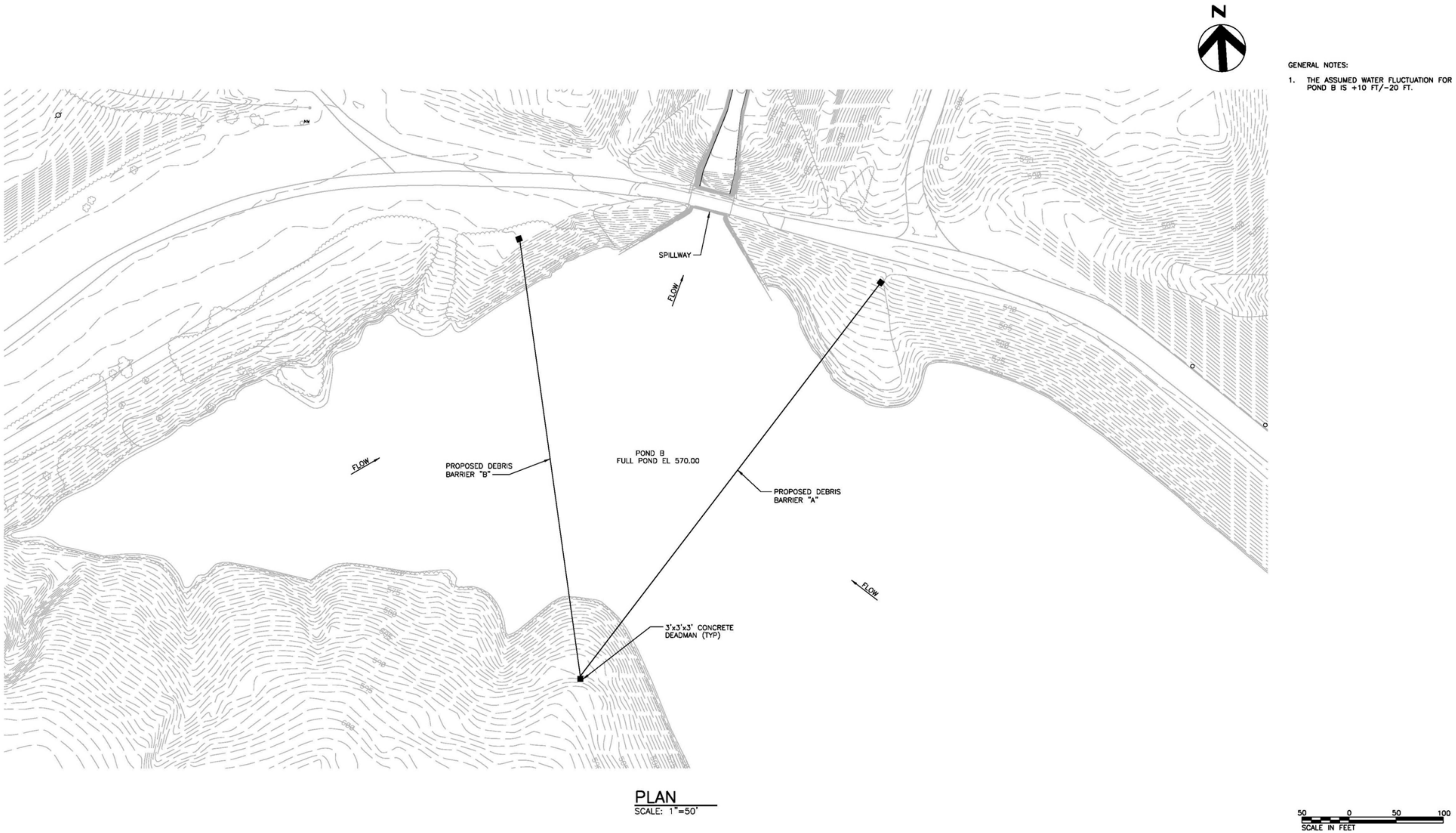
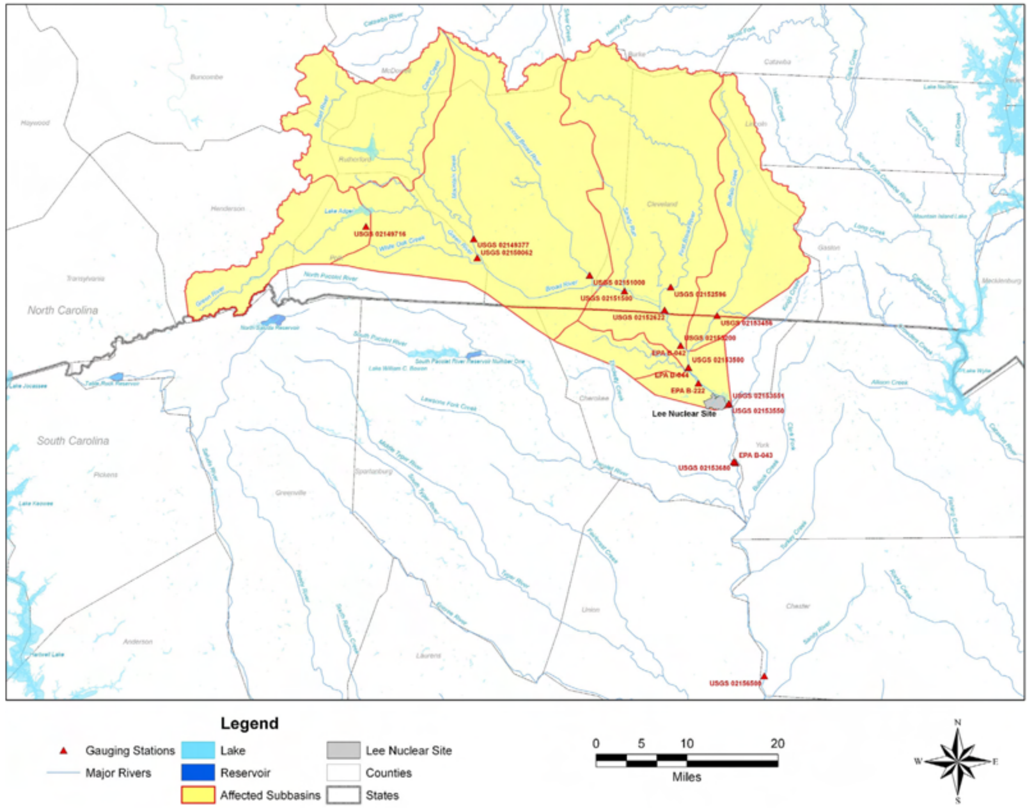


Figure 2.4.1-214
Pond B Conceptual Debris Barrier



Datum: Geographic Coordinate System NAD 83, UTM Zone 17N.

Figure 2.4.2-201
Gauging Station Locations

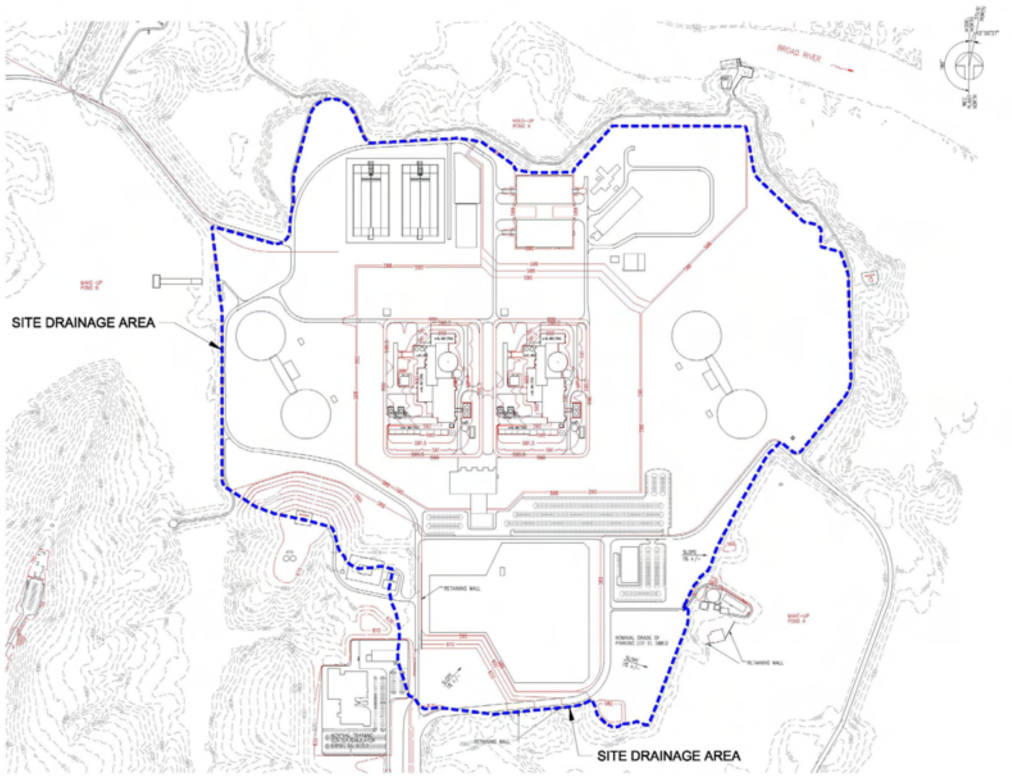
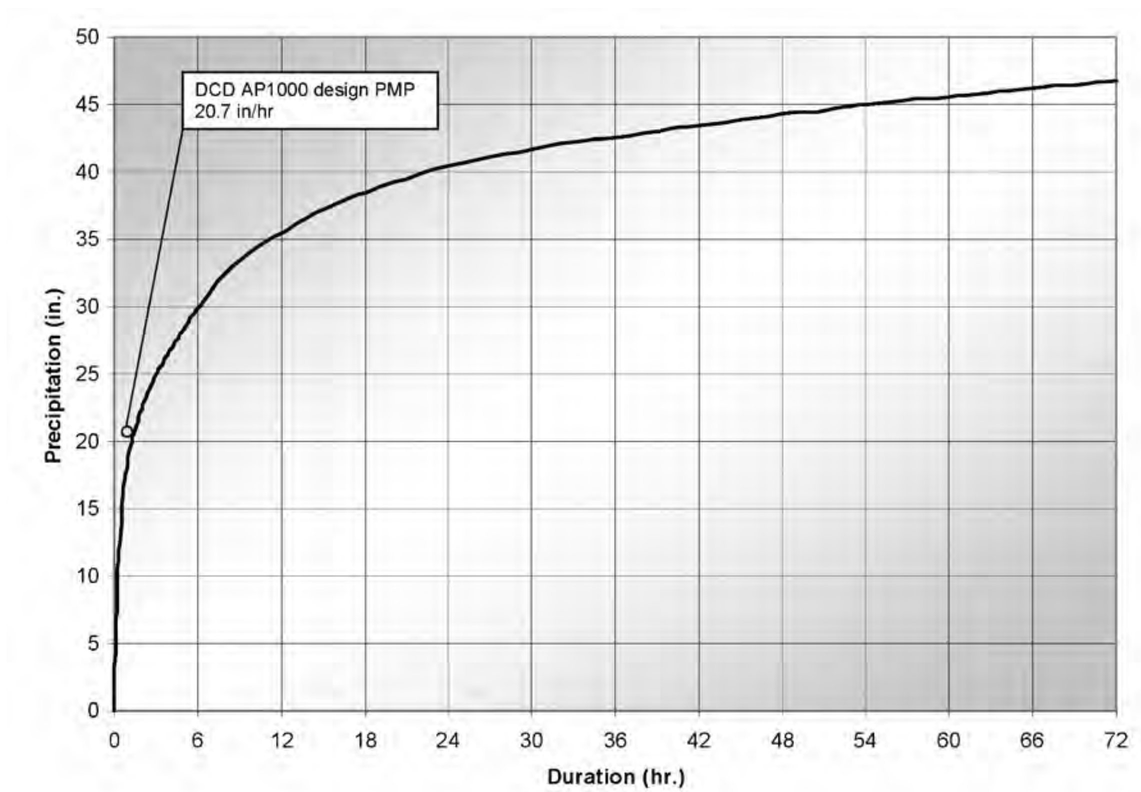


Figure 2.4.2-202
Grading and Drainage Plan



Reference Table 2.4.2-203

Figure 2.4.2-203
Local Intense Probable Maximum Precipitation Depth-Duration Curve

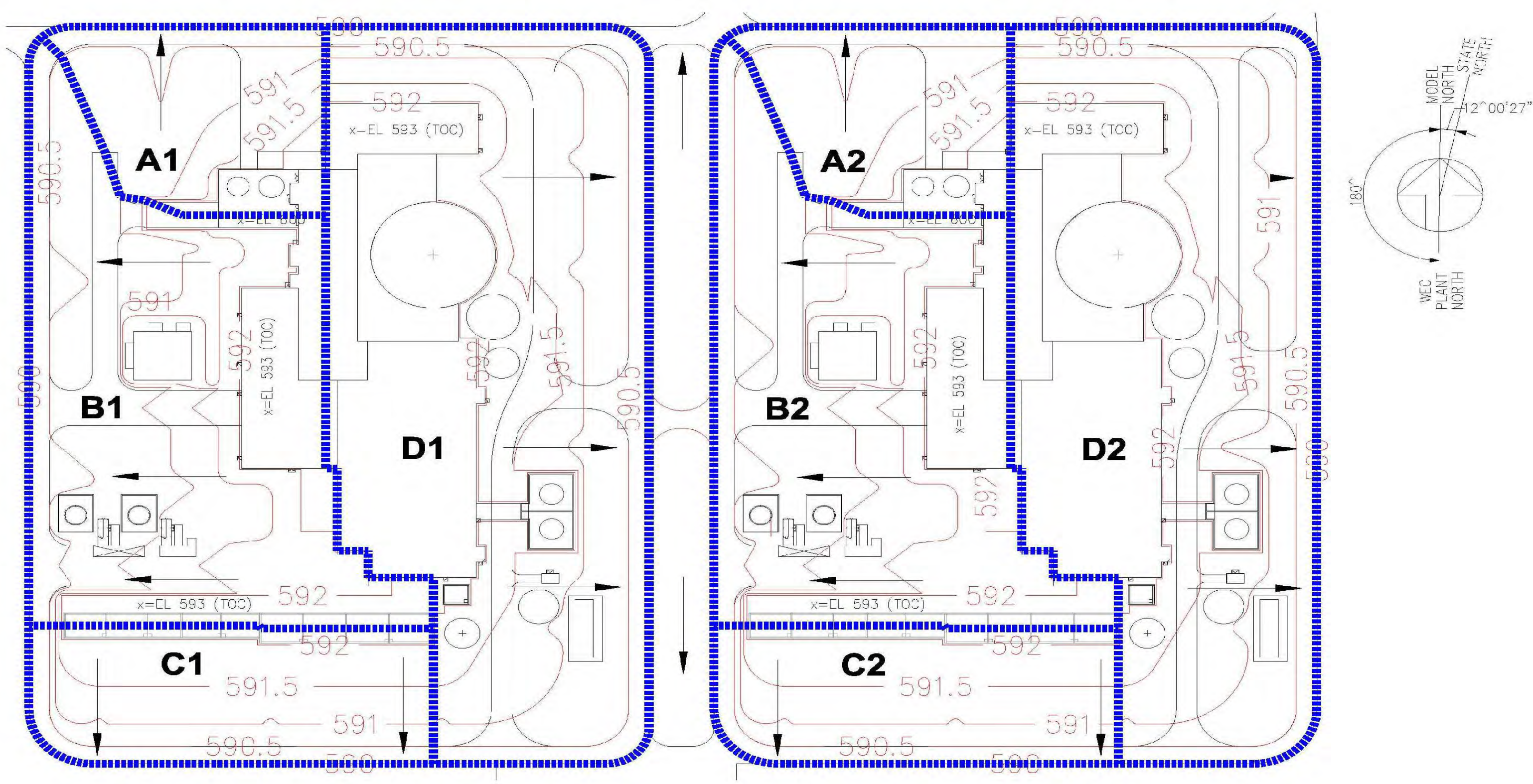


Figure 2.4.2-204
Site Analysis Drainage Areas

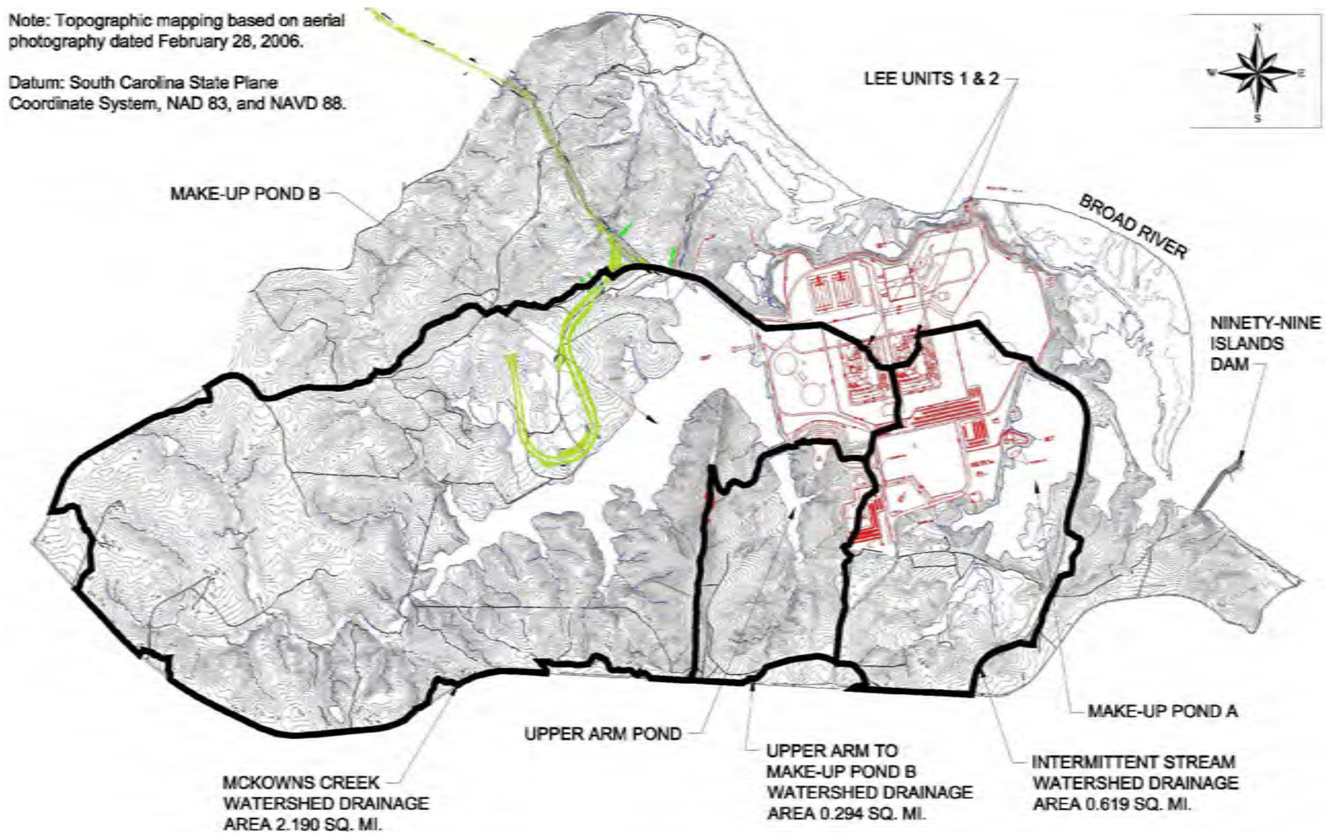
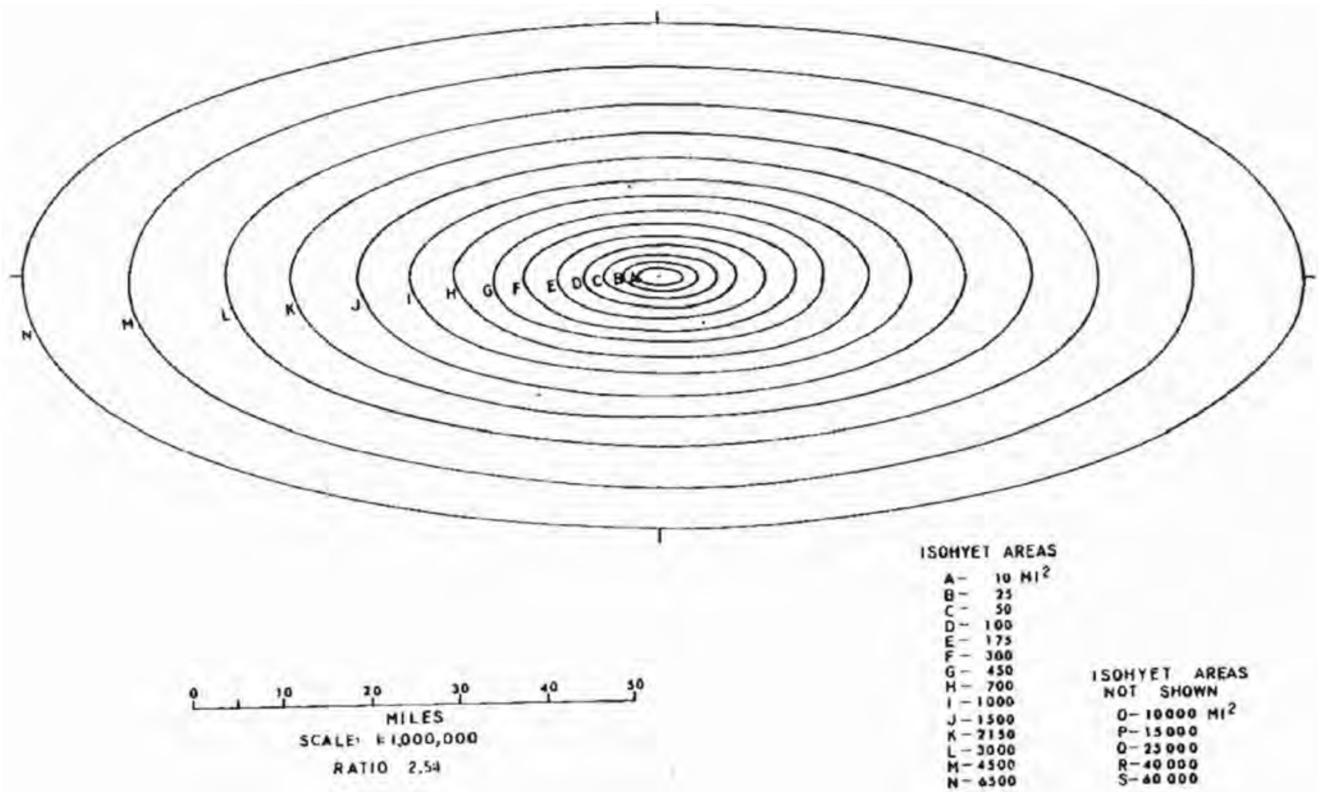
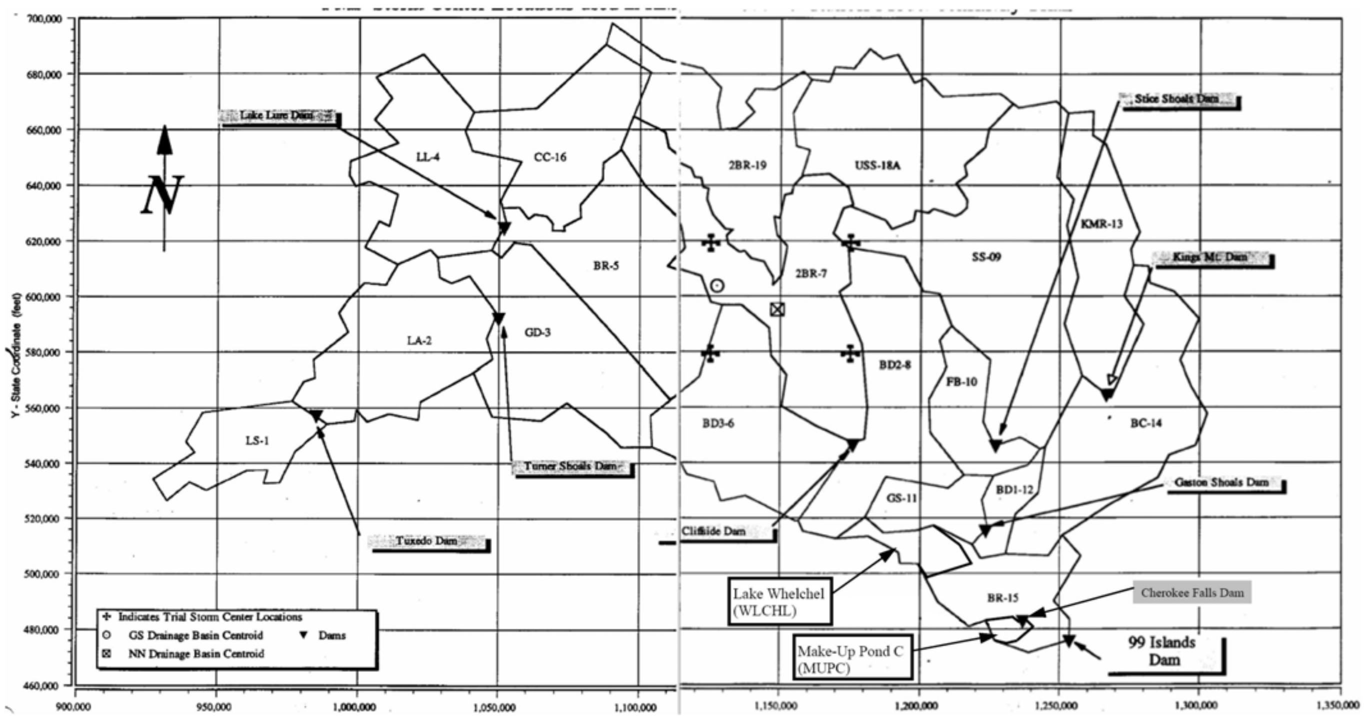


Figure 2.4.3-201
Make-Up Pond A and Make-Up Pond B Watersheds



Reference 225

Figure 2.4.3-202
HMR-52 Standard Isohyetal Pattern



Reference 217

Datum: North Carolina and South Carolina State Plane Coordinate System, NAD 27.

Figure 2.4.3-203
Broad River Watershed Subbasins

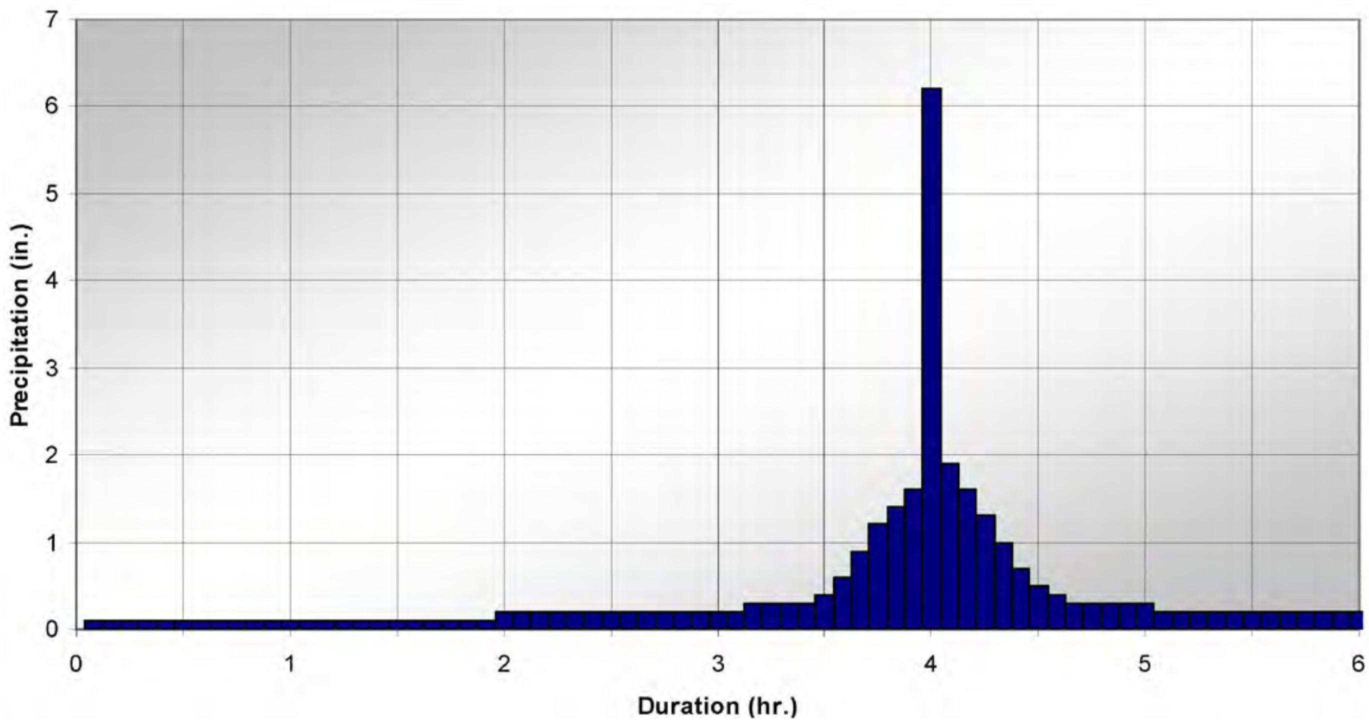


Figure 2.4.3-204
Local Intense Probable Maximum Precipitation 6-Hour Hyetograph

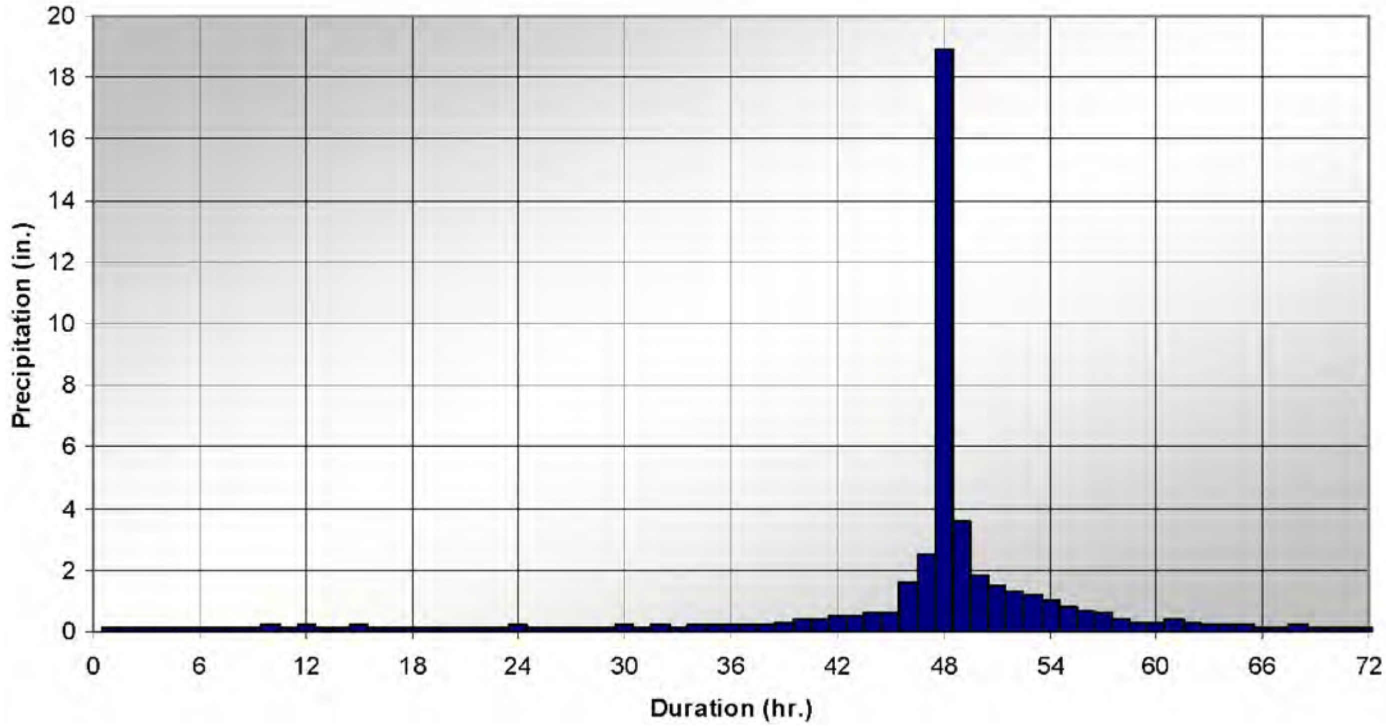


Figure 2.4.3-205
Local Intense Probable Maximum Precipitation 72-Hour Hyetograph

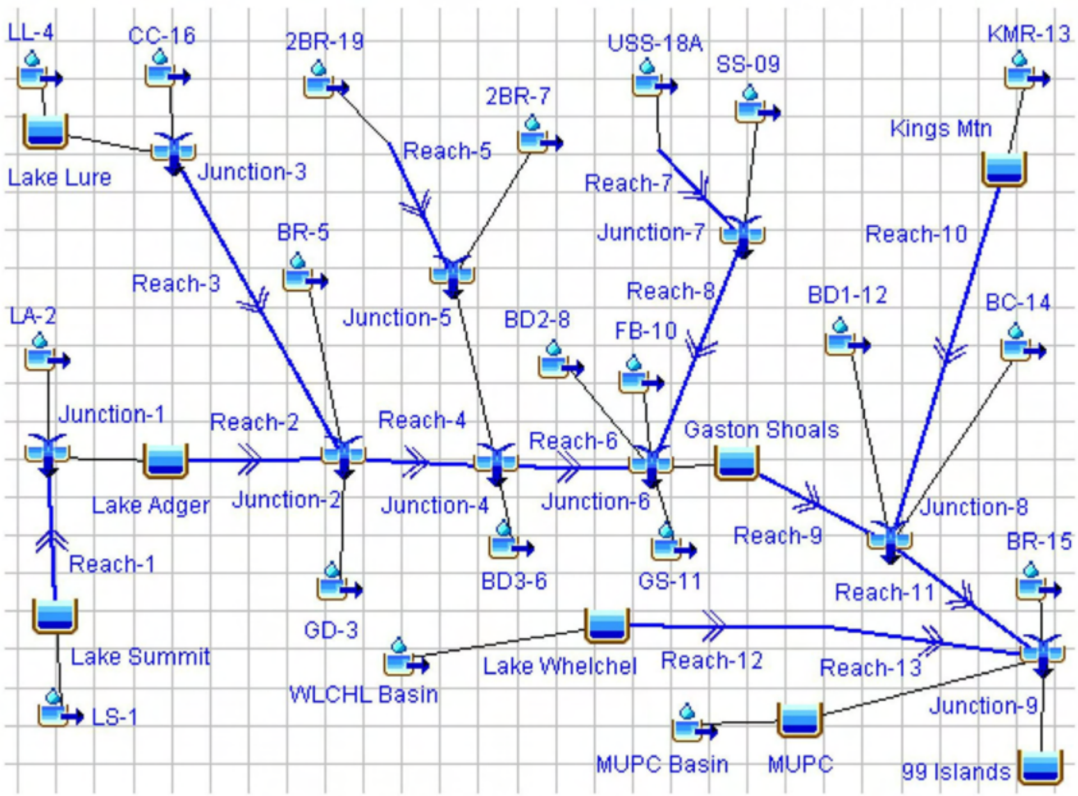
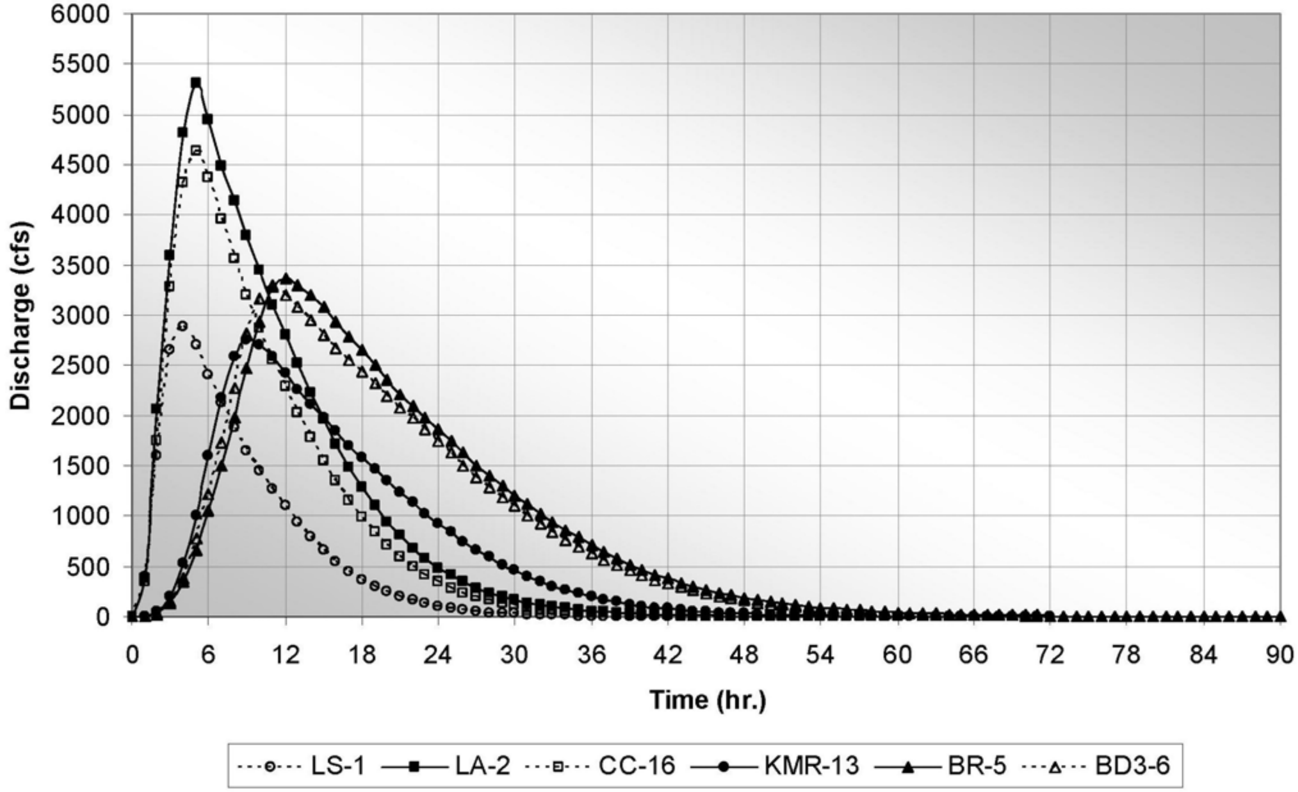
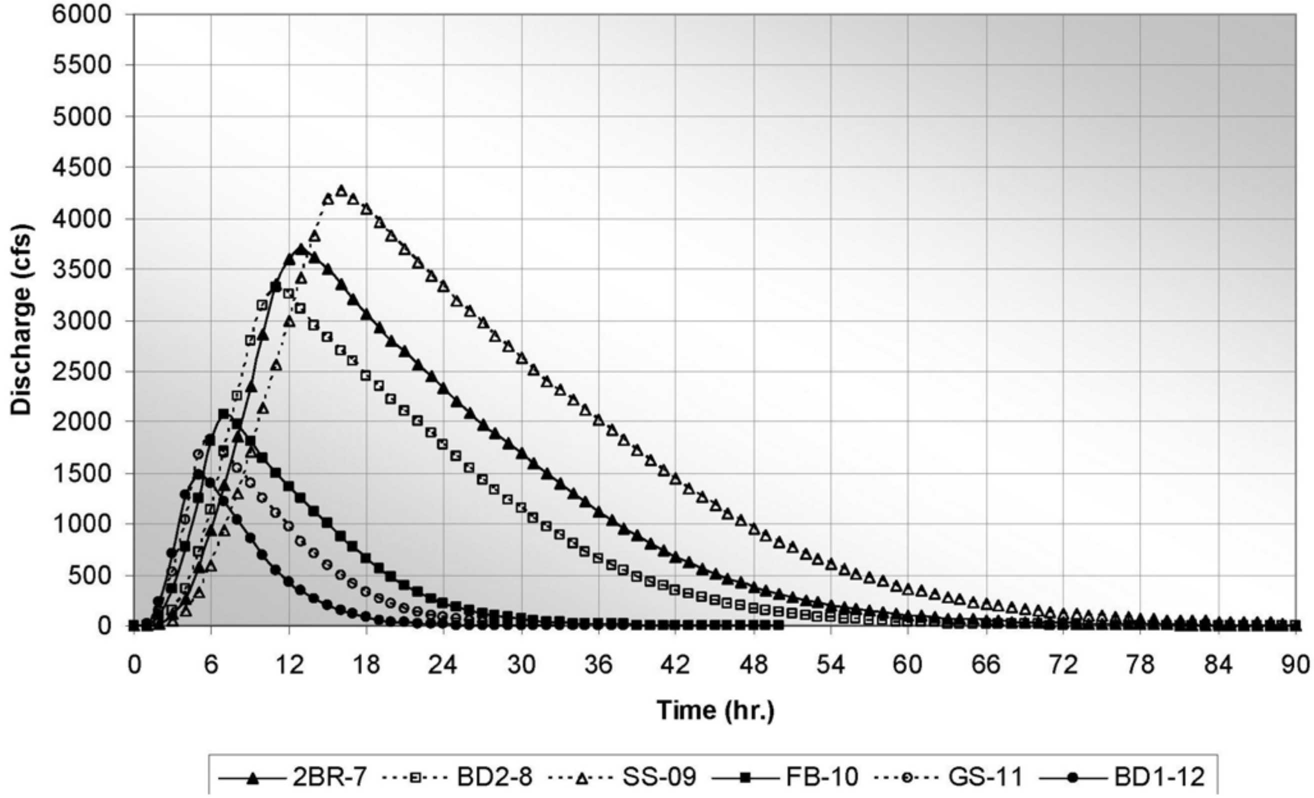


Figure 2.4.3-206
HEC-HMS Broad River Watershed Subbasin Schematic



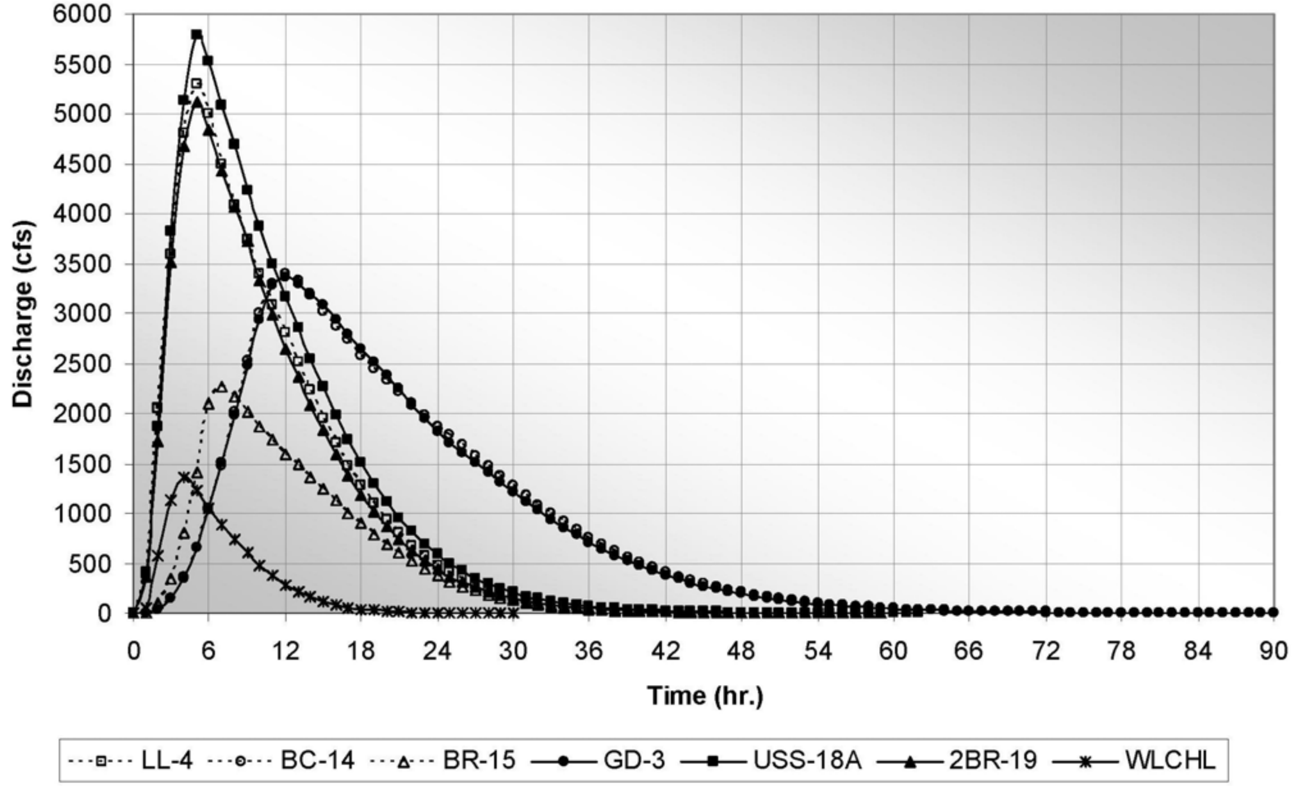
Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-207
Subbasin Unit Hydrographs: LS-1, LA-2, CC-16, KMR-13, BR-5, and BD3-6



Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-208
Subbasin Unit Hydrographs: 2BR-7, BD2-8, SS-09, FB-10, GS-11, and BD1-12



Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-209
Subbasin Unit Hydrographs: LL-4, BC-14, BR-15, GD-3, USS-18A, 2BR-19, and WLCHL

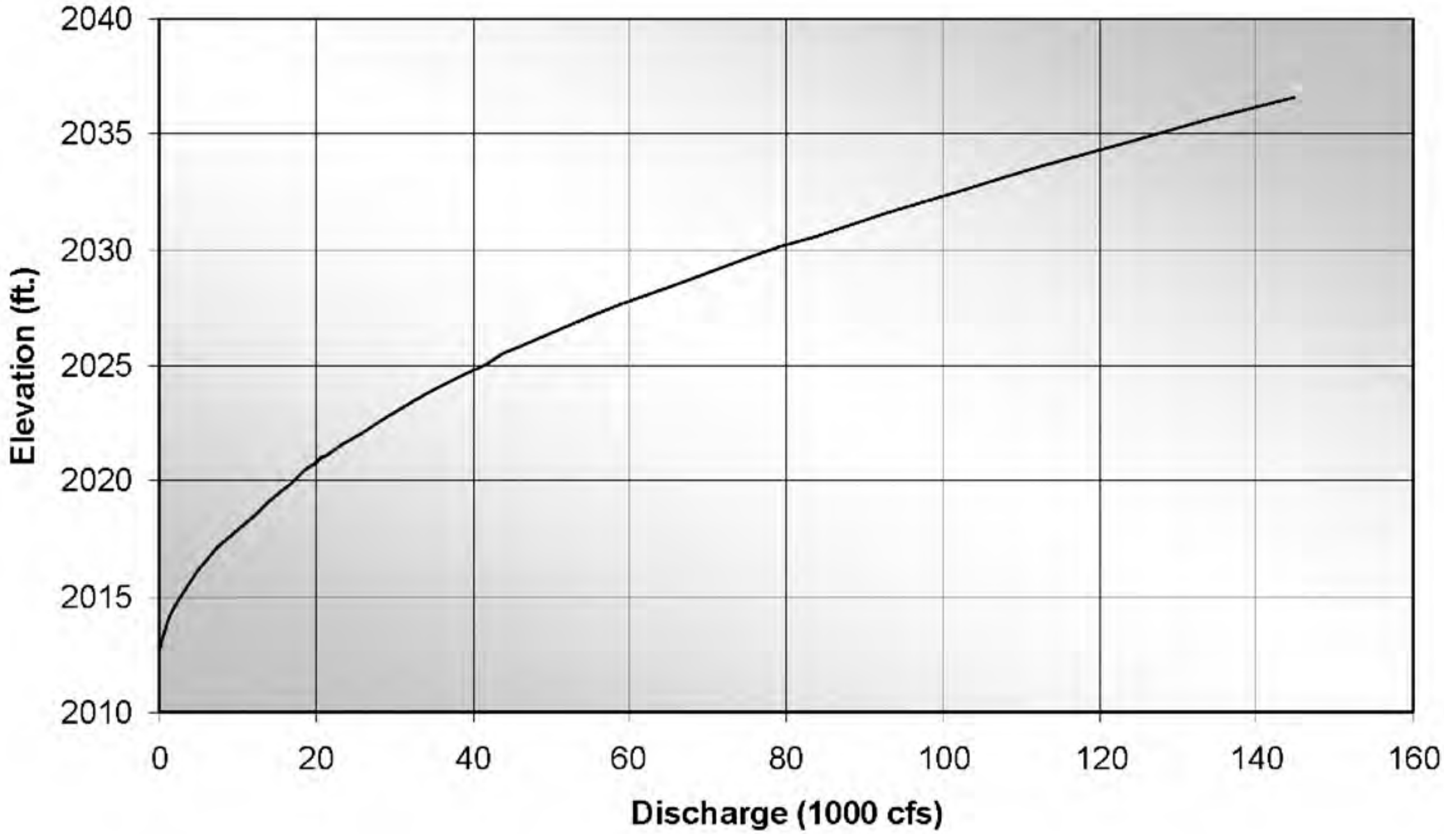


Figure 2.4.3-210
Discharge Rating Curve, Tuxedo Dam

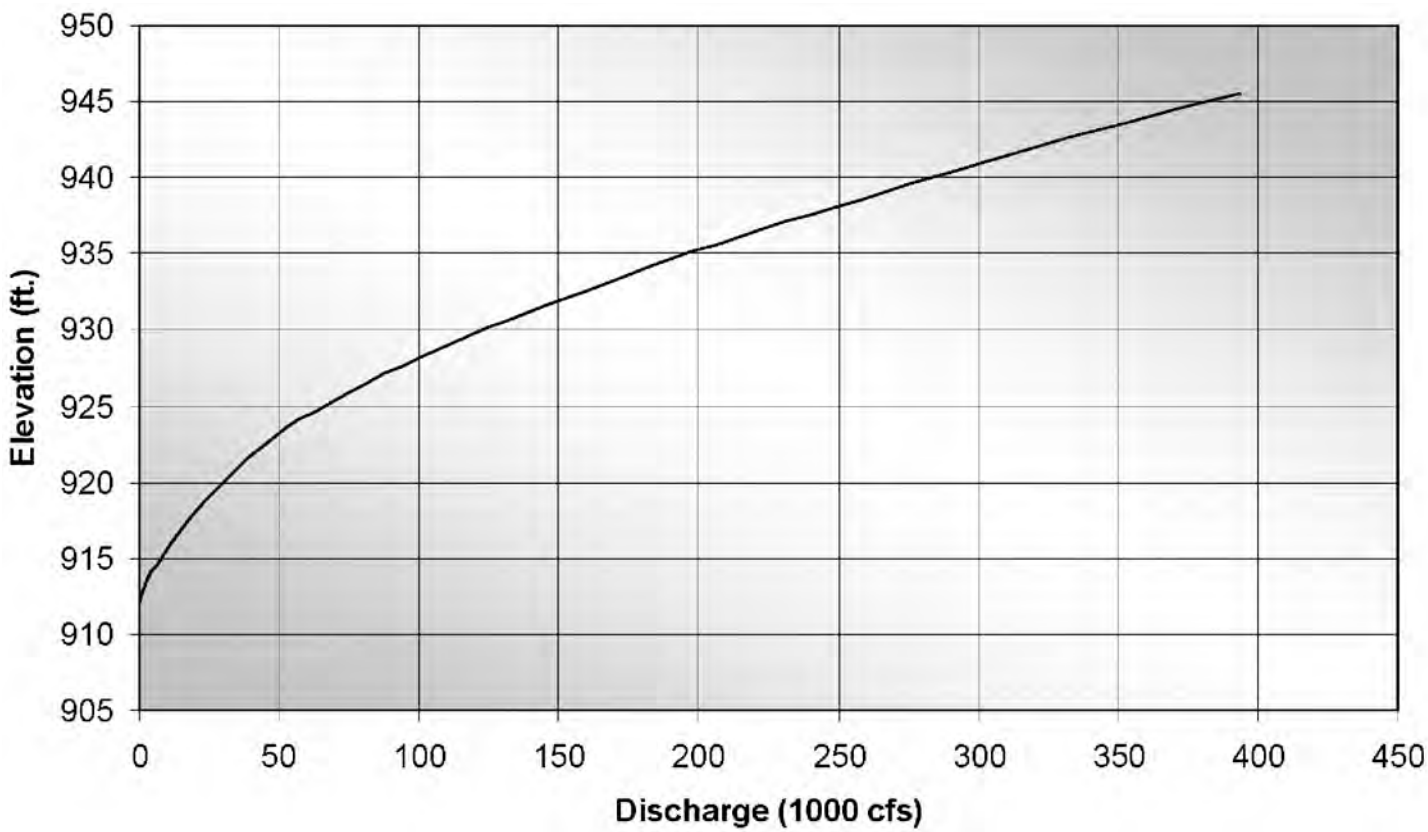


Figure 2.4.3-211
Discharge Rating Curve, Turner Shoals Dam

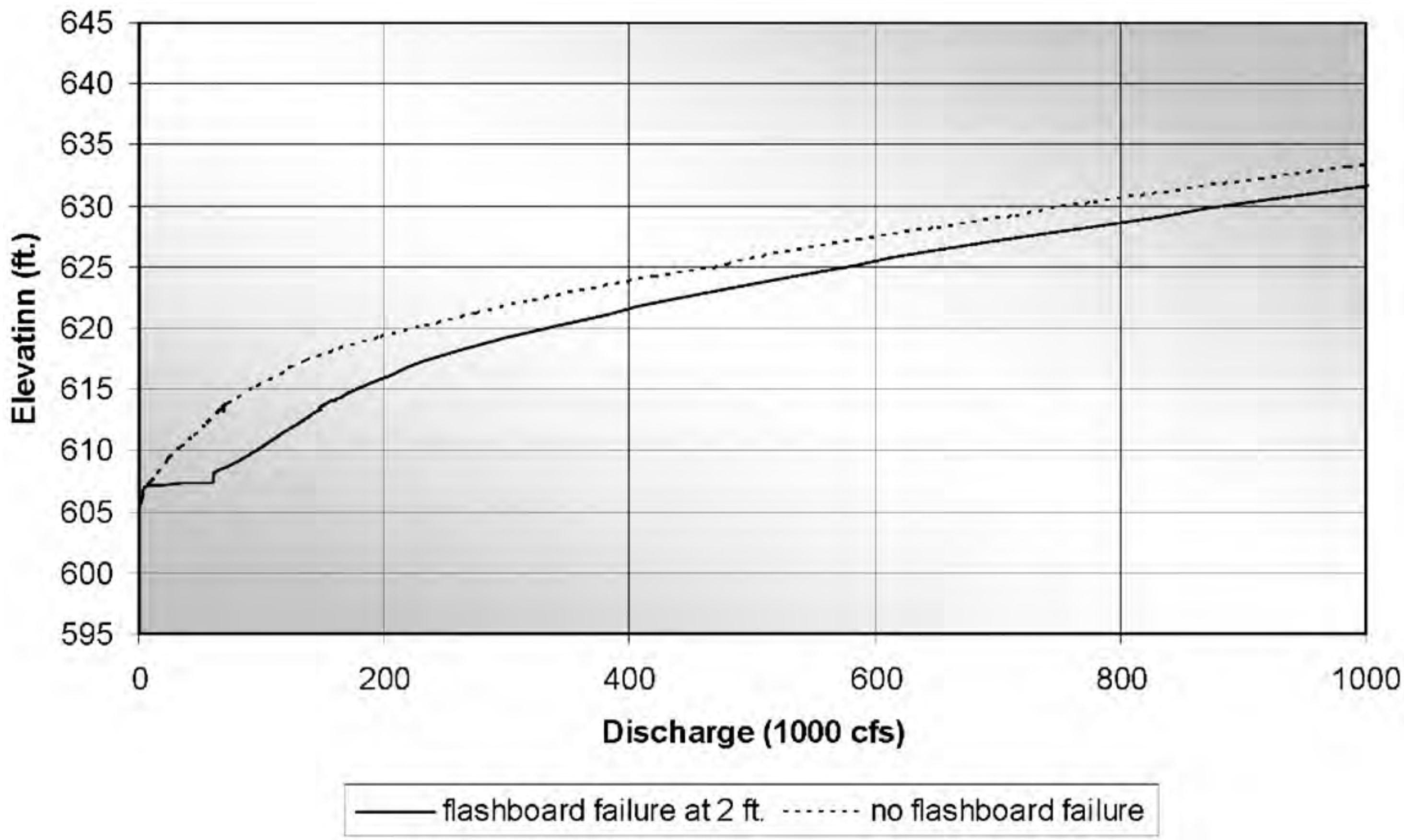


Figure 2.4.3-212
Discharge Rating Curve, Gaston Shoals Dam

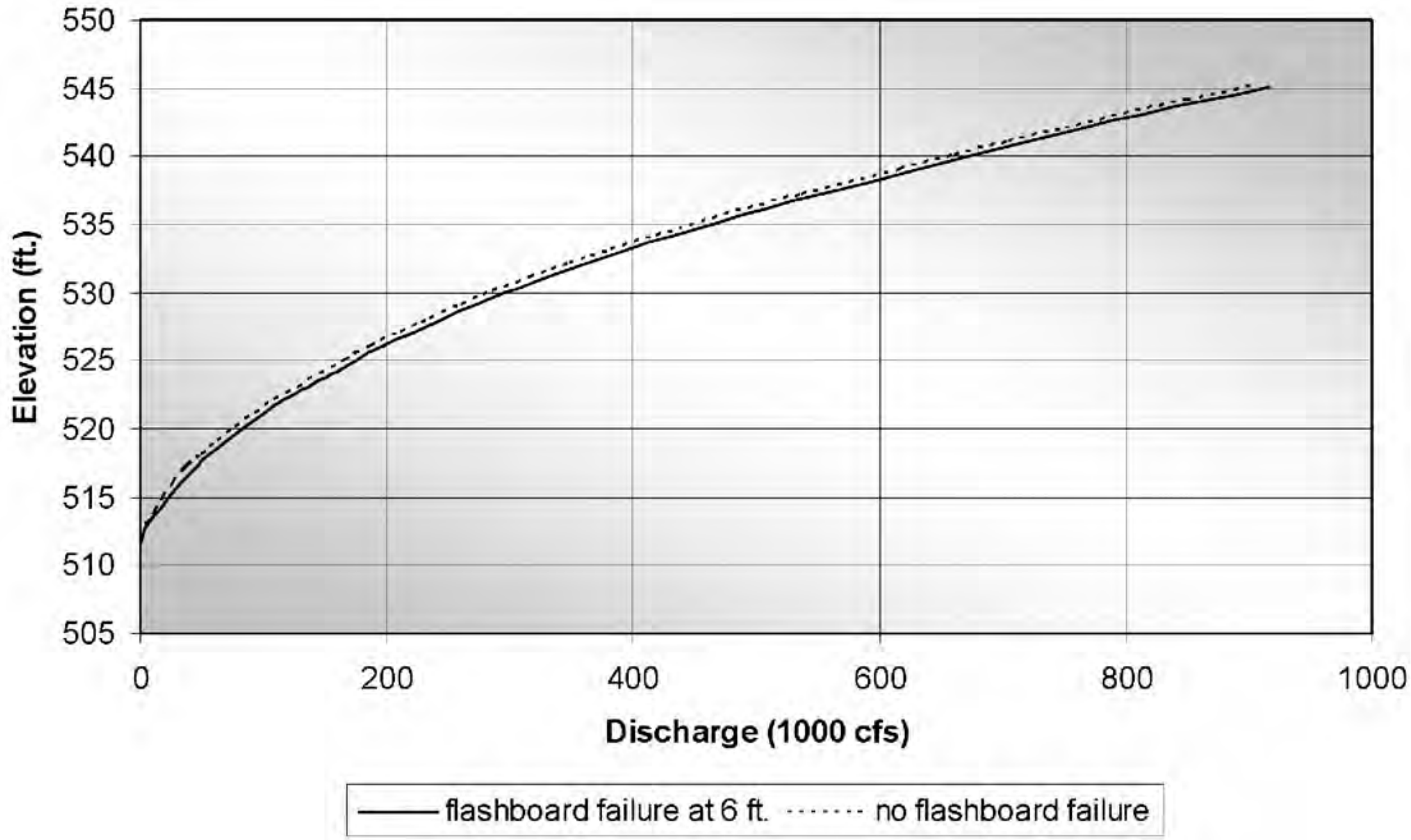


Figure 2.4.3-213
Discharge Rating Curve, Ninety-Nine Islands Dam

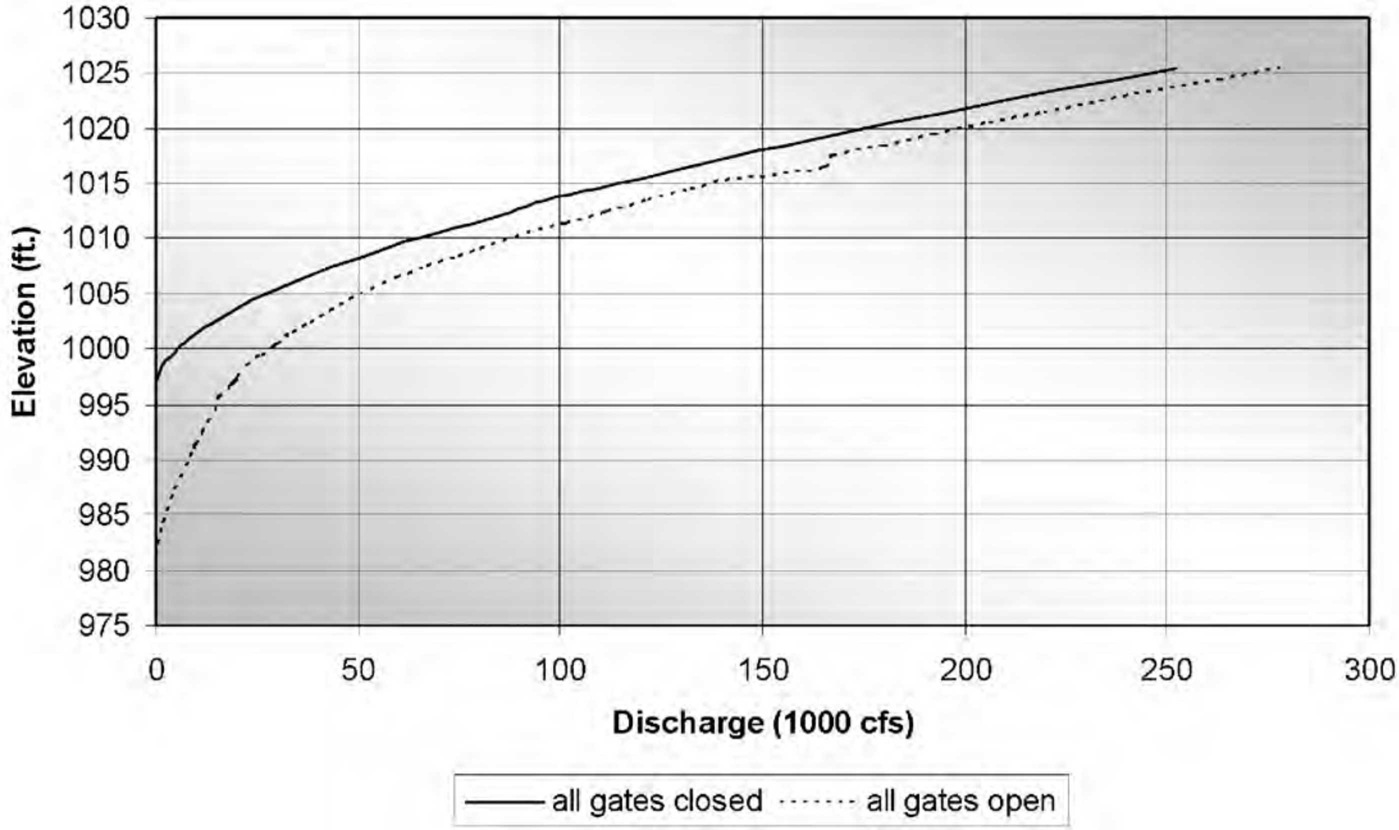


Figure 2.4.3-214
Discharge Rating Curve, Lake Lure Dam

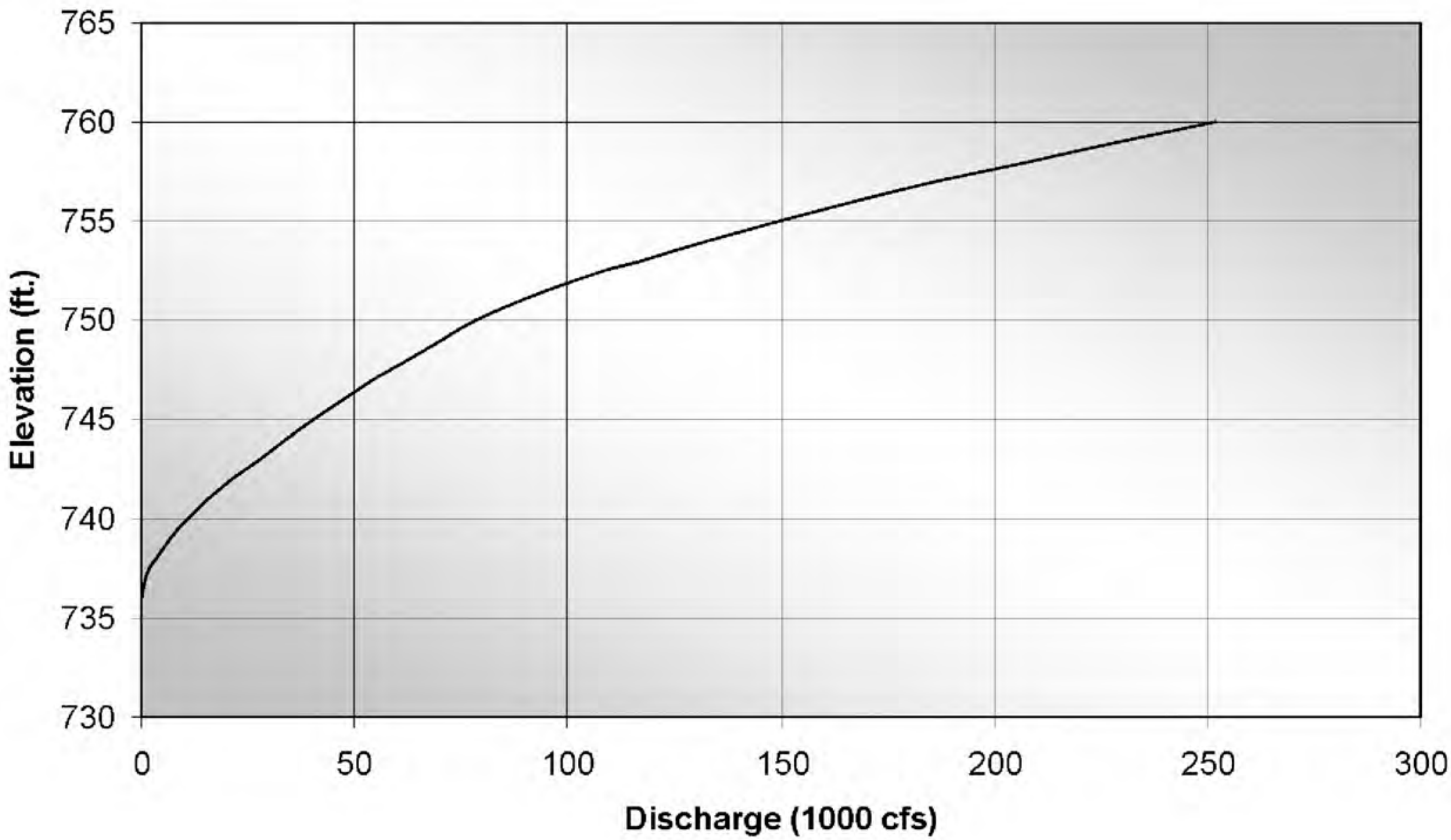


Figure 2.4.3-215
Discharge Rating Curve, Kings Mountain Reservoir Dam

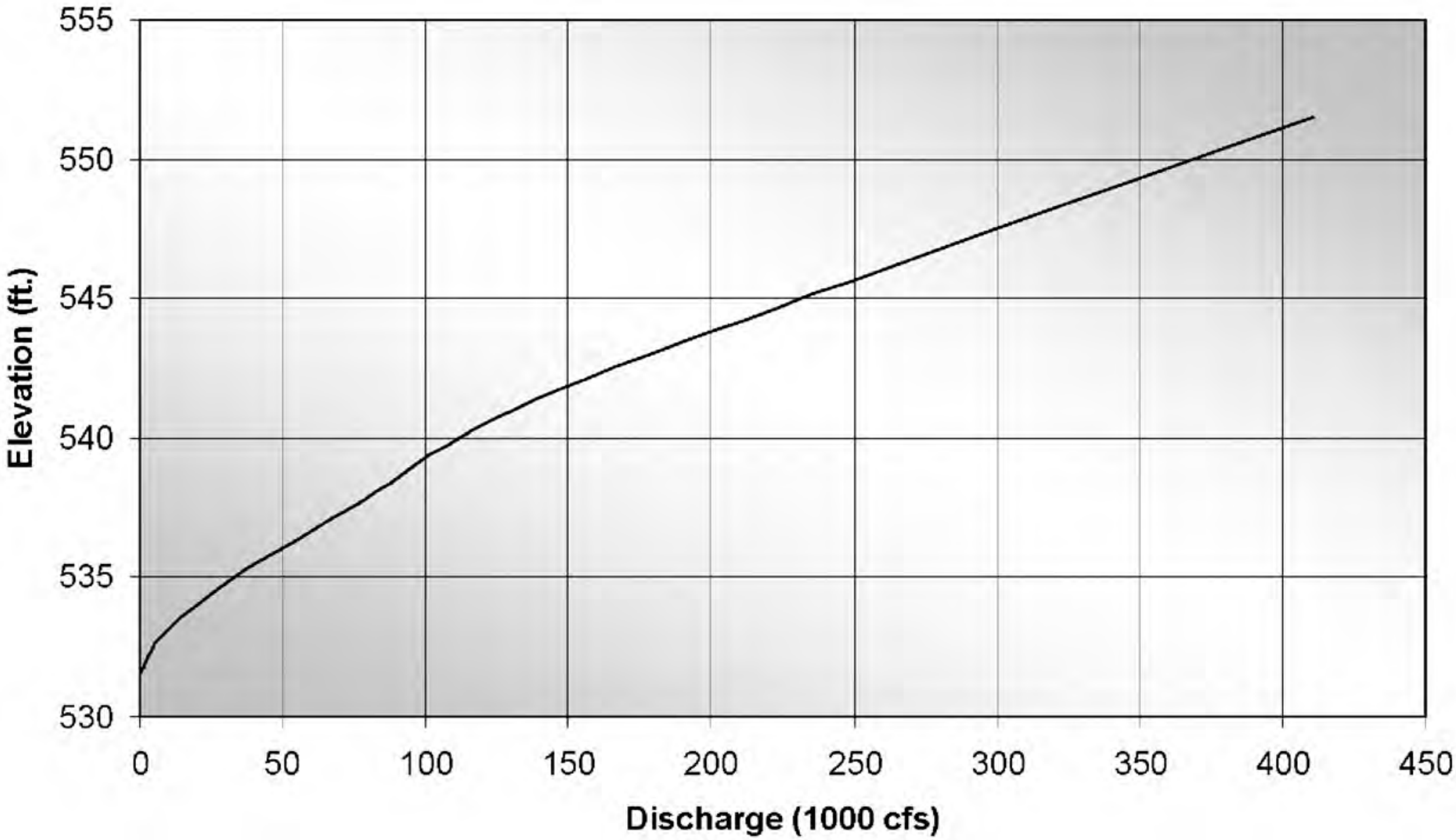


Figure 2.4.3-216
Discharge Rating Curve, Cherokee Falls Dam

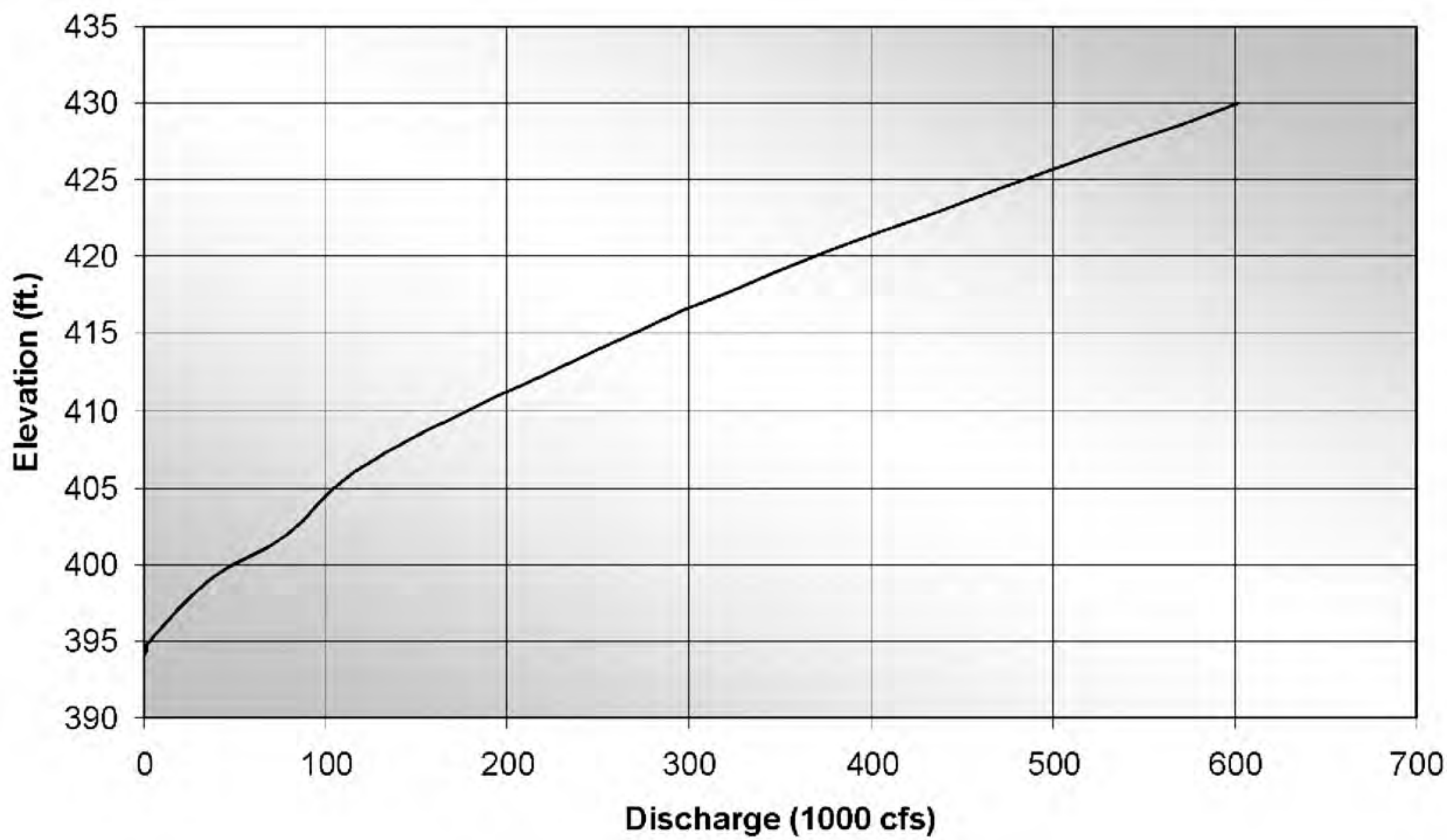


Figure 2.4.3-217
Discharge Rating Curve, Lockhart Dam

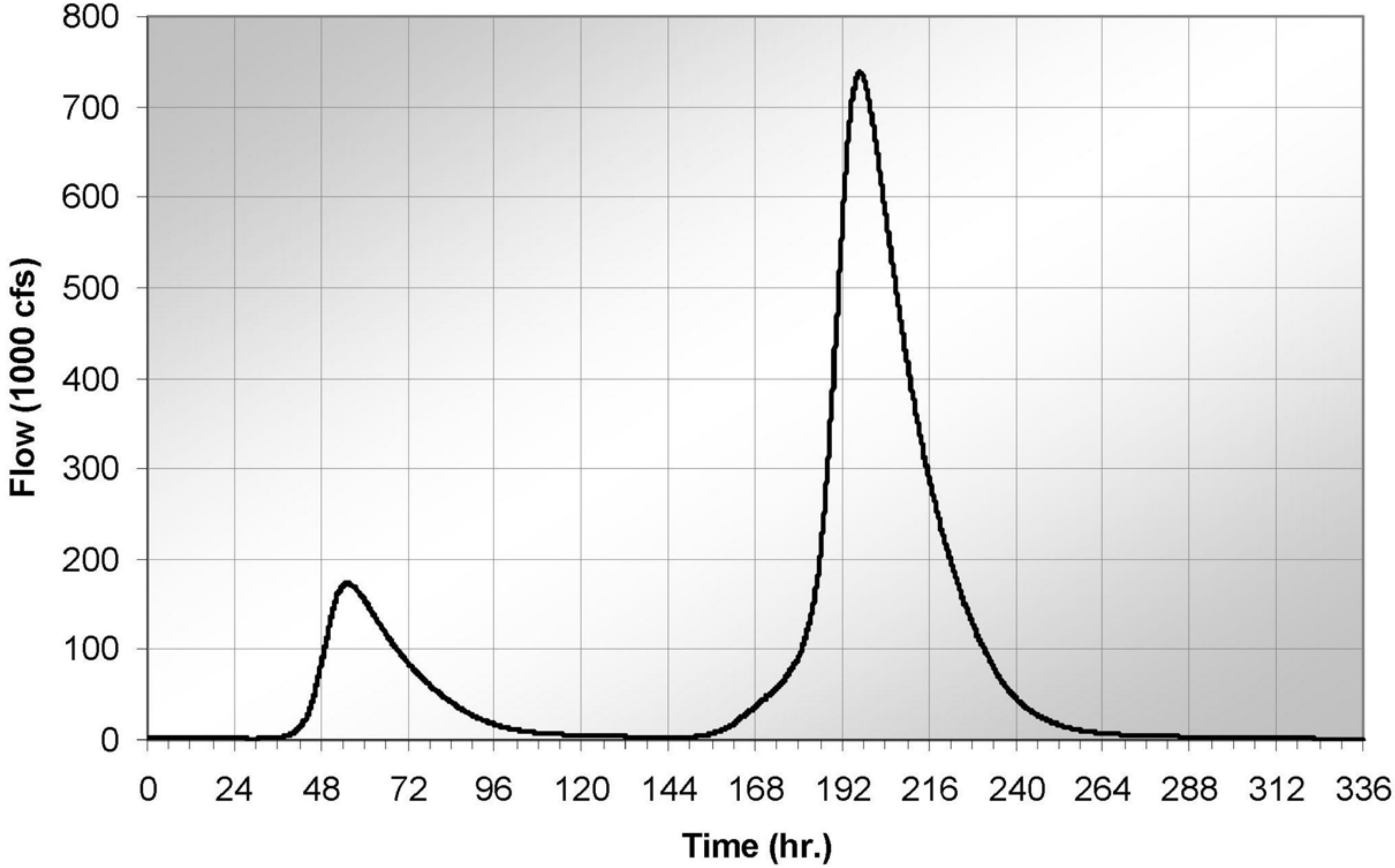
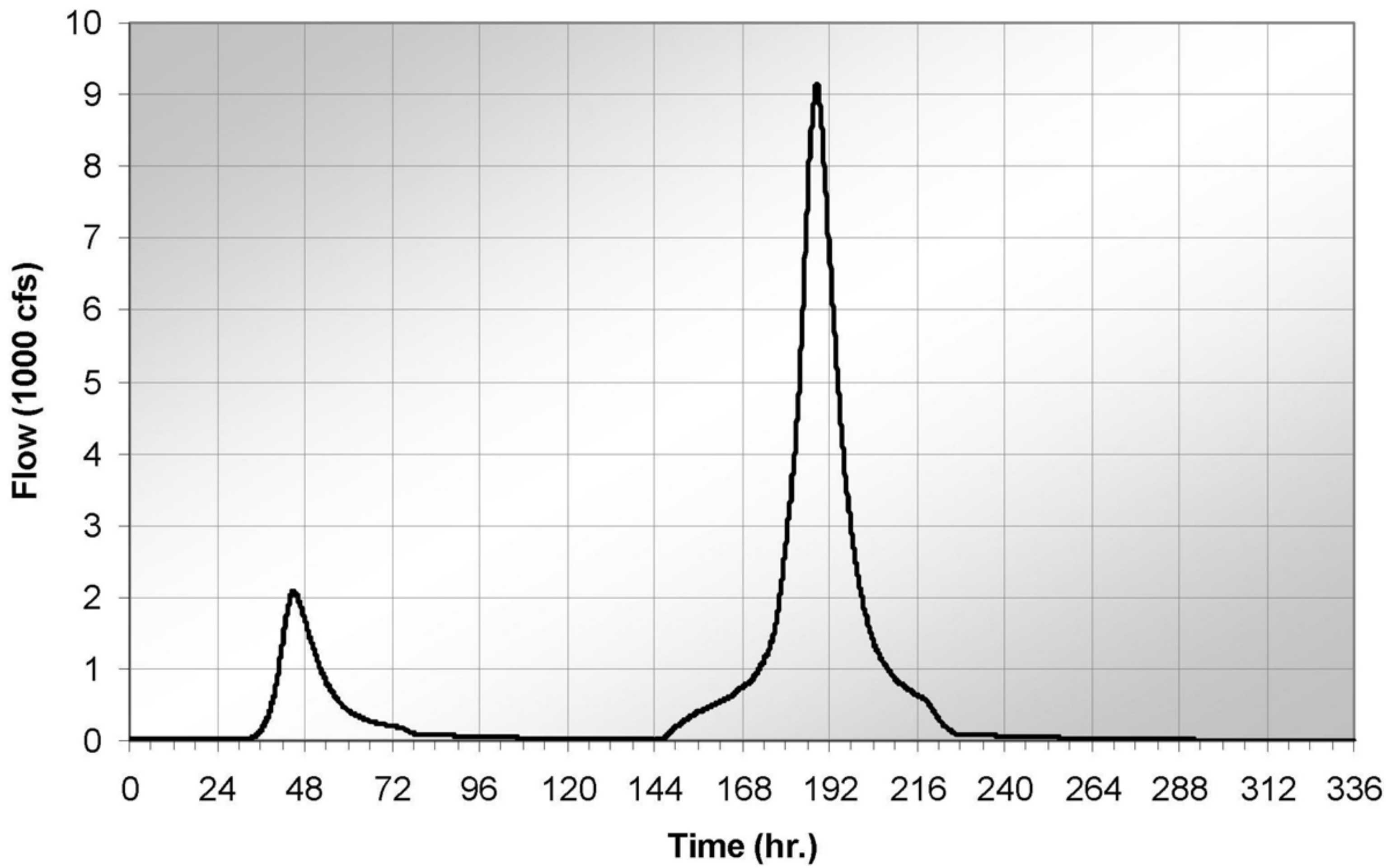
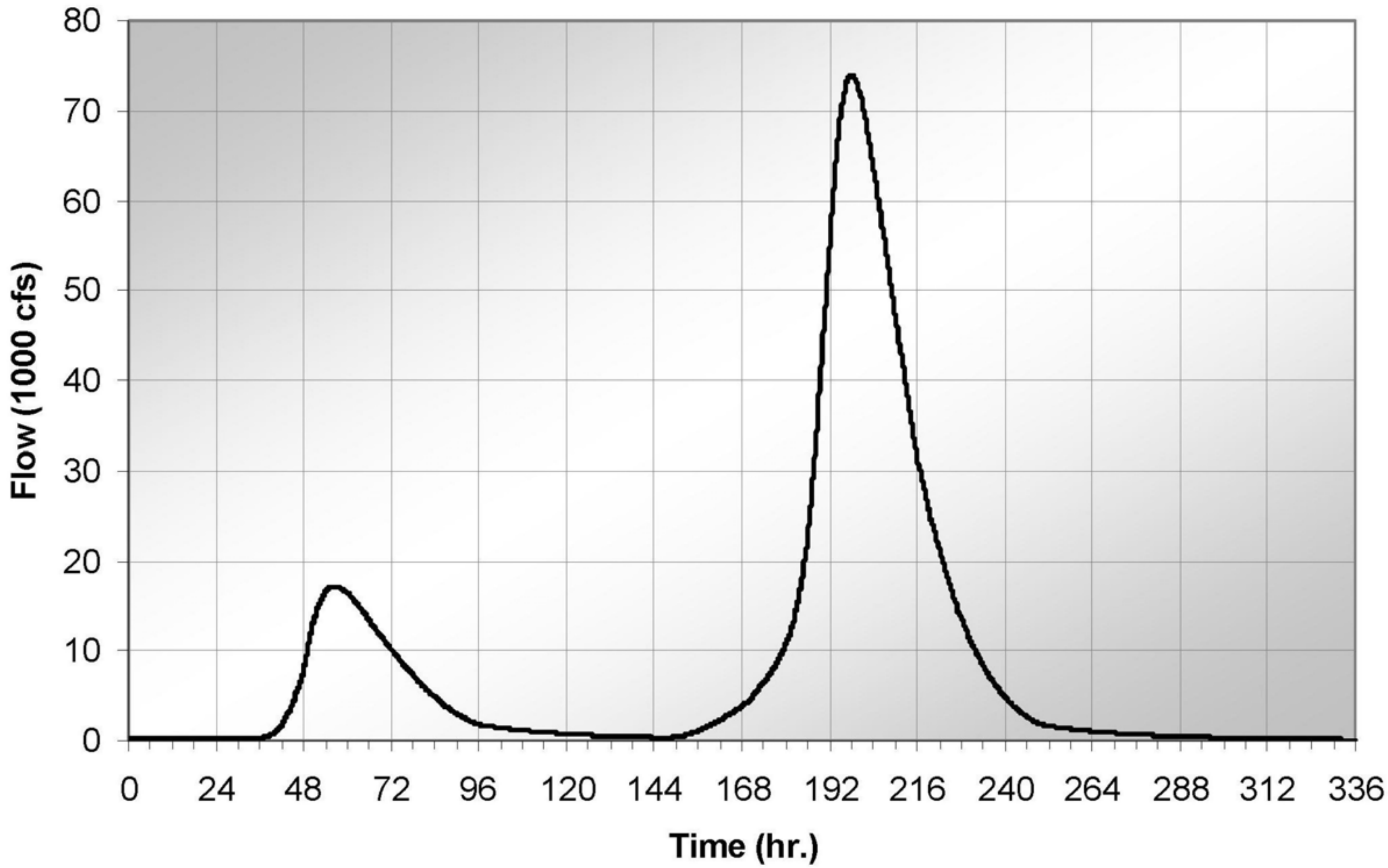


Figure 2.4.3-218
Gaston Shoals Dam PMF Combined Inflow Hydrograph



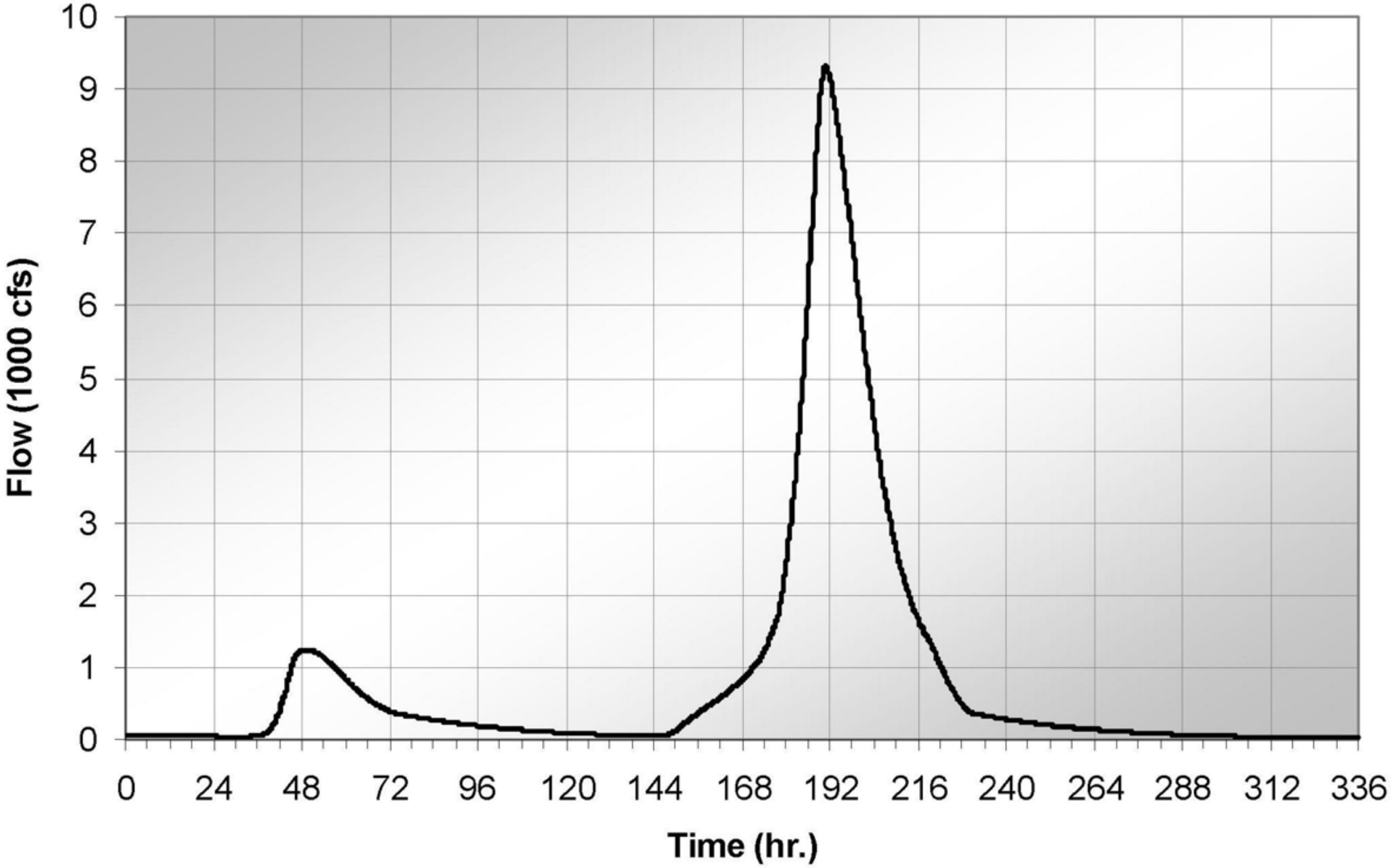
Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-219
Subbasin BD1-12 PMF Inflow Hydrograph



Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-220
Subbasin BC-14 and Routed Subbasin KMR-13 PMF Combined Inflow Hydrograph



Reference Figure 2.4.3-203 for Subbasin Locations

Figure 2.4.3-221
Subbasin BR-15 PMF Inflow Hydrograph

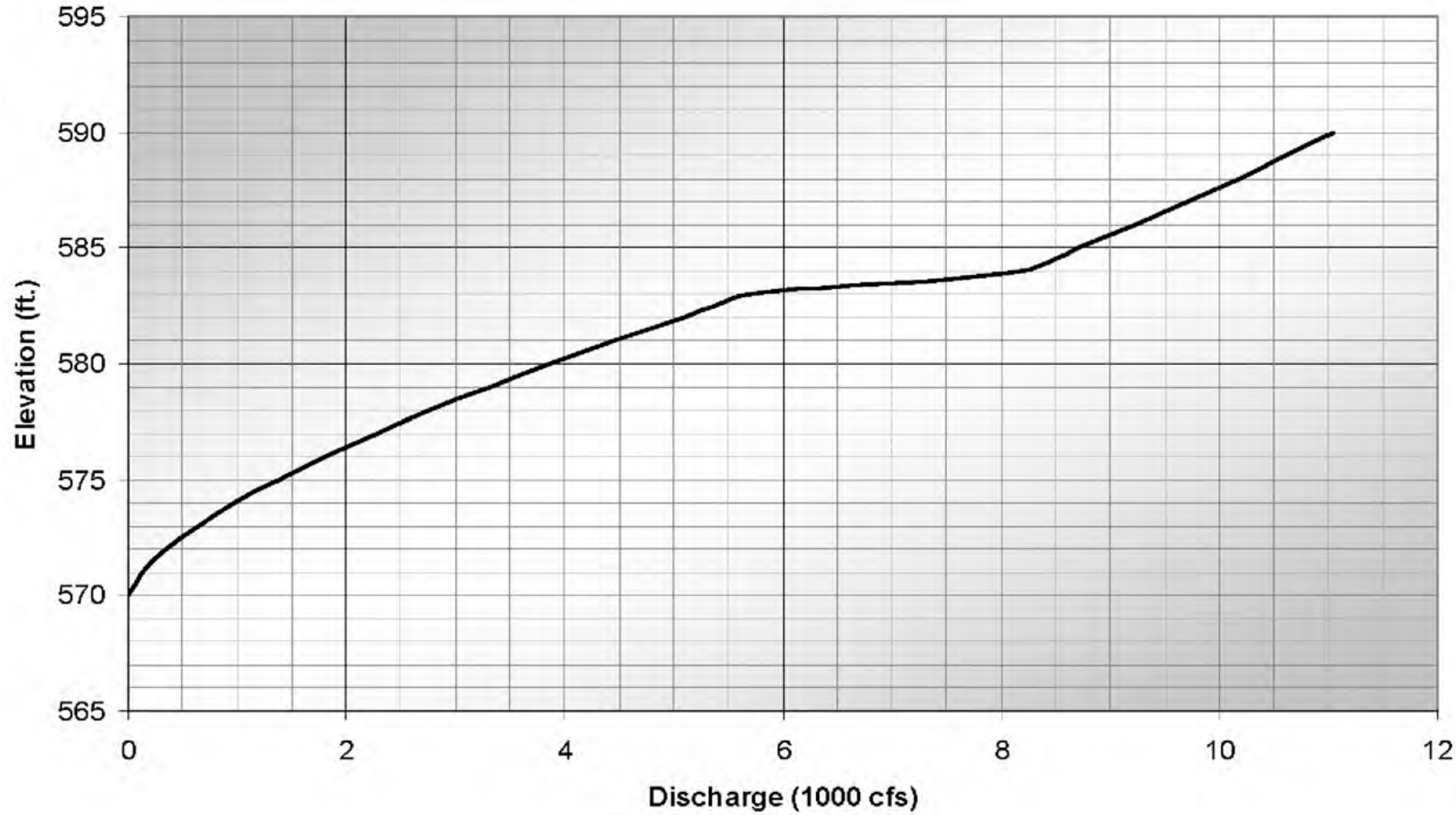


Figure 2.4.3-222
Discharge Rating Curve, Make-Up Pond B

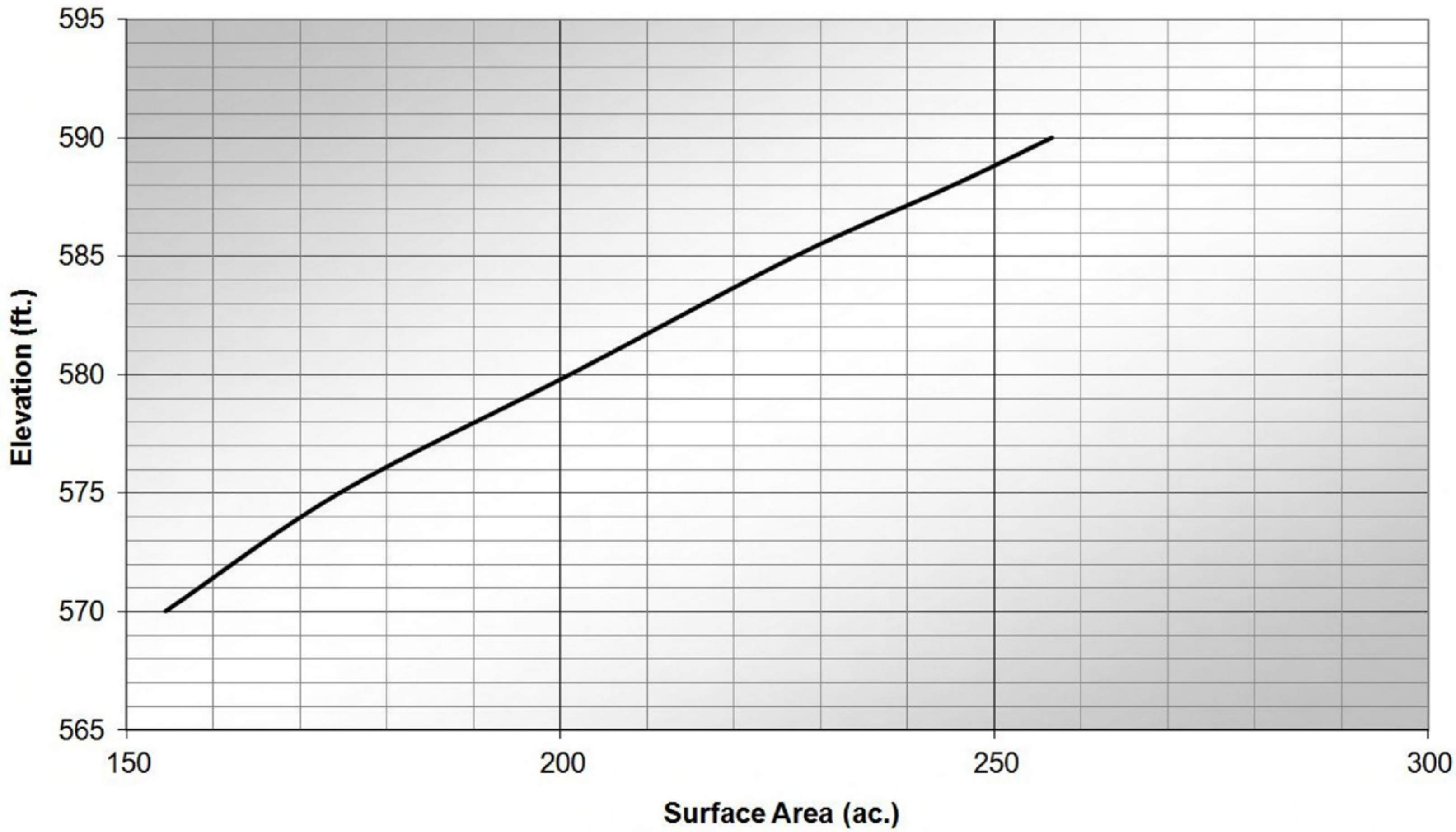


Figure 2.4.3-223
Storage Capacity Curve, Make-Up Pond B

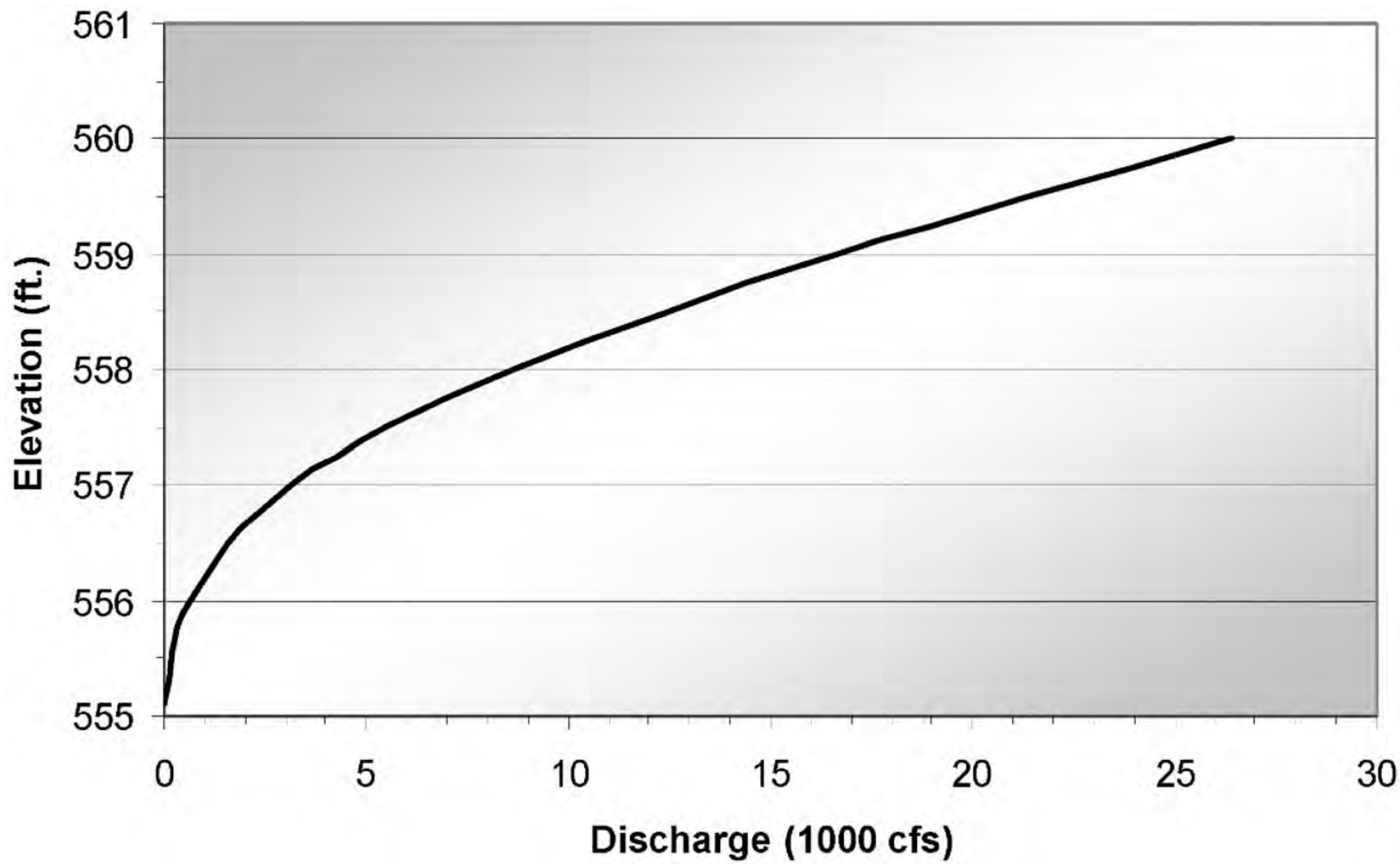


Figure 2.4.3-224
Discharge Rating Curve, Make-Up Pond A

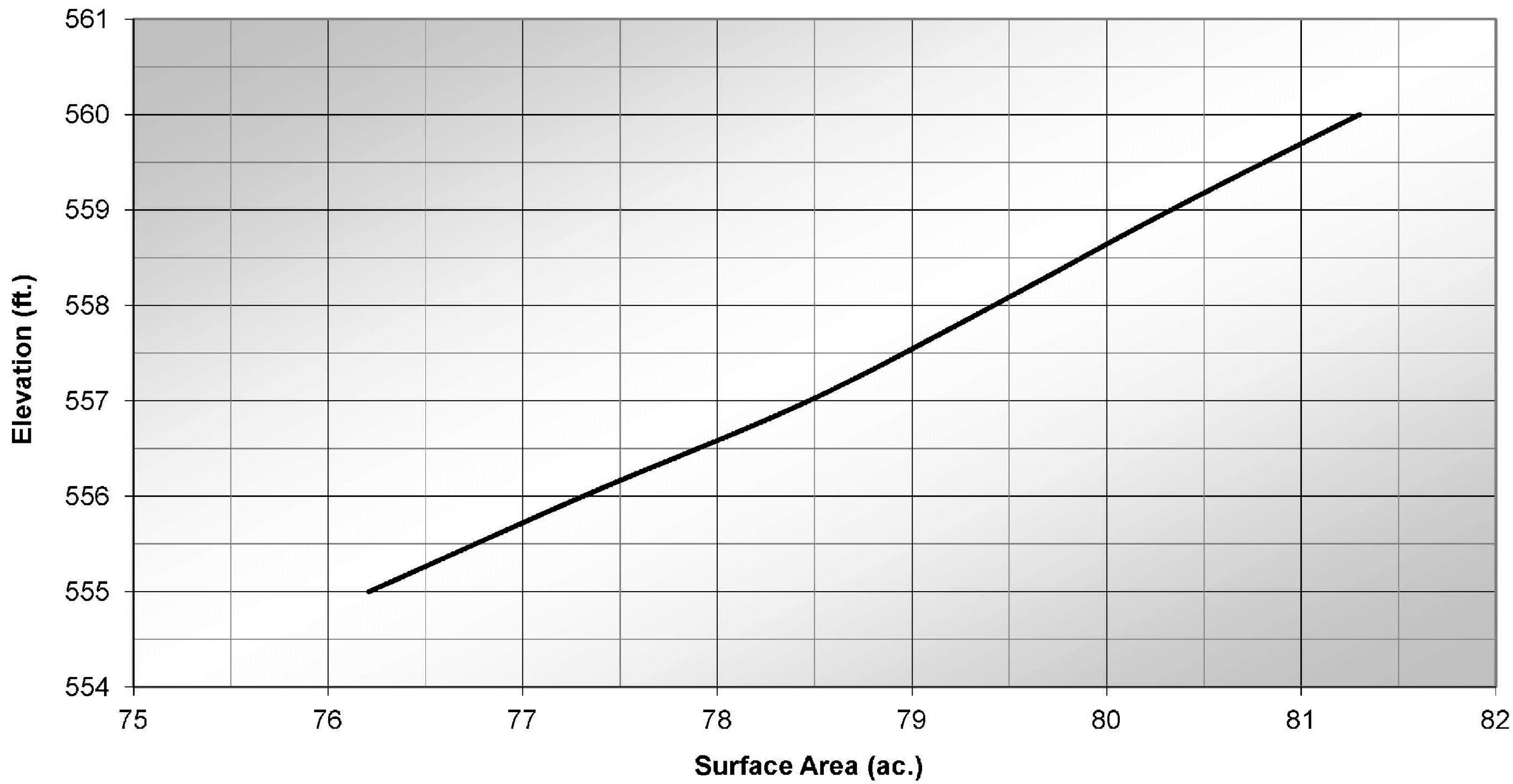


Figure 2.4.3-225
Storage Capacity Curve, Make-Up Pond A

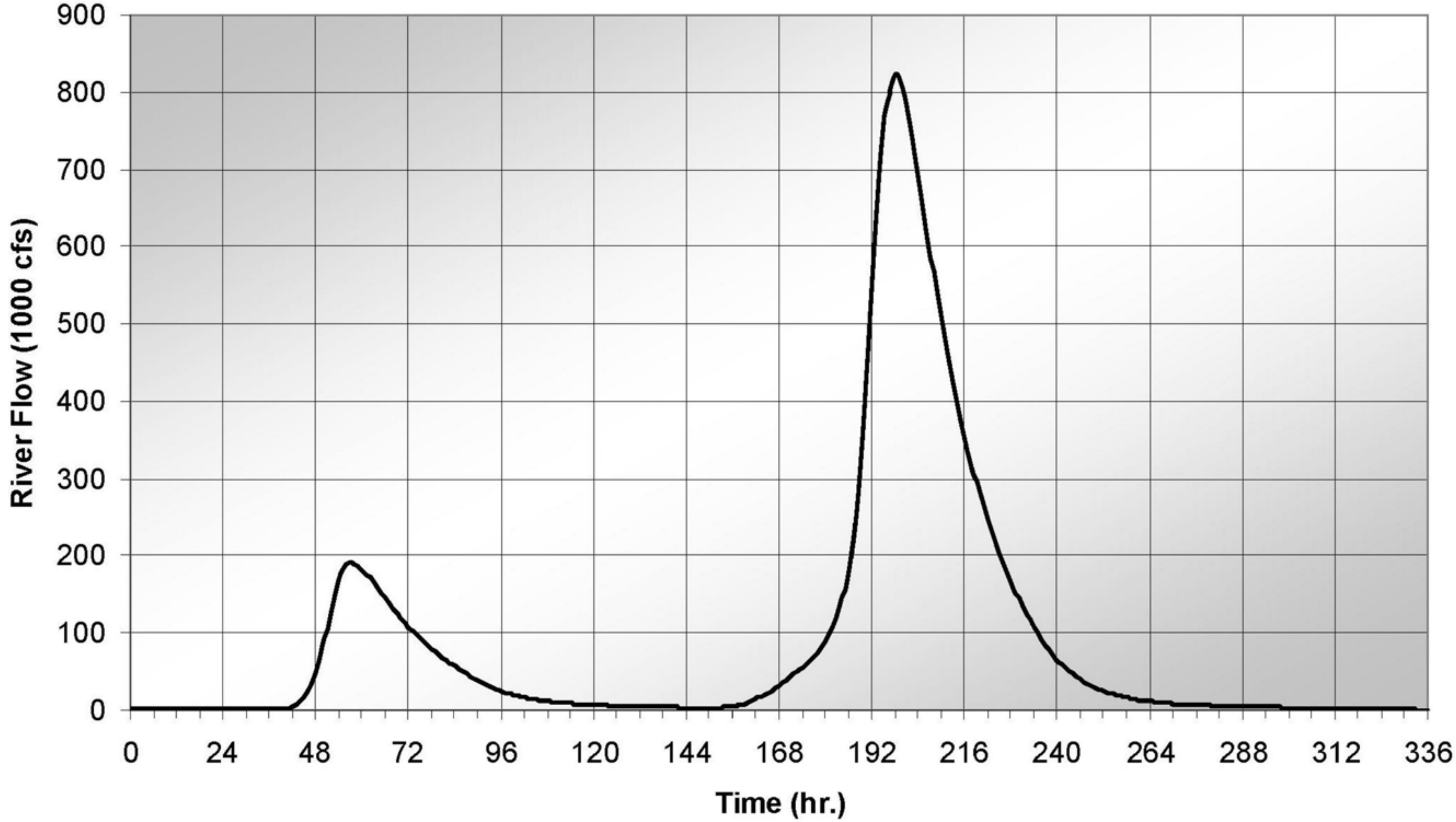


Figure 2.4.3-226
PMF Hydrograph, Broad River at Lee Nuclear Station

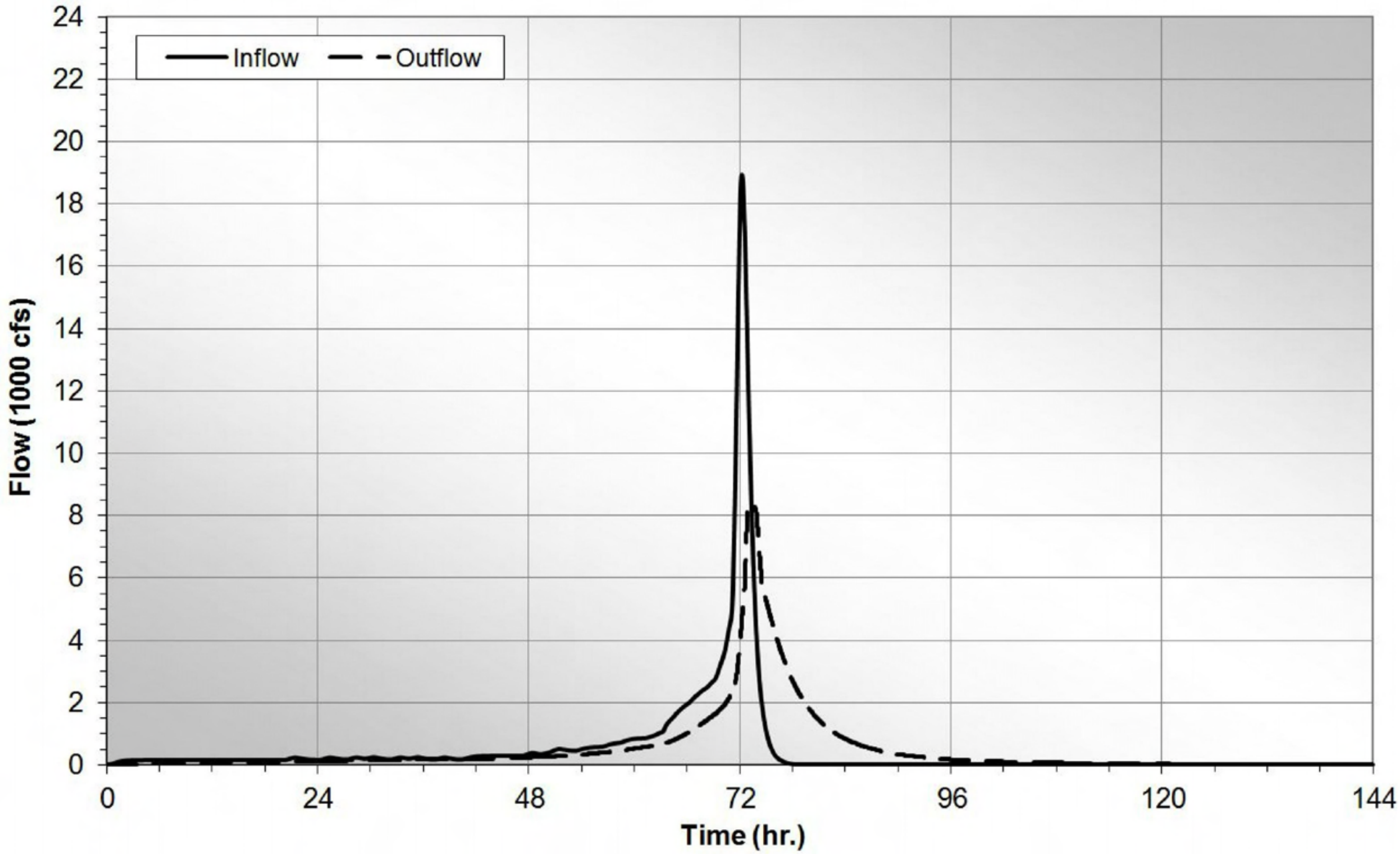


Figure 2.4.3-227
PMF Hydrograph Without Upper Arm Dam Failure, Make-Up Pond B

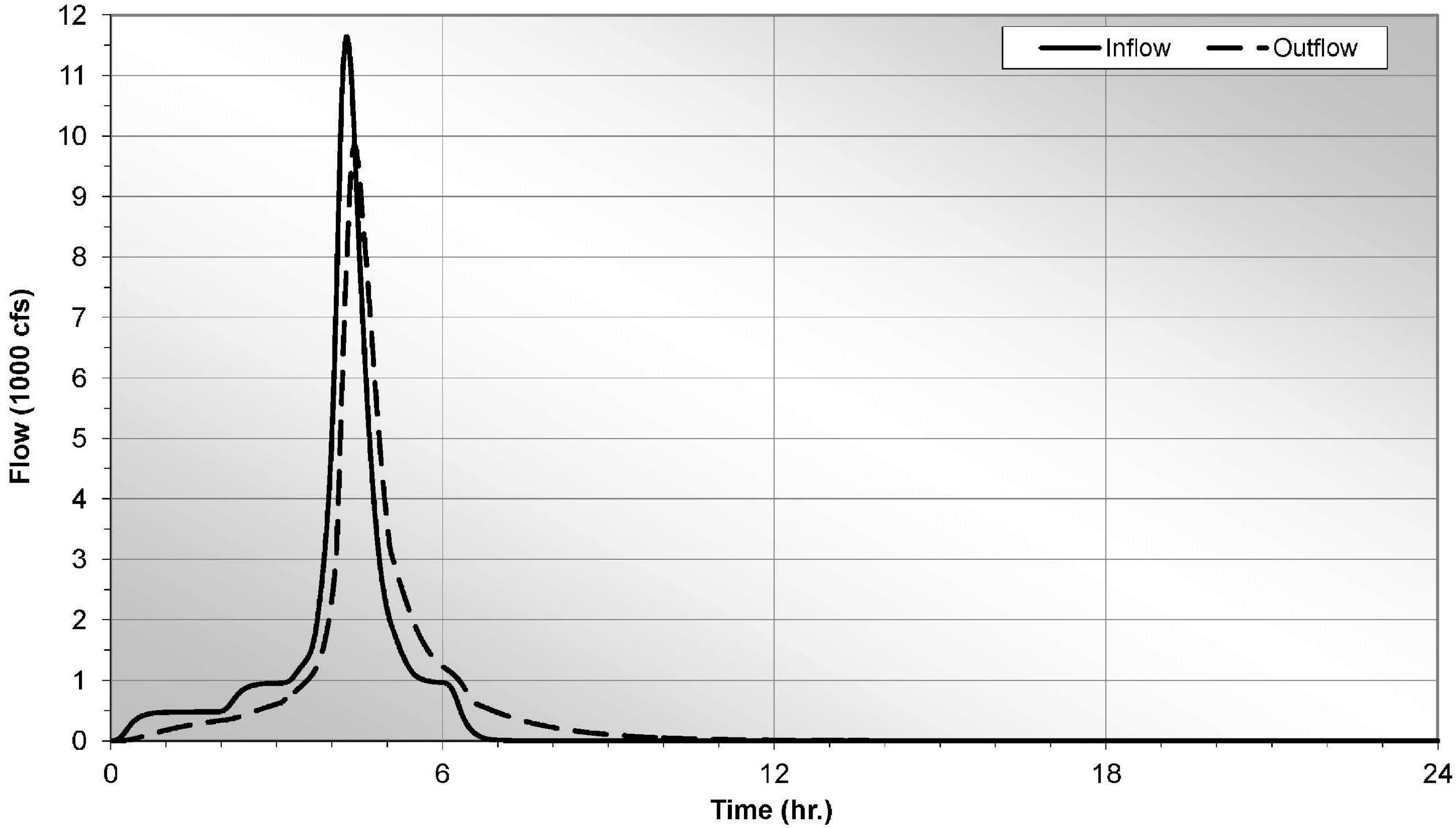


Figure 2.4.3-228
PMF Hydrograph Make-Up Pond A

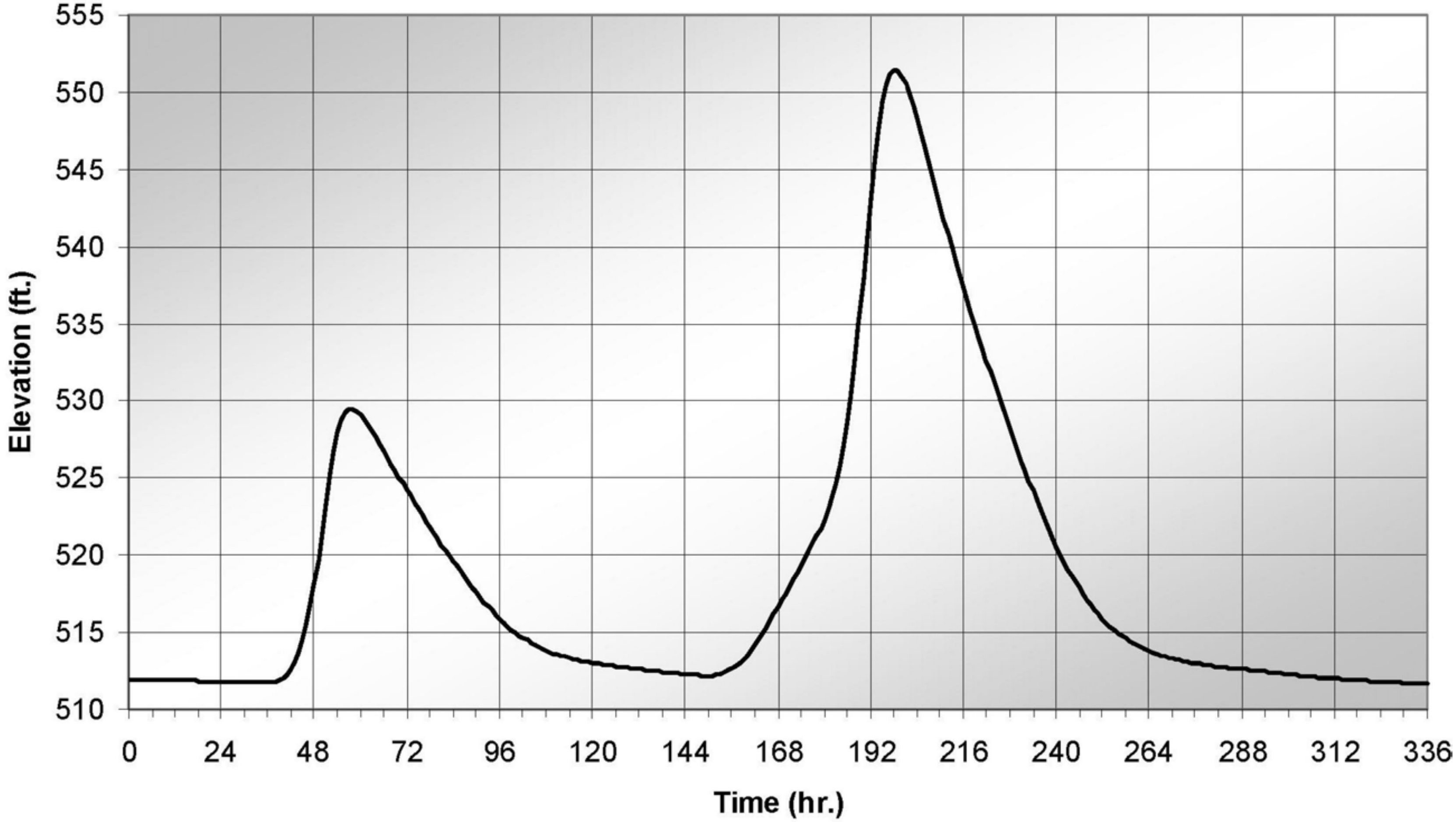


Figure 2.4.3-229
PMF Elevation Hydrograph, Broad River at Lee Nuclear Station

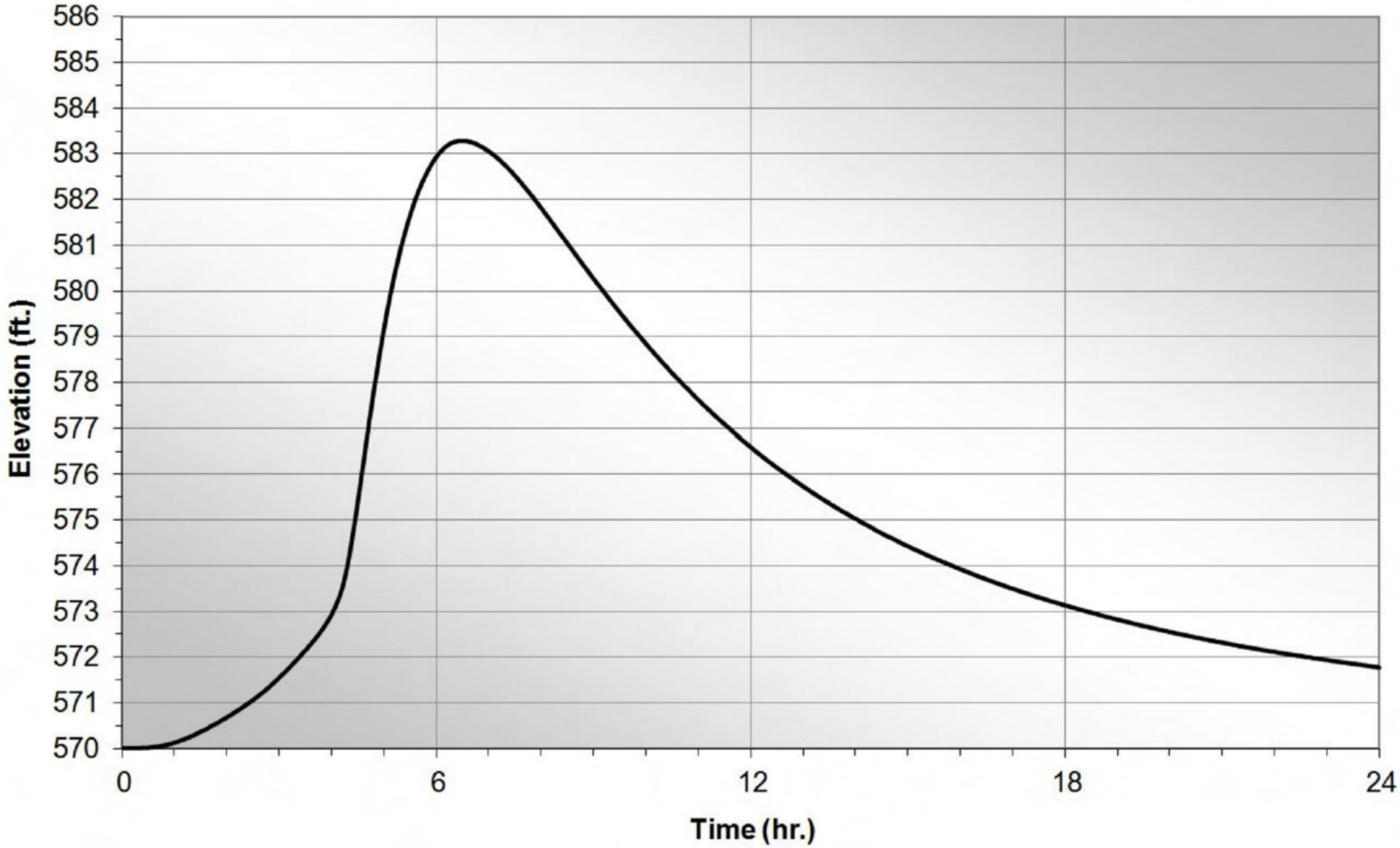


Figure 2.4.3-230
Flood El. Hydrograph Make-Up Pond B Without Upper Arm Dam Failure, 6-hr. Local Intense Precipitation

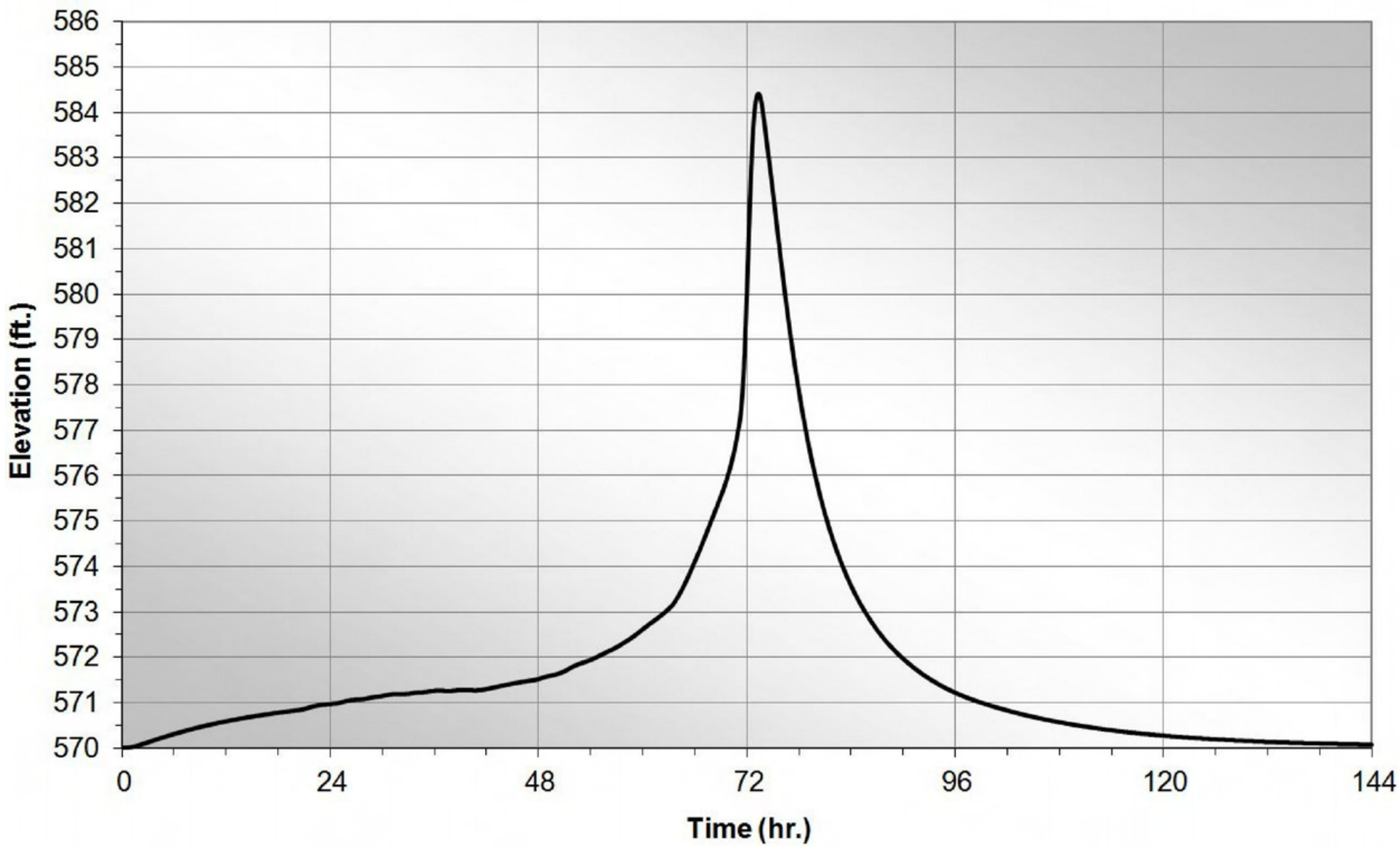


Figure 2.4.3-231
Flood El. Hydrograph Make-Up Pond B Without Upper Arm Dam Failure, 72-hr. Local Intense Precipitation

Figure 2.4.3-232
Not Used

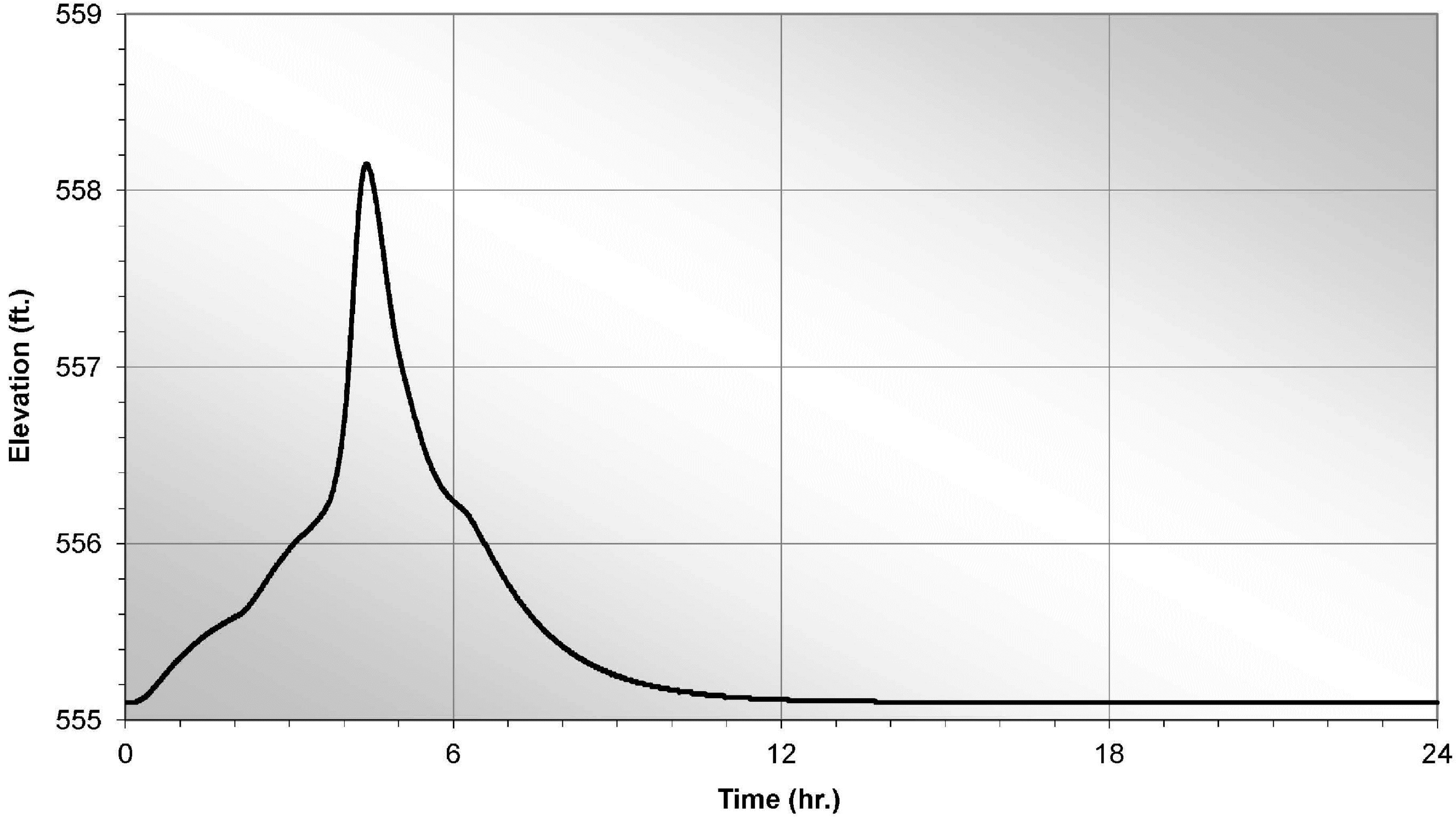


Figure 2.4.3-233
Flood Elevation Hydrograph Make-Up Pond A 6-Hour Local Intense Precipitation

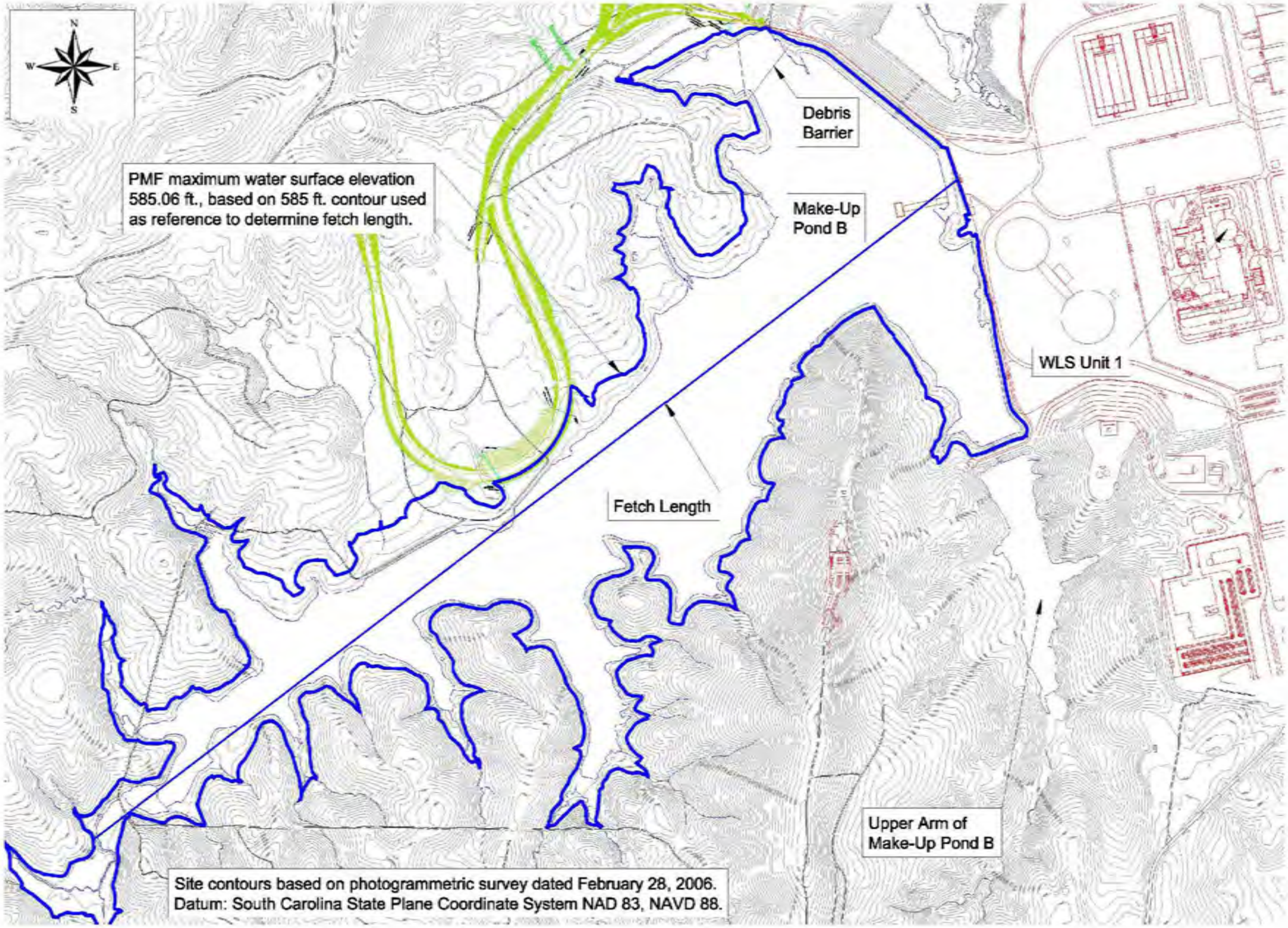


Figure 2.4.3-234
Make-Up Pond B Coincident Wind Wave Fetch Length

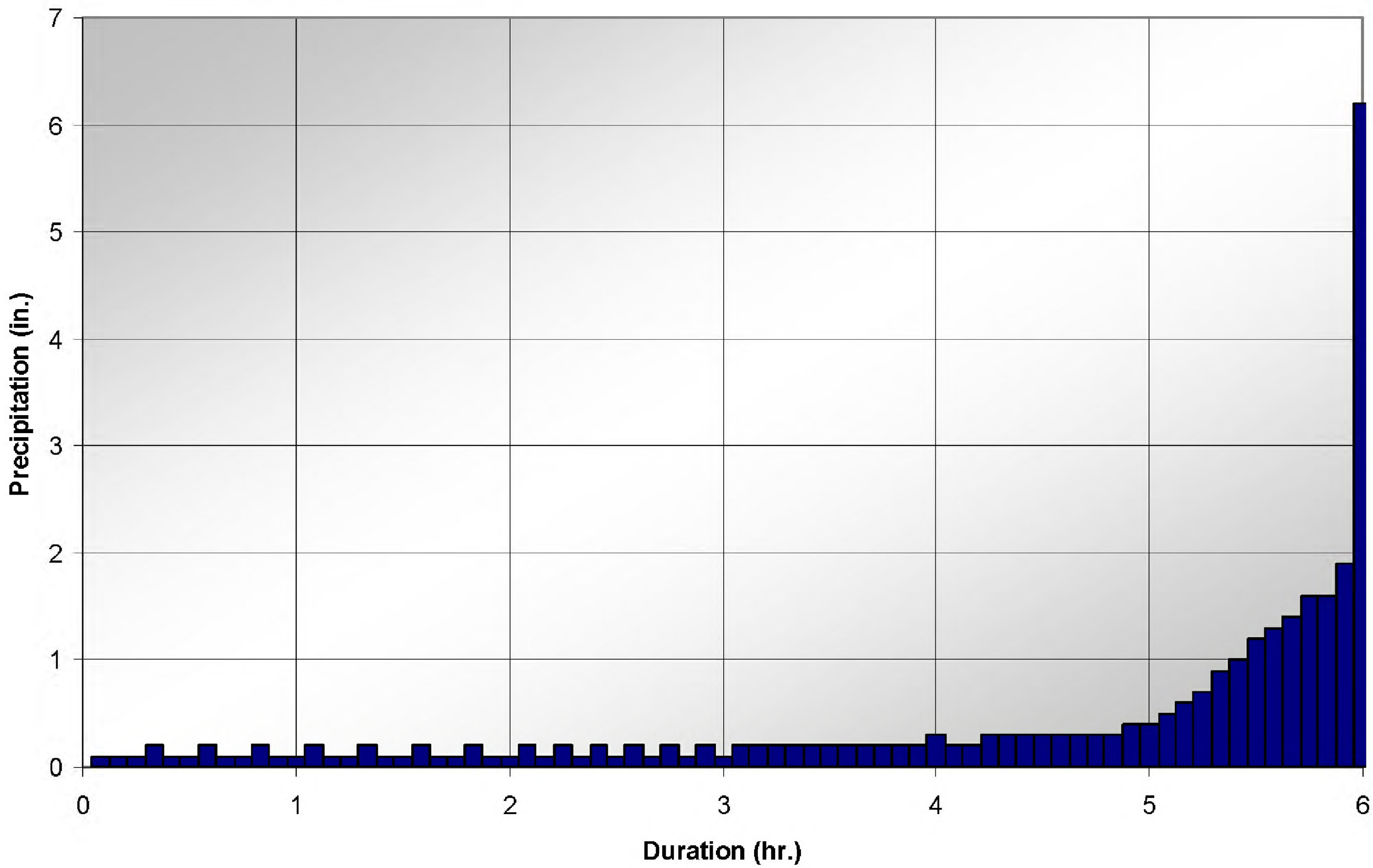


Figure 2.4.3-235
Local Intense Probable Maximum Precipitation 6-Hour End Peaking Hyetograph

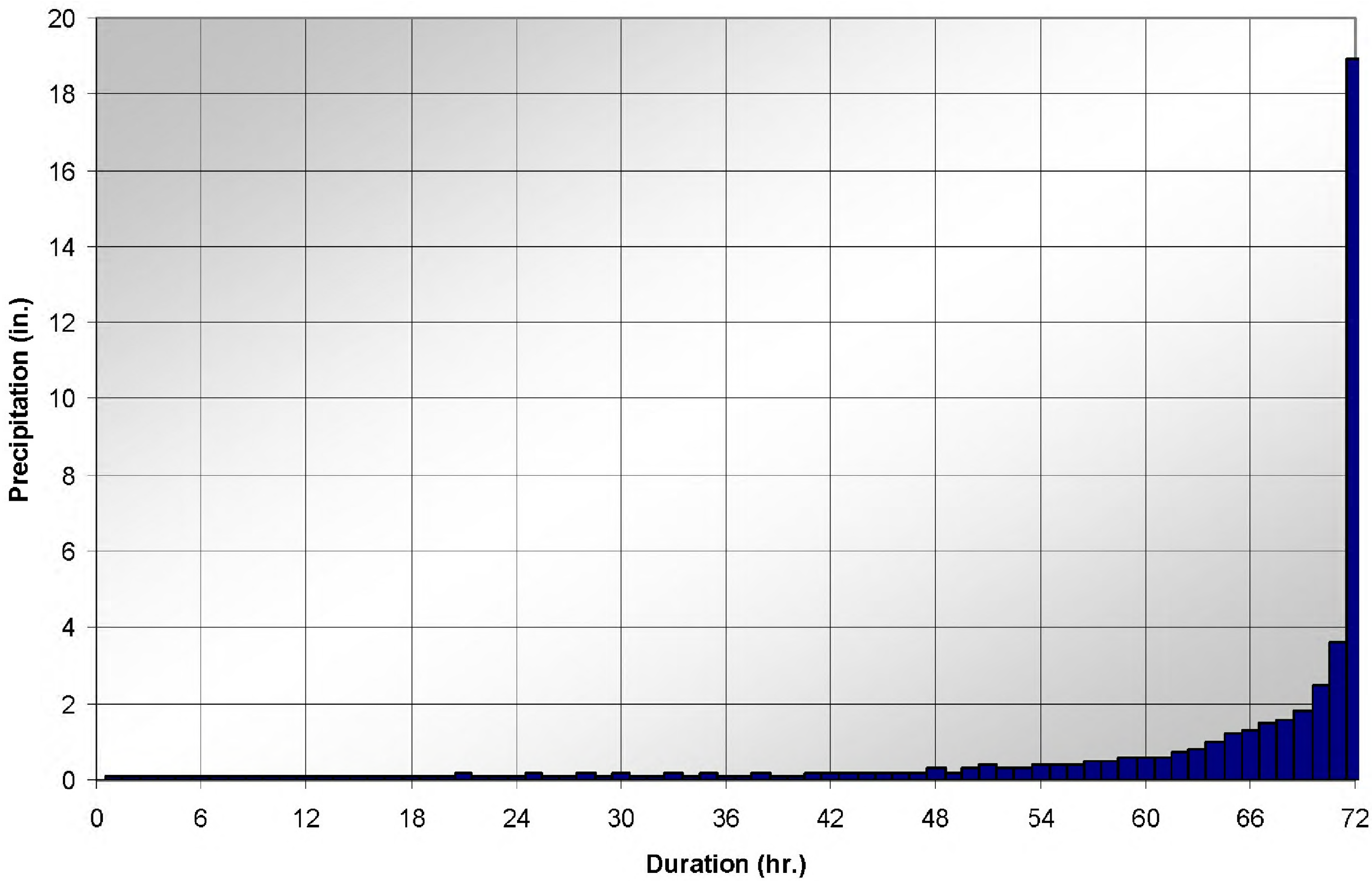


Figure 2.4.3-236
Local Intense Probable Maximum Precipitation 72-Hour End Peaking Hyetograph

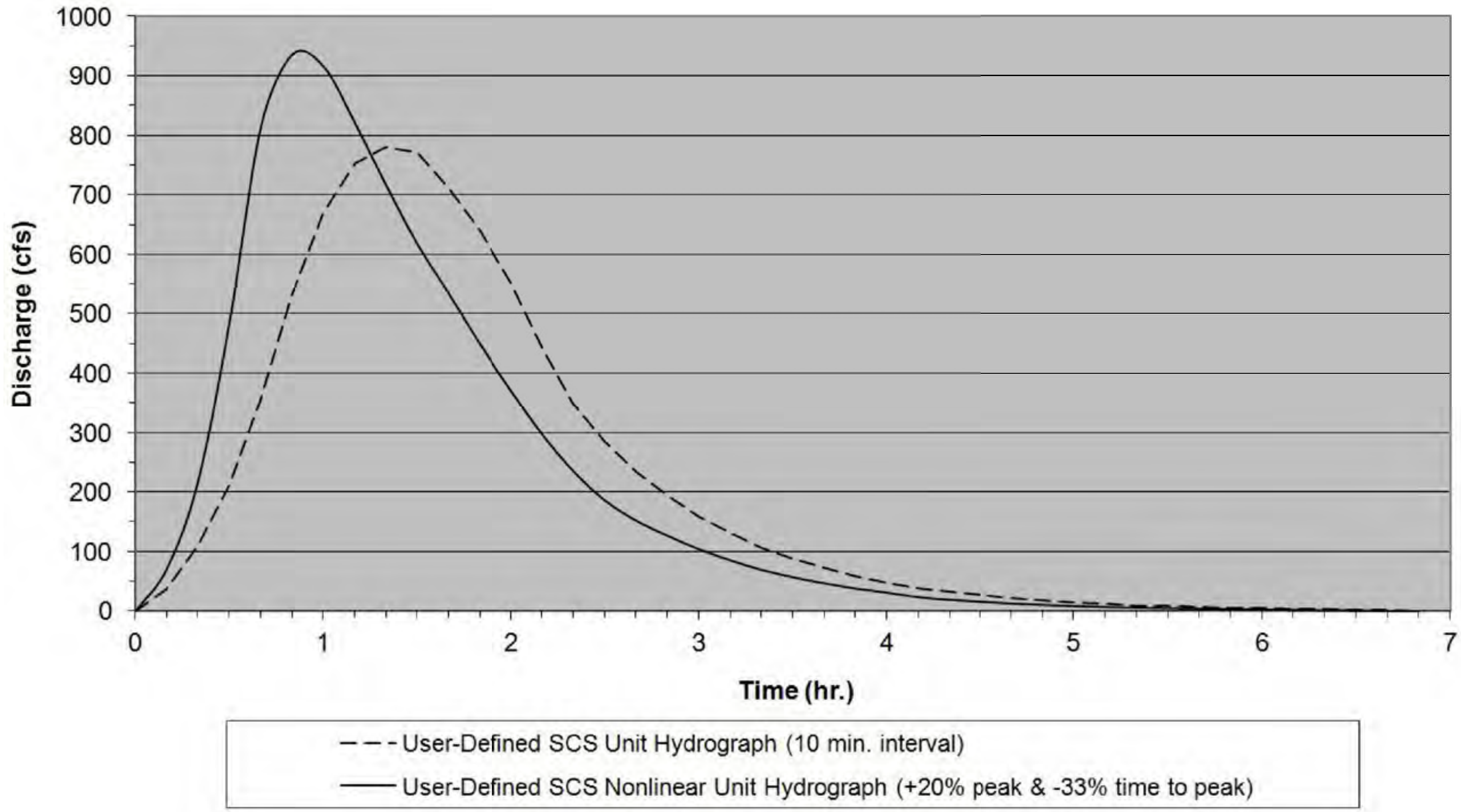


Figure 2.4.3-237
Make-Up Pond B Unit Hydrographs

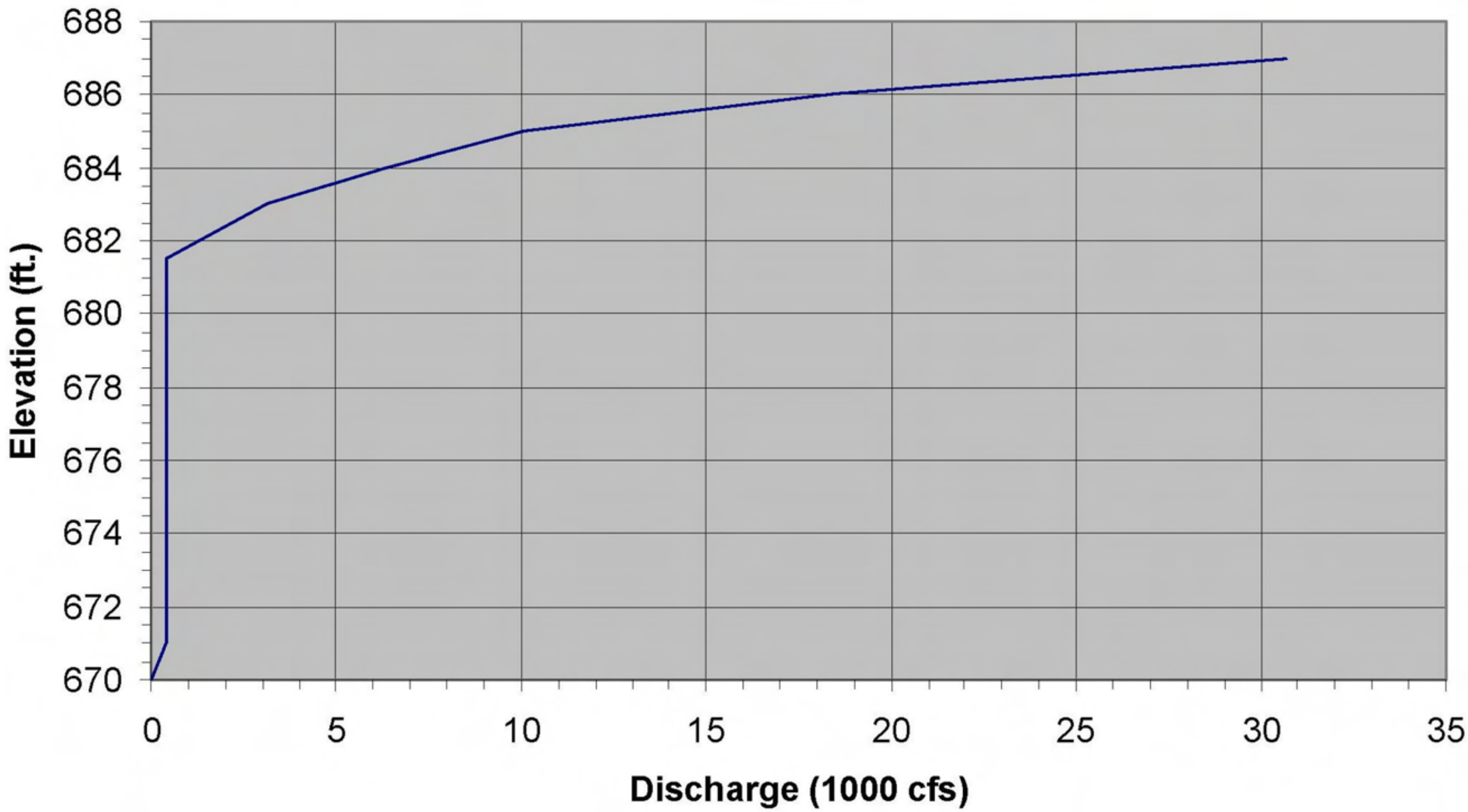


Figure 2.4.3-238
Discharge Rating Curve, Lake Whelchel

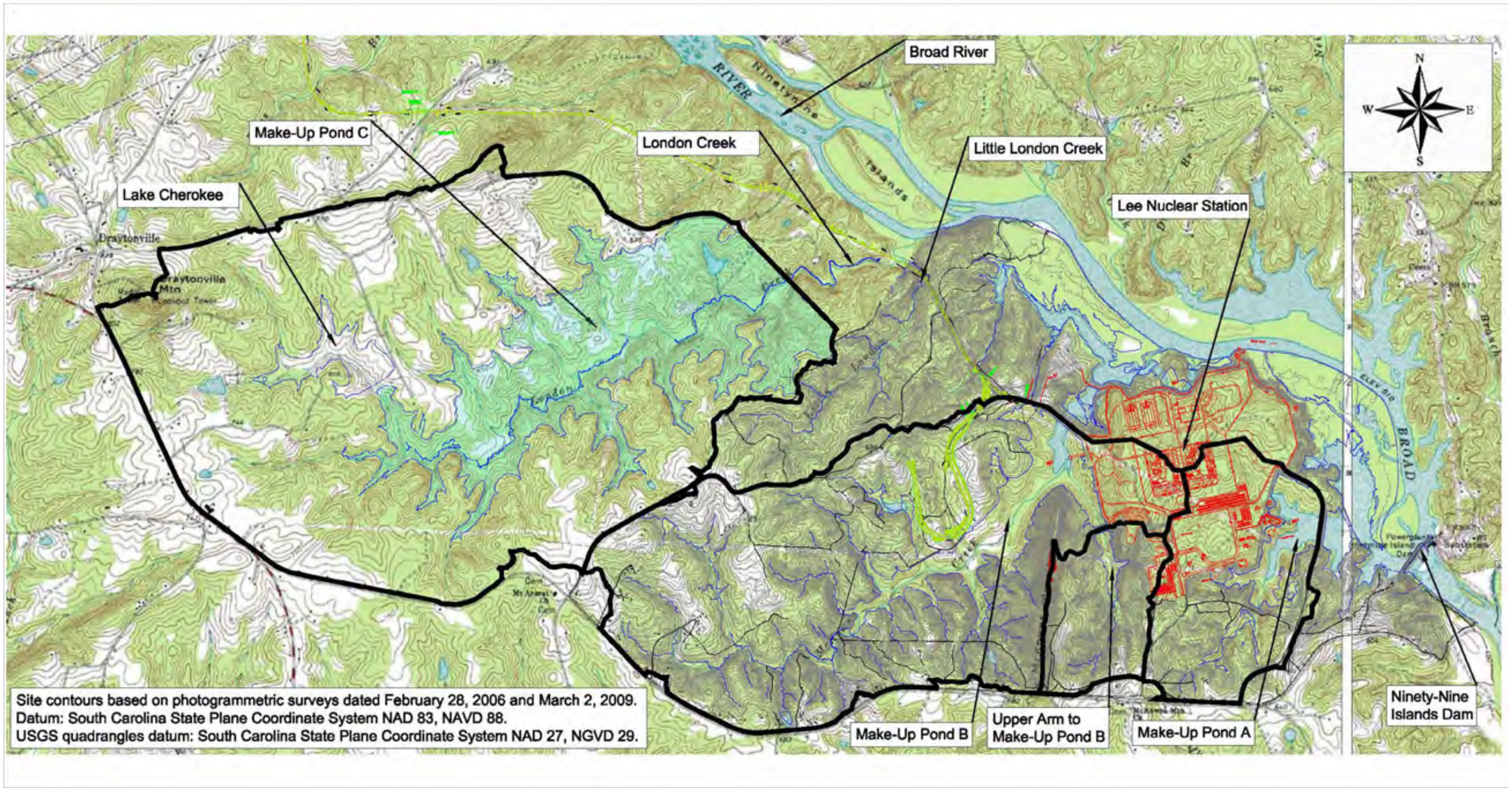


Figure 2.4.3-239
Make-Up Pond C Watershed

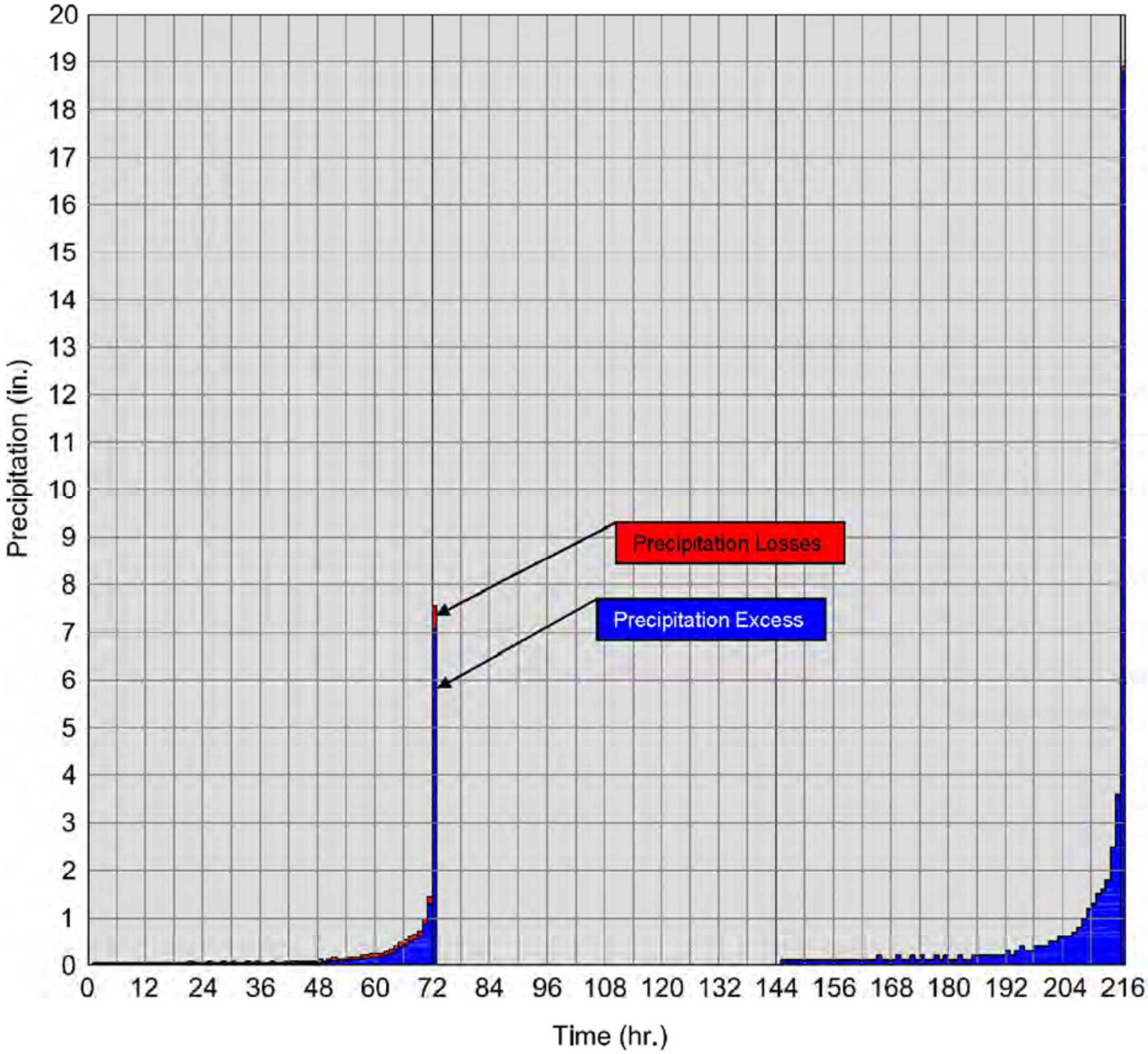


Figure 2.4.3-240
Make-Up Pond C Watershed 72-Hour End Peaking Hyetograph

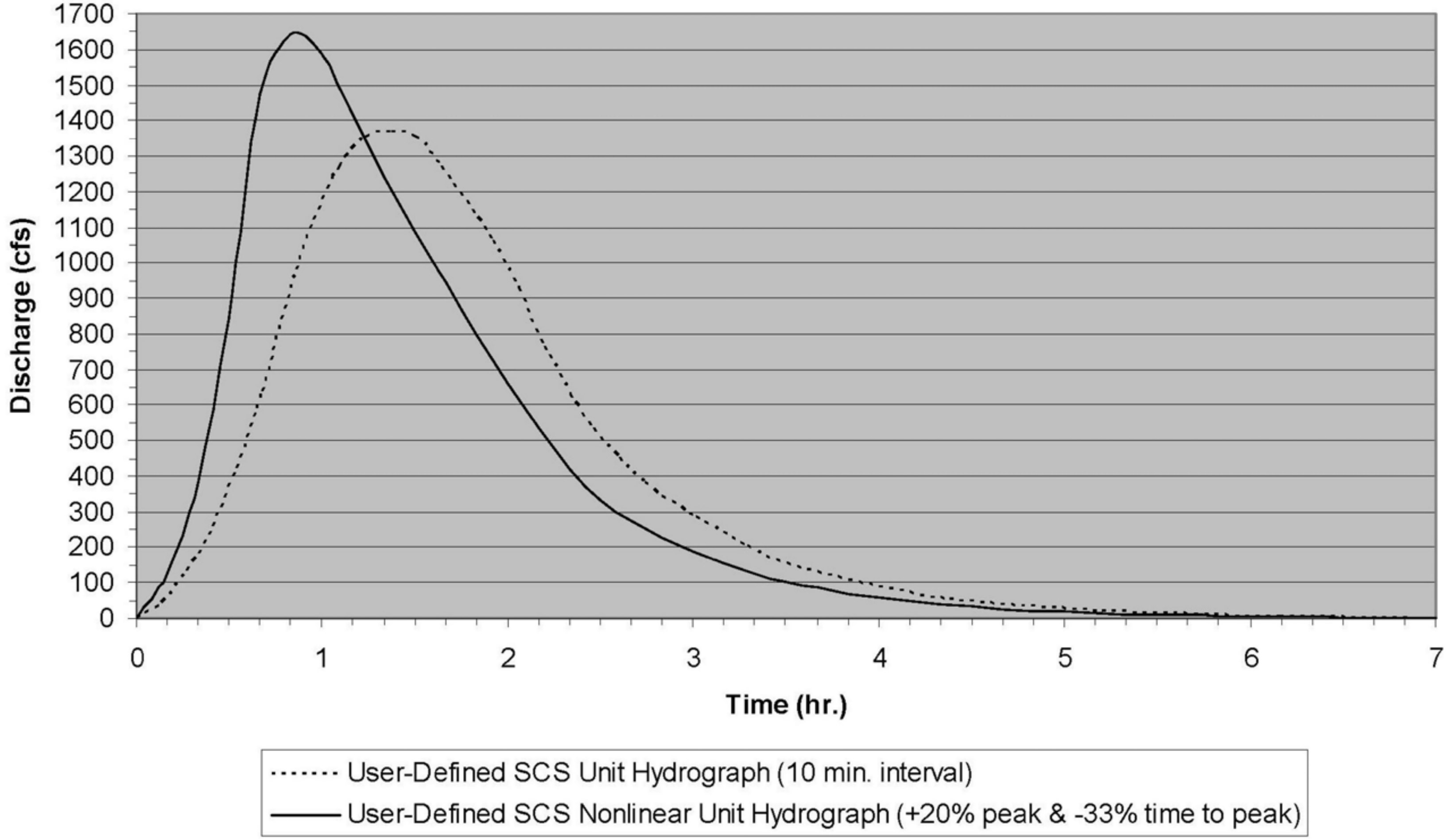


Figure 2.4.3-241
Subbasin Unit Hydroraphs: Make-Up Pond C (MUPC)

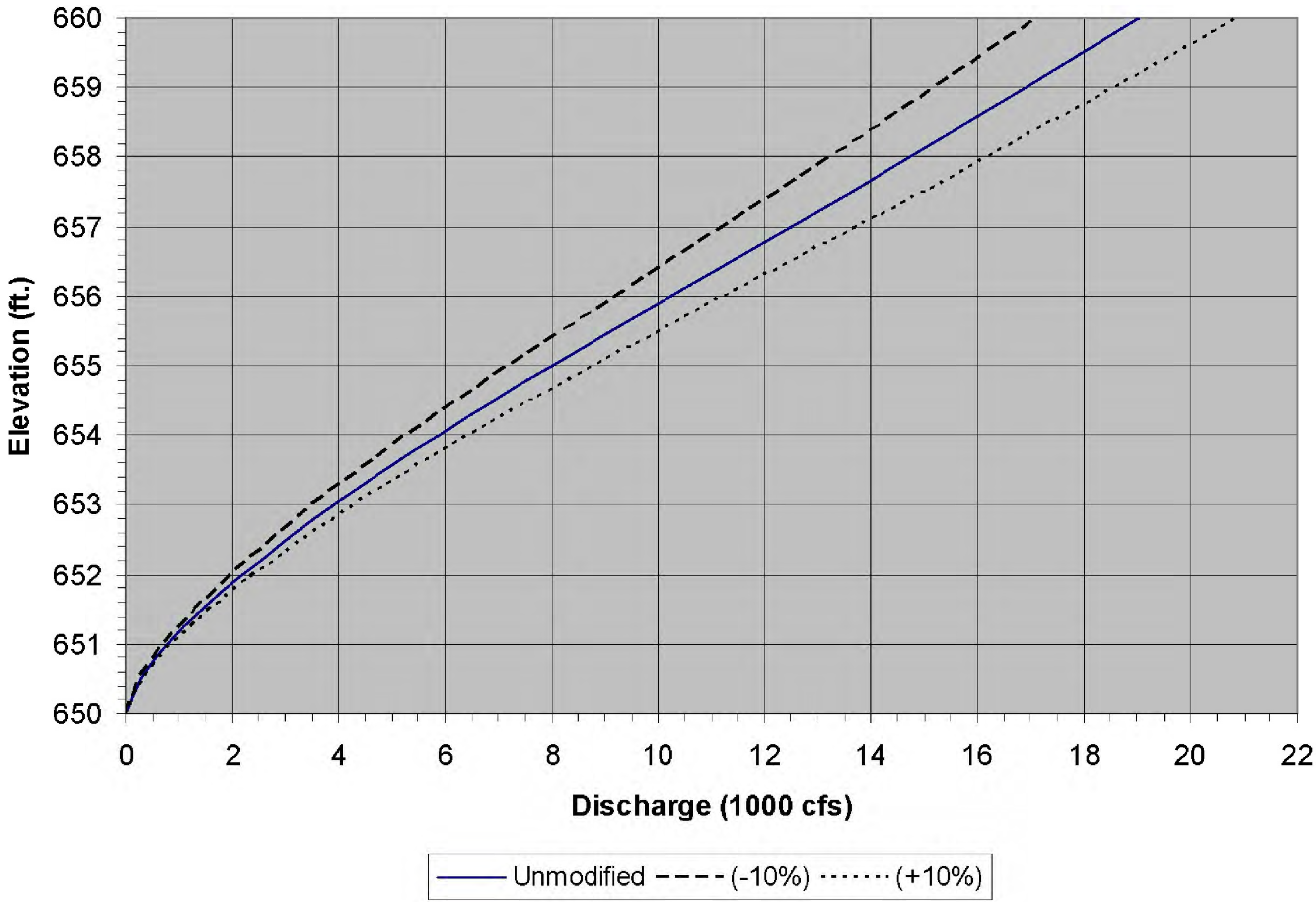


Figure 2.4.3-242
Discharge Rating Curve, Make-Up Pond C

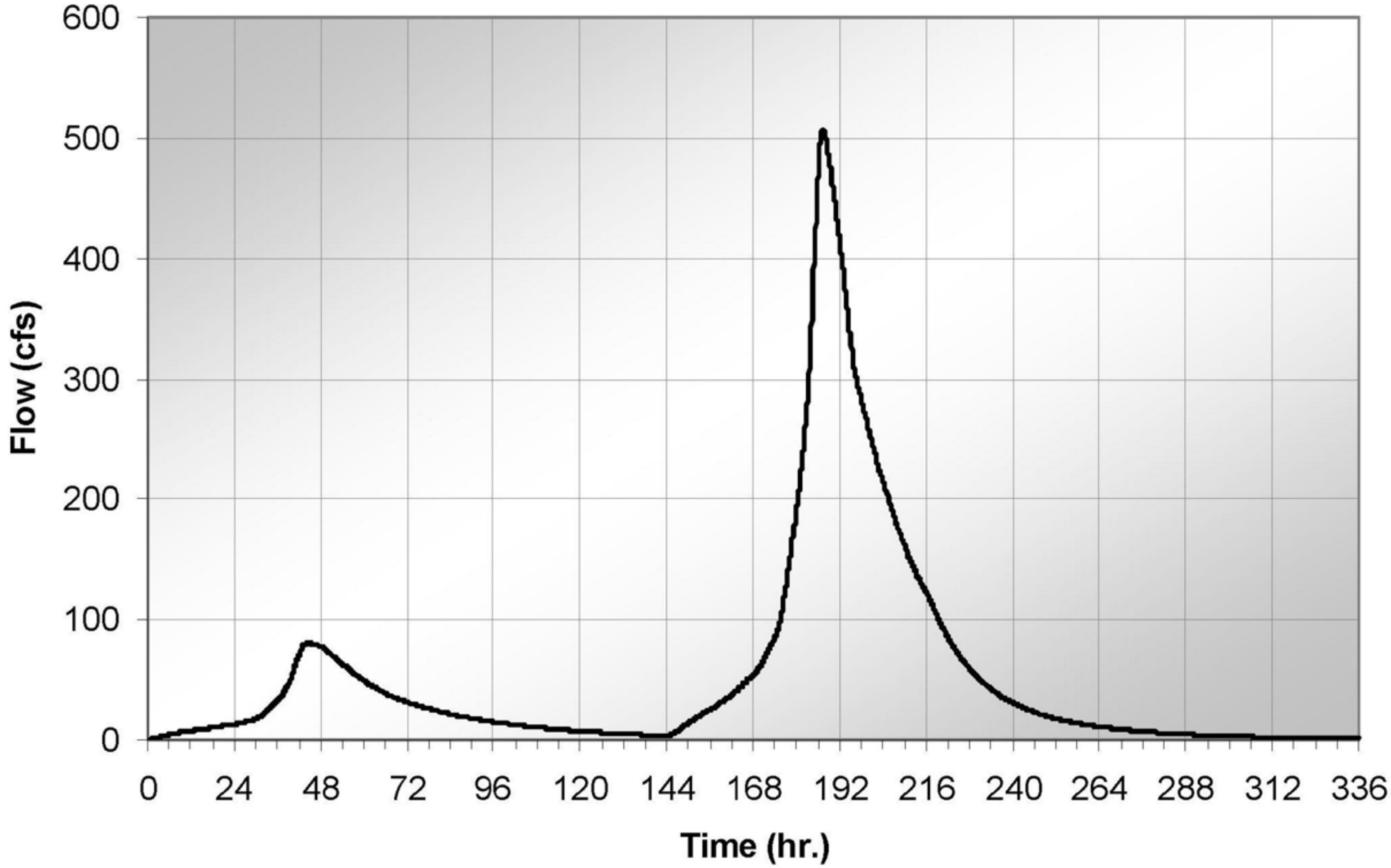


Figure 2.4.3-243
Subbasin Make-Up Pond C (MUPC) PMF Inflow Hydrograph

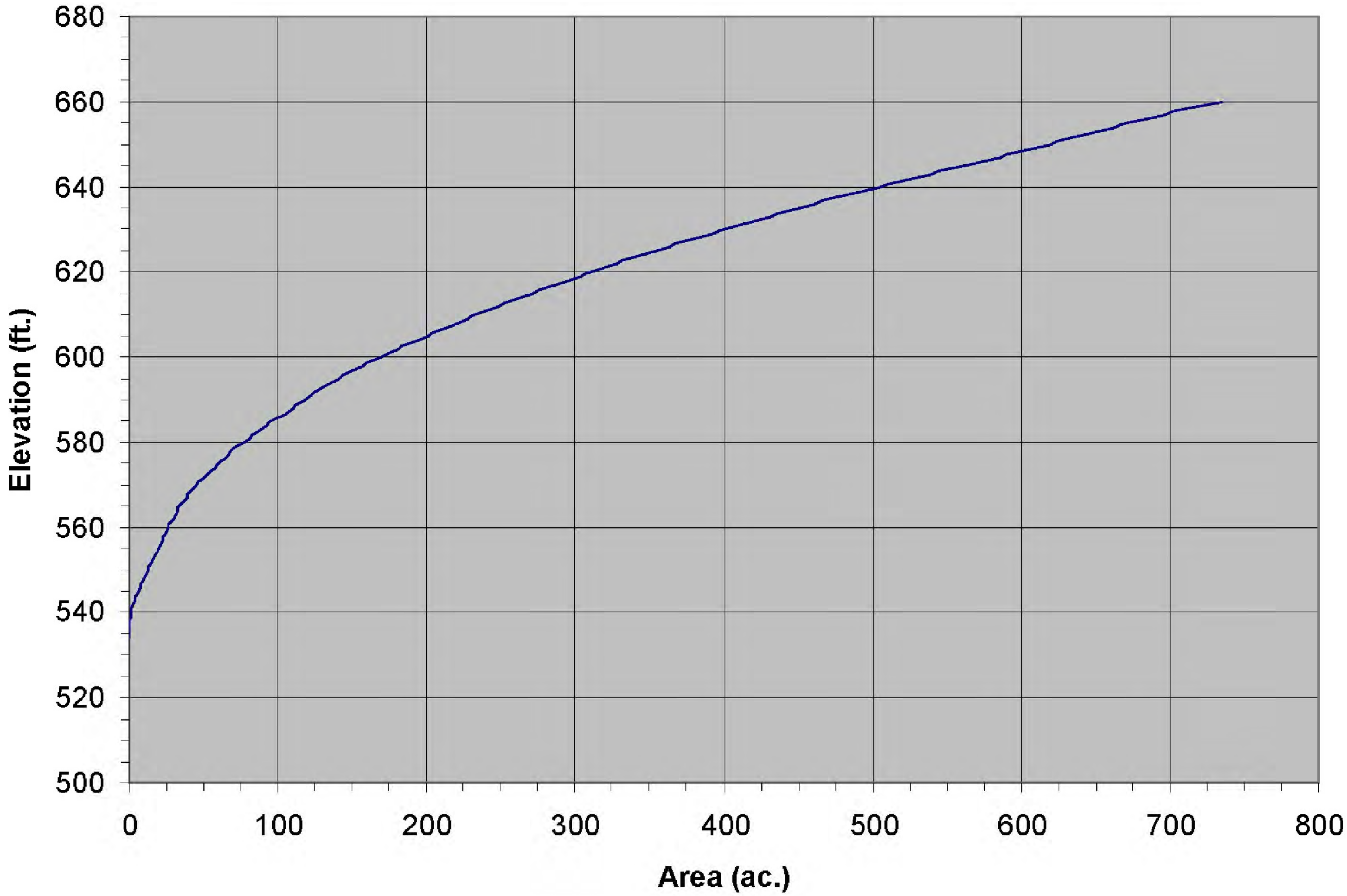


Figure 2.4.3-244
Storage Capacity Curve, Make-Up Pond C

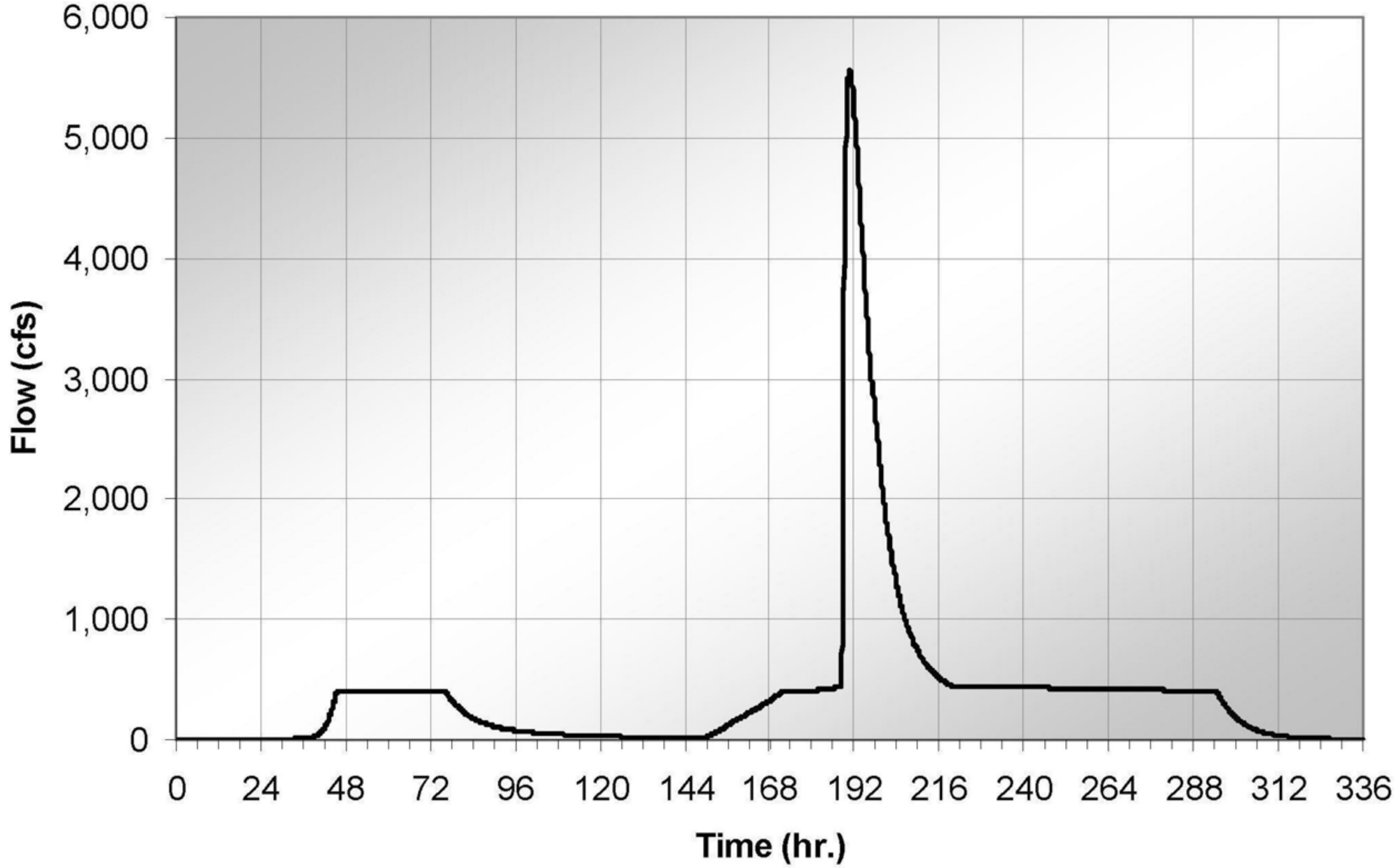


Figure 2.4.3-245
Subbasin Lake Whelchel (WLCHL) PMF Inflow Hydrograph

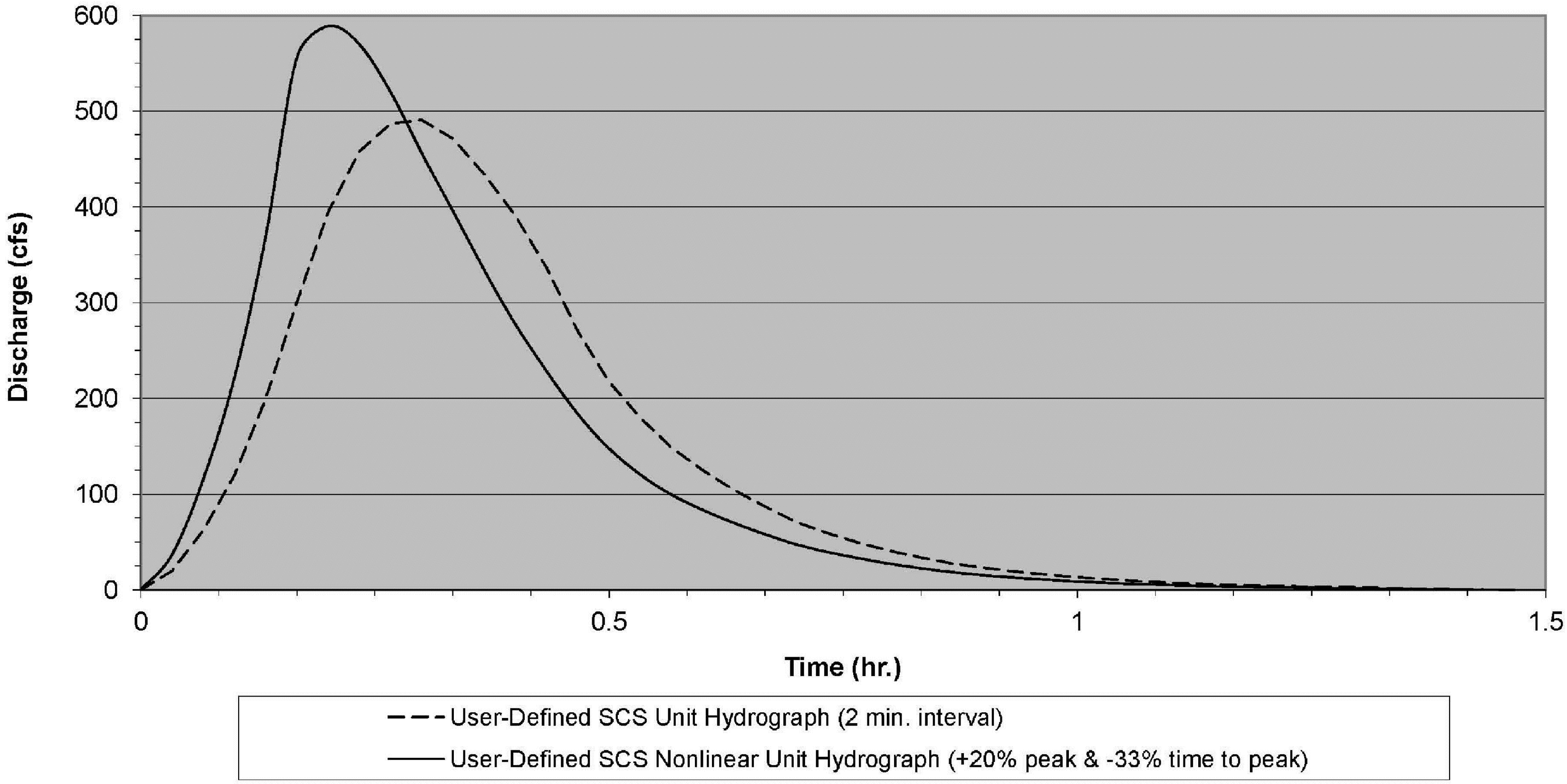


Figure 2.4.3-246
Upper Arm Dam Unit Hydrographs

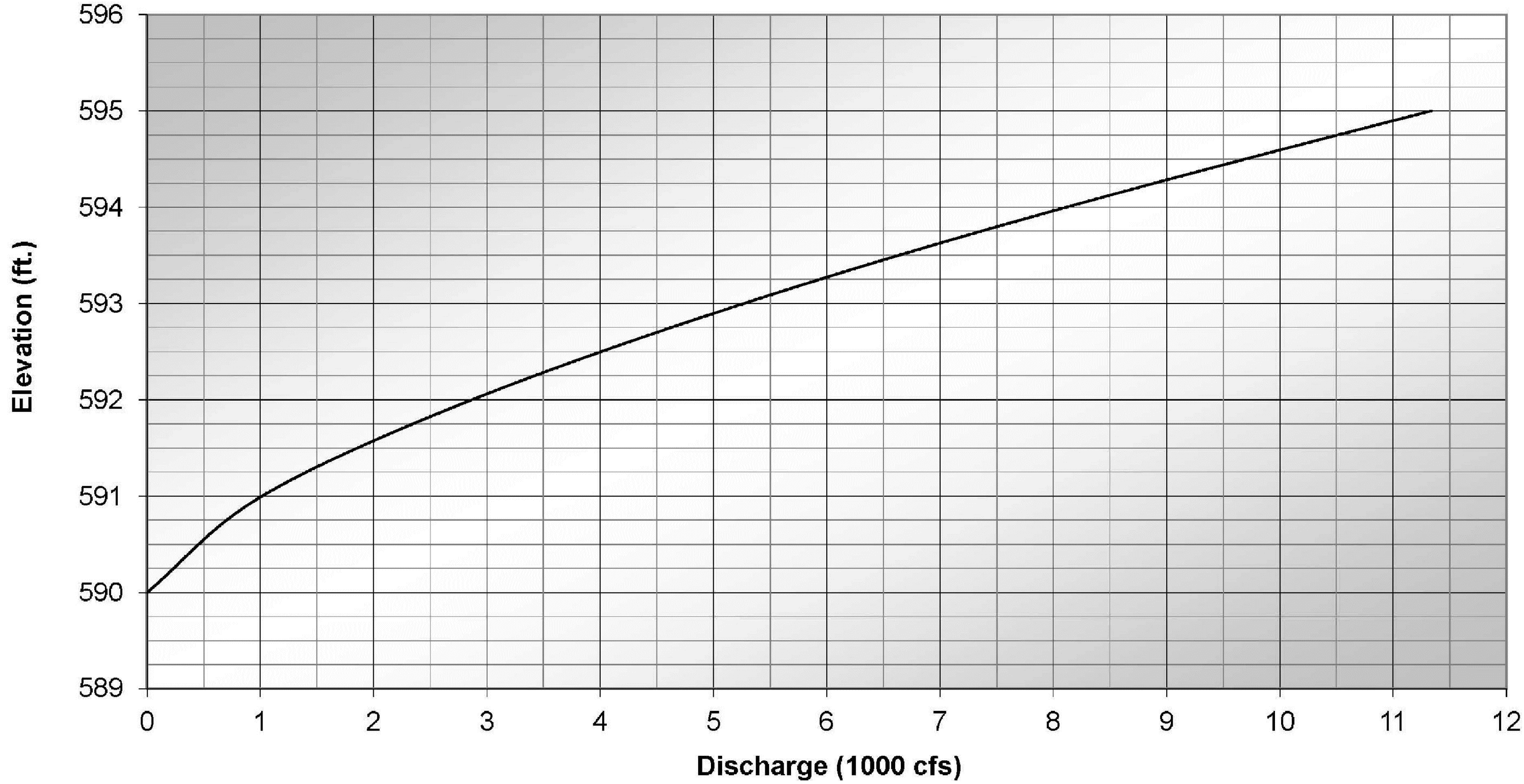


Figure 2.4.3-247
Discharge Rating Curve, Upper Arm Dam

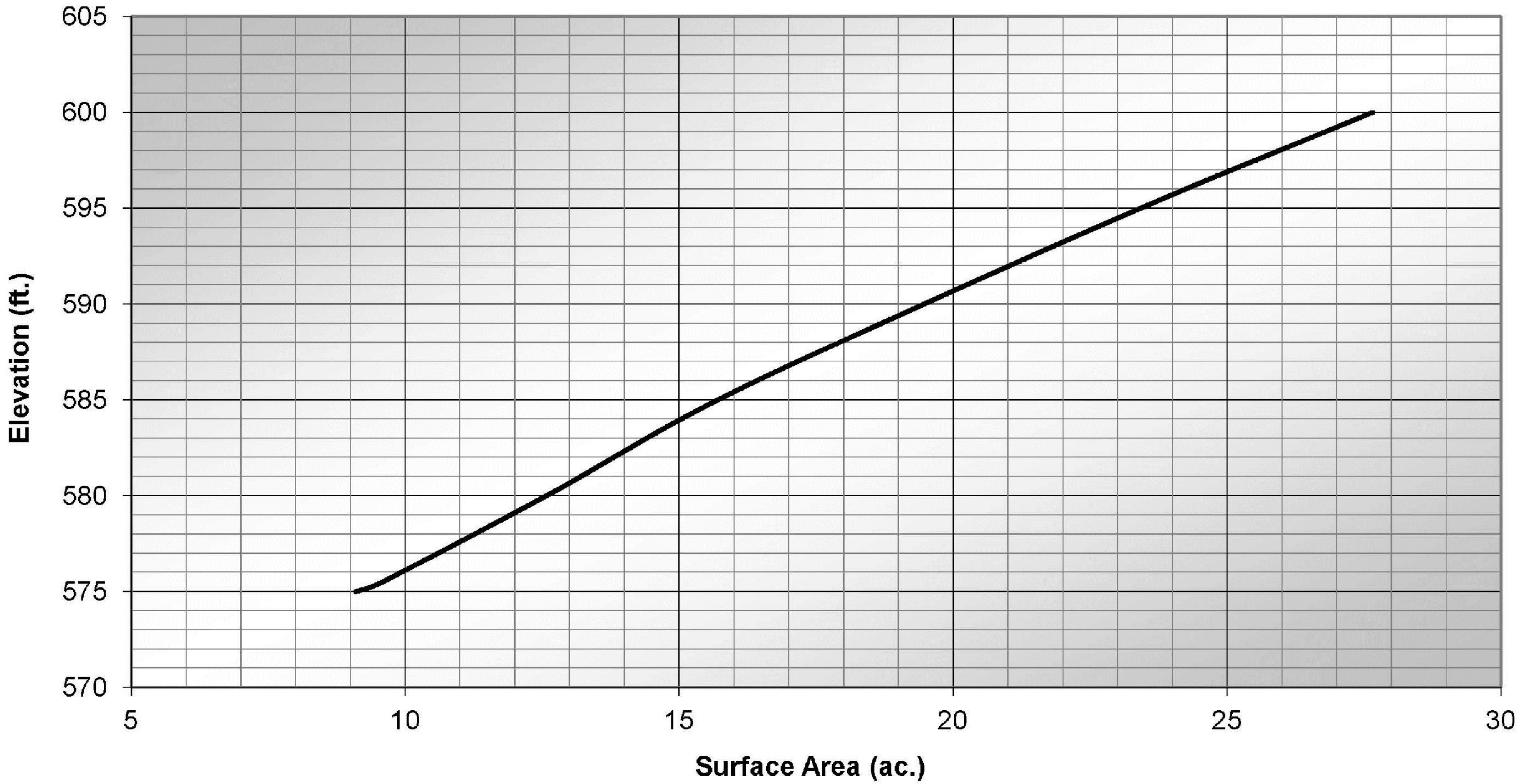


Figure 2.4.3-248
Storage Capacity Rating Curve, Upper Arm Dam

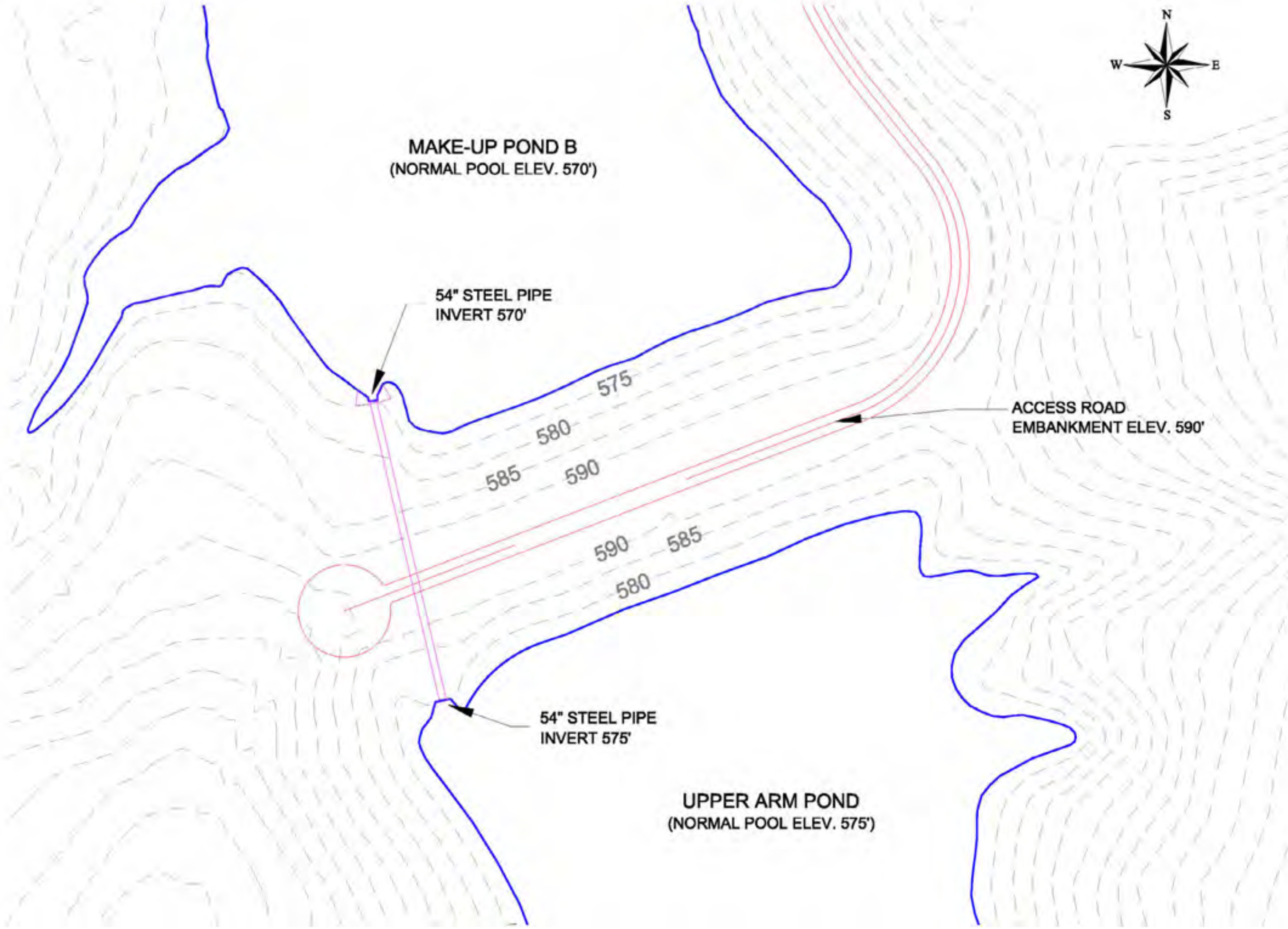


Figure 2.4.3-249
Drainage Culvert Schematic, Upper Arm Dam

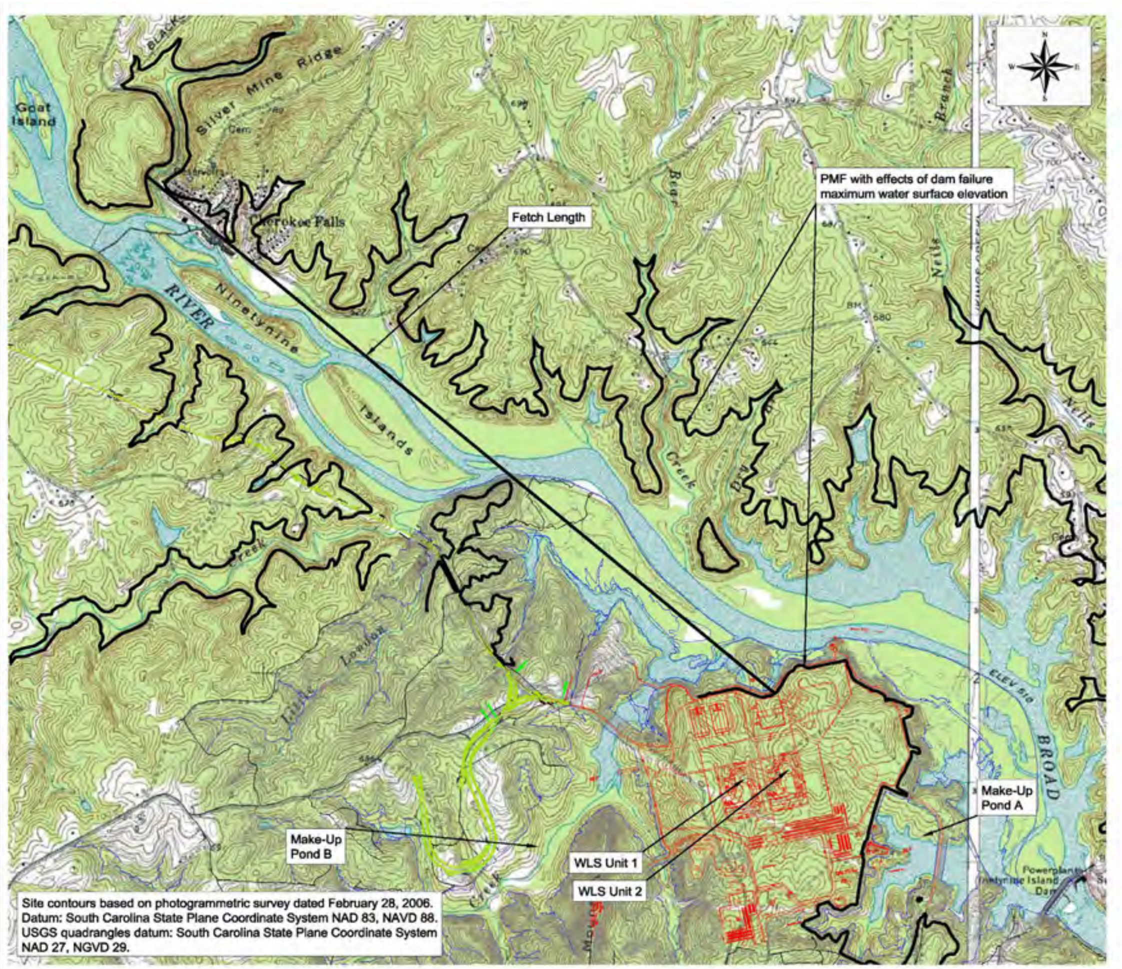


Figure 2.4.4-201
Broad River Coincident Wind Wave Fetch Length

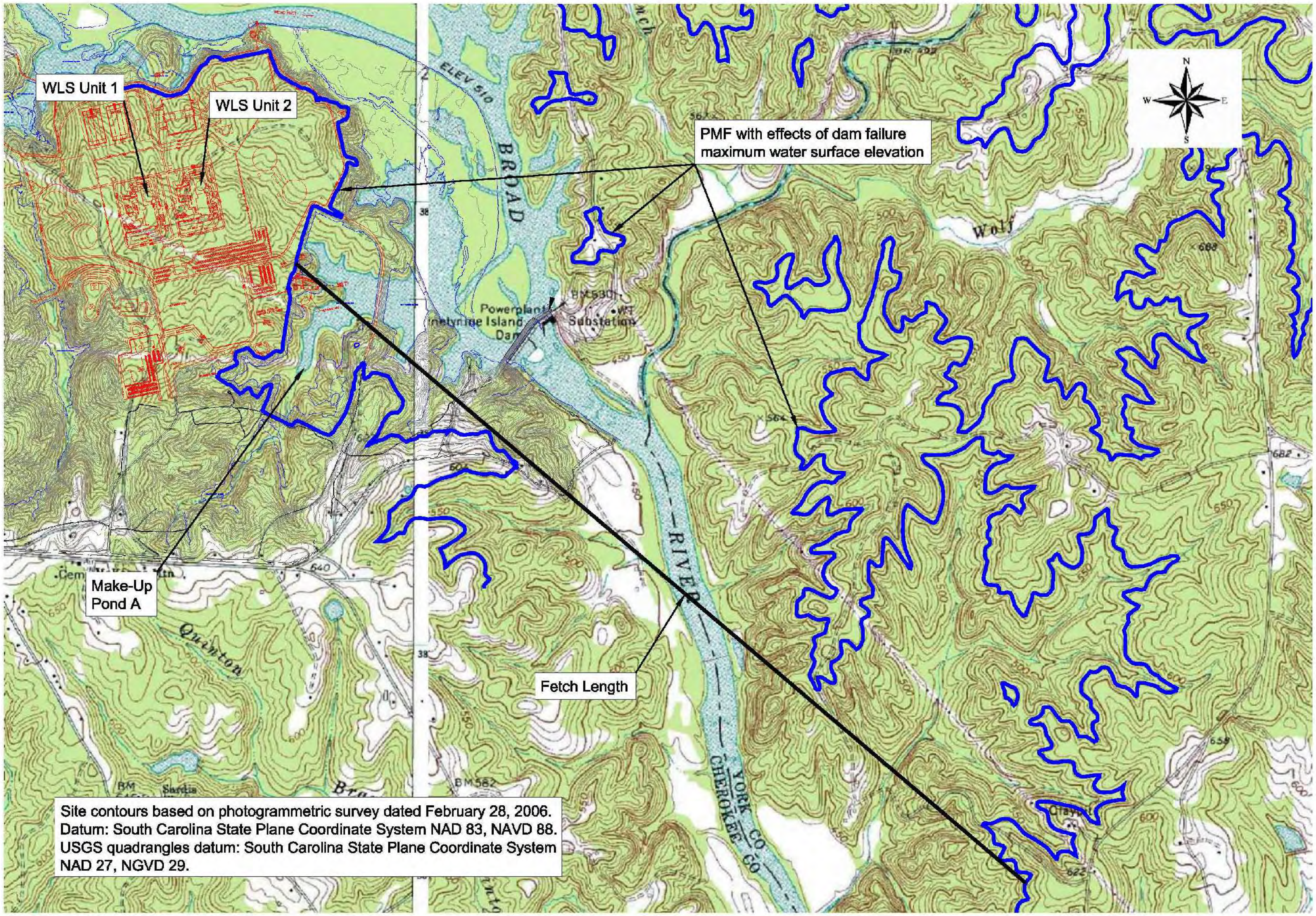


Figure 2.4.4-202
Make-Up Pond A Coincident Wind Wave Fetch Length

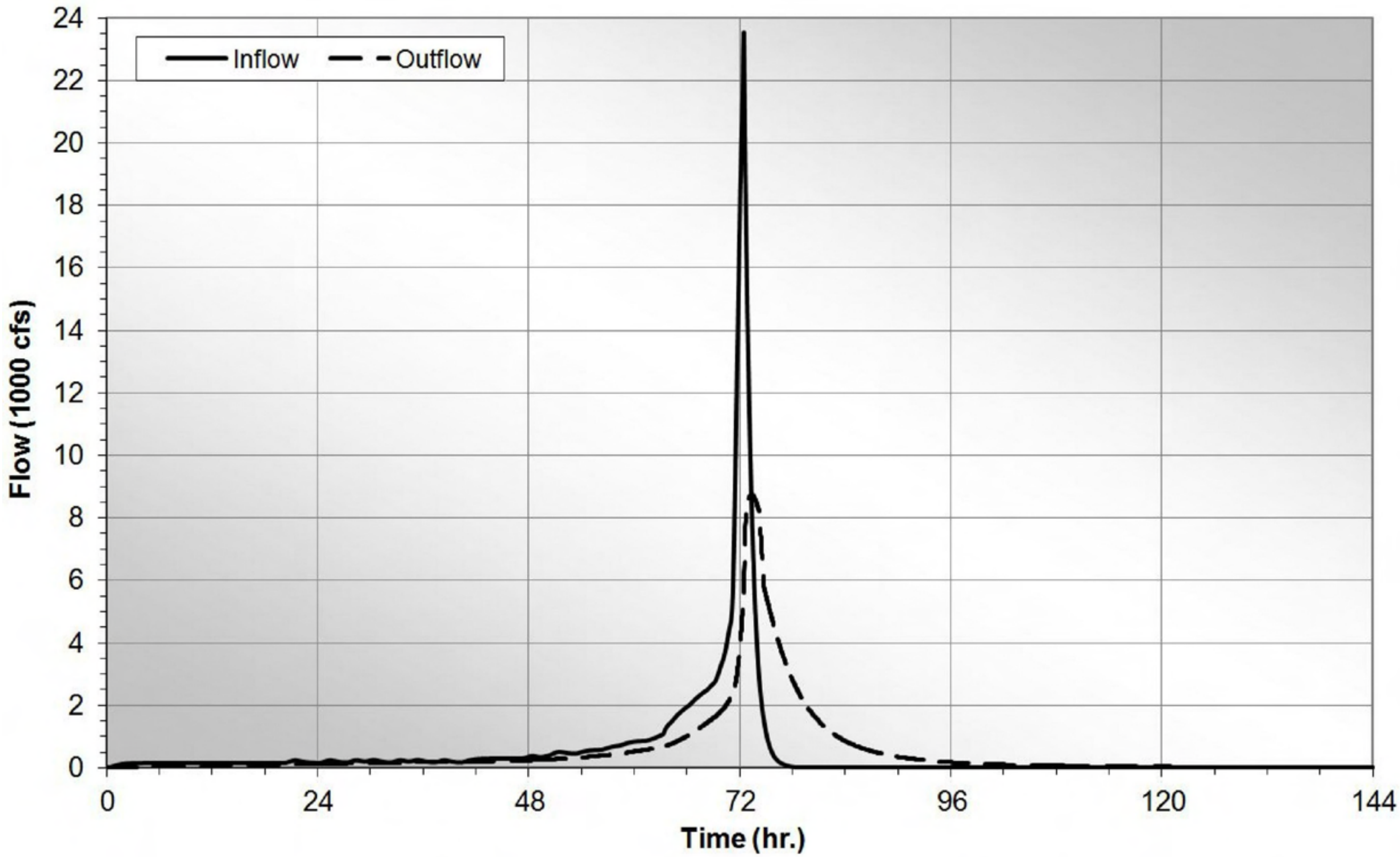


Figure 2.4.4-203
PMF Hydrograph With Upper Arm Dam Failure, Make-Up Pond B

Figure 2.4.4-204
Not Used

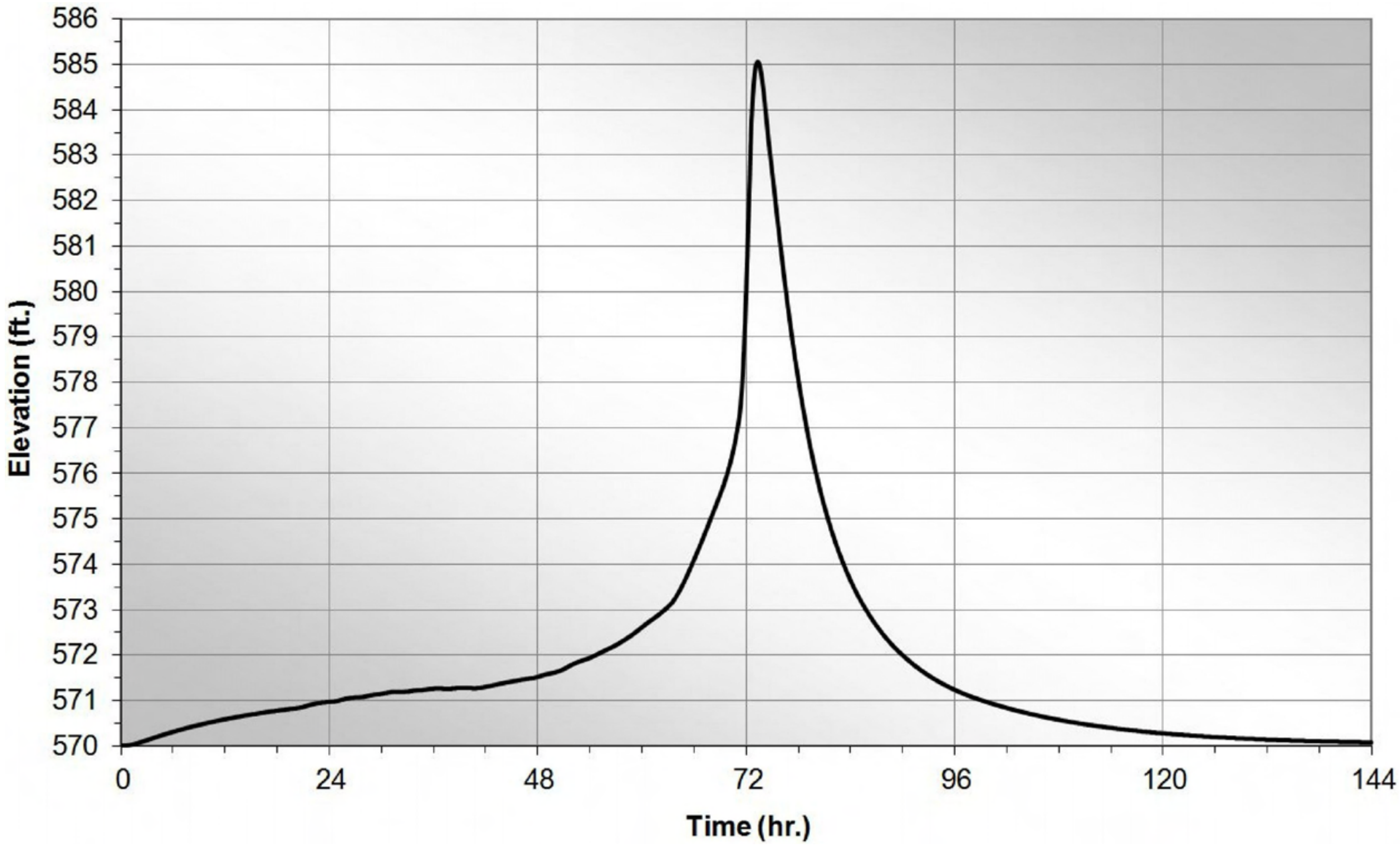


Figure 2.4.4-205
Flood El. Hydrograph Make-Up Pond B With Upper Arm Dam Failure, 72-hr. Local Intense Precipitation