2.4 Hydrologic Engineering

Section 2.4 describes the hydrological characteristics of the VEGP site. The site location and description are provided in sufficient detail to support the safety analysis. This section addresses characteristics and natural phenomena that have the potential to affect the design basis for the proposed AP1000 units. The Section is divided into thirteen sections:

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

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The 3,169-acre VEGP site is located on a coastal plain bluff on the southwest side of the Savannah River in eastern Burke County. The site is approximately 30 river miles above the U.S. Highway 301 bridge and directly across the river from the Department of Energy's Savannah River Site (Barnwell County, South Carolina). The VEGP site is approximately 15 miles east-northeast of Waynesboro, Georgia and 26 miles southeast of Augusta, Georgia, the nearest population center (i.e., having more than 25,000 residents). It is also about 100 miles north-northwest of Savannah, Georgia and 150 river miles from the mouth of the Savannah River. The contributing drainage area of the Savannah River at the site is 8,304 square miles, as estimated from digital mapping.

The Savannah River Basin and its subbasins, as delineated by the National Weather Service **(NWS 2005)**, and further subdivided into USGS Hydrologic Unit Code (HUC-12) subbasins

(USGS 2006f), are shown in Figure 2.4.1-1. The drainage areas of the NWS subbasins are given in Table 2.4.1-1

Two Westinghouse pressurized water reactors (PWRs), rated at 3,565 MWt each, are currently in operation at the VEGP site. Unit 1 began commercial operation in May 1987; Unit 2 began commercial operation in May 1989. All structures, including the containment structures, two natural draft cooling towers (one per unit), associated pumping and discharge structures, water treatment building, switchyard, and training center, are located at or above El. 220 ft mean sea level (msl).

SNC has selected the Westinghouse AP1000 certified plant design **(NRC 2006)** for the VEGP ESP application. The proposed AP1000 units, to be referred to as Units 3 and 4, will be located west of and adjacent to existing Units 1 and 2 as shown in SSAR Figure 1-4. The AP1000 is rated at 3,400 MWt, with a net electrical output of 1,117 megawatts electrical (MWe). The new units will use natural draft towers for circulating water system cooling, with make-up water coming from the Savannah River, and mechanical draft towers for service water system cooling, with make-up water coming from site wells. The Units 3 and 4 grade elevation will also be at or above 220 feet msl. An extensive site storm water drainage system was developed during construction of Units 1 and 2 and will be used for Units 3 and 4 with some modifications.

2.4.1.2 Hydrosphere

The Savannah River is the main hydrologic feature that may affect or be affected by power plants constructed at the VEGP site.

The watershed of the Savannah River extends into the mountains of North Carolina, South Carolina, and Georgia near Ellicott Rock, the point where the borders of those three states meet. The river system drains a basin of 10,577 sq mi, divided between the three states as follows **(SR 2006)**:

- 4,581 sq mi in South Carolina
- 5,821 sq mi in Georgia
- 175 sq mi in North Carolina

Within the three states, the basin includes portions of 44 counties and borders two major metropolitan centers, Augusta and Savannah. The lower 50 mi is tidally influenced **(USACE 1996)**.

The Savannah River basin, which is described as long and relatively narrow, crosses through three distinct physiographic provinces: Mountain, Piedmont, and Coastal Plain. The Mountain and Piedmont provinces are within the Appalachian Mountain range, with the border between them extending from northeast to southwest, crossing the Tallulah River at Tallulah Falls. The

Fall Line, or division between the Piedmont province and the Coastal Plain, also crosses the basin in a generally northeast to southwest direction, near Augusta, Georgia **(USACE 1996)**.

Watershed elevations range from 5,030 ft msl at Little Bald Peak in North Carolina to sea level at Savannah. The approximate range of elevations for each physiographic region is **(USACE 1996)**:

- 5,030 to 1,800 ft msl within the Mountain Province
- 1,800 to 500 ft msl within the Piedmont Province
- 500 to 0 ft msl within the Coastal Plain

The Savannah River, together with certain of its tributaries, forms the border between the states of Georgia and South Carolina. The confluence of the Seneca and Tugaloo Rivers, formerly known as "The Forks," but now inundated by Hartwell Lake, marks the upstream end of the Savannah River. The length of the Savannah River from “The Forks” to the mouth is approximately 312 mi **(USACE 1996)**.

The following principal streams make up the Savannah River stream system **(USACE 1996)**:

- The Tallulah and Chatooga rivers combine to form the Tugaloo River at River Mile 358.1.
- Twelve Mile Creek and the Keowee River join to form the Seneca River at River Mile 338.5.
- The Tugaloo and Seneca rivers join to form the Savannah River proper at River Mile 312.1, at the point known as “The Forks.”

The entire 312-mi length of the Savannah River is regulated by three adjoining US Army Corps of Engineers (USACE) multipurpose projects, forming a chain along the Georgia–South Carolina border 120 mi long. The three reservoirs, each with appreciable storage, are, from upstream to downstream:

- Hartwell Lake and Dam
- Richard B. Russell Lake and Dam
- J. Strom Thurmond Lake and Dam (also known as Clarks Hill Lake and Dam)

Of the 6,144 sq mi drainage basin above Thurmond Dam, 3,244 sq mi (53 percent) are between Thurmond and Russell Dams, 802 sq mi (13 percent) are between Russell and Hartwell Dams, and 2,088 sq mi (34 percent) are above the Hartwell Dam **(USACE 1996)**. Table 2.4.1-2 lists the River Miles of key landmarks along the Savannah River.

The climate in the upper Savannah River watershed is classified as temperate, with generally mild winters and long summers. The basin is protected from the extremes of winter continental weather experienced in the nearby Tennessee Valley by the Blue Ridge Mountains. The annual mean temperature for the basin is 60 ºF. January, which is usually the coldest month of the year, frequently has night temperatures of 20 ºF or lower. July and August, the hottest months of the year, have many days with temperatures over 90 ºF. In the lower section of the basin, the winters are milder and the summer temperatures higher **(USACE 1996)**.

There are generally two periods of maximum rainfall in the upper basin: February–March and July–August, although heavy rainfall has occurred in practically every calendar month. The mean annual precipitation decreases from 83.5 in. in Highlands, North Carolina, to 49.2 in. at Savannah, Georgia **(USACE 1996)**.

2.4.1.2.1 Hydrologic Characteristics

Average daily and annual peak flow series data have been tabulated by the USGS for nine stream gages that have been maintained along the Savannah River between River Miles 288.9 and 60.9. Table 2.4.1-3 identifies location, gage elevation, upstream drainage area, and start and stop date and number of records for the annual and daily time series for each gage. Annual peak discharge data for these gages are used in Section 2.4.2; daily discharge data for these gages are used in Section 2.4.11.3. Summary statistics characterizing the seasonal flow variability are discussed below.

As indicated in Table 2.4.1-2, the USGS gage at Jackson, South Carolina, is approximately 6 river miles upstream of the VEGP site. Based on the mean daily flow series for this gage, presented in Table 2.4.1-6, the average daily discharge at the site is 8,913 cfs, calculated as the mean of the average daily flows for each day of the 31-year record. For this gage, the monthly mean daily flow varies from a minimum of 7,216 cfs in September to a maximum of 11,347 cfs in March. A plot of the monthly variation in mean daily flow on the Savannah River recorded at the Jackson, South Carolina, stream gage (with plots for the Calhoun Falls and Augusta, Georgia, gages included for comparison) is provided in Figure 2.4.1-2, based on USGS records for the years of record of each gage, without accounting for the impact of changes in upstream regulation. Tables 2.4.1-4 through 2.4.1-6 show the mean daily discharge for the years of record for each of the three gages presented in Figure 2.4.1-2.

2.4.1.2.2 Local Site Drainage

Local drainage is shown in Figure 2.4.1-3, which was developed from the Shell Bluff Landing, Girard NW, Alexander, and Girard USGS quadrangle sheets. The site is on a high, steep bluff on the west bank of the Savannah River, overlooking the extensive floodplain on the east bank. Georgia State Highway 23 runs roughly parallel to the river, about 4 mi from the VEGP site. It runs along the ridge line that separates local drainage running northeast to the river from runoff draining generally to the southwest.

An unnamed, highly incised creek drains the northern area of the site, including Mallard Pond, into the Savannah River just upstream of the site, near the point identified as Hancock Landing in Figure 2.4.1-3.

To the west, the site is drained by the Red Branch and Daniels Branch, which combine and drain along with Beaverdam Creek and High Head Branch into Telfair Pond, south of the site. Beaverdam Creek intercepts three streams draining runoff from north of State Road 23 before they reach the site.

The names, estimated channel lengths, and slopes of the natural channels draining the site area are provided in Table 2.4.1-7.

2.4.1.2.3 Dams and Reservoirs

There are a number of water control structures on the Savannah River and its major tributaries **(USGS 1990**, **USACE 1993**, and **USACE 1996)**. Table 2.4.1-8 presents a list of these structures with hydraulic design information for each project and identification of its location with respect to the VEGP site.

Three major projects run by the USACE upstream of the VEGP site have a significant influence on the discharge of the Savannah River due to their large storage volume. These are:

- Hartwell Lake and Dam,
- Richard Russell Lake and Dam, and
- J. Strom Thurmond Lake and Dam (also known as Clarks Hill Lake and Dam on the Georgia side)

The authorized water management goals of the three-dam multi-use project are specified for normal operation, flood operation, and drought condition operation as follows **(USACE 1996)**:

<u>For normal conditions</u>, the operation policy is designed to maximize the public benefits of hydroelectric power, flood damage reduction, recreation, fish and wildlife, water supply, and water quality.

<u>Under flood conditions</u>, the water management objective of the multipurpose projects is to operate the reservoir system to minimize flooding downstream by timing turbine discharges, gate openings, and spillway discharges as required.

<u>For drought conditions</u>, the water management objectives of the projects are:

- To prevent draw-down of lake levels below the bottom of the conservation pool,
- To make use of most of the available storage in the lake during the drought-of-record,
- To maintain hydroelectric plant capacity throughout the drought, and
- To minimize adverse impacts to recreation during the recreation season (generally considered to be from May 1 through Labor Day)

The USACE also operates the New Savannah Bluff Lock and Dam upstream of the VEGP site, but this project has very little impact on flows at the site, due to its small run-of-river storage volume **(USACE 1996)**.

Each project is described briefly in the following paragraphs **(USACE 1996)**.

<u>The Hartwell Lake and Dam</u> is at River Mile 288.9, 7 mi east of Hartwell, Georgia. The top of the conservation pool is set at El. 660 ft msl. At this level, the reservoir extends 49 mi up the Tugaloo River in Georgia and 45 mi up the Seneca and Keowee Rivers in South Carolina. The shoreline at El. 660 ft msl is approximately 962 mi long, excluding island areas. Operation of the project began in 1965.

The reservoir has a total storage capacity of 2,550,000 acre-feet below El. 660 ft msl. The dam consists of a concrete gravity section 1,900 ft in length and rising about 204 ft above the streambed, and two earth embankment sections extending to high ground on the Georgia and South Carolina shores of the river, for a total length of 17,880 ft.

<u>The Richard B. Russell Lake and Dam</u> is at River Mile 259.1 in Elbert County, Georgia, and Abbeville County, South Carolina. The dam is 18 mi southwest of Elberton, Georgia; 4 mi southwest of Calhoun Falls, South Carolina; and 40 mi northeast of Athens, Georgia. Operation of the project began in January 1985.

The top of the conservation pool is set at El. 475 ft msl. The reservoir has a total storage capacity of 1,026,200 acre-feet at this level, and 1,166,166 acre-feet of total storage at the top of the flood control pool (El. 480 ft msl).

The dam consists of a concrete gravity section 1,883.5 ft in length and two earth embankment sections, 2,180 ft in length in Georgia and 460 ft in length in South Carolina. A concrete overflow spillway section is located in what was formerly the stream channel. It has an ogee-shaped crest controlled by 10 tainter gates.

A flip bucket for dissipating the energy of spillway discharges is located at the bottom of the spillway. The spillway tainter gates are designed for a maximum discharge of 800,000 cfs at pool El. 490 ft msl.

<u>The J. Strom Thurmond Lake and Dam</u> is at River Mile 221.6 on the Savannah River, 22 mi upstream of Augusta, Georgia. The reservoir at the top of the flood control pool (El. 335 ft msl) has an area of 78,500 acres. At El. 330 ft msl, the top of the conservation pool, the reservoir extends about 40 mi up the Savannah River and about 30 mi up the Little River in Georgia and has approximately 1,050 mi of shoreline, excluding island areas. The reservoir has a total storage capacity of 2,510,000 acre-feet below El. 330 ft msl. Operation of the project began in 1952.

The dam consists of a concrete gravity section 2,282 ft in length and two earth embankment sections with a total length of 5,680 ft, extending to high ground on the Georgia and South Carolina shores.

The spillway is a concrete gravity ogee section extending across the west floodplain and river channel. A bucket anchored to solid rock and constructed at four levels ranging from

El. 163.0 ft msl to El. 179.0 ft msl, is provided at the toe of the spillway. The spillway discharges are controlled by 23 tainter gates separated by concrete piers 8 ft thick.

The embankments and earth dam are of rolled fill construction. An impervious core, graded from coarse and medium sand to fine silt and clay, extends to rock and is contained by a more pervious shell, consisting of well-graded coarse and medium sand to silt. The embankments are covered with rip-rap from the top down to El. 295 ft msl on the upstream side, and from the toe up to an elevation above maximum tailwater on the downstream side. U.S. Highway 221 crosses the dam.

The New Savannah Bluff Lock and Dam is located at River Mile 187.7. The function of the lock was originally to provide adequate draft depths for navigation, but there is currently very little commercial navigation above Savannah Harbor. Today the structure's main function is to maintain an adequate river stage for upstream water supply intake structures.

The structure crosses the Savannah River about 13 mi below Augusta. It is a concrete dam 360 ft long containing five vertical-lift crest control gates. The lock chamber, located on the Georgia side of the river, is 56 ft by 360 ft and is closed by mitering lock gates. The lift is 15 ft, the depth over the lower miter sill being about 10 ft at low water and over the upper miter sill being 14 ft at normal pool level. Elevation of the normal pool is about 115.0 ft msl, and low water at the downstream entrance to the lock is at El. 101.8 ft msl, based on a flow of 6,300 cfs.

2.4.1.2.4 Proposed Water Management Changes

The USACE, working in response to US Environmental Protection Agency (EPA) recommendations, is currently reviewing operating rules for the dams under its jurisdiction in the Savannah River watershed. The study goal is to determine if changes are warranted to meet current and future water resource management goals, including flood control, water supply, fish and wildlife enhancement, drought control, water quality, recreation, and aquatic plant control. The study is scheduled for completion in 2009 **(USACE 2004)**.

Pending the results of the watershed study, current USACE operations along the river are limited to the maintenance of existing structures and minor flood control improvements with no significant impact on the VEGP site.

It has been reported **(SR 2006)** that the Ports Authority of Georgia is considering deepening the harbor in Savannah to accommodate the new very large container ships that will be visiting ports on the East coast. The possibility that dredging would force the salinity gradient further upstream with possible adverse impact on the Savannah National Wildlife Refuge has been the subject of some study, but the possible change in policy would have no impact on safety issues at the VEGP site.

2.4.1.2.5 Surface Water Users

Historically, the Savannah River was an important transportation corridor, but today it serves primarily as a source of water for industry and municipalities, a receiving body for the subsequent discharge of effluent, and an avenue for power generation and recreational activities **(SR 2006)**.

Agencies with important roles in the watershed include the USACE, which is responsible for maintaining reservoirs on the main stem of the Savannah River, and the EPA in cooperation with the Georgia Environmental Protection Division and the South Carolina, which are responsible for maintaining water quality in the basin.

Current in-stream use of Savannah River water includes minimum stream flow requirements for navigation and environmental maintenance, and diversions for industrial use, including once-through cooling. Consumptive use of Savannah River water is predominantly for industrial withdrawals for cooling water towers and processing and diversions to water treatment plants for municipal water use.

Table 2.4.1-9 presents a summary of data on surface-water users adjacent to or downstream from VEGP whose intakes could be adversely impacted by an accidental release of contaminants from the site; the summary includes information on the owner, facility type, estimated distance from the VEGP site, and average daily withdrawal rate.

Information about groundwater users is presented in Section 2.4.12, while Section 2.4.13 discusses the consequences of liquid effluent releases to surface waters.

2.4.1.2.6 Water Consumption

The new AP1000 units require water for both plant cooling and operational uses. The Savannah River provides makeup water for the circulating water system (CWS) to replace the water lost to evaporation, drift, and blowdown. Onsite wells provide groundwater makeup for the service water system (SWS). The wells also provide water for other plant systems, including the fire protection system, the plant demineralized water supply system, and the potable water system. Surface water consumptive use for the two AP1000 units' normal operation is 27,924 gpm, with a maximum of 28,904 gpm. Groundwater consumptive use is 752 gpm on average, with a maximum of 3,140 gpm. During normal operation, approximately 305 gpm of groundwater is returned as surface water to the Savannah River. Table 2.4.1-10 identifies the normal and maximum water demand and effluent streams for the AP1000 units.

The CWS and SWS cooling towers lose water from evaporation and drift. Evaporation and drift from the CWS cooling towers is estimated at 27,924 gpm during normal operations. Evaporation and drift for the SWS cooling tower is estimated at 403 gpm. These values are based on site characteristics and AP1000 design parameters for cooling.

Table 2.4.1-10 also provides the water release estimates for wastewater and blowdown discharged to the Savannah River. These include estimates for all wastewater flows from the site, including radiological effluent releases, sanitary waste, miscellaneous drains, and demineralizer discharges. The normal values listed are the expected values for normal plant operation with two new units in operation. The maximum values are those expected for upset or abnormal conditions with two new units in operation.

Table 2.4.1-1 Savannah River Subbasins and Drainage Areas above VEGP Site

NWS Subbasin		NWS Subbasin Name	Drainage Area, mi^2	
No.	I.D.		upstream of site (1)	downstream of site (2)
1	TIGG1	Burton Dam, GA	122.3	0.0
2	JCSS1	Jocassee Dam, SC	157.7	0.0
3	KEOS1	Keowee Dam, SC	288.0	0.0
4	HRTG1	Hartwell Dam, GA	1544.7	0.0
5	RBRS1	R.B. Russell Dam	738.2	0.0
6	CARG1	Carlton Bridge, GA	760.6	0.0
7	CHDS1UP	Clark Hill - Thurmon Dam (upstream)	665.9	0.0
8	CHDS1	Clark Hill Dam	1847.7	0.0
9	MODS1	Modoc, S.C.	539.9	0.0
10	AGTG1	Steven Creek Dam, GA	454.8	0.0
11	AGSG1	Augusta 5th Street	77.1	0.0
12	AUGG1	Augusta/Butler Creek	273.6	0.0
13	JACS1	Jackson, S.C.	651.2	0.0
14	BFYG1	Burton's Ferry, GA	182.5	293.4
15	BRIG1	Millhaven, GA	0.0	646.2
16	CLYG1	Clyo, GA	0.0	634.7
Estimated Savannah River drainage area at site			**8304.2**	

1) Based on data from Southeast River Flood Forecasting Center, Atlanta, GA. (NWS 2005)
2) As estimated from HUC-12 shapefiles

Table 2.4.1-2 River Miles for Key Landmarks Along the Savannah River

Land Mark	River Mile *
Confluence of White Water & Toxaway Rivers	368.6
Confluence of Tallulah & Chatooga (forming the Tugaloo)	358.1
Confluence of the Keowee & Twelve Mile Creek (forming Seneca River)	338.5
Confluence of the Senaca & Tugaloo Rivers (forming the Savannah)	312.1
Hartwell Dam (USGS gage 02187250)	288.9
Iva gage (USGS gage 02187500)	280.4
Confluence of Broad River	269.6
Calhoun Falls (USGS gage 02189000)	263.6
Richard B. Russell Dam (USGS gage 02189004)	259.1
Conflence of Little River	223.4
J. Strom Thurmond Dam (USGS gage 02194500)	221.6
Confluence of Stevens Creek	208.1
Augusta City Dam	207.0
Augusta, GA at Fifth Street gage site (02197000)	199.6
Horse Creek at mouth	197.4
New Savannah Bluff Lock and Dam	187.7
Shell Bluff Landing, Georgia	161.9
Jackson, SC gage (02197320)	156.8
Vogtle Electric Generating Plant	150.9
Burtons Ferry Gage (02197500)	118.7
Confluence of Brier Creek	102.5
Clyo gage (02198500)	60.9
Ebenezer Landing, Georgia	48.1
Houlihan Bridge (U.S. Highway 17)	21.6
City of Savannah, GA at Bull Street	14.4
Mouth of the Savannah River	0.0

* River miles measured from the mouth of Savannah Harbor, as reported by USACE 1996.

Table 2.4.1-3 USGS Gage Data for the Savannah River

USGS Gage ID	Location on Savannah River	River Mile *	Coordinates		Altitude, feet MSL **	Area drained, mi^2	Average daily flow series			Annual Peak flow series		
							Start	End	No.	Qp start	Qp end	No.
2187252	below Hartwell Lake nr Hartwell, GA	288.9	34°21'15" N,	82°48'55" W	470.00	2,090	10/1/1984	9/30/1999	4,502	1/21/1985	8/24/1999	15
2187500	near Iva, SC	280.4	34°15'20" N,	82°44'42" W	432.26	2,231	10/1/1950	9/30/1981	11,323	10/8/1949	7/24/1981	32
2189000	near Calhoun Falls, SC	263.6	34°04'15" N,	82°38'30" W	363.53	2,876	10/1/1896	9/30/1979	17,044	4/5/1897	3/28/1980	82
2195000	near Clarks Hill, SC	NR	33°38'40" N,	82°12'05" W	182.69	6,150	5/14/1940	6/30/1954	5,161	--	--	0
2196484	near North Augusta, SC	207.0	33°33'06" N,	82°02'19" W	150.00	7,150	10/1/1988	9/30/2002	5,113	9/21/1989	3/4/2002	13
2197000	at Augusta, GA	199.6	33°22'25" N,	81°56'35" W	96.58	7,508	10/1/1883	9/30/2003	35,793	1/17/1796	6/14/2004	133
2197320	near Jackson, SC	156.8	33°13'01" N,	81°46'04" W	77.00	8,110	10/1/1971	9/30/2002	10,733	1/21/1972	3/5/2002	30
2197500	at Burtons Ferry Bridge nr Millhaven, GA	118.7	32°56'20" N,	81°30'10" W	52.42	8,650	10/1/1939	9/30/2003	18,993	10/1/1929	3/21/2003	53
2198500	near Clyo, GA	60.9	32°31'41" N,	81°16'08" W	13.39	9,850	10/1/1929	9/30/2003	25,567	1/24/1925	3/3/2004	80

* River miles measured from the mouth of Savannah Harbor, as reported by USACE 1996.
** NGVD 1929

Source: Adapted from USGS 2006a

Table 2.4.1-4 Daily Mean Flow Data for the Savannah River at Calhoun Falls, South Carolina (USGS Gage 2189000)

Day of month	Mean of daily mean values for this day for 49 years of record[1], in ft³/s											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	5,364	5,898	6,560	8,923	6,925	5,443	4,455	3,872	4,237	3,286	3,630	4,632
2	5,084	6,221	6,427	8,229	6,832	5,161	4,482	4,081	3,904	3,078	3,827	4,534
3	5,719	5,796	6,734	7,558	6,824	4,698	4,020	4,167	3,718	2,960	3,821	4,435
4	5,632	6,219	7,497	7,158	6,529	5,023	3,008	4,237	3,547	3,205	4,180	5,338
5	5,596	5,686	6,972	8,424	5,786	5,796	3,114	4,531	3,558	3,488	4,082	6,139
6	6,324	5,925	6,452	8,819	5,454	5,555	3,935	4,285	3,642	3,323	4,048	5,638
7	7,437	7,683	7,408	8,529	5,380	5,587	4,638	4,310	4,473	3,224	3,810	5,778
8	6,593	6,761	7,349	8,164	5,243	6,334	4,592	4,356	4,503	3,887	3,820	5,563
9	5,991	6,038	6,340	8,194	5,215	5,651	4,681	4,450	4,410	3,780	3,864	4,983
10	6,304	6,226	5,744	6,916	5,039	4,783	4,567	4,226	3,976	3,412	3,780	5,151
11	6,274	6,374	6,054	6,539	5,265	4,809	4,260	3,953	3,885	3,451	3,932	4,961
12	5,577	6,749	6,824	7,098	5,606	4,912	4,617	3,676	3,593	3,463	3,866	5,437
13	5,061	8,015	7,053	7,949	5,521	5,155	5,113	5,354	3,819	3,246	4,227	5,333
14	5,664	8,108	7,193	8,068	5,405	5,225	4,718	5,460	3,958	3,128	3,872	5,486
15	5,451	6,564	6,791	7,346	5,621	4,838	4,503	4,829	4,023	3,178	4,062	6,332
16	5,840	6,167	7,183	7,791	5,561	4,552	4,880	4,299	3,899	3,248	4,064	5,910
17	6,253	6,370	6,959	7,460	5,493	4,819	4,899	4,407	3,956	3,186	4,004	5,658
18	6,401	6,974	6,071	6,864	5,345	5,148	4,658	4,863	3,937	3,299	4,532	5,487
19	6,468	6,621	6,076	6,996	5,339	4,973	5,127	4,654	3,711	3,282	4,809	5,520
20	7,141	6,584	6,982	7,193	5,422	5,021	4,759	4,114	3,667	3,340	4,662	5,688
21	7,074	7,106	7,352	6,842	5,789	5,171	4,663	4,012	3,741	3,639	4,303	6,548
22	6,061	7,211	8,108	6,423	5,717	5,128	4,353	4,114	3,478	3,333	4,507	6,862
23	5,743	6,675	8,035	6,193	5,491	4,999	4,414	4,290	3,301	3,131	4,308	6,130
24	5,919	6,069	8,340	6,133	5,611	5,239	4,326	4,160	3,375	3,287	4,284	5,631
25	6,107	5,968	7,747	6,176	5,157	5,323	4,268	4,246	3,428	3,189	4,317	4,358
26	5,687	6,205	7,591	6,311	4,968	5,114	4,391	3,963	3,705	3,524	4,400	4,748
27	5,432	6,620	7,547	6,261	4,722	4,701	4,367	3,760	3,852	3,427	4,870	6,071
28	5,945	6,525	7,624	6,064	4,845	4,901	4,231	4,016	3,731	3,201	5,000	5,934
29	5,903	5,381	7,737	6,111	5,369	5,269	4,003	4,081	3,386	3,481	5,503	6,425
30	5,555		8,100	6,932	5,325	4,942	4,129	4,709	3,125	3,492	5,053	6,429
31	6,005		8,063		5,419		4,098	5,175		3,446		5,769
Average:	5,987	6,508	7,126	7,255	5,555	5,142	4,396	4,344	3,785	3,342	4,248	5,578

1 -- Available period of record may be less than value shown for certain days of the year.

Source: Adapted from USGS 2006b

Table 2.4.1-5 Daily Mean Flow Data for the Savannah River at Augusta, Georgia (USGS Gage 2197000)

Day of month	Mean of daily mean values for this day for 98 years of record[1], in ft^3/s											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	10,790	11,320	17,390	16,289	10,680	8,129	7,708	8,359	8,281	7,717	5,987	8,172
2	11,380	11,860	15,900	16,230	10,950	8,078	8,381	8,139	8,205	10,460	6,316	7,694
3	11,360	11,960	14,110	17,210	10,570	8,107	7,871	8,541	7,546	10,080	6,574	7,651
4	12,460	12,860	13,420	15,820	10,130	7,917	7,126	8,446	7,586	8,478	6,847	8,232
5	13,170	13,380	14,440	14,099	9,711	7,943	7,085	7,901	7,451	7,249	6,990	8,680
6	12,130	13,339	14,920	15,170	9,621	8,233	7,356	8,065	7,634	7,143	6,782	8,617
7	11,860	13,850	15,029	15,920	9,875	8,760	7,357	8,125	7,709	6,793	6,303	8,444
8	12,600	15,250	15,910	15,740	10,160	8,985	7,993	7,921	7,986	6,526	6,310	8,281
9	12,650	15,590	16,410	15,490	10,140	8,532	8,653	8,440	7,689	6,696	6,763	8,289
10	12,080	15,459	16,070	15,120	10,110	8,316	8,541	8,329	8,819	7,243	6,846	8,670
11	11,550	15,330	14,549	14,560	9,318	8,103	7,732	7,352	9,687	7,243	6,650	8,512
12	11,790	15,190	13,940	13,650	8,830	8,026	7,387	7,287	7,867	7,047	6,635	8,372
13	12,240	14,620	14,520	12,780	8,648	8,111	7,342	7,680	6,671	7,058	6,901	8,580
14	11,610	14,330	14,940	12,730	8,600	8,570	7,788	8,807	6,223	6,582	7,357	8,793
15	11,200	14,090	14,690	13,110	8,388	8,829	7,669	9,442	6,372	6,121	7,344	9,559
16	10,860	13,469	15,490	13,619	8,393	9,036	7,872	9,381	6,331	5,916	7,227	10,260
17	11,570	13,880	15,880	13,450	8,369	8,825	7,699	9,570	6,543	6,188	7,475	9,995
18	12,350	15,020	14,779	12,270	7,988	8,540	7,635	9,034	7,583	6,975	7,398	9,486
19	13,900	15,020	13,869	11,650	7,629	8,056	7,612	8,447	7,598	6,931	7,311	9,025
20	15,450	14,170	14,490	11,670	8,318	7,589	7,735	8,776	6,913	6,854	7,297	8,854
21	14,820	14,130	15,780	11,620	9,137	7,369	7,393	8,078	6,540	7,215	6,879	9,797
22	12,730	15,110	16,450	11,370	9,283	7,657	7,171	7,790	6,591	7,233	6,834	9,845
23	11,580	14,790	16,189	10,830	9,216	7,228	6,961	7,473	6,438	7,373	6,792	9,854
24	11,800	14,010	16,550	10,380	8,788	7,318	6,879	7,321	6,270	7,584	7,131	9,289
25	11,990	13,780	15,960	10,060	8,499	8,373	7,196	7,213	6,418	7,035	7,296	9,232
26	12,190	13,880	15,079	10,500	7,805	8,399	7,623	7,367	6,989	6,491	7,352	9,595
27	11,760	14,160	15,370	10,500	7,795	7,699	7,499	7,301	8,905	6,709	7,551	10,100
28	11,260	16,089	15,380	10,190	7,904	7,406	7,428	7,615	8,902	6,778	7,584	10,090
29	11,310	11,980	15,300	9,767	7,866	7,209	7,655	8,207	7,516	6,342	7,950	10,160
30	11,450		16,800	10,480	7,794	7,598	8,445	8,447	7,140	6,319	8,448	11,020
31	11,250		16,920		7,823		8,962	8,352		6,173		11,100
Average:	12,101	14,066	15,372	13,076	8,979	8,098	7,669	8,168	7,413	7,115	7,038	9,169

1 -- Available period of record may be less than value shown for certain days of the year.

Source: Adapted from USGS 2006c

Table 2.4.1-6 Daily Mean Flow Data for the Savannah River at Jackson, South Carolina (USGS Gage 2197320)

Day of month	Mean of daily mean values for this day for 31 years of record[1], in ft^3/s											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	8,843	10,990	10,650	11,520	9,351	8,778	8,337	7,511	7,725	7,052	7,188	8,115
2	9,091	11,140	11,050	10,540	8,757	8,383	7,974	7,581	7,334	7,079	7,167	8,850
3	9,807	11,920	11,320	10,560	8,860	7,941	7,691	7,778	7,141	7,541	7,088	8,730
4	9,931	11,990	11,470	10,660	8,858	8,393	7,922	7,877	7,433	7,708	7,193	8,524
5	9,759	11,430	12,559	10,900	9,146	8,316	7,743	7,420	7,791	7,885	7,261	8,674
6	9,677	11,560	12,140	11,150	8,650	8,323	8,097	7,441	7,891	7,779	7,233	8,840
7	9,407	11,650	12,040	10,630	8,578	8,328	8,102	7,409	7,778	7,589	7,218	8,908
8	9,032	11,730	12,160	10,290	7,630	8,169	7,924	7,463	7,395	7,581	7,141	9,053
9	9,086	11,620	12,240	10,180	7,377	8,247	7,316	7,566	7,322	7,791	7,225	9,121
10	9,402	11,830	12,020	10,470	8,088	7,944	7,700	7,752	7,428	7,937	7,354	8,978
11	9,922	11,430	11,100	10,920	7,937	8,374	7,524	7,465	7,247	7,994	7,435	9,219
12	10,540	11,980	11,480	10,510	8,381	8,175	7,107	7,766	7,042	7,991	7,510	9,271
13	10,800	12,060	11,790	10,360	8,695	8,682	7,079	7,695	7,059	7,850	7,542	9,356
14	10,870	11,850	11,920	9,937	8,551	8,554	7,042	7,798	7,047	7,693	7,745	9,084
15	10,640	11,930	11,740	9,614	8,096	8,441	7,183	7,859	7,299	7,367	8,222	9,007
16	10,430	11,840	11,510	10,490	8,221	8,061	7,270	7,835	7,208	7,330	8,354	9,235
17	10,510	10,920	11,570	10,510	8,368	7,730	7,478	7,945	7,015	7,739	7,940	9,326
18	10,770	10,540	11,340	10,150	8,784	7,774	7,583	8,110	6,855	7,308	7,681	9,248
19	11,290	11,110	10,750	9,529	9,375	7,715	7,551	8,038	6,841	7,717	7,734	9,064
20	11,480	10,840	10,560	9,320	8,814	7,670	7,688	7,437	6,826	7,695	7,644	9,841
21	11,260	10,200	10,800	9,484	8,461	8,276	7,558	7,482	6,702	7,905	7,584	9,628
22	11,430	10,260	10,990	9,388	8,173	8,800	7,393	7,431	7,010	7,758	7,739	9,536
23	11,580	10,760	10,220	9,379	8,739	8,878	7,469	7,361	7,161	7,848	8,381	9,469
24	11,300	11,080	9,758	9,780	9,255	8,404	7,360	7,312	7,366	8,257	8,387	9,350
25	11,240	11,250	10,010	9,456	9,503	8,230	7,209	7,335	7,141	8,340	8,529	9,362
26	10,980	11,090	11,160	9,380	9,236	8,154	7,234	7,284	7,216	8,108	8,117	9,653
27	10,900	11,380	11,150	9,780	9,021	8,113	7,057	7,332	7,115	7,974	7,992	9,524
28	11,230	10,990	10,860	9,542	8,956	8,240	6,866	7,430	6,977	8,022	7,863	9,155
29	10,720	10,540	11,550	9,237	9,177	8,481	6,835	8,035	7,106	7,759	8,077	8,781
30	10,850		11,950	9,728	9,396	8,469	7,195	7,984	7,017	7,360	8,527	8,777
31	10,870		11,900		9,236		7,465	7,957		7,160		8,816
Average:	10,440	11,307	11,347	10,113	8,699	8,268	7,482	7,635	7,216	7,713	7,702	9,113

1 -- Available period of record may be less than value shown for certain days of the year.

Source: Adapted from USGS 2006d

Table 2.4.1-7 Approximate Lengths and Slopes of Local Streams

Map ID	Stream Identification	Approximate length, ft **	Upstream Elevation	Outfall Elevation	Approximate Slope
1	Unnamed creek at Hancock Landing to the Savannah River	7,000	163	85	0.0111
2	Unnamed tributary to Daniels Branch to Daniels Branch	6,000	190	105	0.0142
3	Red Branch to Daniels Branch	10,500	235	115	0.0114
4	Daniels Branch D/S of embankment dam to confluence with Red Br.	5,500	140	115	0.0045
5	Unnamed tributary to Beaverdam Creek	8,500	235	87	0.0174
6	Beaverdam Creek to Telfair Pond	13,500	100	85	0.0011
7	Beaverdam Creek, D/S of Telfair Pond to Savannah River	21,000	190	105	0.0040

* Identifier for streams shown in Figure 2.4-3

** from outfall to end of longest tributary

Table 2.4.1-8 Inventory of Savannah River Watershed Water Control Structures

Name of Dam or Reservoir	Owner or Operator	Stream	Savannah River Mile	Distance U/S of Vogtle Site	Drainage Area above dam (sq. mi.)	Total Storage, in 1000's of acre-feet	Normal Pool Elev, ft MSL	Spillway Crest Elevation, ft. MSL	Top of Dam Elevation, ft. MSL	Generator Capacity, MW
New Savannah Bluff Lock & Dam	USACE	Savannah River	187.7	36.8	7,508	RoR	115.0	n/a	n/a	n/a
Stevens Creek	SC Electric & Gas	Savannah River	208.1	57.2	7,173	11	n/a	n/a	n/a	19.2
J. Strom Thurmond Lake & Dam	USACE	Savannah River	221.6	70.7	6,144	2,510	335.0	300	351	280
Richard B. Russell Lake & Dam	USACE	Savannah River	259.1	108.2	2,900	1,026	475.0	436	495	300
Hartwell Lake & Dam	USACE	Savannah River	288.9	138.0	2,088	2,550	660.0	630	679	330
Yonah Dam	GA Power Company	Tugaloo-Chatooga	340.0	189.1	470	10.2	744.2	742	757	22.5
Keowee Lake & Dam	Duke Power Company	Senaca-Keowee	341.0	190.1	439	940	800.0	765	815	157.5
Tugaloo Lake & Dam	GA Power Company	Tugaloo	343.1	192.2	464	43.2	891.5	885	905	45
Tallulah Falls Dam	GA Power Company	Tallulah River	346.7	195.8	186	2.46	1,500.0	1493	1514	72
Mathis Lake & Dam	GA Power Company	Tallulah River	353.4	202.5	151	31.4	1,689.6	1681	1704	16
Jocassee Lake & Dam	Duke Power Company	Senaca-Keowee	357.0	206.1	148	1,100	1,110.0	1077	1125	612
Nacoochee Dam	GA Power Company	Tallulah River	362.1	211.2	136	8.2	1,752.5	1753	1765	4.8
Little River Lake & Dam	Duke Power Company	Senaca-Keowee	366.0	215.1	439	940	800.0	765	815	n/a
Burton Lake & Dam	GA Power Company	Tallulah River	366.4	215.5	118	108	1,866.6	1860	1873	6.1

Source: Compiled from USACE 1996

Table 2.4.1-9 Surface Water Users on the Savannah River Near or Downstream of Proposed Units

Owner	Facility Type and Description	Source Water	River mile	Distance from VEGP	Average Daily Withdrawal	Reference
Savannah River Site, US DOE	Tritium Extraction Facility	Savannah River	151.0	-0.1	2.9 MGD (1)	DOE/EIS 1997
Georgia Power Company	Vogtle Electric Generating Plant	Savannah River	150.9	0.0	171.3 MGD (1)	DOE/EIS 1997
SCE&G	Coal-fired plant cooling water at SRS	Savannah River	151.0	-0.1	44.2 MGD (1)	DOE/EIS 1997
City of Savannah	Cherokee Hill Water Treatment Plant in Port Wentworth for treatment of industrial & domestic water	Savannah River	29.0	121.9	50.0 MGD	DOE/EIS 1997
Beaufort/Jaspar Water & Sewer Authority	W.T.P. Intake for WTP facility serving 75% of Beaufort Co. & 1% of Jasper Co.	Savannah River	39.2	111.7	16.0 MGD	DOE/EIS 1997
City of Waynesboro, Burke Co.	Water Treatment Plant intake for municipal water supply (12 miles overland from site)	Brier Creek	102.5	48.4	1.5 MGD (2)	Georgia DNR 2006
International Paper Corporation in Chatham Co., GA	Water treatment plant intake for industrial water supply (approximate river mile)	Savannah River	18.5 (3)	132.4	50.0 MGD (2)	Georgia DNR 2006
Kerr-McGee Chemical, LLC in Chatham Co., GA	Water treatment plant intake for industrial water supply (approximate river mile)	Savannah River	18.5 (3)	132.4	20.0 MGD (2)	Georgia DNR 2006
Georgia Power Company Riverside, GA	Water treatment plant intake for industrial water supply	Savannah River	18.5 (3)	132.4	174.0 MGD (2)	Georgia DNR 2006
Savannah Electric & Power Co-Pt Wentworth, GA	Water treatment plant intake for industrial water supply (approximate river mile)	Savannah River	18.5 (3)	132.4	267.0 MGD (2)	Georgia DNR 2006
Weyerheauser Company, Chatham Co., GA	Water treatment plant intake for industrial water supply (approximate river mile)	Savannah River	18.5 (3)	132.4	27.5 MGD (2)	Georgia DNR 2006
Weyerheauser Company, Chatham Co., GA	Water treatment plant intake for industrial water supply (approximate river mile)	Savannah River	18.5 (3)	132.4	30.0 MGD (2)	Georgia DNR 2006
Fort James Operating Company, Effingham, GA	Water Treatment Plant intake for municipal water supply	Savannah River	45	105.9	35.0 MGD (2)	Georgia DNR 2006
Savannah Electric & Power Co, McIntosh, GA	Water treatment plant intake for industrial water supply	Savannah River	45	105.9	130.0 MGD (2)	Georgia DNR 2006
Savannah Industrial & Domestic Water, Effingham Co., GA	Combined municipal and industrial water supply (near confluence with Savannah R.)	Abercorn Creek	29	121.9	55.0 MGD (2)	Georgia DNR 2006
J M Huber Corp -Brier Creek, in Warren Co., GA	Water treatment plant intake for industrial water supply (near confluence with Savannah R.)	Brier Creek	102.5	48.4	4.0 MGD (2)	Georgia DNR 2006

1) Average water use, 1998 interpolated to 2006 using 2010 projected value
2) Average water use, Georgia DNR 2006
3) Midpoint of the reach identified in Georgia DNR 2006

Table 2.4.1-10 Plant Water Use

Stream Description	Normal Case[a] gpm	Maximum Case[a,b] gpm	Comments
Groundwater (Well) Streams:			
Plant Well Water Demand	752	3,140	
Well Water for Service Water System Makeup	537	2,353	
• Service Water System Consumptive Use	403	1,177	
- Service Water System Evaporation	402	1,176	
- Service Water System Drift	1	1	c
• Service Water System Blowdown	134	1,176	d
Well Water for Power Plant Make-up/Use	215	787	
• Demineralized Water System Feed	150	600	
- Plant System Make-up/Processes	109	519	
- Misc. Consumptive Use	41	81	
• Potable Water Feed	42	140	
• Fire Water System	10	12	
• Misc. Well Water Users	13	35	
Surface Water (Savannah River) Streams			
River Water for Circulating Water / Turbine Plant Cooling Water System Make-up	37,224	57,784	
• Circulating Water / Turbine Plant Cooling Water System Consumptive Use	27,924	28,904	
- Circulating Water / Turbine Plant Cooling Water System Evaporation	27,900	28,880	
- Circulating Water / Turbine Plant Cooling Water System Drift	24	24	c
• Circulating Water / Turbine Plant Cooling Water System Blowdown	9,300	28,880	d

Table 2.4.1-10 (cont.) Plant Water Use

Stream Description	Normal Case[a] gpm	Maximum Case[a,b] gpm	Comments
Plant Effluent Streams			
Final Effluent Discharge to River	9,608	30,761	
• Blowdown Sump Discharge	9,605	30,561	
- Wastewater Retention Basin Discharge	171	505	
○ Miscellaneous Low Volume Waste	129	365	
○ Treated Sanitary Waste	42	140	
- Service Water System Blowdown	134	1,176	d
- Circulating Water / Turbine Plant Cooling Water System Blowdown	9,300	28,880	d
- Start-up Pond Discharge	0	0	e
• Treated Liquid Radwaste	3	200	f

NOTES:

[a] The flow rate values are for two AP1000 units.

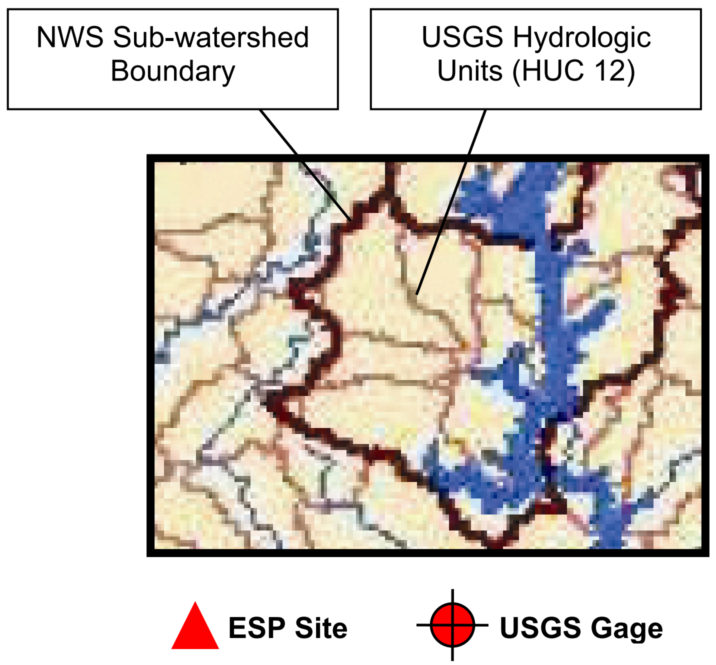
[b] These flows are not necessarily concurrent.

[c] The cooling tower drifts are 0.002% of the tower circulating water flow.

[d] For the normal case, the cooling towers are assumed operating at four cycles of concentration. For the service water cooling tower (maximum case), both unit towers are assumed operating at two cycles of concentration. For the main condenser / turbine auxiliary cooling water tower (maximum case), both towers are assumed operating at two cycles of concentration. Flows are determined by weather conditions, water chemistry, river conditions (circulating water / turbine plant cooling water system only) and operator discretion.

[e] Start-up flushes and start-up pond discharge would occur only during the initial plant start-up phase and potentially after unit outages when system flushes are required.

[f] The short-term liquid waste discharge flow rate may be up to 200 gpm. However, given the waste liquid activity level, the discharge rate must be controlled to be compatible with the available dilution (cooling tower blowdown) flow.



NWS Sub-Watersheds	
1	Burton Dam, GA
2	Jocasse Dam, SC
3	Keowee Dam, SC
4	Hartwell Dam, GA
5	R. B. Russell Dam
6	Carlton Bridge, GA
7	Clark Hill – Thurmon Dam (upstream)
8	Clark Hill Dam
9	Modoc, SC
10	Steven Creek Dam, GA
11	Augusta 5th Street
12	Augusta/Butler Creek
13	Jackson, SC
14	Burton's Ferry, GA
15	Millvaven, GA
16	Clyo, GA

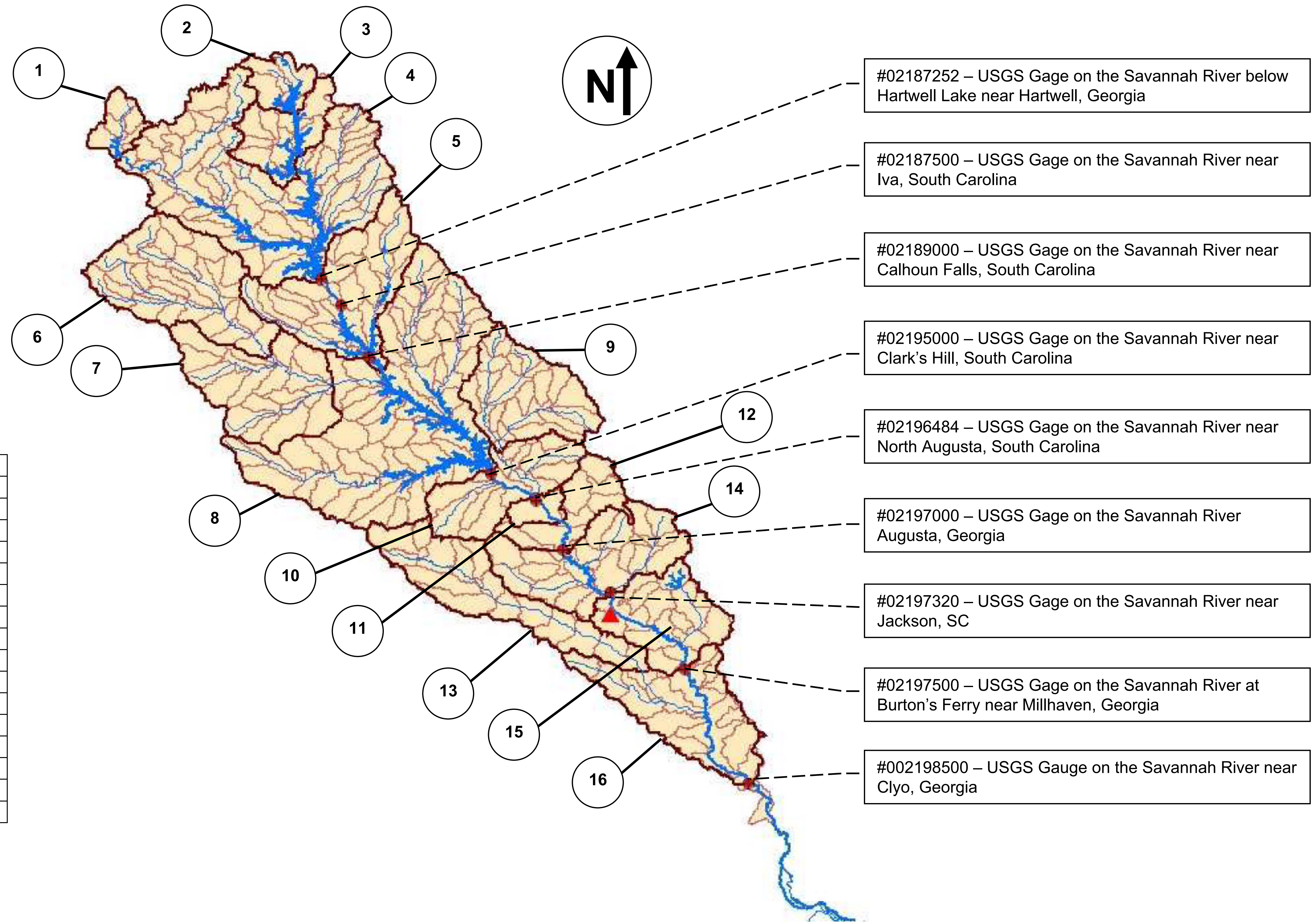


Figure 2.4.1-1 Savannah River Watershed and HUCs (No Scale)

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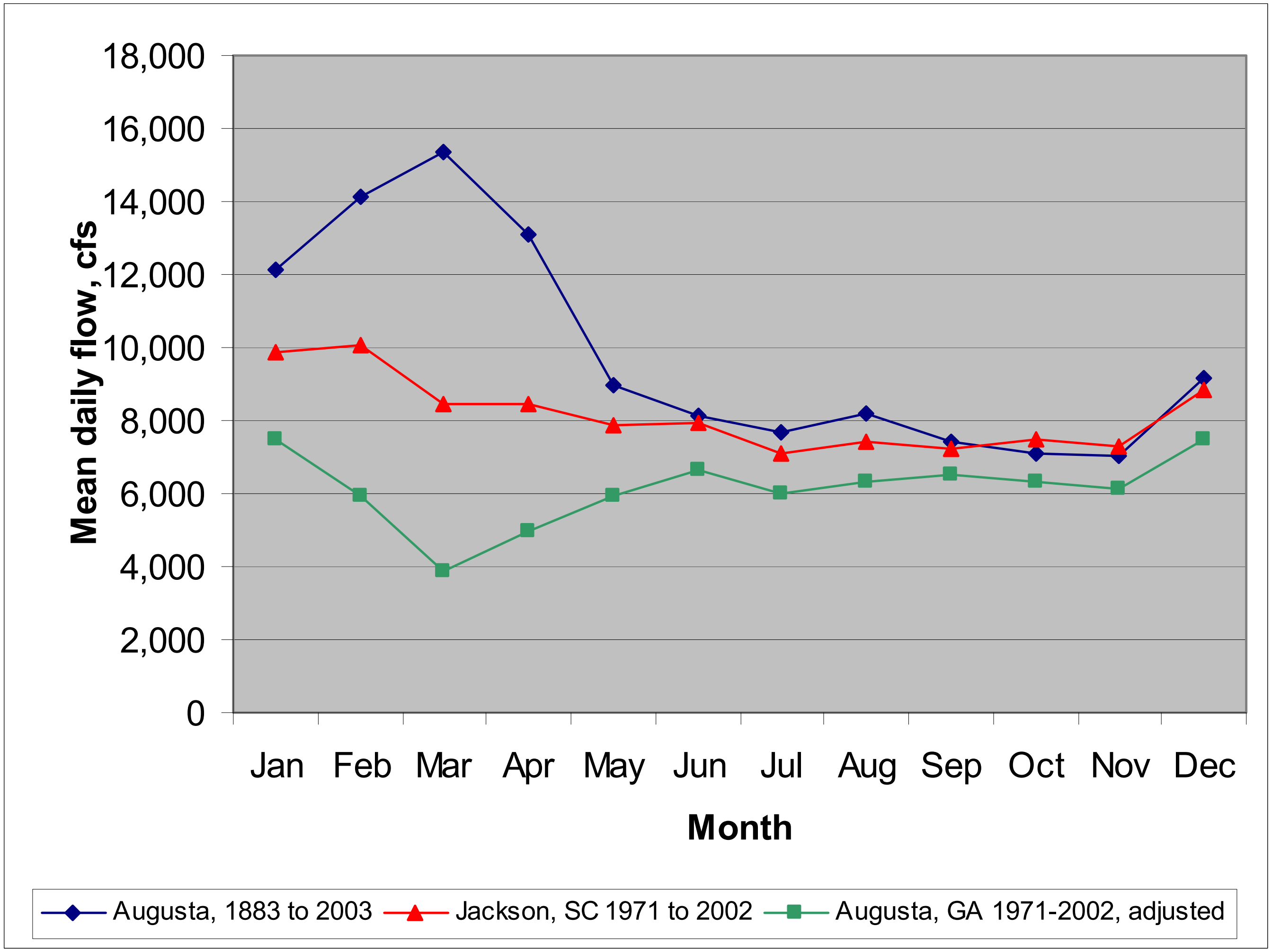

Figure 2.4.1-2 Mean Daily Discharge for the Year – Selected Gages of the Savannah River

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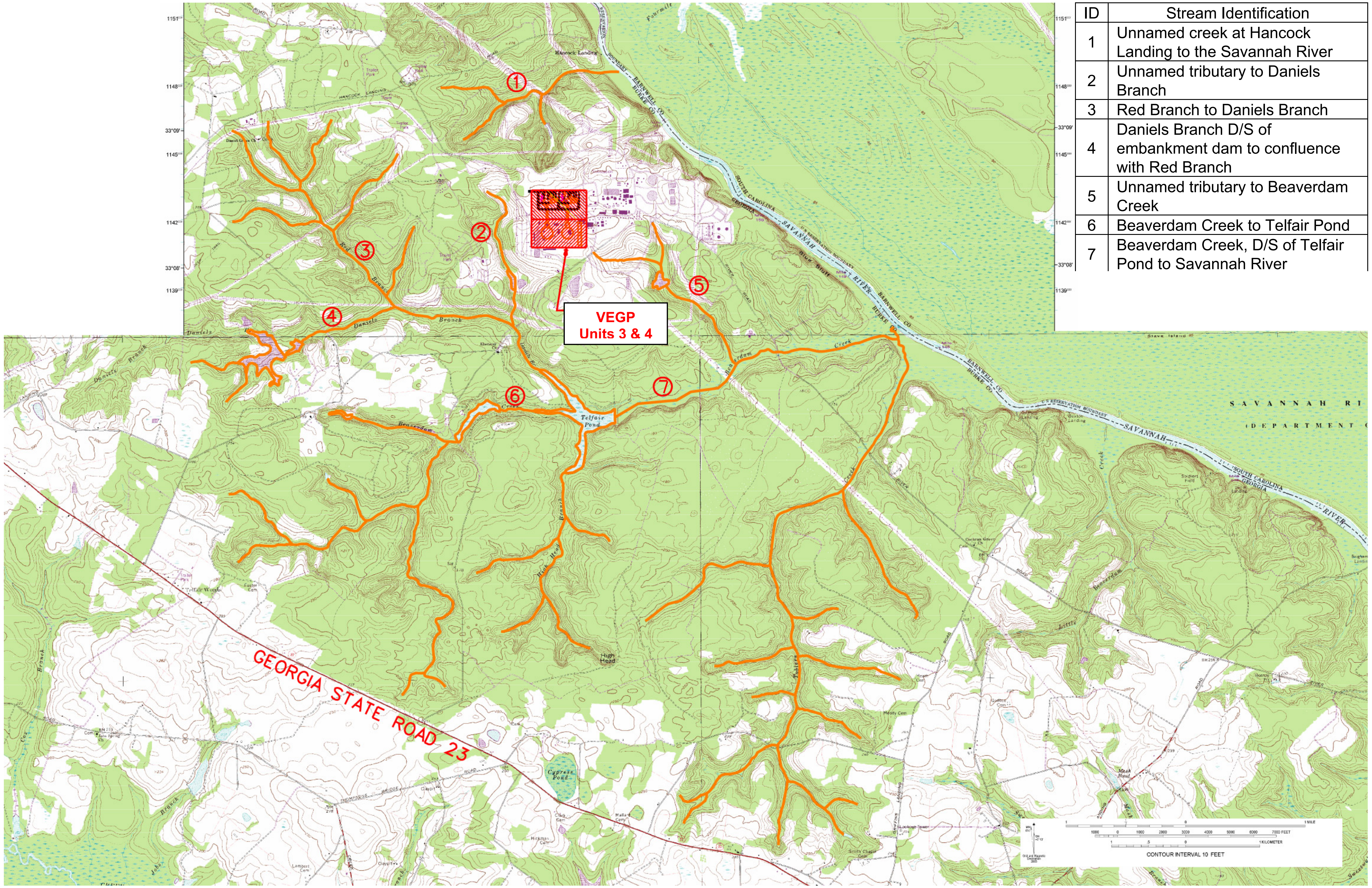

ID	Stream Identification
1	Unnamed creek at Hancock Landing to the Savannah River
2	Unnamed tributary to Daniels Branch
3	Red Branch to Daniels Branch
4	Daniels Branch D/S of embankment dam to confluence with Red Branch
5	Unnamed tributary to Beaverdam Creek
6	Beaverdam Creek to Telfair Pond
7	Beaverdam Creek, D/S of Telfair Pond to Savannah River

Figure 2.4.1-3 Site Drainage

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Section 2.4.1 References

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(USGS 2006b) USGS Stream Gage 302198000 Savannah River at Calhoun Falls, South Carolina http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02198000 (accessed 1-17-2006).

(USGS 2006c) USGS Stream Gage 302197000 Savannah River at Augusta, Georgia http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02197000. (accessed 1-17-2006).

(USGS 2006d) USGS Stream Gage 302197320 Savannah River near Jackson, South Carolina http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02197320 (accessed 1-17-2006).

(USGS 2006e) USGS Stream Gage 302198500 Savannah River near Clyo, Georgia http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02198500 (accessed 1-17-2006).

(USGS 2006f) US Geological Survey, South Carolina Office; Contact for access to HUC-12 shapefiles for the Savannah River: malowery@usgs.gov (accessed 1-26-2006).

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2.4.2 Floods

2.4.2.1 Flood History

Potential causes of flooding at the site are limited to local runoff events due to intense point-rainfall near the site and flooding from the Savannah River. There is no historical record of flooding due to storm surges or tsunamis at the site, which is consistent with its location approximately 150 River Miles inland from the ocean. Because there are no large bodies of water near the site, flooding due to seiche motion was not considered (see Sections 2.4.5 and 2.4.6).

Table 2.4.2-1 **(USGS 2006a)** provides the date, stage elevation, and annual peak discharge for the entire period of record of USGS stream gage 02197000 on the Savannah River at Augusta, Georgia, approximately 48.7 River Miles upstream of the VEGP site. The annual peak floods include estimated values from historic floods reported in 1796, 1840, 1852, 1864, 1865, and 1876.

The maximum annual peak flood discharge for the period of record is 350,000 cfs from the storm of October 2, 1929. The storm of January 17, 1796, estimated from reported stages using slope-conveyance methods, is the oldest event used to extend the record length. The estimated value of the peak flow for this storm ranges from 280,000 cfs for a reported stage of 38 ft **(USGS 2006a)** to 360,000 cfs for a reported maximum flood stage of 40 ft **(USGS 1990)**. This puts the maximum flood elevation of the Savannah River at Augusta, Georgia, for the historic period between 134.6 and 136.6 ft msl, based on an elevation of 96.58 ft msl for the Augusta, Georgia, stream gage datum (see Table 2.4.2-1).

Since 1952, annual peaks on the Savannah River at Augusta, Georgia, have been impacted by regulation from upstream reservoirs: J. Strom Thurmond (also known as Clarks Hill) Lake and Dam in 1952, Hartwell Lake and Dam in 1961, and Richard B. Russell Lake and Dam in 1984 **(USACE 1996)**. In Figure 2.4.2-1 **(USGS 1990)**, which is based on the historical record from 1796 to 1985, this impact is shown by the pronounced reduction of peak flows after 1952. The addition of annual peak stream gage data from 1986 to 2002 would not significantly affect this graph, as indicated by the following averages:

Average annual peak discharge 1796 - 1950:	232,696 cfs
Average annual peak discharge 1876 - 1950:	113,086 cfs
Average annual peak discharge 1951 - 2004:	34,343 cfs
Average annual peak discharge 1951 - 1985:	37,569 cfs
Average annual peak discharge 1986 - 2004:	28,734 cfs

The USGS stream gage at Jackson, South Carolina, which is approximately 5.9 River Miles upstream of the VEGP site (see Table 2.4.1-2), has a record length significantly shorter than that of the Augusta gage and contains no observations before upstream dams were closed.

Table 2.4.2-2 compares the annual peak discharges on the Savannah River at Augusta, Georgia, and Jackson, South Carolina, for the 29 coincident years of record. During this period, the peak annual discharge at the two sites was not associated with the same storm event in seven instances. These cases are indicated by the grayed-out rows of Table 2.4.2-2, for which the dates of the peaks differ by a significant number of days. There is a 1-to-2-day lag in the occurrence of annual maximum peaks at the two gages derived from the same flood event. A very strong linear correlation exists between flood stages at the two sites for the annual peak floods derived from the same event, as shown in Figure 2.4.2-2, making it feasible to extend the historical record at Jackson, South Carolina. The annual peak flood stage at the VEGP site could then be estimated from the stages at Jackson, with a level of confidence dependent on the ability to establish a reliable estimate of the stage at the VEGP site from the river stage at Jackson, South Carolina, based on hydraulic considerations.

Annual peak flood frequency curves for regulated and unregulated conditions for the Savannah River at Augusta, Georgia, were developed for the period between 1796 and 1985 and are presented in Figure 2.4.2-3 **(USGS 1990)**. Unregulated annual peak discharge values for the period after 1952 and regulated annual peak discharge values for the years before 1952 were generated by modeling reservoir operation based on the stage-storage-discharge characteristics reported for the three projects, using the 1990 operating rule set for the entire period **(USGS 1990)**.

Figure 2.4.2-3 clearly shows the convergence of the regulated and unregulated annual flood frequency plots with increasing flood size. On the left side of the graph, for the 80 percent chance-of-exceedence event (a 1.25-year return period), the unregulated peak discharge exceeds the regulated peak by more than 100 percent; on the right side, for the 0.2 percent chance-of-exceedence event (500-year return period), the unregulated peak discharge exceeds the regulated peak by about 30 percent. Based on this trend, regulation would not be expected to significantly affect the probable maximum flood on the Savannah River downstream of Augusta, provided that the regulating structures do not fail. Flooding due to dam-breaks is discussed in Section 2.4.4.

2.4.2.2 Flood Design Considerations

The location of VEGP Units 3 and 4 would be adjacent to and generally to the west of existing VEGP Units 1 and 2, as illustrated in Figure 1-4. The site is located on a high bluff on the west bank of the Savannah River. The proposed site grade for the new units will be at or above El. 220 ft msl, similar to the existing VEGP units, well above the probable maximum flood stage of the Savannah River, as discussed in Section 2.4.3.

The annual maximum flood at the VEGP site can occur in any month of the year and is not associated specifically with icing, which does not normally occur to any significant degree, as indicated in Section 2.4.7). For this reason, the effect of ice accumulation on runoff was not taken into account in selecting the design flood.

The design basis flood for the VEGP site was determined by selecting the maximum flood elevation on the Savannah River obtained by considering all flooding scenarios applicable to the location, including an approximate estimate of the probable maximum flood (PMF), flooding due to probable maximum precipitation (PMP) over local drainage courses, and potential dam failures coincident with wind set-up and wave run-up. Flood surge from ocean storms and tsunami-caused flooding were not considered because the VEGP site is approximately 151 river miles inland.

Each applicable flooding scenario was evaluated following guidelines provided in Regulatory Guide 1.59, *Design Basis Floods for Nuclear Power Plants*, 1977 (RG 1.59) and ANSI/ANS-2.8, *Determining Design Basis Flooding at Power Reactor Sites* **(ANSI/ANS-2.8-1992)**, as detailed in Sections 2.4.3 through 2.4.7.

The controlling event for the VEGP site was determined to be from the breach of the upstream dams, estimated as described in Section 2.4.4, using the Standard Project Flood discharge as a starting condition, including wind set-up and wave run-up. The design basis flooding level derived from this event, including wave setup, is El. 178.10 ft msl, which is 41.9 ft below the proposed site grade elevation of 220.0 ft msl.

Elevations for safety-related components and structures are not yet established for the proposed units. However, the grade elevation in the power block area of the VEGP site would be approximately the same as the existing units, elevation 220 ft msl, providing over 41 ft of freeboard above the design basis flooding level. Freeboard for all above-grade, safety-related structures, systems, and components of the new units will be equal to or greater than this value.

2.4.2.3 Effects of Local Intense Precipitation

The design basis for local intense precipitation at the site is the PMP, which is defined as the "greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of year" **(HMR-52 1982)**. Maps of the PMP are published for durations ranging from 6 to 72 hours and for watershed areas ranging from 10 to 20,000 sq mi **(HMR-51 1978)**.

As can be seen in Figure 2.4.1-3, the VEGP site is situated on high ground in such a manner that the areas to be drained by each conveyance system serving the site will be on the order of 1 sq mi, with times of concentration considerably less than 6 hours. The 1-sq-mi PMP for the VEGP site is calculated for a range of durations between 5 and 60 minutes from the 10-sq-mi, 6-hour, all-season average PMP depth, using multipliers following accepted engineering practice **(HMR-52, 1982)**. These values of depth are used to develop a relation between rainfall intensity and durations for the PMP, which will be used for storm drain designs at the VEGP site. The point values used for developing the relation are listed in Table 2.4.2-3 and the estimated curve is plotted in Figure 2.4.2-4.

The existing storm water system provides positive drainage away from the site for the runoff generated by the PMP: surface runoff flows away from the high ground on which the Unit 1 and 2 structures are located and is collected in four principal drainage channels aligned in concert with access roads and railroad facilities to outfall to the north, south, east, and west.

The locations and designs of storm water management systems for the new units at the VEGP site have not been determined for this ESP application. This will be done as part of detailed engineering and will be described in the COL application. In general, the storm water management system developed for Units 3 and 4 will be integrated with the existing facilities as possible; runoff from Units 3 and 4 will be directed away from Unit 1 and 2 structures, to outfall to the west and south of the VEGP site.

The storm drain system will be designed in accordance with good engineering practice, following all applicable federal, state, and local storm water management regulations. In addition, site grading will be sufficiently sloped to convey runoff overland from the PMP event, away from all buildings and safety-related equipment, without flooding, even if all catch basins and roof drains are plugged.

Table 2.4.2-1 Annual Peak Discharge for USGS Gage 2197000 on the Savannah River at Augusta, Georgia

Water Year	Date	Gage Height (feet)	Stream- flow (cfs)	Water Year	Date	Gage Height (feet)	Stream- flow (cfs)
1796	Jan. 17, 1796	38	280,000 (2)	1937	Jan. 04, 1937	30.1	91,400
1840	May 28, 1840	37.5	260,000 (2)	1938	Oct. 21, 1937	30.1	91,400
1852	Aug. 29, 1852	36.8	230,000 (2)	1939	Mar. 02, 1939	24.1	90,900
1864	Jan. 01, 1864	34	160,000 (2)	1940	Aug. 15, 1940	29.4	239,000
1865	Jan. 11, 1865	36.4	220,000 (2)	1941	Jul. 08, 1941	22.89	53,300
1876	Dec. 30, 1875	28.6	86,400	1942	Mar. 23, 1942	24.56	105,000
1877	Apr. 14, 1877	31.4	119,000	1943	Jan. 20, 1943	25.1	117,000
1878	Nov. 23, 1877	23.5	51,500	1944	Mar. 22, 1944	25.53	128,000
1879	Aug. 03, 1879	22	44,000	1945	Apr. 27, 1945	23.16	64,000
1880	Dec. 16, 1879	30.1	102,000	1946	Jan. 09, 1946	24.43	97,200
1881	Mar. 18, 1881	32.2	130,000	1947	Jan. 22, 1947	23.97	86,000
1882	Sep. 12, 1882	29.3	93,300	1948	Feb. 10, 1948	23.9	83,200
1883	Jan. 22, 1883	30.8	111,000	1949	Nov. 30, 1948	26.61	154,000
1884	Apr. 16, 1884	28	81,000	1950	Oct. 09, 1949	20.1	32,500
1885	Jan. 26, 1885	27.5	77,000	1951	Oct. 22, 1950	22.32	46,300
1886	May 21, 1886	32.5	135,000	1952	Mar. 06, 1952	21.53	39,300 (5)
1887	Jul. 31, 1887	34.5	173,000	1953	May 8, 1953	20.8	35,200 (6)
1888	Sep. 11, 1888	38.7	303,000	1954	Mar. 30, 1954	17.39	25,500 (6)
1889	Feb. 19, 1889	33.3	149,000	1955	Apr. 15, 1955	16.77	23,900 (6)
1890	Feb. 27, 1890	22.9	48,500	1956	Apr. 12, 1956	14.7	18,600 (6)
1891	Mar. 10, 1891	35.5	197,000	1957	May 7, 1957	14.08	18,000 (6)
1892	Jan. 20, 1892	32.8	140,000	1958	Apr. 18, 1958	22.91	66,300 (6)
1893	Feb. 14, 1893	25	60,000	1959	Jun. 08, 1959	18.65	28,500 (6)
1894	Aug. 07, 1894	24	54,000	1960	Feb. 14, 1960	20.58	34,900 (6)
1895	Jan. 11, 1895	30.4	106,000	1961	Apr. 02, 1961	20.56	34,800 (6)
1896	Jul. 10, 1896	30.5	107,000	1962	Jan. 09, 1962	20.09	32,500 (6)
1897	Apr. 06, 1897	29.3	93,300	1963	Mar. 23, 1963	19.52	31,300 (6)
1898	Sep. 02, 1898	31.3	117,000	1964	Apr. 09, 1964	24.16	87,100 (6)
1899	Feb. 08, 1899	31	113,000	1965	Dec. 27, 1964	20.62	34,600 (6)
1900	Feb. 15, 1900	32.7	138,000	1966	Mar. 06, 1966	21.5	39,300 (6)
1901	Apr. 04, 1901	31.8	124,000	1967	Aug. 25, 1967	18.1	26,500 (6)
1902	Mar. 01, 1902	34.6	175,000	1968	Jan. 12, 1968	20.94	35,900 (6)
1903	Feb. 09, 1903	33.2	147,000	1969	Apr. 21, 1969	22.24	45,600 (6)
1904	Aug. 10, 1904	25.5	63,000	1970	Apr. 01, 1970	17.68	25,200 (6)
1905	Feb. 14, 1905	25.8	64,800	1971	Mar. 05, 1971	23.3	63,900 (6)
1906	Jan. 05, 1906	29.6	96,600	1972	Jan. 20, 1972	20.36	33,700 (6)
1907	Oct. 05, 1906	23.6	52,000	1973	Apr. 08, 1973	21.63	40,200 (6)
1908	Aug. 27, 1908	38.8	307,000	1974	Feb. 23, 1974	20.13	32,900 (6)
1909	Jun. 05, 1909	28.7	87,300	1975	Mar. 25, 1975	22.24	45,600 (6)
1910	Mar. 02, 1910	26.4	69,800	1976	Jun. 05, 1976	20.27	33,300 (6)
1911	Apr. 14, 1911	19.1	32,800	1977	Apr. 07, 1977	20.5	34,200 (6)
1912	Mar. 17, 1912	36.8	234,000	1978	Jan. 26, 1978	21.98	43,100 (6)
1913	Mar. 16, 1913	35.1	156,000	1979	Feb. 27, 1979	21.13	37,300 (6)
1914	Dec. 31, 1913	24.3	48,000	1980	Mar. 31, 1980	22.33	47,200 (6)
1915	Jan. 20, 1915	28.2	61,000	1981	Feb. 12, 1981	14.7	17,700 (6)
1916	Feb. 03, 1916	31	82,400	1982	Jan. 02, 1982	19.39	30,700 (6)
1917	Mar. 06, 1917	29.2	68,000	1983	Apr. 10, 1983	23.21	66,100 (6)
1918	Jan. 30, 1918	25.5	45,500	1984	5-May-84	20.35	34,000 (6)
1919	Dec. 24, 1918	35	128,000	1985	Feb. 07, 1985	17.89	25,700 (6)
1920	Dec. 11, 1919	35.4	133,000	1986	Oct. 03, 1985	15.74	21,000 (6)
1921	Feb. 11, 1921	35.1	129,000	1987	Mar. 06, 1987	18.98	29,200 (6)
1922	Feb. 16, 1922	32	92,000	1988	Feb. 05, 1988	10.61	13,600 (6)
1923	Feb. 28, 1923	28	59,700	1989	Sep. 22, 1989	15.33	20,200 (6)
1924	Sep. 22, 1924	28	59,700	1990	Feb. 27, 1990	20.69	35,300 (6)
1925	Jan. 20, 1925	36.5	150,000	1991	Oct. 13, 1990	22.8	59,200 (6)
1926	Jan. 20, 1926	27.3	55,300	1992	Mar. 27, 1992	16.29	22,100 (6)
1927	Dec. 30, 1926	24	39,000	1993	Jan. 14, 1993	21.81	45,100 (6)
1928	Aug. 17, 1928	40.4	226,000	1994	Jul. 01, 1994	21.4	40,700 (6)
1929	Sep. 27, 1929	46.3	343,000	1995	Feb. 19, 1995	20.28	33,600 (6)
1930	Oct. 02, 1929	45.1	350,000	1996	Feb. 05, 1996	20.48	34,400 (6)
1931	Nov. 17, 1930	19.9	26,100	1997	Mar. 10, 1997	18.11	26,300 (6)
1932	Jan. 09, 1932	30.4	93,800	1998	Feb. 07, 1998	21.63	43,000 (6)
1933	Oct. 18, 1932	30.3	92,600	1999	Feb. 02, 1999	14.72	19,000 (6)
1934	Mar. 05, 1934	28.5	73,200	2000	Jan. 25, 2000	13.25	16,800 (6)
1935	Mar. 14, 1935	27.4	63,700	2002	Mar. 04, 2002	7.14	8,510 (6)
1936	Apr. 08, 1936	41.2	258,000	2003	24-May-03	20.42	31,600 (6)
				2004	Jun. 14, 2004	13.82	17,600 (6)

2 -- Discharge is an Estimate
5 -- Discharge affected to unknown degree by Regulation or Diversion
6 -- Discharge affected by Regulation or Diversion

Source: USGS 2006c

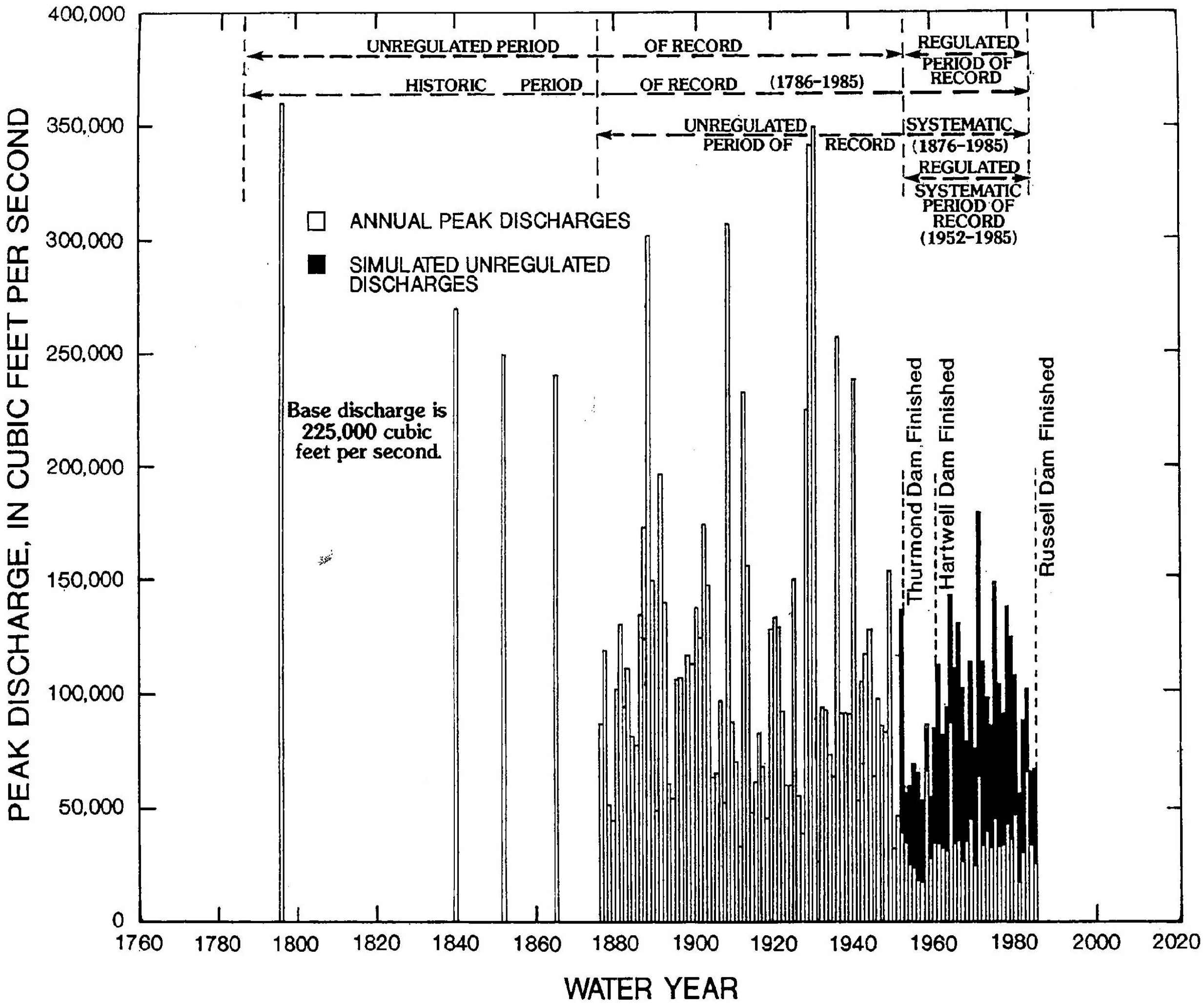
Table 2.4.2-2 Comparison of Annual Peak Discharges on the Savannah River at Augusta, Georgia and Jackson, South Carolina for 1972 to 2002

	Savannah River at Augusta, GA			Savannah River at Jackson, SC		
Water Year (Oct - Sept)	Date of annual peak discharge	Gage Height (feet)	Stream-flow (cfs)	Date of annual peak discharge	Gage Height (feet)	Stream-flow (cfs)
1972	Jan. 20, 1972	20.36	33,700	Jan. 21, 1972	19.02	n/r
1973	Apr. 08, 1973	21.63	40,200	Apr. 09, 1973	19.71	n/r
1974	Feb. 23, 1974	20.13	32,900	Feb. 24, 1974	18.64	n/r
1975	Mar. 25, 1975	22.24	45,600	Sep. 16, 1975	20.22	n/r
1976	Jun. 05, 1976	20.27	33,300	Jul. 06, 1976	18.84	n/r
1977	Apr. 07, 1977	20.5	34,200	Apr. 08, 1977	18.85	n/r
1978	Jan. 26, 1978	21.98	43,100	Jan. 28, 1978	19.65	n/r
1979	Feb. 27, 1979	21.13	37,300	Apr. 28, 1979	19.12	n/r
1980	Mar. 31, 1980	22.33	47,200	Apr. 01, 1980	20.72	n/r
1981	Feb. 12, 1981	14.7	17,700	Feb. 13, 1981	15.16	17300
1982	Jan. 02, 1982	19.39	30,700	Feb. 20, 1982	17.12	20500
1983	Apr. 10, 1983	23.21	66,100	Apr. 11, 1983	21.57	n/r
1984	May 5, 1984	20.35	34,000	Mar. 09, 1984	19.3	n/r
1985	Feb. 07, 1985	17.89	25,700	Feb. 08, 1985	17.21	20600
1986	Oct. 03, 1985	15.74	21,000	Nov. 24, 1985	14.29	15900
1987	Mar. 06, 1987	18.98	29,200	Mar. 07, 1987	18.35	n/r
1988	Feb. 05, 1988	10.61	13,600	Feb. 06, 1988	12.42	13200
1989	Sep. 22, 1989	15.33	20,200	Sep. 23, 1989	14.9	16800
1990	Feb. 27, 1990	20.69	35,300	Feb. 28, 1990	19.61	n/r
1991	Oct. 13, 1990	22.8	59,200	Oct. 14, 1990	20.05	n/r
1992	Mar. 27, 1992	16.29	22,100	Mar. 27, 1992	16.26	18800
1994	Jul. 01, 1994	21.4	40,700	Jul. 03, 1994	19.19	n/r
1995	Feb. 19, 1995	20.28	33,600	Feb. 20, 1995	18.91	n/r
1996	Feb. 05, 1996	20.48	34,400	Mar. 16, 1996	18.86	n/r
1997	Mar. 10, 1997	18.11	26,300	Mar. 11, 1997	18.41	n/r
1998	Feb. 07, 1998	21.63	43,000	Feb. 09, 1998	19.83	n/r
1999	Feb. 02, 1999	14.72	19,000	Oct. 28, 1998	15.23	17300
2000	Jan. 25, 2000	13.25	16,800	Jan. 26, 2000	14.86	16500
2002	Mar. 04, 2002	7.14	8,510	Mar. 05, 2002	8.77	8870

Source: Based on data from USGS 2006c and 2006d

Table 2.4.2-3 Probable Maximum Precipitation Values for Point Rainfall at VEGP Site

Duration	Watershed Area, mi^2	Multiplier	Applied to	Source	PMP depth (inches)
6-hour	10	n/a	n/a	HMR-51, Fig 18	31.0
1-hour	1	0.620	6-hr 10 mi^2 value	HMR-52, Fig 23	19.2
30-minutes	1	0.736	1-hr 1 mi^2 value	HMR-52, Fig 38	14.1
15-minutes	1	0.509	1-hr 1 mi^2 value	HMR-52, Fig 37	9.8
5-minutes	1	0.323	1-hr 1 mi^2 value	HMR-52, Fig 36	6.2



Source: Figure 2 from USGS 1990

Figure 2.4.2-1 Unregulated and Regulated Peak Discharge Frequency Curves for the Savannah River at Augusta, Georgia (02197000)

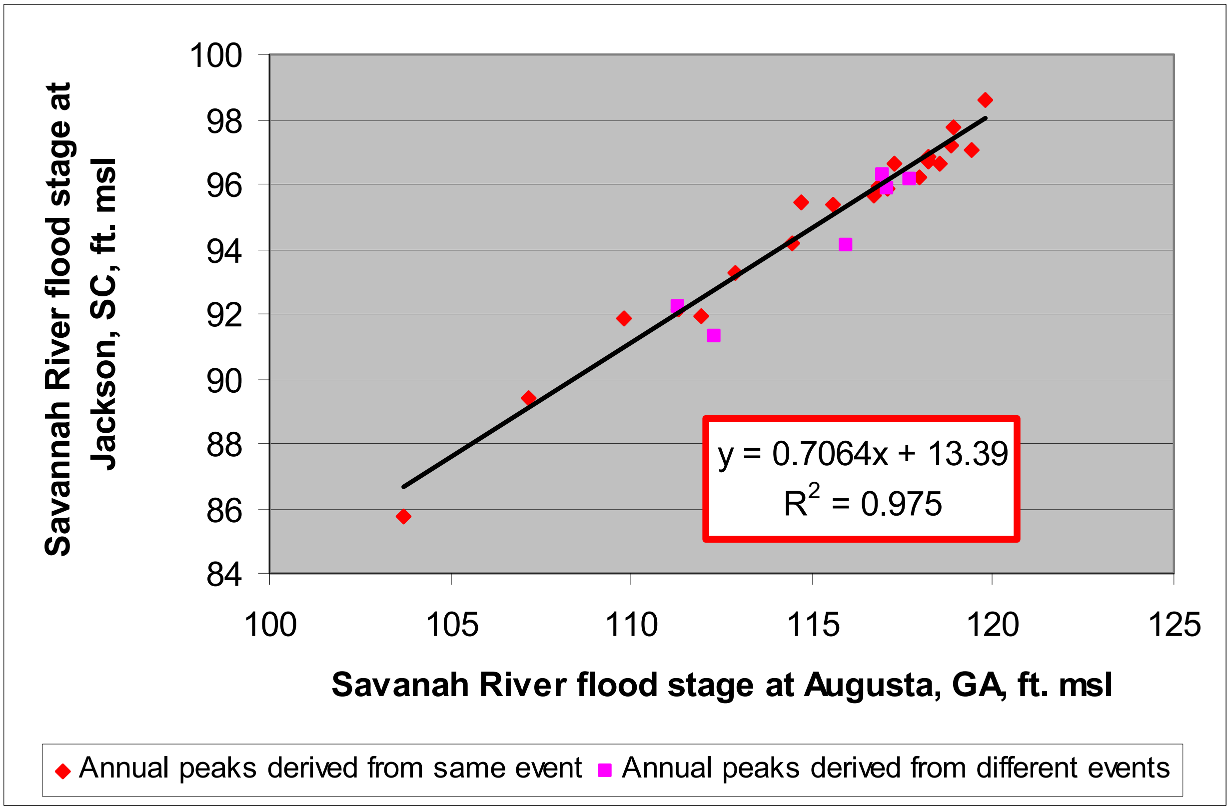
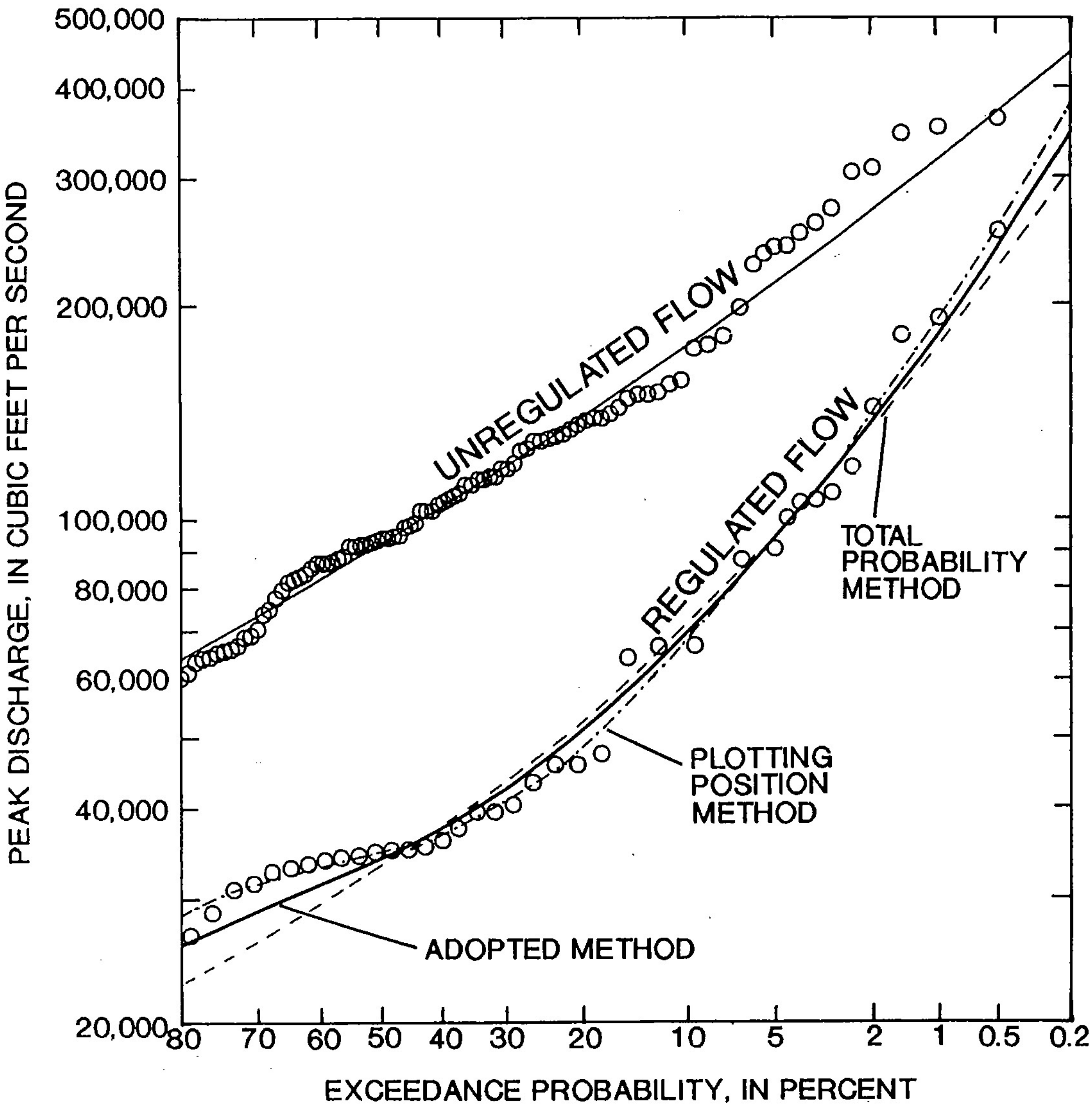


Figure 2.4.2-2 Correlation of Annual Peak Discharges on the Savannah River at Augusta, Georgia (02197000), and Jackson, South Carolina (2197320), for Years with Annual Peak Derived from Same Storm Event



Source: Figure 35 from USGS 1990

Figure 2.4.2-3 Unregulated and Regulated Annual Peak Discharge Frequency Curves for the Savannah River at Augusta, Georgia

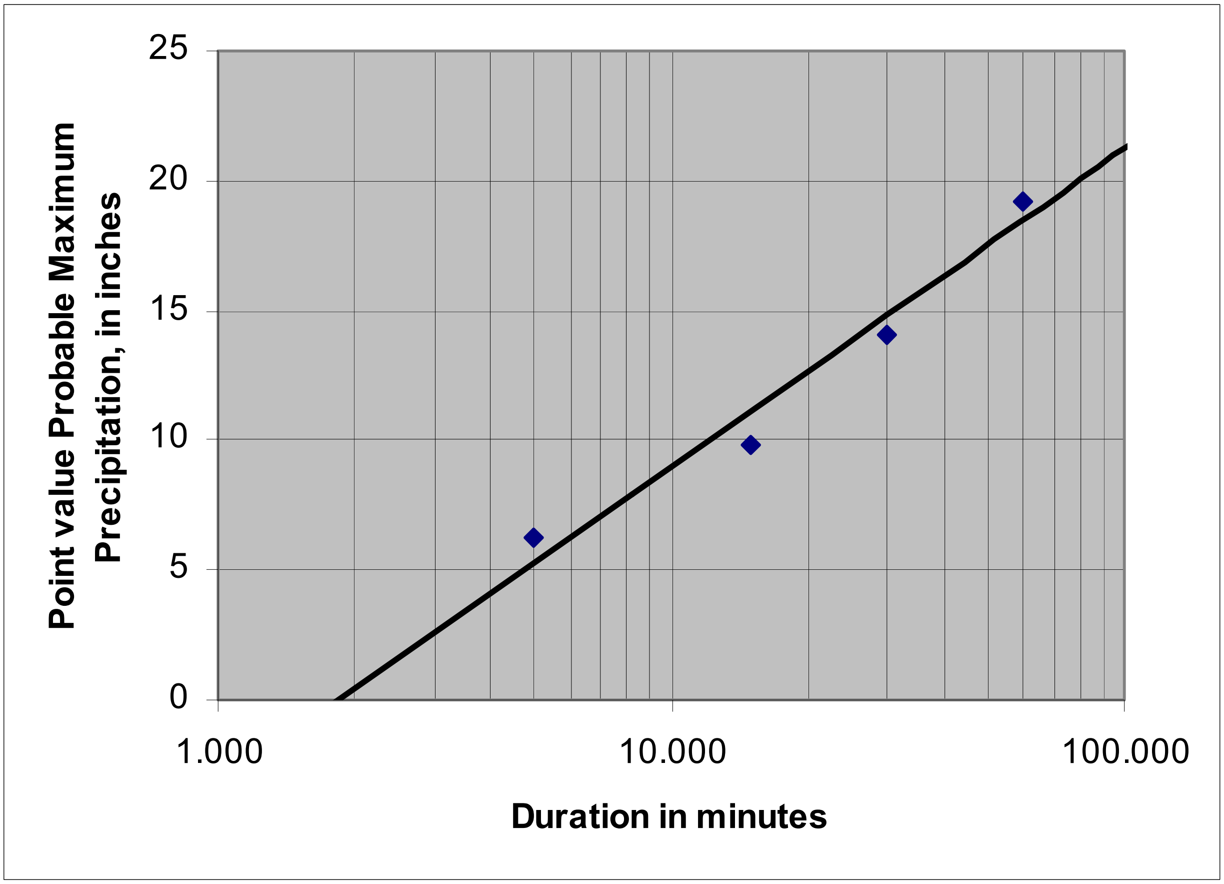


Figure 2.4.2-4 Probable Maximum Precipitation Values as a Function of Duration for Point Rainfall at VEGP Site

Section 2.4.2 References

(ANSI/ANS-2.8-1992) ANSI/ANS-2.8-1992, Determining Design Basis Flooding at Power Reactor Sites, American Nuclear Standards Institute/American Nuclear Society, 1992.

(HMR-51 1978) Hydrometeorological Report No. 51, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, U.S. Department of Commerce, 1978.

(HMR-52 1982) NOAA Hydrometeorological Report No. 52, Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, U.S. Department of Commerce, 1982.

(USACE 1996) Water Control Manual – Savannah River Basin Multiple Purpose Projects: Hartwell Dam & Lake; Richard B. Russell Dam & Lake; J. Strom Thurmond Dam & Lake, Georgia and South Carolina. Savannah District USACE, 1996.

(USGS 1990) Curtis L. Sanders, Jr., Harold E. Kubik, Joseph T. Hoke, Jr., and William H. Kirby, "Flood Frequency of the Savannah River at Augusta, Georgia," US Geological Survey Water Resources Investigations Report 90-4024, Columbia, South Carolina, 1990.

(USGS 2006c) USGS Stream Gage 302197000 Savannah River at Augusta, Georgia. http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02197000 (annual peak data accessed 3-16-2006).

(USGS 2006d) USGS Stream Gage 302197320 Savannah River near Jackson, South Carolina http://nwis.waterdata.usgs.gov/nwis/dvstat/?site_no=02197320 (accessed 1-17-2006).

2.4.3 Probable Maximum Flood

In this section, the hydrometeorological design basis of any necessary flood protection measures is presented for those structures, systems, and components necessary to ensure the capability to shut down the proposed VEGP Units 3 and 4 and maintain them in a safe shutdown condition. One of the scenarios investigated to determine the design basis flood for ensuring the safety of nuclear power plants is the Probable Maximum Flood (PMF). PMF flows and stages at a site can be the result of local flooding, as discussed in Section 2.4.2, or riverine flooding, as described below.

The location of VEGP Units 3 and 4 would be adjacent to and generally to the west of the existing VEGP units, as illustrated in Figure 1-4. The site is located on a high bluff on the west bank of the Savannah River. The proposed site grade for the new units will be at or above Elevation 220 ft msl, similar to the existing VEGP units, which is well above the probable maximum flood stage of the Savannah River.

Based on calculations, site visits, an assessment of site conditions, and a review of previous studies, it was determined that the maximum water surface elevation resulting from the PMF on the Savannah River at the VEGP site and the additional combined action of wind setup and wave run-up would be substantially below El. 220 ft msl.

Considering this assessment, the VEGP site can be characterized as a “flood-dry site,” as described in Section 5.1.3 of the American National Standard Report, *Determining Design Basis Flooding at Power Reactor Sites*, because the safety-related structures of both the existing VEGP and proposed AP1000 units are or will be so high above the Savannah River that safety from flooding is “obvious or can be documented with minimum analysis” **(ANSI/ANS-2.8-1992)**.

A review of studies and analysis performed for the existing units was carried out to confirm that the conclusions continue to be valid for Units 3 and 4. This characterization of the VEGP site is reported in Section 2.4.3.1.

A calculation of the PMF discharge using approximate methods was developed for the ESP application from Regulatory Guide 1.59, *Design Basis Floods for Nuclear Power Plants*, Revision 2, August 1977, reported in Section 2.4.3.2, and the calculation of the associated flood stage using a steady-state hydraulic model and wave run-up, reported in Section 2.4.3.3. These calculations indicate that the maximum flood stage associated with Savannah River flooding is approximately 70 ft below the base slab elevation of the proposed units, confirming the assessment of the VEGP site as “flood dry.”

2.4.3.1 Review of Studies for Units 1 and 2

As part of the hydrologic study carried out for Units 1 and 2, the PMF values for the Savannah River at the site were first estimated using a hydrologic model of the entire upstream watershed and then were checked with a dynamic hydraulic model of the reach of the Savannah River between the last storage reservoir and the VEGP site, as summarized below:

1. The HEC-1 Flood Hydrograph Computer Program, developed by the USACE, was used to develop the PMF hydrograph of the Savannah River near the VEGP site, using the unit hydrographs of the 10 subbasins developed by the National Weather Service (NWS) together with Probable Maximum Precipitation (PMP) estimates derived from methodology outlined in National Weather Service Hydrometeorological Reports (NWS HMR 51 and HMR 52). Valley storage was accounted for by separately modeling the Strom Thurmond Dam HEC-1 outflow hydrograph with the NWS DAMBRK program.
2. The HEC-1 model was independently verified by routing the USACE-derived PMF outflow hydrograph from the Strom Thurmond Dam down to the VEGP site and combining it with the PMF hydrographs from the intervening drainage areas developed from HEC-1.

The results of these previous modeling efforts are summarized in Table 2.4.3-1 and are described in more detail below.

Table 2.4.3-1 Results of Previous PMF Modeling Efforts

Model	Description	PMF and Flood Elevation Results	PMF Stage Including Wave Action	Freeboard wrt El. 220 ft msl
HEC-1 Model with HMR 51 and 52 PMP	Ignoring Valley Storage	895,000 cfs, 136 ft msl	163 ft msl	57 ft
	Valley Storage Modeled in NWS DAMBRK	540,000 cfs, 126 ft msl	153 ft msl	67 ft
USACE PMF with NWS DAMBRK Model		710,000 cfs, 138 ft msl	165 ft msl	55 ft

2.4.3.1.1 Savannah River Watershed Hydrologic Model

In the HEC-1 hydrologic model, the watershed for the Savannah River at the VEGP site was subdivided into 10 subbasins with a total drainage area estimated at that time as 8,015 sq mi (the subwatershed areas used by the NWS for the current flood forecasting model of the Savannah River basin are different from the values used in previous modeling; the updated watershed areas are presented in Table 2.4.1-1 and are used for the PMF approximation described in Section 2.4.3.2). The PMF hydrograph for each subbasin was developed using the unit hydrograph obtained from NWS for the respective subbasins and the corresponding PMP estimates pertaining to the subbasin in question.

Starting from the most upstream subbasin, the PMF hydrograph was then routed and combined in succession in the downstream direction to the VEGP site, including reservoir routing through the upstream Burton, Hartwell, Strom Thurmond, and Stevens Creek dams.

Below Augusta, Georgia, significant floodplain storage exists that could significantly reduce the flood peak. Two PMF values at the VEGP site are presented in the study for licensing Units 1 and 2: a value of 540,000 cfs, with valley storage effects considered, and a value of 895,000 cfs without storage. Without the wind wave activities included, the maximum Savannah River PMF water levels at the VEGP site were estimated to be at El. 126 ft msl and 136 ft msl, respectively, for these two cases.

2.4.3.1.2 Dynamic Hydraulic Model Check on Hydrologic Model Results

An independent check of the reliability of the HEC-1-based estimate of the PMF at the VEGP site was carried out by routing the USACE-derived PMF outflow hydrograph from the Strom Thurmond Dam down to the VEGP site using the NWS dynamic hydraulic model DAMBRK and combining it with the HEC-1-derived PMF hydrographs from the intervening drainage areas between the Strom Thurmond Dam and the site.

The PMF outflow hydrograph at the Strom Thurmond Dam was obtained from the 1962 USACE *Reservoir Regulation Manual* (revised in 1968) developed by the Savannah District before the HMR 51 and 52 PMP guidelines were published and before the closure of the upstream dams.

The PMF peak discharge at the VEGP site was found to be 710,000 cfs, with a corresponding maximum water level at EL 138 ft msl.

It appears that a PMF value of 710,000 cfs was adopted in the study for Units 1 and 2 because it gave a higher water level than the 540,000 cfs value derived from the HEC-1/NWS modeling effort, when valley storage effects were considered.

2.4.3.2 Estimation of PMF by Approximate Methods

An alternative method for estimating the PMF is described in the NRC Regulatory Guide 1.59 for flood dry sites. The method consists of obtaining a relationship for the PMF discharge as a function of drainage area, based on PMF iso-line maps developed for regions of the United States east of the 105th Meridian, and utilizing the drainage area at a given site, obtain the PMF from the relation determined for that region. No PMP is required for this method. Calculations for the estimated PMF at the VEGP site are presented below.

The PMF values determined from the 100-, 500-, 1,000-, 5,000-, 10,000-, and 20,000 sq mi contributing area maps at the location of the Savannah River watershed upstream of the VEGP site are tabulated in Table 2.4.3-2.

Table 2.4.3-2 PMF Values for an Area-PMF Relationship at the VEGP Site

Watershed Area, sq. mi.	PMF in cfs from isolines	Supporting Figure (RG 1.59)
100	110,000	B-2
500	250,000	B-3
1,000	330,000	B-4
5,000	750,000	B-5
10,000	1,050,000	B-6
20,000	1,300,000	B-7

A logarithmic plot of the power curve fit to these values is presented in Figure 2.4.3-1. Based on the curve fit to the data and the currently estimated drainage area of 8,304 sq mi (as discussed in Section 2.4.1), the estimated PMF for the VEGP site is about 920,000 cfs. This point is located on the curve in Figure 2.4.3-1, along with a data point for VEGP (reported as Alvin W. Vogtle), presented on page 4 of 17 in Table B.1 of RG 1.59 as 1,001,000 cfs for a drainage area of 6,144 sq mi. Considering current and previously reported measurements, the drainage area reported for the VEGP site in Table B.1 appears to be incorrect and inconsistent with the RG 1.59 method, which was used to derive the value. However, it is presented as a published reference value.

2.4.3.3 Estimation of Flood Stage at VEGP Site for PMF

A stage-discharge relationship or "rating curve" is required to estimate the water surface elevation of the Savannah River near the VEGP site associated with the PMF discharge. This relationship was obtained from a steady-state hydraulic backwater analysis of the Savannah River run in HEC-RAS, a computer model developed by the USACE **(USACE 2005).**

The steady-state model was adapted from the dynamic model used for the analysis of the dam-break scenario described in Section 2.4.4, using the same channel roughness (Manning's n) values as in that model. All bridges were removed from the dynamic model; they were not put back into the steady-state model, which is equivalent to assuming that any downstream bridges are either swept away or have a negligible impact on water surface elevations at the VEGP site during the PMF event.

Changes in the HEC-RAS model used to estimate stages at the VEGP site included:

- The reaches of the model upstream of the Augusta City Dam (River Mile 199.667) were removed.

- The model was converted from dynamic to steady-state mode with the downstream boundary condition at River Mile 99.406 determined by normal depth using an estimated energy slope of 0.0005 (the downstream water surface elevation will have a negligible impact on water surface elevations some 90 mi upstream near the VEGP site).
- The PMF and reference discharges were input for the entire model reach.
- The cross-section nearest the VEGP site (River Mile 150.906) was extended to the proposed top-of-slab elevation using 1:24,000-scale topography from 7.5-minute USGS quadrangles **(USGS MAPS 1989)**

The results for the cross-section nearest to the VEGP site (River Mile 150.906 in the model) are shown in Table 2.4.3-3.

Table 2.4.3-3 PMF Flood Stages for Cross-Section Nearest VEGP Site

Profile	Q Total, cfs	W.S. Elev, ft	E.G. Elev, ft	E.G. Slope	Vel Chnl, fps	Flow Area, sf	Top Width, ft	Froude # Chl
Avg Daily Max	13,669	88.22	88.25	0.000056	1.50	31,765	8,238	0.07
Avg Annual Peak	28,734	92.37	92.39	0.000056	1.64	66,743	8,551	0.07
Historic Max	360,000	118.55	118.63	0.000093	4.12	384,032	14,534	0.11
PMF	917,965	138.82	138.95	0.000102	5.66	680,627	14,681	0.13
2 x PMF	1,835,930	160.50	160.71	0.000120	7.50	999,754	14,784	0.14

The longitudinal profile output for the Savannah River for this model is reproduced as Figure 2.4.3-2. The cross section developed for the VEGP site is shown in Figure 2.4.3-3.

The estimated maximum stages at the VEGP site for the PMF estimated per the approximate method outlined in RG 1.59 are shown in Table 2.4.3-4.

Table 2.4.3-4 Estimated Probable Maximum Flood Stage at VEGP Site

PMF Stage:	138.82 ft msl –HEC-RAS WSL at River Mile 150.906
Wave run-up & wind set-up	11.31 ft – result for 2h:1v slope w/ 50 mph wind from NE over an 11-mile fetch resulting from dam-break
Total PMF Stage:	150.13 ft msl
Minimum Slab elevation	220.00 ft msl
Estimated Freeboard	69.87 feet

Based on the fact that the estimated maximum stage reached by the Savannah River at the site for the approximate PMF flood is over 69 feet below the minimum top-of-slab elevation of any safety-related systems, structures, or components at the VEGP site, the characterization of a flood-dry site should be established.

2.4.3.4 Conclusions

The PMF discharge on the Savannah River at the VEGP site estimated using the approximate methodology recommended for flood-dry sites is approximately 920,000 cfs, which corresponds to an approximate flood stage of about El. 139 ft msl. Accounting for wave run-up and wind set-up, the probable maximum water surface elevation on the Savannah River at the VEGP site would be less than elevation 151 ft msl.

The peak flood discharge associated with the dam-break analysis presented in Section 2.4.4 is about 2,332,000 cfs – significantly higher than the estimated PMF, which is consistent with the very significant volume of storage in the reservoirs upstream of the site. The maximum water surface elevation of the Savannah River at the VEGP site associated with the dam-break scenario is El. 166.79 ft msl at a discharge of 2,233,000 cfs (occurring several hours after the wave front associated with peak discharge, at which time the water surface is lower). Including 11.31 feet of wave run-up and wind set-up, the estimated maximum water stage at the VEGP site is El. 178.1 ft msl, significantly higher than the stage resulting from the PMF event with no dam failure.

In either case, the probable maximum flood stage is so far below the proposed grade elevation for the new units that the site can be classified as flood dry without reservation, and it can be concluded that the site is not susceptible to flooding from the Savannah River.

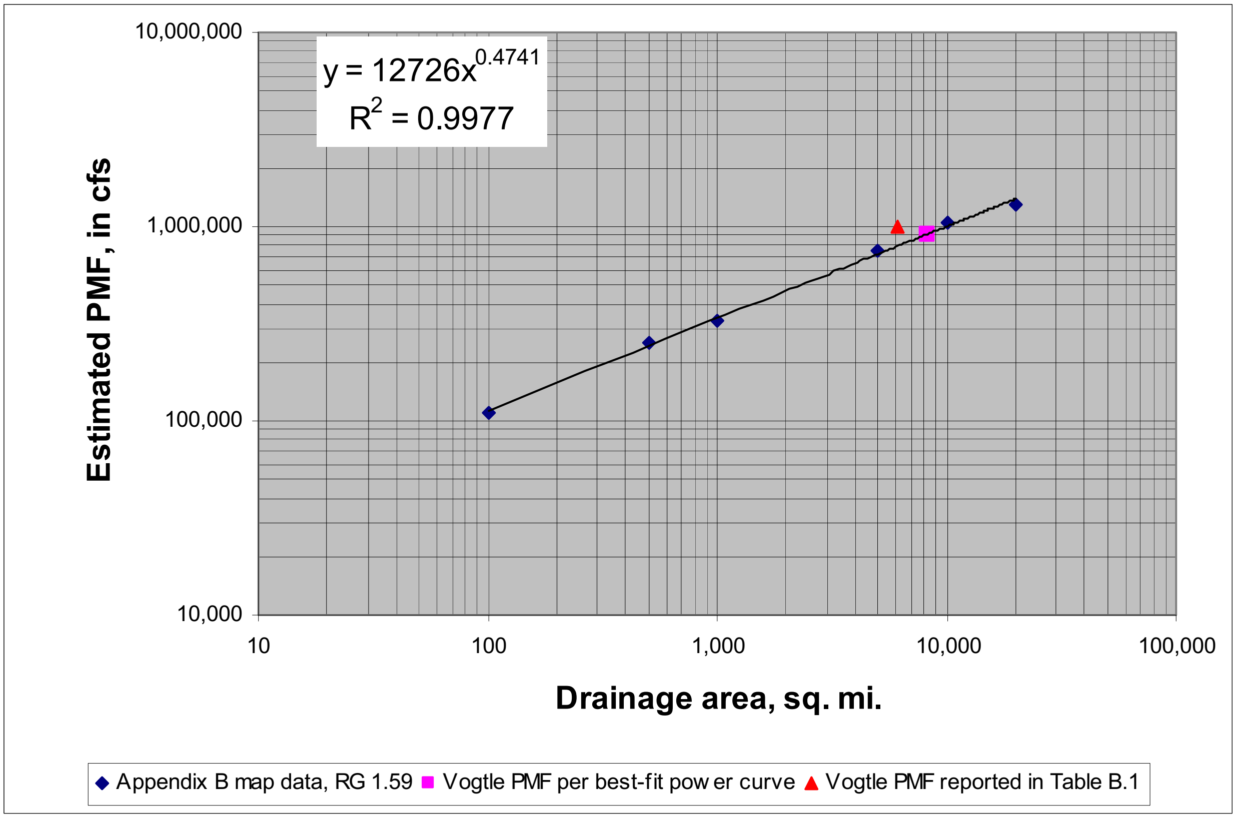


Figure 2.4.3-1 Area-PMF Plot for VEGP Site per Approximate Method from RG 1.59)

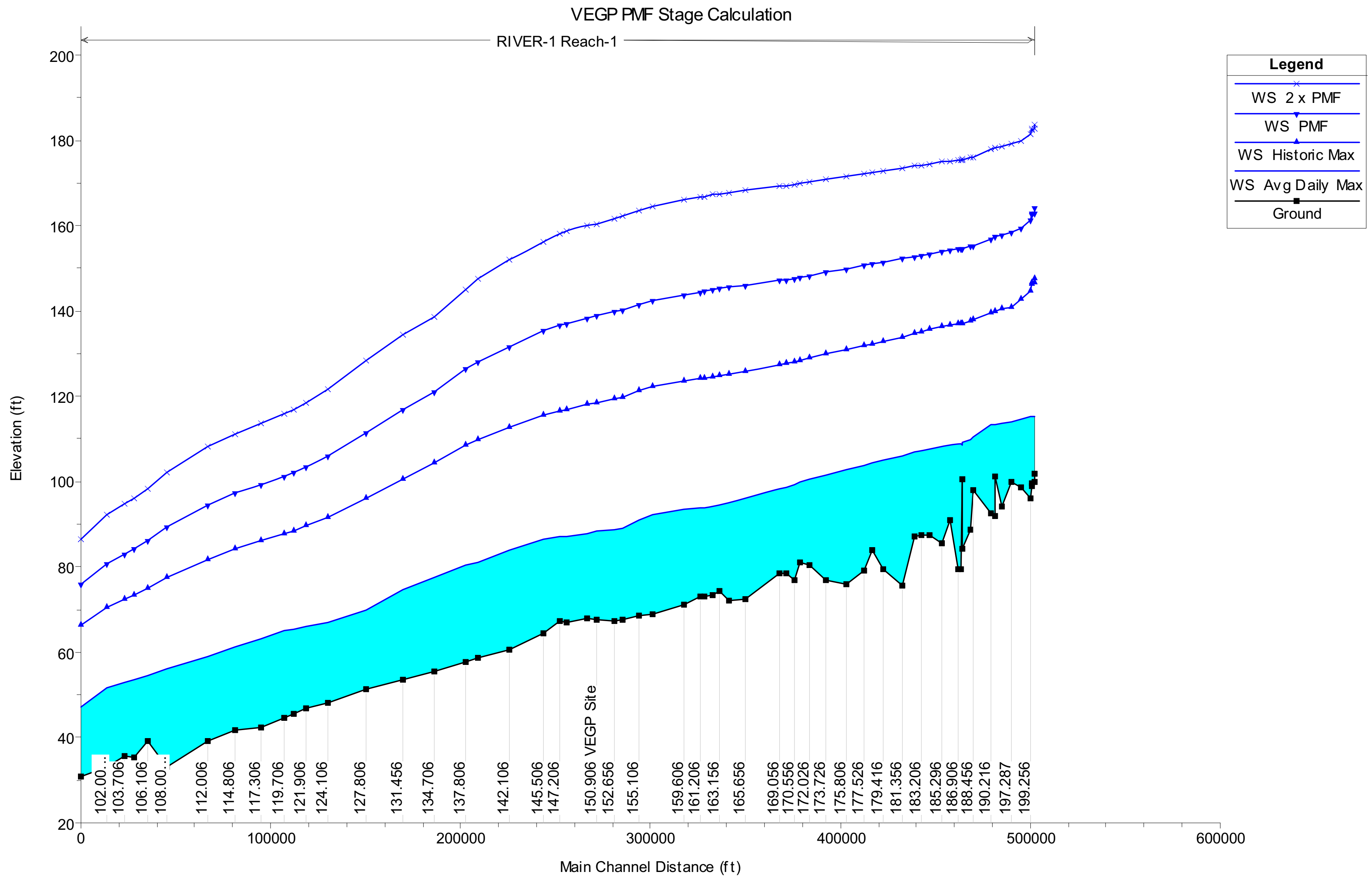


Figure 2.4.3-2 Longitudinal Profiles of the Savannah River from Steady-State HEC-RAS Model Run

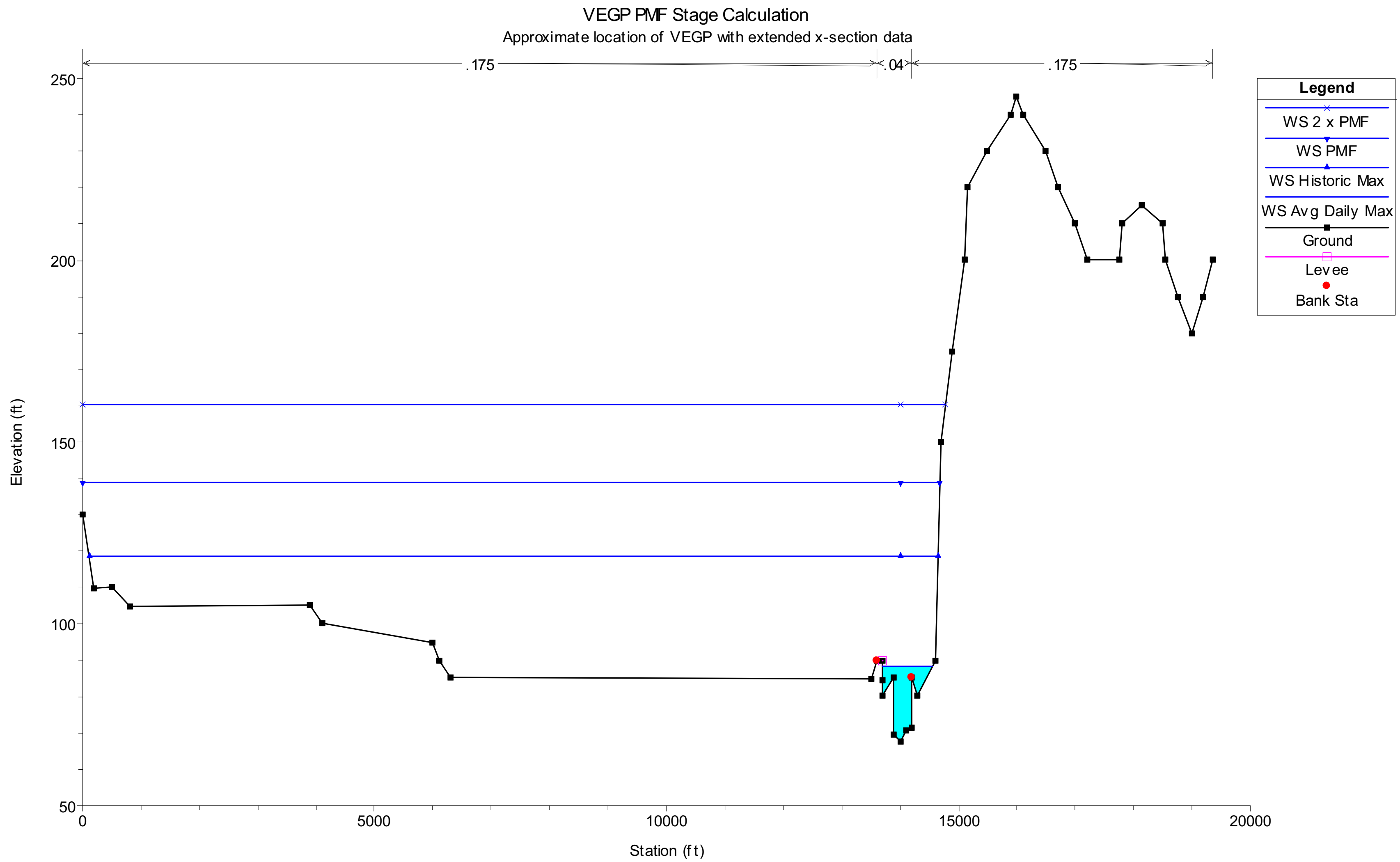


Figure 2.4.3-3 HEC-RAS Model Section at VEGP Site (Looking Downstream)

Section 2.4.3 References

(ANSI/ANS-2.8-1992) ANSI/ANS-2.8-1992, *Determining Design Basis Flooding at Power Reactor Sites*, American National Standards Institute/American Nuclear Society, 1992

(USACE 2005) HEC-RAS, River Analysis System, Version 3.1.3, Computer Program, Hydrologic Engineering Center, US Army Corps of Engineers, May 2005.

(USGS MAPS 1989) 7.5 Minute Series, Topographic Maps, US Geological Survey, Shell Bluff Landing, GA, 1989, Girard NW, GA-SC, 1989.

2.4.4 Potential Dam Failures

The VEGP site is located on the west bank of the Savannah River about 50 River Miles downstream of the City of Augusta, Georgia. There are 14 dams in the Savannah River Basin upstream of the VEGP site. These dams are owned and operated by either the U.S. Army Corps of Engineers (USACE) or one of several electric power generation companies located in Georgia and South Carolina. Table 2.4.1-9 lists the dams, their owners, and other pertinent data. The dams owned and operated by electric power generators fall under the jurisdiction of the Federal Energy Regulatory Commission (FERC); the other dams fall under the jurisdiction of the USACE.

Both FERC and USACE regulations require that dams for which failures pose a risk to human life be designed to survive very large earthquakes without risk of failure. Thus, it is unlikely that failure of any of the upstream dams would occur during a Safe Shutdown Earthquake (SSE). However, to demonstrate that the VEGP site will not be subject to flooding due to potential dam failures, a domino-type failure of the upstream dams is assumed, and this section analyzes the resulting flood wave and corresponding flood elevations at the VEGP site.

2.4.4.1 Dam Failure Permutations

Figure 2.4.4-1 shows the locations of the Savannah River Basin dams. Two of these dams, Stevens Creek Dam and New Savannah Bluff Lock and Dam, are relatively small weir structures used for flow diversion and small hydropower generation and do not have significant storage volumes. Both of these dams are located downstream of J. Strom Thurmond (also known as Clark's Hill) Dam and would be completely inundated by a breach of the upstream dams. Therefore, they are not included in the dam breach analysis presented in this subsection.

Table 2.4.1-9 lists each dam, its location, and size. Note that Little River Lake and Dam and Keowee Lake and Dam are hydraulically connected and share a common reservoir. All discharge from the common reservoir is through the Keowee Dam. Little River Dam has no outlet works.

Three large hydroelectric and storage dams on the Savannah River are operated by the USACE. They are J. Strom Thurmond Lake and Dam, Richard B. Russell Lake and Dam, and Hartwell Lake and Dam. Each dam comprises an earth embankment with a concrete gravity section in the center where the hydroelectric generation facilities and spillway gates are located. Upstream of Hartwell Dam, the remaining dams are located on tributaries to the Savannah River. Keowee/Little River Dam and Jocassee Dam are located on the Keowee River. Yonah Dam and Tugaloo Dam are located on the Tugaloo River. Tallulah Falls Dam, Mathis Dam, Nacoochee Dam, and Burton Dam are located on the Tallulah River, which is a tributary to the Tugaloo River.

For the dam breach analysis, conservatism of coincident flow rates in the Savannah River and water levels in the dams are assumed. The dam failure is assumed to be coincident with the standard project flood (SPF) water levels in the reservoirs behind the dams and the USACE-defined SPF discharge in the Savannah River.

Upstream of Thurmond Dam, there are essentially no free-flowing reaches of the Savannah River or the Keowee River. Each dam discharges into the reservoir pool of the next downstream dam. The failure mode that produces the largest flood wave and flood elevations at the VEGP site would produce the highest water level and largest volume of water at Thurmond Dam (the dam closest to the site) just before the assumed breach of Thurmond Dam. Based on the configuration of the dams upstream of Thurmond Dam, two breach scenarios are possible.

The first scenario consists of breaching all dams simultaneously. In this scenario, the water level at Thurmond Dam would be the SPF flood level in the lake, El. 342.1 ft msl **(USACE 1996)**. Initially, the stored water behind the reservoir would be the storage volume associated with the SPF water level. The inflow into Thurmond Lake would be equal to the flow through the breach at Russell Dam, which would be based on the SPF water level at Russell Dam, and so on upstream for all dams.

The second scenario consists of initially breaching only the most upstream dam in one of the stream reaches upstream of Hartwell Dam and allowing it to fill the next downstream reservoir, overtopping the downstream dam and breaching it. This scenario would continue breaching dams downstream by overtopping until Thurmond Dam is breached. In this scenario, when the breach occurs at Thurmond Dam, the water level would be at the top of the dam, El. 351.0 ft msl **(USACE 1996)**. Since the water level would be higher than the SPF level, the storage volume would also be larger. Additionally, the flow from Russell Dam into Thurmond Lake would have already started before Thurmond Dam was breached and would also be based on a higher water level in Russell Lake, resulting in a larger discharge into Thurmond Lake. Thus, with higher water levels and larger storage volumes and with the discharges from the upstream breaches already established before Thurmond Dam is breached, the second alternative would produce the higher flood wave downstream.

In the second scenario, there are two possible failure modes. The first mode (Mode 1) consists of Jocassee Dam breaching and progressing downstream through Keowee Dam to Lake Hartwell. The second mode (Mode 2) consists of Burton Dam breaching and progressing downstream through Nacoochee Dam, Mathis Dam, Tallulah Falls Dam, Tugaloo Dam, and Yonah Dam to Lake Hartwell. By comparing the normal pool storage volumes for the upstream dams listed in Table 2.4.1-9, the most severe failure mode is estimated. The combined normal pool storage volumes behind the dams in each mode are shown in Table 2.4.4-1.

Table 2.4.4-1 Normal Pool Storage Volumes

Dam	Mode 1 Reservoir Storage Volume (1000 ac-ft)	Mode 2 Reservoir Storage Volume (1,000 ac-ft)
Jocassee	1,100	
Keowee	940	
Burton		108
Nacoochee		8.2
Mathis		31.4
Tallulah Falls		2.46
Tugalo		43.2
Yonah		10.2
Total	2,040	203

Table 2.4.4-1 indicates that the normal pool storage volume in Mode 1 is 10 times the volume in Mode 2. Thus, an assumed dam failure scenario following Mode 1 with the Jocassee Dam failing is analyzed.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The dam breach option of the USACE River Analysis System computer program (HEC-RAS) **(USACE 2005a)** was used to develop the dam breach flood wave. The unsteady flow option of HEC-RAS was then used to route the flood wave downstream to the VEGP site. Multiple dams were breached in the analysis to determine the maximum flood elevation at the site. Although HEC-RAS is capable of routing several dam breaches in succession, this analysis used an alternative modeling approach for simplicity and conservatism. In this analysis, only two dams (Russell Dam and Thurmond Dam) were breached in succession. The storage volume behind the upstream dam (Russell Dam) was assumed to be equivalent to the SPF storage volume of all the upstream reservoirs (Lake Jocassee, Lake Keowee, Hartwell Lake, and Lake Russell). This approach conservatively models the successive failure of the three upstream dams and the simultaneous arrival of their combined storage volumes at Russell Dam. Russell Dam is breached by overtopping, which then causes the overtopping breach of Thurmond Dam and a subsequent flood wave down the Savannah River.

The *Savannah River Basin Water Control Manual* **(USACE 1996)** contains the SPF water levels, SPF discharges, and storage volumes from the Thurmond, Russell, and Hartwell dams, as well as storage data for the Jocassee and Keowee dams. Jocassee and Keowee dam SPF peak discharges and water levels are not available. However, probable maximum flood (PMF) water levels and discharges, which are greater than SPF values, are available and were used instead of the SPF values in the analysis. The PMF water levels and peak discharges for these two dams were obtained from LBC&W Associates of South Carolina **(LBC&W 1972)**. Area-capacity curves for each of the five reservoirs are shown on Figures 2.4.4-2 through 2.4.4-6, respectively.

For the purposes of this analysis, the Russell and Thurmond dams were assumed to fail by overtopping. The HEC-RAS computer program dam breach option requires the input of several breach parameters. These include the final bottom width (B) and the bottom elevation of the breach along with the side slopes (Z) of the breach. The time (tf) to reach the final breach dimensions is also required input. Several methodologies are available to estimate these parameters. The Bureau of Reclamation has summarized many of these methodologies in a single document, Prediction of Embankment Dam Breach Parameters **(USBR 1998)**. These methodologies give various results. The breach parameters for the Thurmond and Russell Dams are estimated using many of the procedures described in USBR 1998 and the results compared.

The formulas used for each of the breach parameter estimation methods are shown in Table 2.4.4-2. The input and output variables for each of these formulas are meters, cubic meters, and hours. Several variables for each of these methods are required. The required variables are listed below:

h_w = Depth of water at dam at failure, above the breach bottom (m)

h_b = Height of breach (m)

h_d = Height of dam (m)

S = Storage volume at breach elevation (m^3)

S* = Dimensionless storage (S/h_b^3)

W_c = Width of dam crest (m)

W_b = Width of dam bottom (m)

W* = Dimensionless average dam width ($(W_c + W_d)/2h_b$)

V_{er} = Volume of material eroded, estimated by ($0.0261(S^*h_w)^{0.769}$) (m^3)

K_o = Overtopping correction factor (1.4 if failure mode is overtopping)

K_c = Core wall correction factor (0.6 if dam has a core wall, 1.0 if not)

The breach for each dam will consist of an overtopping breach. The breach depth for each dam is also assumed to reach to the upstream reservoir invert. This is a conservative assumption for both the Russell and Thurmond Dams because the majority of the portions of each dam that reach the upstream inverts are the portions constructed of concrete where the tainter gate spillways and hydroelectric turbines are located. In order for the earth sections to breach to the invert depths for the widths calculated in the following discussion, native material will have to be eroded. However, for the purpose of this analysis, it will be assumed that the embankment and native material will erode to the upstream invert elevation.

The input variables along with the estimated breach parameters, by the various methodologies, for each dam are shown in Tables 2.4.4-3 through 2.4.4-6.

For the Thurmond Dam, the FERC (1987) equation from Table 2.4.4-2, as well as other sources in the literature, indicates that the breach width should be 2 to 5 times the height of the dam. This guidance is confirmed by the USBR report **(USBR 1998)**, which shows the 84 data points for observed breach widths used in their analysis of dam breach parameters. The Froehlich (1995b) relationships in the Table 2.4.4-2 were developed using a regression analysis of the data, which is biased by the fact that the majority of the data points are for breach widths less than 50 m (164 ft). In fact, the USBR report **(USBR 1998)** states that the Froehlich relationships are apparently the best fit for cases with observed breach widths less than 50 m (164 ft). Extrapolation of the Froehlich relations to the anticipated breach width on the order of 5 times the height of the dam (230 m [755 ft]) indicates that the Froehlich relations are not in agreement with the observed data for breach widths greater than 150 m (492 ft). Because all of the other methods shown in Table 2.4.4-4 are of the same order of magnitude, and are also within the range of accepted engineering practice for FERC-mandated dambreak analyses, a breach width of 755 ft was selected for this study. The value of 755 ft also is the maximum of the values obtained by all other methods, and is therefore conservative. The following considerations of the dam layout and river cross-section at the dam show that the use of a 755-ft breach width is also conservative in light of the physical layout of J. Strom Thurmond Dam and appurtenances:

- The HEC-RAS dam breach model and the equations used to determine discharges from the breach assume a “flat” bottom breach with a constant elevation. This means that bottom elevation of the entire 755-ft breach width is assumed to be at El. 200 ft msl, which is the minimum elevation of the original streambed on the upstream side of J. Strom Thurmond Dam.
- As shown on Figure 2.4.4-7, the total dam width at the top of the dam is about 5,700 ft **(USACE 1996)**. The width of the dam at the upstream invert elevation (El. 200 ft msl) is about 2,840 ft. Located within the portion of the dam that extends to El. 200 ft is a concrete embankment section 2,282 ft wide where the tainter gate spillways and powerhouse are located **(USACE 1996)**. The failure mode assumes that only the earth section of the dam will erode during the breach. Consequently, the 755-ft bottom width of the breach extends beyond the area in which the actual ground elevation is at the minimum ground elevation of El. 200 ft msl.
- Superposing the 755-ft bottom width at El. 200 ft msl on the cross-section of the valley on the upstream side of the dam shows that more than 200 ft of the breach would be above El. 200 ft msl. Therefore, the entire bottom of the breach was taken as El. 200 ft msl to be conservative. The cross section shown in Figure 2.4.4-7 has been artificially widened at El. 200 ft msl to accommodate the 755-ft-wide breach.

Based on a review of data and analyses for 84 dam failure cases, and the physical layout of J. Strom Thurmond Dam, a breach width of 755 ft, with 2 to 1 side slopes was selected for this analysis. Additionally, most of the breach time predictions are close to 1.0 hour. Thus, a breach time of 1.0 hour was selected for this analysis.

The breach width for the Richard B. Russell dam is also much larger than 50 m and thus, the Froehlich equations predict values much greater than the observed data. Since all of the other methods shown in Table 2.4.4-6 are of the same order of magnitude, and are also within the range of accepted engineering practice for FERC-mandated dambreak analyses, a breach width of 750 ft was selected for this study. The value of 750 ft also is the maximum of the values obtained by all other methods, and is therefore conservative. The following considerations of the dam layout and river cross-section at the dam show that the use of a 750-ft breach width is also conservative in light of the physical layout of Richard B. Russell Dam and appurtenances:

- The HEC-RAS dam breach model and the equations used to determine discharges from the breach assume a “flat” bottom breach with a constant elevation. This means that the bottom elevation of the entire 750-ft breach width is assumed to be at El. 345 ft msl, which is the minimum elevation of the original streambed on the upstream side of Richard B. Russell Dam.
- As shown on Figure 2.4.4-8, the total dam width at the top of the dam is about 4,500 ft. **(USACE 1996)**. The width of the dam at the upstream invert elevation (El 345 ft msl) is about 2,200 ft. Located within the portion of the dam that extends to El. 345 ft msl is a concrete embankment section 2,180 ft wide where the tainter gate spillways and powerhouse are located **(USACE 1996)**. Only 1,000 ft of the concrete section extends to El. 345 ft msl, the remaining portion extends up the embankment. The failure mode assumes that only the earth section of the dam will erode during the breach. Consequently, the 750-ft bottom width of the breach extends beyond the area in which the actual ground elevation is at the minimum ground elevation of El. 345 ft msl.
- Superposing the 750-ft bottom width at El. 345 ft msl on the cross-section of the valley on the upstream side of the dam shows that more than 150 ft of the breach would be above El. 345 ft msl. Therefore, the entire bottom of the breach was taken as El. 345 ft msl to be conservative. The cross section shown in Figure 2.4.4-8 has been artificially widened at El. 345 ft mls to accommodate the 750-ft-wide breach.

Based on a review of data and analyses for 84 dam failure cases, and the physical layout of Richard B. Russell Dam, a breach width of 750 ft, with 2 to 1 side slopes was selected for this analysis. Additionally, most of the breach time predictions are close to 1.0 hour. Thus, a breach time of 1.0 hour was selected for this analysis.

Once the dam breach occurred, the HEC-RAS computer program determined the flood wave discharge from the dam based on the breach dimensions, water level in the reservoir behind the dam, and the water level downstream of the dam. The program then used an unsteady flow option to model the progression of the flood wave downstream to the VEGP site. Additionally, HEC-RAS continued to model the flows through the dam breaches until the stored water in the reservoirs was evacuated. Since the combined volume of all five reservoirs is more than 10 million acre-feet, the flood wave from the dam breaches would last for several days at the VEGP site.

Cross-section data for the Savannah River used in the HEC-RAS computer model were obtained directly from the USACE, Savannah District **(USACE 2002)**. The data were supplied in HEC-RAS format and assembled from various floodplain studies on the Savannah River. To ensure that the cross-section data were accurate, several representative cross-sections near the site, in the City of Augusta, and near Thurmond Dam, were compared with cross-sections developed independently from USGS topographic maps **(USGS 1984–2000)**. In each instance, the cross-section data supplied provided a good match with those developed from USGS topographic maps.

The USACE elevation data for most of the cross-sections did not extend to the computed water surface elevation for the dam breach analysis. Therefore, HEC-RAS extended the left-most and right-most cross-section elevations vertically to meet the computed water surface. Usually, this approach is conservative in that it produces a cross-sectional area less than the actual cross-section. However, downstream of the breached dam, a constricted cross-section could produce water levels high enough to restrict the flow from the breach due to tail water submergence. Thus, four cross-sections downstream of the dam were sufficiently extended horizontally, based on USGS topographic information, to cover the range of the computed water levels.

A sensitivity analysis was performed to assess the effect of extending the remaining cross-sections to higher elevations. The results of this analysis indicated that extending the cross-sections lowered the water level and peak discharge at the VEGP site by less than 0.5 ft. Thus, for the most part, these cross-sections were not modified. However, the cross-section data through the City of Augusta extend only to the top of the levee on the right (west) bank of the Savannah River. Flood elevations for the dam breach event would overtop the levee and extend out into the City of Augusta. Thus, cross-section data through the City of Augusta were extended horizontally using topographic maps **(USGS 1984–2000)** to include additional area to these cross-sections and account for overtopping of the levee.

At least two sets of River Mile stationing have appeared in different USACE publications for the Savannah River. There is an approximately 16-mi discrepancy between the two stationing sets. The River Mile stationing set used in this analysis matches the stationing set used in the VEGP UFSAR and most of the *Savannah River Basin Water Control Manual* **(USACE 1996)**. The VEGP site is located at River Mile 150.9 in the HEC-RAS model. The other River Mile stationing reference would have the site at approximately River Mile 167.

Several bridges cross the Savannah River downstream of Thurmond Dam and through the City of Augusta. The last of these bridges is about 40 river miles upstream of the VEGP site. Modeling the dam breach flood wave through the City of Augusta with the bridges intact would produce results that impede the travel of the flood wave and reduce the computed flood levels at the VEGP site. However, during a dam breach event, all bridges would be significantly overtopped and it is likely that most, if not all, would be washed out. Thus, to provide more reasonable results, which allow the flood wave to progress unimpeded downstream (a conservative assumption for modeling the flood elevations at the VEGP site), the bridge structures were removed from the HEC-RAS model.

The Savannah River cross-section data supplied by the USACE stopped just downstream of Thurmond Dam. Cross-sections upstream and downstream of Thurmond and Russell dams were obtained from USGS topographic maps **(USGS 1984–2000)**. The below-water portions of the cross-section data were obtained from fishing maps with depth contours **(FHS L649; FHS L650)**.

Roughness coefficients (Manning's n) were estimated using procedures developed by the US Geological Survey **(USGS 1989)**. Additionally, roughness coefficients were estimated for the flood studies performed for the existing VEGP Units 1 and 2 by calibrating water surface profile models with known flood elevations. The USGS estimation procedures produce roughness coefficients that are higher, and more conservative, than those presented in the UFSAR. Thus, the USGS-estimated roughness coefficients were used in the HEC-RAS dam breach model. The use of higher roughness coefficients is consistent with observations of dam-break floods that show that roughness coefficients for exceptionally high flow depths associated with dam-break floods are higher than those associated with lower flood flows in a river.

The starting water levels at three locations were required in the HEC-RAS dam breach model—in each of the two reservoirs and at the downstream end of the model. The cross-section farthest downstream in the HEC-RAS model is located at the River Mile 99.41, 51.5 mi downstream of the VEGP site. The normal depth option in HEC-RAS was used to determine the starting water surface elevation at this location. Given the distance from the site, any changes in the downstream boundary condition water level will not affect the computed flood elevations at the VEGP site.

The starting water level in Thurmond Lake was set at the SPF water level (i.e., El. 344.7 ft msl). Additionally, at this point an initial inflow was added equal to the SPF discharge of 560,000 cfs from Thurmond Dam. Once Russell Dam breaches, the overtopping breach of Thurmond Dam is triggered when the water level reaches El. 351.1 ft msl, 0.1 ft above the top of the dam (**USACE 1996)**, due to inflows from the breach of Russell dam.

The starting water level at Russell Dam was treated slightly differently. The model was set up as if the breaches of the Jocasse, Keowee, and Hartwell dams have already occurred and the combined SPF storage volume from these reservoirs is already at Russell Dam. Any upstream breaches would have already raised the water level to the top of Russell Dam. Therefore, the starting water level at Russell Dam was set at the top of the dam at El. 495.0 ft msl **(USACE 1996)**. The overtopping breach of Russell Dam was triggered 2 hours after the start of the HEC-RAS simulation. This 2-hour time delay allowed the SPF flood flow in the Savannah River downstream of Thurmond Dam to stabilize in the HEC-RAS model prior to initiating the Russell Dam breach.

2.4.4.3 Water Level at the Plant Site

The results of the HEC-RAS dam breach and unsteady flow routing analysis indicate that the peak water level at the VEGP site due to dam failure is El. 166.79 ft msl, which is 53.21 ft below the proposed site grade at El. 220.0 ft msl. The computed discharge at the time of the peak water level is 2,232,605 cfs.

The computed peak discharge rate, however, occurs 5 hours before the peak water level. The peak discharge is 2,331,582 cfs, with a corresponding water level at El. 164.71 ft msl. The delay in the peak water level at the site is due to backwater effects caused by the peak flood wave moving downstream of the site. The results are quoted to more significant figures than is physically possible to measure so that, if necessary, a direct correlation between the numerical results presented here and the computer output in supporting calculations can be obtained easily.

A plot of the Savannah River discharge and stage hydrograph at the VEGP site location is shown in Figure 2.4.4-9. Plots of the SPF water surface profile, maximum water surface profile, and water surface profile at the time of the maximum water level at the VEGP site are shown on Figures 2.4.4-10 through 2.4.4-12, respectively.

The flood elevations determined for this section have been determined to demonstrate that a postulated dam-break flood wave cannot adversely impact the VEGP site. The analysis to determine these elevations is based on very conservative assumptions, and the computed flood elevations should not be used for any other purposes or locations.

In accordance with ANSI/ANS-2.8 (1992), the maximum wave height and wave run-up at the shoreline generated by a 2-year wind speed must be estimated in conjunction with the dam breach flood level at the site. The fastest mile 2-year wind speed at the site is 50 mph **(ANSI/ANS-2.8 1992)**. The *Coastal Engineering Manual* **(USACE 2005b)** is used to estimate the wave height and run-up elevations at the VEGP site. The procedures outlined in the *Coastal Engineering Manual* use the wind speed, wind speed duration, water depth, and over-water fetch length to determine wave heights and run-up. The maximum fetch length during the dam breach flood is from the northeast and is about 11.14 miles long. The maximum fetch length is shown on Figure 2.4.4-13.

Various wind speed durations were analyzed to determine the maximum wave height and run-up elevation at the site. The wave run-up was determined based on the steep embankment condition that will exist during a dam breach flood event at the VEGP site. The estimated slope of the embankment is 2H:1V for the wave run-up determination.

The estimated wave height and run-up values at the VEGP site during the dam breach flooding event are as follows:

- Maximum Wave Height, HMAX = 7.46 ft
- Spectral Peak Period, TP(MAX) = 4.09 s
- Maximum Wave Length, L0 = 85.73 ft
- Maximum Wave Run-up, R = 11.31 ft

The calculated wave run-up also includes wave setup effects. To obtain the maximum flood elevation due to wind-induced waves at the VEGP site, the maximum wave run-up elevation was added to the still water elevation due to dam breach flooding. Adding these two numbers gives a maximum flood level of El. 178.10 ft msl, which is 41.9 ft below the proposed site grade of El. 220.0 ft msl. Therefore, the VEGP site is precluded from flooding due to potential dam failures and coincident wind-generated waves.

Table 2.4.4-2 Breach Parameter Estimation Formulas

Reference	Number of Case Studies	Relations Proposed (S.I. units, meters, m^3/s, hours)
Johnson & Illes (1976)		$0.5h_d \leq B \leq 3h_d$ for earthfill dams
Singh & Snorrason (1982, 1984)	20	$2h_d \leq B \leq 5h_d$ 0.15 m ≤ d_{ovtop} 0.61 m 0.25 hr ≤ t_f ≤ 1.0 hr
MacDonald & Langridge-Monopolis (1984)	42	<u>Earthfill dams:</u> $V_{er} = 0.0261\ (V_{out} * h_w)^{0.769}$ [best-fit] $T_f = 0.0179(V_{er})^{0.364}$ [upper envelope] <u>Non-earthfill dams:</u> $V_{er} = 0.00348(V_{out} * h_w)^{0.852}$ [best-fit]
FERC (1987)		B is normally 2-4 times h_d B can range from 1-5 times h_d Z = 0.25 – 1.0 [engineered, compacted dams] Z = 1 – 2 [non-engineered, slag or refuse dams] t_f = 0.1 - 1 hr [engineered, compacted earth dams] t_f = 0.1 - 0.5 hr [non-engineered, poorly compacted earth dams]
Froehlich (1987)	43	$\overline{B}^* = 0.47K_0\ (S^*)^{0.25}$ K_0 = 1.4 overtopping; 1.0 otherwise $Z = 0.75\ K_c(h_w^*)^{1.57}(\overline{W}^*)^{0.73}$ K_c = 0.6 with corewall; 1.0 without corewall $t_f^* = 79(S^*)^{0.47}$
Reclamation (1988)	52	$B = 3h_w$ $t_f = 0.011B$
Von Thun & Gillette (1990)	57	B, Z, t_f see guidance in USBR 1998
Froehlich (1995b)	63	$\overline{B} = 0.1803K_0V_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{(-0.90)}$ K_0 = 1.4 for overtopping; 1.0 otherwise

Source: USBR 1998

Table 2.4.4-3 J. Strom Thurmond Dam Input Variables

Input Variable	English Units		SI Units	
h_w	151.1	ft	46.1	m
h_b	151.0	ft	46.0	m
h_d	151.0	ft	46.0	m
S	4360000	ac-ft	5378009947	m^3
S*			55162.75	
W_c	40	ft	12.2	m
W_b	740	ft	225.6	m
W*			8.47	
V_{er}			15085176.57	m^3
K_o			1.4	
K_c			0.6	

Table 2.4.4-4 J. Strom Thurmond Dam Breach Parameters

Reference	B (m)	B (ft)	Z	tf (hrs)
Johnson and Illes	138.1	453		
Singh and Snorrason (1982, 1984)	230.1	755		0.25 to 1.0
MacDonald and Langridge-Monopolis (1984)				7.34
FERC (1987)	230.1	755	1 to 2	0.1 to 1.0
Froehlich (1987)	365.6	1199	2.1	
Bureau of Reclamation (1988)	138.2	453		1.52
Von Thun and Gillette	170.0	558		1.17
Froehlich (1995b)	679.0	2228		11.62

Table 2.4.4-5 Richard B. Russell Dam Input Variables

Input Variable	English Units	SI Units
Hw	150.1 ft	45.8 m
Hb	150.0 ft	45.7 m
Hd	150.0 ft	45.7 m
Storage	1700000 ac-ft	2096930484 m^3
S*		21941.45
W_c	20 ft	6.1 m
W_b	865 ft	263.7 m
W*		9.68
V_{er}		7274160.639 m^3
K_o		1.4
K_c		0.6

Table 2.4.4-6 Richard B. Russell Dam Breach Parameters

Reference	B (m)	B (ft)	Z	tf(hrs)
Johnson and Illes	137.2	450		
Singh and Snorrason (1982, 1984)	228.6	750		0.25 to 1.0
MacDonald and Langridge-Monopolis (1984)				5.63
FERC (1987)	228.6	750	1 to 2	0.1 to 1.0
Froehlich (1987)	258.3	847	2.4	
Bureau of Reclamation (1988)	137.3	450		1.51
Von Thun and Gillette	169.3	555		1.17
Froehlich (1995b)	501.7	1646		7.10

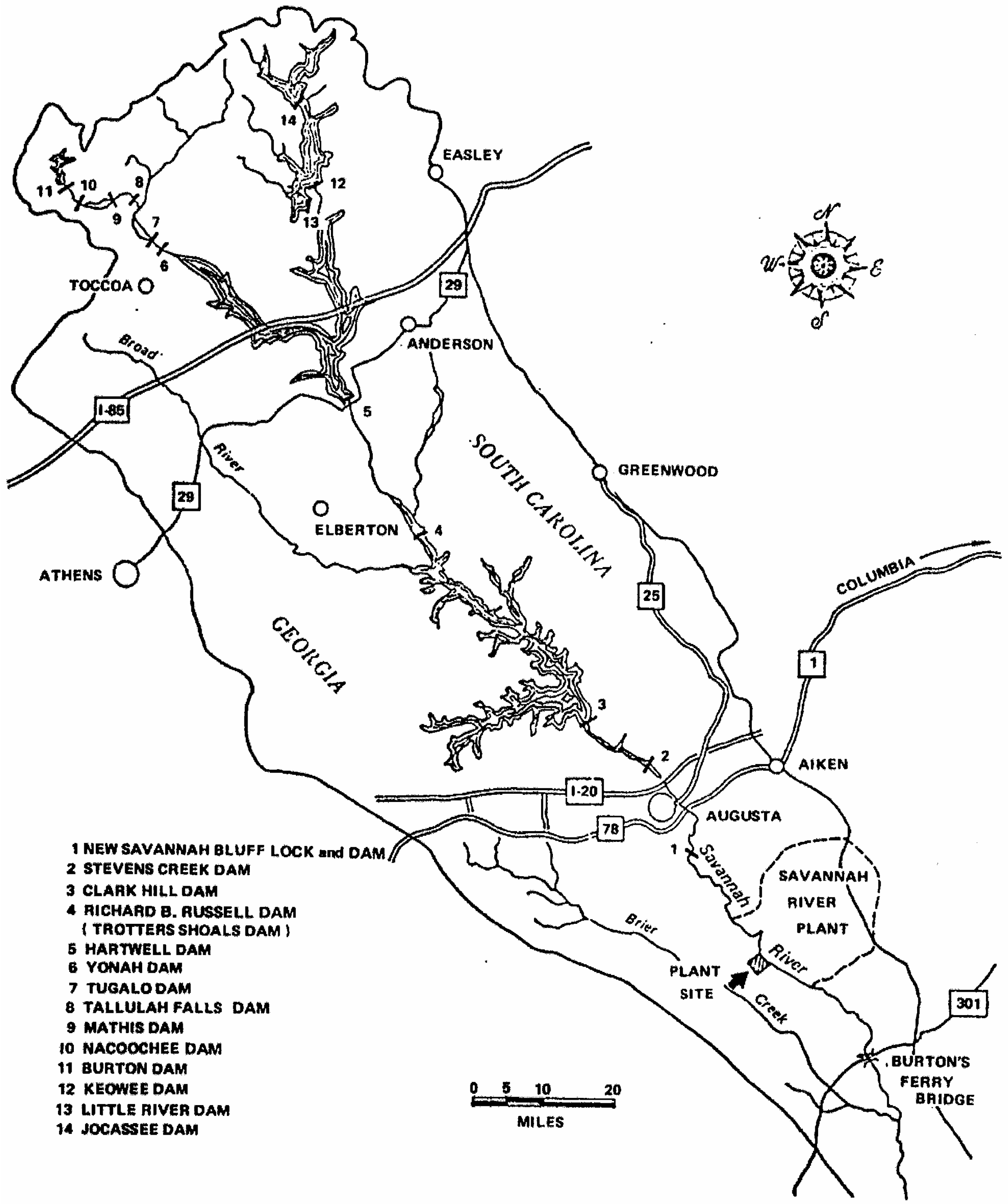
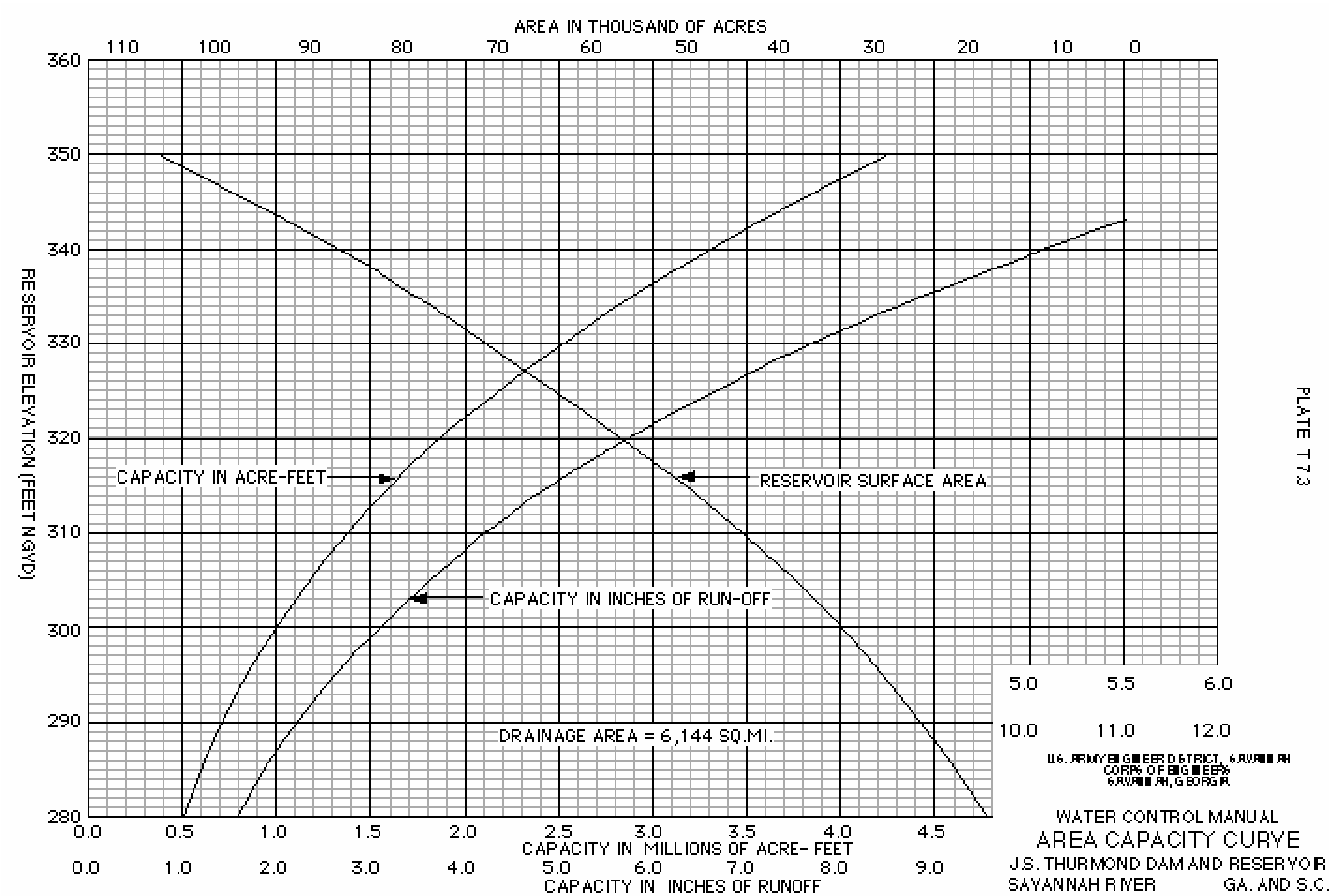
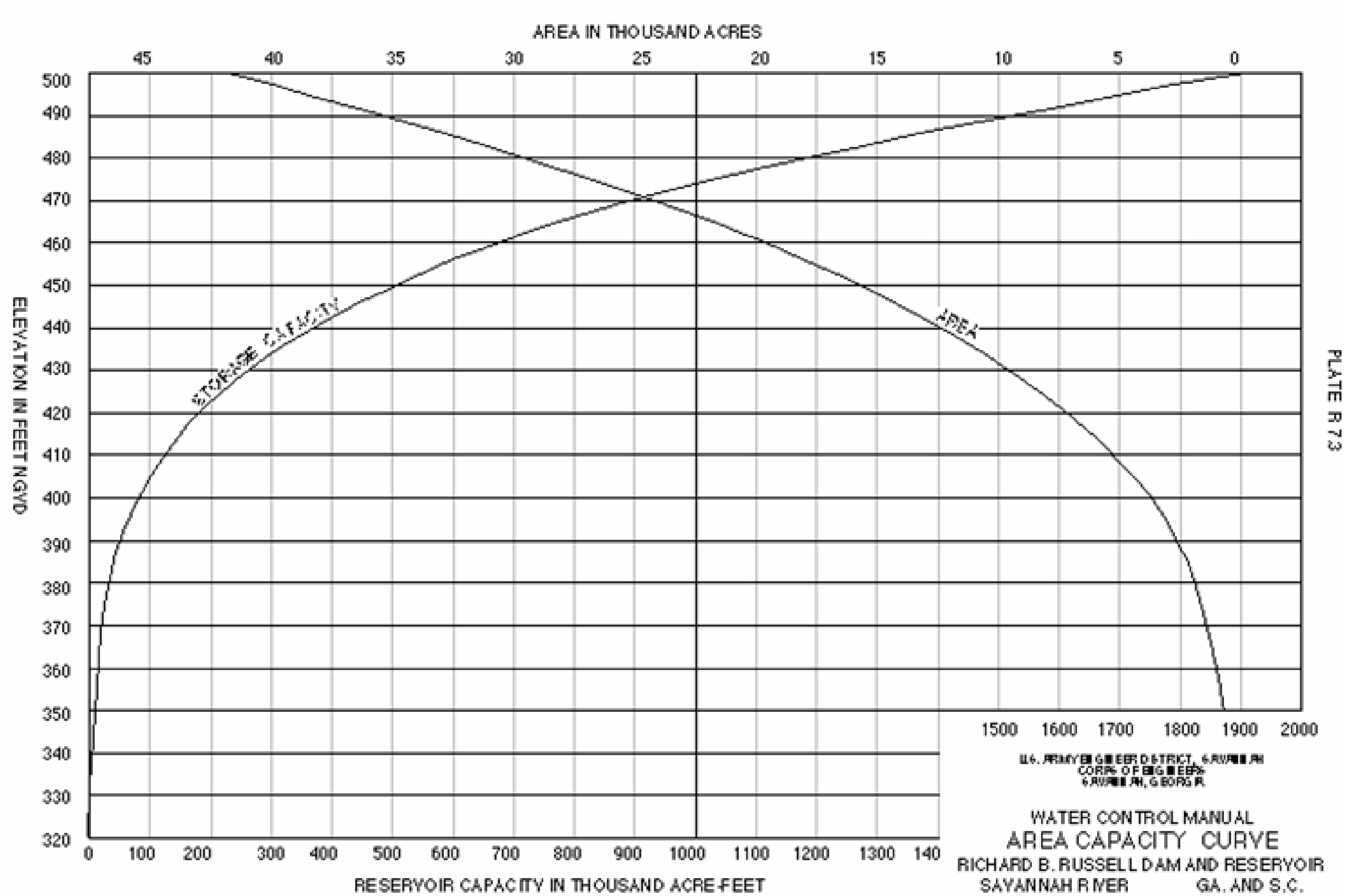


Figure 2.4.4-1 Savannah River Basin Dam Locations



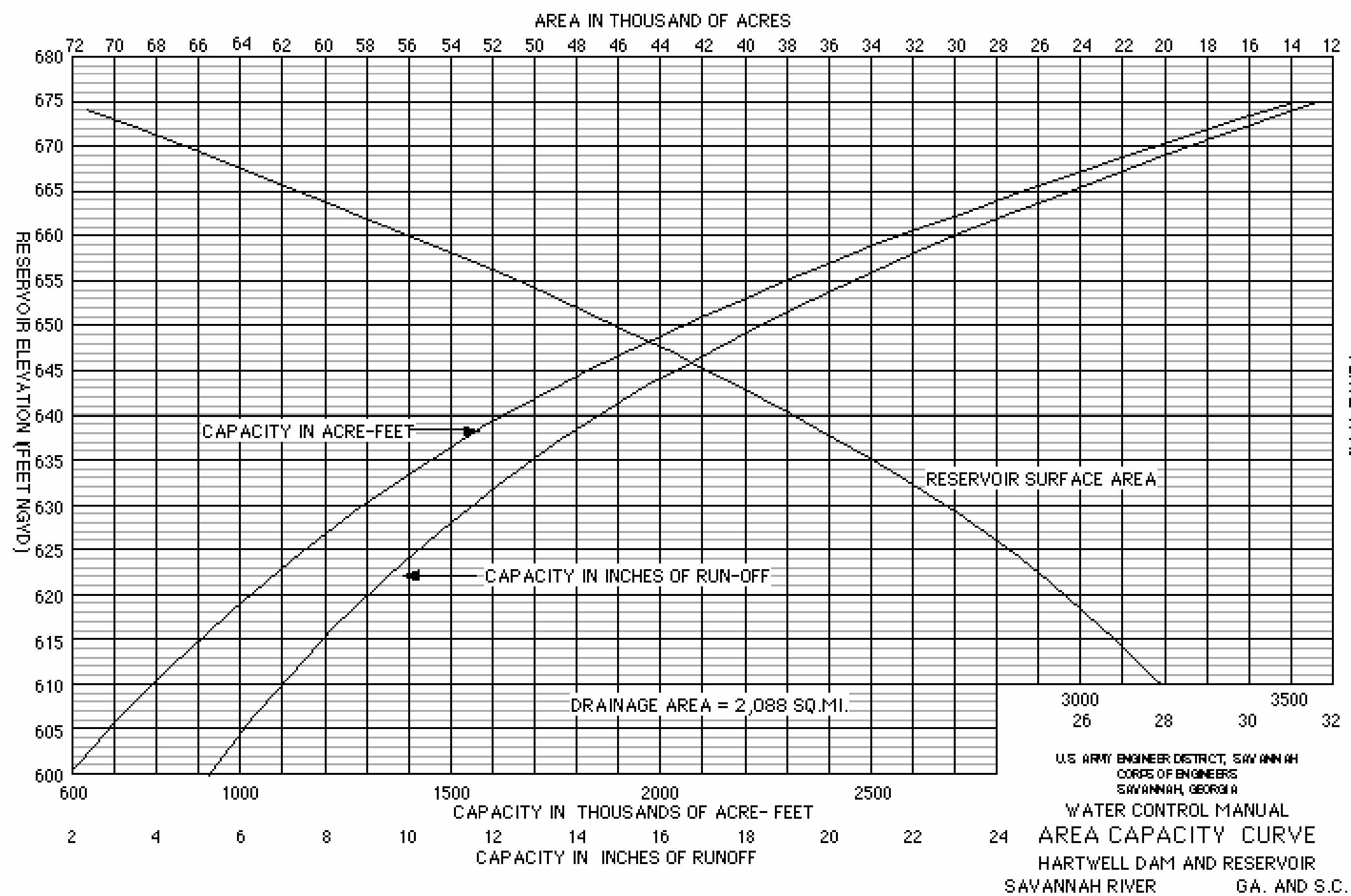
Source: USACE 1996

Figure 2.4.4-2 J. Strom Thurmond Area Capacity Curve



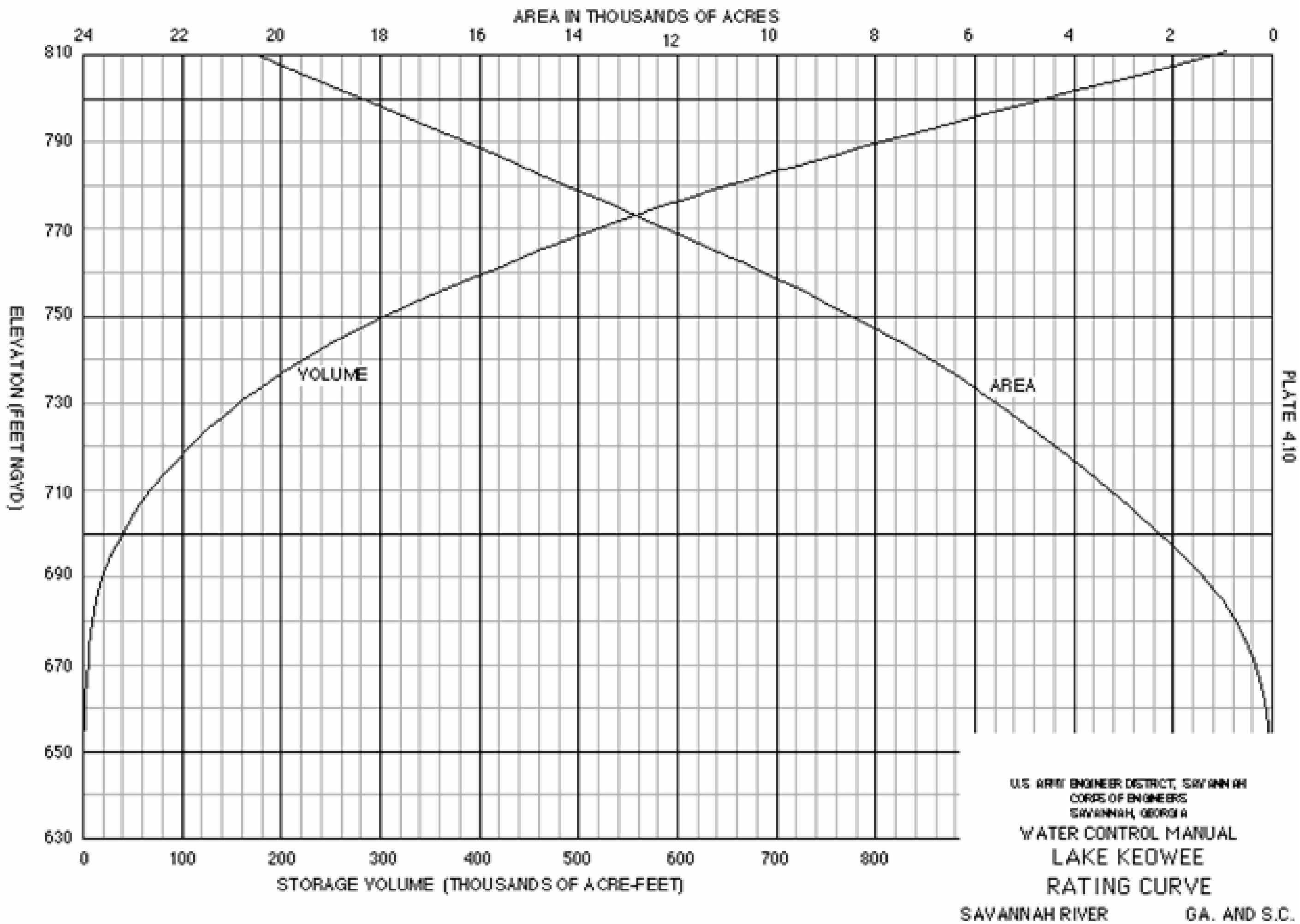
Source: USACE 1996

Figure 2.4.4-3 Richard B. Russell Area Capacity Curve



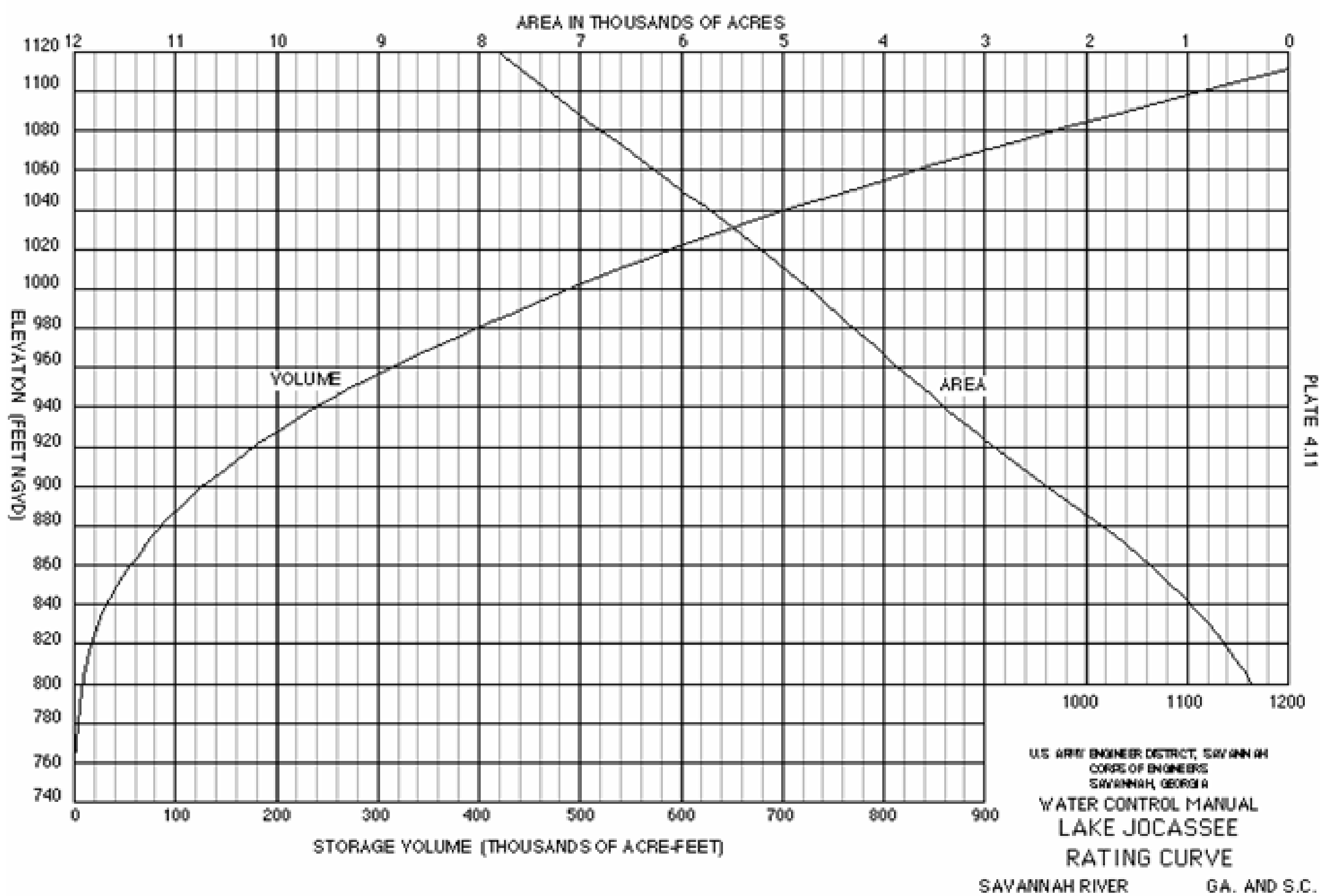
Source: USACE 1996

Figure 2.4.4-4 Hartwell Dam and Reservoir Area Capacity



Source: USACE 1996

Figure 2.4.4-5 Keowee Area Capacity Curve



Source: (USACE 1996)

Figure 2.4.4-6 Jocassee Area Capacity Curve

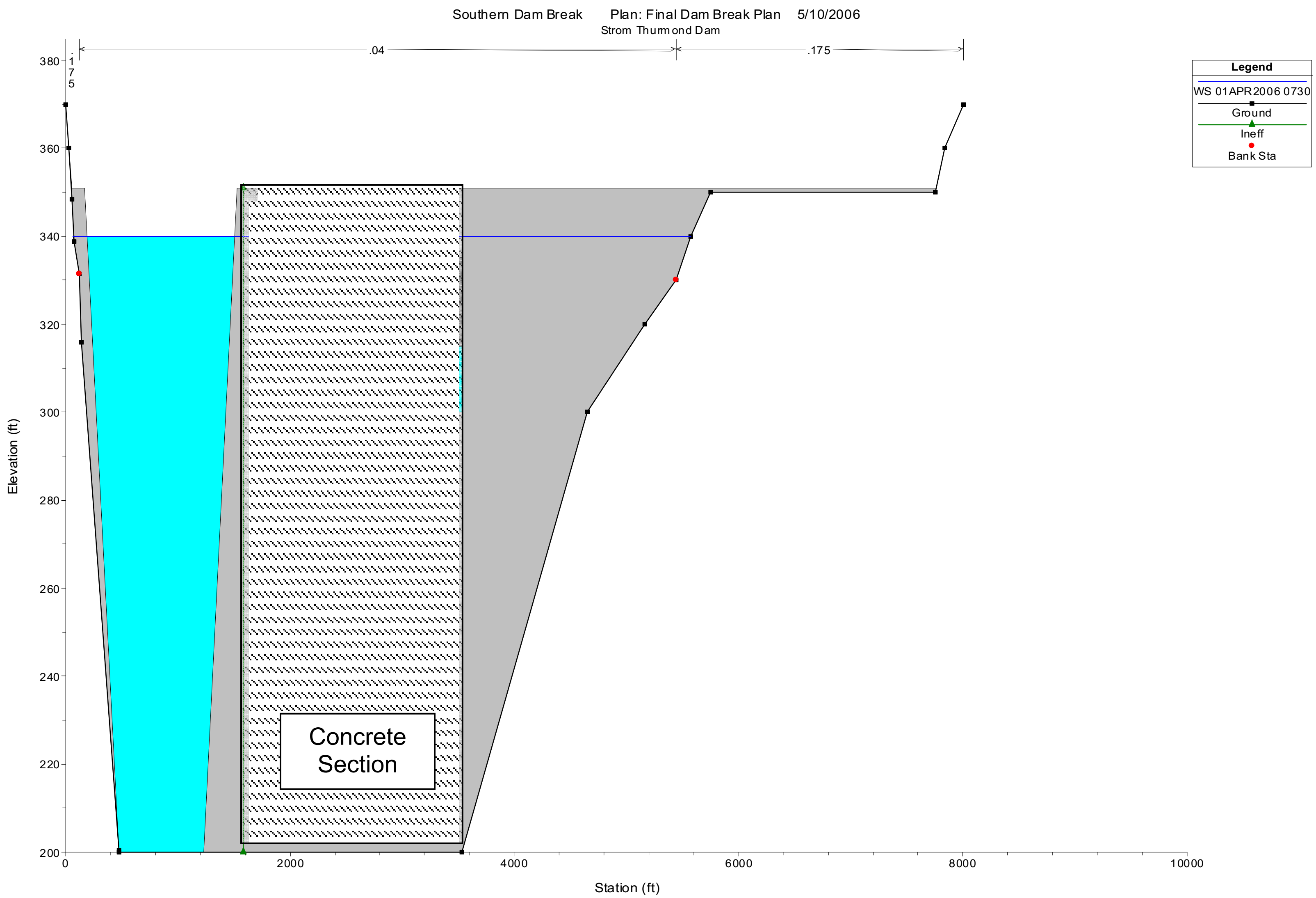


Figure 2.4.4-7 J. Strom Thurmond Dam Cross Section

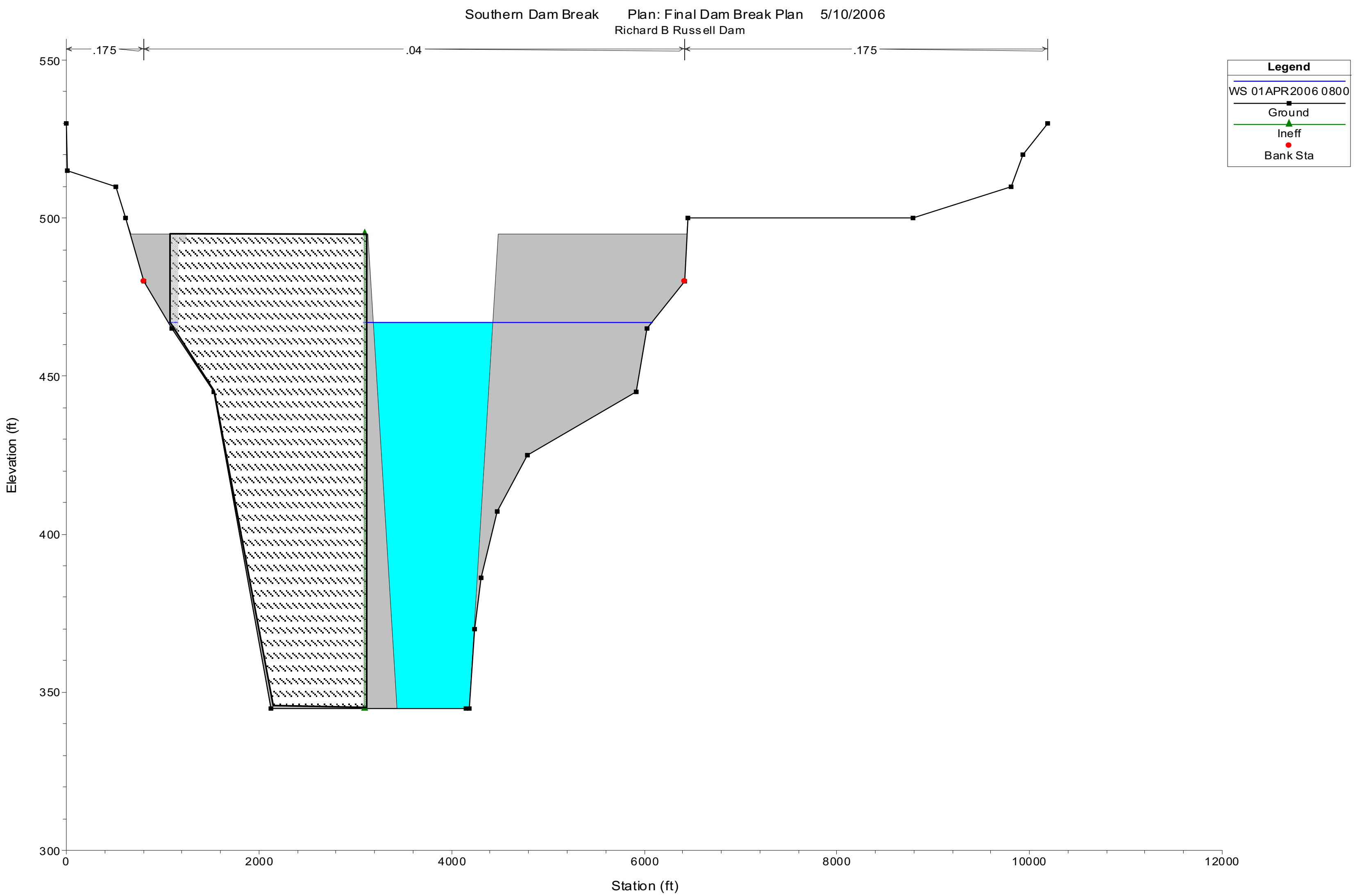


Figure 2.4.4-8 Richard B. Russell Dam Cross Section

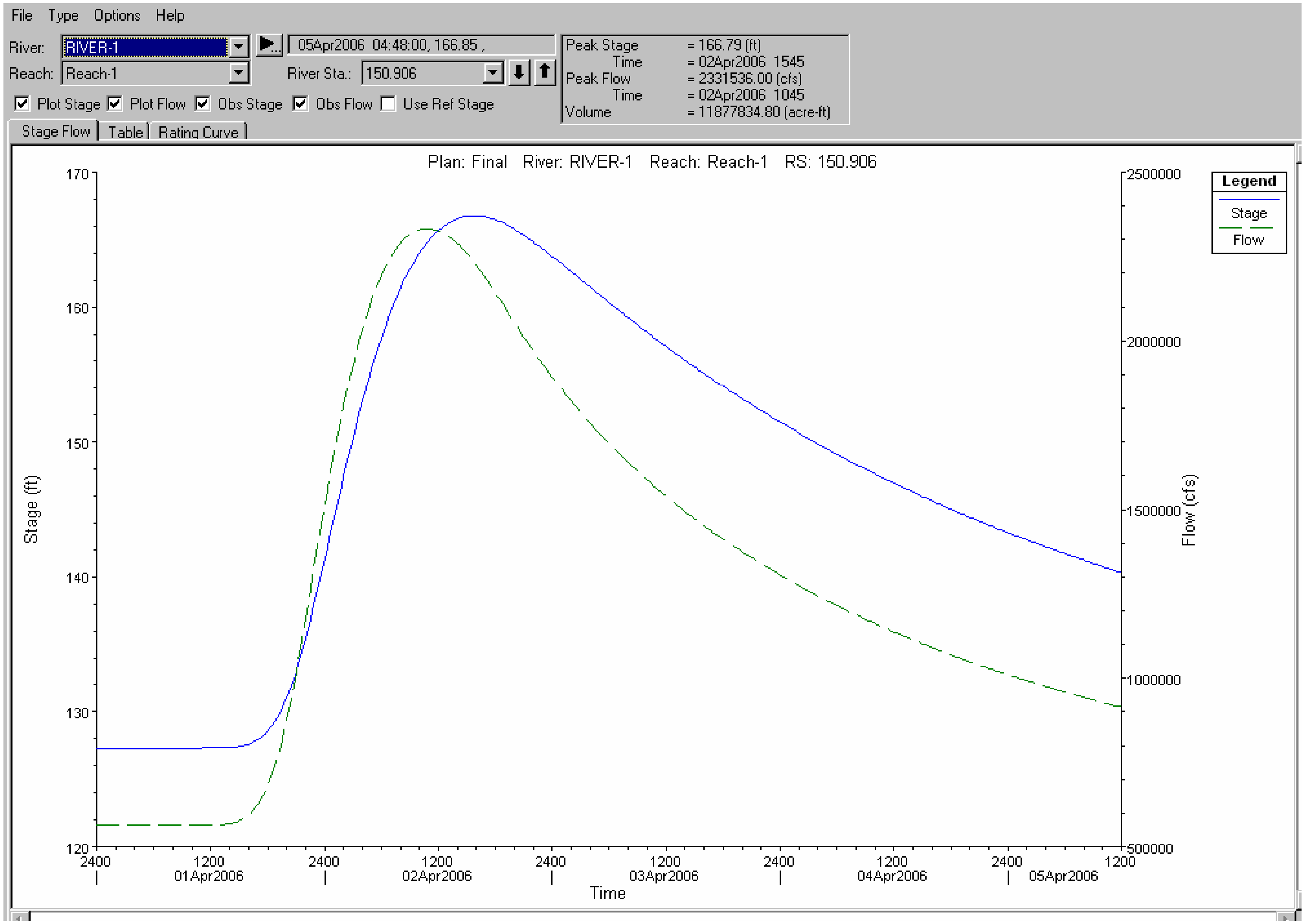


Figure 2.4.4-9 Dam Breach Flood Flow and Stage Hydrograph at the VEGP Site

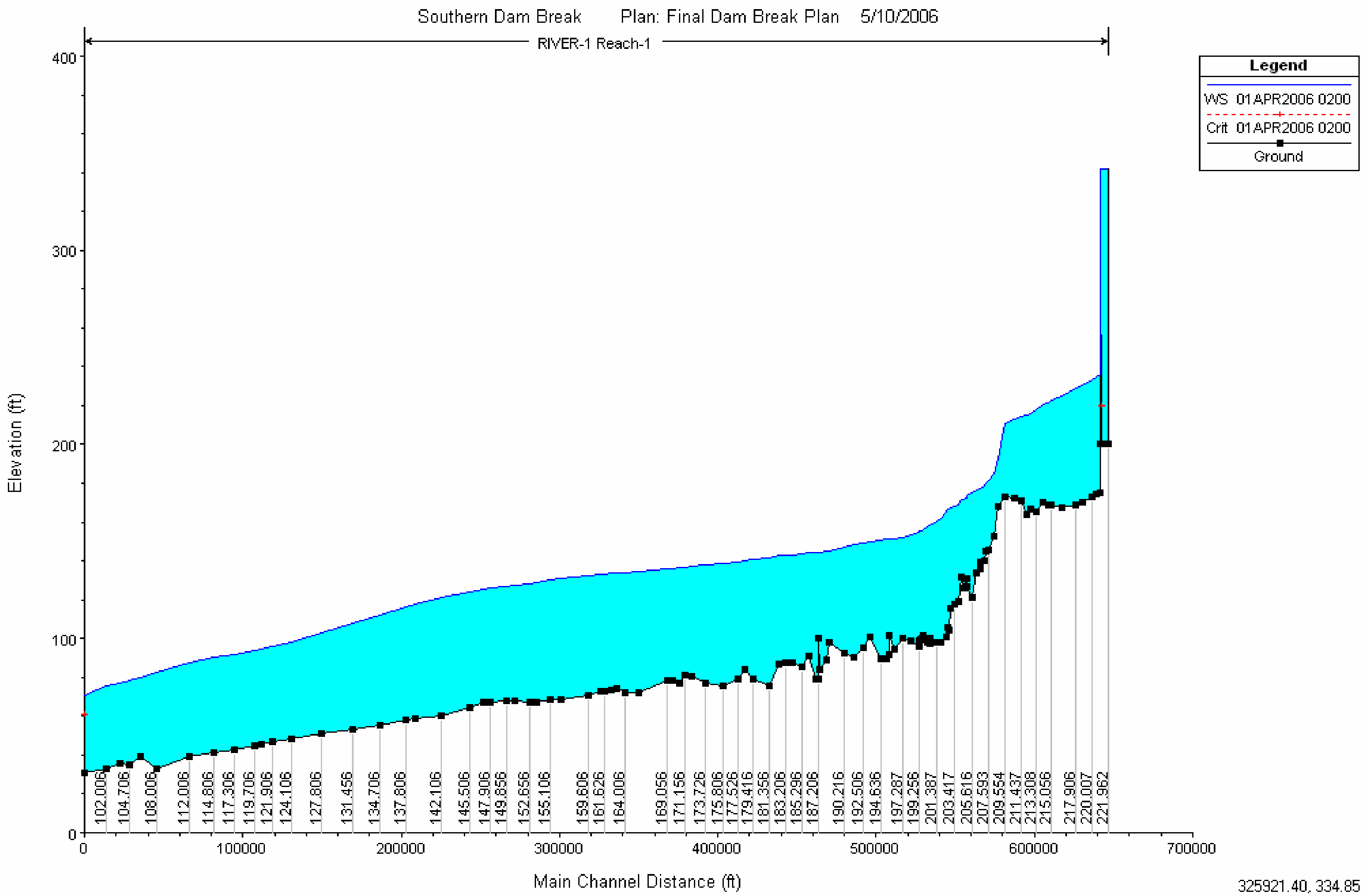


Figure 2.4.4-10 Savannah River SPF Water Surface Profile

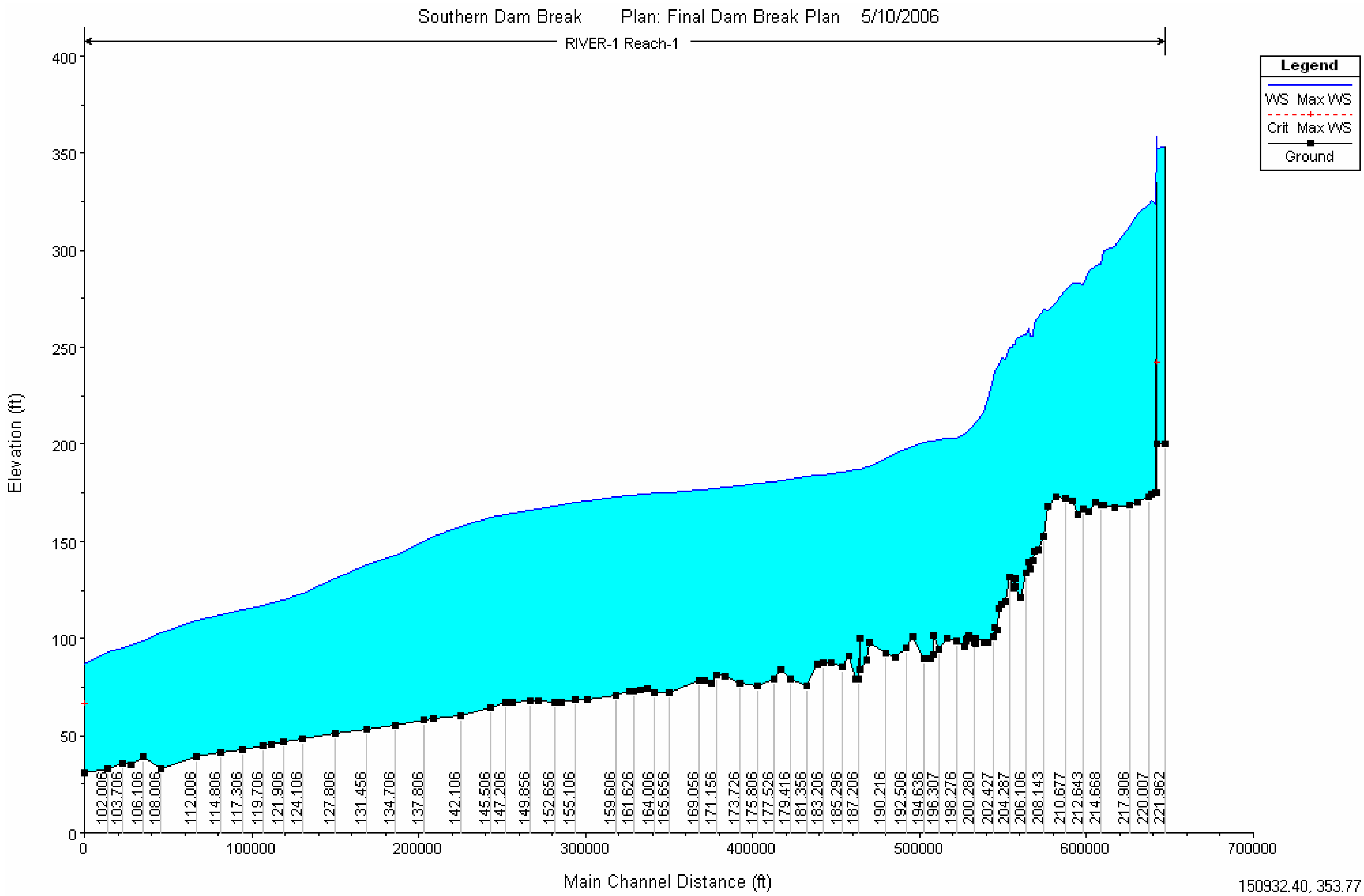


Figure 2.4.4-11 Savannah River Dam Breach Flood Maximum Water Surface Profile

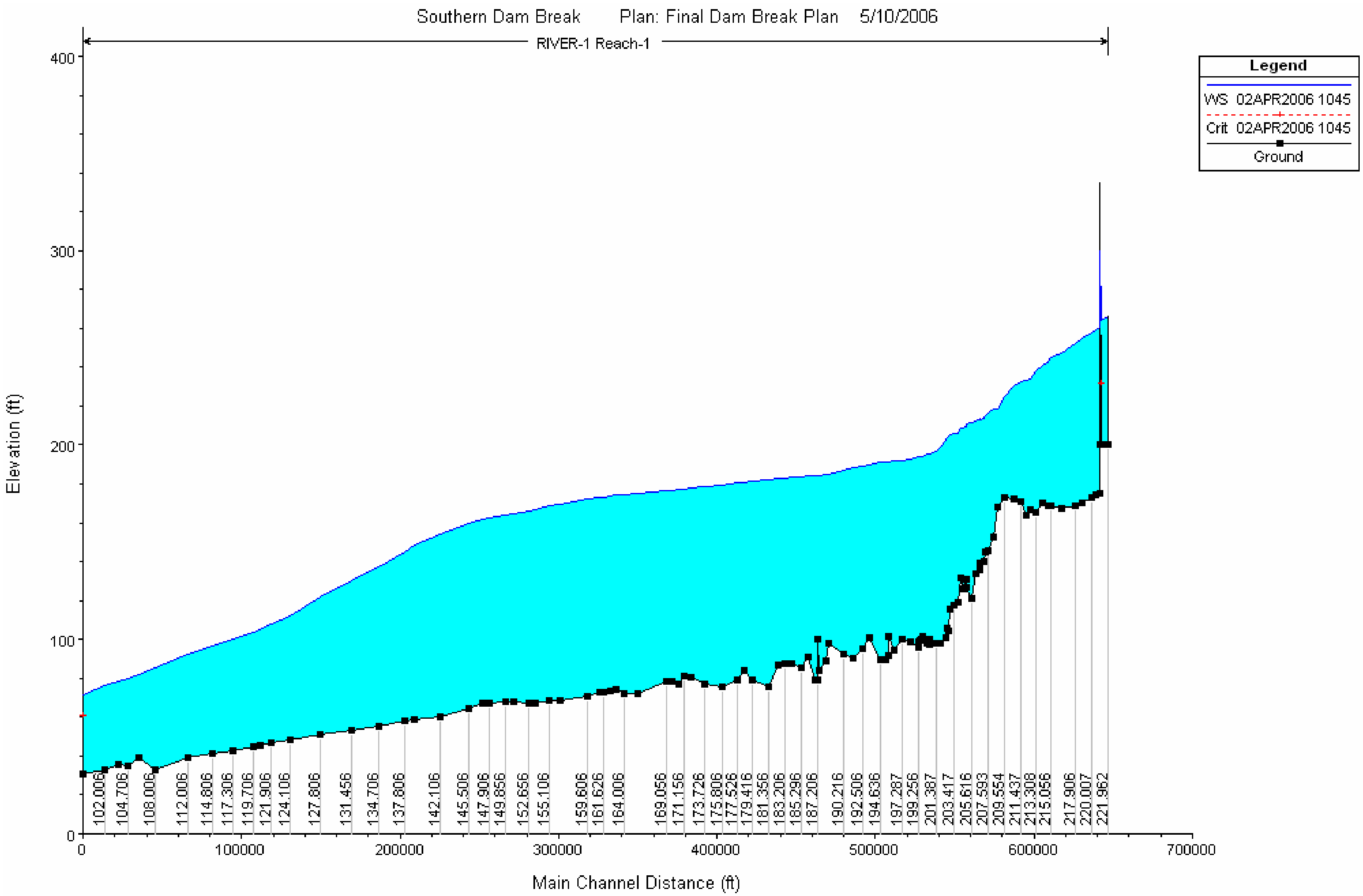


Figure 2.4.4-12 Savannah River Dam Breach Flood Water Surface Profile for Peak Discharge at VEGP Site

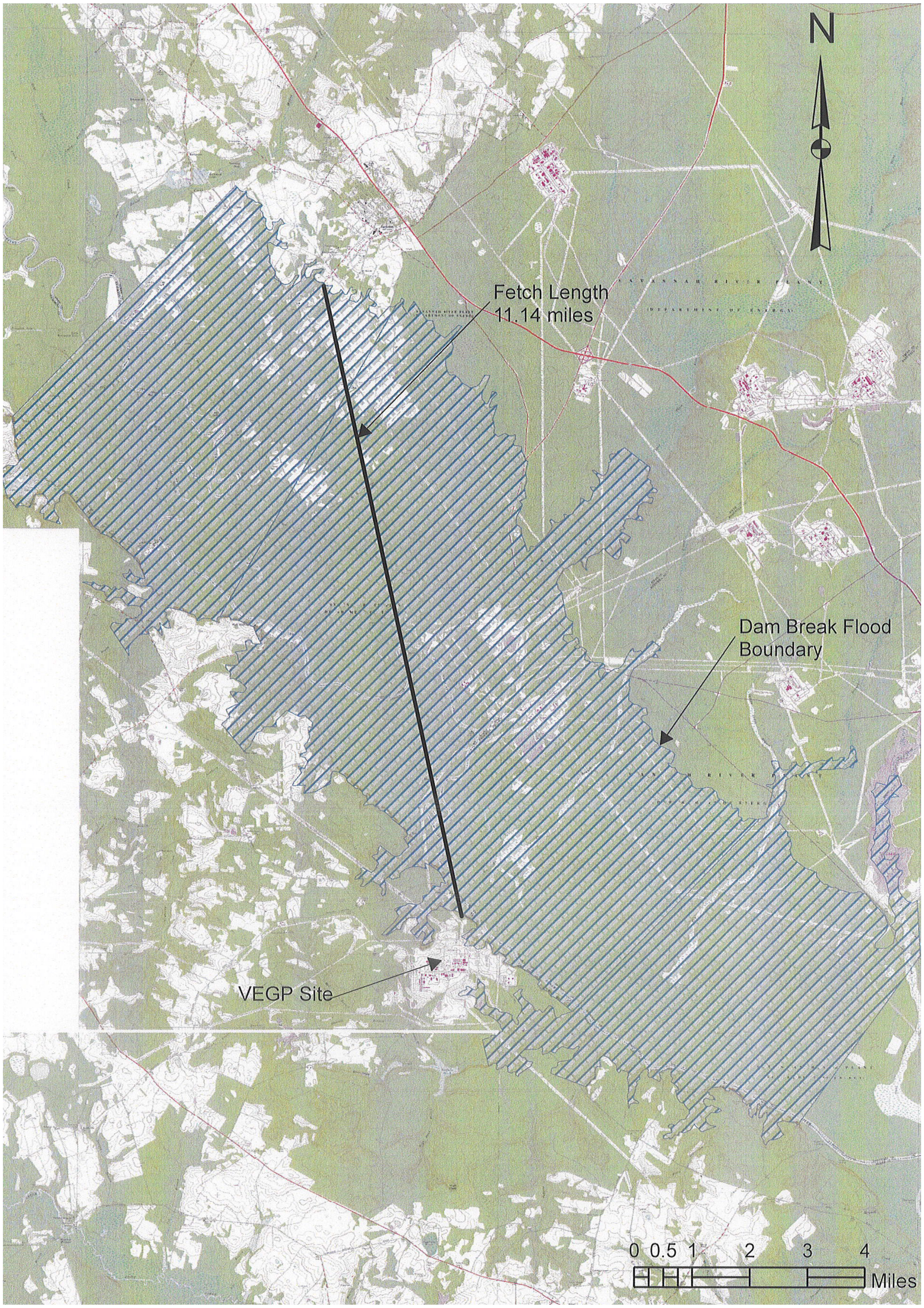


Figure 2.4.4-13 Maximum Fetch Length

Section 2.4.4 References

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2.4.5 Probable Maximum Surge and Seiche Flooding

The VEGP site is located on a coastal plain bluff on the west bank of the Savannah River approximately 151 River Miles inland from the Atlantic Ocean at grade El. 220 ft msl. Since the site is not located on an open or large body of water, surge or seiche flooding will not produce the maximum water levels at the site.

The Savannah River estuary region is occasionally exposed to extreme mid-Atlantic hurricanes. Between 1841 and 2004, only three major hurricanes, Category 3 or over (measured using the Saffir/Simpson Hurricane Scale), hit the coast of Georgia **(Blake et al. 2005)**. The most devastating hurricane on record with a landfall within approximately 100 miles of the Savannah River estuary was Hurricane Hugo, which hit the coast of South Carolina near Charleston in 1989. This Category 4 hurricane produced a 20-foot-high storm surge in the Cape Romain-Bulls Bay area in South Carolina **(NHC 2006)**.

Regulatory Guide 1.59, *Design Basis Floods for Nuclear Power Plant*, Revision 2, August 1977 (RG 1.59), Appendix C provides the distribution of probable maximum surge levels from hurricanes along the Atlantic coast. It shows maximum surge heights of 28.2 ft mean low water (mlw) at Folly Island, South Carolina, and 33.9 ft mlw at Jekyll Island, Georgia, located northeast and southwest of the Savannah River estuary, respectively. The probable maximum storm surge height at the mouth of the Savannah River can be estimated from these values following the procedure described in RG 1.59 Appendix C, as shown in Table 2.4.5-1.

The high tide at the estuary with a 10 percent exceedance level is defined as 9.0 ft mlw, and the mlw at the entrance to Savannah River, Georgia is 1.2 ft below msl **(ANSI/ANS-2.8 1992)**. Considering the coincidence of the probable maximum surge with a 10-percent-exceedence high tide at the river mouth, a probable maximum surge height of 32.3 ft mlw or 31.1 ft msl may be obtained for the Savannah River estuary, as shown in Table 2.4.5-1.

If it is assumed that a storm surge of such a magnitude is generated in the Savannah River estuary moving inland, the surge height would dissipate before reaching the VEGP site (151 River Miles inland and at grade El. 220 ft msl), and the site would be free from any resultant flood. Also, because the VEGP site is not located on a large enclosed body of water, flooding due to seiche is precluded.

The probable maximum surge data from RG 1.59 have not included those from the hurricanes after 1975. The inclusion of the data from recent hurricanes, including Hurricane Hugo, may have changed the probable maximum surge data from RG 1.59 somewhat. However, because the VEGP site is 151 River Miles inland and at grade El. 220 ft msl, the effects of probable maximum surge at the estuary of Savannah River would be insignificant at the site, and would not cause flooding of the site.

Table 2.4.5-1 Estimated Probable Maximum Surge at the Savannah River Mouth

Components	Unit	Folly Island[a]	Jekyll Island[b]	Savannah Estuary[c]	Comments
Wind Setup	ft mlw[d]	17.15	20.63	18.89	Taken as average of wind set-up from Folly Island and Jekyll Island
Pressure Set-up	ft mlw	3.23	3.34	3.29	Taken as average of pressure set-up from Folly Island and Jekyll Island
Initial Water Level	ft mlw	1.00	1.20	1.10	Taken as average of initial water level from Folly Island and Jekyll Island
10 % Exceedence High Tide	ft mlw	6.80	8.70	9.00	Magnitude at the Savannah River estuary taken from ANSI/ANS-2.8-1992; others from NRC RG 1.59 1977
Total Surge Height	ft mlw	**28.2**	**33.9**	**32.3**	Sum of wind and pressure set-up, initial water level, and 10% exceedence high tide
mlw to msl conversion[e]	ft			-1.2	Magnitude at the Savannah estuary obtained from ANSI/ANS-2.8-1992
Sea Surface Anomaly	ft			0.0	Magnitude at the Savannah estuary obtained from ANSI/ANS-2.8-1992
Total Surge Height	ft msl			**31.1**	

[a] NRC RG 1.59 1977
[b] NRC RG 1.59 1977
[c] Wind and pressure set-up, and initial water level averaged from Folly Island and Jekyll Island, tidal data was obtained from ANSI/ANS-2.8-1992
[d] Mean low water (mlw)
[e] Mean sea level (msl) = (mlw +1.2) ft at the Savannah estuary **(ANSI/ANS-2.8 1992)**

Section 2.4.5 References

(ANSI/ANS-2.8 1992) Determining Design Basis Flooding at Power Reactor Sites, American National Standard/American Nuclear Society, July 1992.

(Blake et al. 2005) Blake, E.S., E.N. Rappaport, J.D. Jarrell, and C.W. Landsea, *The Deadliest, Costliest, and Most Intense United States Tropical Cyclones from 1851 to 2004 (and Other Frequently Requested Hurricane Facts)*, Tropical Prediction Center, National Hurricane Center, Miami, Florida, August 2005.

(NHC 2006) Hurricane History, National Hurricane Center, Web site address: http://www.nhc.noaa.gov/HAW2/english/history.shtml#hugo, accessed April 7, 2006.

2.4.6 Probable Maximum Tsunami Flooding

Since the VEGP site is not located on an open ocean coast or large body of water, tsunami-induced flooding will not produce the maximum water level at the site.

The Atlantic Ocean region is characterized by infrequent seismic and volcanic activities, resulting in few recorded tsunamis. The majority of tsunamis in the Atlantic Ocean and Caribbean Sea have been either triggered by seismic (earthquake) activity or the result of volcanic eruption. The most notable Atlantic tsunami was generated by the Great Lisbon Earthquake of 1755. The tsunami hit the coasts of Portugal, Spain, and northern Africa and traveled across the Atlantic Ocean with a 10-to-15-ft wave reportedly reaching the Caribbean coasts **(Maine DOC 2006)**. Computer models suggested a wave height of 10 ft along the east coast of the US **(NOAA 2006)** from this tsunami.

The effects of any tsunami with similar height approaching the Savannah River estuary would be dissipated before reaching the VEGP site (151 River Miles inland and at grade El. 220 ft msl), and the site would be free from any resultant flood.

Section 2.4.6 References

(Maine DOC 2006) *Tsunamis in the Atlantic Ocean*, Maine Geological Survey, Maine Department of Conservation, Web site address: http://www.maine.gov/doc/nrimc/mgs/explore/hazards/tsunami/jan05.htm, accessed April 10, 2006.

(NOAA 2006) *Tsunami, Tidal Waves and Other Extreme Waves*, National Weather Service Forecast Office, Philadelphia/Mount Holly, National Oceanic and Atmospheric Administration, Web site address: http://www.erh.noaa.gov/er/phi/reports/tsunami.htm, accessed April 10, 2006.

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2.4.7 Ice Effects

2.4.7.1 Ice Conditions and Historical Ice Formation

Long-term air temperature records available at the National Weather Service (NWS) weather station at Augusta, Georgia (Bush Field), and seven other cooperative observation stations around the VEGP site are used to analyze historical extreme air temperature variations at the VEGP site. The analysis was also supported by onsite temperature data measured at the VEGP site. A detailed description of station locations and data availability is presented in Section 2.3.2.

The climate at the VEGP site is characterized by short, mild winters and long, humid summers. Local climatology data at Augusta, Georgia, for a period of 129 years show an average annual air temperature of 64.2°F (17.9°C) **(NCDC 2003)**. January is the coldest month, with an average temperature of 46.8°F (8.2°C). July is the warmest, with an average temperature of 81.3°F (27.4°C). Based on temperature records at Augusta and seven surrounding stations, the lowest air temperature on record was observed to be -4.0°F (-20.0°C) at Aiken in January 1985 (Table 2.3-3). The January 1985 event produced a minimum air temperature of -0.1°F (-17.8°C) at the VEGP site, with the air temperature remaining below freezing (32°F [0°C]) for only about 50 hours (Figure 2.4.7-1). VEGP temperature data from 1984 through 2002 show that the average daily air temperature has remained below freezing for a maximum of 3 consecutive days (Table 2.4.7-1). In three instances, the average daily air temperature remained above freezing the entire year.

Historical water temperatures recorded at five USGS stations located on the Savannah River **(Dyar and Alhadeff 1997)** are presented in Table 2.4.7-2. These USGS stations include: No. 02187500 near Iva, South Carolina, at River Mile 280.4; No. 02189000 near Calhoun Falls, South Carolina, at River Mile 263.6; No. 02197000 at Augusta, Georgia, at River Mile 187.4; No. 02197500 at Burtons Ferry near Milhaven, Georgia, at River Mile 118.7; and No. 02198500 near Clyo, Georgia, at River Mile 60.9. The data cover a river reach that includes the VEGP site. Within this river reach, the minimum water temperature is observed in February, which shows a variation between 39.2°F (4.0°C) and 42.8°F (6.0°C).

Based on the record of air and water temperatures, it is very unlikely that surface or frazil ice formation would occur in the Savannah River in the vicinity of the proposed VEGP Units 3 and 4 river intake location.

2.4.7.2 Ice Jam Events

There are no recorded ice jam events in the lower reach of the Savannah River based on a search of the *Ice Jam Database* of the US Army Corps of Engineers **(USACE 2006)**.

The large dams and reservoirs on the Savannah River located upstream of the VEGP site reduce the possibility of any surface ice or ice floes moving downstream. Since the water temperatures in the lower reach of the Savannah River remain consistently above freezing, as seen in Table 2.4.7-2, the formation of frazil ice or ice jams would be very unlikely at the proposed VEGP Units 3 and 4 intake location.

2.4.7.3 Description of the Cooling Water System

The VEGP Units 3 and 4 will be Westinghouse AP1000 reactors and use a closed cycle cooling system with wet, natural-draft cooling towers for circulating water system cooling. The river intake system, comprising an intake canal and a pump intake structure, will be located upstream from the existing river intake structure for the VEGP Units 1 and 2. Makeup water from the Savannah River will be required to replace evaporative water losses, drift losses, and blowdown discharge from the circulating water system cooling towers.

For safety-related cooling, AP1000 reactors use passive ultimate heat sink (UHS) systems with in-plant storage water. These reactor plants do not require an external safety-related UHS system to reach safe shutdown. Also, the AP1000 design have a non-safety-related heat removal auxiliary heat sink–service water system (SWS) used for shutdown, normal operations, and anticipated operational events. Make-up water to the SWS will be supplied from site groundwater wells or a site water storage tank. Consequently, no water will be necessary from the Savannah River or from any other open surface water sources for the AP1000 UHS and SWS. Therefore, even a very unlikely ice event on the Savannah River will not have any impact on safety-related UHS or non-safety-related SWS of the proposed AP1000 units.

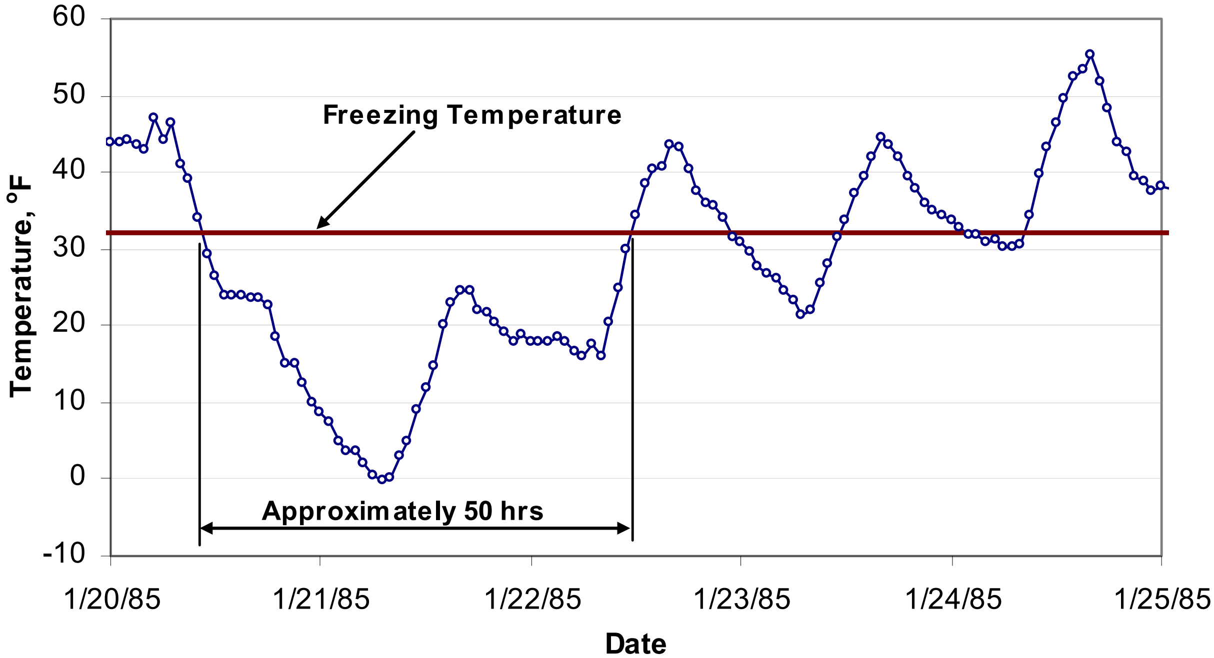
Table 2.4.7-1 Variation in Lowest Average Daily Temperatures and Number of Days with Average Daily Temperature Below Freezing

Year	Lowest Average Daily Temp ºF (ºC)		Date Lowest Average Daily Temp Occurred	Maximum No. of Consecutive Freezing Days	Total No. of Freezing Days
1984	25.7	-(3.5)	12/7/1984	1	3
1985	11.9	-(11.2)	1/21/1985	3	5
1986	20.7	-(6.3)	1/28/1986	2	3
1987	31.2	-(0.4)	1/27/1987	1	1
1988	25.2	-(3.8)	1/8/1988	3	6
1989	19.0	-(7.2)	12/23/1989	3	6
1990	37.3	(2.9)	12/25/1990	0	0
1991	26.0	-(3.3)	2/16/1991	1	1
1992	33.4	(0.8)	1/16/1992	0	0
1993	30.4	-(0.9)	3/14/1993	1	1
1994	21.3	-(5.9)	1/19/1994	2	4
1995	29.2	-(1.6)	2/9/1995	2	4
1996	20.8	-(6.2)	1/8/1996	3	8
1997	28.9	-(1.7)	1/18/1997	2	2
1998	34.8	(1.6)	12/26/1998	0	0
1999	25.2	-(3.8)	1/14/1999	3	3
2000	26.5	-(3.1)	12/20/2000	2	4
2001	30.9	-(0.6)	1/3/2001	2	2
2002	29.7	-(1.3)	1/4/2002	2	2
Average days				1.7	2.9

Table 2.4.7-2 Variation in the Minimum Water Temperatures at Five Locations on the Savannah River

USGS Station No.	Location	River Mile	Data Period	Observed Minimum Temperature, ^oF (^oC)											
				Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
02187500	Savannah River near Iva, SC	280.4	1958-1984	62.6 (17.0)	55.4 (13.0)	46.4 (8.0)	44.6 (7.0)	39.2 (4.0)	42.8 (6.0)	48.2 (9.0)	48.2 (9.0)	57.2 (14.0)	55.4 (13.0)	53.6 (12.0)	57.2 (14.0)
02189000	Savannah River near Calhoun Falls, SC	263.6	1957-1974	65.3 (18.5)	59 (15.0)	46.4 (8.0)	46.4 (8.0)	42.8 (6.0)	51.8 (11.0)	53.6 (12.0)	59.9 (15.5)	64.4 (18.0)	66.2 (19.0)	68 (20.0)	71.6 (22.0)
02197000	Savannah River at Augusta, GA	207.0	1958 - 1973	64.4 (18.0)	59 (15.0)	51.8 (11.0)	42.8 (6.0)	42.8 (6.0)	50 (10.0)	57.2 (14.0)	59.9 (15.5)	66.2 (19.0)	66.2 (19.0)	64.4 (18.0)	69.8 (21.0)
02197500	Savannah River at Burtons Ferry near Milhaven, GA	118.7	1957-1979	63.5 (17.5)	58.1 (14.5)	46.4 (8.0)	43.7 (6.5)	39.2 (4.0)	44.6 (7.0)	55.4 (13.0)	59 (15.0)	66.2 (19.0)	73.4 (23.0)	71.6 (22.0)	71.6 (22.0)
02198500	Savannah River near Clyo, GA	60.9	1938 - 1984	59.9 (15.5)	46.4 (8.0)	44.6 (7.0)	41 (5.0)	40.1 (4.5)	44.6 (7.0)	57.2 (14.0)	57.2 (14.0)	68 (20.0)	73.4 (23.0)	71.6 (22.0)	67.1 (19.5)

Source: Dyer and Alhadeff 1997



(The temperature remained below freezing for approximately 50 consecutive hours.)

Figure 2.4.7-1 Lowest Temperature Observed at the VEGP Site in 1985

Section 2.4.7 References

(Dyar and Alhadeff 1997) Dyar, T.R., and S.J. Alhadeff, *Stream-Temperature Characteristics in Georgia*, U.S. Geological Survey, Water Resources Report 96-4203, Atlanta, Georgia, 1997.

(NCDC 2003) *Local Climatological Data, Annual Summary with Comparative Data, Augusta, Georgia*, National Climatic Data Center, ISSN 0198-1587, Asheville, North Carolina, 2003.

(USACE 2006) *Ice Jam Database*, U.S. Army Corps of Engineers, Cold Region Research and Engineering Laboratory, Web site address: https://rsgis.crrel.usace.army.mil/icejam/index.html, accessed April 11, 2006.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 Cooling Water Canals

The proposed VEGP Units 3 and 4 will use a closed cycle cooling system for condenser heat rejection and will use wet, natural-draft, cooling towers for circulating water system cooling. Makeup water from the Savannah River will be required to replace evaporative water losses, drift losses, and blowdown discharge. The river intake for VEGP Units 3 and 4 will withdraw makeup water from the Savannah River at a maximum rate of approximately 57,784 gpm (128.7 cfs). The intake system will be located upstream of the river intake of the existing VEGP units. The makeup water will be pumped directly to the cooling tower basin.

For safety related cooling, AP1000 reactor plants use passive ultimate heat sink (UHS) systems with sufficient in-plant storage water for safety-related water cooling. These reactor plants do not require an external safety-related UHS system to reach safe shutdown. Therefore, the river intake system will not be part of the safety-related facilities for VEGP Units 3 and 4, and the river intake canal and structure will have no safety-related functions. These reactor plants also have a non-safety-related heat removal auxiliary heat sink–service water system (SWS) used for shutdown, normal operations, and anticipated operational events. Make-up water to the SWS will be supplied from site groundwater wells; therefore, the SWS will not depend on the river intake system.

The river intake system for VEGP Units 3 and 4 would consist of an intake canal and an intake structure. The design details of the river intake system will be established during the COL applications. An overview of the conceptual design is provided below.

The river intake canal will be approximately 200 ft long and 150 ft wide, with a bottom elevation of about El. 70 ft msl. The bottom of the canal would be unpaved and bordered by vertical sheet piles, the tops of which would be extended to about El. 98 ft msl. The river intake canal would also act as a siltation basin and will incorporate a sill to reduce sediment inflow into the canal. At the minimum river operating level (78 ft msl), the flow velocity in the new canal would be about 0.1 fps, calculated based on a maximum makeup water demand of 128.7 cfs. Because the river intake canal would also act as the siltation basin, maintenance dredging may be necessary to maintain the canal invert elevation. Also, the canal embankment slopes would be protected using rip-rap of appropriate design specifications.

The intake structure, located at the end of the river intake canal, would house multiple makeup water pumps, traveling band screens, and trash racks with raking mechanisms. For each of the two new units, three 50-percent-capacity, vertical wet-pit pumps would be installed in the intake structure, with one makeup water pump at each pump bay, along with one dedicated traveling band screen and a trash rack.

Because VEGP Units 3 and 4 will not rely on the Savannah River for safe shutdown, a minimum river water level will not be necessary for safety-related cooling water supply.

2.4.8.2 Reservoirs

VEGP Units 3 and 4 will not have any cooling water reservoirs.

2.4.9 Channel Diversions

The VEGP site area lies in the Upper Coastal Plain of the Atlantic Coastal Plain physiographic province and is bordered by the Savannah River to the east. The surrounding topography consists of gently rolling hills with surface topography elevation ranges from about 200 to nearly 300 ft msl. Local site drainage consists of a principally dendritic drainage pattern where all major streams are tributary to the Savannah River. The VEGP site and surrounding areas are shown in Figure 2.4.1-3.

Near the site area, incision of the Savannah River has produced a deep valley with topographic relief of nearly 150 ft from the river surface and a valley width of over 4 mi. The present-day river course is located at the western side of the valley, forming steep bluffs near the VEGP site. The river floodplain consists of a broad alluvial surface extended on the eastern side at heights of 5–10 ft above the riverbank.

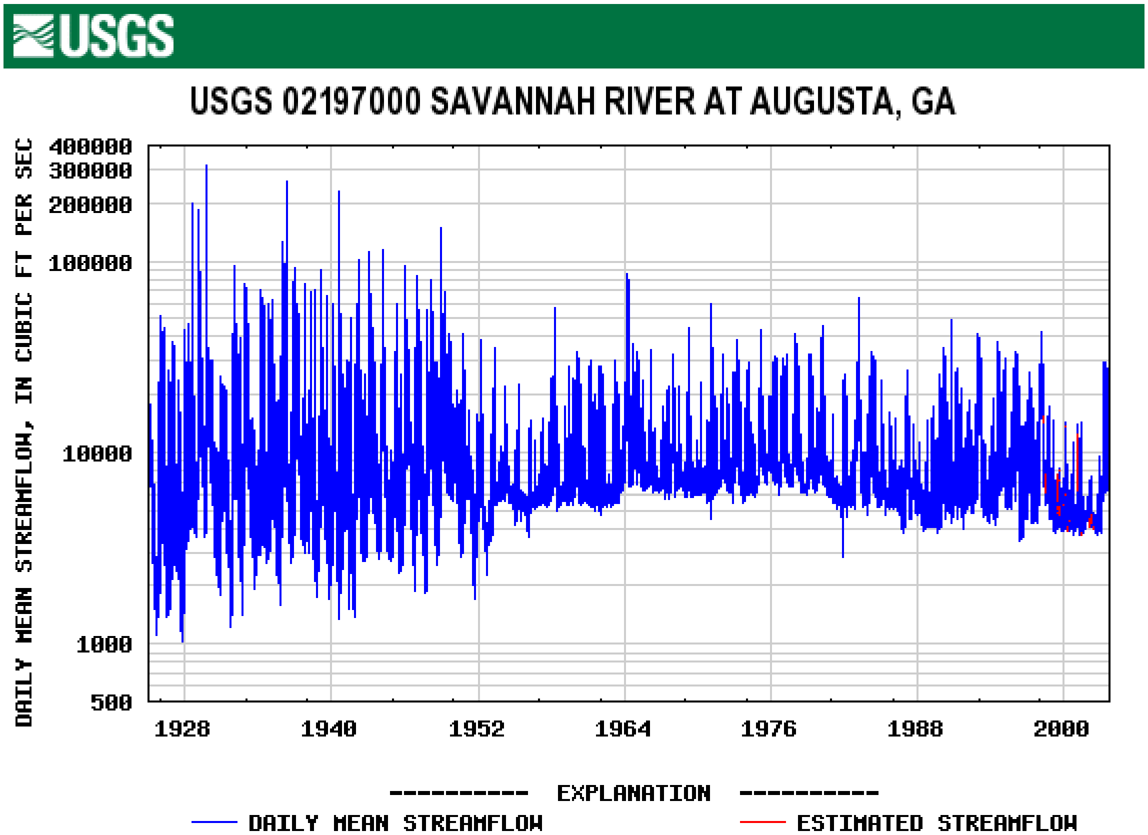
Rivers in the Upper Coastal Plain are typically underlain by sands, clays, limestones, and gravels and exhibit gentle to moderate bed slopes, wide floodplain development, and increased sinuosity. Consequently, diversion of the river channel in this region cannot be completely discounted.

Historical development of the river plan-form, which is the shape on map of river bank-line, near the VEGP site is well-represented in the USGS 7.5-minute series (topographic) maps. Oxbow lakes, meander cutoffs, abandoned meanders, low-lying swamps, and forested wetlands provide considerable evidence of historical channel plan-form development. Although meander river plan-form is present upstream and downstream of the site, the Savannah River near the site has a relatively straight and stable reach extending approximately from River Mile 143 to River Mile 152. A comparison of river bank-lines between 1965 and 1989, obtained from USGS topographic maps **(USGS 1989a**; **USGS 1989b**; **USGS 1989d)** and topographic maps used for VEGP Units 1 and 2, shows a nearly unchanged river plan-form within the reach during this period.

Since 1952, the Savannah River flow has been regulated by large federal multipurpose projects: Hartwell Dam, Richard B. Russell Dam, and J. Strom Thurmond (also known as Clarks Hill) Dam. A major impact of dam operation on river flow downstream of the J. Strom Thurmond Dam is the modulation of the outflow hydrograph, with reduced peaks and increased low-flow rates, as can be seen from Figure 2.4.9-1. Such flow modulation results in much-reduced river morphological activity, and a sudden river plan-form change is unlikely.

It is, therefore, unlikely that the river at the VEGP site will be diverted from the river intake by natural causes. Furthermore, analysis for existing VEGP Units 1 and 2 indicate that any possible effect on water supply to the intake from river channel diversion should come from extremely slow changes, which can be remedied as they occur.

While it is unlikely that a diversion of the main river channel will occur, such a diversion, either upstream or downstream of the proposed river intake, cannot be discounted. The river upstream and downstream from the proposed river intake has bluffs and steep slopes along the west bank. If it is assumed that a bluff slid into the river bed just upstream from the river intake structure, it may obstruct the flow of the main river channel, and river flow would divert over the floodplain on the eastern side of the river and away from the river intake. This could result in loss of the river intake due to river water starvation. Likewise, if a bluff slid into the river bed just downstream of the river intake structure, it again may obstruct the flow of the main river channel, but could possibly flood the river intake structure before diverting river water over the floodplain on the eastern side of the river. In this case, the river intake structure would be lost due to flooding. However, all the safety-related cooling water systems for the proposed AP1000 reactor plants would not use water from the river intake. Hence, the river intake would not be classified as a safety-related structure and loss of the river intake for either of these described scenarios would have no adverse affect on plant safety.



Source: USGS 2006b

Figure 2.4.9-1 Variation in Daily Mean Streamflow Rates at Augusta, Georgia, on the Savannah River (USGS Stream Gauging Station 02197000, Savannah River at Augusta, Georgia), Showing Streamflow Modulation After the Construction of the Dams

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(USGS 1989a) United States Geological Survey, Shell Bluff Landing Quadrangle, Georgia-South Carolina, 7.5 Minute Series (Topographic), DMA 4650 III NE – Series V845, 1965, Photorevised 1989.

(USGS 1989b) United States Geological Survey, Girard NW Quadrangle, Georgia-South Carolina, 7.5 Minute Series (Topographic), DMA 4650 III NW – Series V846, 1964, Photorevised 1989.

(USGS 1989d) United States Geological Survey, Girard Quadrangle, Georgia-South Carolina, 7.5 Minute Series (Topographic), DMA 4650 II SW – Series V845, 1964, Photorevised 1989.

(USGS 2006b) Daily Streamflow Data – Savannah River at Augusta, Website of the Unites States Geological Survey Surface Water for Georgia, http://nwis.waterdata.usgs.gov/ga/nwis/discharge?site_no=02197000&agency_cd=USGS&begin_date=1925-01-01&end_date=2003-09-30&format=gif&set_logscale_y=1&date_format=YYYY-MM-DD&rdb_compression=file&survey_email_address=&submitted_form=brief_list, accessed March 29, 2006.

2.4.10 Flood Protection Requirements

The maximum design basis flood elevation, including wind setup and wave run-up, at the VEGP site is El. 178.10 ft msl, as discussed in Section 2.4.4. This elevation is well below the VEGP site grade at El. 220.0 ft msl. Entrances and openings to all safety-related structures for the proposed VEGP Units 3 and 4 will be located at or above the site grade. Since the site grade is well above the maximum design basis flood elevation, the possibility is precluded of flooding VEGP Units 3 and 4 safety-related structures, systems, and components.

The effects of intense local precipitation on the safety-related structures, systems, and components of VEGP Units 3 and 4 will be considered in the design of site drainage facilities. The VEGP Units 3 and 4 site is on locally high ground, and natural drainage flow-paths slope away from the site, as shown in Figure 2.4.1-3. Thus, the topography of the proposed site facilitates drainage of intense rainfall events. Drainage facilities for the VEGP Units 3 and 4 site will be designed so that the peak discharge from the local probable maximum precipitation (PMP) do not produce flood elevations that could cause a flooding hazard to any safety-related structure, system, or component at the VEGP Units 3 and 4 site. The design will also assume that all drainage structures (e.g., culverts, storm drains, and bridges) are blocked during the PMP event. The safety-related structures, systems, and components would still be safe from resulting flood hazards.

Additionally, the design of the drainage facilities and the development of construction and operation plans will incorporate measures to ensure that existing VEGP Units 1 and 2 safety-related facilities are not subject to flooding during construction and operation of VEGP Units 3 and 4. Drainage from the VEGP Units 3 and 4 site during construction and operation of the new VEGP units will be directed away from the existing drainage facilities of VEGP Units 1 and 2. Hence, drainage from the VEGP Units 3 and 4 site will not interfere with the safety-related structures, systems, and components of VEGP Units 1 and 2.

The roofs of all safety-related structures will be designed to prevent flooding of, or leakage into, safety-related structures, systems, and components as a result of PMP on the roofs. The design basis combination of a 100-year return period ground-level snowpack and 48-hour probable maximum winter precipitation, as applied to safety-related roofs, is discussed in Section 2.3.1.3.4. Application of these two climate-related components of design basis snow load will be described in the COL Application.

Although the river intake will not be a safety-related facility, rip-rap protection of embankment slopes will be provided at the river intake location on the west bank of the Savannah River to prevent intake canal bank erosion.

Applicable NRC, federal, state, and local stormwater management regulations will be followed in the design of the drainage facilities.

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2.4.11 Low Water Considerations

This section identifies the natural events that may reduce or limit the available cooling water supply and demonstrates that an adequate water supply will exist to operate or shut down the plant under normal operations, anticipated operations, and emergency conditions.

2.4.11.1 Low Flow in Streams

VEGP Units 3 and 4 will be Westinghouse AP1000 reactors that do not require a conventional ultimate heat sink to provide safety-related cooling during emergency shutdown. Consequently, river water will not be necessary to achieve safe shutdown of the units. The only use of water from the Savannah River for the reactor units will be for the circulating water system/turbine plant cooling water system makeup, where river water will be required to replace evaporative water losses, drift losses, and blowdown discharge.

2.4.11.1.1 Observed Low Flow Data

The Savannah River flow near the VEGP site is regulated by the operation of three large federal multipurpose projects located upstream: Hartwell Dam, Richard B. Russell Dam (Russell Dam), and J. Strom Thurmond (also known as Clarks Hill) Dam. The operation of the dams during low flow periods is controlled by the drought contingency plan for the Savannah River basin **(USACE 1989)**. The contingency plan was developed in 1989 during one of the most severe droughts in the region in recent history. The objectives **(USACE 1989)** of the plan are to:

- Maintain reservoir levels at or above the bottom of the conservation pools for the three reservoirs
- Maintain a minimum release no less than 3,600 cfs at J. Strom Thurmond Dam (Thurmond Dam) for downstream use
- Use most of the available storage in the reservoirs during the drought-of-record while maintaining reservoir levels above the bottom of the conservation pools as a contingency against a drought that exceeds the drought-of-record
- Maintain project capacity throughout the drought
- Maintain releases required to meet state water quality standards from J. Strom Thurmond Dam for as long as possible without jeopardizing water supplies
- Minimize impact to recreation during the recreational season, from the first of May through Labor Day

Depending on the pool elevations at Hartwell and Thurmond reservoirs, four levels of actions are defined in the drought contingency plan, as summarized in Table 2.4.11-1. Actions for Level 3, which corresponds to the severe drought of 1988–89 (drought-of-record), will maintain a minimum of 3,600 cfs of water released through Thurmond Dam. Thurmond Dam Level 4 actions require maintaining the minimum flow of 3,600 cfs for as long as possible and, thereafter, allow the same outflow as the reservoir inflow. Consequently, the drought contingency plan for the Savannah River basin will impact water availability at the VEGP site during low flow periods.

Low water conditions in the Savannah River in the vicinity of the VEGP site are analyzed using flow records at three USGS stream gage stations. These are USGS Station No. 02197000 at Augusta, Georgia, at River Mile 187.4; 02197320 at Jackson, South Carolina, at River Mile 156.8; and 02197500 at Burtons Ferry near Milhaven, Georgia, at River Mile 118.7. The VEGP site, located at River Mile 150.9, is nearest to the Jackson gage and nearly halfway between the gages at Augusta and Burtons Ferry.

Daily-mean stream flow data are available at these three stations from the USGS Web site **(USGS 2006g)**. USGS maintains stream flow records covering a water year, which starts on October 1 of the preceding year and ends on September 30 of the current year. The longest daily-mean flow record is available at Augusta, with a period of record from the water years 1884–1891, 1896–1906, and 1925–2003. At Burtons Ferry, the flow period of record is available between the water years 1940 and 2003, with missing data periods from 1971 to 1982. The Jackson gage presents the shortest period of record of daily stream flow data, with data available between the water years 1972 and 2002. Data from the Jackson gage also include numerous periods of missing flow values. However, these periods with missing data are generally during peak flow discharges with the low flow data remained mostly unaffected.

Streamflow gage and water level measurement data are also available near the VEGP site at USGS Station No. 021973269 – Savannah River near Waynesboro at approximate River Mile 150.6. However, flow records at this gage are only available since January 2005. The short duration of the record for this gage makes it unsuitable for the calculation of low flow statistics. These data are used instead for developing a stage-discharge relationship near the site as discussed in Section 2.4.11.1.4. Details of gage locations and data availability are shown in Table 2.4.11-2.

Annual minimum daily-mean stream flow data from the three gages are shown in Figure 2.4.11-1 and Table 2.4.11-3. The data show that the annual minimum daily-mean flow within the river reach between Augusta and Burtons Ferry increased considerably after the construction of the Thurmond and Hartwell dams. The annual minimum daily-mean flow decreased during the drought-of-record (1986–1989) and has remained lower, since the implementation of the drought contingency plan in 1989, than prior to the onset of the drought. Russell Dam, the last of the three major projects, was commissioned in 1985. Because of increased catchment area downstream from Augusta, the flow at Jackson and Burtons Ferry generally is higher than the flow at Augusta. However, occasionally, the annual minimum daily-mean flow at Augusta remains higher than that at Jackson or Burtons Ferry.

Figure 2.4.11-2 shows the variation of annual minimum daily-mean flow at Jackson and Burtons Ferry corresponding to that at Augusta for the period of available data. As indicated before, the annual minimum daily-mean flow at Jackson and Burtons Ferry remains higher than that at Augusta most of the time, except a few occasions when flow at Jackson or Burtons Ferry becomes similar to or less than that at Augusta. This may indicate that although the daily-mean flow generally increases at Jackson and Burtons Ferry compared to that at Augusta because of the increase in catchment area, during certain years the additional catchment area may not contribute additional flow to the low-flow available at Augusta.

Within the period from 1985 to 2003, after the completion of Richard B. Russell Dam and representing present-day river regulation, the lowest daily-mean flow at Augusta was observed as 3,460 cfs on May 16, 1996; at Jackson it was 3,960 cfs on September 13, 2002; and at Burtons Ferry a minimum flow of 3,920 cfs was observed on September 14, 2002 (Table 2.4.11-3). The low flow measured at Augusta is also the lowest observed after the completion of all three dams within the river reach that includes the VEGP site. This data period of record also includes two of the most severe droughts in recent history in the region, 1986–1989 **(USACE 1989)** and 1998–2003 **(USACE 2006c; USGS 2006h)**.

American National Standard ANSI/ANS-2.13-1979, *Evaluation of Surface-Water Supplies for Nuclear Power Sites* **(ANSI/ANS-2.13 1979)**, recommends that for ungaged sites that have gage stations located upstream and downstream, the flow at the site may be estimated by interpolation between the gaged records based on catchment areas at the site and at the gage stations. An analysis was performed following the procedure of ANSI/ANS-2.13 (1979), which showed that the data from the Augusta gage would be the most suitable for the analysis of low flow statistics at the VEGP site. Consequently, only data from the Augusta gage is used to obtain the low flow statistics at the VEGP site. Also, because the low flow data at Augusta are generally lower than the low flow data at Jackson or Burtons Ferry, it is more conservative to use the Augusta gage data to calculate low flow statistics at the VEGP site.

2.4.11.1.2 Low Flow Statistics

Analyses for low flow statistics were performed based on historical flow data at Augusta for daily-mean annual minimum flow conditions. Because of the regulation of the Savannah River due to the construction of the dams, the complete flow record at Augusta could not be used for the analyses. Instead, flow statistics were computed within discrete segments of homogenous data periods of record. Historical annual minimum daily-mean flow data from the water years 1884 to 1952 were first analyzed using six different probability density functions: normal, log-normal, exponential, generalized extreme value – type 1 (Gumbel), Pearson – type 3 (P3), and log-Pearson – type 3 (LP3) distributions. The parameters for the distributions were estimated using the method of moments. Goodness-of-fit of the distributions was evaluated using standard χ^2 – and Kolmogorov-Smirnov tests. A distribution is considered acceptable when the test value is lower than a standard test value for a certain confidence interval. Also, further consideration is given to distributions with a smaller standard error and that fit the observed data near the desired return period.

The results of the analyses are summarized in Table 2.4.11-4. It shows that five distributions—normal, log-normal, Gumbel, P3, and LP3—are acceptable when both goodness-of-fit tests are considered for 95 percent confidence interval. Considering the goodness-of-fit, standard error magnitude, and comparison with observed data, the LP3 distribution was found to be the most suitable. The LP3 distribution with data from 1884 to 1952 is presented in Figure 2.4.11-3. Weibull plotting position formula was used for observed data, and the frequencies of the distributions were modified to reflect low flow conditions following the methodology proposed by Riggs (1972). LP3 distribution was then used to obtain flow statistics for annual minimum daily-mean flow values for the water years 1985–2003, the period representative of present-day river regulation. A similar goodness-of-fit analysis with annual minimum daily-mean flow data for water years 1985 to 2003 also showed a best fit for the LP3 distribution with observed data.

Figure 2.4.11-4 shows the LP3 distribution of the data for the water years 1953–2003. This period of record corresponds to the first regulation of the Savannah River by J. Strom Thurmond Dam. However, additional regulation of the river was added in 1965 and 1985 when Hartwell Dam and Richard B. Russell Dam, respectively—the last two of the three major projects—were constructed. The effect of this additional river regulation can be observed in the figure with a reduced fit of the distribution with observed data. The distribution is also found to be unacceptable according to the χ^2 goodness-of-fit test (Table 2.4.11-5).

Table 2.4.11-5 also shows the summary of low flow statistics for water years 1985–2003 for annual minimum daily-mean flow at Augusta. Although the period of record for this data is small, it represents the present-day full regulation of the river flow and shows acceptable goodness-of-fit for annual minimum daily-mean flows. The low flow volume thus estimated for a 100-year return period is 3,298 cfs, as shown in Table 2.4.11-5. A 7-day average 10 year return period minimum discharge (7Q10) of 3,829 cfs was calculated for the flow at Augusta, as shown in Table 2.4.11-5.

The corresponding low flow for a 100-year return period at Jackson (3,746 cfs) is also presented in Table 2.4.11-5 to facilitate a comparison. Figure 2.4.11-5 is a plot of the low flow frequency curve derived using the minimum daily-mean flow data observed at the Augusta gage for the period of 1985-2003. A similar frequency curve for the Jackson gage is presented in Figure 2.4.11-6.

2.4.11.1.3 Probable Minimum Flow

Because the river water will not be used for any safety-related activities for VEGP Units 3 and 4, probable minimum flow at the VEGP site has not been determined.

2.4.11.1.4 River Water Level for the 100-year Drought Condition

The flow rate for a 100-year drought event is estimated as 3,298 cfs in Table 2.4.11-5. The river stage corresponding to this flow rate was estimated from the stage-discharge relationship developed at USGS stream gage station 021973269 at Waynesboro, Georgia on the Savannah River near the VEGP site. Details of the stream measurements at this gaging location are presented in Table 2.4.11-2.

Streamflow measurements by the USGS at this gage were established very recently, and only eight records of measured data are available from the USGS Web site **(USGS 2006j)**. Details of these flow measurements and corresponding river stages are shown in Table 2.4.11-6. The data show five measurement events in 2005 and one each in 1986, 1987, and 1988. Flow measurements in 2005 were performed using an acoustic Doppler current profiler (ADCP). Measurements in the previous years were performed using current meters from boats.

The gage datum at this station is given on the USGS Web site as El. 90 ft above sea level NGVD29, which is equivalent to El. 90 ft msl. Using this datum, the converted water surface elevation for the measurements in 1988, 1987, and 1986 becomes close to El. 170 ft msl, which clearly is not correct. Based on the stage-discharge relationship presented in a VEGP Unit 1 and 2 analysis, it is assumed that these levels, which are shown as gage heights on the USGS Web site (also in Table 2.4.11-6), likely represent the river stage in feet msl after datum conversion.

Uncertainties also remain with the gage datum in converting the measured water surface gage heights from 2005, where the water levels become too high after conversion; for example, a flow of 8,120 cfs show a river stage of over El. 100 ft msl. This uncertainty in defining the gage datum for the Waynesboro gage was also identified at the site, where a gage datum of 70.75 ft msl was established based on a discussion with USGS and onsite geodetic marker of Georgia Power Company (GPC). Accordingly, a gage datum of 70.75 ft msl is used in this analysis.

The stage-discharge rating relationship at the site was developed using the measured flow discharges and river stages, as shown in Figure 2.4.11-7. The following approach was used to develop the rating relationship. First, the measured water levels for the years 1988, 1987, and 1986 were assumed to be the river stages in feet msl. Second, using data from all the measurement points, a best fit of the rating relation was investigated. A river stage corresponding to a no flow condition in the river at the station (H_0) was assumed, and all river stage data were converted to H-H_0 values. H-H_0 was then plotted against corresponding measured streamflow values. Last, an optimization of the best-fit rating relation was performed by modifying the assumed H_0 to maximize the root-mean-square value (R^2) of the best-fit equation. The final estimated relationship is shown in Figure 2.4.11-7. The optimization provided a zero flow level (H_0) of El. 67.56 ft msl, and an R^2 value of nearly 100 percent. The H_0 magnitude of El. 67.56 ft msl also lies within the range of river bottom elevations measured near the VEGP Units 3 and 4 river intake location during a bathymetric survey conducted in January 2006, as shown in Figure 2.4.11-8.

Using the stage-discharge relationship developed in Figure 2.4.11-7, a river stage of El. 76.26 ft msl was estimated at the VEGP site for the drought event with 100-year return period (3,298 cfs).

2.4.11.2 Low Water Resulting from Surges, Seiches, Tsunamis, or Ice Effects

Since the VEGP site is not located on a large body of water or in a coastal region, low water conditions resulting from storm surges, seiches, or tsunamis do not apply. Since there is no evidence of ice jam events near the VEGP site (see Section 2.4.7), low water conditions due to ice effects are also precluded. There are no dams downstream from the VEGP site; therefore, downstream dam failure is not a factor that could cause low flow condition at the site. Furthermore, no VEGP Unit 3 and 4 safety-related facilities will be dependent on water supply from the Savannah River.

2.4.11.3 Historical Low Water

Table 2.4.11-3 shows the annual minimum daily-mean flow recorded at the three USGS stations: Augusta, Jackson, and Burtons Ferry. Within the period of data availability, the lowest recorded daily-mean flow at Augusta was 1,040 cfs on October 2, 1927. At Jackson the record lowest flow of 3,220 cfs was observed on December 9, 1981, and at Burtons Ferry it was 2,120 cfs on September 9, 1951. The lowest flow on record at Augusta and Burtons Ferry occurred prior to construction of the dams on the Savannah River. However, because of the short length of flow records, the lowest flow at Jackson occurred after the J. Strom Thurmond and Hartwell dams were completed. The corresponding low flow at Augusta was 2,810 cfs, observed on December 7, 1981. Burtons Ferry data for this water year are not available.

Low water conditions in the river reach between Augusta and Burtons Ferry after completion of all three dams are discussed in Section 2.4.11.1.1. Since construction of the dams, the lowest flow measurement of 3,460 cfs was observed at Augusta on May 16, 1996. The corresponding flow at Jackson and Burtons Ferry, however, was considerably higher, with 5,730 cfs at Jackson on May 17, 1996, and 5,590 cfs at Burtons Ferry on May 18, 1996.

The lowest ever-recorded instantaneous flow at Augusta was 648 cfs on September 24, 1939, which was caused by the operation of the gates at the New Savannah Bluff Lock and Dam. The low flow stage-discharge rating curve at the Augusta gage was established based on the lowest measured flow magnitude of 1,400 cfs. The instantaneous low flow magnitude in 1939 was estimated by extrapolating the stage-discharge relationship at the gage station below the lowest measured discharge value of 1,400 cfs. The daily-mean flow for that day, however, was higher, at 2,940 cfs.

2.4.11.4 Future Controls

Present consumptive use of water from the Savannah River includes public supply, industrial and commercial use, power generation, and irrigation. A compilation of water use data for Georgia indicates that surface water use within the state remained nearly unchanged between 1980 and 2000 **(Fanning 2003)**. For South Carolina, while surface water use between 1990 and 2000 remained nearly the same, an increase of approximately 50 percent in surface water use is projected for the year 2045 **(SC DNR 2004)**. The projected increase also includes water demand for power generation.

The US Army Corps of Engineers, Savannah District, along with the states of Georgia and South Carolina, are developing an updated comprehensive water resources management plan for the Savannah River basin. As part of the comprehensive water management scenarios, a revised drought management plan is now being actively considered. Under the proposed plan and for proposed alternative (Alternative 2), flow through Thurmond Dam would be increased (from 3,600 cfs) to 3,800 cfs for a Level 3 drought **(USACE 2006c)**. This would also increase the low water flow available in the Savannah River near the VEGP site. The proposed drought triggers for this alternative are shown in Table 2.4.11-7.

2.4.11.5 Plant Requirements

VEGP Units 3 and 4 will be Westinghouse AP1000 reactor designs with a closed-cycle wet cooling system for condenser heat rejection. The only use of water from the Savannah River for the reactor units will be for the circulating water system/turbine plant cooling water system makeup, where river water will be required to replace evaporative water losses, drift losses, and blowdown discharge. Under normal operating conditions and design ambient conditions, river water demand for two-unit operation will be 82.9 cfs (37,212 gpm). The maximum water requirement for plant operation will be 128.7 cfs (57,784 gpm).

2.4.11.6 Heat Sink Dependability Requirements

The AP1000 reactor plants selected for VEGP Units 3 and 4 do not require a conventional ultimate heat sink to provide safety-related cooling during emergency shutdown. The AP1000 reactors make use of a passive cooling system and use water stored in onsite tanks. Consequently, river water will not be necessary to achieve safe shutdown of the units.

Table 2.4.11-1 Summary of Action Levels for Drought Management in the Savannah River Basin

Level	Reservoir Pool Levels				Action
	Hartwell Dam		J.S. Thurmond Dam[a]		
	Apr 18 – Oct 15 ft msl[b]	Dec 1 – Jan 1 ft msl	May 1 – Oct 15 ft msl	Dec 15 – Jan 1 ft msl	
1	656	655	326	325	Public Safety Information
2	654	652	324	322	Reduce Thurmond discharge to 4,500 cfs; reduce Hartwell discharge as appropriate to maintain balanced pool
3	646	646	316	316	Reduce Thurmond discharge to 3,600 cfs; reduce Hartwell discharge as appropriate to maintain balanced pool
4	625	625	312	312	Continue Level 3 discharge as long as possible; thereafter Inflow = Outflow

[a] J. Strom Thurmond Dam

[b] mean sea level

Source: USACE 1989

Table 2.4.11-2 Locations, Catchment Areas, and Data Availability of the USGS Gage Stations

Station Name	County/Town	USGS Station ID	Location			River Mile[a]	Catch-ment Area (mi²)	Daily Streamflow Data Availability		
			Latitude	Longitude	HU[b]			Start Date	End Date	Count
Savannah River at Augusta	Richmond, GA	02197000	33°22'25"	81°56'35"	03060106	187.4	7,508	10/1/1883	9/30/2003	35,793
Savannah River near Jackson	Aiken, SC	02197320	33°13'01"	81°46'04"	03060106	156.8	8,110	10/1/1971	9/30/2002	10,733
Savannah River at Burtons Ferry near Millhaven	Millhaven, GA	02197500	32°56'20"	81°30'10"	03060106	118.7	8,650	10/1/1939	9/30/2003	18,993
Savannah River near Waynesboro	Burke, GA	021973269	33°08'59"	81°45'18"	03060106	150.6[c]	8,300	1/22/2005	9/30/2005	252

[a] USACE 1996

[b] Hydrological Unit

[c] Approximate River Mile

Source: USGS 2006g

Table 2.4.11-3 Variation of Annual Minimum Daily-mean Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages

	Flow (ft^3/sec) at Locations			
	Augusta	**Jackson**	**Burtons Ferry**	
River Mile	**187.7**	**156.8**	**118.7**	**Comments**
1884	2,060			
1885	1,980			
1886	3,500			
1887	2,780			
1888	3,300			
1889	4,340			
1890	2,700			
1891	4,480			
1896	2,230			
1897	1,990			
1898	2,080			
1899	2,350			
1900	3,000			
1901	3,940			
1902	3,920			
1903	3,740			
1904	2,060			
1905	1,450			
1906	2,650			
1925	1,100			
1926	1,380			
1927	1,160			
1928	1,040			Historical low flow at Augusta on Oct. 2, 1927
1929	3,580			
1930	1,970			
1931	1,420			
1932	1,230			
1933	2,280			

Table 2.4.11-3 (cont.) Variation of Annual Minimum Daily-mean Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages

	Flow (ft^3/sec) at Locations			
	Augusta	Jackson	Burtons Ferry	
River Mile	187.7	156.8	118.7	Comments
1934	1,950			
1935	2,090			
1936	1,590			
1937	2,970			
1938	1,860			
1939	1,770			
1940	1,340		2,400	
1941	1,510		2,320	
1942	1,390		2,240	
1943	2,700		3,600	
1944	2,780		3,440	
1945	2,350		3,120	
1946	2,550		3,530	
1947	1,840		2,720	
1948	1,900		3,230	
1949	2,930		4,900	
1950	2,850		4,120	
1951	1,710		2,120	Lowest flow (within available data) at Burtons Ferry on Sep. 9, 1951
1952	1,770		2,550	J. Strom Thurmond Dam
1953	3,260		3,850	
1954	5,460		5,500	
1955	4,180		4,770	
1956	3,580		4,590	
1957	5,170		5,500	
1958	5,000		5,500	
1959	5,260		5,500	
1960	5,350		6,440	
1961	4,930		6,060	

Table 2.4.11-3 (cont.) Variation of Annual Minimum Daily-mean Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages

	Flow (ft³/sec) at Locations			Comments
	Augusta	Jackson	Burtons Ferry	
River Mile	187.7	156.8	118.7	
1962	4,760		5,700	
1963	5,130		6,260	
1964	6,120		6,900	
1965	6,300		7,600	Hartwell Dam
1966	6,160		7,110	
1967	5,740		6,780	
1968	5,890		6,950	
1969	5,800		6,900	
1970	5,870		6,710	
1971	4,460			
1972	6,220	6,330		
1973	5,460	6,390		
1974	5,450	6,330		
1975	5,830	6,760		
1976	6,750	6,770		
1977	6,000	6,420		
1978	6,110	5,800		
1979	5,940	5,770		
1980	5,970	5,930		
1981	5,120	5,190		
1982	2,810	3,220		Lowest flow (within available data) at Jackson on Dec. 9, 1981
1983	5,080	5,050	5,870	
1984	4,740	4,900	5,210	
1985	4,750	4,760	4,830	Richard B. Russell Dam
1986	4,590	4,760	4,390	
1987	3,790	4,120	3,960	
1988	3,880	4,150	4,000	
1989	3,800	4,360	4,100	

Table 2.4.11-3 (cont.) Variation of Annual Minimum Daily-mean Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages

	Flow (ft^3/sec) at Locations			Comments
	Augusta	Jackson	Burtons Ferry	
River Mile	187.7	156.8	118.7	
1990	4,010	4,880	4,730	
1991	4,310	4,640	4,330	
1992	4,000	4,610	4,620	
1993	4,560	5,620	5,320	
1994	4,200	5,160	4,930	
1995	5,110	5,590	5,410	
1996	3,460	5,730	5,360	After 1985, lowest flow at Augusta on May 16, 1996
1997	4,230	4,790	4,480	
1998	4,300	5,310	5,370	
1999	3,800	4,710	4,490	
2000	3,880	4,300	4,160	
2001	3,670	4,380	4,550	
2002	3,730	3,960	3,920	After 1985, lowest flow at Jackson on Sep. 13, 2002; at Burtons Ferry on Sep. 14, 2002
2003	3,470		4,360	
Record Low Flow	1,040	3,220	2,120	
Low Flow between 1983–2002	3,460	3,960	3,920	Period of common data availability
Low Flow after 1985	3,460	3,960	3,920	Period after the completion of three major dams (present-day regulation of the Savannah River)

Source: USGS 2006g

Table 2.4.11-4 Summary of Statistical Parameters for Different Probability Density Functions Calculated with Annual Minimum Daily-mean Streamflow Values at Augusta for the Water Years 1884–1952

Distribution	Mean	SD[a]	Cs[b]	Goodness-of-Fit (95% confidence level)				Comments
				Standard Test Value		Present set of Data		
				χ^2	K-S[c]	χ^2	K-S	
Normal	2331.1	881.64	0.713	21.92	0.159	11.5	0.115	Acceptable
Exponential	2331.1	881.64	0.713			23.7	0.129	Not acceptable
Gumbel[d]	2331.1	881.64	0.713			6.9	0.046	Acceptable
P3[e]	2331.1	881.64	0.713			6.4	0.044	Acceptable
Log-Normal	7.7	0.37	0.011			11.0	0.050	Acceptable
LP3[f]	7.7	0.38	0.011			7.4	0.046	Acceptable, selected

a Standard Deviation

b Coefficient of Skewness

c Kolmogorov-Smirnov

d Extreme Value Type I

e Pearson Type 3

f Log-Pearson Type 3

Table 2.4.11-5 Summary of Low Flow Statistics for Log-Pearson Type 3 Distribution with Annual Minimum Daily-mean and 7-Day Moving-average Streamflow Values at Augusta and Jackson for Different Water Years

Gage Station	Water Years	Data Type	Mean Ln (cfs)	SD[a]	Cs[b]	Goodness-of-Fit[c]		Low Flow Magnitudes (cfs) for Return Periods (years)				
						χ^2	K-S[d]	5	10	20	50	100
Augusta	1953-2003	Daily-mean	8.47	0.21	-0.38	23.6	0.093	3,985	3,684	3,465	3,246	3,115
	1985-2003	Daily-mean	8.31	0.11	0.49	6.9	0.079	3,708	3,569	3,466	3,361	3,298
	1985-2003	7-Day Moving-average	8.40	0.12	0.17	11.9	0.149	4,018	3,829	3,682	3,528	3,430
Jackson	1985-2002	Daily-mean	8.46	0.11	0.26	8.7	0.083	4,316	4,130	3,988	3,839	3,746
	1985-2002	7-Day Moving-average	8.52	0.14	0.27	10.0	0.083	4,478	4,238	4,056	3,868	3,752

[a] Standard deviation

[b] Coefficient of Skewness

[c] For 95% confidence limit, standard χ^2 test value is 21.92; for Kolmogorov-Smirnov tests the standard values are 0.154 for water years 1953-2003, 0.231 for 1985-2003, and 0.236 for 1985-2002

[d] Kolmogorov-Smirnov

Table 2.4.11-6 Summary of Streamflow Measurement at USGS Station No. 021973269 Savannah River Near Waynesboro

Measurement No.	Date	Width ft	Area ft^2	Mean Velocity fps	Gage Height ft	Streamflow cfs	Measurement Type
8	10/14/2005	359	2740	1.89	7.81	5,180	ADCP
7	5/18/2005	369	4000	2.03	10.56	8,120	ADCP
6	3/31/2005	423	6740	3.22	19.28	21,700	ADCP
5	3/17/2005	371	5540	2.63	14.80	14,600	ADCP
4	1/19/2005				12.03	9,840	ADCP
3	8/29/1988	333	2270	1.96	77.56	4,450	Boat
2	2/4/1987	310	3300	2.32	80.60	7,640	Boat
1	9/24/1986	300	2300	1.98	77.84	4,570	Boat

Note: A detailed discussion on gage heights for different years is included in Section 2.4.11.1.4

Source: USGS 2006j

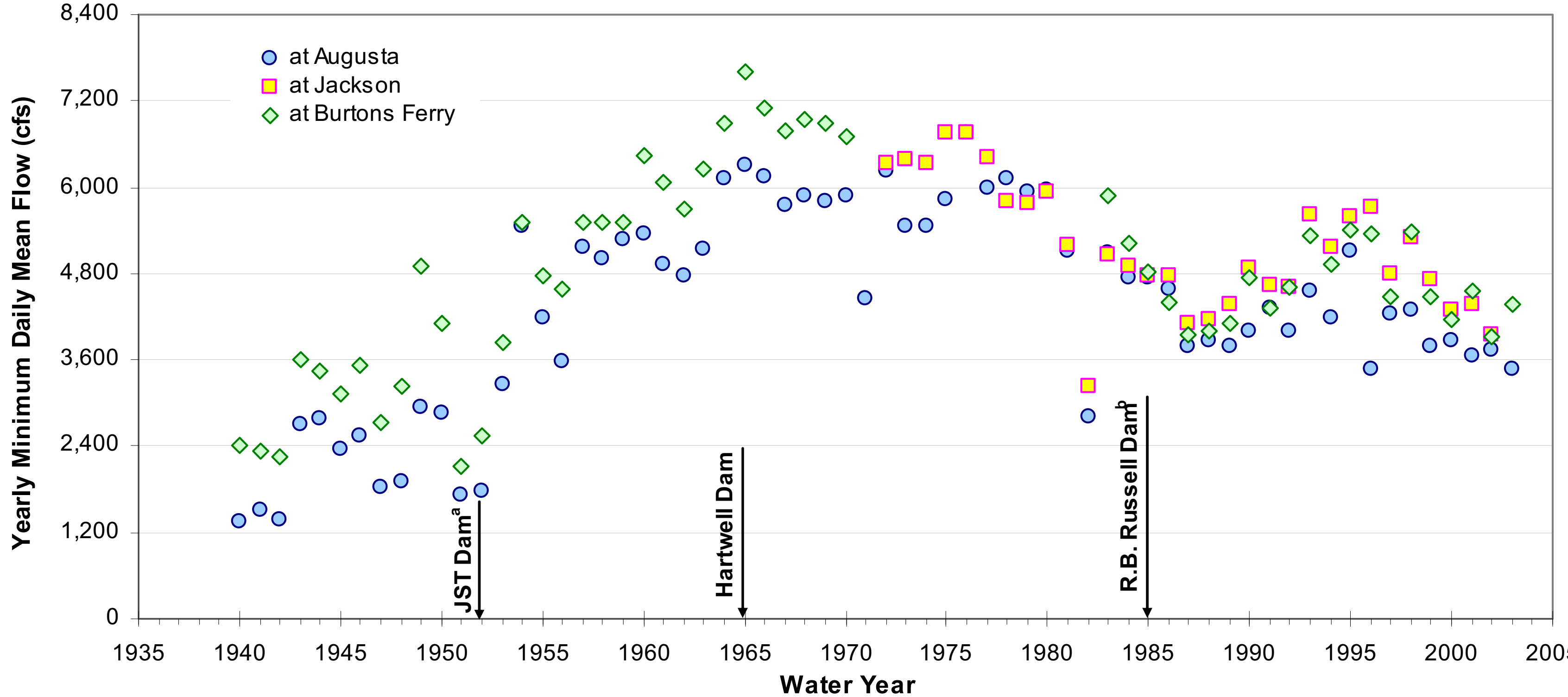
Table 2.4.11-7 Summary of Proposed Modifications in Action Levels for Drought Management in the Savannah River Basin

Level	Reservoir Pool Levels				Action
	Hartwell Dam		J.S. Thurmond Dam[a]		
	Apr 1 – Oct 15 ft msl[b]	Dec 15 – Jan 1 ft msl	Apr 1 – Oct 15 ft msl	Dec 15 – Jan 1 ft msl	
1	656	654	326	324	Reduce Thurmond discharge to 4,200 ft^3/sec
2	654	652	324	322	Reduce Thurmond discharge to 4,000 ft^3/sec
3	646	646	316	316	Reduce Thurmond discharge to 3,800 ft^3/sec
4	625	625	312	312	Inflow = Outflow

[a] J. Strom Thurmond reservoir

[b] mean sea level

Source: USACE 2006c

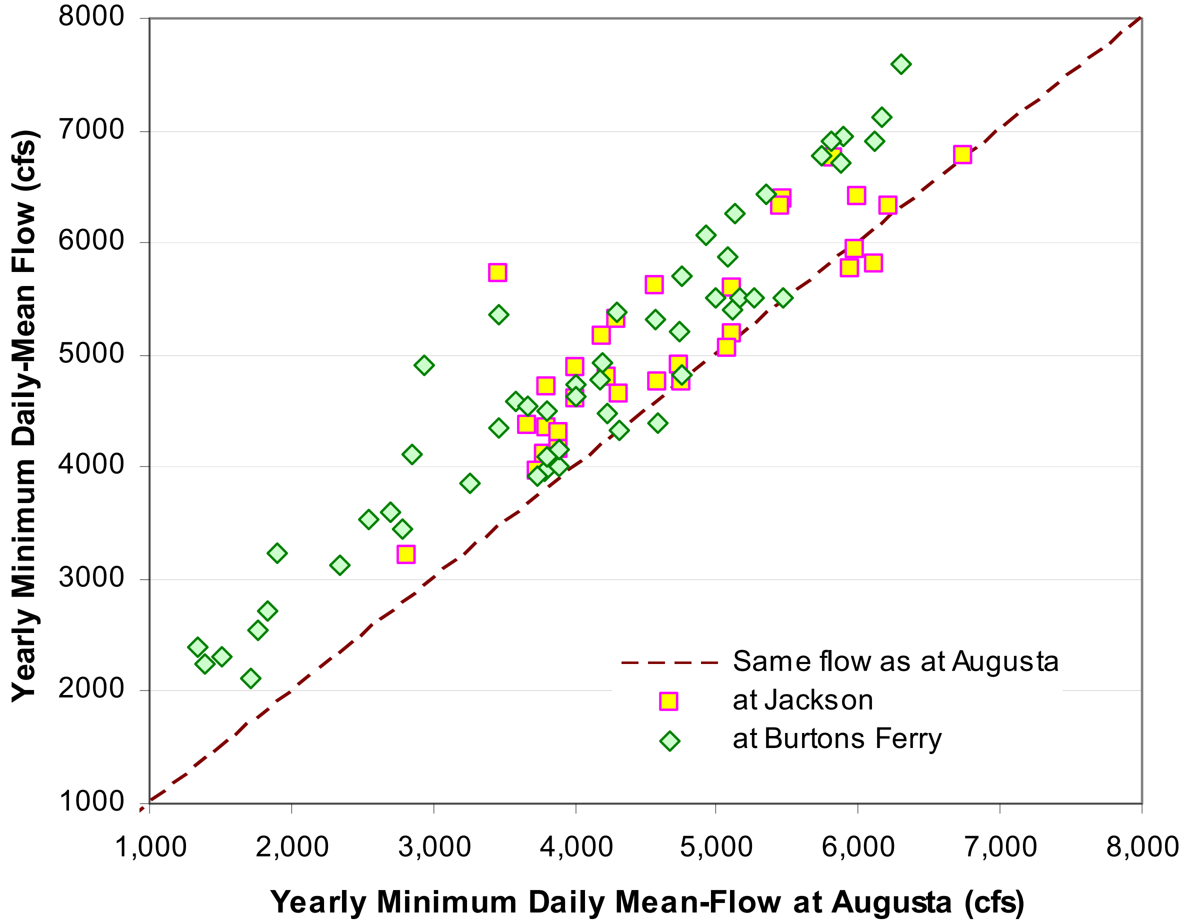


[a] J. Strom Thurmond Dam

[b] Richard B. Russell Dam

Source: USGS 2006g

Figure 2.4.11-1 Variation in Annual Minimum Daily-mean Stream Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages



Source: USGS 2006g

Figure 2.4.11-2 Change in Annual Minimum Daily-mean Flow at Jackson and Burtons Ferry Corresponding to that at Augusta for the Period of 1940-2003

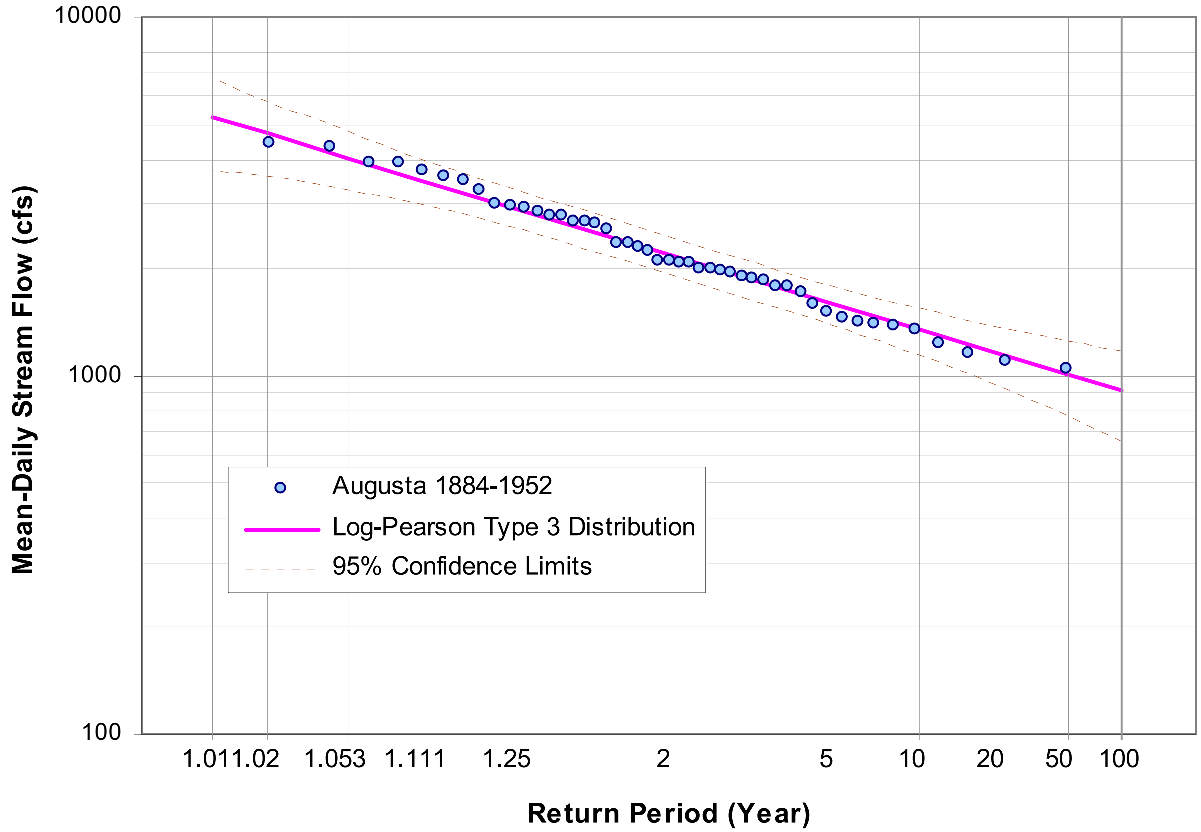


Figure 2.4.11-3 Log-Pearson Type 3 Distribution with Annual Minimum Daily-mean Flow Data from Augusta for the Water Years 1884–1952

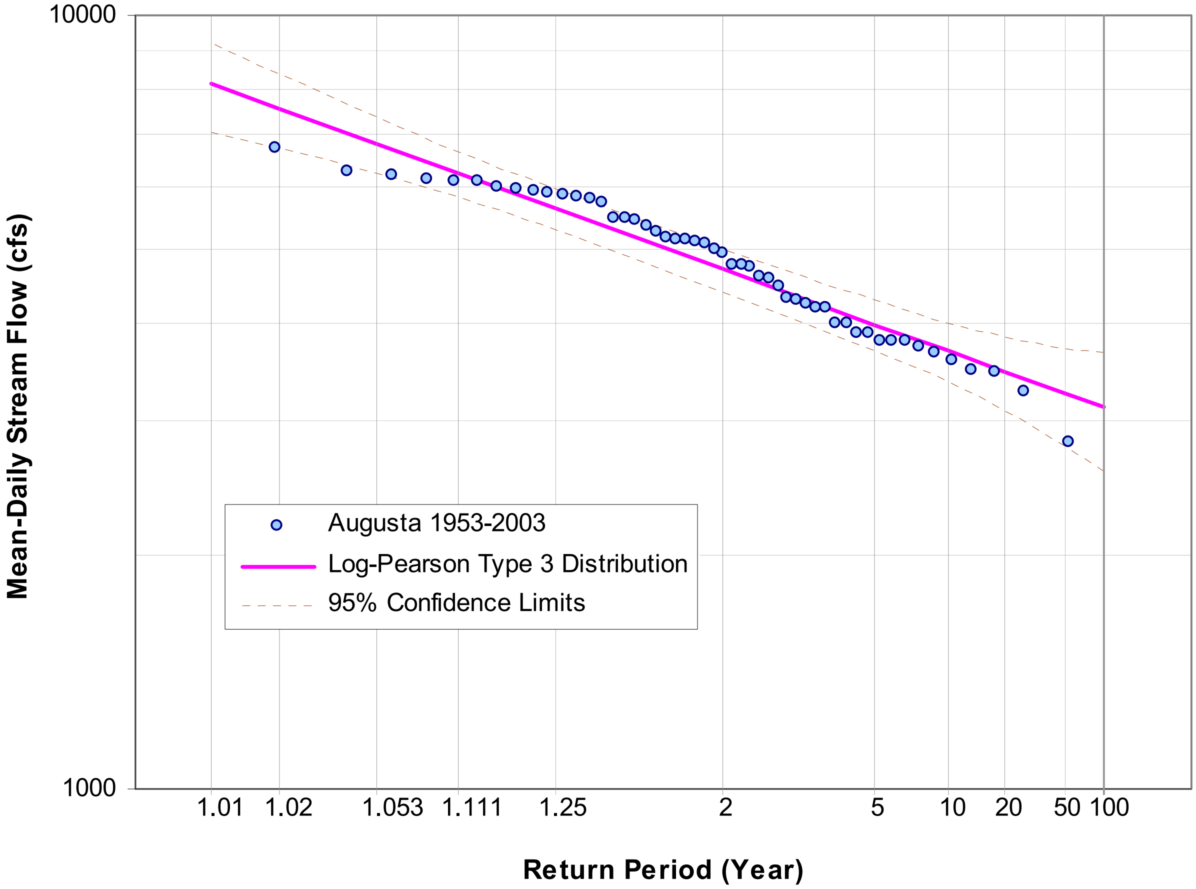


Figure 2.4.11-4 Log-Pearson Type 3 Distribution with Annual Minimum Daily-mean Flow Data from Augusta for the Water Years 1953–2003

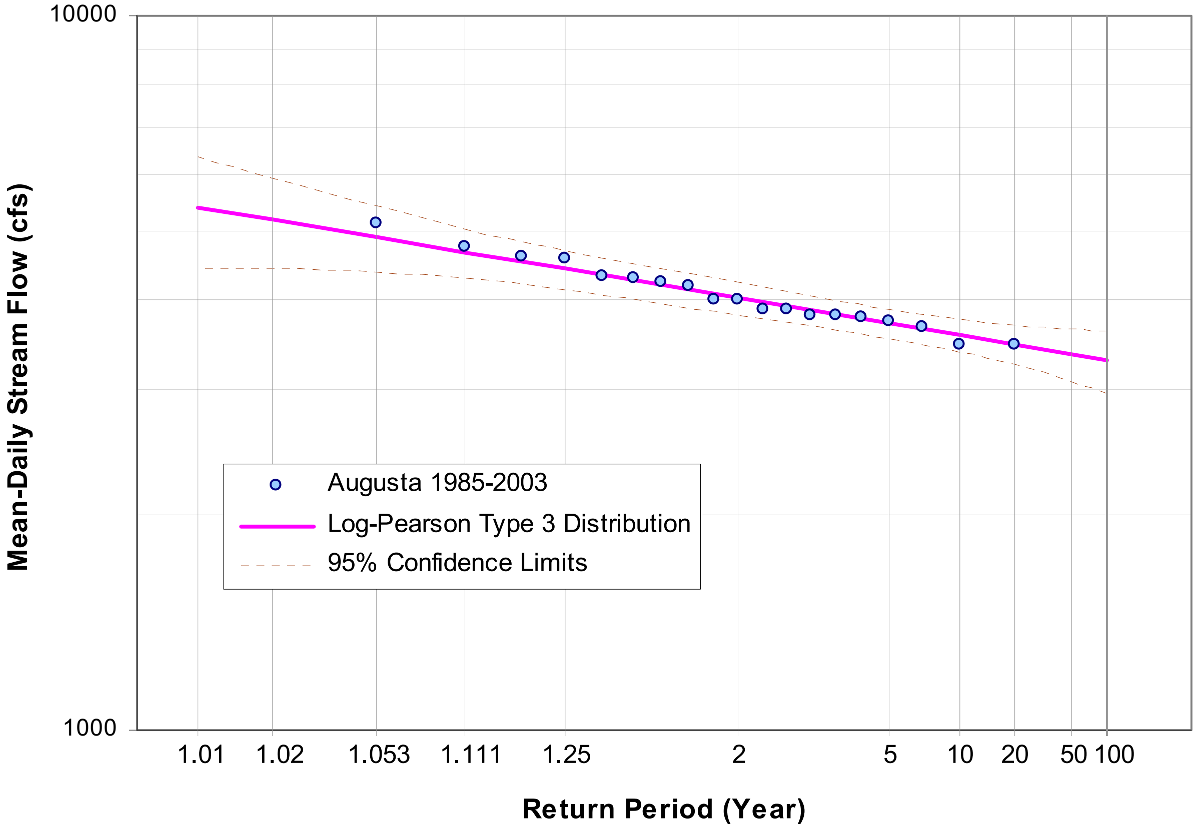


Figure 2.4.11-5 Log-Pearson Type 3 Distribution with Annual Minimum Daily-mean Flow Data from Augusta for the Water Years 1985–2003

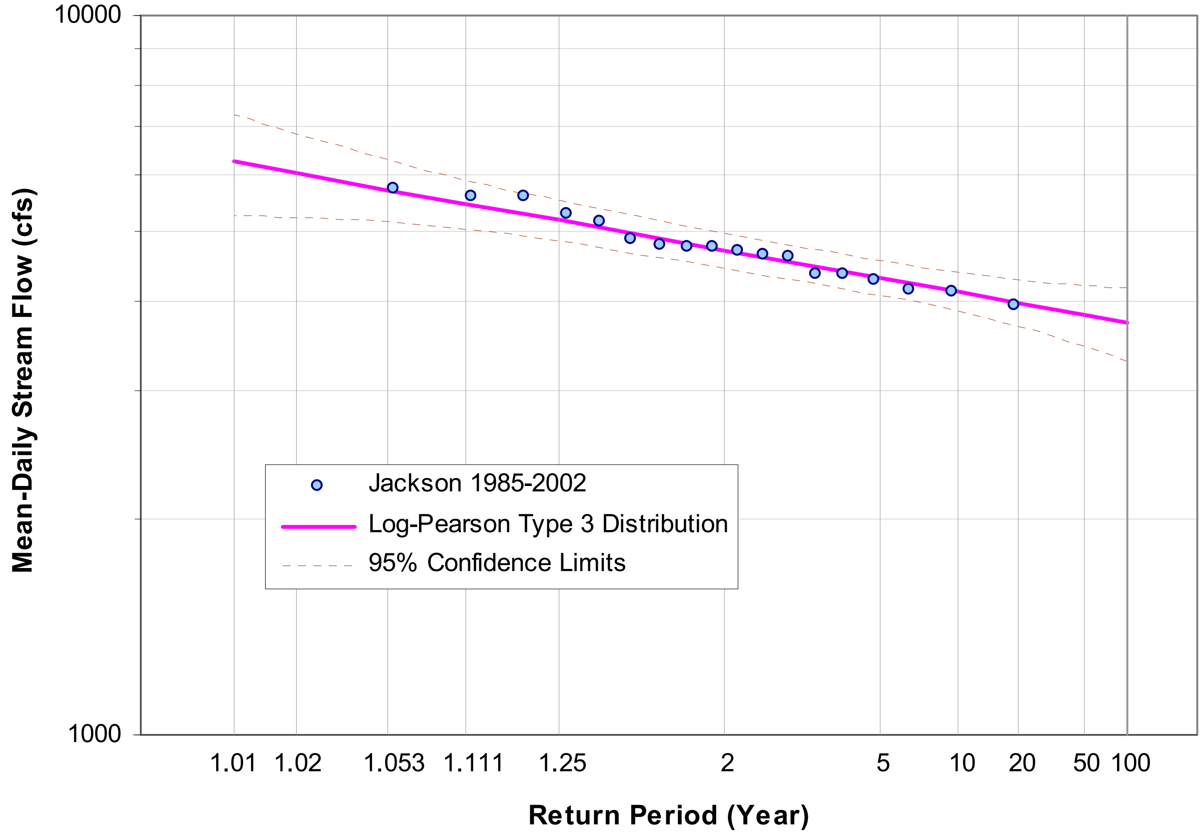
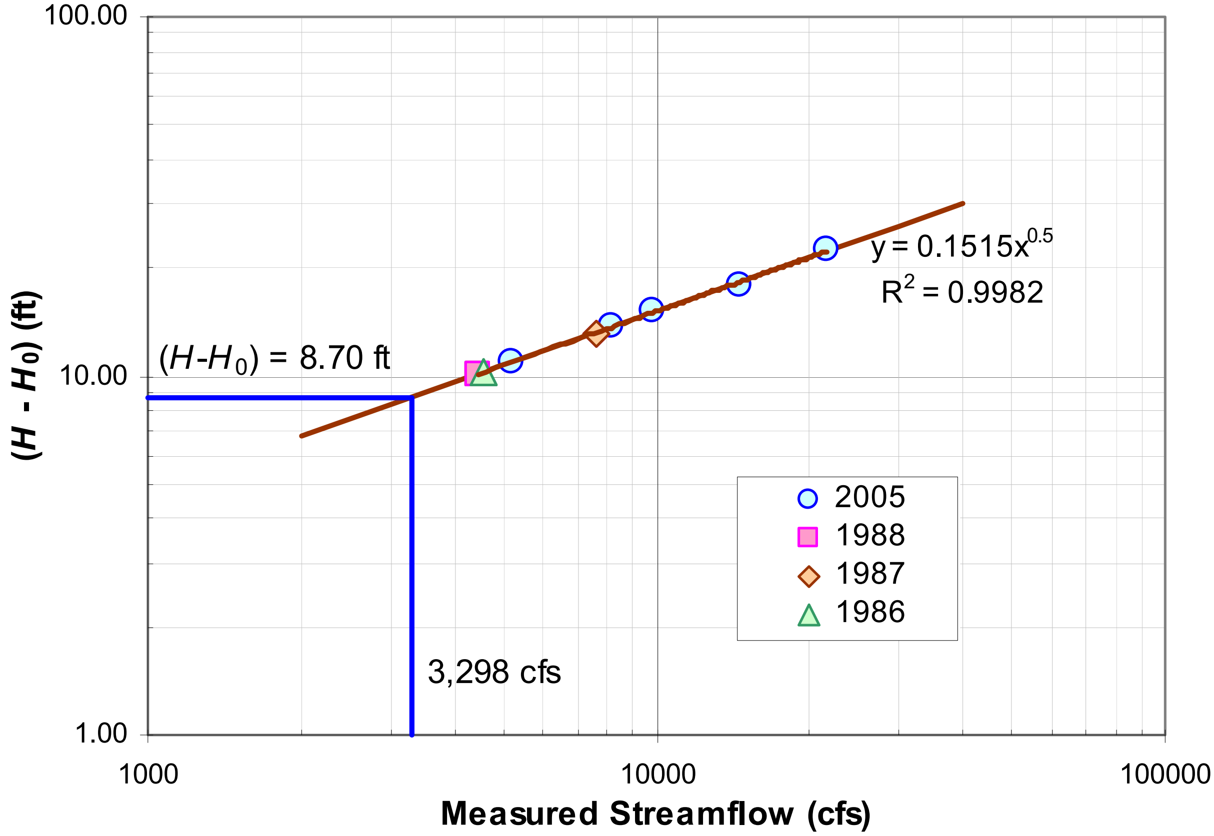


Figure 2.4.11-6 Log-Pearson Type 3 Distribution with Annual Minimum Daily-mean Streamflow from Jackson for the Water Years 1985–2002



H = Water surface elevation in El. ft msl

H_0 = Elevation corresponding to zero flow = El. 67.56 ft msl

Figure 2.4.11-7 River Stage-Discharge Rating Relationship at USGS Waynesboro Gage Station Near the VEGP Site Using Data for the Years 2005, 1988, 1987 and 1986

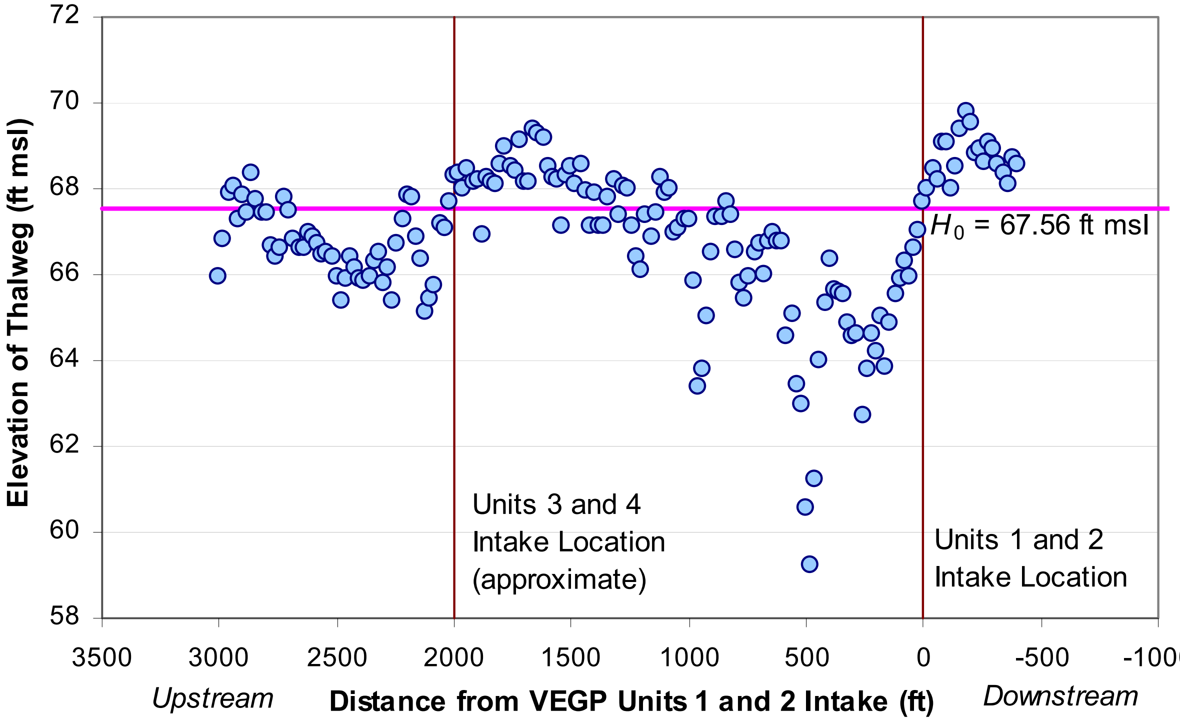


Figure 2.4.11-8 Comparison of Estimated River Stage Corresponding to Zero Discharge (H_0) with Measured River Thalweg Levels Near the Intake Location

Section 2.4.11 References

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(USACE 1989) U.S. Army Corps of Engineers, Savannah District, *Savannah River Basin PDrought Management Plan*, March 1989.

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(USACE 2006c) U.S. Army Corps of Engineers, Savannah District, *Draft Environmental Assessment and Finding of No Significant Impact Drought Contingency Plan Update, Savannah River Basin*, May 2006.

(USGS 2006g) *Daily Stream Flow for Georgia*, U.S. Geological Survey, Web site: http://nwis.waterdata.usgs.gov/ga/nwis/discharge?search_criteria=county_cd&search_criteria=search_station_nm&submitted_form=introduction, accessed April 24, 2006.

(USGS 2006h) *Summary of Hydrologic Condition in Georgia*, U.S. Geological Survey, Web site: http://ga.water.usgs.gov/news/drought99/hydrsumm.html, accessed April 24, 2006.

(USGS 2006j) *Streamflow Measurements for Georgia*, Savannah River near Waynesboro, GA, U.S. Geological Survey, Web site: http://nwis.waterdata.usgs.gov/ga/nwis/measurements/?site_no=021973269&agency_cd=USGS, accessed May 12, 2006.

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2.4.12 Groundwater

This section describes the groundwater resources as it relates to the design bases for the Westinghouse AP1000 reactor design. The hydrogeology of the VEGP regional and local area including the site and the interface with the new AP1000 units are discussed in this section. Current and projected groundwater uses in the VEGP region are also discussed.

The 3,169 acre VEGP site is located on a bluff on the southwest side of the Savannah River in eastern Burke County, Georgia, within the Coastal Plain Physiographic Province (Figure 2.5.1-1). The proposed AP1000 units referred to as VEGP Units 3 and 4 will have a finished grade level elevation of approximately 220 ft msl. The bottom of the foundation slab for the safety related AP1000 containment structure will be 39.5 ft (180.5 ft msl) below grade level. The Westinghouse AP1000 reactor design has no safety-related ultimate heat sink that relies on surface water or groundwater supplies. On-site wells will provide make-up water for the service water system (SWS). The wells will also supply water for power plant systems, including the fire protection system, the plant demineralized water supply system, and the potable water system. Groundwater withdrawn for the proposed two new units will be 752 gpm on average, with a maximum of 3,140 gpm. During normal operation, approximately 305 gpm of the withdrawn groundwater is returned as surface water to the Savannah River **(Westinghouse 2005)**.

In constructing the new units, the site will be excavated approximately 80 to 140 ft below existing grade to remove the in situ soil down to the principal bearing strata, the Blue Bluff Marl. The in situ soil will be replaced with seismically-designed fill material. Foundations for the new units will be poured on this new backfill material and the fill material will be placed around the structures and continue up to the finished grade elevation of 220 ft msl. Seismic analysis of the geological formations under the proposed new units including the seismically designed backfill are discussed in Section 2.5.

2.4.12.1 Regional and Local Groundwater Aquifers and Conceptual Model Description

The following primary sources of information were used to develop the regional and local hydrogeological description and the conceptual model description presented in this section:

- Vogtle ALWR ESP Project Final Data Report, ES1374, Southern Company Services Inc., November 2005. (Appendix 2.4A)
- Data Report of Geotechnical Investigation and Laboratory Testing MACTEC Engineering and Consulting Inc., January 2005. (Appendix 2.5A)
- Groundwater Atlas of the United States, Segment 6, Alabama, Florida, Georgia, and South Carolina, U.S. Geological Survey, Hydrologic Investigations Atlas 730-G, J.A. Miller, 1990. **(Miller 1990)**

- The Lithostratigraphic Framework of the Upper Cretaceous and Lower Tertiary of Eastern Burke County, Georgia, Bulletin 127, Georgia Department of Natural Resources, Huddlestun, P.F., and J.H. Summerour, 1996. **(Huddlestun and Summerour 1996)**
- Final Safety Analysis Report for Vogtle Electric Generating Plant (VEGP) Units 1 and 2.
- An Investigation of Tritium in the Gordon and Other Aquifers in Burke County, Georgia, Phase II: Georgia Geologic Survey Information Circular 102, J.H., Summerour, E.A. Shapiro, and P.F. Huddlestun, 1998. **(Summerour et al 1998)**
- Ground-Water Levels, Predevelopment Ground-Water Flow, and Stream-Aquifer Relations in the Vicinity of Savannah River Site, Georgia and South Carolina: U.S. Geological Survey Water-Resources Investigations Report 97-4197, 1997. J.S. Clarke, and C.T. West. **(Clarke and West 1997)**
- Simulation of Ground-Water Flow and Stream-Aquifer Relations in the Vicinity of the Savannah River Site, Georgia and South Carolina: U.S Geological Survey Water-Resources Investigations Report 98-4062, 134 p. J.S. Clarke, and C.T. West, 1998. **(Clarke and West 1998)**
- Simulation and Particle-Tracking Analysis of Ground-Water Flow Near the Savannah River Site, Georgia and South Carolina, 2002, and for Selected Water-Management Scenarios, 2002 and 2020: U.S. Geological Survey Scientific Investigations Report 2006-5195, G.S. Cherry, 2006. **(Cherry 2006)**

2.4.12.1.1 Regional Hydrogeology

The region within a 200-mi radius around the VEGP site encompasses parts of four physiographic provinces. These include, from northwest to southeast, the Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain Physiographic Provinces. Figure 2.5.1-1 shows the physiographic provinces and indicates a 200-mi radius from the VEGP site. Several major aquifers or aquifer systems are present with these physiographic provinces. The VEGP site and associated groundwater are located within the Coastal Plain province **(Miller 1990)**. However, groundwater within the other provinces is discussed below to provide a complete picture of regional hydrogeologic conditions.

The Valley and Ridge Physiographic Province lies about 180 mi northwest of the VEGP site. Aquifers underlying the Valley and Ridge province occur within Paleozoic-age folded and faulted sedimentary rock. The sedimentary strata consist predominantly of sandstone, shale, and limestone, with minor amounts of dolomite, conglomerate, chert, and coal. The carbonate and sandstone layers form the principal aquifers in the province. Typical well yields are from 10 gpm in sandstone formations to 10 to 50 gpm within the limestone units. Locally high yields, equal to 100 gpm or greater, are possible within highly fractured strata or solution cavities. Localized weathered rock and alluvium can provide lesser, but adequate, groundwater yields for domestic use. **(Miller 1990)**

The Piedmont and Blue Ridge Physiographic Provinces are hydrologically similar in nature. Both provinces are composed primarily of metamorphic and igneous rocks. Surface materials in the Blue Ridge Province consist mainly of thin residual soils, alluvium and colluvium. Surface materials in the Piedmont Province consist generally of more deeply weathered residual soils (saprolite) and alluvium. Groundwater occurs both in the fractured portions of bedrock and within the saprolite and alluvium material. Well yields generally depend on the local fracture density and fracture connectivity of the bedrock and range from a few to 30 gpm. Localized groundwater well yields of 100 gpm or greater are possible. **(Miller 1990)**

The majority of Georgia's groundwater use occurs in the Coastal Plain Physiographic Province. The Coastal Plain sediments are thin, less than 200 ft thick, along the western boundary of the province (where they terminate at the contact with the Piedmont province, the Fall Line) and thicken to over 4,000 ft in an eastern-to-southeastern direction. The sediments range in age from Holocene to Cretaceous and overlie crystalline igneous and metamorphic bedrock, which is an eastward extension of the Piedmont province **(Miller 1990)**.

Groundwater in the Coastal Plain is withdrawn from both unconfined, shallow aquifer systems and deeper, confined aquifer systems. These aquifers are recharged principally in their outcrop area along the western boundary of the province near the Fall Line and from localized infiltration of precipitation within the province. Precipitation migrates downward and laterally through the unconsolidated surficial materials and discharges to nearby streams and low areas or percolates downward into the deeper unconsolidated and consolidated material. The thickness and areal extent of the Coastal Plain sediments result in higher groundwater storage than for any other physiographic provinces in Georgia **(Miller 1990)**.

Coastal Plain sediments comprise three aquifer systems consisting of seven aquifers that are separated hydraulically by confining units. As presented by **Clarke and West (1997)**, the aquifer systems are, in descending order: (1) the Floridan aquifer system, which consists of the Upper Three Runs and Gordon aquifers in sediments of Eocene age; (2) the Dublin aquifer system, consisting of the Millers Pond, upper Dublin, and lower Dublin of Paleocene-Late Cretaceous age; and (3) the Midville aquifer system, consisting of the upper Midville and lower Midville aquifers in sediments of Late Cretaceous age. It is important to note that nomenclature used by the U.S. Geological Survey **(Clarke and West 1997)** for geologic and hydrogeologic units differs from the **Huddleston and Summerour (1996)** nomenclature used in Section 2.4.12.1.2 of the ESP application to describe the local hydrogeologic units. In this ESP application, the Water Table aquifer comprises the Upper Three Runs aquifer, the Tertiary sand aquifer comprises the Gordon aquifer, and the Cretaceous aquifer comprises the Dublin and

Midville aquifers. Figure 2.4.12-1 and Figure 4 of Clarke and West (1997) provide additional details.

The Upper Three Runs aquifer is the shallowest aquifer and is unconfined to semi-confined throughout most of the area. Groundwater levels in the Upper Three Runs aquifer respond to a local flow system and are affected mostly by topography and climate. Groundwater flow in the deeper Gordon aquifer and Dublin and Midville aquifer systems is characterized by local flow to the northwest near outcrop areas, changing to intermediate flow and then regional flow downdip (southeastward) as the aquifers become more deeply buried. Water levels in these deeper aquifers show a pronounced response to topography and climate in the vicinity of outcrops that diminishes southeastward where the aquifer is more deeply buried. Stream stage and pumpage affect groundwater levels in these deeper aquifers to varying degrees throughout the area. **(Clarke and West 1997)**

The geologic characteristics of the Savannah River alluvial valley substantially control the configuration of potentiometric surfaces, groundwater flow directions, and stream-aquifer relations. Data from 18 shallow borings **(Leeth and Nagle 1996)** indicate incision into each aquifer by the paleo Savannah River, and subsequent infill by permeable alluvium has resulted in direct hydraulic connection between the aquifers and the Savannah River along various parts of its reach. This hydraulic connection may be the cause of large groundwater discharge to the river near Jackson, South Carolina, as evidenced by stream baseflow and potentiometric measurements, where the Gordon aquifer is in contact with Savannah River alluvium, and also the cause of lows or depressions in potentiometric surfaces of confined aquifers that are in contact with the alluvium. Groundwater in these aquifers flows toward the depressions. The influence of the river diminishes downstream where the aquifers become deeply buried beneath the river channel, and where upstream and downstream groundwater flow is possibly separated by a groundwater flow divide or “saddle.” Water-level data indicate that saddle features probably exist in the Gordon aquifer and Dublin aquifer system, with the groundwater divide occurring just downstream of the VEGP site, and also might be present in the Midville aquifer system. **(Clarke and West 1997)**

Basin-wide potentiometric-surface maps for the unconfined Upper Three Runs aquifer and confined Gordon, Dublin, and Midville aquifer systems have been prepared using historical data **(Clarke and West 1997)** and numerical simulation **(Cherry 2006)**. Detailed discussions of these maps are provided in the cited references. Data from observation wells installed and monitored for an 18-month period at the VEGP site have also been used to develop potentiometric-surface maps on a more highly resolved, site-specific basis. These maps are discussed in detail in Section 2.4.12.1.3. The groundwater flow directions inferred from these maps are generally consistent with the larger-scale maps produced by Clarke and West (1997) and Cherry (2006), i.e., groundwater flow in the Upper Three Runs (Water Table) aquifer generally conforms with surface topography, while that in the confined Gordon (Tertiary) aquifer is towards the Savannah River.

Recharge to the Upper Three Runs (Water Table) aquifer is almost exclusively by precipitation, while discharge is primarily to local drainages. Recharge to the confined Gordon (Tertiary) and Dublin and Midville (Cretaceous) aquifers occurs primarily by direct infiltration of rainfall in their outcrop areas northwest of the VEGP site that are generally parallel to the Fall Line. Because the permeable alluvium of the Savannah River valley allows for direct hydraulic connection between aquifers and the Savannah River, the river serves as the major discharge area for the confined aquifers in hydraulic connection with the river valley alluvium. Potentiometric maps presented by Clarke and West (1997) indicate groundwater discharge from the confined Gordon, Dublin, and Midville aquifers to the Savannah River. For the shallower Gordon confined aquifer, groundwater flow directions are generally perpendicular to the river reach. In the case of the deeper Dublin and Midville aquifers, there are upriver components to the groundwater flow directions that depend on where the paleo river channel has breached confining units. Clarke and West (1997) provide a detailed discussion of this phenomenon.

Although a water budget for the VEGP site has not been quantified, recharge and discharge rates have been estimated on a basin-wide basis by other investigators. Clarke and West (1997) estimated groundwater discharge to the Savannah River based on the net gain in stream discharge for local, intermediate, and regional groundwater flow systems and for different hydrologic conditions. Groundwater discharge ranged from 910 ft3/s during a drought year (1941), to 1,670 ft^3/s during a wet year (1949), and averaged 1,220 ft^3/s. Of the average discharge, the local flow system contributed an estimated 560 ft^3/s and the intermediate and regional flow systems contributed an estimated 660 ft^3/s. Clarke and West (1997) approximated the long-term average recharge by weighting these values according to drainage area, and estimated the average groundwater recharge in the Savannah River basin to be 14.5 inches, of which 6.8 inches is to the local flow system, 5.8 inches is to the intermediate flow system, and 1.9 inches is to the regional flow system. Mean-annual precipitation in the basin ranges from 44 to 48 inches. Cherry (2006) presents simulated water budgets for different hydrologic conditions using a numerical model for groundwater flow in Georgia and South Carolina near the Savannah River Site. The numerical model contains estimates of inflow or outflow across lateral boundaries, recharge, discharge, groundwater pumpage, and vertical flow upward and downward across confining units.

The potential for trans-river flow in the vicinity of the Savannah River Site and VEGP site has been discussed by Clarke and West (1997). Trans-river flow is a term that describes a condition under which groundwater originating on one side of a river migrates beneath the river floodplain to the other side of the river. Although some groundwater could discharge into the river floodplain on the opposite side of the river from its point of origin, such flow would likely be discharged to the river because flow in the alluvium is toward the river. Potentiometric-surface maps developed by Clarke and West (1997) for the Upper Three Runs aquifer and Gordon aquifers do not indicate the possible occurrence of trans-river flow. However, flow lines on potentiometric-surface maps of the confined Dublin and Midville aquifer systems do suggest the possible occurrence of trans-river flow for a short distance into the Savannah River alluvial valley. The possible occurrence of trans-river flow in the Dublin aquifer system also is suggested by the chemical and isotopic composition of water from the Brighams Landing well-cluster site in Georgia. Clarke and West (1997) suggest that the potential for trans-river flow may be facilitated by groundwater withdrawal, particularly at pumping centers located near the Savannah River. Pumped wells on one side of the river could intercept groundwater that originates on the other side. For this to occur, pumping would need to be sufficient to reverse the hydraulic gradient away from the river and towards the pumping center.

Numerical simulation techniques have been used to further evaluate areas of previously documented trans-river flow on the Georgia side of the Savannah River **(Clarke and West, 1998; Cherry 2006)**. At such areas, local head gradients might allow the migration of contaminants from the Savannah River Site into the underlying aquifers and beneath the Savannah River into Georgia. Cherry (2006) identified the area near Flowery Gap Landing (covering about 1 mi^2) as an area of potential trans-river discharge. Backward particle tracking analysis was conducted to better quantify trans-river flow. Between 29 and 37 percent of the particles released in this area backtracked to recharge areas on the Savannah River Site (trans-river flow), depending on the scenario being evaluated. Of the particles exhibiting trans-river flow, the median time-of-travel ranged from 366 to 507 years. For the worst-case scenario evaluated (deactivation of Savannah River Site production wells), the median time-of-travel decreased to about 370 years, with a shortest time-of-travel period of about 80 years.

While the potential for trans-river flow exists, it is likely that such flow would be quickly discharged to the river because flow in the river alluvium is toward the river. Also, any tritiated water originating from the Savannah River Site and participating in trans-river flow would undergo significant radioactive decay, considering its 12.35-year half-life, relative to even the worst-case 80-year time-of-travel. Furthermore, pumping of the current make-up water wells for VEGP Units 1 and 2 does not appear to have intercepted groundwater originating from the other side of the river, based on the particle tracking results presented by Cherry (2006). It is also unlikely that pumping the additional water needed to supply VEGP Units 3 and 4 would be sufficient to reverse that hydraulic gradient and cause groundwater originating from South Carolina to be drawn any further into Georgia, given the high transmissivities of the confined Tertiary and Cretaceous aquifers. Therefore, trans-river flow does not appear to be a mechanism that would contribute to the contamination of aquifers underlying the VEGP site.

There is no evidence to suggest that the potential for groundwater leakage between the Upper Three Runs (Water Table) aquifer and Gordon (Tertiary sand) aquifer in the vicinity of the Pen Branch fault exists at the VEGP site. SSAR Section 2.5.1.2.4 describes previous investigations of the Pen Branch fault and the site subsurface investigation of the fault that was conducted for the ESP application. Results of this investigation, which included seismic reflection and refraction surveys, clearly document that the Pen Branch fault strikes northeast and dips southeast beneath the VEGP site. SSAR Figure 2.5.1-42 shows the vertical projection of the Pen Branch fault from the top of basement rock in relation to VEGP Units 3 and 4. The plan projection of the intersection of the Pen Branch fault with the top of basement rock is located beneath or slightly southeast of the antiformal hinge at the top of the monocline in the Blue Bluff Marl (SSAR Figure 2.5.1-39). Because of its spatial association with the Pen Branch fault, it is likely that this monocline feature is the result of reverse or reverse-oblique slip on the Pen Branch fault. The seismic survey data indicate that the fault terminates in the Cretaceous Coastal Plain deposits and that the overlying Tertiary deposits, including those comprising the Gordon (Tertiary sand) aquifer, Gordon aquitard (Blue Bluff Marl), and Upper Three Runs (Water Table) aquifer, are not considered to be affected by the Pen Branch fault. This result is consistent with that of Summerour et al. (1998) who reported that none of the faults identified in their seismic surveys appear to have disturbed the Gordon aquitard (Blue Bluff Marl), which isolates the unconfined from the underlying confined aquifers.

Based on the results and discussion presented above, the Pen Branch fault has not affected the Tertiary age deposits at the VEGP site and would be neither a barrier nor conduit for groundwater transport in these deposits. Insufficient data are available to determine if the fault would be a barrier or conduit in the deeper, Cretaceous deposits that have been affected by the fault.

2.4.12.1.2 Local Hydrogeology

The VEGP site is located approximately 40 mi southeast of the Fall Line, the northwestern boundary of the Coastal Plain physiographic province, and is adjacent to the Savannah River. Geologic conditions beneath the VEGP site generally consist of about 1000 ft of Coastal Plain sediments with underlying Triassic Basin rock southeast of the Pen Branch fault and Paleozoic crystalline rock northwest of this fault (Section 2.5.1). The Savannah River lies along the northeast border of the VEGP site and influences the local hydrogeologic conditions within the site area. This local hydrogeology discussion is restricted to the VEGP site vicinity (approximate radius of 5 mi) south of the Savannah River.

Geotechnical and hydrogeological investigations performed for this ESP application provide information on the VEGP site from the Triassic Basin rock to the ground surface. The geotechnical logs are provided in Appendix 2.5A and further discussed in Section 2.5.4. The boring logs from the observation well installation are presented in Appendix 2.4A. In addition, reviews of the original site investigations for VEGP Units 1 and 2, existing unit well monitoring programs, and published literature were included in the analysis. Results from these investigations indicate that there are three aquifers underlying the VEGP site, the Cretaceous, Tertiary, and Water Table (or Upper Three Runs), all being part of the Southeastern Coastal Plain aquifer system. Although present regionally, the Surficial aquifer system, consisting of Miocene (Hawthorne Formation) through Quaternary deposits, is not continuous over Burke County or the VEGP site **(Miller 1990)** and was not encountered in the investigations performed for this ESP application.

The lower aquifer at the VEGP site overlies the bedrock and is comprised of Cretaceous-age sediments. Locally, this aquifer system is known as the Cretaceous aquifer. The sediments include sands, gravels, and clays of the Cape Fear Formation, Pio-Nono Formation and associated unnamed sands, Gaillard Formation, Black Creek Formation, and Steel Creek Formation. The middle aquifer system is made up of Tertiary-age sediments occurring over the Cretaceous-age sediments described above. The middle aquifer is known locally as the Tertiary aquifer system. It consists primarily of the permeable sands of the Still Branch and Congaree Formations. The relatively impermeable clays and silts of the Snapp and Black Mingo Formations overlie and confine the Cretaceous aquifer, while the clays and clayey sands of the Lisbon Formation overlie and confine the Tertiary aquifer. The upper aquifer is unconfined and is comprised of Tertiary-age sands, clays, and silts of the Barnwell Formation, which overlie the relatively impermeable Lisbon Formation. This aquifer is known locally as the Water Table aquifer or Upper Three Runs aquifer. Figure 2.4.12-1 illustrates the hydrostratigraphic column for the VEGP site and surrounding area, identifying geologic units, confining units, and aquifers. Figures 2.4.12-2A and 2B present hydrogeologic cross sections for the VEGP site. The aquifers underlying the VEGP site and surrounding area are discussed below.

Cretaceous Aquifer

The Cretaceous aquifer locally comprises the Cape Fear Formation, Pio-Nono Formation/unnamed sands, Gaillard Formation/Black Creek Formation, and Steel Creek Formation. These formations generally consist of fluvial and estuarine deposits of cross-bedded quartzitic sand and gravel interbedded with silt and clay. The coarse-grained sediments are mostly unconsolidated and are generally permeable, while the fine-grained sediments are partially consolidated and are generally impermeable. In addition to the varying lithology, the formation also exhibits lateral facies changes, on-lap and off-lap relationships, and discontinuous lenses **(Huddlestun and Summerour 1996)**. The elevations, thicknesses, and descriptions of these geologic formations, as determined from VEGP geotechnical boring B-1003, are summarized below:

- The basal Cape Fear Formation overlies the Triassic Dunbarton Basin bedrock, which consists of alternating mudstone, sandstone, and breccia. Boring B-1003 encountered top of bedrock at an elevation of approximately -826 ft msl. The Cape Fear Formation consists of interbedded sands, silts, clays, and gravels. The formation is approximately 191 ft thick, with the top of the formation being at El. -635 ft msl.

- The Pio-Nono Formation and other unnamed sands overlie the Cape Fear Formation. This formation consists of sand, silt, and clay. The formation is approximately 60 ft thick, while the top of the formation is at approximately El. -575 ft msl.

- The undifferentiated Gaillard Formation and Black Creek Formation overlie the Pio-Nono Formation and unnamed sands. Most of the formation consists of sand with silt and clay, and layers of gravel. The deposit is approximately 211 ft thick, with the top of the formation being at approximately El. -364 ft msl.

- The Steel Creek Formation overlies the undifferentiated Gaillard Formation and Black Creek Formation. It consists mainly of sand with clay and silt. The formation is approximately 110 ft thick; the top of the formation is at approximately El. -254 ft msl.

The Cretaceous aquifer system has not been extensively developed, primarily because the shallower Tertiary system is adequate for most groundwater needs and is available for use throughout the region. Quantitative data from the limited number of test and production wells in the Cretaceous strata, and inferred data from geologic and stratigraphic studies, indicate clearly that the Cretaceous aquifer system is highly transmissive and is capable of providing good quality groundwater.

Recharge to the Cretaceous aquifer system is primarily by direct infiltration of rainfall in its outcrop area, located north of the VEGP site in a 10- to 30-mile-wide belt extending from Augusta, Georgia, northeastward across South Carolina to near the state line separating North and South Carolina. In the outcrop areas, precipitation penetrates the Cretaceous sediments. Groundwater in the outcrop areas is under water table conditions, but as it moves progressively downdip, it becomes confined beneath the overlying Snapp and Black Mingo Formations in the vicinity of the VEGP site. Hence, the Cretaceous aquifer system is under confined conditions for most of its areal extent. Discharge of the Cretaceous aquifer system is primarily from subaqueous exposures of the aquifer that are presumed to occur along the Continental Shelf. Other discharge sources are to the Savannah River and by pumping.

Tertiary Aquifer

The most productive aquifer at the VEGP site consists of the Congaree and Still Branch Formations, which are hydraulically connected and are referred to as the Tertiary aquifer. The overlying Lisbon Formation, containing the Blue Bluff Marl, acts as a confining layer. The elevations, thicknesses, and descriptions of geologic formations comprising the Tertiary aquifer, as encountered in boring B-1003, are described below:

- The Black Mingo and Snapp Formations constitute a semi-confining hydrogeologic unit under the VEGP site that separates the underlying Cretaceous aquifer from the overlying Tertiary sand aquifer as they dip to the southeast. The Paleocene-age Black Mingo Formation is approximately 39 ft thick and consists of sand, clay, and silt. The top of the formation is at approximately El. -215 ft msl. The Snapp Formation overlies the Black Mingo Formation and consists of sand, clay and silt, and includes a basal gravel layer. The stratum is also Paleocene in age. The formation is approximately 107 ft thick. The top of the formation is at approximately El. -108 ft msl.

- Above the Snapp is the Eocene-age Congaree Formation. The Congaree Formation has a thickness of about 115 ft and consists primarily of sand with clay and silt, and a basal gravel layer. The top of the formation is at an elevation of approximately 7.3 ft msl. The overlying Still Branch and Bennock Millpond Sands Formation consist of sand, clay, and silt and has a weak carbonate component. The formation thickness is approximately 67 ft, with the top of the formation being approximately El. 74 ft msl.

- The Lisbon Formation overlies the Tertiary sediments. The Lisbon Formation is Eocene in age and is comprised of sand, clay, and silt with interbedded layers of fossiliferous limestone. The Lisbon Formation contains a marl known as the Blue Bluff Member (Blue Bluff Marl). The Lisbon Formation also contains the McBean Limestone Member, a fossiliferrous limestone layer. The formation has a thickness of approximately 63 ft, and the top of the formation is at approximately El. 137 feet msl. This formation separates the confined and unconfined aquifer systems beneath the VEGP site.

  In addition, the VEGP Units 1 and 2 UFSAR Section 2.5.1.2.2.2.1.1 indicates that the Blue Bluff marl is a distinct unit that is relatively constant in thickness over many square miles, although variable in lithology. Contours of the upper and lower surfaces as well as an isopach map of the marl in the vicinity of the plant are shown on drawings AX6DD352, AX6DD371, and AX6DD372 of the UFSAR. These drawings indicate the Blue Bluff Marl to be continuous over the entire VEGP site.

Recharge to the Tertiary aquifer is primarily by infiltration of rainfall in its outcrop area, which is a belt 20 to 60 miles wide extending northeastward across central Georgia and into portions of Alabama to the west and South Carolina to the east. Discharge from the Tertiary aquifer occurs from pumping, from natural springs in areas where topography is lower than the piezometric level of the aquifer, and from subaqueous outcrops that are presumed to occur offshore. Discharge also occurs to the Savannah River where the river has completed eroded the Blue Bluff Marl confining layer allowing discharge from the aquifer to the river bed.

Water Table Aquifer

The uppermost aquifer at the VEGP site is unconfined and consists of the Barnwell Group, including the discontinuous deposits of the Utley limestone. The saturated interval within the Barnwell Group is commonly referred to as the Water Table aquifer (also known as the Upper Three Runs aquifer) and is the first water-bearing zone encountered beneath the VEGP site. The elevations, thicknesses, and descriptions of geologic formations comprising the Barnwell Group were determined from VEGP ESP geotechnical and hydrogeological borings and are described below.

- The Utley Limestone Member of the Barnwell Group consists of sand, clay, and silt with carbonate-rich layers. The stratum is discontinuous across the VEGP site and was not encountered in several of the borings. To assess its degree of discontinuity, borings logged for the hydrogeological and geotechnical investigations have been examined for the presence/absence of the Utley limestone. Logs for these borings are included in Appendices 2.4A and 2.5A. In completing this assessment, effort was made to eliminate spatial bias. Therefore, only one boring log was considered when there were adjacent borings from OW-series well pairs, or adjacent B- and OW-series borings. The results are summarized in Table 2.4.12-13.

  The data presented in Table 2.4.12-13 indicate that the Utley limestone is absent in 8 out of 18 borings, or 44 percent of the borings. Spatial trends in the presence/absence of the Utley limestone indicate that the unit tends to be present in the power block area for VEGP Units 3 and 4 and the area to the north towards Mallard Pond. The Utley limestone tends to be absent in the cooling tower area for VEGP Units 3 and 4 and the area to the south. These results are consistent with the Utley limestone isopachs presented in the VEGP Units 1 and 2 UFSAR (Drawing No. AX6DD376). These isopachs indicate that the limestone increases in thickness to a maximum of about 80 ft and then decreases in thickness to 10 ft or less along a profile extending from the power block to Mallard Pond, with the long axis of this unit trending in a northeast-southwest direction.

- Overlying the Utley limestone are undifferentiated sands, clays, and silts. The thickness of the group is variable with a range of approximately 14 to 119 ft. The top of the group extends to the ground surface and ranges from approximately El. 205 to 264 ft msl. At boring B-1003, the formation is approximately 48 ft thick with the top of the formation being at an elevation of approximately 223 ft msl.

Recharge to the Water Table aquifer is almost exclusively by infiltration of direct precipitation. The presence of porous surface sands and the moderate topographic relief in the VEGP site area suggest that a significant fraction of the precipitation infiltrates the ground or is lost to the atmosphere by evapotranspiration. Discharge is to localized drainages and wells.

2.4.12.1.3 Observation Well Data

Data from a combination of new wells installed for the ESP application and existing VEGP site wells were used to develop groundwater elevation contour maps and present groundwater elevation trends. The new wells, designated OW-1001 through OW-1015, were installed in May and June 2005. Ten of the new wells are screened in the Water Table aquifer and five are screened in the confined Tertiary aquifer system below the Blue Bluff Marl. No wells were installed into the deeper Cretaceous aquifer. Existing wells 142 and 179, remaining from the pre-construction monitoring network for VEGP Units 1 and 2, are screened in the Water Table aquifer. Existing wells with identifications beginning with the number 8 were installed between 1979 and 1985 to monitor construction dewatering of VEGP Units 1 and 2. These wells are screened in either the Water Table or Tertiary aquifers. Existing wells with an LT designation were installed in 1985 as part of post-construction monitoring activities and are screened in the Water Table aquifer.

Observation well OW-1001A was installed at the site in October 2005 to replace OW-1001. Observation well OW-1001A was the only new “A” well installed at the site for the ESP application. Observation well OW-1001A may be confused with the borings or drill logs contained in Appendix 2.4A which also use the suffix “A” to indicate abandoned wells. OW-1001A was installed during the geotechnical subsurface investigation performed at the site and is not discussed in Appendix 2.4A report. A summary of borings or holes drilled at the site to accommodate installation of the new observation wells is provided in Table 2.4.12-14.

Groundwater level elevations in OW-1001 measured between the period June 2005 and November 2006 (groundwater level data continues to be collected in wells OW-1001 and OW-1001A for observation purposes) range from about 114 to 118 ft msl with a seasonal fluctuation of about 4.4 ft. These groundwater levels and seasonal fluctuations are not consistent with the groundwater levels and seasonal fluctuations of groundwater levels in the Water Table aquifer and suggest that the screened portion of the well is not in good hydraulic communication with the Water Table aquifer. Review of the boring log, daily field log, well development log and in situ hydraulic conductivity test results for the well indicate that either the formation material adjacent to the well was adversely impacted by well construction or that the well was inadvertently installed in the confining unit underlying the formation material. Observation well OW-1001A was installed to replace well OW-1001, as discussed above. The construction log for OW-1001A contained in Appendix 2.5A (report Appendix D) indicates that the screened portion of the well ranges in elevation from 146.13 to 136.13 ft msl. Groundwater level elevations for the 18-month monitoring period range from 135.91 to 135.99 ft msl. Based on these groundwater level data, it is evident that the groundwater level in the well is close to or below the bottom of the screened interval of the well, indicating no hydraulic communication with the Water Table aquifer. Groundwater data obtained from OW-1001 and OW-1001A are considered invalid and are not used in the following groundwater evaluations.

Monthly water levels in the observation wells were measured to characterize seasonal trends in groundwater levels and flow directions for the VEGP site. Monthly monitoring of these wells began in June 2005 and is continuing. An 18-month data set representing June 2005 through November 2006 is utilized for the ESP application. In addition, some long-term data are available for certain existing wells completed in the Water Table and Tertiary aquifers and are used to characterize historic trends.

The locations of VEGP site observation wells that are being monitored are shown in Figure 2.4.12-3. Table 2.4.12-1 lists the observation wells currently being used to monitor the Water Table aquifer, while Table 2.4.12-2 lists the observation wells currently being used to monitor the Tertiary aquifer.

The following groundwater piezometric surface discussion is based on the information presented in Tables 2.4.12-1 and 2.4.12-2, Figures 2.4.12-7 through 2.4.12-11, Figures 2.4.12-14 through 2.4.12-18, and Figures 2.4.12-21 through 2.4.12-26.

Water Table Aquifer

Groundwater level data for the Water Table aquifer available for the 1979 through 2006 period are provided in Figure 2.4.12-21. Table 2.4.12-15 summarizes the historical groundwater levels for the Water Table aquifer. Also shown on this figure is annual precipitation measured at three climate stations close to the VEGP site, which includes the Augusta WSO Airport, Waynesboro 2 NE, and Milen 4N climate stations. Precipitation data were obtained from the South Carolina Department of Natural Resources website (SC DNR 2007). In addition, the Palmer Drought Severity Index (PDSI) and Palmer Hydrological Drought Index (PHDI) are plotted on Figure 2.4.12-22 for the same period. The PDSI attempts to measure the duration and intensity of the long-term cumulative meteorological drought and wet conditions. The PDHI is another long-term drought index intended to measure the hydrological impacts of drought (e.g., reservoir levels, groundwater levels, etc.). PDSI and PHDI data were obtained from the National Climatic Data Center (NCDC) website (NCDC 2007). These indices provide an indication of the severity of a wet or dry spell. The indices generally range from +6 to -6 with negative values denoting dry spells and positive values denoting wet spells. Values of +0.5 to -0.5 indicate normal conditions.

Figure 2.4.12-21 shows that during the period 1979 to 1984, groundwater level elevations in the Water Table aquifer were impacted (lowered) by construction dewatering of the power block excavation for VEGP Units 1 and 2 that was in effect from June 1976 to March 1983. Groundwater levels for subsequent years exhibit variability in response to meteorological conditions. The magnitude of the variability can be estimated using data from the wells having the longest period of record, which include wells 802A, 805A, 808, LT-7A, LT-12, and LT-13. Table 2.4.12-16 summarizes the minimum and maximum water levels recorded at each of these wells. These results indicate a 5-to 8-ft range in water levels over the 17-year period of record for these wells. Inspection of the long-term hydrographs for these wells in conjunction with the drought severity indices for the same period indicates that groundwater levels in the Water Table aquifer generally correlate with the PDSI and PDHI. Water levels tend to remain unchanged when the drought severity indices remain near normal (±1). During drought periods when the PDSI or PDHI index falls to -2 or below, groundwater levels tend to decline. Conversely, during wet periods when the PDSI or PDHI increases to +2 or more, groundwater levels tend to rise. Increases or decreases in the drought indices would be associated with the increases or decreases in the rate of recharge of the Water Table aquifer. Because of the relatively large depth to the water table (at least 60 ft), prolonged wet or dry periods on the order of a year in duration are apparently required to affect the recharge to the water table at these depths.

Recent groundwater data from June 2005 to November 2006 for the Water Table aquifer are summarized in Table 2.4.12-1 and shown in Figure 2.4.12-23. During the 18-month monitoring period, groundwater elevations ranged from about 133 to 165 ft msl with seasonal fluctuations averaging about 1 foot. These data exhibit very little variability because the recharge during this period was evidently relatively constant. Comparison of historical groundwater level elevations to precipitation events and other meteorological indices over a longer period of time suggest that persistent and significant wet weather is required to elicit any significant water table response, as discussed above. The annual precipitation, the PDSI, and the PDHI for the 2004 to 2006 period have been relatively stable and near normal values. Due to the absence of any upward or downward trends in these indices, it is therefore expected that groundwater elevations in the Water Table aquifer would be relatively steady over this period.

The groundwater elevation data summarized in Table 2.4.12-1 were used to develop groundwater surface elevation contour maps for the Water Table aquifer on a quarterly basis. These maps are presented in Figures 2.4.12-7 through 2.4.12-11 and Figure 2.4.12-24 for June 2005 through November 2006. Note that a contour map for November 2006 was developed as no groundwater level data are available for September and October 2006. For each quarter, the spatial trend in the piezometric surface is similar, with elevations ranging from a high of approximately El. 165 ft msl in the vicinity of well OW-1013 to a low of less than El. 135 ft msl at well OW-1005. The groundwater surface contour maps indicate that horizontal groundwater flow across the VEGP site is in a north-northwest direction toward Mallard Pond (also known as Mathes Pond). This surface water feature is a local discharge point for the shallow groundwater flowing beneath the VEGP site. The horizontal hydraulic gradient across the site for the Water Table aquifer is relatively consistent between the five figures and is approximately 0.014 ft/ft.

Tertiary Aquifer

Historical groundwater elevations from 1971 through 1985 for Tertiary aquifer wells 27 and 29 are provided in Figure 2.4.12-12.

Recent groundwater elevation data from June 2005 to November 2006 for the Tertiary aquifer are summarized in Table 2.4.12-2 and shown in Figure 2.4.12-25. Groundwater elevations for this 18-month monitoring period range from about 82 to 128 ft msl. Elevations are relatively constant from June to August 2005. In most cases, the piezometric head of the aquifer declines from August 2005 through November 2005. The elevations begin to rebound in December 2006, continuing through February 2006. The lowering of the piezometric surface is likely in response to a decrease in precipitation. October and November are the months with the lowest precipitation during the year for this area. Well 27 shows a higher degree of variability than the others and is likely influenced by its proximity to the river.

The groundwater elevation data summarized in Table 2.4.12-2 were used to develop piezometric surface maps for the Tertiary aquifer. The Tertiary aquifer piezometric surface is presented in Figures 2.4.12-14 through 2.4.12-18 and Figure 2.4.12-26 for June 2005 through November 2006. The piezometric surfaces for the Tertiary aquifer show a relatively consistent flow pattern. In general, the groundwater in this aquifer unit shows an east-to-northeast flow pattern, toward the Savannah River. Head elevations range from approximately El. 125 ft msl in the western portion of the VEGP site to less than El. 100 ft msl in the vicinity of the bluff next to the Savannah River flood plain. The elevation of the piezometric head at the bluff and that of the Savannah River flood plain suggest groundwater is discharging to the Savannah River. The piezometric elevations in the Tertiary aquifer decreased at least 1.5 ft across the VEGP site in December 2005, reflecting the seasonal decrease in precipitation.

The horizontal hydraulic gradient across the site for the Tertiary aquifer is relatively consistent among the five figures and is approximately 0.006 ft/ft. In the center of the VEGP site, there is a downward head difference of approximately 50 ft between the Water Table aquifer and the Tertiary aquifer, suggesting hydraulic separation of the two aquifers. The Blue Bluff Marl confining unit that separates the aquifer systems has an average thickness of about 70 ft at the VEGP site.

Cretaceous Aquifer

At the VEGP site, both the Cretaceous and the Tertiary aquifers are considered confined beneath the Blue Bluff Marl but are in apparent hydraulic connection with each other. At some distance downdip of the VEGP site, the Cretaceous aquifer becomes hydraulically separated from the Tertiary aquifer. This separation is believed to be due to facies changes in the intervening clays and silts of the Snapp and Black Mingo formations becoming relatively impermeable. The point at which this occurs is not well defined but it is believed to be a few miles downdip (south) of the site.

The regional direction of the groundwater flow in the Cretaceous (and the Tertiary) aquifer system is south-by-southeast at a hydraulic gradient of approximately 6 to 20 ft/mi (0.001 to 0.004 ft/ft) **(Siple 1967)**. From the vicinity of the Fall Line to a point expected to be a few miles south of the site, the Savannah River has downcut through the Blue Bluff Marl confining layer and into the underlying strata. The Savannah River channel cut allows both the Cretaceous and the Tertiary aquifers to discharge to the riverbed, resulting in a localized hydraulic (groundwater) sink. The aquifer flow directions in the vicinity of the river cut are affected by the hydraulic sink and do not follow regional trends.

2.4.12.1.4 Hydrogeologic Properties

Slug tests were performed in the new groundwater observation wells installed in connection with the ESP application to determine in situ hydraulic conductivity values for the Water Table and Tertiary aquifers. Table 2.4.12-3 summarizes the test results. Soil samples collected from selected geotechnical and hydrogeological borings were submitted for laboratory tests to determine grain size, moisture content, and specific gravity, results from which are included in Tables 2.4.12-4 through 2.4.12-6. Similar data are available for the adjacent VEGP Units 1 and 2 site. The hydrogeological properties of the Water Table aquifer, Lisbon Formation (Blue Bluff Marl) confining unit, Tertiary aquifer, and Cretaceous aquifer at the VEGP site are discussed below.

Water Table Aquifer

In the vicinity of the VEGP site, the basal unit of the Barnwell Group, the Utley limestone member, is capable of transmitting groundwater but is of limited areal and vertical extent. In addition, the horizontal and vertical hydraulic conductivity of the saturated clays, silts, and sands within the Barnwell Group varies considerably, due to variable clay content.

The hydraulic conductivity of the Water Table aquifer within the vicinity of the VEGP site was previously measured by both in situ and laboratory testing methods during site characterization investigations for VEGP Units 1 and 2. In situ hydraulic conductivity values for the Barnwell Group sands, silts, and clays were found to range between 200 and 267 ft/yr (0.5 to 0.7 ft/day). Laboratory values varied beyond the range of the in situ tests from 9.8 to 302 ft/yr (0.03 to 0.8 ft/day). Well pumping tests conducted in the Utley limestone resulted in hydraulic conductivities ranging from 3,250 to 125,400 ft/yr (9 to 343 ft/day), while falling and constant head tests suggested lower values, ranging from 96 to 5,800 ft/yr (0.3 to 16 ft/day). These results indicate the possibility of localized, highly permeable zones in the Utley limestone. Laboratory porosity values for the Barnwell Group sands, silts, and clays were found to range from 34 to 61 percent, with a mean value of 44 percent.

Hydraulic conductivities were determined for the VEGP Units 3 and 4 site as part of the ESP investigation. Slug test results for the Water Table aquifer range from 0.12 to 2.65 ft/day, with a geometric mean of 0.5 ft/day (Table 2.4.12-3). Table 2.4.12-4 summarizes the laboratory test results for geotechnical samples of the Barnwell Formation, which were at depths ranging from El. 108 to 248 ft msl. Sand and clay make up the majority of samples, with some gravel present. Measured moisture contents, by weight, range from 4 to 93 percent and have a median value of about 25 percent. Specific gravity analysis was performed only for the samples collected from the observation well borings. Values range between 2.59 to 2.75 and have a median value of 2.66. Using the median moisture content of 25 percent and a value of 2.66 for the specific gravity, the void ratio is estimated to be about 0.67. A total porosity of 40 percent is calculated from this void ratio **(Craig 1994),** and an effective porosity of about 32 percent is estimated based on 80 percent of the total porosity **(de Marsily 1986)**. The specific yield for the Water Table aquifer was not determined; however, an estimate of this value taken from published literature for similar aquifer materials indicates that it may be in the range of 0.20 to 0.33 **(McWhorter and Sunada 1977)**.

The groundwater travel time in the Water Table aquifer was calculated from the ESP site to the projected discharge point (Mallard Pond). A horizontal hydraulic gradient of 0.014 ft/ft was estimated using the maximum water level observed at OW-1009 (163.03 ft msl), the minimum water level observed at OW-1005 (132.53 ft msl), and the distance between the two observation wells of about 2,200 ft. A hydraulic conductivity value of 0.5 ft/day was used, which is considered to be a representative hydraulic conductivity value for the Barnwell Formation which includes the Utley limestone. Using this hydraulic conductivity of 0.5 ft/day and an effective porosity of 32 percent, an average horizontal groundwater velocity of 0.02 ft/day was calculated **(Heath 1998).** Using a distance of approximately 2,450 ft between either auxiliary building and the south side of Mallard Pond, the groundwater travel time is estimated to be about 336 years.

The geotechnical boring logs contained in Appendix 2.5A, which report some occurrence of water loss during drilling through the Utley limestone, and high hydraulic conductivity test results for the Utley limestone obtained during site investigations for VEGP Units 1 and 2 indicate the possibility of localized highly permeable zones in the Utley limestone. These zones could act as preferential pathways for groundwater flow if there was an accidental liquid release of effluents to the groundwater at the VEGP site.

As described in SSAR Section 2.5.4.5, construction of the new VEGP Units 3 and 4 will require a substantial amount of excavation and backfill. The excavation will be necessary to completely remove the sands, silt, clays, and Utley limestone of the Barnwell Group. Total excavation depth to the Blue Bluff Marl bearing stratum is expected to range from approximately 80 to 90 ft below existing grade. Backfilling will be performed from the top of the Blue Bluff Marl to the bottom of the containment and auxiliary buildings at a depth of about 40 ft below final grade. The construction duration for excavation then backfill to the bottom of the containment and auxiliary buildings is currently projected to be about 18 months. Filling will continue up around these structures to final grade. The fill will primarily consist of granular materials, selected from portions of the excavated sands and from other available borrow sources. Following the guidelines used during construction of VEGP Units 1 and 2, structural fill will be a sandy or silty sand material with no more than 25 percent of the particle sizes smaller than the No. 200 sieve. This structural fill will be compacted to a minimum of 97 percent of the maximum dry density.

Excavating existing soils and replacing these soils with structural fill will alter the hydrogeologic characteristics of the subsurface materials within the footprint of VEGP Units 3 and 4. In situ hydraulic testing of fill material for VEGP Units 1 and 2 indicates a hydraulic conductivity range of 480 ft/yr (1.3 ft/day) to 1,220 ft/yr (3.3 ft/day) based on data included in UFSAR Table 2.4.12-15. Values for Units 3 and 4 are expected to be similar because the borrow sources and compaction criteria for the fill will be the same. Compared to the hydraulic conductivities for the Water Table aquifer, as described above, it can be seen that the hydraulic conductivity of the fill is generally higher than that of the in situ soils.

Development of VEGP Units 3 and 4 will also increase the impervious area across the VEGP site where power generation and associated facilities are constructed. Storm-water management facilities (e.g., catch basins, storm sewers) will be used to convey runoff from precipitation offsite. The increased impervious area and use of storm-water management facilities will tend to reduce the recharge to the Water Table aquifer in areas affected by Unit 3 and 4 construction.

Construction of VEGP Units 3 and 4 will entail the placement of relatively large and impermeable structures below grade. The base elevations of the major structures (containment and auxiliary buildings) will be at about El. 186.5 ft msl. This elevation is at least 25 to 35 ft above the water table. Because these structures will not extend below the water table, they would not affect the hydrogeologic characteristics of the underlying saturated zone.

Lisbon Formation (Blue Bluff Marl) Confining Unit

The hydraulic conductivity of the marl layer is very low, and it effectively confines the aquifer underlying it. It is considered a vertical barrier to groundwater movement. In situ permeability tests (packer tests) were performed in the marl during site characterization investigations for VEGP Units 1 and 2. In 90 percent of the intervals tested, no measurable water inflow occurred. Laboratory permeability tests were also conducted on core samples collected from the marl. Laboratory measurements ranged from 0.0052 to 8.8 ft/yr (1.4×10^{-5} to 2.4×10^{-2} ft/day) with a geometric mean of 1.3×10^{-3} ft/day, indicating the marl is nearly impermeable. Porosity values ranged from 24 to 62 percent, with a mean value of 48 percent.

Geotechnical laboratory results for the Lisbon Formation (Blue Bluff Marl) confining unit are summarized in Table 2.4.12-5 for the VEGP site. Soil samples were collected between El. 51

and 135 ft msl. The samples consist of gravel, sand, and clay. Moisture contents range from 13.5 to 67 percent, with porosities of 25 to 59 percent. Using the median moisture content of 29 percent from geotechnical laboratory results and an assumed specific gravity of 2.65, the void ratio of the confining unit is estimated to be 77 percent. Based on the void ratio value, total porosity is calculated to be 44 percent. The effective porosity of the Blue Bluff Marl was estimated using de Marsily (1986) Figure 2.17. This figure plots total and effective porosity as a function of grain size. To estimate the effective porosity for the marl, the ratio of effective to total porosity determined from Figure 2.17 was applied to the site-specific total porosity value for the VEGP site. Using the median D50 value of 0.24 mm as a representative grain size (Table 2.4.12-5), a ratio of effective to total porosity of about 0.8 was determined. Multiplying the median total porosity of 0.44 by this ratio yields an effective porosity of 0.35.

The effective porosity was also estimated as the difference between the total porosity and the residual water content, as given by Yu et al. (1993) Equation 4.4. Grain size distribution data indicate that most of the Blue Bluff Marl samples can be classified as a silty sand (SM) or clayey sand (SC). The residual water content for SM or SC soils obtained from Carsel and Parrish (1988) using equivalent USDA-SCS soil textural classifications ranges from 0.07 to 0.10. The effective porosity would then range from 0.34 to 0.37. This result indicates that the 0.35 value for effective porosity is representative of the Blue Bluff Marl.

Tertiary Aquifer

Hydraulic conductivities determined from Tertiary aquifer slug tests range from 0.35 to 2.1 ft/day, with a geometric mean of 0.83 ft/day (Table 2.4.12-3). These results are consistent with those for the VEGP Units 1 and 2 site for which the geometric mean was determined to be 0.51 ft/day. The laboratory results from the selected geotechnical samples collected in the Tertiary aquifer are presented in Table 2.4.12-6. Sample elevations range from El. -273 ft msl to 69 ft msl, with the samples consisting mainly of sand and fine particles, with some gravel. Moisture content ranges from 19 to 41 percent, with specific gravity values varying from 2.62 to 2.69. Using the median moisture content of 24 percent and a value of 2.67 for the specific gravity, the void ratio of the Tertiary aquifer is estimated to be about 0.64. A total porosity of 39 percent is calculated from this void ratio **(Craig 1994),** and an effective porosity of about 31 percent is estimated based on 80 percent of the total porosity **(de Marsily 1986)**. The storage coefficient for the Tertiary aquifer alone has not been determined; however, previous tests of wells completed in the combined Cretaceous/Tertiary aquifers suggest that a value on the order of 10^{-4} would be a reasonable estimate (see below).

The horizontal hydraulic gradient of the Tertiary aquifer is approximately 0.005 ft/ft, based on the maximum water level observed at well OW-1008 (127.99 ft msl), the minimum water level observed at well 27 (81.5 ft msl), and the distance between the two observation wells of about 8,700 ft. The average horizontal groundwater velocity was calculated at 0.013 ft/day using a hydraulic conductivity of 0.83 ft/day, a hydraulic gradient of 0.005 ft/ft, and an effective porosity of 31 percent **(Heath 1998)**. Using a distance of 5,600 ft from center of the power block area for the new AP1000 units to the closest point of the Savannah River, the groundwater travel time from the power block area to the Savannah River in the Tertiary aquifer is estimated to be about 1180 years.

Cretaceous Aquifer

Two makeup water wells (designated as MU-1 and MU-2A) for VEGP Units 1 and 2 were reported to be capable of supplying water at 2,000 gal./min and 1,000 gal./min, respectively. The water is withdrawn from the combined Cretaceous/Tertiary aquifers. Pumping tests were conducted at these wells in 1977. Transmissivity values ranged between 110,400 to 130,900 gallons per day per foot (gpd/ft). A storage coefficient was calculated at 1.07×10^{-4}.

A pumping test was also conducted in a Cretaceous aquifer test well identified as TW-1 during site characterization activities for VEGP Units 1 and 2. A transmissivity value of 158,000 gpd/ft was calculated as an average value for the aquifer. The storage coefficient ranged between 3.3×10^{-4} and 2.1×10^{-4}, indicating the aquifer is effectively under confined conditions.

Vertical hydraulic conductivities were estimated assuming that the anisotropy ratio between the vertical and horizontal directions is 1:3, based on measured horizontal and vertical hydraulic conductivities for sandstone deposits **(Freeze and Cherry 1979)**. The vertical hydraulic conductivities for the Water Table aquifer, Lisbon Formation confining unit, and Tertiary aquifer are estimated to be 0.14, 0.00045, and 0.28 ft/day, respectively.

2.4.12.2 Regional and Local Groundwater Use

Present groundwater uses within 25 mi of the VEGP site are primarily municipal, industrial, and agricultural. Most of the groundwater wells withdraw water from the Cretaceous aquifer. Apart from water withdrawals for VEGP Units 1 and 2, the immediate area near the VEGP site has mainly domestic users, with no other nearby large groundwater users. The nearest domestic well is located west of the VEGP site across River Road.

The Georgia Environmental Protection Division (EPD) issues permits for wells having average daily withdrawals that exceed 100,000 gpd during any single month. Table 2.4.12-7 lists the permitted groundwater users, aquifer and withdrawal rates, and annual average withdrawal rates for municipal and industrial wells within 25 mi of the VEGP site and permitted by the Georgia EDP. Table 2.4.12-8 lists similar data for agricultural wells for the counties within 25 mi of the VEGP site and permitted by the Georgia EPD. The Safe Drinking Water Information System (SDWIS) maintained by the US EPA lists community, non-transient non-community, and transient non-community water systems serving the public. Community water systems are defined as those that serve the same people year-round (e.g., in homes or businesses). Non-transient non-community water systems are those that serve the same people, but not year-round (e.g., schools that have their own water system). Transient non-community water systems are those that do not consistently serve the same people (e.g., rest stops, campground, gas stations). Table 2.4.12-9 lists the community, non-transient non-community, and transient non-community water systems using groundwater as their primary water source within 25 mi of the VEGP site.

The locations of the agricultural, industrial, and municipal wells permitted by the Georgia EPD along with the public water system wells listed in the SDWIS database within 25 mi of the VEGP site are shown in Figure 2.4.12-19. These data indicate the nearest permitted agricultural well (William Hatcher, A-28) to be about 3.4 mi northwest of the VEGP site, while the nearest permitted industrial well (International Paper, I-1) is about 8.5 mi northwest of the site. The nearest municipal well (City of Waynesboro, M-1) is seen to be about 14.5 mi west-southwest of the VEGP site. The nearest SDWIS-listed well (Dealigle Mobile Home Park, C-6) is about 4.9 mi southwest of the VEGP site These wells are sufficiently distant from the VEGP site such that pumping these wells would have no effect on groundwater levels at the VEGP site. The recharge areas for the source aquifers for the nearest Georgia EPD-permitted wells are in their outcrop areas located up-gradient of the VEGP site and beyond the influence of the new units.

Regionally, projected overall water use is expected to increase through 2035 for Burke County. Surface water usage is increasing; however, it is increasing at a much slower rate than groundwater usage, approximately 5 percent versus 17 percent. Burke County's water usage, including both surface and groundwater, is projected as 100 to 120 mgpd for 2035 **(Fanning et al. 2003)**. Projections for Burke County total water use in 2050 are provided in the Comprehensive Water Supply Management Plan for Burke County and its Municipalities **(Rutherford 2000)**. Assuming the same water usage patterns, groundwater demand with the population increasing to 43,420 people is projected to be 10.94 mgpd for domestic use, 14.73 mgpd for industrial use, and 40.96 mgpd for agricultural use, which totals 66.63 mgpd **(Rutherford 2000)**.

Local groundwater use includes domestic wells and wells supplying water to existing VEGP Units 1 and 2. Operating plant uses include makeup process water, utility water, potable water, and supply for the fire protection system. Table 2.4.12-10 lists these wells, while Figure 2.4.12-20 identifies their location. Current permitted withdrawal rates are a monthly average of 6 mgpd and an annual average of 5.5 mgpd, as permitted by the Georgia EPD. Three of the wells are in the Cretaceous aquifer at depths varying from 851 to 884 ft, with design yields of 1,000 to 2,000 gpm. These wells provide makeup water for the plant processes. The remaining six wells extend into the Tertiary aquifer, range in depth from 200 to 370 ft, and have design yields of 20 to 150 gpm. Average annual usage levels for 1999 to 2004 from all wells excluding SEC are from 0.79 to 1.44 mgpd **(SNS 2005a)**. The SEC well was added in 2005 and will be included on water usage data from 2006. Recent groundwater usage from June 2005 to December 2005 is in Table 2.4.12-11.

Table 2.4.12-12 shows projected groundwater use for two AP1000 units with normal and maximum usage values. Service water system make-up, potable water system, demineralized water system, fire protection system, and miscellaneous users are the intended uses. Groundwater needed to supply VEGP Units 3 and 4 will be obtained from two 1,500 gpm wells installed in the Cretaceous aquifer. The maximum case water demand is conservatively based on several plant operating modes, which are not expected to operate concurrently. Based on the wells that currently supply makeup water for plant processes for the existing Units 1 and 2 (MU-1 and MU-2A) the proposed wells will extend to a depth of approximately 850 ft below the ground surface and will be open to selected aquifer zones within the Cretaceous aquifer. The proposed locations of the new wells are shown on Figure 2.4.12-27. SNC's groundwater use permit **(SNS 2005a)** will be modified accordingly.

2.4.12.3 Monitoring or Safeguard Requirements

Groundwater monitoring for the VEGP site takes place through programs implemented both for the existing units and as part of the ESP effort by SNC. Current groundwater monitoring programs for the existing units are addressed in VEGP Procedure Number 30140-C, Revision 22 **(VEGP 2006)**. The results of these programs are reported semiannually.

As part of detailed engineering, the existing SNC groundwater monitoring programs will be evaluated with respect to placement of the new units to determine if any additional monitoring of existing or construction of new observation wells will be required to adequately monitor impacts on groundwater. This evaluation will include a review of the observation wells installed for the ESP application to determine if they can be used as part of any longer-term groundwater monitoring program. The results will be described in the COL application.

Safeguards will be used to minimize the potential for adverse impacts to the groundwater by construction and operation of the new units. These safeguards could include the use of lined containment structures around storage tanks and hazardous materials storage areas, emergency cleanup procedures to capture and remove surface containments, and other measures deemed necessary to prevent or minimize adverse impacts to the groundwater beneath the VEGP site.

2.4.12.4 Design Basis for Subsurface Hydrostatic Loading

The design basis for subsurface hydrostatic loading for existing VEGP Units 1 and 2 is El. 165 ft msl. For new VEGP Units 3 and 4, the design basis for groundwater-induced loadings on subsurface portions of safety-related structures, systems, and components is also El. 165 ft msl as discussed in Section 2.5.4.6. Note that the lowest elevation of a safety-related structure, system, or component is El. 180.5 ft msl (bottom elevation of the containment building slab). This elevation is about 20 to 30 ft above the highest water table elevation recorded in the power block area based on the contours plotted in Figures 2.4.12-7 through 2.4.12-11 and Figure 2.4.12-24. Because the subsurface portions of all safety-related structures, systems, and components are well above the highest recorded water table elevations, there will be no groundwater-induced loadings. No permanent dewatering system will be required to lower the design basis groundwater level. No wells will be used for safety-related purposes.

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Table 2.4.12-1 Monthly Groundwater Level Elevations in the Water Table Aquifer

Well No.	Groundwater Level Elevation (ft msl)																	
	Jun-05	Jul-05	Aug-05	Sep-05	Oct-05	Nov-05	Dec-05	Jan-06	Feb-06	Mar-06	Apr-06	May-06	Jun-06	Jul-06	Aug-06	Sep-06	Oct-06	Nov-06
142	154.37	154.38	154.49	154.64	154.75	154.69	154.60	154.71	154.78	154.71	154.63	154.55	154.48	154.41	154.36	0.00	0.00	154.16
179	147.42	148.40	148.42	148.72	148.69	148.75	148.52	148.61	148.64	148.72	148.66	148.76	148.78	148.56	148.75	0.00	0.00	148.79
802A	157.88	157.86	158.07	158.23	158.29	158.34	158.28	158.28	158.39	158.23	158.17	158.09	157.99	157.91	157.89	0.00	0.00	157.56
803A	159.98	159.91	160.15	160.32	160.39	160.48	160.39	160.37	160.48	160.45	160.30	160.20	160.12	159.96	159.88	0.00	0.00	159.64
804	163.73	163.62	163.92	164.10	164.21	164.23	164.05	164.08	164.23	164.30	164.11	163.99	163.88	163.69	163.69	0.00	0.00	162.84
805A	158.53	158.57	158.84	158.98	159.09	159.09	159.05	158.94	158.92	158.98	158.82	158.82	158.63	158.53	158.45	0.00	0.00	158.19
806B	155.62	155.65	155.78	155.90	155.96	155.98	155.88	155.97	155.98	156.03	155.85	155.78	155.73	155.68	155.62	0.00	0.00	155.42
808	158.88	159.14	159.42	159.55	159.49	159.37	159.15	159.04	159.19	159.15	158.99	158.53	158.80	158.72	158.65	0.00	0.00	158.40
809	152.78	152.70	152.75	152.89	152.98	152.97	152.98	153.10	153.22	153.18	153.05	153.02	153.00	152.88	152.86	0.00	0.00	152.71
LT-1B	154.92	154.82	155.01	155.16	155.18	155.22	155.06	155.18	155.52	155.28	155.18	155.15	154.95	154.95	154.95	0.00	0.00	154.78
LT-7A	154.39	154.15	154.33	154.46	154.48	154.46	154.31	154.57	154.83	154.59	154.57	154.50	154.41	154.30	154.34	0.00	0.00	154.25
LT-12	158.21	157.90	158.07	158.22	158.31	158.28	158.21	158.53	158.66	158.48	158.54	158.48	158.23	158.19	158.18	0.00	0.00	158.11
LT-13	156.10	155.92	156.13	156.30	156.32	156.37	156.23	156.36	156.66	156.35	156.32	156.32	156.23	156.08	156.14	0.00	0.00	155.93
OW-1003	155.94	155.89	156.06	156.29	156.24	156.36	156.26	156.34	156.37	156.43	156.32	157.24	156.16	156.03	155.98	0.00	0.00	155.90
OW-1005	132.95	132.73	132.88	133.01	132.67	132.65	132.53	132.74	133.04	133.12	133.14	133.20	133.12	132.94	132.84	0.00	0.00	132.50
OW-1006	147.66	147.48	147.57	147.60	147.49	147.20	147.18	147.41	147.40	147.37	147.35	147.12	147.05	146.88	146.80	0.00	0.00	146.47
OW-1007	151.82	151.72	151.78	151.63	151.45	151.15	151.05	151.41	151.49	151.45	151.22	151.11	150.99	150.76	150.53	0.00	0.00	150.08
OW-1009	162.38	162.40	162.71	162.90	163.01	163.03	162.87	162.93	163.01	163.01	162.89	162.79	162.65	162.50	162.44	0.00	0.00	162.17
OW-1010	163.06	163.26	163.59	163.77	163.81	163.78	163.62	163.60	163.63	163.57	163.44	163.29	163.09	162.91	162.84	0.00	0.00	162.51
OW-1012	161.83	161.93	162.07	162.06	161.98	161.80	161.71	161.82	161.86	161.80	161.68	161.53	161.37	161.22	161.00	0.00	0.00	160.49
OW-1013	164.95	165.00	165.29	165.47	165.48	165.42	165.21	165.29	165.46	165.31	165.23	165.11	164.96	164.79	164.68	0.00	0.00	164.25
OW-1015	159.63	159.58	159.78	159.90	159.96	159.96	159.82	159.81	159.79	159.89	159.75	159.66	159.58	159.45	159.35	0.00	0.00	159.06

Note.
Groundwater level data for the period between June 2005 and February 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00027, SNC ALWR ESP Project. **(Bechtel Power Corporation, March 2006).**
Groundwater level data for the period between March 2006 and June 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00038, SNC ALWR ESP Project. **(Bechtel Power Corporation, June 2006).**
Groundwater level data for the period between July 2006 and November 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00039, SNC ALWR ESP Project **(Bechtel Power Corporation, November 2006).**

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Table 2.4.12-2 Monthly Groundwater Level Elevations in the Tertiary Aquifer

	Groundwater Level Elevation (ft msl)																	
Well No.	Jun-05	Jul-05	Aug-05	Sep-05	Oct-05	Nov-05	Dec-05	Jan-06	Feb-06	Mar-06	Apr-06	May-06	Jun-06	Jul-06	Aug-06	Sep-06	Oct-06	Nov-06
27	91.50	89.96	91.63	83.96	82.13	88.24	82.57	84.62	85.77	84.49	83.42	83.08	83.03	84.54	84.73	0.00	0.00	81.50
29	98.88	97.80	98.33	93.17	91.86	91.89	92.59	93.97	94.19	93.63	93.05	92.16	91.76	91.86	91.44	0.00	0.00	89.97
850A	105.27	104.68	104.76	101.04	100.03	99.91	100.70	101.86	101.69	101.48	101.14	100.07	99.63	99.23	98.57	0.00	0.00	97.56
851A	114.54	114.40	114.02	111.59	111.38	110.60	112.34	112.32	112.43	112.42	112.23	111.08	110.36	109.31	108.00	0.00	0.00	107.77
852	114.71	114.49	114.00	111.88	111.09	111.21	111.88	113.06	113.51	113.14	112.82	111.74	110.38	108.78	107.20	0.00	0.00	108.35
853	108.60	108.17	107.98	104.51	103.64	103.45	104.18	105.32	105.14	104.97	104.65	103.58	103.15	102.57	101.86	0.00	0.00	101.13
854	107.06	106.88	106.65	103.37	102.38	102.23	102.38	104.13	103.85	103.73	103.45	102.31	101.86	101.31	100.57	0.00	0.00	99.87
855	102.63	101.74	102.00	97.22	96.08	96.21	96.85	98.43	98.48	98.15	97.53	96.75	95.93	95.85	94.96	0.00	0.00	94.12
856	114.07	113.94	113.49	111.37	110.57	110.63	111.31	112.52	112.46	112.39	112.07	111.21	109.94	108.36	106.75	0.00	0.00	107.75
OW-1002	120.76	120.61	120.04	118.65	117.81	117.71	118.44	119.36	119.63	119.64	119.43	118.37	117.65	116.45	114.48	0.00	0.00	114.77
OW-1004	108.27	108.14	108.01	105.06	104.05	103.75	104.51	105.56	105.38	105.28	105.12	103.88	103.54	102.81	102.06	0.00	0.00	101.26
OW-1008	126.06	127.99	125.09	124.24	123.49	123.51	124.19	125.10	125.46	125.54	125.21	124.33	123.42	122.18	119.64	0.00	0.00	120.42
OW-1011	122.50	122.38	121.49	120.37	119.59	119.73	120.46	121.41	121.64	121.70	121.48	120.47	119.37	117.67	115.35	0.00	0.00	116.59
OW-1014	111.18	111.00	110.74	108.34	107.34	107.11	107.81	108.87	108.73	108.75	108.66	107.41	106.94	105.98	104.86	0.00	0.00	104.44

Note.
Groundwater level data for the period between June 2005 and February 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00027, SNC ALWR ESP Project. **(Bechtel Power Corporation, March 2006).**
Groundwater level data for the period between March 2006 and June 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00038, SNC ALWR ESP Project. **(Bechtel Power Corporation, June 2006).**
Groundwater level data for the period between July 2006 and November 2006 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00039, SNC ALWR ESP Project. **(Bechtel Power Corporation, November 2006).**

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Table 2.4.12-3 Hydraulic Conductivity Values

Observation Well No.	Depth Test Interval	Aquifer	Material	Hydraulic Conductivity	
	(ft)			(cm/sec)	(ft/day)
OW-1003	72 - 91	Water Table	Reddish brown silty SAND (SM) with Light tan silty SAND with Tan and grey clayey COQUINA.	4.4E-05	0.12
OW-1005	143 - 169	Water Table	Pale yellow, silty SAND, calcareous (SM), fine-coarse-grained with shell pieces.	1.1E-04	0.32
OW-1006	113 - 136	Water Table	Very light tan silty SAND (SM) with light gray COQUINA, unconsolidated (OW-1006A). Tan sandy and shelly CLAY (CH), saturated with light tan, fine-coarse grained SAND with shell (SW) (OW-1006).	4.8E-04	1.4
OW-1007	99 - 120	Water Table	Tan fine-grained silty SAND (SM), saturated with very light tan silty SAND (SM) becoming shelly with light olive grey CLAY (CH).	9.3E-04	2.65
OW-1009	81 - 98	Water Table	Very light tan silty SAND (SM) with Tan limestone shell hash, very light tan silty SAND (SM) WITH "Brown silty CLAY.	4.0E-04	1.1
OW-1010	70 - 92	Water Table	Tan poorly graded SAND with silt (SP-SM) with brownish yellow clayey silty SAND (SC-SM), soft with white SHELL HASH.	6.4E-05	0.18
OW-1012	71 - 94	Water Table	Brown SAND, fine-to-medium-grained with pale yellow silt (SM) with Pale olive silt (ML) with pale yellow SILT, micaceous (ML).	1.4E-04	0.39
OW-1013	81 - 104	Water Table	Tan fine-to-medium-grained SAND (SP-SM) with tan or clay tubes or bioturbation with light olive tan calcareous silty fine grained-grained SAND (SP-SM) with light olive tan calcareous CLAY (CL), wet but not saturated.	1.3E-04	0.38
OW-1015	90 - 120	Water Table	Grayish white, fine-to-medium-grained SAND (SP) saturated with very light tan poorly graded SAND with silt (SP-SM) with tan shelly (coarse) fine to medium grained clayey SAND (SC).	1.5E-04	0.44

Table 2.4.12-3 (cont) Hydraulic Conductivity Values

Observation Well No.	Depth Test Interval	Aquifer	Material	Hydraulic Conductivity	
OW-1002	216 - 237	Tertiary	Light greenish gray fine- to medium- grained silty, glauconitic SAND with gray clay layer (SM).	3.2E-04	0.9
OW-1004	150 - 187	Tertiary	Fine- to medium- grained dark gray SAND with organics, wet, poorly graded with silt (SP-SM).	1.3E-04	0.35
OW-1008	226 - 247	Tertiary	Gray, fine SAND (SW) with light gray fine sand (SM).	7.5E-04	2.1
OW-1011	197 - 218	Tertiary	Dark bluish-gray silty fine- to medium- grained SAND, very moist with gray, poorly graded sand with silt (SP-SM) with silty gravelly sand with fossils, shark teeth with gray medium- to coarse-grained SAND.	3.8E-04	1.1
OW-1014	179 - 197	Tertiary	Dark gray silty SAND (SM-SP), high organic content, saturated with light gray fine quartz SAND (SP), silty SAND (SM) and dark gray Sandy SILT (ML).	1.9E-04	0.54

Geometric Mean Water Table Aquifer	1.75E-04	0.5
Geometric Mean Tertiary Aquifer	2.95E-04	0.83

Note.
Hydraulic conductivity values provided in Appendix 2.5A (report Appendix D)
Material descriptions from the borings logs provided in Appendix 2.4A (report Appendix E)

Table 2.4.12-4 Summary of Laboratory Test Results on Grain Size, Moisture Content and Specific Gravity for the Barnwell Formation

Borehole / Well No.	Sample Elevation	Grain Size Distribution			Moisture Content	Specific Gravity
		Gravel	Sand	Clay/Silt		
	(ft msl)	(%)	(%)	(%)	(%)	
OW-1003	144.5	0.0	65.1	34.9	ND	2.69
OW-1003	139.5	31.1	50.0	18.4	ND	2.68
OW-1005	115.9	8.9	57.0	34.1	ND	2.63
OW-1005	110.9	18.2	47.6	34.3	ND	2.61
OW-1006	113.6	7.0	61.1	31.9	ND	2.67
OW-1006	108.6	3.6	74.4	22.0	ND	2.59
OW-1007	113.4	0.0	85.0	15.0	ND	2.65
OW-1007	108.4	0.0	85.0	18.1	ND	2.66
OW-1009	135.9	2.7	74.6	22.7	ND	2.61
OW-1009	130.9	34.7	45.9	19.2	ND	2.75
OW-1010	143.4	0.0	89.3	10.7	ND	2.67
OW-1010	138.4	0.0	63.5	36.5	ND	2.63
OW-1012	131.9	0.0	76.1	23.9	ND	2.66
OW-1012	126.9	0.0	14.1	85.9	ND	2.66
OW-1013	132.9	0.0	91.1	8.9	ND	2.65
OW-1013	122.9	0.0	91.1	8.9	ND	2.65
OW-1015	126.9	0.0	97.7	2.8	ND	2.63
OW-1015	125.4	0.0	93.2	6.8	ND	2.67
B-1002	214.3	6.6	84.0	9.4	6.2	ND
B-1002	203.5	0.0	62.9	37.1	24.4	ND
B-1002	193.5	0.0	75.1	24.9	31.8	ND
B-1002	188.5	0.0	68.4	31.6	58.8	ND
B-1002	168.5	0.0	89.5	10.5	42.9	ND
B-1002	158.5	0.0	92.8	7.2	29.3	ND
B-1002	148.5	0.4	89.6	10.0	24.5	ND
B-1002	138.5	0.0	93.9	6.1	27.6	ND
B-1003	208.2	0.0	79.1	20.9	13.4	ND
B-1003	185.2	0.0	70.2	29.8	42.1	ND
B-1003	168.2	52.2	34.4	13.4	17.5	ND
B-1003	148.2	0.0	91.8	8.2	32.3	ND
B-1004	240.8	0.0	75.6	24.4	13.8	ND
B-1004	237.8	0.7	76.2	23.1	14.5	ND
B-1004	226.3	0.2	84.9	14.9	18.5	ND
B-1004	206.3	0.0	40.0	60.0	46.2	ND
B-1004	196.3	0.0	59.0	41.0	62.9	ND
B-1004	181.3	10.5	69.6	19.9	24.1	ND
B-1004	166.3	0.0	88.5	11.5	28.8	ND
B-1004	126.3	48.6	32.2	19.2	19.7	ND
B-1006	248.5	0.0	92.7	7.3	3.8	ND
B-1006	222.5	0.1	73.8	26.1	19.7	ND

Table 2.4.12-4 (cont.) Summary of Laboratory Test Results on Grain Size, Moisture Content and Specific Gravity for the Barnwell Formation

Borehole / Well No.	Sample Elevation	Grain Size Distribution			Moisture Content	Specific Gravity
		Gravel	Sand	Clay/Silt		
	(ft msl)	(%)	(%)	(%)	(%)	
B-1006	197.5	0.0	41.7	58.3	92.8	ND
B-1006	187.5	0.1	96.8	3.1	25.4	ND
B-1006	167.5	0.0	84.3	15.7	51.9	ND
B-1006	147.5	30.7	47.8	21.5	22.0	ND
B-1010	211.1	0.0	92.2	7.8	5.7	ND
B-1010	185.1	0.0	83.0	17.0	18.9	ND
B-1010	160.1	0.0	86.7	13.3	27.3	ND
				Median	25	2.66

Note.
ND – Not Determined
OW-series data are provided in Appendix 2.4A
B-series data are provided in Appendix 2.5A
Moisture content is by weight percent.

Table 2.4.12-5 Summary of Laboratory Test Results on Grain Size, Moisture Content, and Porosity for the Lisbon Formation

Borehole / Well No.	Sample Elevation	Grain Size Distribution			Moisture Content	D50	Porosity
		Gravel	Sand	Clay/Silt			
	(ft msl)	(%)	(%)	(%)	(%)	(mm)	
B-1002	130.0	49.4	21.7	28.9	52.1	3.49	0.59
B-1002	118.5	22.9	41.2	35.9	56.5	0.26	0.56
B-1002	108.5	12.8	53.4	33.8	25.5	0.21	0.36
B-1002	98.5	53.7	21.8	24.5	13.5	7.52	0.25
B-1002	88.5	26.3	49.4	24.3	28.6	0.87	0.45
B-1003	135.2	16.5	50.1	33.4	67.4	0.43	ND
B-1003	130.2	1.6	57.8	40.6	30.6	0.14	0.46
B-1003	118.5	1.2	67.1	31.7	40.6	0.27	0.52
B-1003	101.5	11.7	45.8	42.5	28.0	0.12	0.42
B-1003	81.5	7.3	58.5	34.2	25.9	0.15	0.39
B-1004	105.8	1.0	52.7	46.3	44.6	0.10	0.56
B-1004	96.3	0.7	57.6	41.7	30.1	0.15	0.45
B-1004	86.3	38.0	29.8	32.2	25.1	0.49	0.43
B-1004	72.8	20.9	37.4	41.7	20.8	0.12	0.38
B-1004	61.3	34.9	41.3	23.8	29.0	0.85	0.44
B-1004	51.3	5.2	60.3	34.5	26.2	0.18	0.39
				Median	29	0.24	0.44

Note.
ND – Not Determined
B-series data are provided in Appendix 2.5A
Moisture content is by weight percent.
Porosity calculated assuming specific gravity of 2.65.

Table 2.4.12-6 Summary of Laboratory Test Results on Grain Size, Moisture Content, and Specific Gravity for the Still Branch And Congaree Formations

Borehole / Well No.	**Sample Elevation**	**Grain Size Distribution**			**Moisture Content**	**Specific Gravity**
		Gravel	**Sand**	**Clay/Silt**		
	(ft msl)	(%)	(%)	(%)	(%)	
OW-1002	8.9	0.2	79.6	20.2	ND	2.65
OW-1002	-9.6	0.0	1.4	90.6	ND	2.62
OW-1004	69.4	0.1	89.7	10.2	ND	2.69
OW-1004	64.4	0.0	93.4	6.6	ND	2.67
OW-1008	-11.9	0.0	83.2	16.8	ND	2.69
OW-1008	-21.9	2.2	67.9	20.3	ND	2.68
OW-1011	12.3	0.0	88.9	10.8	ND	2.67
OW-1011	-2.7	4.5	89.6	5.9	ND	2.66
OW-1014	37.4	0.0	87.8	12.2	ND	2.69
OW-1014	32.4	0.0	89.6	10.4	ND	2.66
B-1002	68.5	20.0	40.6	39.4	23.3	ND
B-1002	33.5	0.0	93.4	6.6	40.7	ND
B-1002	-16.5	3.1	84.6	12.3	18.5	ND
B-1003	57.5	0.0	94.6	5.4	23.6	ND
B-1003	37.5	0.9	82.7	16.4	32.3	ND
B-1003	17.5	1.4	77.2	21.4	39.3	ND
B-1003	-17.5	0.0	89.1	10.9	23.2	ND
B-1003	-57.5	0.3	85.5	14.2	23.2	ND
B-1003	-92.5	70.7	26.0	3.3	32.7	ND
B-1003	-127.5	0.0	21.5	78.5	21.3	ND
B-1003	-177.5	0.3	83.9	15.8	18.9	ND
B-1003	-227.5	0.0	84.1	15.9	28.6	ND
B-1003	-273.5	0.0	86.8	13.2	26.4	ND
				Median	24.0	2.67

Note.
ND – Not Determined
OW-series data are provided in Appendix 2.4A
B-series data are provided in Appendix 2.5A
Moisture content is by weight percent.

Table 2.4.12-7 Georgia EPD Permitted Municipal and Industrial Groundwater Users within 25 miles of the VEGP Site

Well ID	Permit Holder	County	Aquifer	Year	Permitted Monthly Average, gpm (mgpd)	Permitted Annual Average, gpm (mgpd)	Average Annual Water Use, gpm (mgpd)
C-2	City of Sardis	Burke	Floridan	2004	278 (0.40)	278 (0.40)	63 (0.09)
				2005	278 (0.40)	278 (0.40)	NA
C-12	East Central Regional Hospital - Gracewood Campus	Richmond	Cretaceous Sand	2004	347 (0.50)	278 (0.40)	146 (0.21)
				2005	NA	NA	76 (0.11)
C-13	City of Hephzibah	Richmond	Cretaceous Sand	2004	833 (1.20)	833 (1.20)	160 (0.23)
				2005	NA	NA	236 (0.34)
C-19	Olin Corporation	Richmond	Cretaceous Sand	2004	847 (1.22)	847 (1.22)	514 (0.74)
				2005	NA	NA	486 (0.70)
C-19	Olin Corporation - Corrective Action Wells	Richmond	Cretaceous Sand	2004	632 (0.91)	632 (0.91)	229 (0.33)
				2005	NA	NA	250 (0.36)
I-1	International Paper	Burke	Cretaceous Sand	2004	660 (0.95)	660 (0.95)	181 (0.26)
				2005	660 (0.95)	660 (0.95)	35 (0.05)
I-2	Prayon, Inc	Richmond	Cretaceous Sand	2004	292 (0.42)	264 (0.38)	35 (0.05)
				2005	NA	NA	63 (0.09)
I-3	Thermal Ceramics, Inc.	Richmond	Cretaceous Sand	2004	625 (0.90)	625 (0.90)	313 (0.45)
				2005	NA	NA	208 (0.30)
I-4	Procter & Gamble Manufacturing Company	Richmond	Cretaceous Sand	2004	486 (0.70)	486 (0.70)	278 (0.40)
				2005	NA	NA	243 (0.35)
I-5	Southern Wood Piedmont Company	Richmond	Cretaceous Sand	2004	451 (0.65)	451 (0.65)	188 (0.27)
				2005	NA	NA	174 (0.25)
M-1	City of Waynesboro	Burke	Cretaceous Sand	2004	2778 (4.00)	2431 (3.50)	NA
				2005	2778 (4.00)	2431 (3.50)	NA
M-2	Augusta-Richmond Utilities Department	Richmond	Cretaceous Sand	2004	12778 (18.40)	12083 (17.40)	8285 (11.93)
				2005	NA	NA	8.40
	Southern Nuclear Operating Co.	Burke	Cretaceous Sand	2004	4167 (6.00)	3819 (5.50)	556 (0.80)
				2005	4167 (6.00)	3819 (5.50)	583 (0.84)

Notes: NA – not available
Groundwater permit and usage data **(Voudy 2006)**
Groundwater aquifer description **(Georgia DNR 2006)**
Well locations are labeled in Figure 2.4.12-19 using the listed Well IDs.
Southern Nuclear Operating Co. well locations are shown on Figure 2.4.12-20.

Table 2.4.12-8 Georgia EPD Permitted Agricultural Groundwater Users within 25 miles of the VEGP Site

Well ID	Permit Holder	County	Depth (ft)	Permit (gpm)
A-1	ANDERSON JOHN	Burke	363	1500
A-2	BLANCHARD HENRY	Burke	500	1200
A-3	BLANCHARD HENRY	Burke	450	1400
A-4	BOLLWEEVIL PLANATION	Burke	300	190
A-5	Chance Bill	Burke	500	450
A-6	CHANDLER FARM	Burke	580	1600
A-7	Chandler Michael	Burke	556	2400
A-8	Chandler Randall	Burke	579	2500
A-9	COCHRAN IRBY	Burke	420	1350
A-10	COLLINS ROBERT	Burke	430	1350
A-11	COLLINS ROBERT	Burke	530	1200
A-12	COLLINS ROBERT	Burke	480	1100
A-13	COLLINS ROBERT	Burke	440	1100
A-14	Collins Robert	Burke	490	1700
A-15	DIXON CARL	Burke	600	2000
A-16	DIXON JAMES	Burke	210	400
A-17	DIXON JAMES	Burke	200	200
A-18	DIXON JOANNE	Burke	640	1150
A-19	DIXON PERCY	Screven	560	2000
A-20	DIXON PERCY	Burke	560	2000
A-21	DIXON PERCY	Burke	350	115
A-22	DIXON PERCY	Burke	350	115
A-23	DIXON PERCY	Burke	550	3400
A-24	DIXON PERCY	Burke	350	200
A-25	DIXON PERCY	Burke	575	2500
A-26	DIXON PERCY	Burke	550	2500
A-27	GWR Partnership LLP	Burke	360	200
A-28	Hatcher William	Burke	300	500
A-29	HEATH CLAXTON	Burke	300	150
A-30	HEATH CLAXTON	Burke	400	250
A-31	HEATWOLE BYARD	Burke	325	200
A-32	HOPKINS HENRY	Burke	363	350
A-33	Horst Isaac	Burke	260	250
A-34	MALLARD CLYDE	Burke	320	400
A-35	MALLARD CLYDE MALLARD FARMS	Burke	210	250
A-36	MALLARD J.	Burke	200	150
A-37	McGregor Charles	Burke	430	350
A-38	MOBLEY DANNY	Burke	396	350
A-39	Mobley Danny	Burke	424	650
A-40	MOBLEY HERBERT	Burke	465	1100
A-41	MOBLEY HERBERT	Burke	500	1250
A-42	MOBLEY JAMES F.	Burke	572	2000
A-43	PENNINGTON FARMS- INC.	Burke	240	250
A-44	RAYMOND NEIL	Burke	430	1350

Table 2.4.12-8 (cont.) Georgia EPD Permitted Agricultural Groundwater Users within 25 miles of the VEGP Site

Well ID	Permit Holder	County	Depth (ft)	Permit (gpm)
A-45	Shepherd Joseph	Burke	421	1500
A-46	SMART DARRELL	Burke	300	350
A-47	SMART DARRELL	Burke	300	350
A-48	SMART DARRELL	Burke	300	350
A-49	SMART DARRELL	Burke	300	400
A-50	MIMS JOHN	Jenkins	445	1500
A-51	MIMS JOHN	Jenkins	460	1500
A-52	MULKEY A.	Jenkins	300	1000
A-53	MULKEY A.	Jenkins	400	500
A-54	PARKER GEORGE	Jenkins	450	700
A-55	PARKER GEORGE	Jenkins	300	450
A-56	PARKER GEORGE	Jenkins	300	450
A-57	Parker George	Jenkins	450	450
A-58	POINTE SOUTH GOLF CLUB- INC.	Richmond	311	400
A-59	BRAGG SOL	Screven	380	240
A-60	BRIAR CREEK COUNTRY CLUB	Screven	180	300
A-61	CAIN BRIAN	Screven	390	600
A-62	Cain Brian	Screven	493	1100
A-63	CLEMENT INVESTMENTS	Screven	282	1250
A-64	FOREHAND FARMS	Screven	160	250
A-65	Lee Mike	Screven	480	1800
A-66	Mill Haven Company Inc.	Screven	600	1200
A-67	MILLHAVEN CO.- INC.	Screven	553	1900
A-68	MILLHAVEN CO.- INC.	Screven	565	1400
A-69	NEWTON JAMES	Screven	350	400
A-70	SOWELL CAROLYN	Screven	275	300
A-71	STEPONGZI FRANK & PEARL	Screven	225	300
A-72	THOMPSON JAMES	Screven	475	750
A-73	THOMPSON ROGER	Screven	500	1000
A-74	WADE PLANTATION	Screven	215	200
A-75	WADE PLANTATION	Screven	250	190
A-76	WADE PLANTATION	Screven	460	1200
A-77	WADE PLANTATION	Screven	119	1000
A-78	WADE PLANTATION	Screven	750	1800
A-79	WADE PLANTATION	Screven	494	900
A-80	WADE PLANTATION	Screven	475	1200
A-81	WADE PLANTATION	Screven	672	1100
A-82	WADE PLANTATION	Screven	475	1100
A-83	WADE PLANTATION	Screven	525	1400
A-84	Wade Plantation	Screven	467	1100

Notes: Groundwater permit data **(Lewis 2006)**
Well locations are labeled in Figure 2.4.12-19 using the listed Well IDs.

Table 2.4.12-9 SDWIS Listed Public Water Systems Supplied From Groundwater Within 25 Miles of the VEGP Site in Georgia

Well ID	Water System ID	Water System Name	County Served	Type	System Status
C-1	GA0330000	Girard	Burke	Community	Active
C-2	GA0330002	Sardis	Burke	Community	Active
C-3	GA0330013	Mamie Joe Rhodes Harrison Subdivision	Burke	Community	Closed
C-4	GA0330006	Burke Academy	Burke	Non-Transient Non-Community	Active
C-5	GA0330022	Burke County Training Center	Burke	Non-Transient Non-Community	Active
C-6	GA0330020	Delaigle Mobile Home Park	Burke	Transient Non-Community	Closed
C-7	GA1650000	Millen	Jenkins	Community	Active
C-8	GA1650001	Perkins Water Authority	Jenkins	Community	Active
C-9	GA1650006	Jockey International, Inc.	Jenkins	Non-Transient Non-Community	Active
C-10	GA1650005	DNR - Magnolia Springs State Pk.	Jenkins	Transient Non-Community	Active
C-11	GA1650008	National Fish Hatchery	Jenkins	Transient Non-Community	Closed
C-12	GA2450023	East Central Regional Hospital	Richmond	Community	Active
C-13	GA2450002	Hephzibah	Richmond	Community	Active
C-14	GA2450017	Hephzibah - Oakridge	Richmond	Community	Active
C-15	GA2450014	Mars Trailer Park	Richmond	Community	Active
C-16	GA2450016	Mobile Home Country Club MHP	Richmond	Community	Active
C-17	GA2450004	Richmond County	Richmond	Community	Closed
C-18	GA2450159	Albion Kaolin Company	Richmond	Non-Transient Non-Community	Closed
C-19	GA2450152	Olin Chemicals	Richmond	Non-Transient Non-Community	Closed
C-20	GA2510000	Hiltonia	Screven	Community	Active
C-21	GA2510015	Buck Creek M.H.P.	Screven	Community	Closed
C-22	GA2510052	Millhaven Plantation	Screven	Community	Closed
C-23	GA2510011	DOT - Georgia Welcome Center	Screven	Transient Non-Community	Active
C-24	GA2510057	Savannah River Challenge Program	Screven	Transient Non-Community	Active
	GA0330035	Southern Nuclear - Simulator Bld	Burke	Non-Transient Non-Community	Active
	GA0330017	Southern Nuclear - Vogtle Makeup	Burke	Non-Transient Non-Community	Active
	GA0330036	Southern Nuclear - Vogtle Rec	Burke	Transient Non-Community	Active

Notes: US EPA SDWIS Database **(EPA 2006b)**
Well locations are labeled in Figure 2.4.12-19 using the listed Well IDs.
Southern Nuclear Operating Co. well locations are shown on Figure 2.4.12-20.

Table 2.4.12-10 Water-Supply Wells for the Existing VEGP Plant

Water Supply Well No.	Well Depth (ft)	Aquifer	Design Yield (gpm)	Water Use
MU-1	851	Cretaceous	2000	Make-up water for plant use (nuclear service water system; make-up to the water treatment plant demineralizer, and potable water source).
MU-2A	884	Cretaceous	1000	Make-up water for plant use (nuclear service water system; make-up to the water treatment plant demineralizer, and potable water source).
TW-1	860	Cretaceous	1000	Back-up water for the production make-up well system.
SW-5	200	Tertiary	20	Water supply for old security tactical training area.
IW-4	370	Tertiary	120	Irrigation water for ornamental vegetation.
CW-3	220	Tertiary	NA	Water supply for nuclear operations garage.
REC	265	Tertiary	150	Potable water supply for recreation area.
SB	340	Tertiary	50	Potable water supply for simulator training building.
SEC	320	Tertiary	10	Non-potable water for lavatory use at a new plant entrance security building

Notes: NA – not available
Water supply well data (excluding SEC well) **(SNS 2005b)**
SEC well data **(SNS 2005a)**
Well locations, excluding Well REC, are shown on Figure 2.4.12-20. Well REC is located approximately 9300 ft southwest from Well IW-4.

Table 2.4.12-11 Groundwater Use of the existing VEGP Plant from January 1, 2005 to December 31, 2005, gpm (Thousands of Gallons)

Month	Well MU-1	Well MU-2A	Well TW-1	Well SW-5	Well IW-4	Well CW-3	Well REC	Well SB
January	445 (19,209)	0	0	0	0	0.07 (3)	0.88 (38)	0.05 (2)
February	403 (17,416)	0	0	0	0	0.05 (2)	1.16 (50)	1.34 (58)
March	500 (21,601)	0	0	0	0	0.05 (2)	0.95 (41)	1.25 (54)
April	607 (26,211)	0	0	0	0	0.02 (1)	1.09 (47)	1.5 (65)
May	686 (29,648)	0	0	0	0	0.05 (2)	1.55 (67)	1.74 (75)
June	825 (35,625)	0	0	0	0.32 (14)	0.05 (2)	0.97 (42)	1.92 (83)
July	552 (23,846)	0	0	0	1.27 (55)	0.05 (2)	2.89 (125)	2.73 (118)
August	569 (24,560)	0	0	0	2.92 (126)	0.14 (6)	2.41 (104)	1.53 (66)
September	649 (28,020)	0	0	0	3.1 (134)	0.09 (4)	1.94 (84)	1.6 (69)
October	701 (30,290)	0	0	0	0	0.07 (3)	1.83 (79)	1.13 (49)
November	469 (20,282)	67 (2,880)	0	0	0	0.05 (2)	1.67 (72)	2.41 (104)
December	610 (26,363)	0	0	0	0	0.05 (2)	0.95 (41)	3.7 (160)
Total	7016 (303,071)	67 (2,880)	0	0	7.62 (329)	0.72 (31)	18.26 (789)	22.55 (974)
Monthly Average	585 (252,56)	6 (240)	0	0	0.625 (27)	0.07 (3)	1.53 (66)	1.88 (81)

Notes: Groundwater use data from Southern Nuclear Operating Company
SEC well is active in 2006

Table 2.4.12-12 Projected Groundwater Use for Two AP1000 Units

Water Use	Normal Case (gpm)	Maximum Case (gpm)
Service Water System Make-up	537	2353
Potable Water System	42	140
Demineralized Water System	150	600
Fire Protection System	10	12
Miscellaneous Users	13	35
Total	752	3140

Table 2.4.12-13 Presence of Utley Limestone in the VEGP ESP Site Borings

Boring	Northing	Easting	Utley Limestone
B-1001	1,142,661.92	620,220.42	Present
B-1002	1,142,998.52	620,985.47	Absent
B-1003	1,142,974.36	621,889.85	Present
B-1004	1,142,985.41	620,131.44	Present
B-1005	1,143,991.57	620,155.35	Present
B-1006	1,143,810.26	621,342.90	Absent
B-1007	1,142,662.29	621,120.13	Present
B-1008	1,142,670.93	621,996.15	Present
B-1009	1,141,000.54	620,361.26	Absent
B-1010	1,141,000.12	621,279.68	Absent
B-1011	1,143,741.13	622,378.01	Present
B-1013	1,140,976.08	622,272.50	Absent
OW-1006	1,143,817.85	619,179.75	Present
OW-1008	1,142,347.94	619,306.69	Present
OW-1009	1,141,891.65	620,888.61	Present
OW-1012	1,139,969.50	621,045.92	Absent
OW-1013	1,140,805.40	621,715.03	Absent
OW-1015	1,140,550.58	623,086.32	Absent

Note.
B-series data are provided in Appendix 2.5A
OW-series data are provided in Appendix 2.4A

Table 2.4.12-14 Summary of Holes Drilled at the Site for the Installation of Observation Wells

Boring / Drill Log No.	Drilling Method	Drill Dates		Sampled Depth		Drilled Depth Below the GS	Boring "Abandoned" or “Well” Installed
		Start	End	From (ft)	To (ft)	(ft)	
OW-1001A	3.25" HSA	25-May	25-May	No sampling		100	Abandoned
OW-1001	4.25" HSA	24-May	29-May	113.5	140	140	Well
OW-1002A	3.25" HSA	24-May	25-May	0	108.5	108.5	Abandoned
OW-1002	Rotosonic	2-Jun	6-Jun	87	237	237	Well
OW-1003A	3.25" HSA	24-May	24-May	0	90	90	Abandoned
OW-1003	4.25" HSA	25-May	25-May	No sampling		90.5	Well
OW-1004	Rotosonic	3-Jun	11-Jun	87	187	187	Well
OW-1005A	3.25" HSA	31-May	31-May	0	75	75	Abandoned
OW-1005	4.25" HSA	2-Jun	7-Jun	68.5	170	170	Well
OW-1006A	4.25" HSA	3-Jun	4-Jun	0	125	125	Abandoned
OW-1006	4.25" HSA	9-Jun	14-Jun	118.5	135	135	Well
OW-1007	4.25" HSA	4-Jun	7-Jun	98.5	122	122	Well
OW-1008A	3.25" HSA	26-May	26-May	0	107.5	105	Well OW-1008
OW-1008	Rotosonic	31 May	1-Jun	108	247	247	Well
OW-1009	4.25" HSA	24-May	27-May	0	100	100	Well
OW-1010	4.25" HSA	1-Jun	1-Jun	0	93.5	93.5	Well
OW-1011	Rotosonic	11-Jun	12-Jun	87	217	217	Well
OW-1012	4.25" HSA	31-May	1-Jun	0	93.6	93.6	Well
OW-1013	4.25" HSA	9-Jun	10-Jun	0	103.5	103.5	Well
OW-1014	Rotosonic	11-Jun	11-Jun	97	197.4	197.4	Well
OW-1015	4.25" HSA	30-May	3-Jun	0	120	120	Well

Note.
Borings OW-1001A, OW-1002A, OW-1003A, and OW-1005A were abandoned due to the use of 3.25-in hollow stem auger, which would not adequately accommodate well installation.
Boring OW-1006A was abandoned due to the of shortage hollow stem auger flights.
Boring OW-1008A is the upper portion of boring OW-1008 and was not abandoned. The “A” is designated to show that the upper portion of this boring was drilled using 3.25-in hollow-stem augers while the lower portion was drilled using the rotosonic drilling method.
Boring log OW-1003 contained in Appendix 2.4A (report Appendix E) should read OW-1003A.
The drilling method for boring OW-1006 is assumed to be 4.25" HSA (not described in Appendix 2.4A (report Appendix E)).

Table 2.4.12-15 Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
23-Oct-71		154.3											
2-Nov-71		156.8											
10-Nov-71		160.3											
17-Nov-71		160.8											
23-Nov-71		161.1											
1-Dec-71		162.1											
7-Dec-71		162.4											
14-Dec-71		164.3											
23-Dec-71		164.6											
29-Dec-71		165.8											
5-Jan-72		166.1											
12-Jan-72		167.3											
19-Jan-72		168.1											
26-Jan-72		168.5											
3-Feb-72		168.6											
9-Feb-72		168.9											
23-Feb-72		169.8											
2-Mar-72		170.1											
9-Mar-72		170.3											
16-Mar-72		167.9											
21-Mar-72		170.2											
18-Apr-72		171.9											
1-May-73		174.1											
30-May-73		173.6											
27-Jul-73		172.3											
13-Oct-73		170.8											
3-Nov-73		170.4											
9-Dec-73		170.1											
7-Jan-74		168.9											

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
10-Feb-74		166.6											
23-Mar-74		168.1											
17-Apr-74		167.4											
15-Aug-74		165.3											
11-Sep-74		165.1											
7-Jul-79		160.2		155.5	161.2	152.4							
26-Nov-79		161.8		155.1		153.0							
2-Jan-1980				155.1	161.2	152.9				137.2	141.6		
11-Jan-1980				155.1						136.8	141.7		
24-Jan-1980		161.0		154.9	161.0	138.2				136.8	141.6		
1-Feb-1980				154.9		138.5				136.5	141.1		
15-Feb-1980				155.0						136.6	141.2		
25-Mar-1980		157.9		154.7	161.0					136.2	142.1		
27-Jun-1980		162.0			161.4	137.5				137.0	140.6		
2-Sep-1980										136.4	139.0		
27-Sep-1980		161.7		154.7	161.1	153.3							
1-Dec-1980										135.6	140.2		
29-Dec-1980		161.1		154.4	160.9								
2-Mar-1981										135.8			
28-Mar-1981		159.3		154.0	160.3								
2-Apr-1981											139.7		
1-Jun-1981										135.4			
29-Jun-1981		158.0		153.6									
2-Jul-1981											139.5		
24-Dec-1981											140.2		
7-Feb-1982											139.6		
23-Mar-1982		158.8		152.6	159.1	150.8							
15-Jun-1982		158.8		152.4	159.0	151.0				135.6			
9-Jul-1982											140.7		

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
15-Sep-1982		159.5		152.7	158.7	151.9							
20-Sep-82										137.0			
11-Dec-82	146.1	160.1		152.6	159.0	153.7							
18-Dec-82										135.1			
8-Mar-83	146.3				158.8	153.6							
9-Mar-83		159.6		152.6									
15-Mar-83										140.9			
22-Jun-83	152.3	159.7		155.1	159.0	156.1	152.7			151.4	149.9		
15-Sep-83	153.3	159.7		156.5			154.5						
21-Sep-83					159.7	156.8							
3-Oct-83											154.2		
15-Oct-83										153.8			
12-Dec-83	154.4	160.4		157.7	160.0	157.9	155.4						
14-Dec-83										156.4	155.9		
12-Mar-84	155.1					158.5	156.2						
13-Mar-84		159.9		158.2	160.1								
22-Mar-84										156.1	156.6		
11-Jun-84				158.9	160.5	159.9							
12-Jun-84		155.8					157.1			157.4	157.4		
13-Sep-84				159.8									
16-Sep-84					161.0								
18-Sep-84	156.5	150.9				160.6	157.4				157.7		
13-Dec-84	155.9	151.1		159.9	160.2	160.1	157.1			157.0			
31-Dec-84											158.0		
4-Feb-85	155.7	148.9		159.6	160.9	159.9	157.0			157.1			
30-Jun-85	155.5	150.2		159.6	161.0	159.5	156.9			152.0	152.0		
7-Jul-85	155.3	148.5		159.5	160.8	159.3	156.6	159.2	155.5	157.0		158.5	157.6
16-Jul-85	155.3	150.0		159.4	160.8	159.3	156.7	159.2	152.7	155.2	158.0	160.2	157.5
23-Jul-85	155.2	150.3		159.5	160.8	159.3	156.7	159.3	152.8	155.2	158.1	160.0	157.6

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

	Observation Well and Water Level Elevations (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
31-Jul-85	155.3	150.6		159.5	160.9	159.3	156.8	159.8	152.8	155.2	158.0	160.0	157.5
7-Aug-85	155.4	148.6		159.4	160.9	159.3	157.0	160.0	152.8	155.3	158.1	160.2	157.7
14-Aug-85	155.3	148.6		159.4	160.8	159.2	156.2	160.3	152.7	155.3	158.0	160.2	157.7
21-Aug-85	155.4	148.6		159.4	160.8	159.3	157.1	160.4	152.8	157.2	158.1	160.4	157.8
28-Aug-85	155.6	148.8		159.5	160.9	159.4	157.2	160.5	152.5	157.3	158.2	160.5	157.7
4-Sep-85	155.5	148.8	159.0	159.6	161.0	159.6	157.2	160.4	152.8	157.4	158.3	160.8	157.8
11-Sep-85	155.5	148.8	159.0	159.5	161.0	159.6	157.2	160.6	152.9	157.4	158.4	161.1	158.1
18-Sep-85	155.4	148.8	159.0	159.5	160.8	159.5	157.2	160.5	152.8	157.3	158.4	161.1	158.0
25-Sep-85	155.6	148.8	159.0	159.3	160.9	159.6	157.3	160.4	152.9	157.5	158.5	161.4	158.1
6-Oct-85	155.6	148.8	159.1	159.6	160.9	159.7	157.3	160.3	152.9	157.5	158.5	161.5	158.1
9-Oct-85	155.5	148.8	159.0	159.6	160.9	159.6	157.2	160.2	152.9	157.3	158.3	161.3	158.0
16-Oct-85	155.5	148.8	159.2	159.7	160.8	159.6	157.4	160.3	152.9	157.6	158.7	161.5	158.2
23-Oct-85	155.5	148.8	159.1	159.7	160.7	159.7	157.3	160.2	152.9	157.5	158.8	161.5	158.3
30-Oct-85	155.7	148.8	159.2	159.8	161.1	159.9	157.5	160.2	153.0	157.7	159.0	162.0	158.5
6-Nov-85	155.5	148.7		159.5	160.8	159.7	157.2	160.1	152.9	157.4	158.5	161.6	158.4
13-Nov-85	155.5	148.8		159.5	161.0	159.8	157.2	160.1	152.9	157.3	158.5	161.5	158.0
20-Nov-85	155.6	148.9	159.2	159.8	161.0	159.7	157.3	160.2	153.1	157.4	158.5	161.5	158.1
27-Nov-85	155.6	148.8	159.1	159.6	160.6	159.8	157.4	160.1	153.0	157.6	158.7	161.6	158.1
4-Dec-85	155.7	148.8	159.1	159.7	160.8	159.6	157.4	160.1	153.0	157.5	158.5	161.3	158.4
11-Dec-85	155.8	148.8	159.2	159.9	161.1	159.9	157.6	160.3	153.0	157.8	158.8	161.6	158.3
18-Dec-85	155.8	148.8	159.2	159.7	160.9	159.9	157.6	160.4	153.0	157.7	158.9	161.5	158.3
28-Dec-85	155.9	148.8	159.3	159.8		159.9	157.7	160.5	153.0	157.8	158.6	161.6	158.6
2-Jan-86	156.0	148.9	159.4	159.8	161.0	159.8	157.7	160.5	153.1	157.8	158.6	161.6	158.4
10-Jan-86	156.1	148.9	159.6	160.0	161.4	159.7	157.9	160.5	153.3	158.2	158.8	161.8	158.3
15-Jan-86	155.7	148.7	159.4	159.8	160.7	159.8	157.7	160.6	152.9	157.9	158.8	161.9	158.7
22-Jan-86	156.0	148.8	159.4	159.8	161.0	160.0	157.2	160.5	153.1	157.8			158.7
29-Jan-86	156.0	148.8	159.5	160.0	161.2	160.2	157.7	160.5	153.1	157.9	159.2	161.8	158.8
5-Feb-86	156.0	148.7	159.5	159.9	161.1	160.1	157.6	160.6	153.0	157.9	159.2	162.0	158.6
12-Feb-86	155.9	148.8	159.4	159.9	160.9	160.0	157.6	160.5	153.0	157.7	158.8	161.5	158.8

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
19-Feb-86	156.0	148.8	159.6	160.0	161.2	160.2	157.7	160.4	153.1	157.9	159.1	162.0	158.8
26-Feb-86	156.0	148.9	159.8	160.3	161.2	160.5	157.9	160.3	153.1	158.2	159.6	162.4	158.7
5-Mar-86	155.8	148.7	159.4	159.9	161.0	160.1	157.5	160.3	153.0	157.7	158.9	161.7	158.7
15-Mar-86	156.1	148.8	159.7	160.2	161.5	160.1	157.8	160.3	153.3	157.7	159.0	161.8	158.6
19-Mar-86	155.8	148.8	159.4	160.0	161.1	160.1	157.5	160.2	153.1	157.6	158.9	161.5	158.4
26-Mar-86	155.8	148.8	159.4	160.1	161.4	160.3	157.5	160.1	153.0	157.7	158.9	161.6	158.5
2-Apr-86	155.9	148.7	159.6	160.3	161.4	160.4	157.6	160.1	153.2	157.8	159.0	161.7	158.5
9-Apr-86	155.9	148.8	159.6	160.1	161.3	160.2	157.6	160.2	153.1	157.9	159.2	161.9	158.7
16-Apr-86	155.7	148.7	159.8	160.3	161.1	160.3	157.4	160.1	153.1	157.5	158.7	161.4	158.1
23-Apr-86	155.9	148.8	159.5	160.2	161.4	160.0	157.5	160.2	153.2	157.7	158.9	161.5	158.7
30-Apr-86	155.8	148.8	159.4	160.1	161.4	160.2	157.4	160.1	153.1	157.7	158.8	161.5	158.5
7-May-86	155.7	148.7	159.4	160.1	161.2	160.2	157.5	160.0	153.0	157.4	158.5	161.2	158.3
14-May-86	155.7	148.8	159.3	160.1	161.3	160.1	157.3	160.0	153.1	157.6	158.8	161.3	158.9
21-May-86	155.8	148.8	159.4	160.1	161.3	160.2	157.4	159.9	153.1	157.6	158.8	161.5	158.4
28-May-86	155.7	148.8	159.4	160.1	161.4	160.2	157.3	159.9	153.1	157.5	158.7	161.3	158.2
4-Jun-86	155.7	148.7	159.3	160.0	161.2	160.0	157.2	159.9	153.1	157.3	158.4	161.0	158.3
11-Jun-86	155.7	148.8	159.4	159.9	161.3	160.0	157.2	159.8	153.0	157.4	158.6	161.4	158.2
18-Jun-86	155.9	148.8	159.3	160.0	161.1	160.0	157.3	159.8	153.1	157.5	158.7	161.1	158.2
25-Jun-86	155.8	148.8	159.4	160.0	160.9	159.6	157.3	159.7	153.1	157.5	158.6	161.2	158.2
2-Jul-86	155.8	148.8	159.3	160.0	161.4	160.0	157.3	159.7	153.1	157.5	158.6	161.1	158.2
9-Jul-86	155.7	148.7	159.2	160.0	161.4	160.0	157.2	159.7	153.0	157.4	158.5	161.0	158.1
16-Jul-86	155.7	148.7	159.2	159.9	160.9	159.9	157.2	159.7	153.0	157.3	158.4	160.9	158.2
23-Jul-86	155.6	148.7	159.0	159.9	161.2	159.9	157.1	159.6	153.0	157.2	158.3	160.7	158.2
30-Jul-86	155.7	148.7	159.0	159.9	161.2	159.9	157.2	159.6	153.0	157.2	158.3	160.9	158.2
6-Aug-86	155.7	148.8	159.3	160.0	161.3	160.0	157.2	159.6	153.1	157.3	158.3	160.8	157.9
13-Aug-86	155.6	148.8	159.0	159.9	161.2	159.9	157.1	159.5	153.0	157.3	158.4	160.8	158.0
20-Aug-86	155.6	148.8	159.1	159.9	161.1	159.9	157.1	159.5	153.0	157.2	158.2	160.6	158.1
27-Aug-86	155.6	148.8	159.1	159.9	161.2	159.8	157.0	159.4	153.0	157.2	158.3	160.7	157.9
3-Sep-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.6	153.0	157.3	158.3	160.7	158.0

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
10-Sep-86	155.6	148.7	159.1	159.9	161.2	159.8	157.1	159.6	152.9	157.3	158.3	160.7	157.9
17-Sep-86	155.5	148.7	159.0	159.9	161.0	159.8	157.0	159.7	152.9	157.4	158.5	160.5	157.8
24-Sep-86	155.5	148.7	159.0	159.8	161.0	159.8	157.0	159.9	152.9	157.6	158.2	160.5	158.0
1-Oct-86	155.7	148.8	158.9	159.9	161.0	159.9	157.0	159.9	153.0	157.6	158.3	160.7	157.8
11-Oct-86	155.6	148.8	159.0	160.0	161.1	159.9	157.0	159.8	152.9	157.1	158.1	160.5	157.9
15-Oct-86	155.5	148.8	159.1	159.9	161.1	159.9	157.1	159.9	152.9	157.0	158.2	160.5	158.0
22-Oct-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.8	153.0	157.0	158.2	160.5	157.7
29-Oct-86	155.5	148.8	159.0	159.8	160.9	159.8	157.1	159.9	152.9	156.9	158.2	160.6	157.9
5-Nov-86	155.6	148.8	159.1	159.6	161.2	159.9	157.2	159.8	153.0	157.2	158.2	160.7	158.0
12-Nov-86	155.6	148.8	159.1	159.6	161.1	159.8	157.2	159.7	153.0	157.2	158.3	160.6	157.9
19-Nov-86	155.5	148.8	159.2	159.7	160.9	160.0	157.3	159.8	152.8	157.5	158.6	160.9	158.0
26-Nov-86	155.6	148.8	159.2	159.6	160.9	159.9	157.2	159.6	152.9	157.3	158.3	160.7	158.2
3-Dec-86	155.6	148.8	159.0	159.7	160.9	160.0	157.2	159.6	152.8	157.1	158.0	160.5	157.9
31-Dec-86	155.9	148.8	159.0	159.8	160.9	159.8	157.5	159.4	153.0	157.6	158.6	160.8	158.1
10-Jan-87	156.0	148.9	159.1	160.1	160.9	160.1	157.8	159.3	153.1	158.0	158.9	161.2	158.1
14-Jan-87	156.0	148.8	159.2	160.1	160.8	160.0	157.6	159.1	153.1	158.1	159.1	161.3	158.3
21-Jan-87	155.9	148.7	159.3	160.1	160.8	159.9	157.5	159.2	152.8	158.2	159.1	161.4	158.4
28-Jan-87	156.2	148.8	159.4	160.1	161.2	159.9	157.9	159.5	153.0	158.1	158.9	161.1	158.3
Jan-88	156.7	148.8	160.5	161.8	161.9	161.4	158.2	159.7	153.4	158.2	159.0	160.9	158.6
Feb-88	156.7	148.9	160.7	163.0	162.1	161.6	158.4	159.7	153.3	158.3	159.2	161.1	159.0
Mar-88	156.6	148.8	160.4	161.8	162.1	161.5	158.2	159.3	153.3	158.3	159.2	161.1	158.7
Apr-88	156.7	148.8	160.4	161.6	162.2	161.4	158.1	159.3	153.4	158.3	159.3	161.2	158.9
May-88	156.3	148.7	159.9	161.3	161.7	161.0	157.8	159.0	153.2	157.9	158.8	160.6	158.3
Jun-88	156.2	148.8	159.9	161.1	161.7	161.2	157.8	159.1	153.2	157.9	158.8	160.5	158.3
16-Dec-94			158.8			160.0	156.0	159.4		156.8	155.8	158.3	156.6
14-Mar-95										157.1	156.2	158.7	157.1
13-Jun-95						161.0	156.6						
29-Jun-95			159.6					160.4		157.3	156.3	158.9	157.2
22-Sep-95										157.7	156.7	159.2	157.6

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)											
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
20-Dec-95			160.1							157.8	157.0	159.8	157.8
21-Dec-95						161.6	157.0	160.2					
21-Mar-96										157.6	156.7	159.7	157.6
12-Jun-96						161.6	157.3						
13-Jun-96			160.1					159.7		157.4	156.5	159.9	157.5
15-Sep-96										156.8	156.4	159.0	156.6
11-Dec-96						160.8	156.5	159.4					
30-Dec-96			159.5							157.3	156.4	159.1	157.3
13-Mar-97										157.1	157.7	159.7	157.7
19-Jun-97			159.0			160.7	156.5	159.2		156.8	156.0	158.6	156.8
29-Sep-97										156.8	156.1	158.6	156.8
31-Dec-97			158.9			160.7	156.6	159.0		156.7	155.8	158.4	156.7
24-Mar-98										157.6	156.5	159.2	157.6
23-Jun-98			158.8			160.8	156.7	159.2		157.1	156.1	159.0	157.1
28-Sep-98										157.3	156.5	159.1	157.4
21-Dec-98			158.6			160.7	156.6	159.1		157.1	156.3	158.9	157.1
23-Mar-99										158.8	157.8	160.0	158.8
8-Jun-99										158.5		160.6	
15-Jun-99							157.6						158.6
17-Jun-99			160.8			162.5		159.0			157.7		
23-Sep-99										157.5	158.4	161.1	158.1
17-Dec-99			159.7			160.9	156.9	158.6		156.9	156.1	159.6	157.6
22-Mar-00										158.5	157.3	159.0	158.1
2-Jun-00						159.7	156.0				157.0	158.3	156.5
5-Jun-00			158.6					158.3		156.8			
8-Sep-00										155.5	156.4	157.7	156.0
7-Dec-00			157.8			158.8	155.3	158.4		155.5	156.4	157.8	155.9
5-Mar-01										155.9	154.4	157.0	155.2
8-Jun-01			157.4			158.5	155.1			155.1	156.0	157.2	155.6

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
12-Jun-01								155.0					
14-Mar-02										155.3	156.1	157.7	155.7
5-Jun-02			157.0							154.7	155.5	156.9	155.3
7-Jun-02						157.7	154.6	158.0					
18-Sep-02										154.2	155.2	156.4	154.7
5-Dec-02			156.1			156.9	154.0	157.6		154.2	153.4	156.2	154.7
10-Mar-03										153.9	154.7	155.8	154.3
18-Jun-03			156.9			159.0	154.8	160.0		154.7	154.0	156.3	155.1
4-Sep-03										155.6	154.5	157.3	155.9
9-Dec-03			158.7			160.0	156.2	160.6		156.2	155.0	158.0	156.7
3-Mar-04										156.3	155.2	158.4	156.9
3-Sep-04										156.1	157.0	158.4	156.7
17-Dec-04			158.5			159.5	155.9	158.6		156.0	155.2	158.2	156.6
15-Jun-05	154.37	147.42	157.88	159.98	163.73	158.53	155.62	158.88	152.78	154.92	154.39	158.21	156.10
16-Jul-05	154.38	148.40	157.86	159.91	163.62	158.57	155.65	159.14	152.70	154.82	154.15	157.90	155.92
20-Aug-05	154.49	148.42	158.07	160.15	163.92	158.84	155.78	159.42	152.75	155.01	154.33	158.07	156.13
17-Sep-05	154.64	148.72	158.23	160.32	164.10	158.98	155.90	159.55	152.89	155.16	154.46	158.22	156.30
17-Oct-05	154.75	148.69	158.29	160.39	164.21	159.09	155.96	159.49	152.98	155.18	154.48	158.31	156.32
19-Nov-05	154.69	148.75	158.34	160.48	164.23	159.09	155.98	159.37	152.97	155.22	154.46	158.28	156.37
17-Dec-05	154.60	148.52	158.28	160.39	164.05	159.05	155.88	159.15	152.98	155.06	154.31	158.21	156.23
15-Jan-06	154.71	148.61	158.28	160.37	164.08	158.94	155.97	159.04	153.10	155.18	154.57	158.53	156.36
27-Feb-06	154.78	148.64	158.39	160.48	164.23	158.92	155.98	159.19	153.22	155.52	154.83	158.66	156.66
15-Mar-06	154.71	148.72	158.23	160.45	164.30	158.98	156.03	159.15	153.18	155.28	154.59	158.48	156.35
15-Apr-06	154.63	148.66	158.17	160.30	164.11	158.82	155.85	158.99	153.05	155.18	154.57	158.54	156.32
15-May-06	154.55	148.76	158.09	160.20	163.99	158.82	155.78	158.53	153.02	155.15	154.50	158.48	156.32
15-Jun-06	154.48	148.78	157.99	160.12	163.88	158.63	155.73	158.80	153.00	154.95	154.41	158.23	156.23
26-Jul-06	154.41	148.56	157.91	159.96	163.69	158.53	155.68	158.72	152.88	154.95	154.30	158.19	156.08
28-Aug-06			157.89							154.95	154.34	158.18	156.14
31-Aug-06	154.36	148.75		159.88	163.69	158.45	155.62	158.65	152.86				

Table 2.4.12-15 (cont) Historical Groundwater Levels for the Water Table Aquifer

Date	Observation Well and Water Level Elevations (ft msl)												
	142	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
15-Nov-06	154.16	148.79	157.56		162.84	158.19	155.42	158.40	152.71	154.78	154.25	158.11	155.93
16-Nov-06				159.64									

Note.
Water level data for 802A (168.1 ft msl) measured on 13-Nov-85 considered invalid.
Water level data for 804 (166.0 ft msl) measured on 28-Dec-85 considered invalid.
Water level data for Oct-71 to Feb-85 provided in Ground Water Supplement for VEGP Units 1 and 2 **(Georgia Power March 1985).**
Water level data for Jun-85 to Dec-85 provided in Observation Well Readings for VEGP Units 1 and 2, July-December 1985 **(Georgia Power July 1985).**
Water level data for Dec-85 to Jun-86 provided in Observation Well Readings for VEGP Units 1 and 2, January-June 1986 **(Georgia Power January 1986).**
Water level data for Jun-86 to Dec-86 provided in Observation Well Readings for VEGP Units 1 and 2, July-December 1986 **(Georgia Power July 1986).**
Water level data for Dec-86 to Jan-87 provided in Piezometer Weekly Readings Report for VEGP Units 1 and 2 **(Georgia Power 1987).**
Water level data for Jan-88 to Jun-88 provided in Ground-Water Monitoring July 1987 – June 1988, Vogtle Electric Generating Plant **(Bechtel Civil, Inc. 1988).**
Water level data for Dec-94 to Dec-04 provided in Request For Information Number 25144-000-GRI-GEX-00028, SNC ALWR ESP Project **(Bechtel Power Corporation 2006).**

Table 2.4.12-16 Minimum and Maximum Water Levels Recorded at Observation Wells 802A, 805A, 808, LT-7A, LT-12, and LT-13.

Observation Well	Minimum Water Level Elevation (ft msl)	Date	Maximum Water Level Elevation (ft msl)	Date
802A	156.1	5-Dec-02	160.8	17-Jun-99
805A	156.9	5-Dec-02	162.5	17-Jun-99
808	155.0	12-Jun-01	160.6	9-Dec-03
LT7A	152.0	30-Jun-85	159.6	19-Feb-86
LT12	155.8	10-Mar-03	162.4	26-Feb-86
LT13	154.3	10-Mar-03	159.0	1-Feb-88

Note.
Water level data provided in Table 2.4.12-15.

GEOLOGIC TIME		SNC ESP NOMENCLATURE		
PERIOD	SERIES	GEOLOGIC UNIT	HYDROGEOLOGIC UNIT	REGIONAL HYDROGEOLOGIC UNIT
TERTIARY	Eocene	Barnwell Gr.	Water Table aquifer	Southeastern Coastal Plain Aquifer System
		Lisbon Fm. / Blue Bluff Mbr.	Confining unit	
		Still Branch Fm. Congaree Fm.	Tertiary sand aquifer	
	Paleocene	Snapp Fm. Black Mingo Fm.	Semi-confining unit	
Cretaceous		Steel Creek Fm.	Cretaceous aquifer	
		Gaillard Fm. / Black Creek Fm.		
		Pio-Nono Fm. / unnamed sands		
		Cape Fear Fm.		

Notes: Geologic unit naming convention **(Huddlestun and Summerour 1996; Falls and Prowell 2001)**
Regional hydrogeologic unit naming convention **(Miller 1990)**

Figure 2.4.12-1 Schematic Hydrostratigraphic Classification for VEGP Site

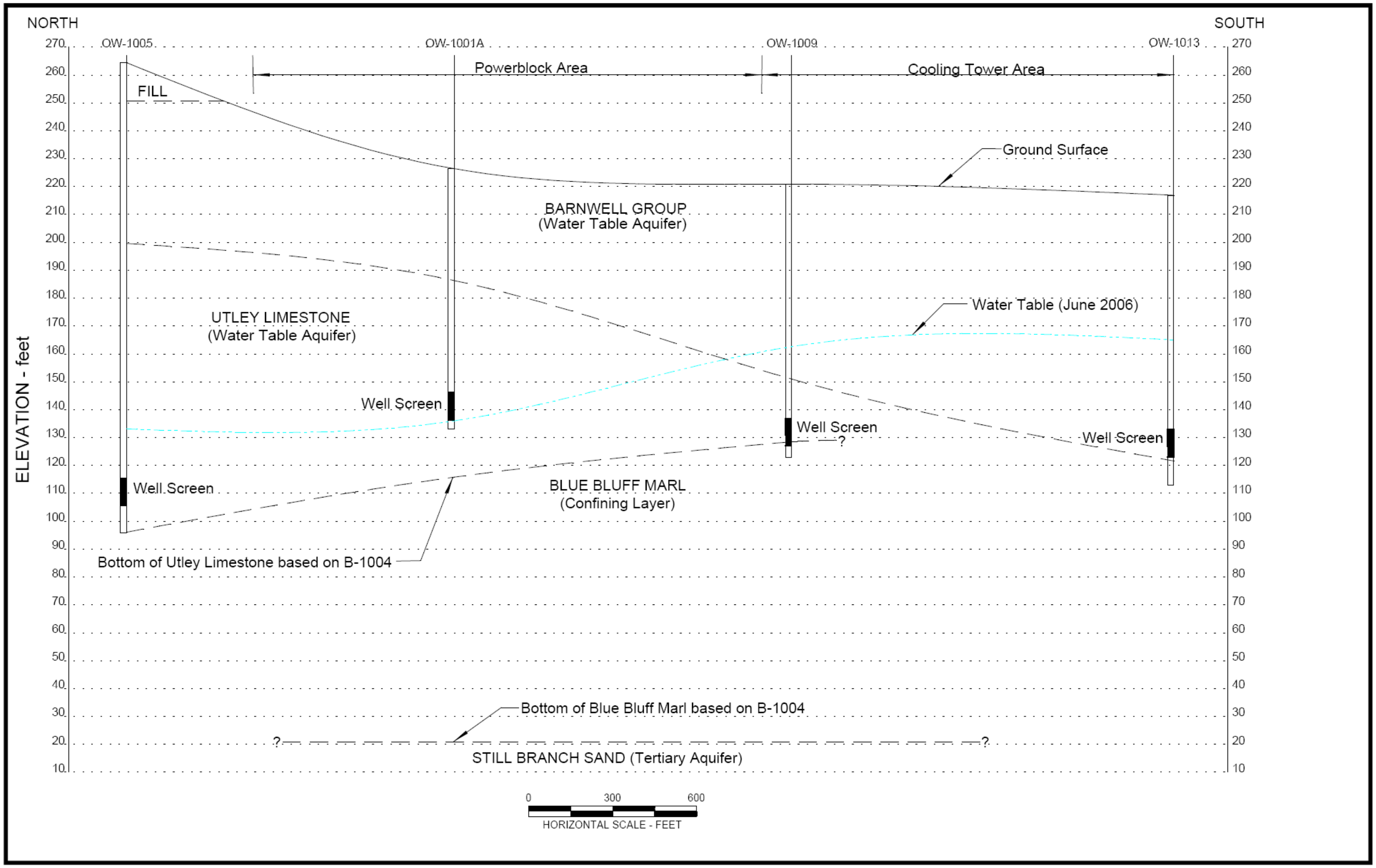

Figure 2.4.12-2A Hydrogeologic Cross-Section of the Water Table Aquifer at the VEGP Site

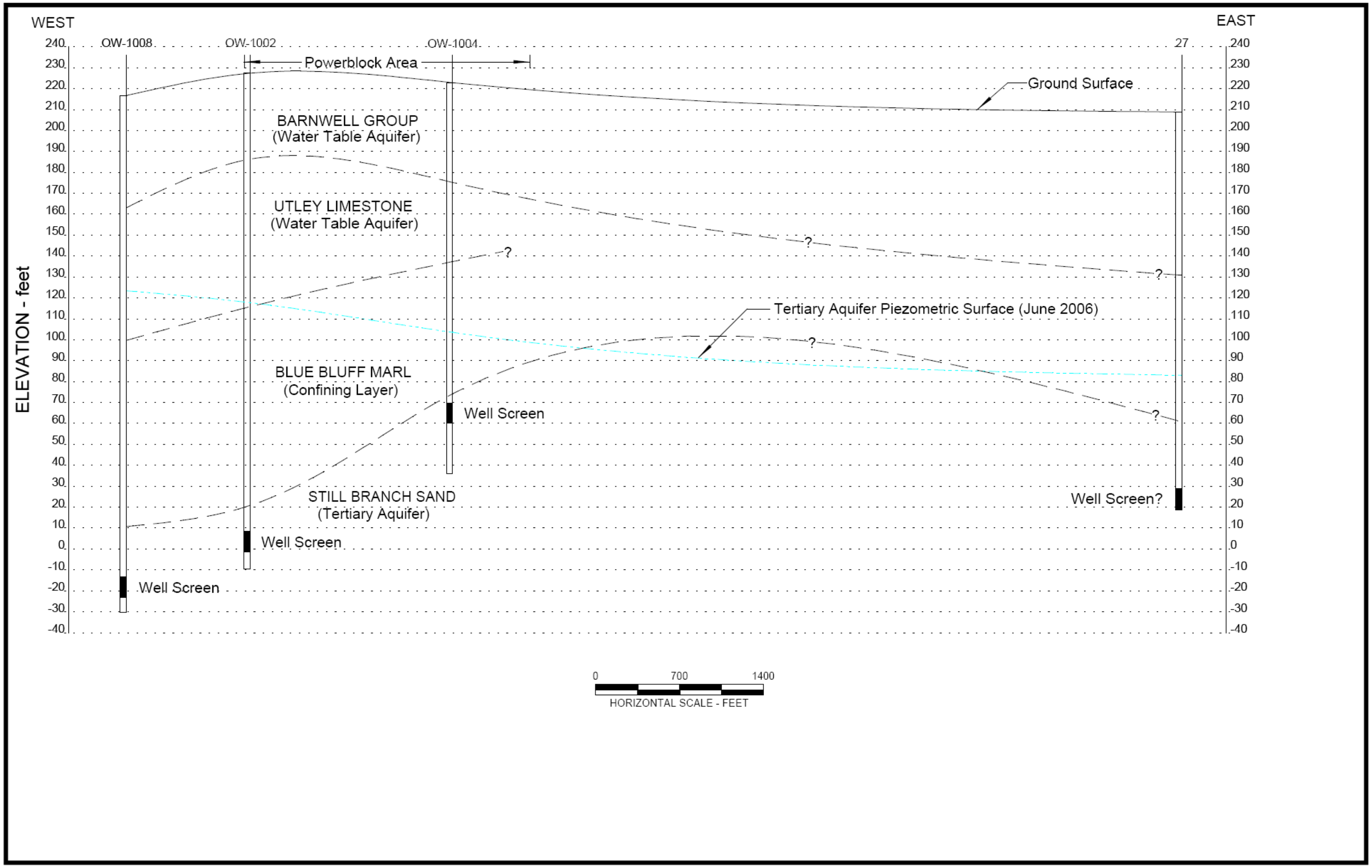

Figure 2.4.12-2B Hydrogeologic Cross-Section of the Tertiary Aquifer at the VEGP Site

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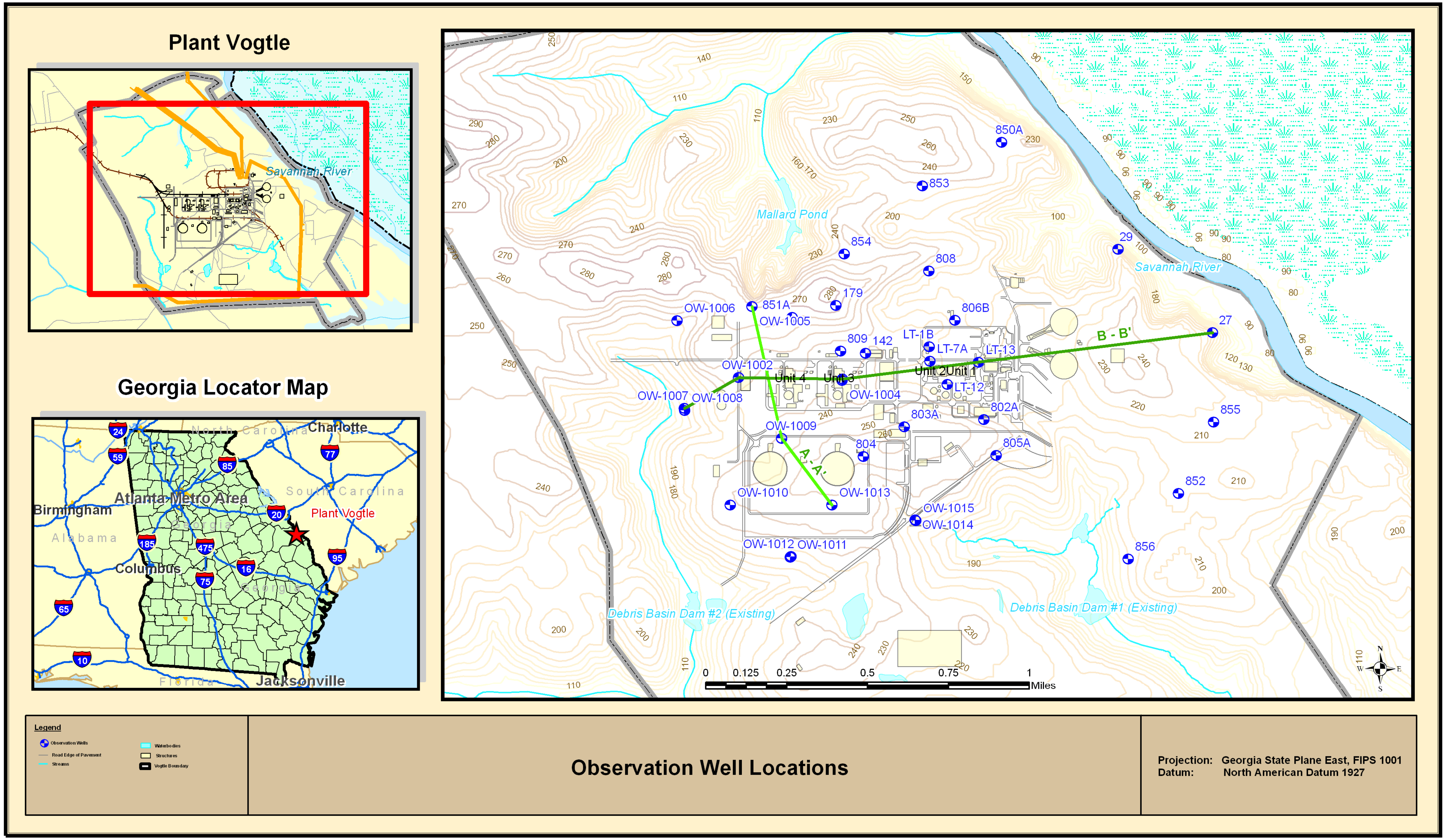

Figure 2.4.12-3 Observation Well Locations

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Figure 2.4.12-4 Deleted at Revision 2

Figure 2.4.12-5 Deleted at Revision 2

Figure 2.4.12-6 Deleted at Revision 2

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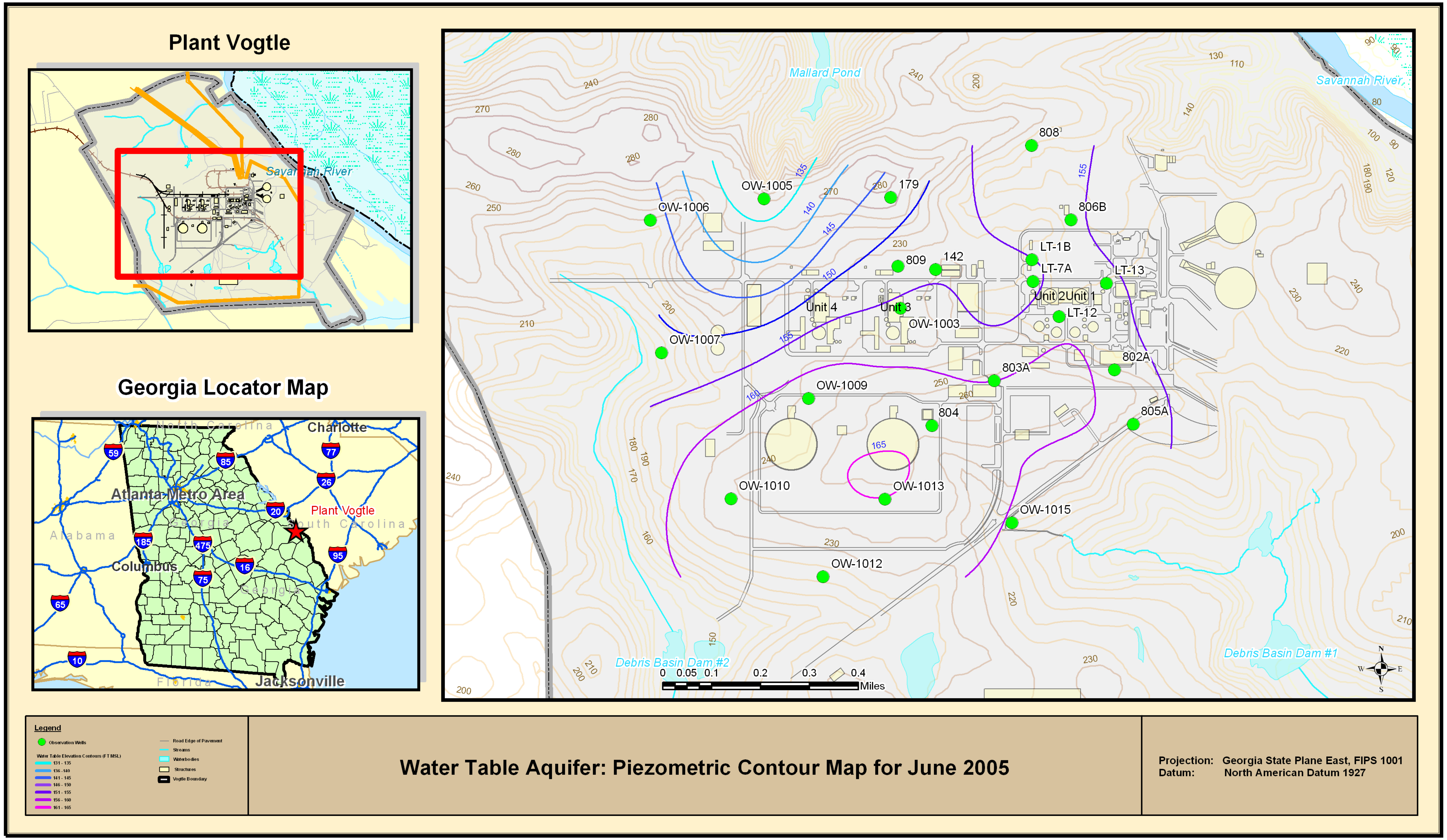

Figure 2.4.12-7 Water Table Aquifer: Piezometric Contour Map for June 2005

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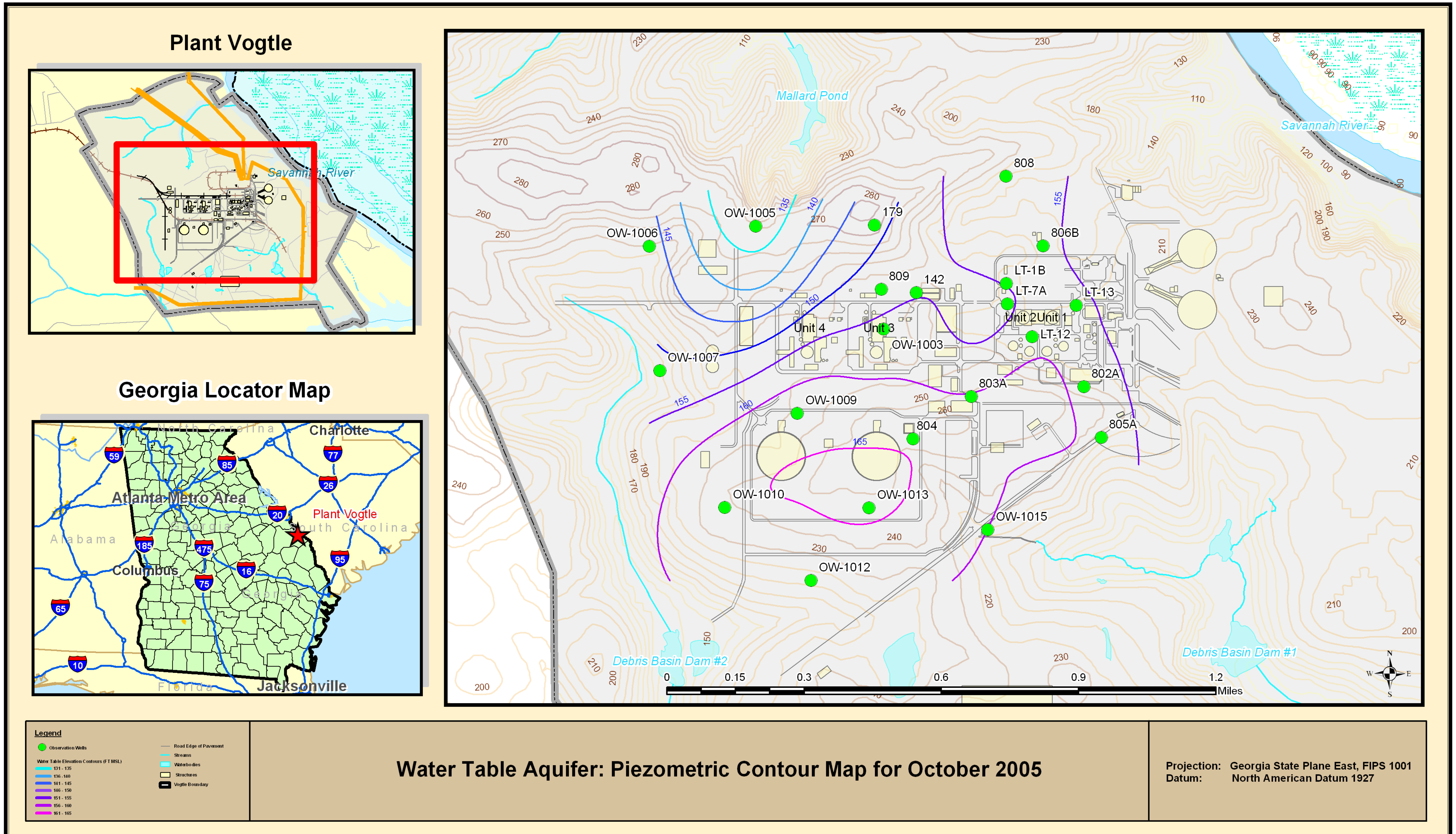

Figure 2.4.12-8 Water Table Aquifer: Piezometric Contour Map for October 2005

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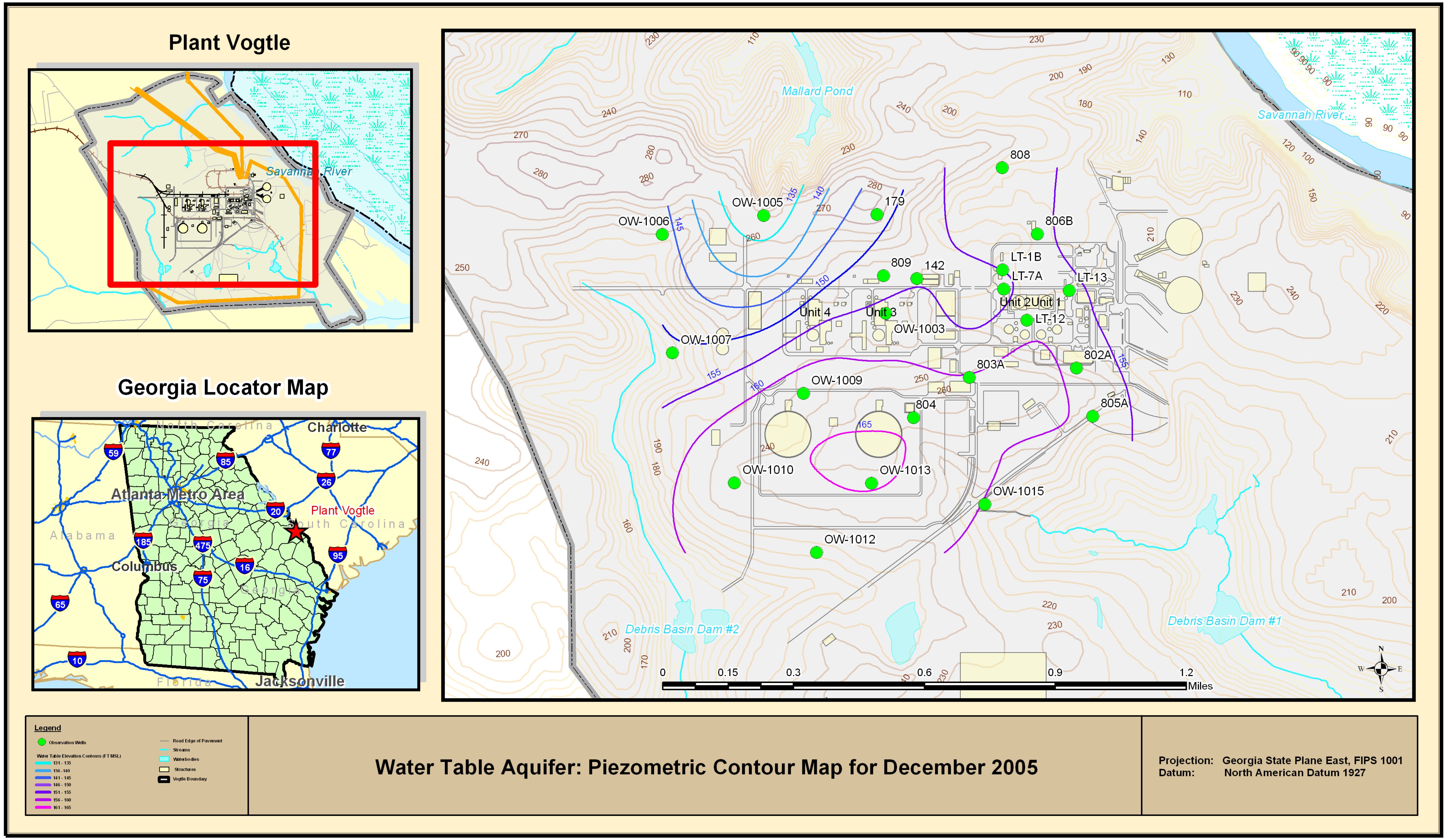

Figure 2.4.12-9 Water Table Aquifer: Piezometric Contour Map for December 2005

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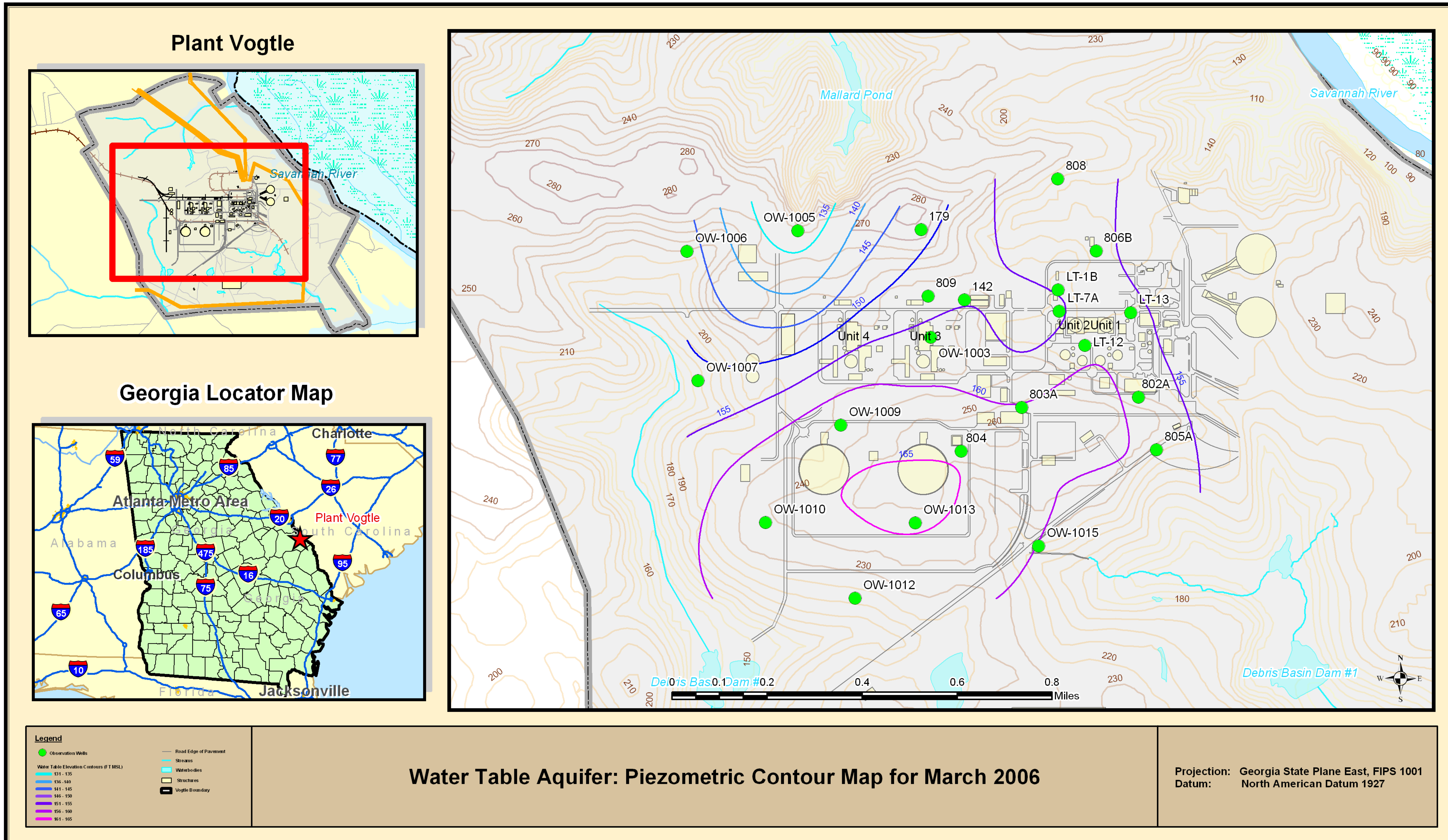

Figure 2.4.12-10 Water Table Aquifer: Piezometric Contour Map for March 2006

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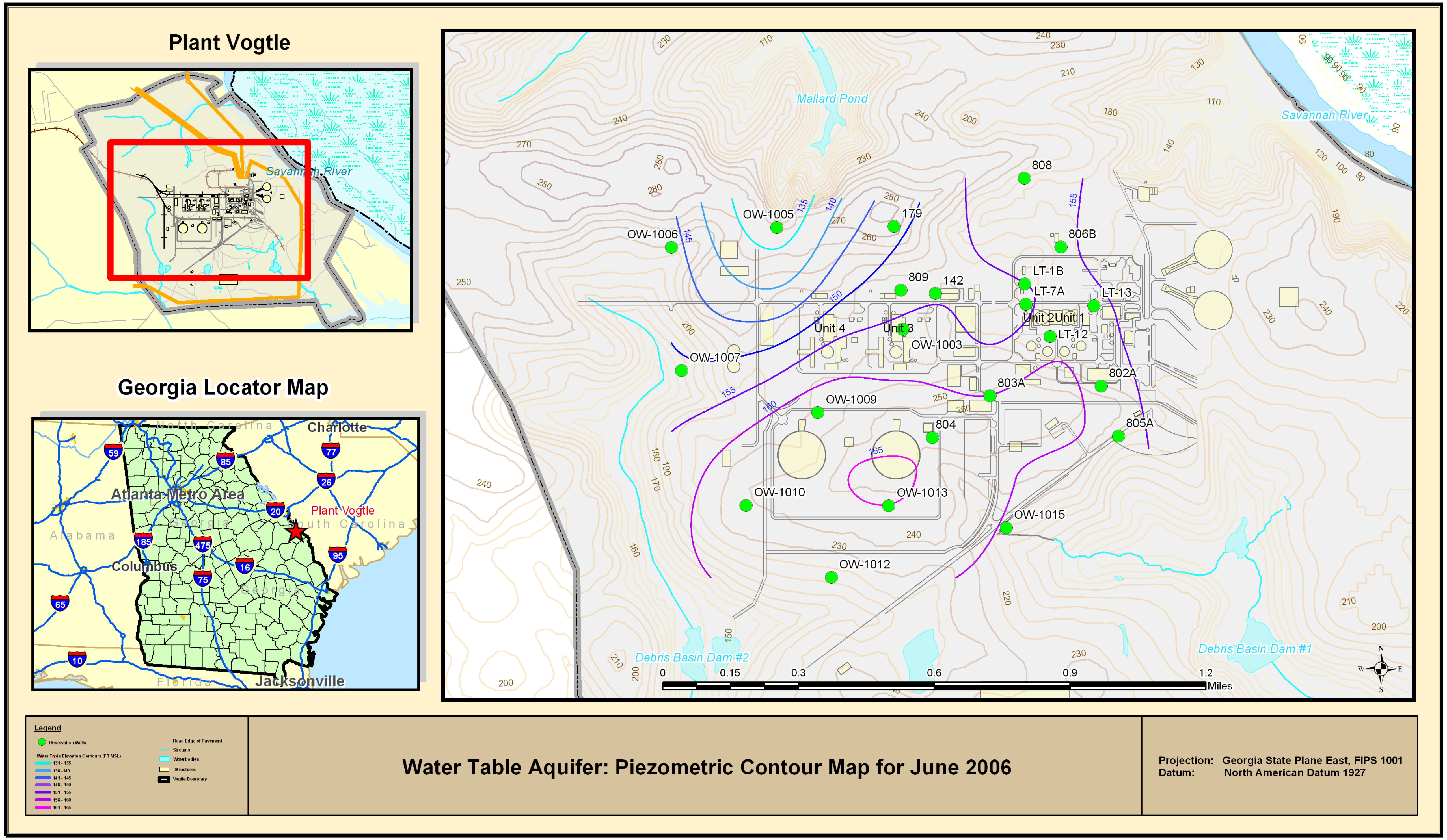

Figure 2.4.12-11 Water Table Aquifer: Piezometric Contour Map for June 2006

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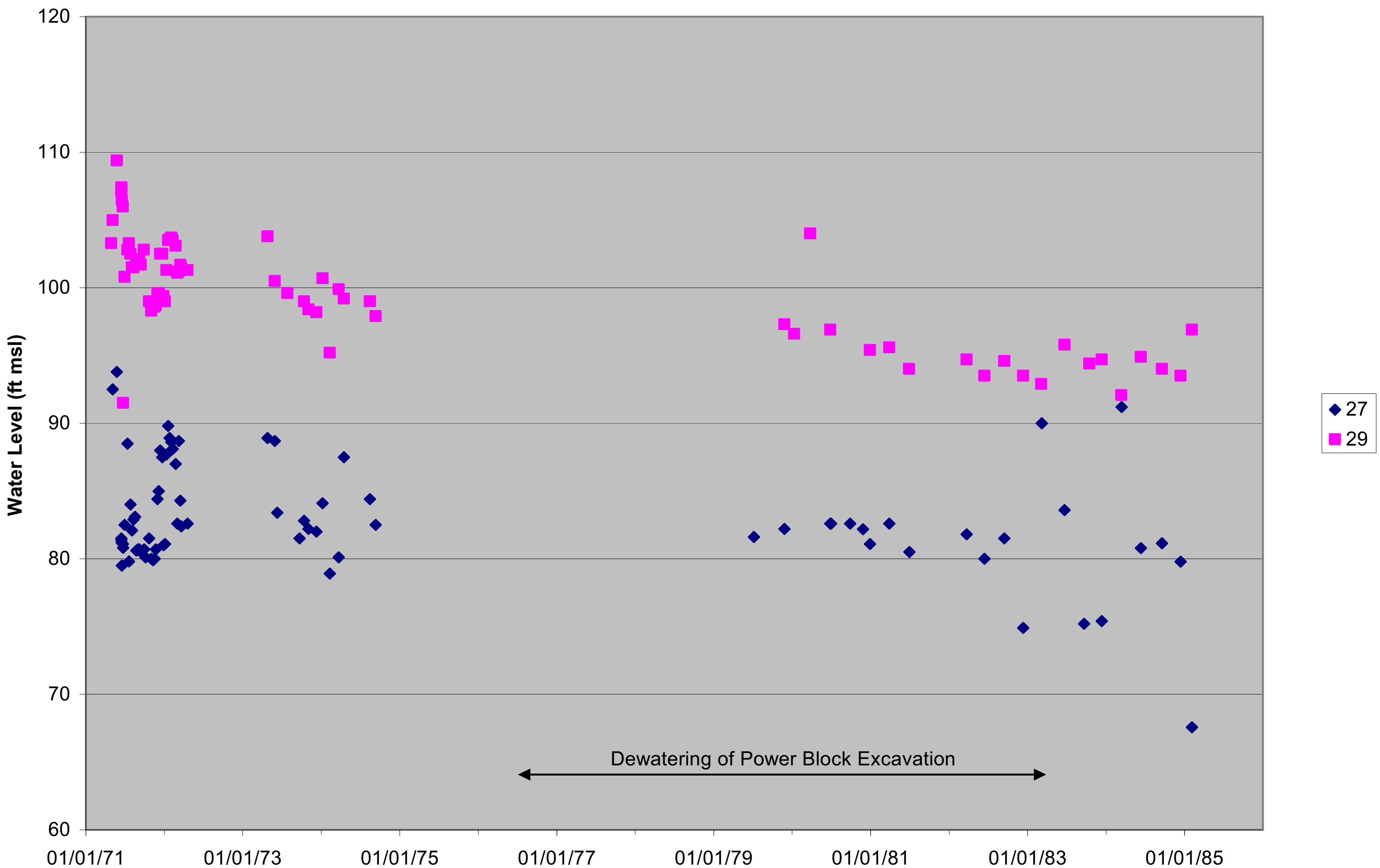

Figure 2.4.12-12 Tertiary Aquifer: 1971–1985 Hydrographs

Figure 2.4.12-13 Deleted at Revision 2

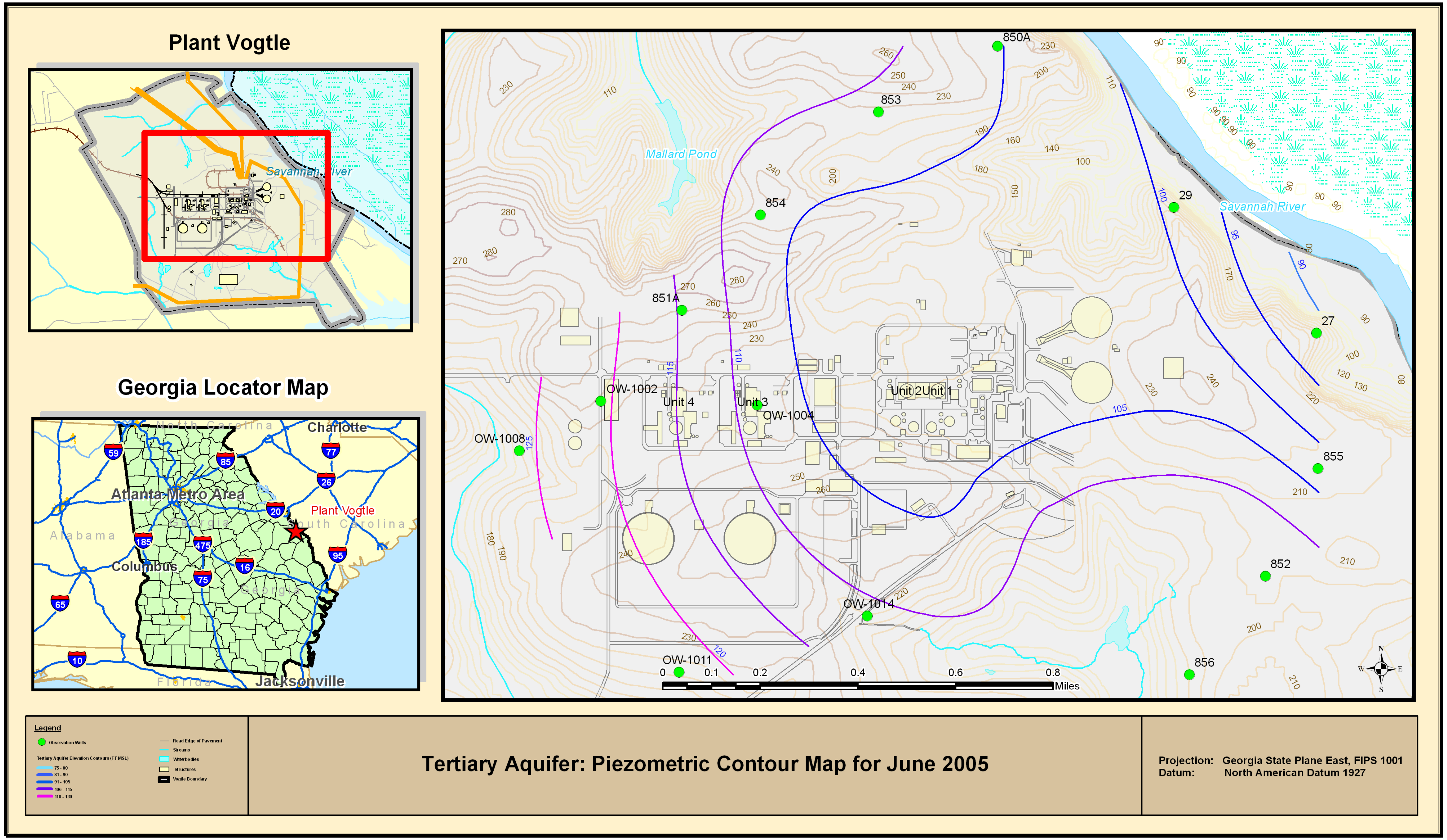

Figure 2.4.12-14 Tertiary Aquifer: Piezometric Contour Map for June 2005

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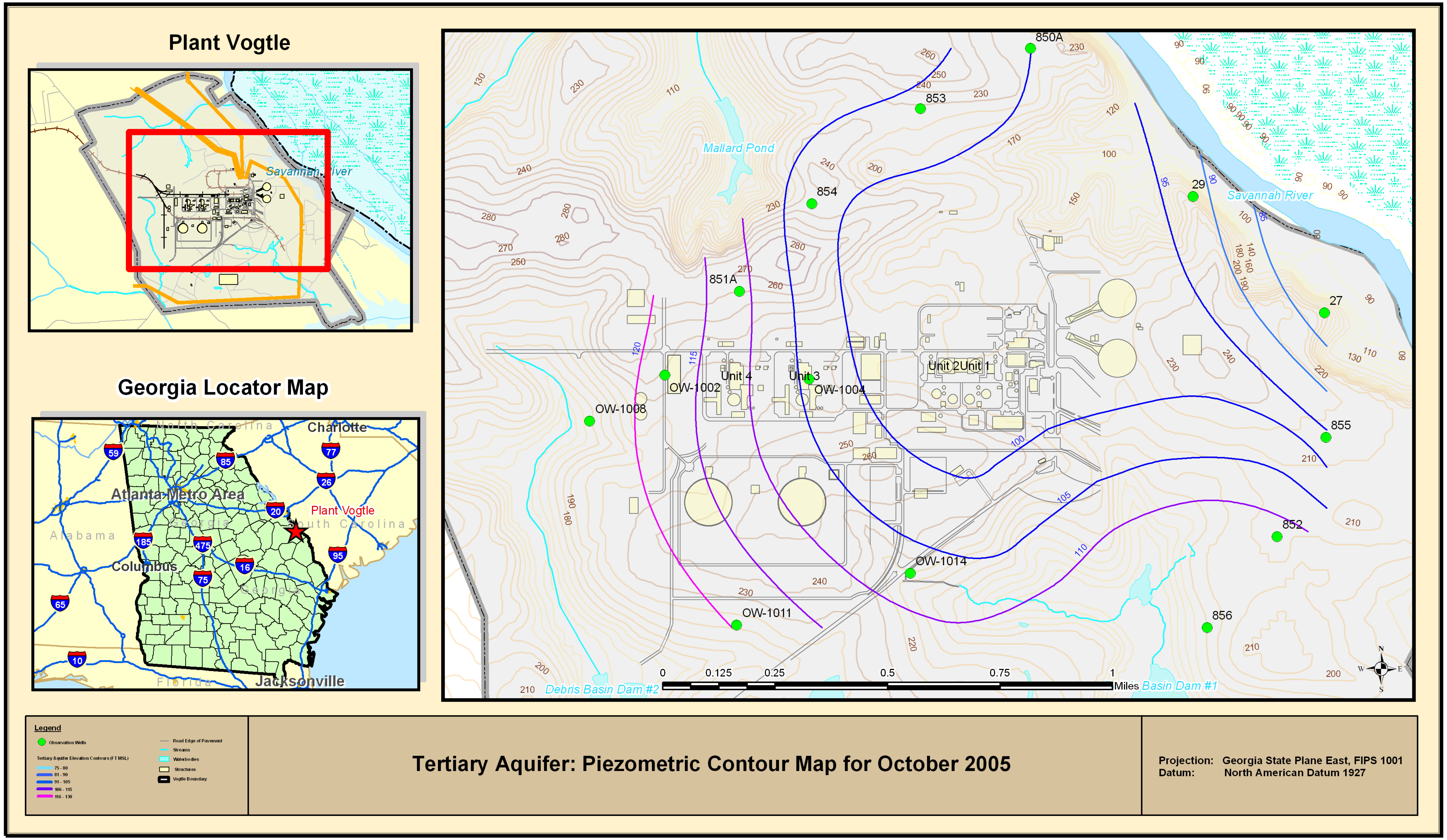

Figure 2.4.12-15 Tertiary Aquifer: Piezometric Contour Map for October 2005

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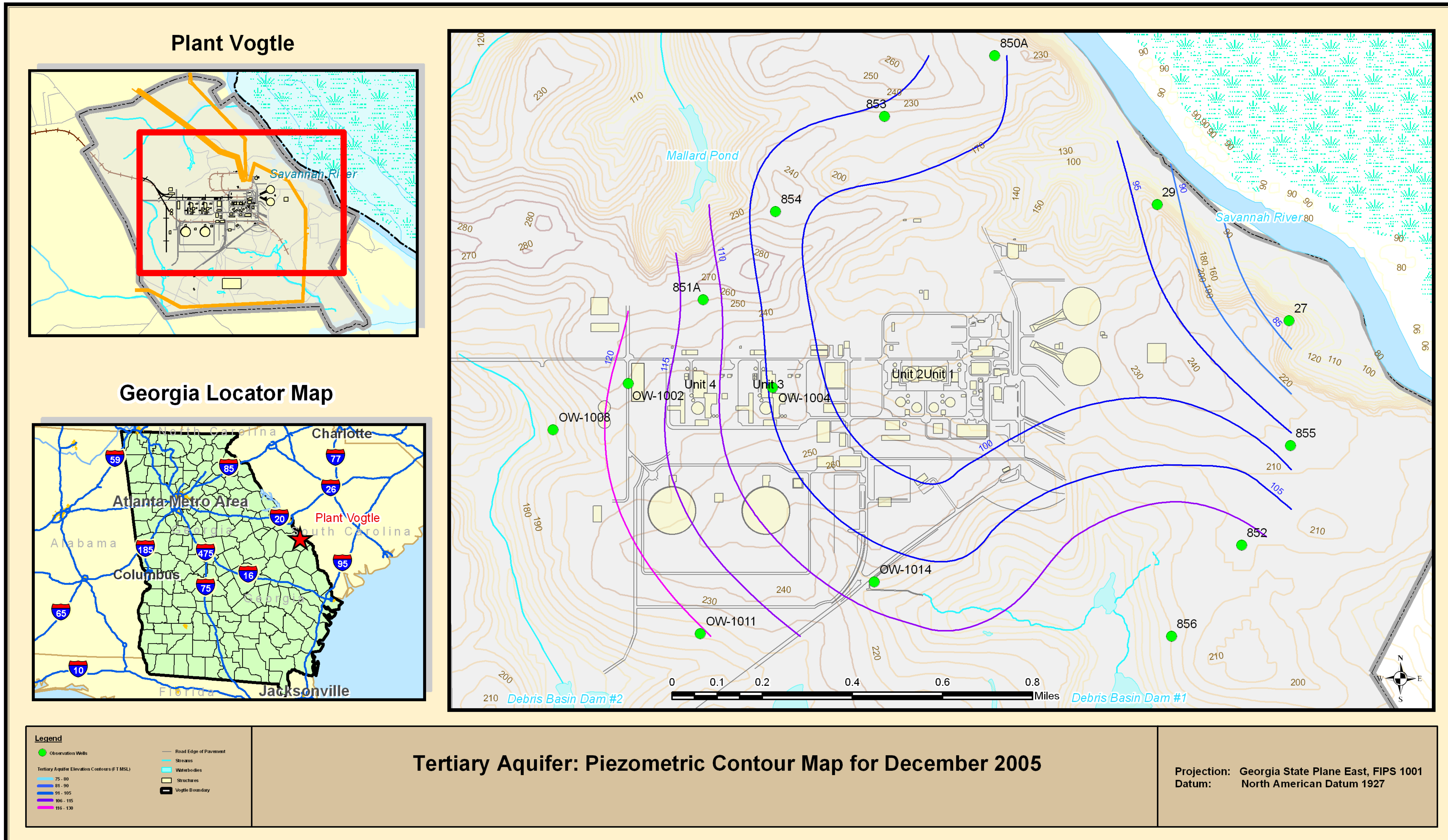

Figure 2.4.12-16 Tertiary Aquifer: Piezometric Contour Map for December 2005

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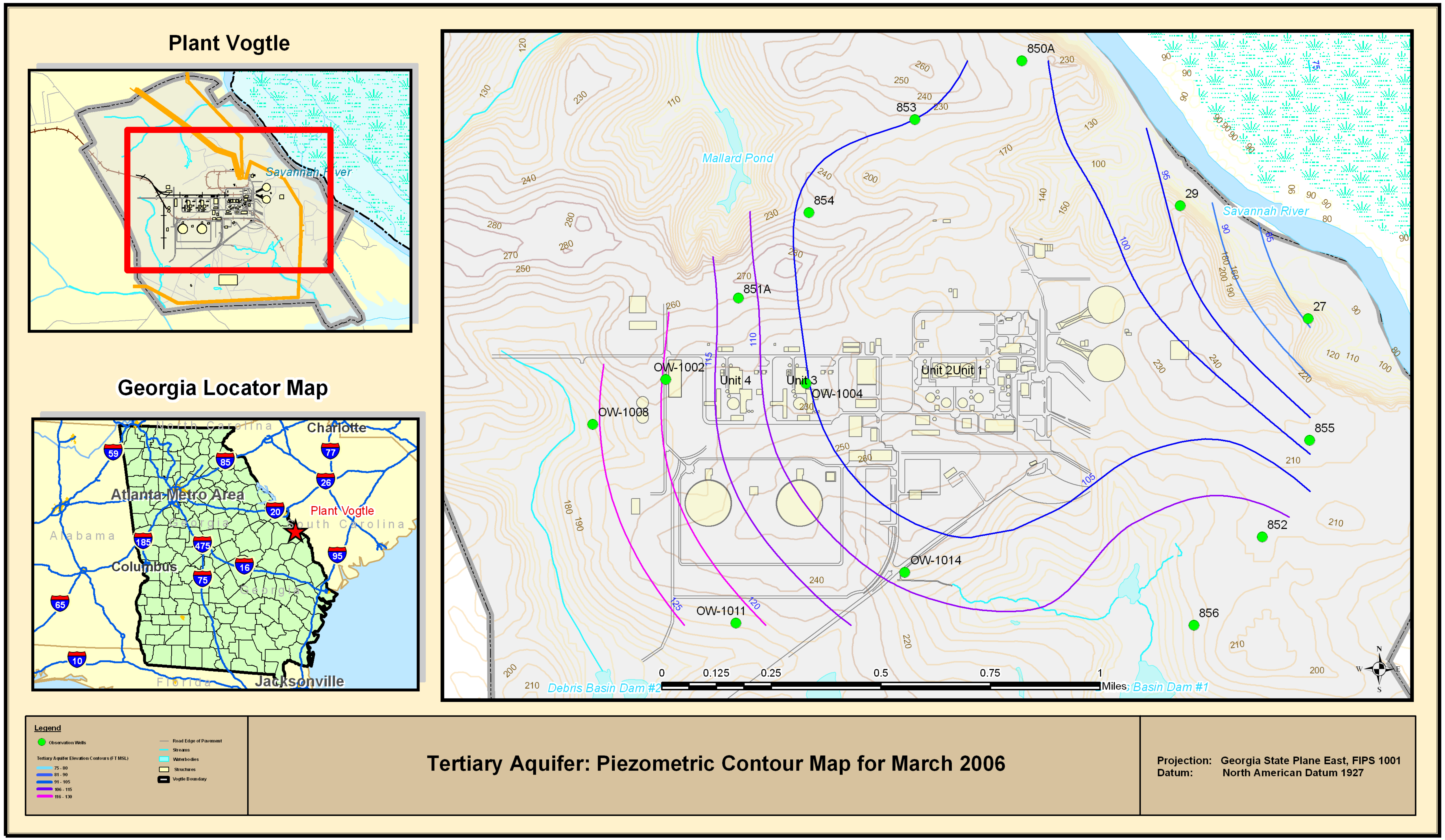

Figure 2.4.12-17 Tertiary Aquifer: Piezometric Contour Map for March 2006

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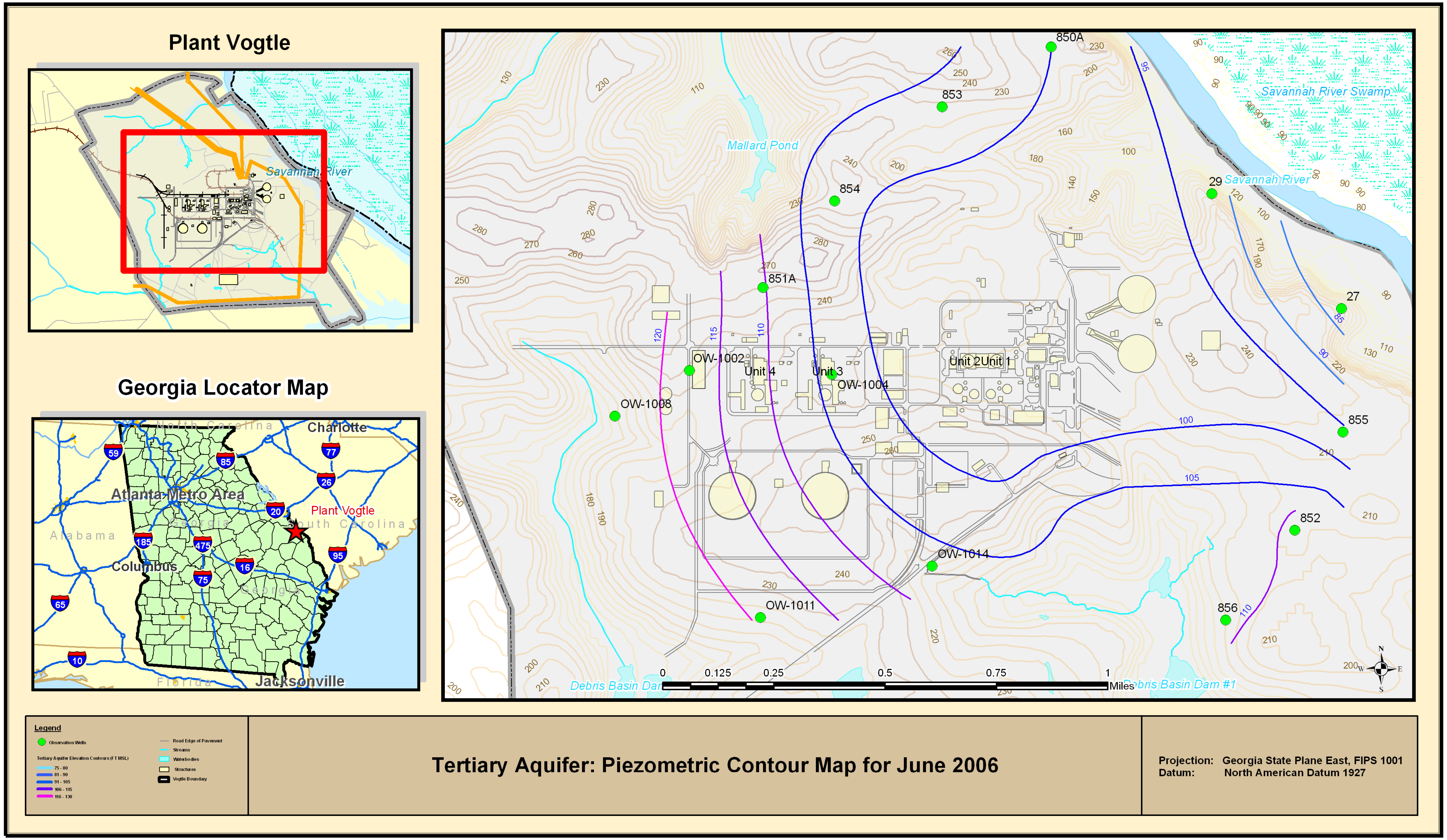

Figure 2.4.12-18 Tertiary Aquifer: Piezometric Contour Map for June 2006

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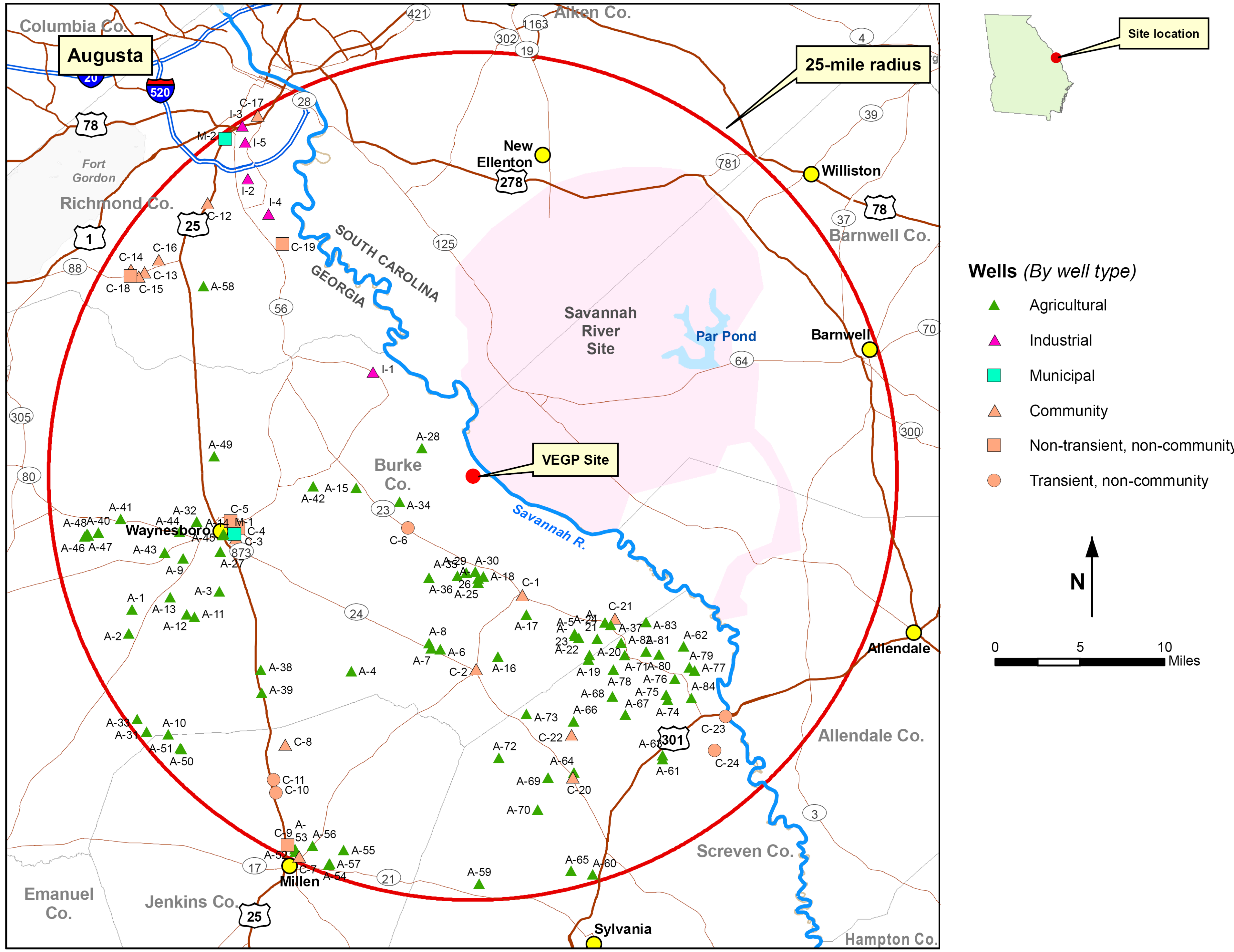

Figure 2.4.12-19 Locations of Agricultural, Industrial, Municipal, and Public Water Supply Wells Within 25 Miles of the VEGP Site

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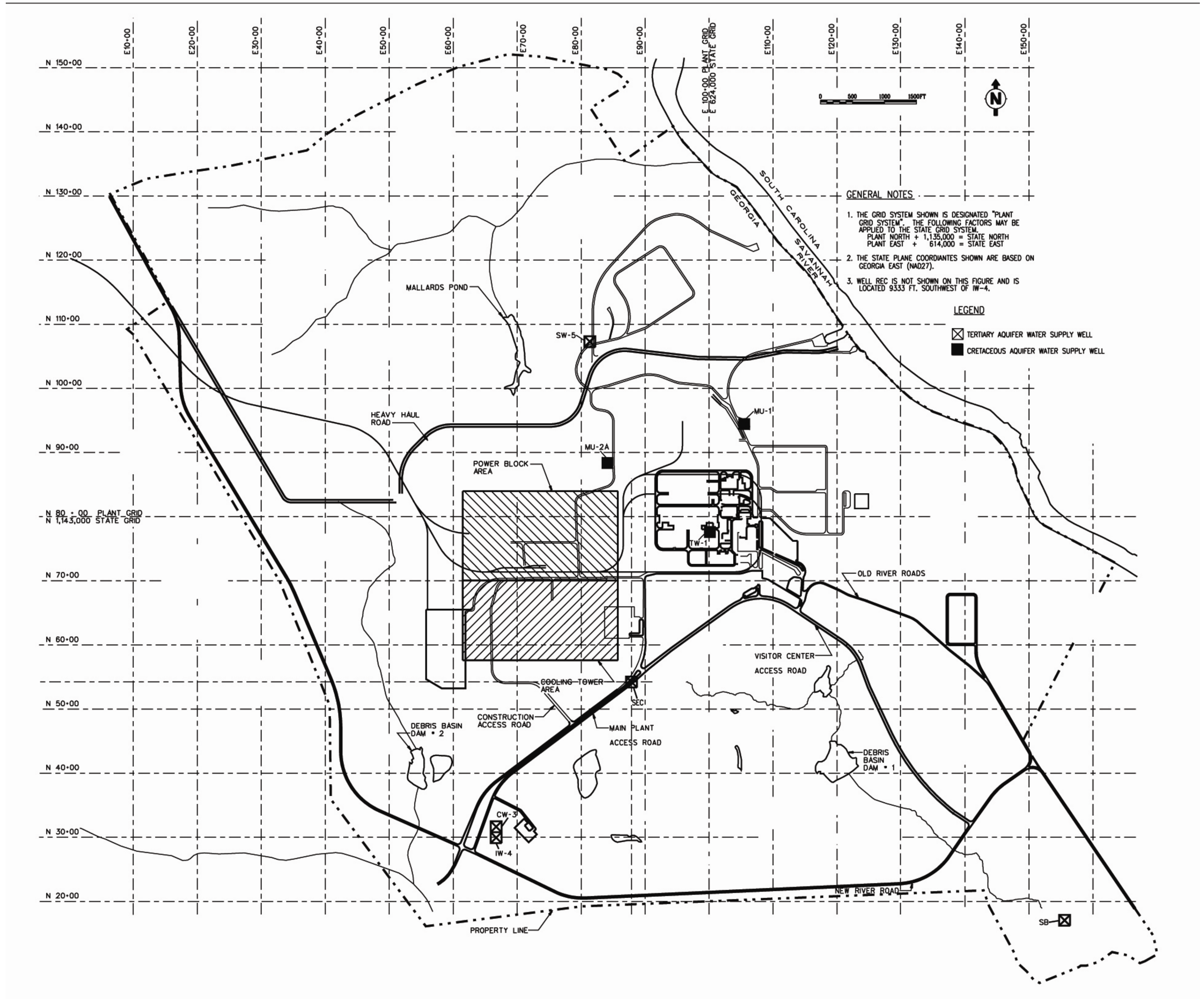

Figure 2.4.12-20 Locations of Existing Supply Wells at the VEGP Site

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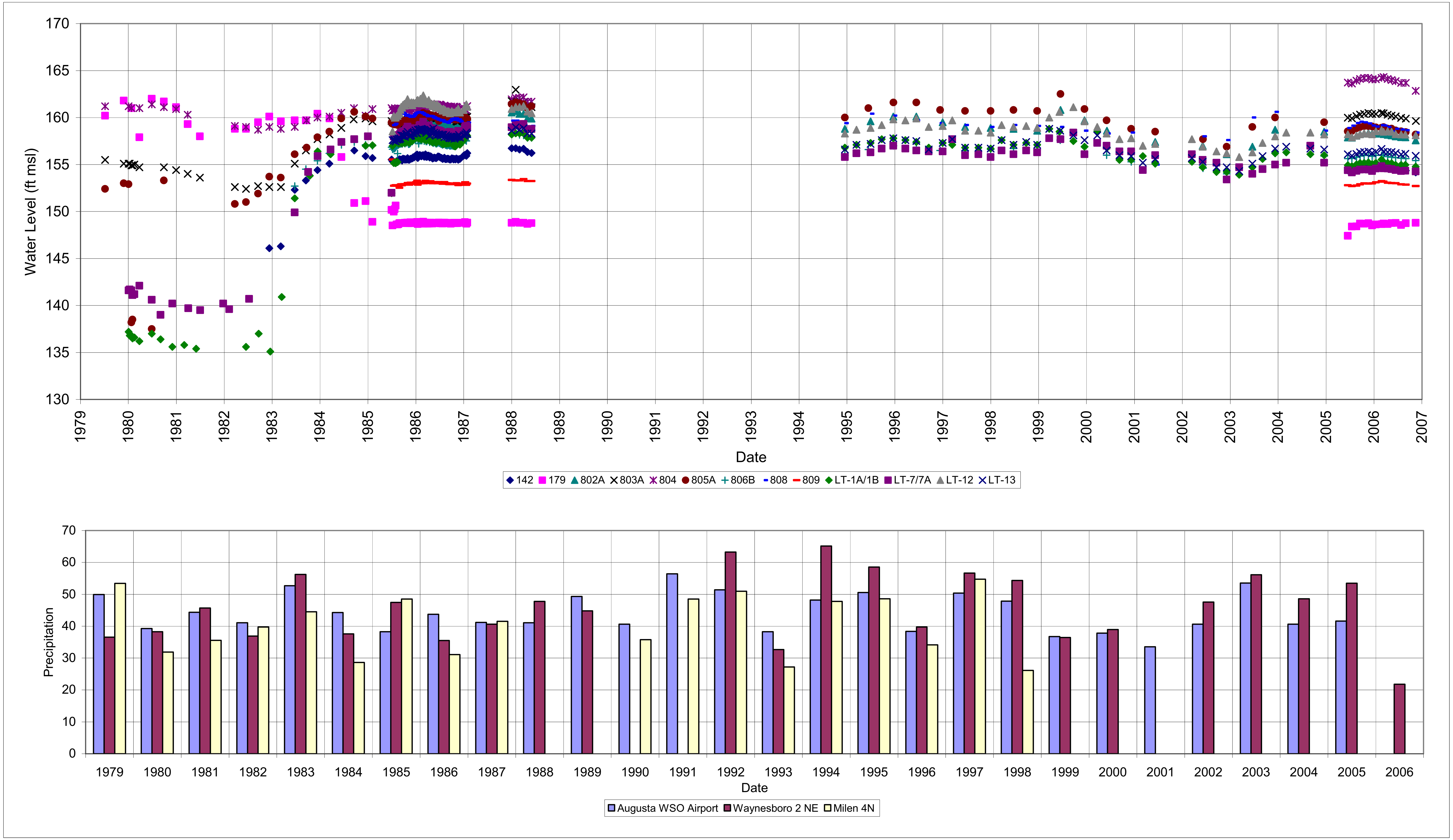

Figure 2.4.12-21 Water Table Aquifer: 1979-2006 Hydrographs

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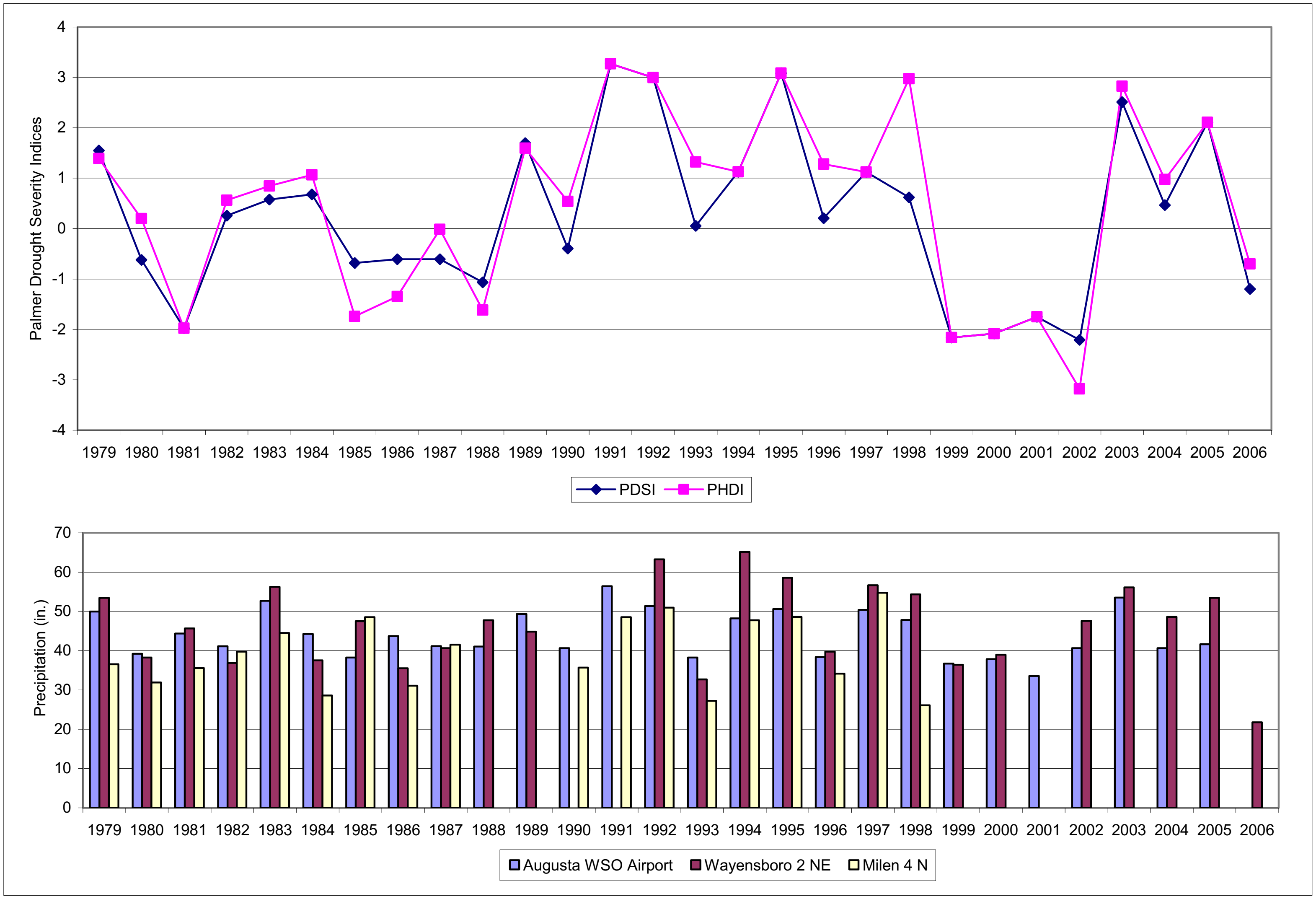

Figure 2.4.12-22 Average Annual PDSI and PHDI for Georgia and Total Annual Precipitation for the Period 1979–2006

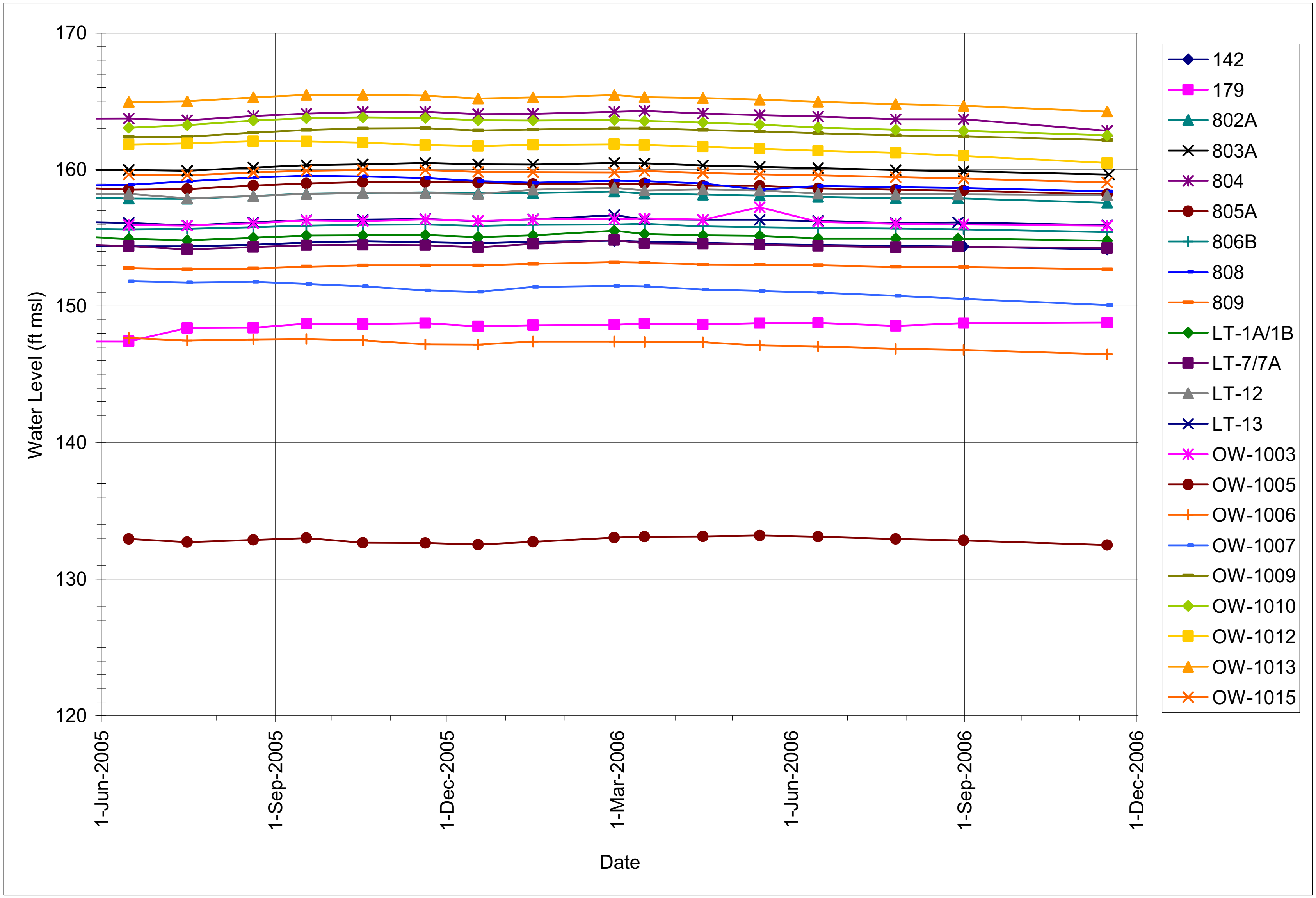

Figure 2.4.12-23 Water Table Aquifer: June 2005 – November 2006 Hydrographs

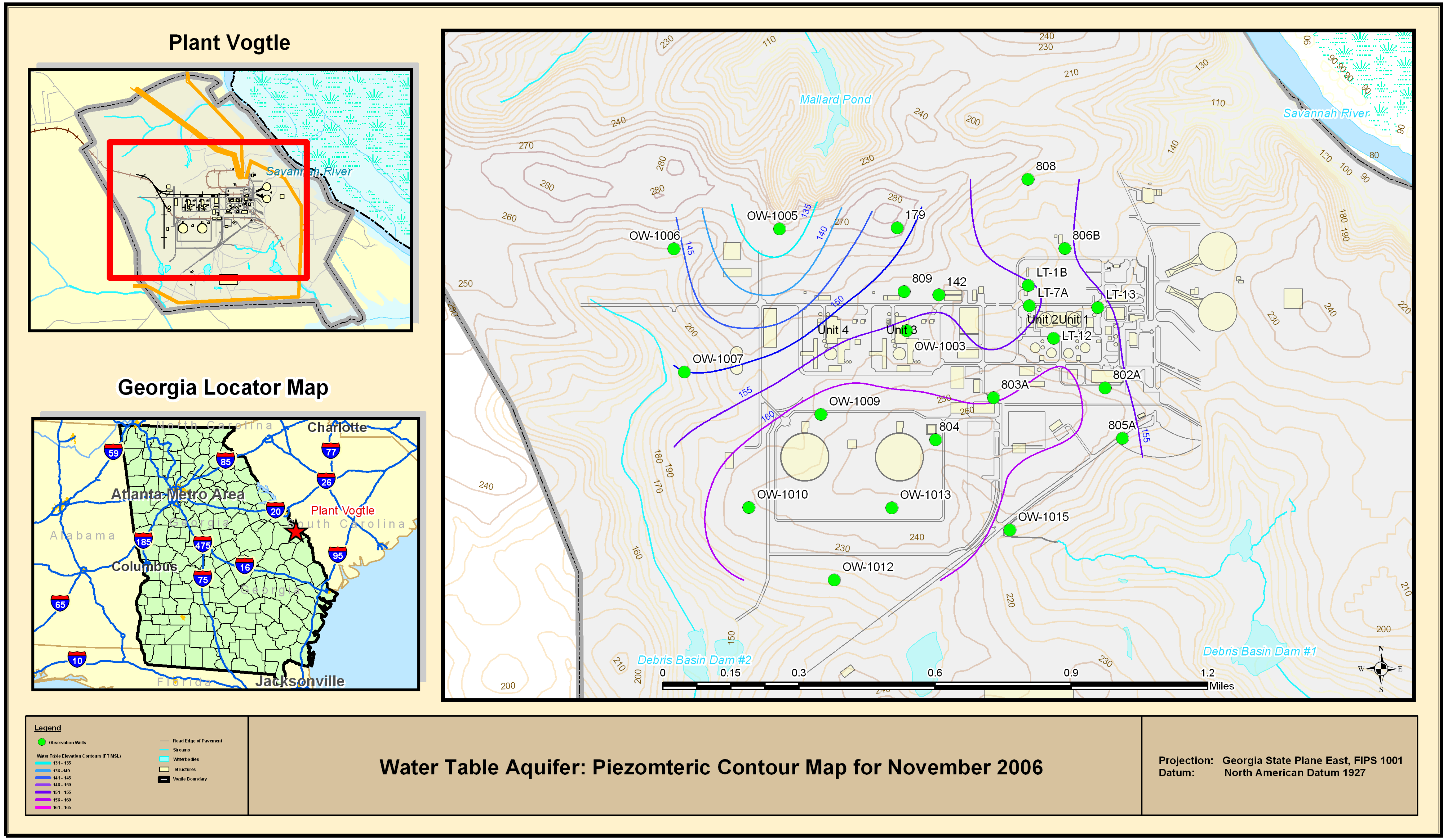

Figure 2.4.12-24 Water Table Aquifer: Piezometric Contour Map for November 2006

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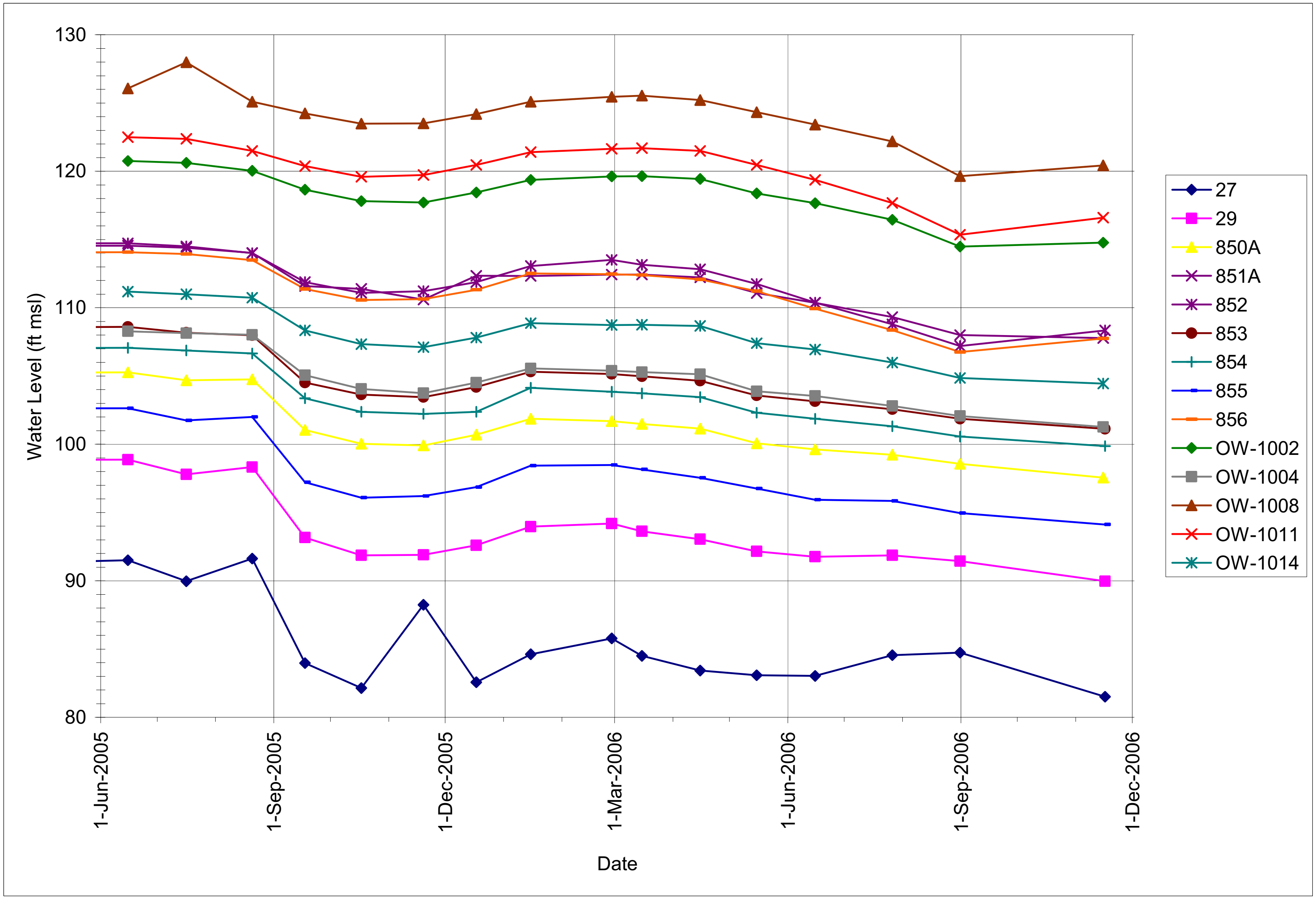

Figures 2.4.12-25 Tertiary Aquifer: June 2005 - November 2006 Hydrographs.

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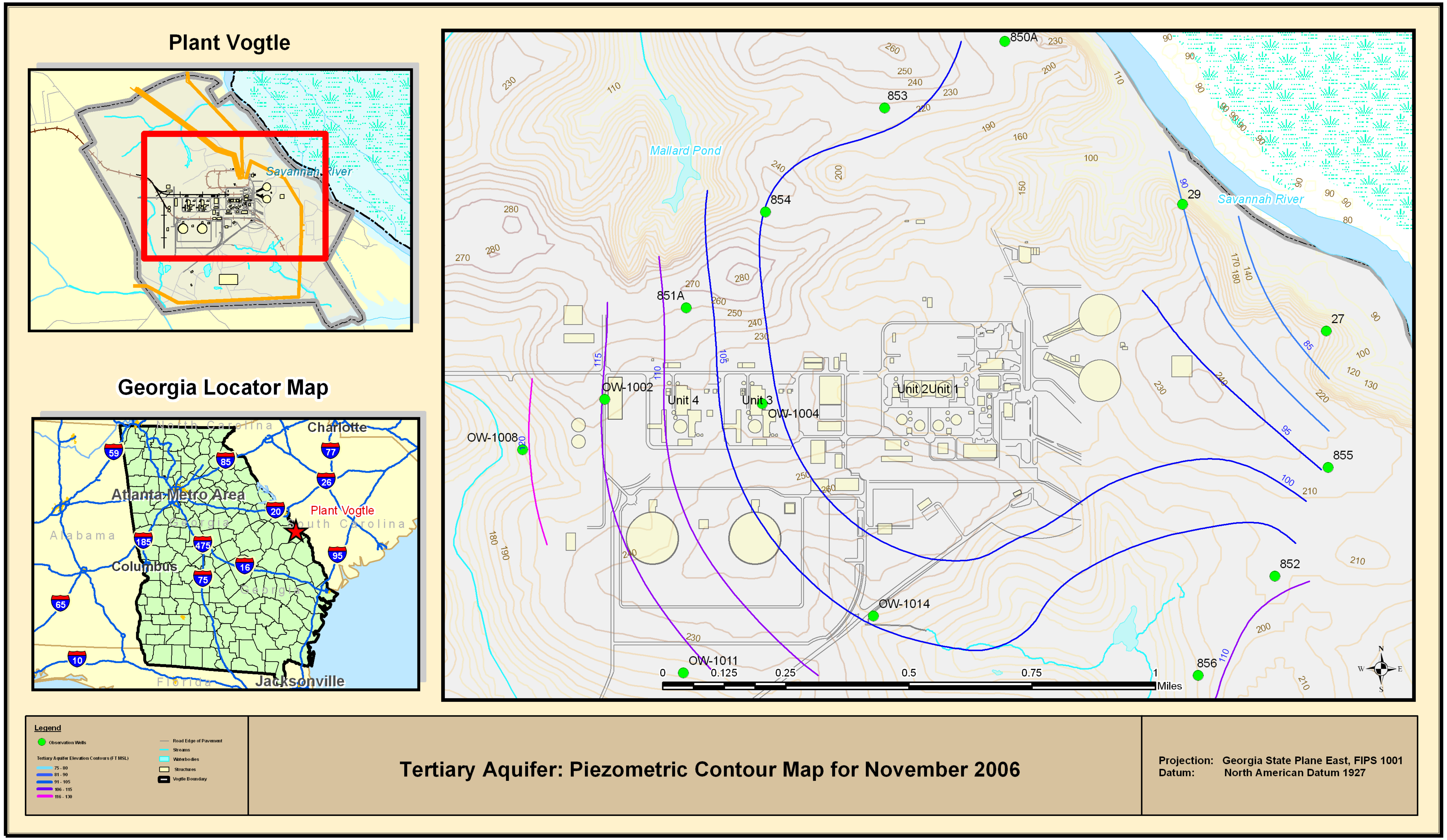

Figures 2.4.12-26 Tertiary Aquifer: Piezometric Contour Map for November 2006

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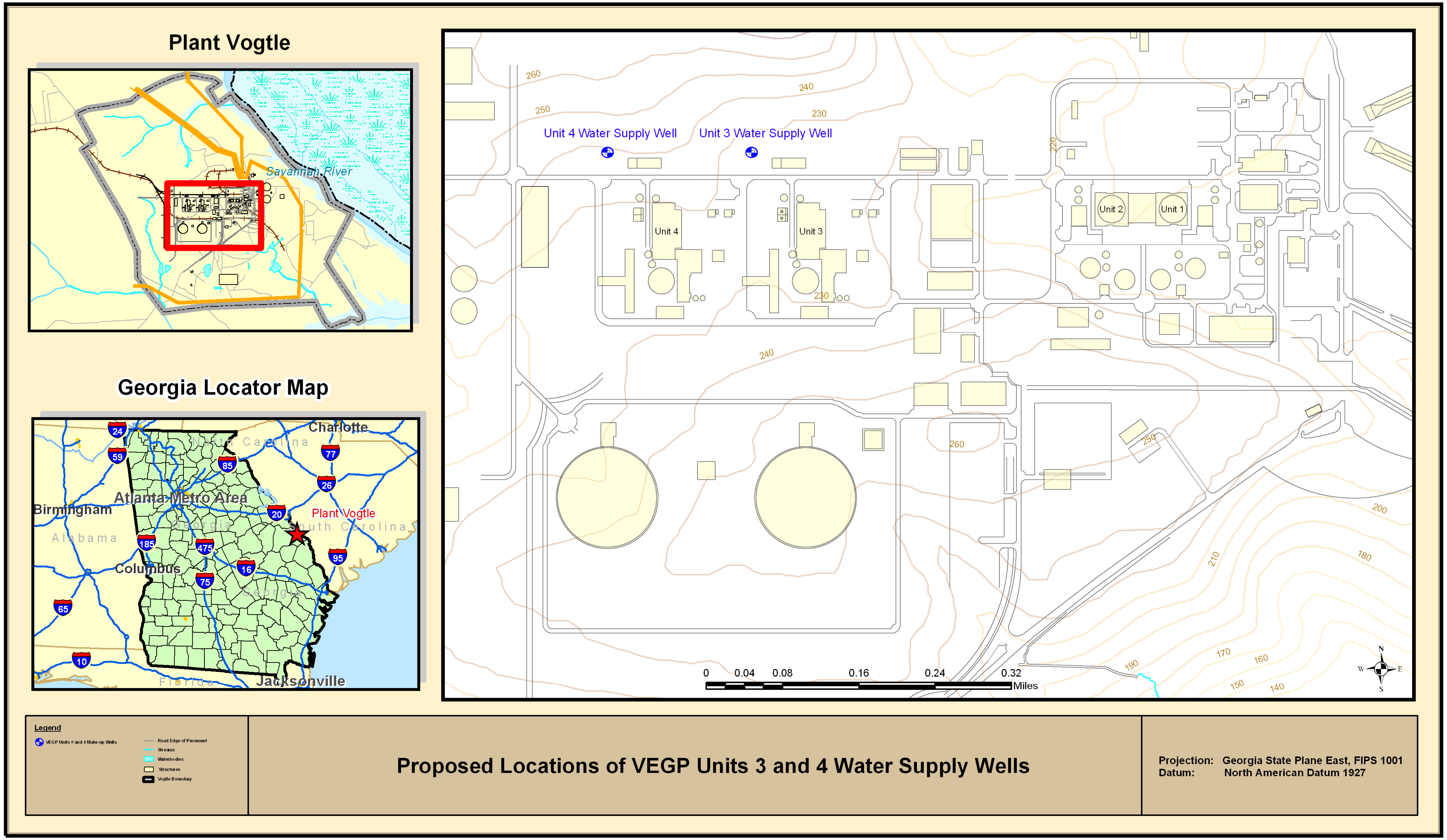

Figures 2.4.12-27 Proposed Locations of VEGP Units 3 and 4 Water Supply Wells

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