Southern Nuclear

UFSAR Table of Contents

UFSAR Formatting Legend

Color	Description
Black	Original Westinghouse AP1000 DCD Revision 19 content (part of plant-specific DCD)
Purple	Departures from AP1000 DCD Revision 19 content (part of plant-specific DCD)
Blue	Standard FSAR content
Green	Site-specific FSAR content
Red	Linked cross-references (chapters, appendices, sections, subsections, tables, figures, and references)

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2.0 Site Characteristics

Chapter 2 describes the characteristics and site-related design parameters of the Vogtle Electric Generating Plant (VEGP), Units 3 and 4. The site location, characteristics and parameters, as described in the following five sections are provided in sufficient detail to support a safety assessment:

- Geography and Demography (Section 2.1)
- Nearby industrial, Transportation, and Military Facilities (Section 2.2)
- Meteorology (Section 2.3)
- Hydrologic Engineering (Section 2.4)
- Geology, Seismology, and Geotechnical Engineering (Section 2.5)

Table 2.0-201 provides a comparison of site-related design parameters for which the AP1000 plant is designed and site characteristics specific to VEGP in support of this safety assessment. The first two columns of Table 2.0-201 are a compilation of the site parameters. The third column of Table 2.0-201 is the corresponding site characteristic for the VEGP. The fourth column denotes the place where this data is presented. The last column indicates whether or not the site characteristic falls within the AP1000 site parameters. “Yes” indicates the site characteristic falls within the parameter. Control room atmospheric dispersion factors (χ/Q) for accident dose analysis are presented in Table 2.0-202. All of the control room χ/Q values fall within the AP1000 parameters.

Table 2.0-203 provides a summary list of the limiting site characteristic values that have been established by analyses presented throughout this document. This list also provides a summary of important site characteristics necessary to establish the findings required by 10 CFR Parts 52 and 100 on the suitability of the site.

Table 2-1
Not Used

Table 2.0-201 (Sheet 1 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Air Temperature				
Maximum Safety[b]	115°F dry bulb/86.1°F coincident wet bulb[h]	115°F dry bulb/77.7°F coincident wet bulb	Table 2.0-203	Yes
	86.1°F wet bulb (noncoincident)	83.9°F wet bulb (noncoincident)	Table 2.0-203	Yes
Minimum Safety[b]	-40°F	-8°F	Table 2.0-203	Yes
Maximum Normal[c]	101°F dry bulb/80.1°F coincident wet bulb	97°F dry bulb/76°F coincident wet bulb	Subsection 2.3.1.5	Yes
	80.1°F wet bulb (noncoincident)[d]	79°F wet bulb (noncoincident)	Subsection 2.3.1.5	Yes
Minimum Normal[c]	-10°F	21°F dry bulb	Subsection 2.3.1.5	Yes
Wind Speed				
Operating Basis	145 mph (3 second gust); importance factor 1.15 (safety), 1.0 (nonsafety); exposure C; topographic factor 1.0	104 mph (3 second gust); exposure C; topographic factor 1.0. (Importance factor is not a property of the wind speed.)	Table 2.0-203 Figure 2.5-235	Yes
Tornado	300 mph	300 mph	Table 2.0-203	Yes
	Maximum pressure differential of 2.0 lb/in^2	2.0 lb/in^2	Table 2.0-203	Yes

Table 2.0-201 (Sheet 2 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[(a)]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Seismic				
CSDRS	CSDRS free field peak ground acceleration of 0.30 g with modified Regulatory Guide 1.60 response spectra (See Figures 5.0-1 and 5.0-2.). The SSE is now referred to as CSDRS. Seismic input is defined at finished grade except for sites where the nuclear island is founded on hard rock. If the site-specific spectra exceed the response spectra in Figures 5.0-1 and 5.0-2 at any frequency, or if soil conditions are outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed. This evaluation will consist of a site-specific dynamic analysis and generation of in-structure response spectra at key locations to be compared with the floor response spectra of the certified design at 5-percent damping. The site is acceptable if the floor response spectra from the site-specific evaluation do not exceed the AP1000 spectra for each of the locations or the exceedances are justified.	Site-specific GMRS values specified and illustrated in Subsection 2.5.2. The seismic design of AP-1000 nuclear island is discussed in Subsection 3.7.1.1.1 Site-specific evaluation performed in Appendix 2.5E	Table 2.0-203 Subsection 3.7.1.1.1 Appendix 2.5E	Yes

Table 2.0-201 (Sheet 3 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[(a)]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
	The hard rock high frequency (HRHF) envelope response spectra are shown in Figure 5.0-3 and Figure 5.0-4 defined at the foundation level for 5% damping. The HRHF envelope response spectra provide an alternative set of spectra for evaluation of site specific GMRS. A site is acceptable if its site specific GMRS fall within the AP1000 HRHF envelope response spectra. Evaluation of a site for application of the HRHF envelope response spectra includes consideration of the limitation on shear wave velocity identified for use of the HRHF envelope response spectra. This limitation is defined by a shear wave velocity at the bottom of the basemat equal to or higher than 7,500 fps, while maintaining a shear wave velocity equal to or above 8,000 fps at the lower depths.			
Fault Displacement Potential	No potential fault displacement considered beneath the seismic Category I and seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the seismic Category I and seismic Category II structures.	No fault displacement potential within the investigative area.	Table 2.0-203	Yes

Table 2.0-201 (Sheet 4 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Soil				
Average Allowable Static Bearing Capacity	The allowable bearing capacity, including a factor of safety appropriate for the design load combination, shall be greater than or equal to the average bearing demand of 8,900 lb/ft² over the footprint of the nuclear island at its excavation depth	34,000 lb/ft^2	Table 2.0-203	Yes
Dynamic Bearing Capacity for Normal Plus Safe Shutdown Earthquake (SSE)	The allowable bearing capacity, including a factor of safety appropriate for the design load combination, shall be greater than or equal to the maximum bearing demand of 35,000 lb/ft² at the edge of the nuclear island at its excavation depth, or Site-specific analyses demonstrate factor of safety appropriate for normal plus safe shutdown earthquake loads.	42,000 lb/ft^2	Table 2.0-203	Yes
Shear Wave Velocity	Greater than or equal to 1,000 ft/sec based on minimum low-strain soil properties over the footprint of the nuclear island at its excavation depth	Greater than 1000 ft/sec	Table 2.0-203	Yes
Lateral Variability	Soils supporting the nuclear island should not have extreme variations in subgrade stiffness. This may demonstrated by one of the following:			

Table 2.0-201 (Sheet 5 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Lateral Variability (Continued)	1 Soils supporting the nuclear island are uniform in accordance with Regulatory Guide 1.132 if the geologic and stratigraphic features at depths less than 120 feet below grade can be correlated from one boring or sounding location to the next with relatively smooth variations in thickness or properties of the geologic units, or	Site is uniform based on boring data and placement of engineered backfill	Subsection 2.5.4.4 and Subsection 2.5.4.5	Yes
	2 Site specific assessment of subsurface conditions demonstrates that the bearing pressures below the footprint of the nuclear island do not exceed 120% of those from the generic analyses of the nuclear island at a uniform site, or	N/A		
	3 Site specific analysis of the nuclear island basemat demonstrates that the site specific demand is within the capacity of the basemat.	N/A		
	As an example of sites that are considered uniform, the variation of shear wave velocity in the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint and 40 feet beyond the boundaries of the nuclear island footprint meets the criteria in the case outlined below:			

Table 2.0-201 (Sheet 6 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[(a)]		VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Lateral Variability (Continued)	Case 1: For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, should have a dip not greater than 20 degrees, and should have less than 20 percent variation in the shear wave velocity from the average velocity than any layer.		N/A		
Limits of Acceptable Settlement Without Additional Evaluation[(i)]	Differential Across Nuclear Island Foundation Mat	1/2 inch in 50 ft	~15/32 inch in 50 ft (projected)	Subsection 2.5.4.10.2	Yes (projected)
	Total for Nuclear Island Foundation Mat	6 inches	4–5 inches (projected)		
	Differential Between Nuclear Island and Turbine Building[(j)]	3 inches	<2 inch (projected)		
	Differential Between Nuclear Island and Other Buildings[(j)]	3 inches	<3 inch (projected)		

Table 2.0-201 (Sheet 7 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Liquefaction Potential	No liquefaction considered beneath the seismic Category I and seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the seismic Category I and seismic Category II structures.	None at the site-specific SSE.	Table 2.0-203	Yes
Minimum Soil Angle of Internal Friction	Minimum soil angle of internal friction is greater than or equal to 35 degrees below the footprint of nuclear island at its excavation depth. If the minimum soil angle of internal friction is below 35 degrees, a site specific analysis shall be performed using the site specific soil properties to demonstrate stability.	36 degrees	Table 2.0-203	**Yes**
Missiles				
Tornado	4000-lb automobile at 105 mph horizontal, 74 mph vertical 275-lb, 8-in. shell at 105 mph horizontal, 74 mph vertical 1-inch-diameter steel ball at 105 mph in the most damaging direction	4000-lb automobile at 105 mph horizontal, 74 mph vertical 275-lb, 8-in. shell at 105 mph horizontal, 74 mph vertical 1-inch-diameter steel ball at 105 mph in the most damaging direction	Subsection 3.5.1.5 Subsection 3.5.1.4 APP-GW-GLR-020, “Wind and Tornado Site Interface Criteria,” Westinghouse Electric Company LLC.[e]	Yes
Flood Level	Less than plant elevation 100 feet	The design basis river flood level is El. 178.10 ft MSL, which is 41.9 feet below plant elevation (220 ft MSL). Maximum local PMP flood elevation is 219.47 ft MSL, which is 0.53 feet below plant elevation (220 ft MSL).	Table 2.0-203 Subsection 2.4.2	Yes

Table 2.0-201 (Sheet 8 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Ground Water Level	Less than plant elevation 98 feet	The maximum groundwater level is 165 ft MSL which is 55 feet below plant elevation (220 ft MSL).	Table 2.0-203	Yes
Plant Grade Elevation	Less than plant elevation 100 feet, except for portion at a higher elevation adjacent to the annex building	The standard plant-floor elevation of the safety-related facilities is established at plant elevation 220 ft MSL; the finished plant grade elevation slopes away from plant structures	Figure 2.4-201	Yes
Precipitation				
Rain	20.7 in/hr [1-hr 1-mi^2 PMP]	19.2 in/hr	Table 2.0-203	Yes
Snow/Ice	75 pounds per square foot on ground with exposure factor of 1.0 and importance factors of 1.2 (safety) and 1.0 (non-safety)	10.0 pounds per square foot	Table 2.0-203	Yes
Atmospheric Dispersion Values - χ/Q[f]				
Site Boundary (annual average)	$\leq 2.0 \times 10^{-5}$ sec/m^3	0.55×10^{-5} sec/m^3	Table 2.0-203	Yes
Site Boundary (0-2 hr)	$\leq 5.1 \times 10^{-4}$ sec/m^3 [g]	3.49×10^{-4} sec/m^3	Table 2.0-203	Yes
Low population zone boundary[g]				
0–8 hr	$\leq 2.2 \times 10^{-4}$ sec/m^3	7.04×10^{-5} sec/m^3	Table 2.0-203	Yes
8–24 hr	$\leq 1.6 \times 10^{-4}$ sec/m^3	5.25×10^{-5}sec/m^3	Table 2.0-203	Yes
24–96 hr	$\leq 1.0 \times 10^{-4}$ sec/m^3	2.77×10^{-5} sec/m^3	Table 2.0-203	Yes
96–720 hr	$\leq 8.0 \times 10^{-5}$ sec/m^3	1.11×10^{-5} sec/m^3	Table 2.0-203	Yes
Control Room	Table 2.0-202	Table 2.0-202	Table 2.0-202	Yes

Table 2.0-201 (Sheet 9 of 9)
Comparison of AP1000 DCD Site Parameters and Vogtle Electric Generating Plant Units 3 & 4 Site Characteristics

	AP1000 DCD Site Parameter[a]	VEGP Site Characteristic	VEGP Reference	VEGP Within Site Parameter
Population Distribution[g]				
Exclusion area (site)	0.5 mi.	The minimum distance from the effluent release boundary to the exclusion area boundary is 0.50 mile.[f]	Table 2.0-203	Yes

(a) AP1000 DCD Site Parameters are a compilation of DCD Tier 1 Table 5.0-1 and DCD Tier 2 Table 2-1.

(b) Maximum and minimum safety values are based on historical data and exclude peaks of less than 2 hours duration.

(c) The maximum normal value is the 1-percent seasonal exceedance temperature. The minimum normal value is the 99-percent seasonal exceedance temperature. The minimum temperature is for the months of December, January, and February in the northern hemisphere. The maximum temperature is for the months of June through September in the northern hemisphere. The 1-percent seasonal exceedance is approximately equivalent to the annual 0.4-percent exceedance. The 99-percent seasonal exceedance is approximately equivalent to the annual 99.6-percent exceedance. See Subsection 2.3.1.5 for further discussion on this relationship.

(d) The noncoincident wet bulb temperature is applicable to the cooling tower only.

(e) Per APP-GW-GLR-020, the kinetic energies of the missiles discussed in Section 3.5 are greater than the kinetic energies of the missiles discussed in Regulatory Guide 1.76 and result in a more conservative design.

(f) For AP1000, the term “site boundary” and “exclusion area boundary” are used interchangeably. Thus, the χ/Q specified for the site boundary applies whenever a discussion refers to the exclusion area boundary. At VEGP the “site boundary” and ”exclusion area boundary” are not interchangeable. See Figure 1.1-202.

(g) Site Interface Values for Post-Accident Dose Consequences and Minimum Distance to Site Boundary are reported per Table 2.0-203. Cooling Tower Make-up Flow Rate, which is not an AP1000 DCD Site Parameter, is 61,145 gpm (2 units) per Table 2.0-203.

(h) The containment pressure response analysis is based on a conservative set of dry-bulb and wet-bulb temperatures. These results envelope any conditions where the dry-bulb temperature is 115°F or less and wet-bulb temperature is less than or equal to 86.1°F.

(i) Additional evaluation may include evaluation of the impact of the elevated estimated settlement values on the critical components of the AP1000, determining a construction sequence to control the predicted settlement behavior, or developing an active settlement monitoring system throughout the entire construction sequence as well as a long-term (plant operation) plan.

(j) Differential settlement is measured at center of Nuclear Island and center of adjacent structures.

Table 2.0-202 (Sheet 1 of 2)
Comparison of Control Room Atmospheric Dispersion Factors for Accident Analysis for AP1000 DCD and VEGP Units 3 & 4

X/Q (sec/m^3) at HVAC Intake for the Identified Release Points[(a)]

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	**Ground Level Containment Release Points[(c)]**	Ground Level Containment Release Points	**PORV and Safety Valve Releases[(d)]**	PORV and Safety Valve Releases	**Condenser Air Removal Stack[(g)]**	Condenser Air Removal Stack	**Steam Line Break Releases**	**Steam Line Break Releases**	**Fuel Handling Area[(e)]**	Fuel Handling Area Blowout Panel	Fuel Handling Area Truck Bay Door
Release Time	**DCD**	VEGP	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	VEGP
0 – 2 hours	**2.53E-03**	2.27E-03	1.71E-03	**4.00E-03**	2.93E-03	**1.92E-02**	1.77E-02	**6.0E-3**	6.60E-04	**2.13E-02**	1.87E-02	**6.0E-3**	1.57E-03	1.30E-03
2 – 8 hours	**1.98E-03**	1.86E-03	1.32E-03	**2.28E-03**	1.75E-03	**1.60E-02**	1.41E-02	**4.0E-3**	4.83E-04	**1.76E-02**	1.51E-02	**4.0E-3**	1.15E-03	9.36E-04
8 – 24 hours	**7.96E-04**	7.36E-04	5.56E-04	**1.03E-03**	7.78E-04	**6.90E-03**	6.25E-03	**2.0E-3**	2.17E-04	**7.50E-03**	6.79E-03	**2.0E-3**	4.62E-04	3.78E-04
1 – 4 days	**6.40E-04**	5.99E-04	4.63E-04	**9.03E-04**	6.81E-04	**4.96E-03**	4.61E-03	**1.5E-3**	1.57E-04	**5.43E-03**	4.94E-03	**1.5E-3**	3.72E-04	2.98E-04
4 – 30 days	**4.78E-04**	4.31E-04	3.43E-04	**7.13E-04**	5.30E-04	**4.16E-03**	3.87E-03	**1.0E-3**	1.17E-04	**4.55E-03**	4.14E-03	**1.0E-3**	2.68E-04	2.09E-04

X/Q (sec/m^3) at Annex Building Door for the Identified Release Points[(f)]

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	**Ground Level Containment Release Points[(c)]**	Ground Level Containment Release Points	**PORV and Safety Valve Releases[(d)]**	PORV and Safety Valve Releases	**Condenser Air Removal Stack[(g)]**	Condenser Air Removal Stack	**Steam Line Break Releases**	**Steam Line Break Releases**	**Fuel Handling Area[(e)]**	Fuel Handling Area Blowout Panel	Fuel Handling Area Truck Bay Door
Release Time	**DCD**	VEGP	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	**DCD**	VEGP	VEGP
0 – 2 hours	**1.0E-3**	5.02E-04	4.62E-04	**1.0E-3**	3.97E-04	**4.0E-3**	1.13E-03	**2.0E-2**	1.72E-03	**4.0E-3**	1.00E-03	**6.0E-3**	3.99E-04	3.83E-04
2 – 8 hours	**7.5E-4**	3.94E-04	3.55E-04	**7.5E-4**	3.26E-04	**3.2E-3**	8.98E-04	**1.8E-2**	1.12E-03	**3.2E-3**	7.97E-04	**4.0E-3**	3.00E-04	2.88E-04
8 – 24 hours	**3.5E-4**	1.61E-04	1.49E-04	**3.5E-4**	1.34E-04	**1.2E-3**	3.69E-04	**7.0E-3**	4.50E-04	**1.2E-3**	3.25E-04	**2.0E-3**	1.22E-04	1.21E-04
1 – 4 days	**2.8E-4**	1.29E-04	1.23E-04	**2.8E-4**	1.10E-04	**1.0E-3**	2.92E-04	**5.0E-3**	3.17E-04	**1.0E-3**	2.58E-04	**1.5E-3**	1.00E-04	9.58E-05
4 – 30 days	**2.5E-4**	9.63E-05	9.12E-05	**2.5E-4**	8.32E-05	**8.0E-4**	2.19E-04	**4.5E-3**	2.60E-04	**8.0E-4**	1.91E-04	**1.0E-3**	7.23E-05	6.78E-05

Table 2.0-202 (Sheet 2 of 2)
Comparison of Control Room Atmospheric Dispersion Factors for Accident Analysis for AP1000 DCD and VEGP Units 3 & 4

a. These dispersion factors are to be used 1) for the time period preceding the isolation of the main control room and actuation of the emergency habitability system, 2) for the time after 72 hours when the compressed air supply in the emergency habitability system would be exhausted and outside air would be drawn into the main control room, and 3) for the determination of control room doses when the non-safety ventilation system is assumed to remain operable such that the emergency habitability system is not actuated.

b. These dispersion factors are used for analysis of the doses due to a postulated small line break outside of containment. The plant vent and PCS air diffuser are potential release paths for other postulated events (loss of-coolant accident, rod ejection accident, and fuel handling accident inside the containment); however, the values are bounded by the dispersion factors for ground level releases.

c. The listed values represent modeling the containment shell as a diffuse area source, and are used for evaluating the doses in the main control room for a loss-of-coolant accident, for the containment leakage of activity following a rod ejection accident, and for a fuel handling accident occurring inside the containment.

d. The listed values bound the dispersion factors for releases from the steam line safety & power-operated relief valves. These dispersion factors would be used for evaluating the doses in the main control room for a steam generator tube rupture, a main steam line break, a locked reactor coolant pump rotor, and for the secondary side release from a rod ejection accident.

e. The listed values bound the dispersion factors for releases from the fuel storage and handling area. The listed values also bound the dispersion factors for releases from the fuel storage area in the event that spent fuel boiling occurs and the fuel handling area relief panel opens on high temperature. These dispersion factors are used for the fuel handling accident occurring outside containment and for evaluating the impact of releases associated with spent fuel pool boiling.

f. These dispersion factors are to be used when the emergency habitability system is in operation and the only path for outside air to enter the main control room is that due to ingress/egress.

g. This release point is included for information only as a potential activity release point. None of the design basis accident radiological consequences analyses model release from this point.

Table 2.0-203 (Sheet 1 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Precipitation		
Maximum Rainfall Rate	19.2 inches in 1 hr 6.2 inches in 5 min	PMP for 1-hr and 5-min duration of precipitation at the site. Refer to Table 2.4-220 and Figure 2.4-210
100-Year Snow Pack 48-Hour Winter Probable Maximum Precipitation (PMP)	10 lb/sq ft 28.3 in.	Weight, per unit area, of the 100-year return period snowpack at the site Maximum probable winter rainfall in 48-hour period. Refer to Subsection 2.3.1.3.4
Seismic		
Design Response Spectra	Site-specific GMRS values specified and illustrated in Subsection 2.5.2	Site-specific response spectra. Refer to Subsection 2.5.2 and Figures 2.5-316, 2.5-317, and 2.5-318.
Capable Tectonic Structures or Sources	No fault displacement potential within the investigative area	Conclusion on the presence of capable faults or earthquake sources in the vicinity of the plant site. Refer to Subsections 2.5.1.1.4, 2.5.1.2.4, and 2.5.3; Table 2.5-235
Water		
Maximum Flood (or Tsunami)	178.10 ft msl	Water level at the site due to dam breach. Refer to Subsections 2.4.2.2, 2.4.3.4, 2.4.4.3, and 2.4.10
Maximum Groundwater	165 ft msl	Site basis for subsurface hydrostatic loading due to difference in elevation between the site grade elevation in the power block area and the maximum site groundwater level. Refer to Subsections 2.4.12.4 and 2.5.4.6.1

Table 2.0-203 (Sheet 2 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Subsurface Material Properties		
Liquefaction	None at site-specific SSE. Compacted structural fill will provide an adequate safety factor against liquefaction (min >1.1).	Liquefaction potential for subsurface material at the site. Refer to Subsection 2.5.4.8.4
Minimum Bearing Capacity (Static and Dynamic)	34,000 lb/sq ft (Static) 42,000 lb/sq ft (Dynamic)	Allowable load-bearing capacity of the layer supporting plant structures. Refer to Subsection 2.5.4.10.1
Minimum Shear Wave Velocity	Values in Tables 2.5-251 and 2.5-253	Propagation velocity of shear waves through the foundation materials. Refer to Subsection 2.5.4.7.1; Tables 2.5-251 and 2.5-253; Figures 2.5-376, 2.5-378, 2.5-379, and 2.5-380
Tornado		
Maximum Pressure Drop	2.0 psi	Decrease in ambient pressure from normal atmospheric pressure at the site due to passage of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2
Maximum Rotational Speed	240 mph	Rotation component of maximum wind speed at the site due to passage of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2
Maximum Translational Speed	60 mph	Translation component of maximum wind speed at the site due to the movement across ground of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2

Table 2.0-203 (Sheet 3 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Maximum Wind Speed	300 mph	Sum of the maximum rotational and maximum translational wind speed components at the site due to passage of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2
Radius of Maximum Rotational Speed	150 ft	Distance from the center of the tornado at which the maximum rotational wind speed occurs at the site due to passage of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2
Maximum Rate of Pressure Drop	1.2 psi/sec	Maximum rate of pressure drop at the site due to passage of a tornado having a probability of occurrence of 10^{-7} per year. Refer to Subsection 2.3.1.3.2
Wind		
Basic Wind Speed	104 mph	Three-second gust wind velocity, associated with a 100-year return period, at 33 ft (10 m) above ground level in the site area. Refer to Subsection 2.3.1.3.2
Selected Site Characteristic Ambient Air Temperatures		*(Site characteristic wet bulb and dry bulb temperatures associated with listed exceedance values and 100-year return period)*
Maximum Dry Bulb		Refer to Subsection 2.3.1.5
• 2% annual exceedance	92°F	
• 0.4% annual exceedance	97°F	
• 100-year return period	115°F	

Table 2.0-203 (Sheet 4 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Minimum Dry Bulb • 1% annual exceedance • 0.4% annual exceedance • 100-year return period	 25°F 21°F -8°	Refer to Subsection 2.3.1.5
Maximum Wet Bulb • 0.4% annual exceedance • 100-year return period	 79°F 88°F	Refer to Subsection 2.3.1.5
Site Temperature Basis for AP1000 • Maximum Safety Dry Bulb and Coincident Wet Bulb • Maximum Safety Wet Bulb (Non-coincident) • Maximum Normal Dry Bulb and Coincident Wet Bulb • Maximum Normal Wet Bulb (Non-coincident)	 115°F dry bulb/77.7°F wet bulb 83.9°F 97°F dry bulb/76°F wet bulb 79°F	Refer to Subsection 2.3.1.5
Airborne Effluent Release Point		
Atmospheric Dispersion (χ/Q) (Accident)		
0-2 hr @ Exclusion Area Boundary (EAB)	3.49E-04 sec/m^3	The atmospheric dispersion coefficients used in the design safety analysis to estimate dose consequences of accident airborne releases. Refer to Subsection 2.3.4.2.
0-8 hr @ Low Population Zone (LPZ)	7.04E-05 sec/m^3	
8-24 hr @ LPZ	5.25E-05 sec/m^3	
1-4 day @ LPZ	2.77E-05 sec/m^3	
4-30 day @ LPZ	1.11E-05 sec/m^3	

Table 2.0-203 (Sheet 5 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Atmospheric Dispersion (χ/Q) (Routine Release)		
Annual Average Undepleted/No Decay χ/Q Value @ EAB	5.5E-06 sec/m^3	The maximum annual average EAB undepleted/no decay atmospheric dispersion factor (χ/Q) value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Subsection 2.3.5.2; Table 2.3-219
Annual Average Undepleted/ 2.26-Day Decay χ/Q Value @ EAB	5.5E-06 sec/m^3	The maximum annual average EAB undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Depleted/ 8.00-Day Decay χ/Q Value @ EAB	5.0E-06 sec/m^3	The maximum annual average EAB depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average D/Q Value @ EAB	1.7E-08 1/m^2	The maximum annual average EAB relative deposition factor (D/Q) value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Resident	3.4E-06 sec/m^3	The maximum annual average resident undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Subsection 2.3.5.2; Table 2.3-219

Table 2.0-203 (Sheet 6 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Annual Average Undepleted/ 2.26-Day Decay χ/Q Value @ Nearest Resident	3.4E-06 sec/m^3	The maximum annual average resident undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Depleted/ 8.00-Day Decay χ/Q Value @ Nearest Resident	3.0E-06 sec/m^3	The maximum annual average resident depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average D/Q Value @ Nearest Resident	1.0E-08 1/m^2	The maximum annual average resident D/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Meat Animal	3.4E-06 sec/m^3	The maximum annual average meat animal undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Subsection 2.3.5.2; Table 2.3-219
Annual Average Undepleted/ 2.26-Day Decay χ/Q Value @ Nearest Meat Animal	3.4E-06 sec/m^3	The maximum annual average meat animal undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Depleted/ 8.00-Day Decay χ/Q Value @ Nearest Meat Animal	3.0E-06 sec/m^3	The maximum annual average meat animal depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219

Table 2.0-203 (Sheet 7 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Annual Average D/Q Value @ Nearest Meat Animal	1.0E-08 $1/m^2$	The maximum annual average meat animal D/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Vegetable Garden	3.4E-06 sec/m^3	The maximum annual average vegetable garden undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Undepleted/ 2.26-Day Decay χ/Q Value @ Nearest Vegetable Garden	3.4E-06 sec/m^3	The maximum annual average vegetable garden undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average Depleted/ 8.00-Day Decay χ/Q Value @ Nearest Vegetable Garden	3.0E-06 sec/m^3	The maximum annual average vegetable garden depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219
Annual Average D/Q Value @ Nearest Vegetable Garden	1.0E-08 $1/m^2$	The maximum annual average vegetable garden D/Q value for use in determining gaseous pathway doses to the maximally exposed individual. Refer to Table 2.3-219

Table 2.0-203 (Sheet 8 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part I Site Characteristics		
Item	**Value**	**Description and Reference**
Population Density		
Population Center Distance	Approximately 26 mi (Augusta, GA)	The minimum allowable distance from the reactor(s) to the nearest boundary of a densely populated center containing more than about 25,000 residents (not less than one and one-third times the distance from the reactor(s) to the outer boundary of the LPZ) (i.e., 2-2/3 mi for VEGP). Refer to Section 1.1, Subsections 1.2.1, 2.1.1, 2.1.3.2, and 2.1.3.5
Exclusion Area Boundary (EAB)	See Figure 1.1-202	The area surrounding the reactor(s), in which the reactor licensee has the authority to determine all activities, including exclusion or removal of personnel and property from the area. Refer to Subsections 2.1.1, 2.1.2, and 2.3.4.1; Figure 1.1-202
Low Population Zone (LPZ)	A 2-mile-radius circle from the midpoint between the containment buildings of Units 1 and 2.	The area immediately surrounding the exclusion area that contains residents. Refer to Subsections 2.1.3.4, 2.3.4.1, 2.3.4.2, and 2.3.5.1; Table 2.3-217
Dose Calculation EAB	See Figure 1.1-202	A circle extending ½ mi beyond the power block area circle (775-ft radius circle encompassing Units 3 and 4). Total radius is 3,415 ft from the centroid of the power block circle. Dose Calculation EAB is completely within the actual plant EAB and is used to conservatively determine χ/Q values and subsequent accident radiation doses. Refer to Subsections 2.3.4.1, 2.3.4.2, and 2.3.5.1; Tables 2.3-216, 2.3-218, and 2.3-219; Figure 1.1-202

Table 2.0-203 (Sheet 9 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part II Design Parameters		
Item	**Single Unit [Two Unit] Value**	**Description and Reference**
Structures		
Building Height	234 ft 0 in.	The height from finished grade to the top of the tallest power blocks structure, excluding cooling towers (i.e., Containment Building). Refer to Subsection 2.3.3.3
Building Foundation Embedment	39 ft 6 in. to bottom of basemat from plant grade	The depth from finished grade to the bottom of the basemat for the most deeply embedded power block structure (i.e., Containment/Auxiliary Building). Refer to Subsections 2.4.12 and 2.5.4.10.1
Cooling Tower Height	600 ft	The height is from the finished grade to the top of the cooling tower Refer to Subsection 2.3.3.3
Cooling Tower Base Diameter	550 ft	The bottom of the cooling tower where it connects to the basin Refer to Subsection 2.3.3.3
Cooling Tower Diameter at the Top	330 ft	The cooling tower diameter at its highest elevation Refer to Subsection 2.3.3.3
Airborne Effluent Release Point		
Release Point Elevation (Post-Accident)	Ground level	The elevation above finished grade of the release point for accident sequence releases. Refer to Subsections 2.3.4.1 and 2.3.5.1; Tables 2.3-216 and 2.3-217

Table 2.0-203 (Sheet 10 of 10)
Site Characteristics, Design Parameters, and Site Interface Values

Part II Design Parameters		
Item	**Single Unit [Two Unit] Value**	**Description and Reference**
Plant Characteristics		
Megawatts Thermal	3,400 MWt [6,800 MWt]	The thermal power generated by one unit. Refer to Section 1.1, Subsections 1.2.2 and 1.3.2

Part III Site Interface Values		
Item	**Single Unit [Two Unit] Value**	**Description and Reference**
Normal Plant Heat Sink		
Cooling Tower Make-up Flow Rate	30,572 gpm [61,145 gpm]	The maximum rate of removal of water from the Savannah River to replace water losses from the circulating watersystem. The bounding Makeup Flow Rate is a calculated value based on the sum of the expected evaporation rate at design ambient conditions plus the bounding blowdown flow rate and drift. Refer to Subsections 2.4.1.2.6, 2.4.8, and 2.4.11.5; Table 2.4-217
Airborne Effluent Release Point		
Minimum Distance to Site Boundary	3,420 ft	The minimum lateral distance from the release point (power block area circle) to the site boundary. Refer to Figure 1.1-202

2.1 Geography and Demography

2.1.1 Site Location and Description

2.1.1.1 Site Location

Units 3 and 4 are on the existing VEGP site. 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 exclusion area boundary (EAB) is bounded by River Road, Hancock Landing Road, and 1.7 miles of the Savannah River (River Miles 150.0 to 151.7). The property boundary entirely encompasses the EAB and extends beyond River Road in some areas. The site is approximately 30 river miles above the US 301 bridge and directly across the river from the Department of Energy's (DOE's) Savannah River Site (SRS) (Barnwell County, South Carolina). The VEGP site is approximately 15 mi east-northeast of Waynesboro, Georgia, and 26 mi southeast of Augusta, Georgia, the nearest population center (i.e., having more than 25,000 residents). It is also about 100 mi from Savannah, Georgia, and 150 river miles from the mouth of the Savannah River.

The VEGP site is situated within three major resource areas: the Southern Piedmont, the Carolina and Georgia Sand Hills, and the Coastal Plain. These characteristics are typical of land forms that resulted from historical marine sediment deposits in central and eastern Georgia. There are no mountains in the general area.

Burke County includes five incorporated towns: Waynesboro, Girard, Keysville, Midville, and Sardis. Of these five towns, only the town of Girard is within 10 mi of the VEGP site. According to the 2000 Census survey, Girard, which has a population of 227, is the largest community within 10 mi of the VEGP site (Reference 207). Figure 2.1-201 shows Girard and its location with respect to the VEGP site. Access to the site is by River Road via US Route 25, Georgia Routes 56, 80, 24, and 23. A railroad spur connects the site to the Norfolk Southern Savannah-to-Augusta track.

Figure 2.1-202 shows highways, railways and airports located in the 50 mi surrounding area. The nearest highway, Interstate 20 (I-20), passing through Augusta and connecting Columbia, South Carolina, with Atlanta, Georgia, is located approximately 29 mi north of the VEGP site.

2.1.1.2 Site Description

VEGP Units 3 and 4 (Westinghouse Electric Company, LLC [Westinghouse] AP1000 certified reactor design plants) is located in the power block area shown in Figure 1.1-202. The centerline of VEGP Unit 3 is located approximately 1,700 ft west and 400 ft south of the center of the existing VEGP Unit 2 containment building. VEGP Unit 4 is approximately 800 ft west of VEGP Unit 3. The coordinates of the center of the containment building for VEGP Units 3 and 4 are as follows:

Unit	Georgia East Coordinates (NAD27) 1001 – Georgia East (US ft)		UTM Coordinates (NAD83) Zone 17 – 84W to 78W (m)		Latitude/Longitude (NAD83)(Deg/Min/Sec)	
3	N	1,142,600	N	3,667,170	N	33 08 27
	E	621,800	E	428,320	W	81 46 07
4	N	1,142,600	N	3,667,170	N	33 08 27
	E	621,000	E	428,070	W	81 46 16

No commercial, industrial, institutional, recreational, or residential structures are located within the site area, with the exception of Plant Wilson, the Georgia Power Company (GPC) combustion turbine plant. The nearest point to the exclusion area boundary (EAB) is located approximately 3,400 ft southwest of the VEGP Units 3 and 4 power block area.

2.1.1.3 Boundary for Establishing Effluent Release Limits

VEGP Units 3 and 4 are located within the power block area, which is the perimeter of a 775-ft-radius circle with the centroid at a point between the two AP1000 units. The EAB as described previously, is the same as the exclusion area boundary for the existing VEGP units. There are no residents in this exclusion area. No unrestricted areas within the site boundary are accessible to members of the public. Access within the property boundary is controlled as discussed in Subsection 2.1.2. Detailed discussion of effluent release points is provided in Subsection 2.3.5.

All areas outside the exclusion areas are unrestricted areas in the context of 10 CFR 20. Additionally, the guidelines provided in 10 CFR 50, Appendix I, for radiation exposures to meet the criterion “as low as is reasonably achievable” would be applied at the EAB.

2.1.2 Exclusion Area Authority and Control

The EAB is bounded by River Road, Hancock Landing Road, and 1.7 miles of the Savannah River (River Miles 150.0 to 151.7) as shown in Figure 1.1-202.

2.1.2.1 Authority

The co-owners own the entire plant exclusion area in fee simple including mineral rights. Pursuant to the VEGP owner’s agreement, GPC, for itself and as agent for the co-owners, has delegated to Southern Nuclear Operating Company, Inc. (SNC) complete authority to regulate any and all access and activity within the entire plant exclusion area.

The perimeter of the VEGP EAB is adequately posted with “No Trespassing” signs on land and with signs along the Savannah River, and indicate the actions to be taken in the event of emergency conditions at the plant.

2.1.2.2 Control of Activities Unrelated to Plant Operation

There are only two facilities within the EAB that have authorized activities unrelated to nuclear plant operations, the visitor’s center and the GPC combustion turbine plant, Plant Wilson.

The exclusion area outside the controlled area fence is posted and is closed to persons who have not received permission to enter the property.

The access route to the visitor's center is from River Road along the main plant access road to the road leading to the visitor’s center. Access to the visitor’s center is controlled by security at the pavilion (access control point) on the plant entrance road. Normally, only a few administrative personnel are located at the visitor’s center. Because of the remote location of the site, the number of visitors at the center is minimal. However, approved persons visiting the center will occupy the center and the area and parking lot immediately adjacent to the center. In the event of emergency conditions at the plant, the emergency plan provides for notification of visitors to the center concerning the proper actions to be taken and evacuation instructions. Plant Wilson is controlled and operated by VEGP staff. Access to the facility from New River Road is limited by locked gates. The emergency plan also provides for notification and evacuation of VEGP personnel at Plant Wilson.

SNC normally will not control passage or use of the Savannah River along the exclusion area boundary. “No trespassing” signs are posted near the river indicating the actions to be taken in the event of emergency conditions at the plant.

2.1.2.3 Arrangements for Traffic Control

No state or county roads, railways, or waterways traverse the VEGP exclusion area.

SNC has made arrangements with the Burke County Sheriff for control of traffic nearby in the event of an emergency. Evacuation of the EAB including the Visitors Center and Plant Wilson is addressed in the Emergency Plan.

2.1.3 Population Distribution

The population distribution surrounding the VEGP site, up to a 50-mi (80 km) radius, was estimated based on the year 2000 US Census Bureau decennial census data (Reference 202). The population distribution is estimated in 10 concentric bands at 0 to 1 mi, 1 to 2 mi, 2 to 3 mi, 3 to 4 mi, 4 to 5 mi, 5 to 10 mi, 10 to 20 mi, 20 to 30 mi, 30 to 40 mi, and 40 to 50 mi from the center of the power block area (generating facilities and switchyard), shown in Figure 1.1-202 and 16 directional sectors, each direction consisting of 22.5 degrees. The population projections for 2010, 2020, 2030, 2040, and 2070 have been estimated by calculating an annualized growth rate using the 1980 and 2000 census data (by county) as the base (Reference 204, Reference 206).

2.1.3.1 Resident Population Within 10 Mi

Figure 2.1-201 shows the general locations of the municipalities and other features within 10 mi (16 km) of the VEGP site. According to the 2000 Census, Girard, with a population of 227, is the largest community within 10 mi of the site (Reference 207). The population of Girard showed an increase of 16.4 percent in the last decade from a population of 195 in 1990 to a population of 227 in 2000 (Reference 205).

The population distribution within 10 mi of the site was computed by overlaying the 2000 Census block points data (the smallest unit of census data) on the grid shown in Figure 2.1-201 and summing the population of the census block points within each sector. SNC used SECPOP 2000, a code developed for the NRC by Sandia National Laboratories, to calculate population by emergency planning zone sectors (Reference 202). SECPOP uses 2000 block data from the US Census Bureau and overlays it into the sectors in the annuli prescribed by the user. The 1980 and 2000 population distributions for each county considered in Georgia and South Carolina were obtained from the U.S Census Bureau and used to calculate a growth rate over 20 years (Reference 204, Reference 206). Each county growth rate was annualized and used to project future populations within each sector, taking into account the percentage of each sector that each county occupied.

The population distributions and related information were collected and the results tabulated for all distances of interest in all 16 directions. All the north-northeast to east sectors in South Carolina are occupied by the SRS, which has no residents. SRS transients are accounted for in the SRS Emergency Plan and, therefore, are not included in the Emergency Plan. The SRS will remain a government-controlled facility in perpetuity. The SECPOP 2000 results show that in 2000, the combined resident and transient populations within 5 mi and 10 mi of the VEGP site were 687 and 3,560 persons, respectively. The resident and transient 10-mi population for 2000 and projections for 2010 through 2070 are shown in Figures 2.1-203 through 2.1-208 and are summarized in the table below.

Year	2000	2010	2020	2030	2040	2070
Population 0–10 miles	3,560	3,822	4,108	4,406	4,737	5,877

2.1.3.2 Resident Population Between 10 and 50 Mi

The 50-mi (80-km) radius centered at the VEGP site includes all, or parts of, 16 counties in Georgia, and 12 counties in South Carolina (Figure 2.1-209). Augusta, Georgia, approximately 26 mi northwest of the VEGP site, had a population of 195,182 in year 2000. Estimates of the year 2000 resident population between 10 and 50 mi from the VEGP site were computed using the same methodology used to develop the 10-mi population distribution.

The population grid to 50 mi is shown in Figure 2.1-209. The 10–50-mi population for 2000 and projections for 2010 through 2070 are shown in Figures 2.1-210 through 2.1-215. The total 0–50 mile population and projections are summarized in the table below.

Year	2000	2010	2020	2030	2040	2070
Population 0–50 miles	674,101	770,243	893,950	1,056,017	1,272,093	2,530,357

2.1.3.3 Transient Population

2.1.3.3.1 Transient Population Within 10 Miles

Information concerning transient population for the 10-mi radius was obtained from the Emergency Plan. The transient population includes hunters and fishermen at recreational areas along the Savannah River. Up to 200 hunters and fishermen may be located along the Savannah River on any weekend day during the hunting season (Reference 203). Although most hunters and fishermen likely reside in the area, this information is not definitive. Therefore, all hunters and fishermen were included as transient population. The construction workforce for VEGP Units 3 and 4 and the existing staff at VEGP were not included as transient population within 10 mi of the plant because they are counted as residents within the 10–50 mi radius area.

Portions of the SRS fall within 10 mi of the VEGP site. However, SRS workers are not counted as transient population in the Emergency Plan because SRS is responsible for its own evacuation plan. (Reference 203)

2.1.3.3.2 Transient Population Between 10 and 50 Miles

Colleges, schools, hospitals, a military base, and the SRS are between 10 and 50 mi from the VEGP site. In addition, thousands of people visit Augusta and the surrounding area out to the 50-mi limit annually during the week of the Masters Tournament and for other annual events within a 50-mi radius. However, compared to the resident population within a 50-mi radius, the transient population is expected to be very small.

2.1.3.4 Low Population Zone

The low population zone (LPZ) for VEGP Units 3 and 4 is the same as the LPZ for the existing VEGP units and consists of the area falling within a 2-mi radius of the midpoint between the VEGP Unit 1 and Unit 2 containment buildings. The resident and transient population distribution within the LPZ is indicated in Figures 2.1-203 through 2.1-208, based on the 2000 Census and projections through 2070. The LPZ population projections are also shown in the table below.

Year	2000	2010	2020	2030	2040	2070
Population	93	100	109	116	126	157

There are no schools in the LPZ. One private school is located approximately 9 mi west of the site, Lord's House of Praise Christian School, with an enrollment of approximately 50 students. S.G.A. Elementary School is the nearest public school and is located in the town of Sardis approximately 11 mi from the VEGP site (Reference 201). As stated in the previous section, the only significant transient population within 10 mi is hunters and fishermen along the banks of the Savannah River. Approximately 50 hunters and fishermen could be considered transient population within the LPZ. River Road is the only road within the LPZ. No towns, recreational facilities, hospitals, schools, prisons, or beaches are within the LPZ (Reference 203). Design basis accidents are evaluated in Chapter 15 to demonstrate that doses at the LPZ will be within the dose limits of 10 CFR 100.21(c) and 10 CFR 50.34(a)(1)(ii). Evacuation of the LPZ is addressed in the referenced Emergency Plan.

2.1.3.5 Population Center

The nearest population center to the VEGP site with more than 25,000 residents is the City of Augusta, Georgia, with a 2000 population of 195,182 (Reference 207). Augusta is approximately 26 miles north-northwest of the VEGP site.

2.1.3.6 Population Density

Regulatory Position C.4 of Regulatory Guide 4.7, *General Site Suitability Criteria for Nuclear Power Plants*, Revision 2, April 1998 (RG 4.7) and NRC Review Standard RS-002, *Processing Applications for Early Site Permits*, May 3, 2004 (RS-002) provide guidance on suitable population densities. Given an ESP approval date of 2010, a conservative startup date of 2030 (at the end of an ESP approval period of 20 years), and an operational period of 40 years, operations could extend until 2070. Figure 2.1-216 is a plot of population density to radial distance from the plant. Three VEGP site curves, one actual and two projected, were plotted to illustrate that the VEGP site vicinity population density is well below the regulatory guidance for population density. The three VEGP curves show the cumulative population in 2000 within 20 mi of the site and projected cumulative populations in 2040 and 2070. On the same figure, spanning the same radial distances, regulatory guidance population curves are plotted for hypothetical densities of 500 persons per square mile and 1,000 persons per square mile. Based on these projections, population densities, averaged over any radial distance out to 20 mi, are expected to be less than 500 persons per square mile over the lifetime of the new units. Figure 2.1-217 provides a representation of the LPZ that includes topographic features, as well as transportation routes (i.e., highways, railways, and waterways).

2.1.4 Combined License Information for Geography and Demography

Site-specific geography and demography information is addressed in Subsections 1.1.1, 1.2.2, and in Section 2.1.

2.1.5 References

201. **(BCS 2006)** *Burke County Schools,* BCS, 2006, available online at: http://www.burke.k12.ga.us, accessed April 5, 2006.

202. **(NRC 2003)** *SECPOP 2000: Sector Population, Land Fraction, and Economic Estimation Program*, Office of Nuclear Regulatory Research, US Nuclear Regulatory Commission, Washington, D.C., August 2003.

203. **(SNC 2004)** *Vogtle Electric Generating Plant Emergency Plan*, Revision 29, Southern Nuclear Operating Company, Inc., 2004.

204. **(USCB 1990a)** CPH-2-1. *1990 Census of Population and Housing, Population and Housing Unit Counts, United States, Table 30: Population and Housing Units: 1940 to 1990*, US Census Bureau, available online at: http://www.census.gov/population/www/censusdata/hiscendata.html, accessed June 1, 2005.

205. **(USCB 1990b)** *DP-1. General Population and Housing Characteristics*: 1990, US Census Bureau, Available online at http://factfinder.census.gov/, accessed June 3, 2005.

206. **(USCB 2000a)** *Census 2000 PHC-T-4. Ranking Tables for Counties;* 1990 and 2000, US Census Bureau, available online at http://www.census.gov, accessed June 2, 2005.

207. **(USCB 2000b)** *GCT-PH1. Population, Housing Units, Area, and Density*: 2000, US Census Bureau, available online at http://factfinder.census.gov, accessed June 3, 2005.

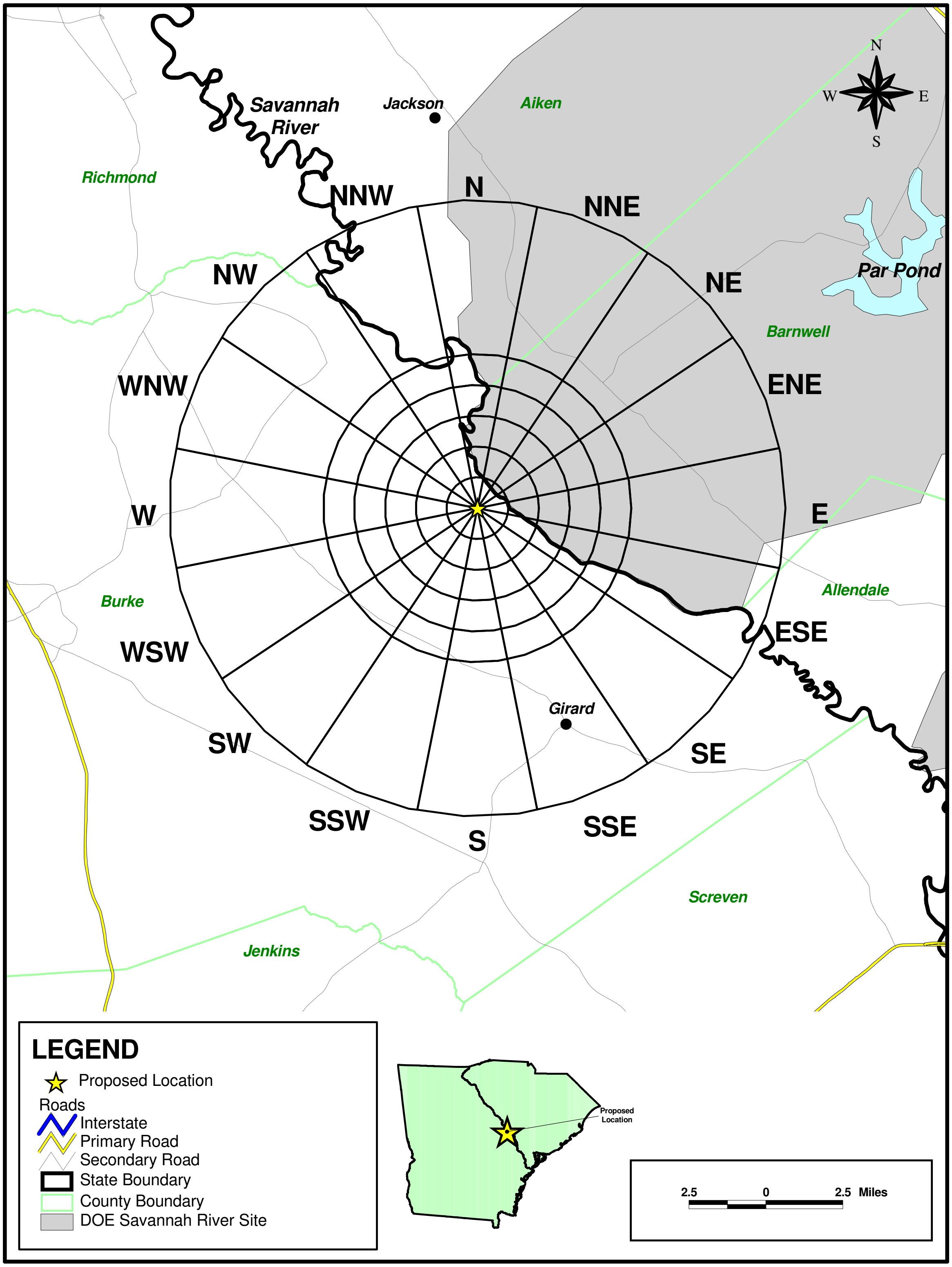


Figure 2.1-201
10-Mile Surrounding Area

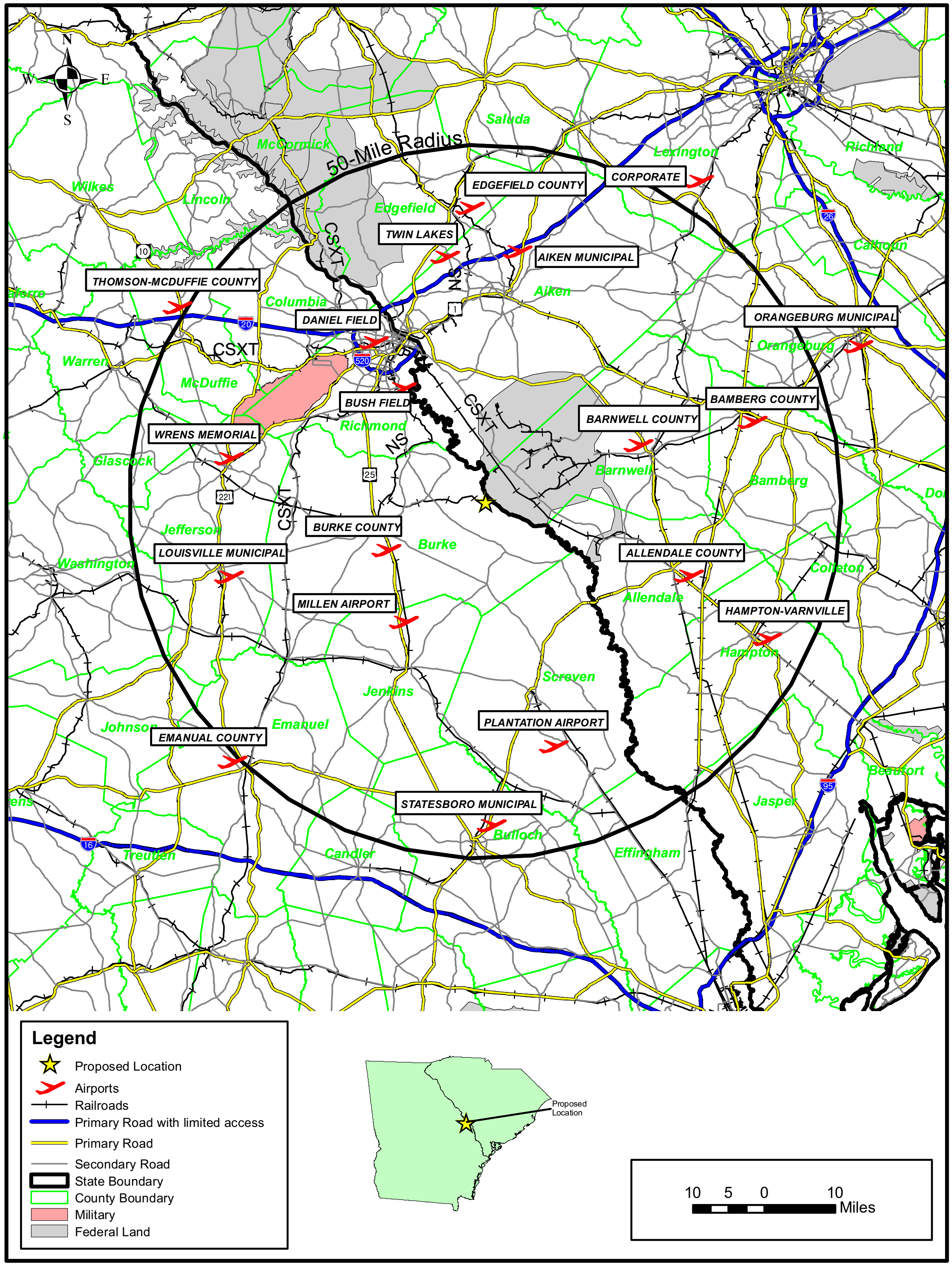


Figure 2.1-202
50-Mile Surrounding Area

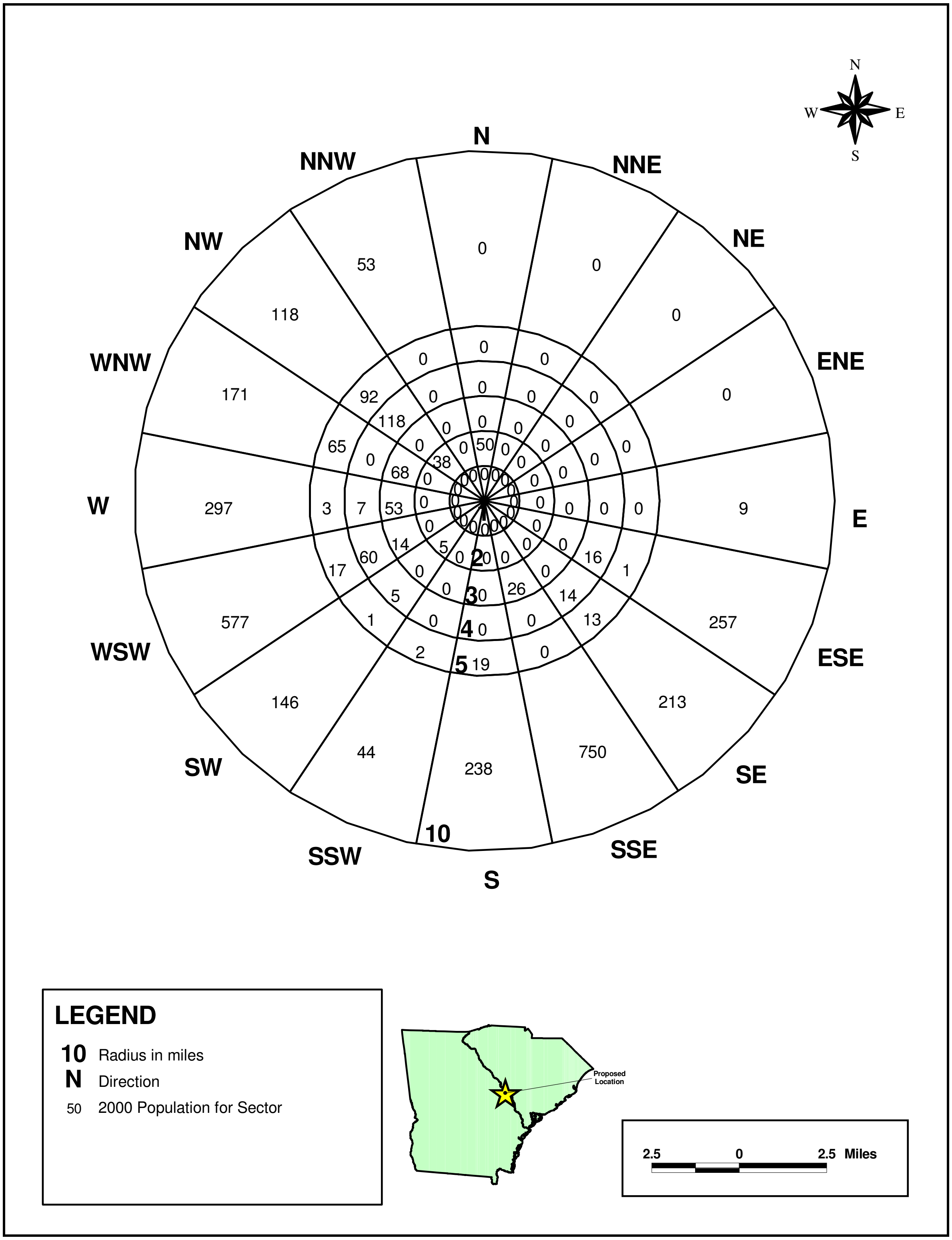


Figure 2.1-203
10-Mile Resident and Transient Population Distribution – 2000

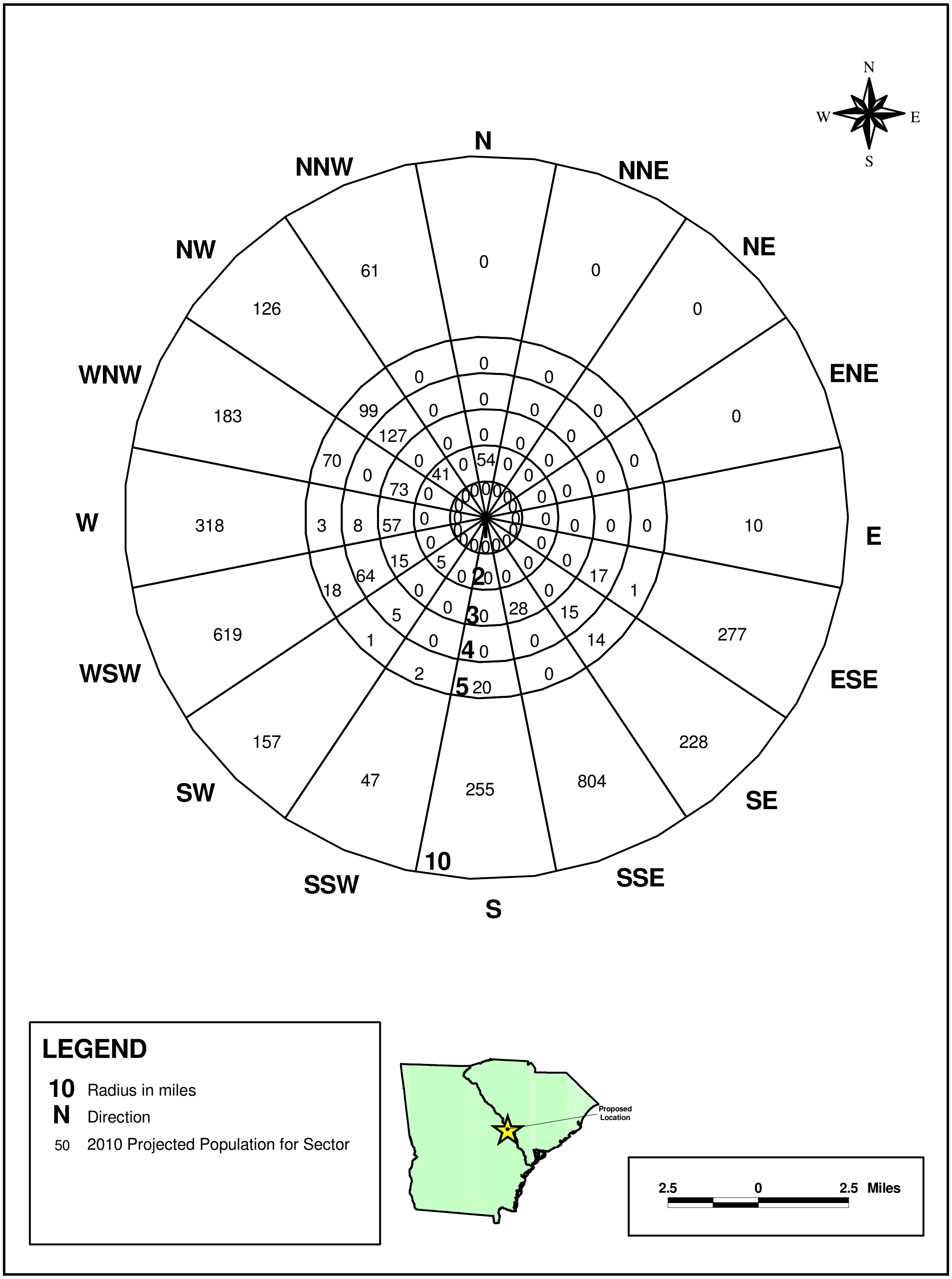


Figure 2.1-204
10-Mile Resident and Transient Population Distribution – 2010

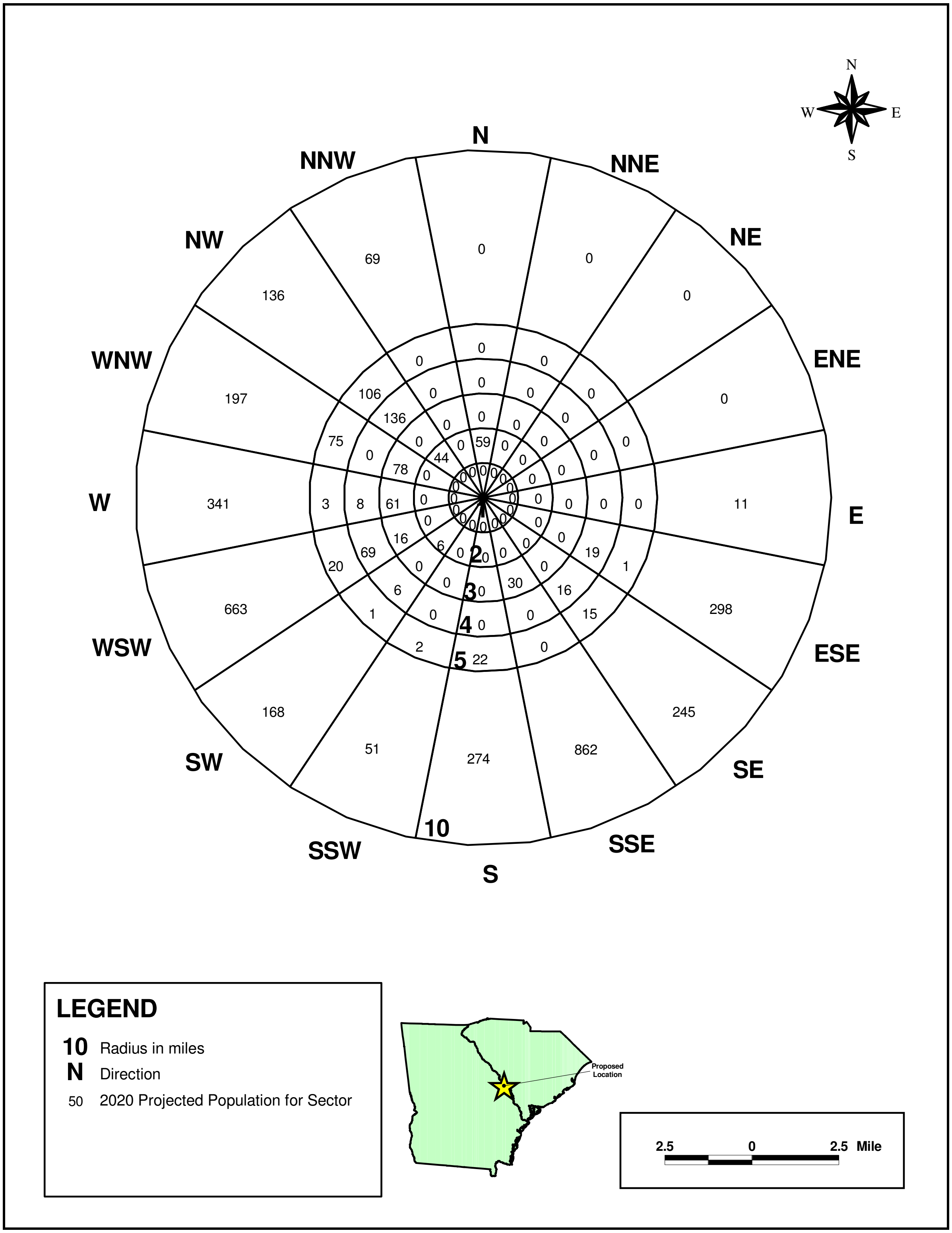


Figure 2.1-205
10-Mile Resident and Transient Population Distribution – 2020

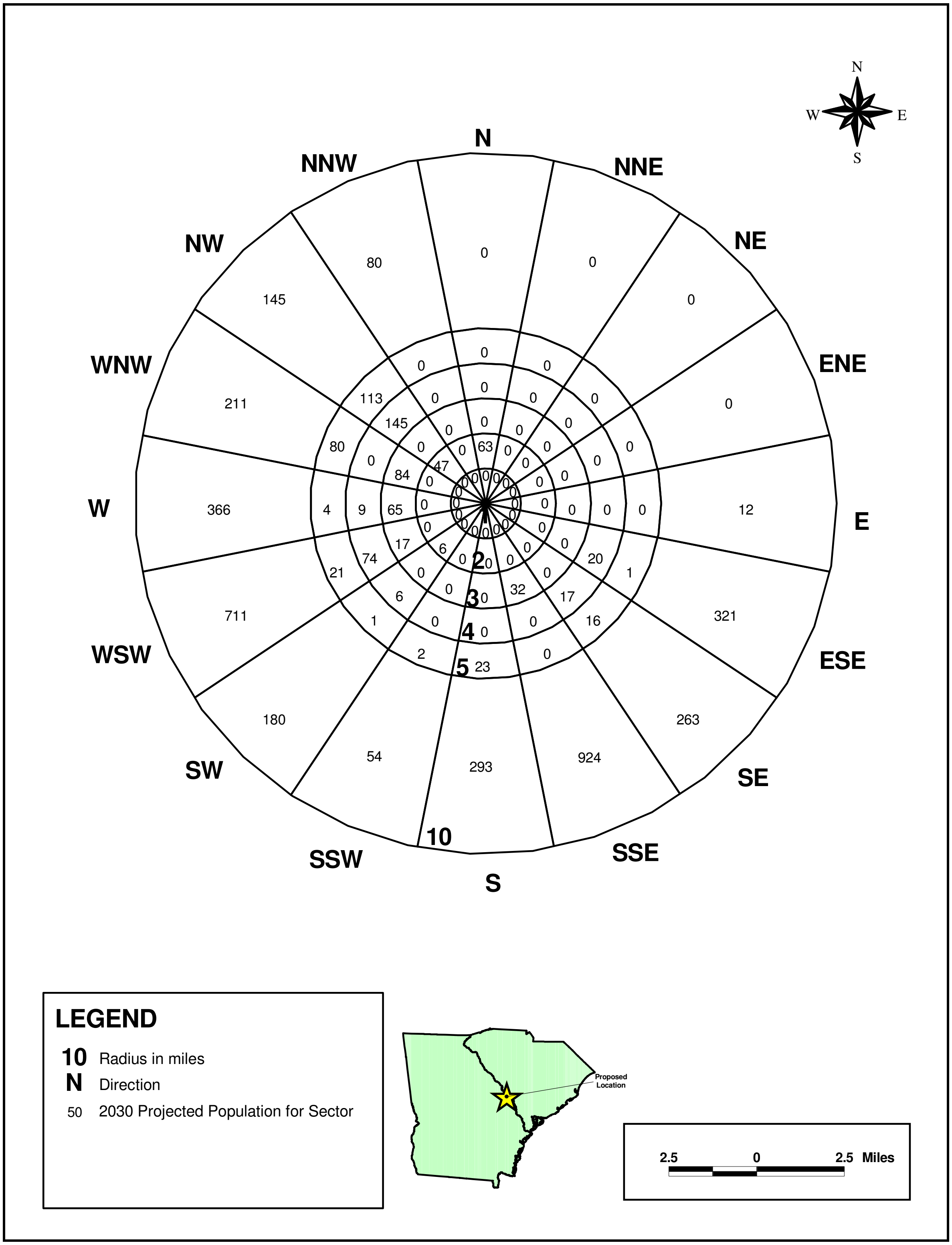


Figure 2.1-206
10-Mile Resident and Transient Population Distribution – 2030

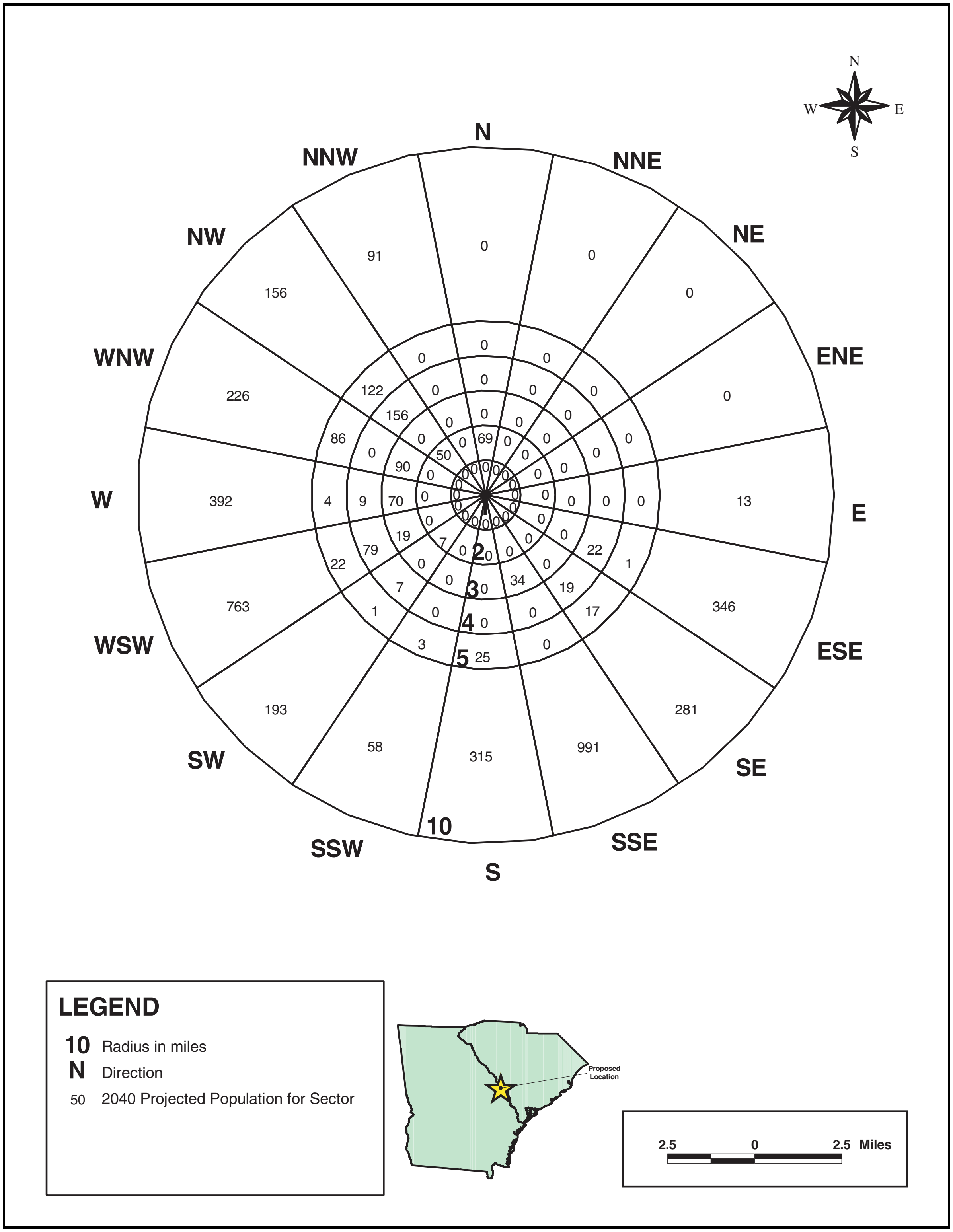


Figure 2.1-207
10-Mile Resident and Transient Population Distribution – 2040

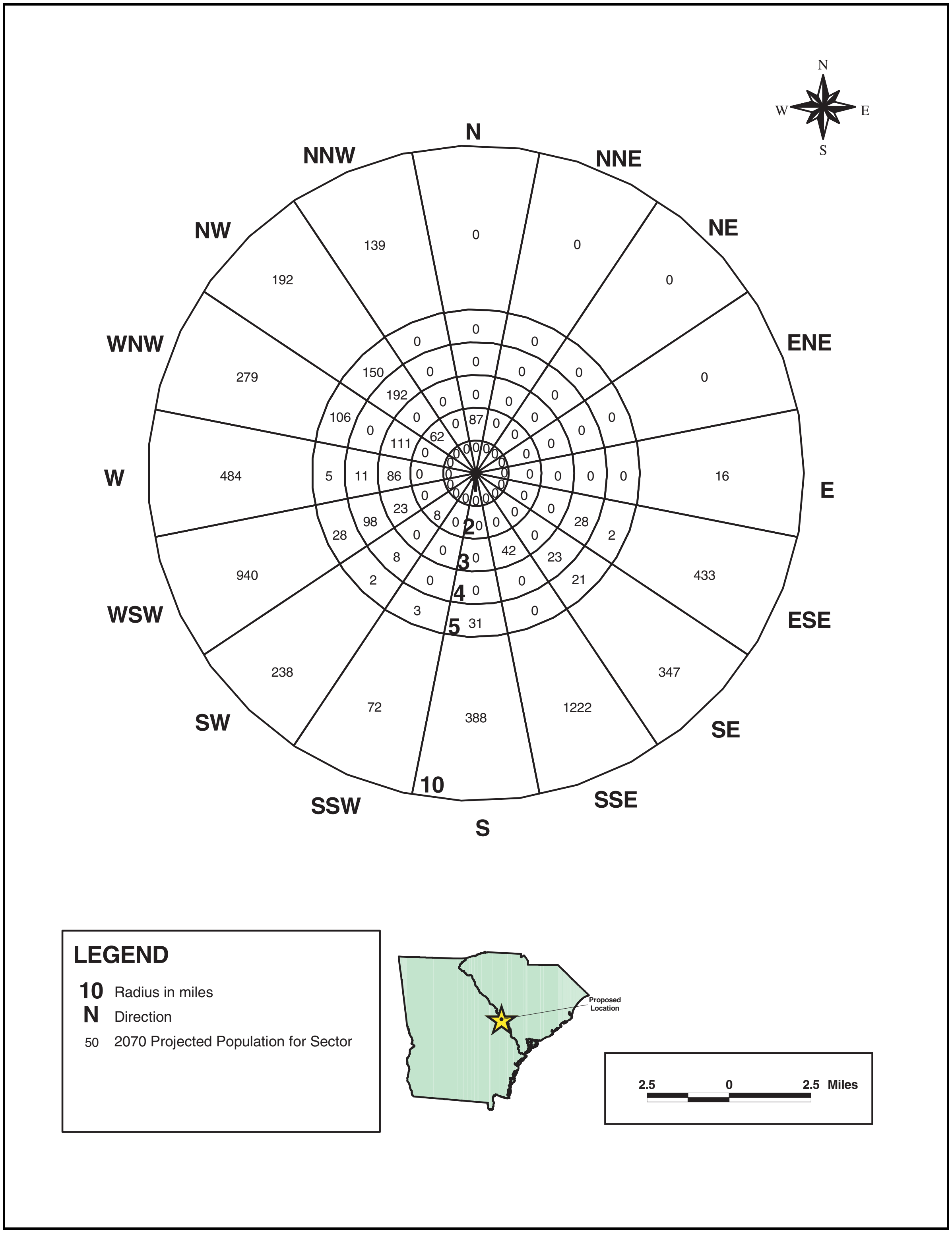


Figure 2.1-208
10-Mile Resident and Transient Population Distribution – 2070

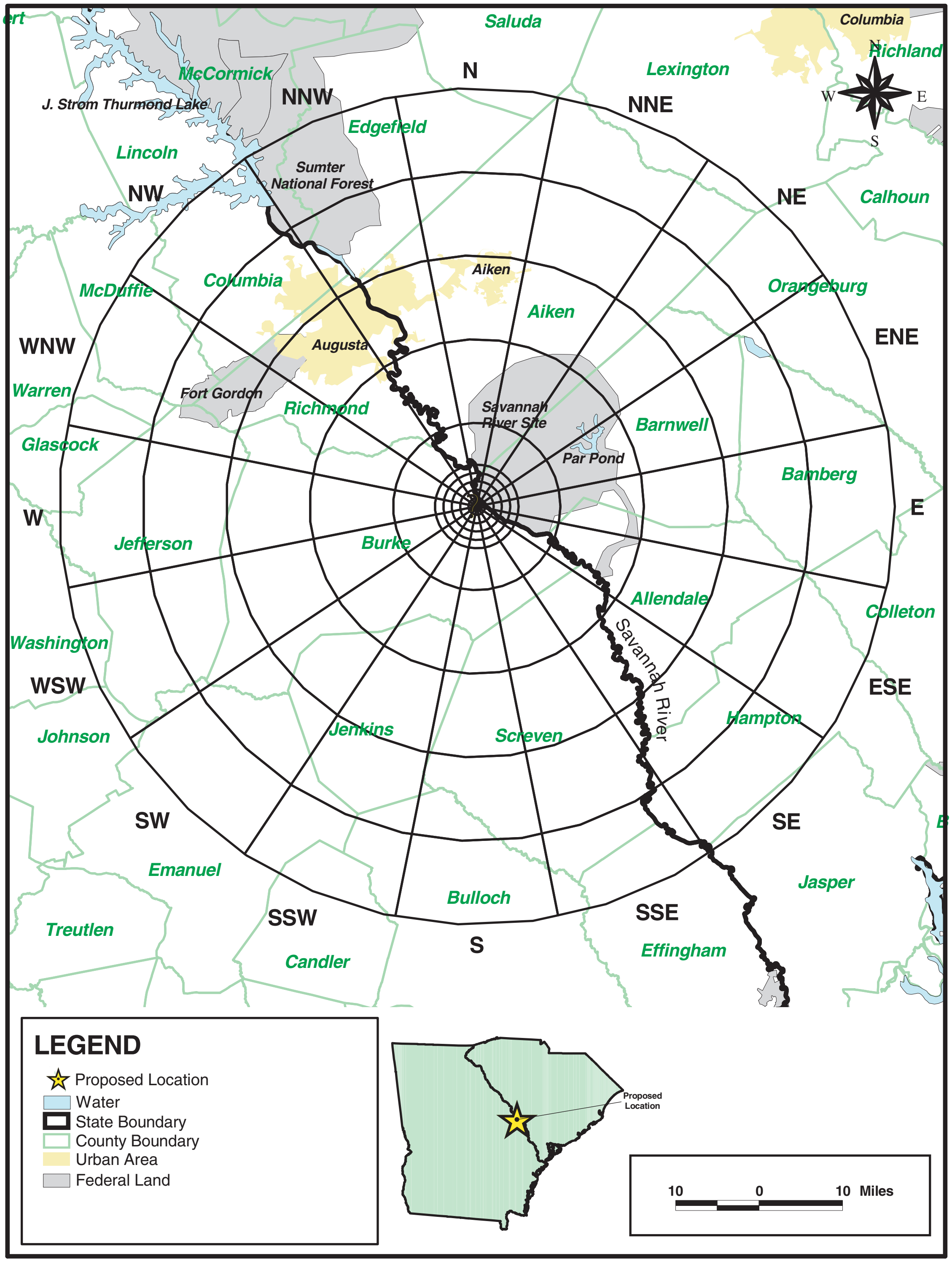


Figure 2.1-209
Population Grid Out to 50 Miles

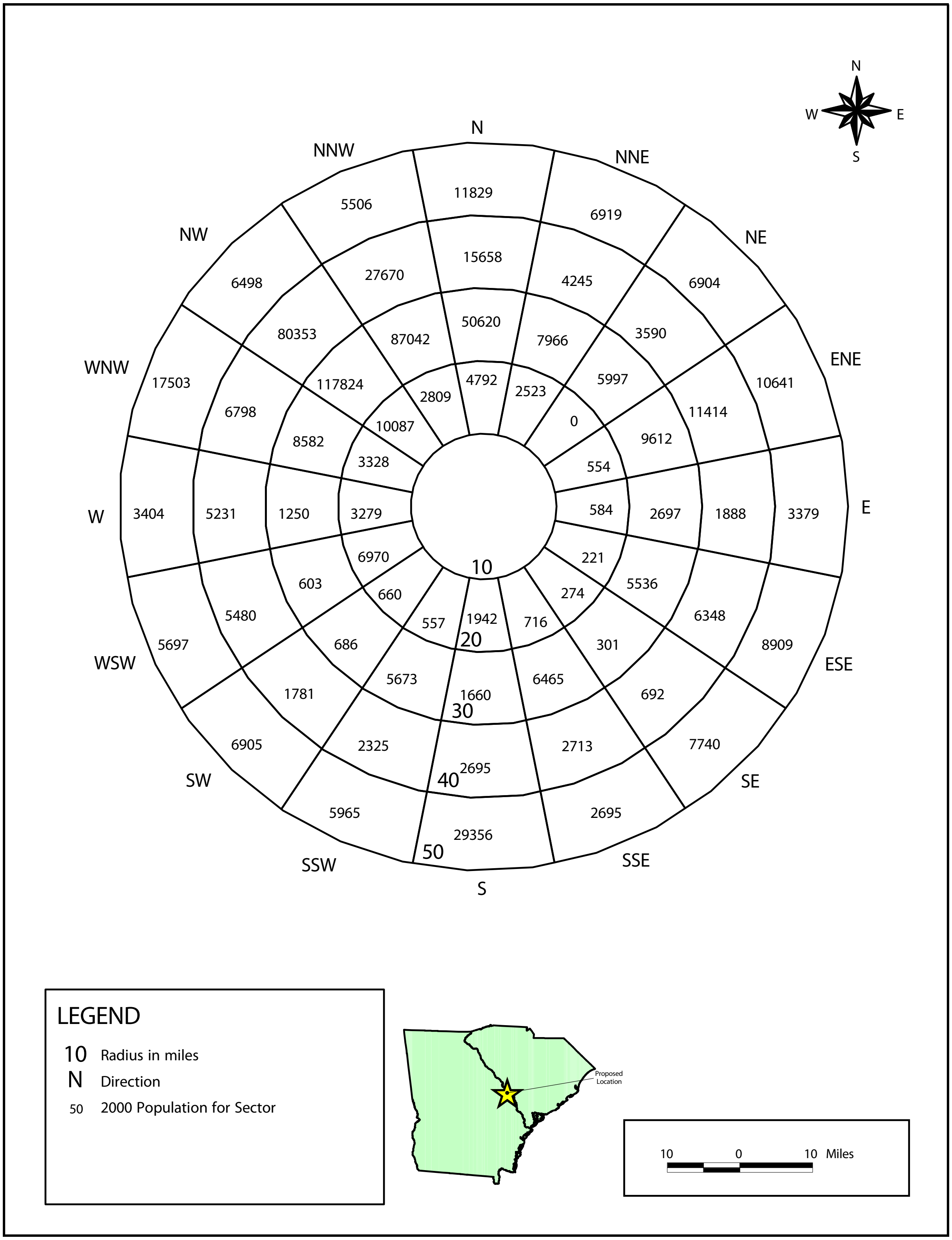


Figure 2.1-210
10 to 50-Mile Resident Population Distribution 2000

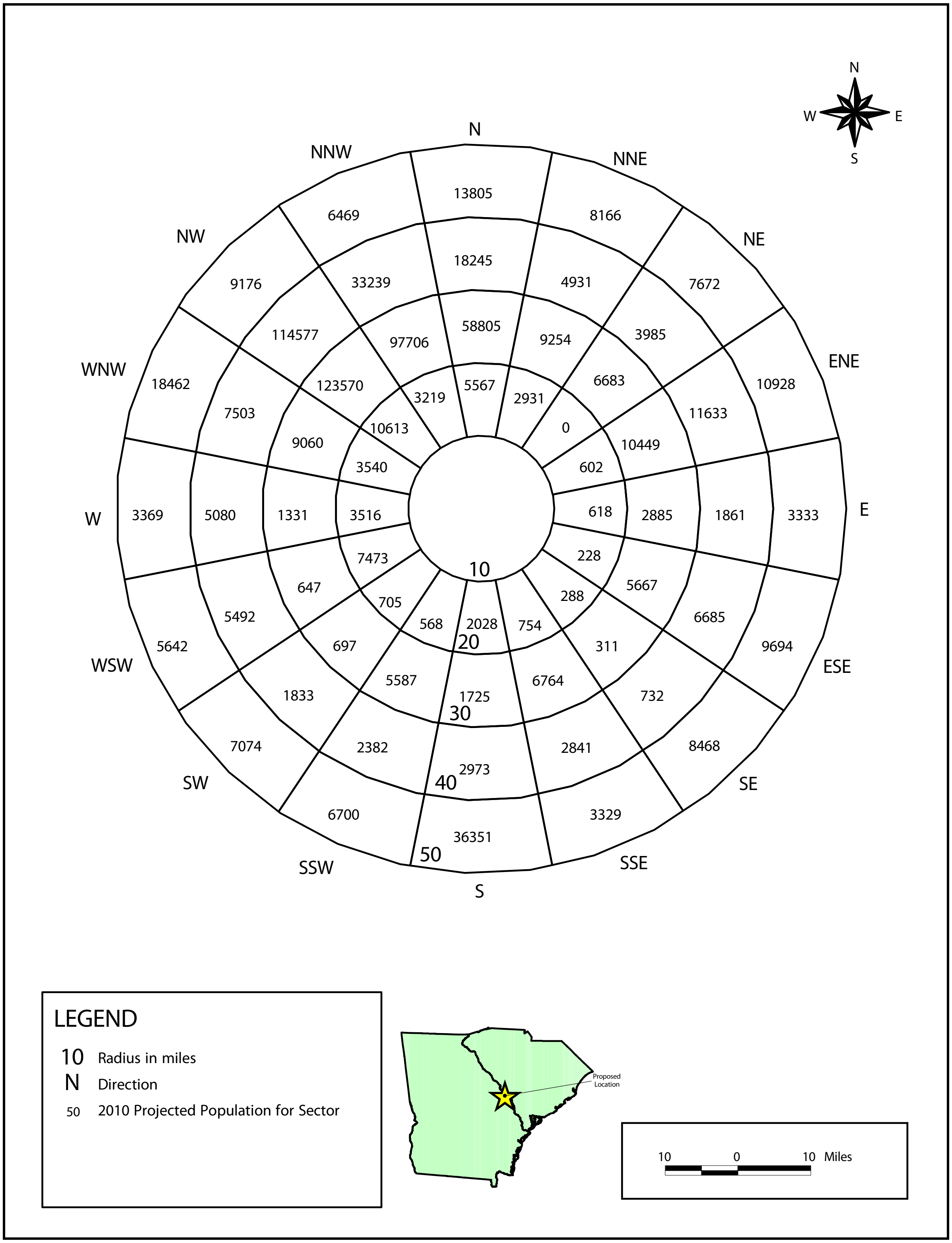


Figure 2.1-211
10 to 50-Mile Resident Population Distribution 2010

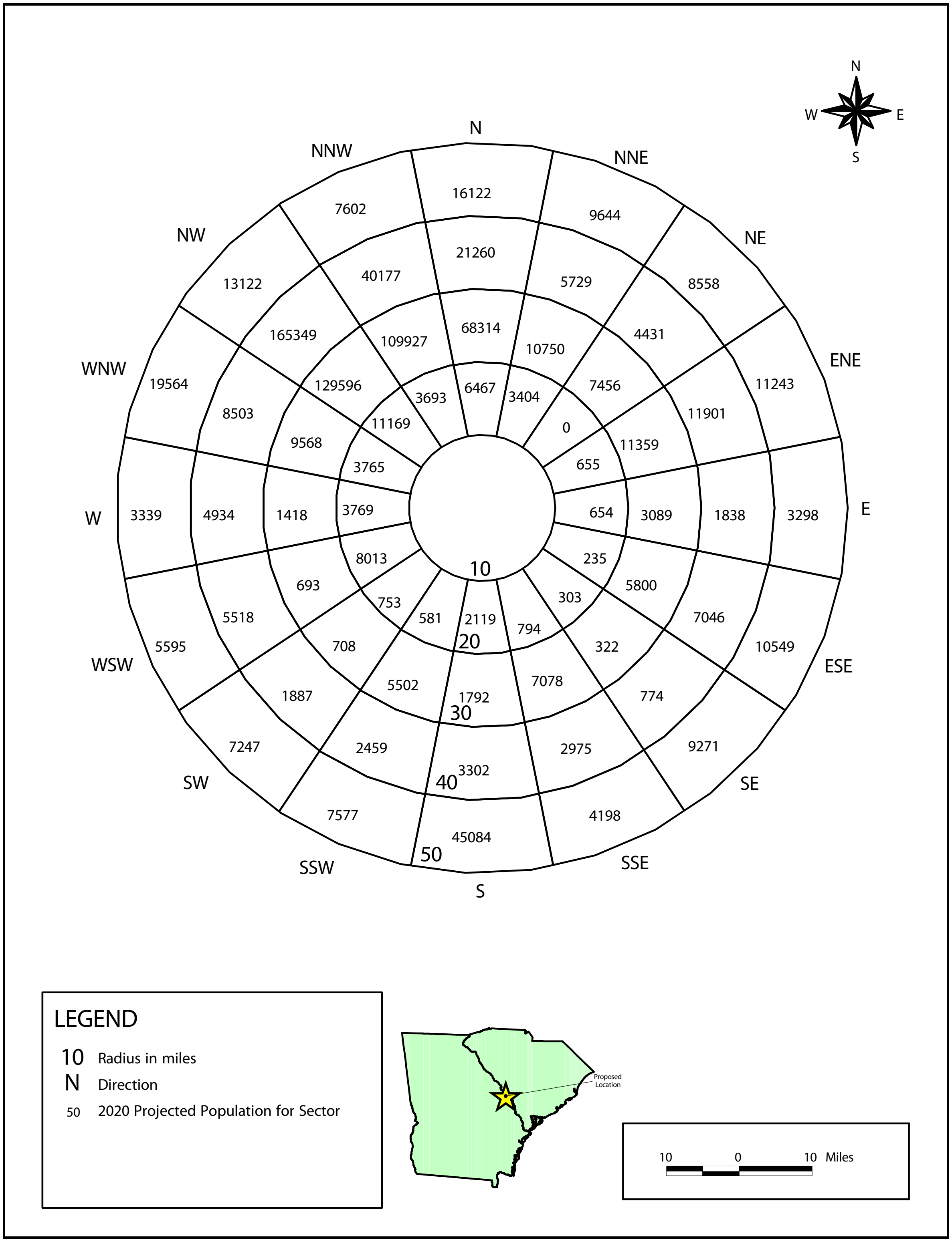


Figure 2.1-212
10 to 50-Mile Resident Population Distribution 2020

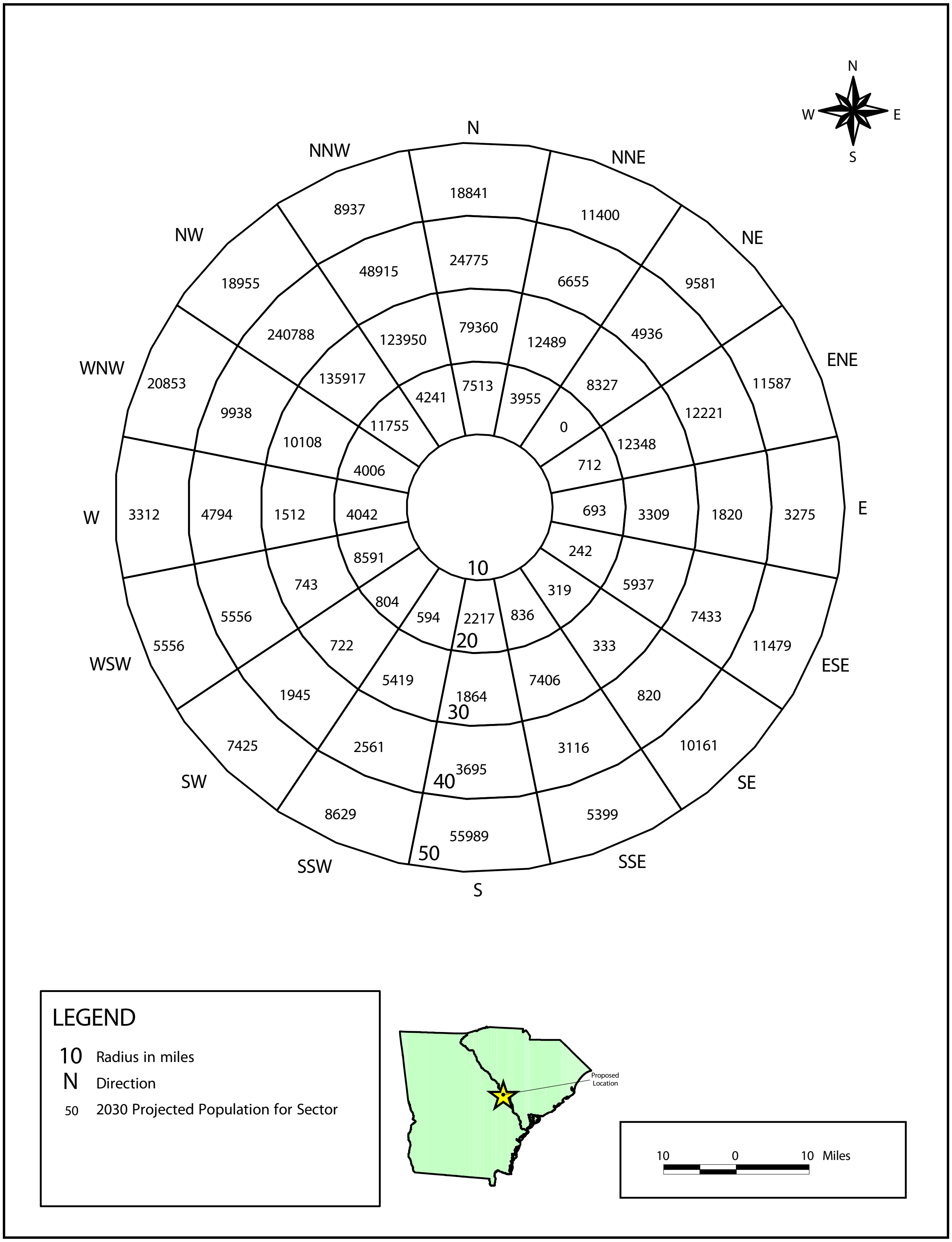


Figure 2.1-213
10 to 50-Mile Resident Population Distribution 2030

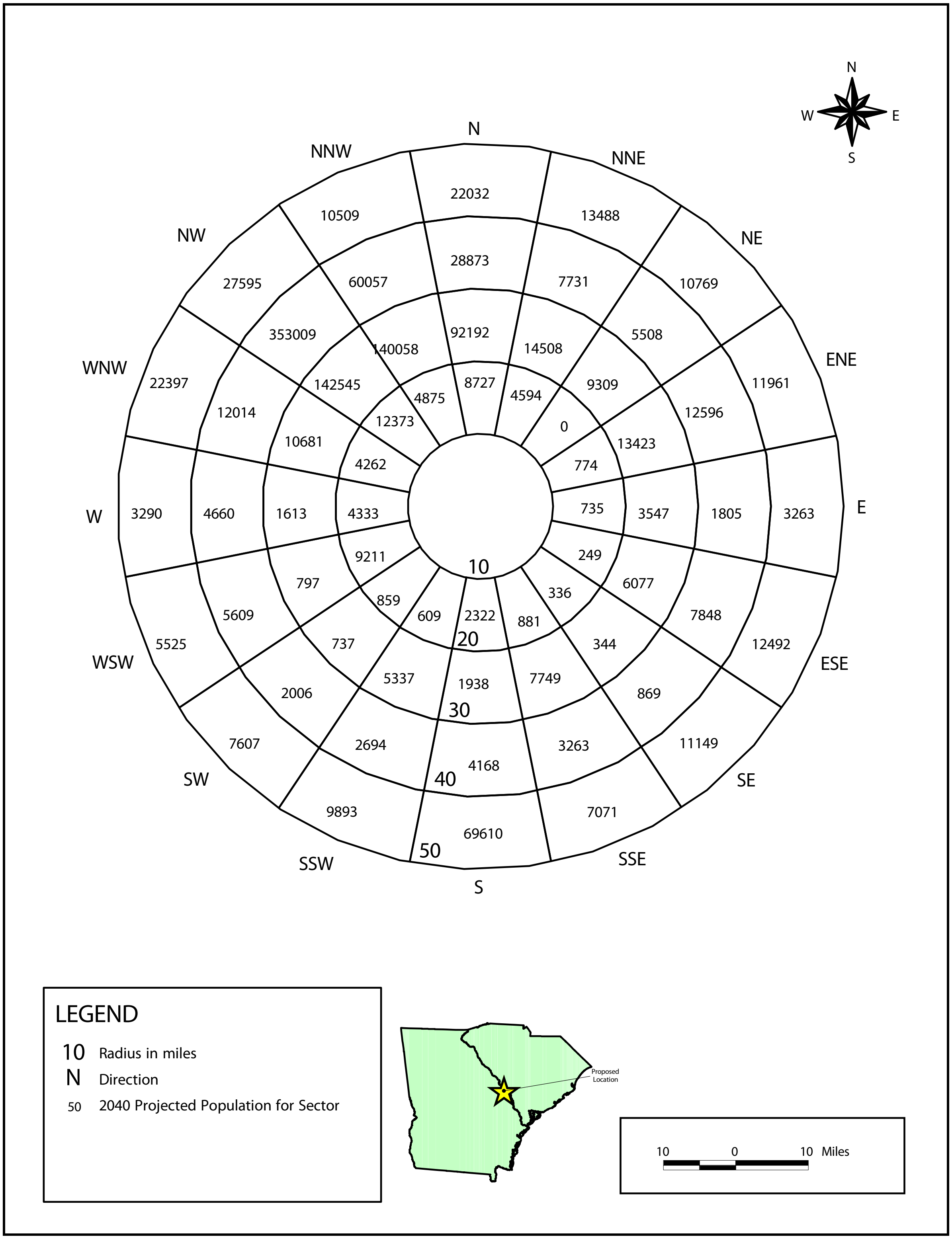


Figure 2.1-214
10 to 50-Mile Resident Population Distribution 2040

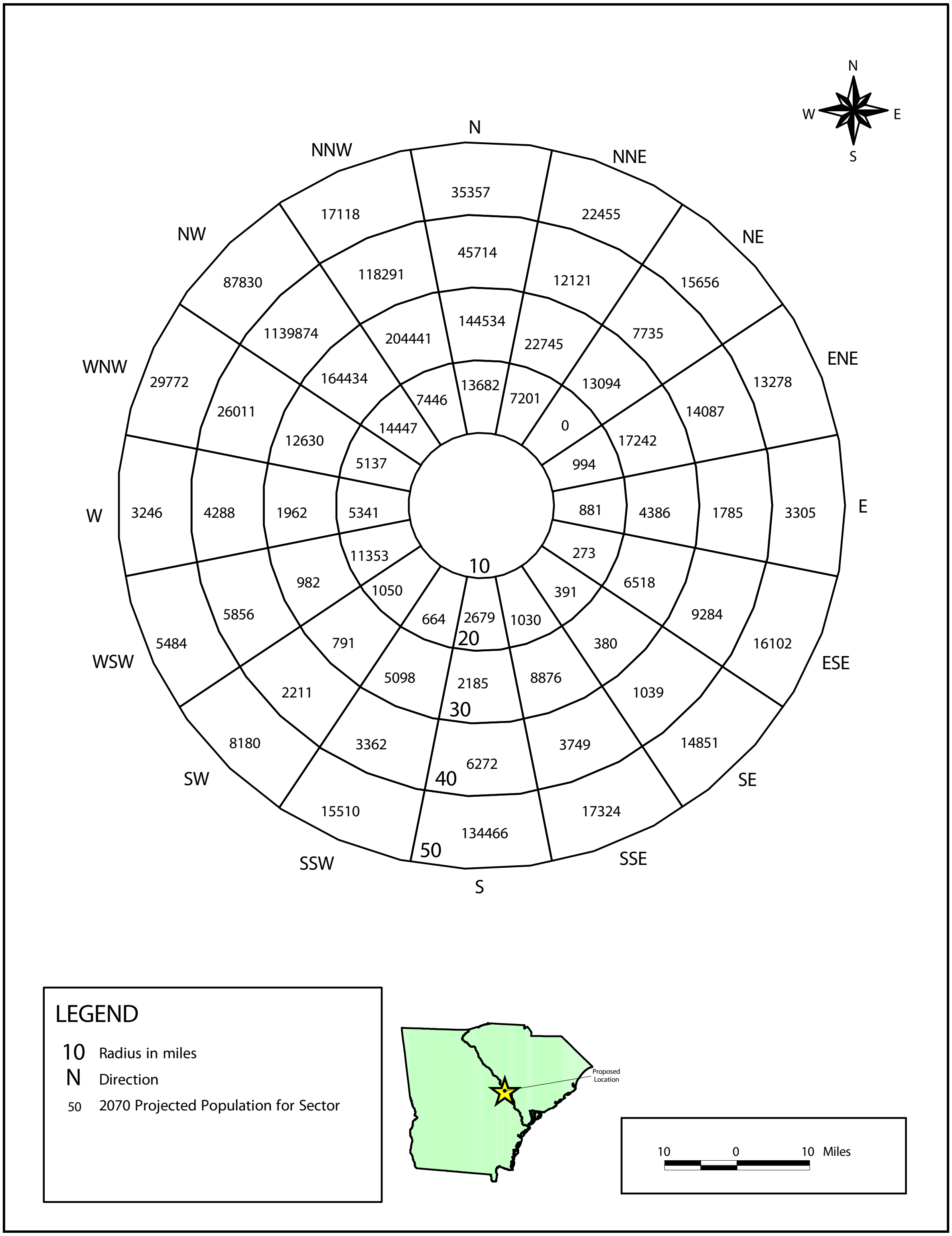


Figure 2.1-215
10 to 50-Mile Resident Population Distribution 2070

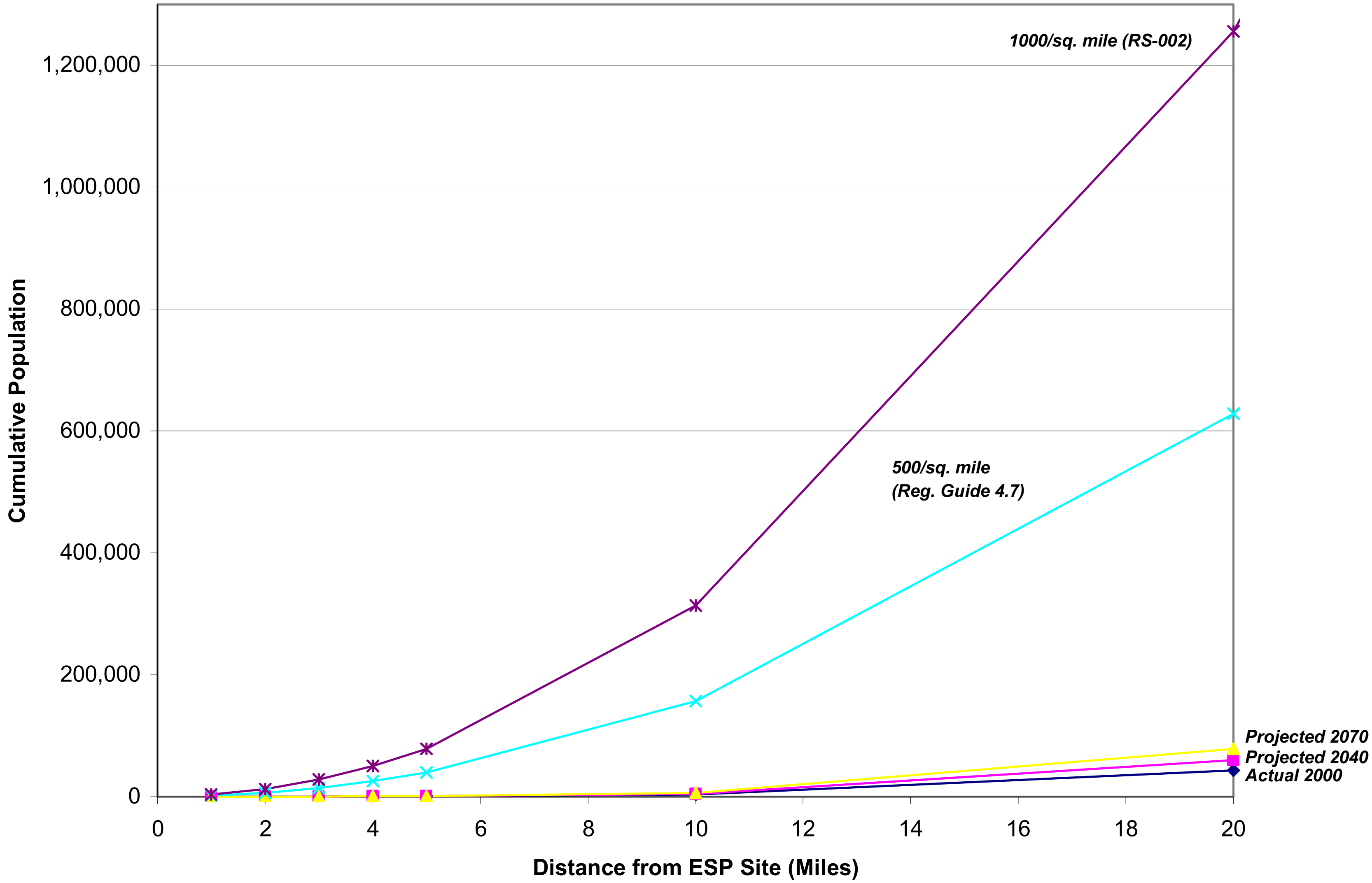
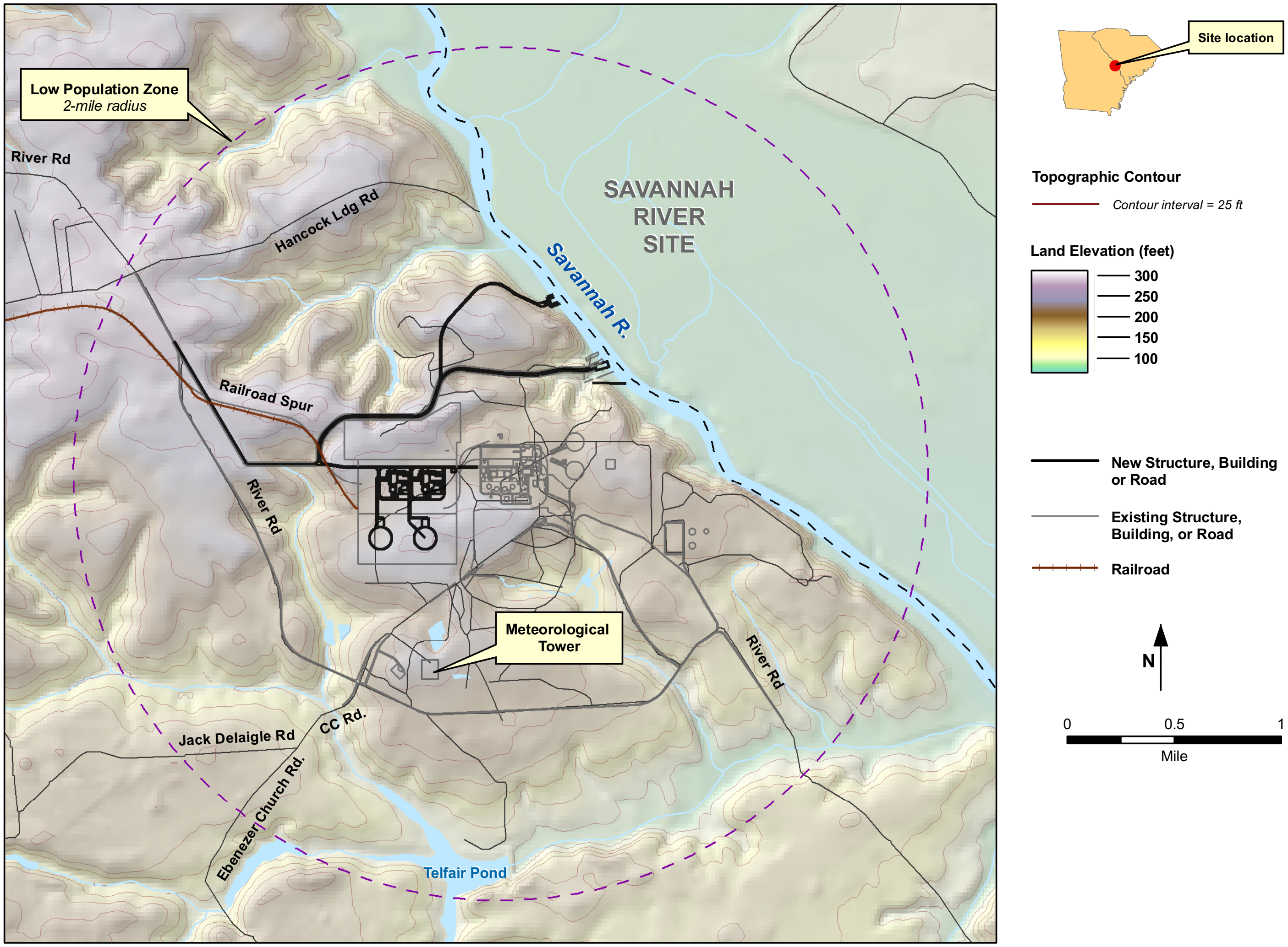


Figure 2.1-216
Population Compared to NRC Siting Criteria



Source data: ESRI, Data & Maps and StreetMap USA, 2005. USGS, seamless data distribution, NED shaded relief.
Vogtle Project Drawing No. 0-C2-00 10-00001.

Figure 2.1-217
Low Population Zone

2.2 Identification of Potential Hazards in Site Vicinity

2.2.1 Location of Nearby Industrial, Transportation, and Military Facilities

Within a 5-mile vicinity of the VEGP site, there are several major industrial facilities, one railroad, and one highway with commercial traffic. Specifically, the following transportation routes and facilities are shown on the indicated figures:

- Plant Wilson (see Figure 2.2-201)
- Savannah River Site (see Figure 2.2-202)
- Georgia State Highway 23 (see Figure 2.2-203)
- CSX Railroad (see Figure 2.2-201)
- A coal-fired steam plant operated by Washington Savannah River Company in D-Area of the SRS
- VEGP Unit 1 and Unit 2

Figures 2.2-202 and 2.2-203 show the location of major industrial facilities, military bases, highway transportation routes, airports, railroads, and pipelines within a 25-mile radius of the site. In addition, Figure 2.2-202 shows nearby airways and military operation areas.

Items illustrated on the maps are described in Subsection 2.2.3. The only military facility within a 50-mile radius is Fort Gordon. The Fort Gordon U.S. Army Signal Corps training facility is barely within 25 miles of the VEGP site. The only major storage facility within 25 miles of the VEGP site, other than those at the SRS and at Chem-Nuclear Systems, is a group of oil storage tanks associated with the existing combustion turbine generators for Plant Wilson on the VEGP site.

2.2.2 Descriptions

2.2.2.1 Industrial Facilities

The Burke County Comprehensive Plan: 2010, Part 1 (Reference 207) shows a relatively slow, stable population growth pattern for the county. This is indicative that the nearby industries have not experienced much growth.

The Comprehensive Plan also reveals that services and manufacturing industries dominate the top 10 employers in the county. Southern Nuclear and Samson Manufacturing Company (curtains and draperies) are the largest Burke County employers. Nearby industries also include the Chem-Nuclear Systems radioactive waste disposal site (18 miles away in South Carolina) operated by Duratek; Unitech Services Group nuclear laundry facility (21 miles away in South Carolina); and the facilities of the SRS (also in South Carolina). Table 2.2-201 lists the largest employers for the three-county region, based on recent data obtained for Burke County (Reference 208) in Georgia, and nearby Aiken and Barnwell counties in South Carolina (Reference 203; Reference 206).

There currently are no projected major increases to industrial, military, or transportation facilities within a 25-mile radius of the VEGP site except for the development of the site for VEGP Units 3 and 4.

2.2.2.1.1 Savannah River Site

The SRS borders the Savannah River for approximately 17 miles opposite the VEGP site. It occupies an approximately circular area of 310 square miles (198,344 acres) encompassing parts of Aiken, Barnwell, and Allendale counties in South Carolina (Reference 230). The SRS is owned by the DOE and operated by an integrated team led by Washington Savannah River Company (WSRC). The site is a closed government reservation except for through traffic on South Carolina Highway 125 (Savannah River Site Road A) and the CSX Railroad.

The SRS processes and stores nuclear materials in support of the national defense and U. S. non-proliferation efforts. The site also develops and deploys technologies to improve the environment and treat nuclear and hazardous wastes left from the Cold War. (Reference 230)

The following is a list of current and near-term operating facilities at the SRS and the activities conducted at these facilities (Reference 230; Reference 213):

- Separations facilities for processing irradiated materials (H Area).
- Waste management facilities that process, dispose or ship solid radioactive waste, hazardous waste, mixed waste, transuranic waste, and sanitary waste (E Area).
- The Defense Waste Processing Facility is processing high-level radioactive waste into stable borosilicate glass for disposal (S Area).
- The Savannah River National Laboratory (a process development laboratory to support production operations and containing two test reactors) and administrative facilities (A Area).
- The L Area Disassembly Basin which provides receipt and interim storage of research reactor fuel (L Area).
- Tritium Extraction Facility to extract tritium from fuel rods irradiated at TVA's reactors and to load the extracted tritium into canisters for shipment to the Department of Defense. Expected to begin operation in fiscal year 2007.
- Replenishment of tritium – recycling, purifying, and reloading nuclear weapons reservoirs.
- MOX Fuel Fabrication Facility (to be constructed) to manage and convert excess weapons-grade plutonium to a form that can be used in commercial nuclear power plants.
- Stabilization, management, and storage of plutonium materials (K Area).
- Salt Waste Processing Facility to remove radioactive constituents from high-level waste (under construction).
- A variety of non-nuclear facilities necessary for plant operations.

Five nuclear production reactors and several small test reactors are deactivated and are awaiting decommissioning and decontamination.

The major waste storage areas for high-level waste are adjacent to the two separations areas and consist of two tank farms linked to the separations areas and to each other by pipelines with secondary containment. In addition, the SRS uses engineered concrete vaults and engineered trenches for the permanent disposal of solid low-level radioactive waste (Reference 230). The deactivated reactors, separations areas, and waste storage areas are at least 4 miles from the nearest VEGP site boundary.

2.2.2.1.2 Unitech Services Nuclear Laundry Facility

Although not located within 5 miles of the VEGP site, the Unitech Services Nuclear Laundry Facility, located in the Barnwell County Industrial Park, is described due to its relative proximity to and association with the SRS (Figure 2.2-203). It was constructed by Unitech Service Group to provide radiological laundry, decontamination and respirator services. The facility has about 50 employees as of May 2006 (Reference 225).

2.2.2.1.3 Chem-Nuclear Systems

Chem-Nuclear Systems developed, constructed, and operates the largest radioactive waste disposal site in the country near Barnwell, South Carolina (Figure 2.2-203). This site contains 308 acres, of which 235 have been deeded to the State of South Carolina as a designated exclusion area. Waste receipts are in the form of solids only; no liquids are accepted. Since the disposal facility began operation in 1971, about 28 million cubic feet, or 90 percent of the available disposal volume, have been used (Reference 210). The facility handles approximately 400 shipments of low-level spent fuel per year. The products and materials associated with Chem-Nuclear Systems are described in Table 2.2-202 (Reference 224).

2.2.2.1.4 Georgia Power Company's Plant Wilson

Plant Wilson is located approximately 6,000 feet east-southeast from the proposed VEGP Units 3 & 4 footprint. The existing combustion turbine plant is an electrical peaking power station of Georgia Power Company. The plant consists of six combustion turbines with a total rated capacity of 351.6 MW. The storage capacity of the fuel storage tanks is 9,000,000 gallons.

2.2.2.1.5 VEGP Units 1 and 2

The existing VEGP Units 1 and 2 reactors are located about 3,600 ft and 3,900 ft, respectively, west of the Savannah River. For these units, the exclusion area is the same as that for the proposed units and it is defined as an irregular shaped area which generally conforms to the site's boundary lines. There are no residents within the exclusion area, and there are no highways, railways, or waterways crossing the area. Besides the activities at Plant Wilson, the only other activities that may occur within the exclusion area that are unrelated to plant operations are those associated with the operation of the Visitor's Center. VEGP has made arrangements to control and, if necessary, evacuate the exclusion area in the event of an emergency.

2.2.2.2 Mining Activities

There are no mining activities within 5 miles of the VEGP site.

2.2.2.3 Roads

The nearest highway with commercial traffic is Georgia State Highway 23 (Figure 2.2-203). Segments of Georgia State Highways 23, 80, and 56 Spur are located within a 5-mile radius of the site. Other than traffic volumes, the Georgia Department of Transportation does not maintain data on the products and materials carried over these roads. However, major commercial traffic occurs only on State Highway 23, which serves as a major link between Augusta and Savannah. The heaviest truck traffic along State Highway 23 near the site consists primarily of timber and wood products and materials. State Highways 80 and 56 Spur serve primarily as minor transportation routes for local traffic. Available statistical data on personal injury accidents on these roads between 1999 and 2003 are presented in Table 2.2-203 (Reference 218).

2.2.2.4 Railroads

CSX is the nearest railroad with commercial traffic and is approximately 4.5 miles northeast of the VEGP site. CSX runs through and services the SRS. Major chemical substances identified as being carried by the CSX Railroad include cyclohexane, anhydrous ammonia, carbon monoxide, and elevated temperature material liquids (ETML). (Reference 220)

Burke County has two local Norfolk Southern rail lines, one through Waynesboro and one through Midville. These are approximately 12 miles west of the VEGP site.

2.2.2.5 Waterways

The Savannah River above the VEGP site (River Mile 151) is primarily used for recreational purposes since 1979, with the closing of the New Savannah Bluff Lock and Dam (River Mile 187) to commercial traffic (Reference 226). No commercial facilities or barge slips/docks are visible on satellite imagery between the VEGP site and the New Savannah Bluff Lock and Dam. This section of the river is primarily forested and otherwise undeveloped land to the river's edge.

Downstream of the VEGP site, barge traffic may be present closer to the Port of Savannah (River Mile 21). In 2005, no barge traffic was reported to the Army Corp of Engineers Waterborne Commerce Statistics Center in New Orleans, Louisiana (Reference 227). In 2004, only 13 commercial vessels were recorded (Reference 219). These vessels were reported to contain a total of less than 500 tons of non-explosive residual fuel oil (less than a full barge load).

Therefore, the current use of the river and the lack of commercial facilities and barge slips/docks upstream of the plant indicate that there is no current or projected barge traffic on the Savannah River past the VEGP site. Based on the above information, SNC has determined that evaluation of hazardous shipments by barge is not necessary for VEGP Units 3 and 4.

2.2.2.6 Airports, Airways, and Military Training Routes

2.2.2.6.1 Airports

There are no airports within 10 miles of the VEGP site. The closest airport, Burke County Airport, is approximately 16 miles west-southwest of the VEGP site. It has a 4,035-foot asphalt runway oriented 250° WSW – 70° ENE. The airport, which has a non-directional radio beacon for runway approach, is used by single-engine private aircraft and by crop-dusting operations. There are only two multi-engine and five single-engine aircraft based at the field. The average number of operations (landings and takeoffs are counted separately) is about 57 per week. Most operations are transient general aviation; only about 33 percent are local general aviation (Reference 209).

The closest commercial airport is Augusta Regional Airport at Bush Field, which is located approximately 17 miles north-northwest of the VEGP site. It has an 8,000-foot primary runway oriented 170° SSE – 350° NNW and a 6,000-foot crosswind runway oriented 80° ENE – 260° WSW. FAA information effective April 13, 2006, indicates that 17 aircraft are based on the field. Ten of these are single-engine airplanes, four are multi-engines airplanes, and three are jet-engine airplanes. The average number of operations is about 91 per day. Most (40 percent) are general transient aviation, 24 percent are air taxi, 12 percent are local general aviation, 14 percent are commercial, and 10 percent are military (Reference 205). Based on the historical flight data recorded prior to 2005, projections for air traffic at Bush Field up to fiscal year 2025 are given in Table 2.2-204

(Reference 204). Approach and departure paths at Bush Field are not aligned with the VEGP site; and no regular air traffic patterns for Bush Field extend into airspace over the VEGP site.

A small un-improved grass airstrip is located immediately north of the VEGP site (north of Hancock Landing Road and west of the Savannah River). At its closest point, the airstrip is more than 1.4 mile from the power block of the new units. This privately owned and operated airstrip has a 1,650-foot turf runway oriented 80° East – 260° West. Thus take-offs and landings are tangential to the site property and oriented away from the plant. While no FAA traffic information is available for this airstrip, informal communication with the owner/operator revealed that the airstrip is for personal use and the associated traffic consists only of small single-engine aircraft (Reference 222). In addition, there is a small helicopter landing pad on the VEGP site. This facility exists for corporate use and for use in case of emergency. The traffic associated with either of these facilities may be characterized as sporadic. Therefore, due to the small amount and the nature of the traffic, these facilities do not present a safety hazard to the VEGP site.

2.2.2.6.2 Airways

The centerline of Airway V185 is approximately 1.5 miles west of the VEGP site (Figure 2.2-202). Additionally, Airway V417 is about 12 miles northeast of the VEGP site, and Airway V70 is approximately 20 miles south of the VEGP site (Figure 2.2-202) (Reference 217). Due to its close proximity to the VEGP site, an evaluation of hazards from air traffic along the V185 airway is presented in Subsection 3.5.1.6. That evaluation shows that the presence of Airway V185 is not a safety concern for the VEGP site.

2.2.2.6.3 Military Training Routes

In August 2005, Shaw Air Force Base (AFB), South Carolina, issued a draft Environmental Impact Statement (EIS) (Reference 223) regarding implementing airspace modifications to the Gamecock and Poinsett Military Operation Areas (MOAs) in South Carolina and the Bulldog MOAs in Georgia. The west edge of the Poinsett MOA is about 75 miles east-northeast of the VEGP site. The Gamecock MOAs are east of the Poinsett MOA. The proposed Gamecock E MOA would be created to form a “bridge,” allowing maneuvering and training between the Gamecock MOAs and the Poinsett MOA. The east edge of the Bulldog MOAs is about 11 miles west of the VEGP site (see Figure 2.2-202). Because of the relatively long distances between the VEGP site and these MOAs, and their related training routes, no aircraft accident analysis is required for flight activities associated with these MOAs and their related training routes.

Under the proposed action, the airspace structure at Bulldog A MOA would be expanded to the east under the Bulldog B “shelf” to match the boundary of the existing Bulldog B. Mainly, the current 500-foot msl floor as allowed at Bulldog A would be laterally expanded into Bulldog B. Because the current Bulldog B floor is 10,000 feet msl, this lateral expansion would increase the airspace volume in the Bulldog MOAs. The overall distance from the MOA boundary to the VEGP site is unchanged.

Military aircraft in the Bulldog MOAs are expected to come mainly from Shaw AFB (about 32 miles east of Columbia, South Carolina) and McEntire Air National Guard Station (about 13 miles east-southeast of Columbia). Among the military training routes, VR97-1059 is located closest to the VEGP site. The distance between the centerline of VR97-1059 and the VEGP site is about 18 miles (Figure 2.2-202). The maximum route width of VR97-1059 is 20 nautical miles (NM); therefore, the width on either side of the route centerline is assumed to be 10 NM (11.5 miles). The VEGP site is located more than 6 miles from the edge of this training route. Additionally, the total number of military aircraft using route VR97-1059 is approximately 833 per year (Reference 223).

According to RS-002, *Processing Applications for Early Site Permits*, May 2004 (RS-002), the aircraft accident probability for military training routes is considered to be less than 10^{-7} per year if the distance from the site is at least 5 statute miles from the edge of military training routes, including low-level training routes, except for those associated with a usage greater than 1,000 flights per year, or where activities may create an unusual stress situation.

In summary, the MOA use is projected to remain relatively unchanged and no modifications are proposed to the military routes. The VEGP site is located more than 5 statute miles from the edge of VR97-1059, and the total military flights using the same route is less than 1,000 per year; therefore, no aircraft accident analysis is required for flights using VR97-1059 (Reference 223).

2.2.2.7 Natural Gas or Petroleum Pipelines

Three pipelines are within 25 miles of the VEGP site (Figure 2.2-203); however, none are located within 10 miles of the VEGP site.

Pipeline 1, located approximately 21 miles northeast of the VEGP site, is an 8-inch-diameter line constructed in 1959. It operates at a maximum pressure of 750 psi; is buried 3 feet deep; has 8-inch Rockwell isolation valves at 25-mile intervals; and carries natural gas. It is not used for storage.

Pipeline 2, located approximately 19 miles southwest of the VEGP site, has a 14-inch-diameter line constructed in 1954 and a 20-inch-diameter line constructed in 1977. Both lines are buried 3-feet deep; operate at a maximum pressure of 1,250 psi; have buried Rockwell isolation valves every 8 to 9 miles; and carry natural gas. They are not used for storage.

Pipeline 3, located approximately 20 miles northwest of the VEGP site, has two 16-inch-diameter lines constructed in 1953 and 1957. Both operate at a maximum pressure of 1,250 psi; are buried 3 feet deep; have buried Rockwell isolation valves every 8 to 9 miles; and carry natural gas.

Because the pipelines identified are well over 10 miles from the VEGP site, there is no need to identify the locations of individual pipeline valves.

2.2.2.8 Military Facilities

There are no military facilities within 5 miles of the VEGP site.

2.2.2.9 VEGP Units 1 and 2 Storage Tanks/Chemicals

Chemicals currently stored at the VEGP site are presented in Table 2.2-205.

2.2.3 Evaluation of Potential Accidents

The plant has inherent capability to withstand certain types of external accidents due to the specified design conditions associated with earthquakes, wind loading, and radiation shielding. Acceptability for external accidents associated with VEGP Units 3 and 4 is covered in this subsection.

The determination of the probability of occurrence of potential accidents which could have severe consequences is based on analyses of available statistical data on the occurrence of the accident together with analyses of the effects of the accident on the plant's safety-related structures and components. If an accident is identified for which the probability of severe consequences is unacceptable, specific changes to the AP1000 are identified. The criteria for not requiring changes to the AP1000 design is that the total annual frequency of occurrence is less than 10^{-6} per year for an external accident leading to severe consequences. The following accident categories are considered in determining the frequency of occurrence, as appropriate:

Explosions – Accidents involving detonations of high explosives, munitions, chemicals, or liquid and gaseous fuels will be considered for facilities and activities in the vicinity of the plant where such materials are processed, stored, used, or transported in quantity.

The AP1000 includes onsite storage facilities for compressed and liquid hydrogen. Accidents involving accidental detonations of hydrogen from these storage facilities are evaluated as part of the AP1000 certified design. It is not required to provide analyses of accidents involving these storage facilities provided that the locations and size of the storage facilities are consistent with the safe distances defined by the AP1000 certified design. The bulk gas storage area for the plant gas system (PGS) is located sufficiently far from the nuclear island that an explosion would not result in damage to safety-related structures, systems, and components.

Evaluation of potential explosions due to exposure of chemical storage tanks to exterior fires has determined that all of these postulated accidents are safe distances away from safety-related items.

The AP1000 certified design does not include liquid oxygen or propane storage facilities. The three 2000 gallon capacity above-ground liquid propane tanks installed at the Vogtle Fire Training Facility (building 695) are addressed in Subsection 2.2.3.2.3.

Flammable Vapor Clouds (Delayed Ignition) – Accidental releases of flammable liquids or vapors that result in the formation of unconfined vapor clouds in the vicinity of the plant.

A flammable vapor cloud (delayed ignition) due to the accidental release of hydrogen from the PGS bulk gas storage area is evaluated as part of the AP1000 certified design. A detonation of such a hydrogen vapor cloud would not result in damage to safety-related structures, systems, and components. No other chemical has the possibility of developing unconfined flammable vapor clouds.

Toxic Chemicals – Accidents involving the release of toxic chemicals from nearby mobile and stationary sources.

Fires – Accidents leading to high heat fluxes or smoke, and to nonflammable gas or chemical-bearing clouds from the release of materials as the consequence of fires in the vicinity of the plant.

Airplane Crashes – Accidents involving aircraft crashes leading to missile impact or fire in the vicinity of the plant.

The safe distance for material in onsite storage facilities for explosions, flammable vapor clouds, and fires is tabulated in Table 2.2-1.

Analyses were performed in order to evaluate the impact on the VEGP Units 3 & 4 following potential accidents resulting in an explosion or flammable cloud or toxic chemical releases within a 5-mile radius of the VEGP site. The postulated accidents that would result in an explosion or chemical release were analyzed at the following locations:

- Nearby transportation routes (Savannah River, Highway 23, and CSX Railroad)
- Nearby chemical and fuel storage facilities (Savannah River Site, Plant Wilson)
- Onsite chemical storage tanks
- Other nearby fire sources

The existing analysis of potential hazards to the Units 1 and 2 was reviewed for applicability to the Units 3 and 4. That analysis evaluated postulated releases of flammable materials and toxic gases transported or stored at industrial facilities within a 5-mile radius of the VEGP site. In addition, new chemicals, which have been identified as being associated with Units 1 and 2, were subsequently evaluated or analyzed to determine their impact to Units 3 and 4. As described below, in each case, these analyses concluded that the potential for hazard is minimal and will not affect safe operation of Units 3 and 4.

2.2.3.1 Explosion and Flammable Vapor Clouds

The effects of explosion and formation of flammable vapor clouds from the nearby sources are evaluated below.

2.2.3.1.1 Truck Traffic

Segments of Georgia State Highways 23, 80, and 56 Spur are located within a 5-mile radius of the VEGP site. Major commercial traffic occurs only on State Highway 23, which serves as a major link between Augusta and Savannah, Georgia.

An analysis of truck-borne hazards that was performed for Units 1 and 2 identified that chlorine (1 ton), anhydrous ammonia (6 tons), liquid nitrogen (6,500 gallons), phosphoric acid (200 lb), nitric acid (5,000 gallons), and diesel oil (6,000 gallons) were transported on nearby Highway 23. At its nearest point, Highway 23 passes about 4.7 miles from the center point of the Units 1 and 2 control rooms. The allowable and actual distances of hazardous chemicals transported on highways were evaluated according to NRC Regulatory Guide 1.91, Revision 1, *Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants*. Regulatory Guide 1.91 cites 1 psi as a conservative value of peak positive incident overpressure, below which no significant damage would be expected. The analysis demonstrated that truck-borne substances transported within a 5-mile radius of the VEGP Units 1 and 2, as well as explosions and flammable vapor clouds induced by these chemicals, would not adversely affect safe operation of the units.

The six chemicals identified above in the analysis of truck traffic were obtained from the original design basis analysis for Units 1 and 2 and were based on a 1975 study performed by the Georgia Institute of Technology for Georgia Power Company. The original study is no longer available, and these chemicals have been re-evaluated as described below.

SNC has obtained the EPA Tier II reports for Burke and Richmond Counties in Georgia, identifying those facilities in the vicinity of the plant that have permits for storing hazardous materials (Reference 216). These reports, along with the EPA Landview 6 database, were used to confirm and/or update the list of chemicals for analysis. (Reference 215) The sites identified from these sources containing chemicals within a 20-mile radius of the VEGP site are depicted on Figure 2.2-204.

A traffic corridor evaluation has been performed to determine whether there are any new or additional chemicals transported by truck within 5 miles of the site related to the facilities described above. The evaluation shows that even fewer chemicals pass by the site now than assumed in the previous analysis performed for the existing units.

Only two EPA regulated sites exist that would likely use State Route 23 to transport materials and equipment. These sites are construction-related sites and are located 7 to 10 miles south of the VEGP site. Neither of these sites currently uses any of the previously identified chemicals, nor have they been identified to use or cause the transport of any hazardous chemicals other than fuel oil or gasoline. The remaining sites are all outside the 5-mile corridor and are likely to transport their materials and equipment via other, more direct, routes, rather than along State Route 23. These remaining sites, therefore, do not warrant further analysis.

The use of bulk anhydrous ammonia has been discontinued at the plant site. Since there are no other users of this chemical in the vicinity of this site, the issue of transportation of this chemical along the roadways or to the site does not require further analysis. (Anhydrous ammonia is still being transported by rail car, and is evaluated in Subsection 2.2.3.1.4).

SNC's re-evaluation concluded that the only remaining hazardous chemicals transported by truck in the vicinity of the site are gasoline and diesel/fuel oil.

For an 8,500 gallon truck on State Road 23 at the closest approach distance of approximately 4.2 miles (22,000 ft), the following calculations were performed in accordance with Regulatory Guide 1.91:

- TNT equivalent safe distance for an explosion of a gasoline vapor cloud
- TNT equivalent safe distance for an explosion of gasoline vapor in a truck

The gasoline truck analysis for the vapor cloud explosion used the industry standard program DEGADIS to calculate the distance from the site of the spill to the boundaries of the upper and lower flammability limits and to obtain the flammable mass within the vapor plume. The concentrations were compared to the lower flammability limits for the respective chemical to determine the maximum distance for the flammable vapor cloud. The input parameters were:

- Quantity of gasoline in the truck = 50,000 lb (56,165 lb TNT equivalent)
- Physical property data:
  - Molecular weight = 95 g/mole
  - Diffusion coefficient = 0.05 cm^2/sec
  - Vapor pressure = 305 mm Hg
  - Boiling point temperature = 130°C
  - Specific gravity = 0.732
- The meteorological conditions assumed were:
  - Stability class = F (stable)
  - Wind speeds = 1 m/s up to 2.5 m/s

For an explosion from an 8,500 gallon truck, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 1,723 feet.

For an explosion from a flammable vapor cloud, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 1,279 feet. The outer edge of the lower flammability limit (LFL) of the flammable portion of the gasoline vapor cloud is 1200 ft downwind from the road. If the blast occurs at the outer edge of the vapor cloud, which is a conservative assumption, then the maximum distance for which a peak incident of 1 psi would occur is the sum of the two distances, or 2,479 ft from the road.

The distance between State Road 23 and Units 3 and 4 is approximately 4.2 miles. This distance is far greater than either of the above calculated critical distances. Therefore, there will not be any impact on Unit 3 or 4 from an explosion of gasoline from a truck or vapor cloud.

The size of gasoline delivery trucks on State Road 23 ranges from 4,000 to 8,500 gallons, so the assumption of an 8,500-gallon truck in the analysis is conservative and bounding.

In addition to road transit, gasoline is delivered to the site by a tank wagon (10-wheel truck) containing a maximum volume of 4,000 gallons. The closest distance from the site delivery route to the power block circle is approximately 2,000 feet.

For an explosion from a 4,000 gallon truck, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 1,340 feet.

For an explosion from a flammable vapor cloud, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 920 feet. The outer edge of the lower flammability limit (LFL) of the flammable portion of the gasoline vapor cloud is 738 ft downwind from the road. If the blast occurs at the outer edge of the vapor cloud, which is a conservative assumption, then the maximum distance for which a peak incident of 1 psi would occur is the sum of the two distances, or 1,658 ft from the road.

As discussed above, since the closest distance from the site delivery route to the power block circle is approximately 2,000 feet, and the 1 psi blast pressure distances for the truck explosion and the vapor cloud explosion are 1,340 ft and 1,658 ft from the road, respectively, there will not be any impact on Unit 3 or 4 from an accident involving the 4,000 gallon gasoline tank wagon.

Since transported diesel/fuel oil is not flammable and is much less volatile than gasoline, the gasoline truck analysis becomes bounding in the evaluation of truck-borne hazards.

The quantity of chemical (diesel and gasoline), distance to Units 3 and 4, the TNT equivalent safe distance (beyond which the blast pressure would be less than 1 psi), the distance from the point of the spill to the point where the vapor concentration is equal to the lower flammability limit, and the lower flammability limit concentrations are shown below:

Chemical	Quantity	Distance to Units 3 and 4	TNT Equivalent Distance	Distance to Lower Flammability Limit	LFL
#2 Diesel	6,000 gal.	~4.2 mi (22,693 ft)	Not applicable	Not applicable	13,000 ppm
#2 Diesel	4,000 gal.	2,000 ft	Not applicable	Not applicable	13,000 ppm
Gasoline	50,000 lb 8,500 gal.	~4.2 mi (22,693 ft)	1,723 ft	1,200 ft	14,000 ppm
Gasoline	23,530 lb 4,000 gal.	2,000 ft	1,340 ft	738 ft	14,000 ppm

2.2.3.1.2 Pipelines and Mining Facilities

No natural gas pipeline or mining facilities are located within 10 miles of the VEGP site. No pipelines carrying potentially hazardous materials are located within 5 miles of the VEGP site. Therefore, the potential for hazards from these sources are minimal and will not adversely affect safe operation of the plant.

2.2.3.1.3 Waterway Traffic

As discussed in Subsection 2.2.2.5, there is no barge traffic past the VEGP site. Therefore, there are no chemicals transported by barge that require evaluation.

2.2.3.1.4 Railroad Traffic

The only railroad within a 5-mile radius of the VEGP site is the CSX Railroad (approximately 4.5 miles northeast of the center point between Units 1 and 2), which runs through, and services, the SRS. A hazards analysis performed for VEGP Units 1 and 2 showed that explosions and flammable vapor clouds induced by chemicals carried by this rail line will not adversely affect safe operation of Units 1 and 2. The critical distance (given by $kW^{1/3}$ in Regulatory Guide 1.91) that could cause overpressures of 1 psi to safety-related structures is approximately 2,291 feet. This scenario is caused by the explosion of a 26-ton ammonia railroad tank car (assumed to contain 132,000 pounds TNT equivalent). Because of the relatively long distance (approximately 4.5 miles) between the railroad and the VEGP site, if an explosion occurred due to an accident involving an ammonia railroad tank car, it would occur at a distance great enough not to pose an overpressure hazard to the safety-related structures. Since the proposed VEGP Units 3 and 4 will be located farther away from the railroad line than Units 1 and 2, the possibility of adverse effects from explosions and flammable vapor clouds is even smaller for the new units.

More recent information obtained from CSX (Director of Infrastructure Security) (Reference 220) indicates that the top four substances carried by CSX during 2005, which qualified as DOT hazardous chemicals, are cyclohexane (64%), anhydrous ammonia (9%), carbon monoxide (3%), elevated temperature material liquids (ETMLs) (3%).

Evaluations were made for each of the above chemicals. Some of these chemicals were already analyzed in a previous analysis for effect on Units 1 and 2, and some were evaluated specifically for their potential effect on Units 3 and 4. In each case, the evaluations concluded that the potential hazard from the chemicals is minimal and will not affect the safe operation of the new units.

Accidental spills of carbon monoxide or ETMLs are not expected to create an explosion or vapor hazard for the site. Carbon monoxide, which can cause asphyxiation, will quickly vaporize and dissipate before coming close to the VEGP plant limits. ETMLs, also referred to as elevated temperature goods, are not necessarily flammable. ETMLs are DOT Class 9 materials, and the main hazard they present is the potential to cause contact burns due to the elevated temperature of the substance. Because of the long distance separation between the CSX Railroad and the new units, no direct contact with these substances is expected. Therefore, no adverse impact is expected from the accidental releases of the ETML substances.

Cyclohexane (used in the manufacture of nylon, paint, resin, etc.) is a hazardous chemical that was not previously considered in the Unit 1 and 2 analyses, so a new analysis has been performed for Units 3 and 4.

For a 67-ton rail car at the closest approach distance of approximately 4.5 miles (23,760 ft), the following calculations were performed in accordance with Regulatory Guide 1.91:

- TNT equivalent safe distance for an explosion of cyclohexane vapor in a rail tank car
- TNT equivalent safe distance for an explosion of a cyclohexane vapor cloud

The cyclohexane rail car analysis for the vapor cloud explosion used the industry standard program DEGADIS to calculate the distance from the site of the spill to the boundaries of the upper and lower flammability limits and to obtain the flammable mass within the vapor plume. The concentrations were compared to the lower flammability limits for the respective chemical to determine the maximum distance for the flammable vapor cloud. The input parameters were:

- Quantity of cyclohexane vapor in the rail car = 48.8 lb (117.5 lb TNT equivalent)
- Physical property data:
  - Molecular weight = 84.16 g/mole
  - Diffusion coefficient = 0.076 cm^2/sec
  - Molecular volume = 133.2
  - Boiling point temperature = 80.7°C
  - Specific gravity = 0.779
- The meteorological conditions assumed were:
  - Stability class = F (stable)
  - Wind speeds = 1 m/s up to 2.5 m/s

For the explosion from a rail car, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 220 feet.

For an explosion from a flammable vapor cloud, the TNT equivalent safe distance beyond which the blast pressure would be less than 1 psi was calculated to be 451 feet. The outer edge of the lower flammability limit (LFL) of the flammable portion of the cyclohexane vapor cloud is 575 ft downwind from the railroad line. If the blast occurs at the outer edge of the vapor cloud, which is a conservative assumption, then the maximum distance for which a peak incident of 1 psi would occur is the sum of the two distances, or 1,026 ft from the rail car.

The distance between the closest point of the rail line and Units 3 and 4 is approximately 4.5 miles. This distance is far greater than either of the above calculated critical distances. Therefore, there will not be any impact on Unit 3 or 4 from an explosion of cyclohexane from a rail car or vapor cloud.

2.2.3.2 Hazardous Chemicals

Regulatory Guide 1.78 requires evaluation of control room habitability for a postulated release of chemicals stored within 5 miles of the control room. As described in Subsection 2.2.2, no manufacturing plants, chemical plants, storage facilities, or oil or gas pipelines are located within 5 miles of the VEGP site. Therefore, three scenarios were evaluated:

1. Potential hazards from chemicals transported on routes within a 5-mile radius of the site, at a frequency of 10 or more per year, and with weights outlined in Regulatory Guide 1.78
2. Potential hazards from major depots or storage areas
3. Potential hazards from onsite storage tanks

Each hazard is discussed and evaluated below. The VEGP Units 1 and 2 analysis was reviewed for applicability to Units 3 and 4 for the effects from each of these hazards. The review determined that the impact to the new units for each of these postulated events is bounded by the impact to Units 1 and 2.

2.2.3.2.1 Release of Hazardous Chemicals Due to a Transportation Accident

As previously discussed, three routes (Georgia State Highways 23, 80, and 56) pass within 5 miles of the VEGP site. Of these three routes, major commercial traffic occurs only on State Highway 23, which serves as a major link between Augusta and Savannah. In addition, rail traffic exists within the 5-mile radius of the plant.

As discussed in Subsection 2.2.2.5, there is no barge traffic past the VEGP site. Therefore, there are no chemicals transported by barge that require evaluation.

The hazardous chemical sources due to a transportation accident were analyzed. The results of the analysis indicated that control rooms of VEGP Units 3 and 4 would remain habitable for all transported chemicals as discussed below.

In the analysis for truck traffic, methods specified in NUREG-0570 were used to estimate vapor emission rates and their dispersion. As discussed in Subsection 2.2.3.1.1, the only hazardous chemicals transported by truck in the vicinity of the VEGP site are gasoline and diesel/fuel oil.

The table below shows, for each chemical transported by truck, the key input parameters and the results of the evaluation using the methodology of NUREG-0570.

Chemical	Quantity	Distance to Control Room	Wind Speed	Stability	Control Room Concentration	Toxicity Limit
#2 Diesel	6,000 gal.	~4.2 mi (22,693 ft)	0.5 m/s	G	0.057 ppm	300 ppm
#2 Diesel	4,000 gal.	2,000 ft	1 m/s	F	Bounded by gasoline	300 ppm
Gasoline	50,000 lb 8,500 gal.	~4.2 mi (22,693 ft)	1 m/s	F	34.9 ppm	300 ppm
Gasoline	23,530 lb 4,000 gal.	2,000 ft	1 m/s	F	95.1 ppm	300 ppm

Therefore, no adverse impact to VEGP Units 3 and 4 is expected from the accidental release of gasoline or diesel/fuel oil.

For a postulated accident on a rail line, cyclohexane and ammonia were evaluated. The potential adverse impact caused by accidental release of cyclohexane was analyzed for the ESP because it was not previously evaluated, it is flammable, and it has an established toxic threshold limit value (TLV). Using approaches specified in NUREG-0570, the analysis has concluded that the accidental release of cyclohexane from a railcar will not have adverse effects to the control room operators. The meteorological conditions used in the ESP analysis were based on guidance provided in Regulatory Guide 1.78. Regulatory Guide 1.78 describes a simplified procedure for calculating weights of hazardous chemicals for control room evaluations. In that simplified procedure, stable atmospheric stability (F stability) is used because it represents the worst 5% meteorology observed at the majority of nuclear plant sites per Regulatory Guide 1.78. Therefore, in the ESP analysis, stable atmospheric meteorological conditions (F stability with a wind speed of 1 m/s) were assumed.

The assumed railcar capacity (67 tons) is similar to that described in Regulatory Guide 1.91. With a control room air intake height about 60 ft above grade, the control room outside concentration was estimated to be 0.12 g/m^3 (34.3 ppm). The immediate danger to life and health (IDLH) value of cyclohexane is 1,300 ppm (Reference 211). Since the control room outside concentration was estimated to be only 34.3 ppm, the accidental release of the cyclohexane tank car will not cause adverse effects to the control room operators.

The evaluation of ammonia was originally performed for Units 1 and 2, and it has been extended to Units 3 and 4. Assuming the release from a rail car containing 26 tons of anhydrous ammonia, the evaluation showed that the Units 1 and 2 control room concentration at 2 minutes after odor detection is 112 ppm, without taking credit for control room isolation. This concentration is much lower than the IDLH value of 300 ppm. In accordance with Regulatory Guide 1.78, the evaluation assumed 2 minutes is sufficient time for a trained operator to put a self-contained breathing apparatus into operation, if they are to be used.

For ammonia and cyclohexane, the factors for estimating the concentration of each chemical at the control room air intake are:

Compound	Quantity	Distance from Railroad to Control Room	Wind Speed	Stability Class	Concentration of Compound at Control Room Air Intake, ppm	IDLH Toxicity Limit, ppm
Ammonia	26 tons	4.5 miles	1 m/s	G	112 @ 2 min	300
Cyclohexane	67 tons	4.5 miles	1 m/s	F	34.3	1,300

In addition the AP1000 design provides manual actuation to initiate the emergency habitability system. Protective measures (including manual actuation of the main control room habitability system) required to be taken by the control room operators are evaluated in accordance with Subsection 6.4.7.

Therefore, no adverse impact to VEGP Units 3 and 4 is expected from the accidental release of ammonia or cyclohexane.

2.2.3.2.2 Potential Hazard from Major Depots or Storage Areas

There are no major depots within 5 miles of the VEGP site. The only chemical storage areas within 5 miles of the VEGP site exist at the SRS and the Wilson combustion turbine plant.

The original analysis (performed for Units 1 and 2) had determined that SRS had the potential to use chlorine and ammonia at the D-Area, which is approximately 4.5 miles distant from Units 1 and 2. However, the 2004 Tier II EPA report for this site (Reference 216), and recent communications with SRS management, have indicated that ammonia and chlorine are no longer in use at D-Area (Reference 228), (Reference 214). The area has been remediated, and nearly all of the facilities have been removed. The only chemicals used at the site, according to the recent Tier II report, are chlorine softener chemicals and biocide, which are used in the waste treatment process to eliminate the bacteria in the water. There were no chemicals identified that would be hazardous to the VEGP site or would require further evaluation.

The chemicals stored at the Plant Wilson combustion turbine plant (6,000 feet from the new AP1000 units' power block), consist of fuel oil, sulfuric acid, and several other chemicals kept in small quantities. These chemicals have low volatility and toxicity, and there would be no potential hazard to the new AP1000 unit control rooms habitability from these substances. The three No. 2 fuel oil tanks located at east of the Service Building for the combustion turbine plant have a capacity of 3,000,000 gallons each (Reference 221). The tanks are surrounded by a dike, which would prevent a fuel leak from spreading into a large spill area. An analysis, based on the methodology of NUREG-0570, has shown that a postulated release of fuel oil from an accidental spill at Plant Wilson will result in a concentration of less than 50 ppm at the air intake for the control room for Unit 3 or 4.

	Quantity	Wind Speed	Stability	Distance to Control Room	Concentration of Vapor at Control Room Air Intake	Toxicity Limit
Fuel Oil	3,000,000 gallons	1 m/s	F	Approximately 5,500 ft	< 50 ppm	300 ppm

Therefore, the Plant Wilson fuel oil storage tanks do not pose a hazard to VEGP Units 3 and 4.

2.2.3.2.3 Potential Hazard from Onsite Storage Tanks

The storage facilities for VEGP Units 1 and 2 are listed in Table 2.2-205. Many of the chemicals listed in that table are excluded from further consideration due to their properties (e.g., low volatility or low toxicity) or due to the relatively small quantities that are stored. The guidelines and methodologies of NUREG-0570 were used to determine the release rates and concentrations of toxic gases at the control room air intake for existing VEGP Units 1 and 2. This analysis shows that the control room would remain habitable for most release scenarios without any operator action and that there would be sufficient time for control room operators to take emergency action (donning emergency breathing apparatus) for the remaining release scenarios. For all releases except hydrazine, the average concentration over an 8-hour period would never exceed the long-term toxicity limit. Where the long-term limit would be exceeded, it has been shown by calculation for VEGP Units 1 and 2 that at least 2 minutes would be available between detection and the time the short-term toxicity limit (as defined in Regulatory Guide 1.78) would be reached. Since hydrazine is stored northeast of the VEGP Unit 1 reactor, this chemical would be separated by a minimum of about 1,800-feet from Units 3 and 4. Therefore, the impact on the new Units 3 and 4 due to an accidental hydrazine release will be expected to be smaller than that for existing Units 1 and 2, and will be evaluated at the time of the COL in accordance with DCD COL Information Item 6.4-1.

The impact on the new Units 3 and 4 due to an accidental hydrazine release is evaluated in Subsection 2.2.3.2.3.1 below.

Additionally, there are three 2000 gallon capacity above-ground liquid propane tanks installed at the Vogtle Fire Training Facility (building 695). These tanks provide fuel to fire training aids installed at the facility. The distance from the tanks to the nearest safety-related structure (the Unit 3 auxiliary building) is approximately 3300 feet and the distance to the control room air intake is approximately 3400 feet. The tanks are designed and installed in accordance with ASME Section VIII (Division 1, 1998 Edition, 2000 Addenda), and the relevant NFPA and insurance standards. The potential hazards considered to be associated with these tanks are:

A. The shockwave from an explosion of the tanks' contents or a flammable vapor cloud emanating from a tank rupture.

B. A dangerous thermal environment resulting from a tank or flammable vapor cloud fire.

C. A potentially lethal concentration of toxic gases in the control room resulting from a tank fire.

D. Diesel generator overspeed, starvation or prevention of a diesel generator start due to a propane tank rupture.

The tanks have been evaluated consistent with the methods described in Regulatory Guide 1.91. Based upon an evaluation, the fire training facility propane tanks are located beyond the distance where an exploding tank or an exploding flammable vapor cloud would yield an overpressure of 1 psi on any safety-related structure. As noted in Regulatory Guide 1.91, the effects of blast-generated missiles will be less than those associated with blast overpressure.

The fire training facility propane tanks have also been evaluated for thermal effects of a fire using the methodology described in NUREG/CR-1748. The calculated temperature rise is less than the maximum allowable temperature rise.

Control room habitability following a propane fire at the fire training facility is bounded by the analysis of Plant Wilson discussed in Subsection 2.2.3.3.2.

The fire training facility propane tank evaluation also determined that the tanks are located beyond the distance where a propane gas cloud could starve or prevent the start of a diesel generator.

Based on the evaluation, the propane tanks at the fire training facility are located at a safe distance from the control room air intake and any safety-related structures. Therefore, the propane tanks are not considered to be a credible hazard.

2.2.3.2.3.1 Hydrazine Hazard from Onsite Storage Tanks

Impact on safety related structures and control room habitability for Units 3 and 4 due to accidental releases from or explosion in the 6,644 gallon Units 1 and 2 hydrazine tank is evaluated below.

The Areal Locations of Hazardous Atmospheres (ALOHA) code (Reference 202) and the TNT equivalency method is used to determine the minimum safe distances for hydrazine that is stored onsite at VEGP. These minimum safe distances for Unit 3 control room are then compared to the distances from where hydrazine is stored to Unit 3. Since the Unit 4 control room is further west of Unit 3, the evaluation is based on Unit 3 only and then the results are applied to Unit 4. The four scenarios evaluated are: toxicity of a vapor cloud, flammability of a vapor cloud, explosive vapor cloud, and a tank explosion.

The assumptions for the three vapor cloud scenarios include the following:

- Hydrazine is a 35% hydrazine solution.
- Atmospheric air flow is turbulent in only one direction (no cross flow) such that the released gases spread downstream in a Gaussian manner.
- Total quantity of hydrazine is released and forms an evaporating puddle with a depth of 1 cm (NUREG-0570). This provides a significant surface area to maximize evaporation and the formation of a vapor cloud.
- Ambient temperature is 95.1°F for daytime releases and 70.1°F for nighttime releases, the relative humidity is 50%, and the atmospheric pressure is 1 atmosphere (40 CFR 68.22).
- A sensitivity study was performed to determine the worst-case meteorological conditions (wind speed and stability class). The worst-case scenario is a wind speed of 2 m/s and stability class “F”.
- Ground roughness is “Urban or Forest” which most accurately represents site conditions.
- Cloud cover selected is based on the appropriate stability class and wind speed (Reference 202).

- Time of accidental release is 12:00 pm on July 21, 2008 for daytime releases and 5:00 a.m. on July 21, 2008 for nighttime releases. The date was selected because it coincides with the highest daily maximum temperature, and 12:00 p.m. was selected because solar radiation is highest during midday. Higher solar radiation leads to a higher evaporation rate and thus a larger vapor cloud. Five o'clock (5:00) a.m. on July 21, 2008 was selected to provide a realistic meteorological condition for the more stable stability classes. ALOHA requires manual override if 12:00 p.m. is used with stability classes "E" and "F", or "D" with a wind speed of 3 m/s (Reference 202).

- Wind input height is 10 meters. ALOHA calculates a wind profile based on where the meteorological data is taken. ALOHA assumes that the MET station is at a height of 10 meters. The National Weather Service usually reports wind speeds from the height of 10 meters.

- There is no temperature inversion.

- It is not known how long after a release ignition occurs for vapor cloud explosions. Therefore, the "unknown" time of vapor cloud ignition option was selected for this case. ALOHA will run explosion scenarios for a range of ignition times that encompass all of the possible ignition times for a scenario. ALOHA takes the results from all of these scenarios and combines them on a single Threat Zone plot.

- Type of vapor cloud ignition is "ignited by detonation." This is the worst case scenario for an accidental explosion.

The assumptions for the tank explosion (TNT mass equivalency) scenario include the following:

- Vapor space is assumed to be the tank volume at the upper flammability limit of hydrazine.

- Air temperature is 32.2°F, the lowest mean daily minimum temperature, which corresponds to an air density of 0.081 lb/ft3.

- Detonation occurs inside the tank.

- Vapor explosion is treated as if it is completely confined. Thus, a yield factor of 100% is used for the confined vapor explosion (NUREG-1805).

Toxicity of a Hydrazine Vapor Cloud

For assessing the toxicity of a vapor cloud from hydrazine release, it is necessary to determine the maximum distance at which the Immediately Dangerous to Life or Health (IDLH) value exists (Regulatory Guide 1.78). This distance represents the minimum safe distance from the hydrazine storage area that a nuclear power plant can operate. The distance depends on the prevailing meteorological conditions, wind speed, relative humidity, atmospheric pressure, ambient temperature, toxicity and the quantity of hydrazine released. It is also necessary to determine the resulting concentration of hydrazine inside the control room to ascertain the effects of a toxic vapor on the operators. ALOHA calculated both the inside and outside concentrations of the control room over time (0 to 1 hour). For this evaluation, a release of 6,644 gallons of 35% hydrazine solution is assumed.

The hydrazine tank is located east of the Unit 1 Turbine Building, 2,200 feet from the Unit 3 control room. The evaluation considers a control room air exchange rate of 0.95 exchanges per hour, and an IDLH for hydrazine of 50 ppm. The maximum vapor cloud distance to the IDLH is calculated to be 927 feet (the resulting maximum concentration at the control room air intake is 15.4 ppm). The maximum concentration of hydrazine inside the control room is calculated to be 7.76 ppm. The resulting hydrazine concentrations inside the Units 3 and 4 control rooms are within the IDLH limit value of 50 ppm.

Results indicate that operators in the Units 3 and 4 control rooms are not impacted by the potential toxicity from a hydrazine vapor cloud.

Flammability of a Hydrazine Vapor Cloud

For assessing the flammability of a vapor cloud from a hydrazine release, the ALOHA air dispersion model is used to determine the distances where the vapor cloud may exist between the upper flammability limit (UFL) and lower flammability limit (LFL) (40 CFR 68.22). Once the concentration of the hydrazine vapor cloud is above the UFL or below the LFL, the vapor is no longer flammable.

For this evaluation, a release of 6,644 gallons of 35% hydrazine solution is analyzed for potential flammable hydrazine vapor threats.

Hydrazine has an LFL of 4.7% and a UFL of 99.9%. The distance from the leak source to the LFL is 54 feet. Though ALOHA does report a distance to the LFL, the vapor cloud does not ever exceed the LFL for any scenario. The distance that is reported is the same for every situation due to near field patchiness. It is further shown that the LFL is never exceeded because, as shown below, no explosions occur, even though a detonation was chosen in every instance.

The distance from the hydrazine storage tank to where the hydrazine vapor cloud exists between the UFL and the LFL is less than the distance from the storage tank to the Units 3 and 4 control rooms. Therefore, results indicate that there is no potential flammable, hydrazine vapor cloud reaching safety related structures or the operators in the Units 3 and 4 control rooms.

Explosive Hydrazine Vapor Cloud

For assessing the explosion from a vapor cloud due to hydrazine release, it is necessary to determine the “safe distance”, the minimum distance required for an explosion to have less than or equal to 1 psi peak incident pressure (Regulatory Guide 1.91). This is the minimum safe distance for no impacts from an explosion of a hydrazine vapor cloud. A peak overpressure of 1 psi will shatter glass but not significantly cause structural damage to buildings (Regulatory Guide 1.91). The peak overpressure to the Unit 3 control room is also established. For this evaluation, a release of 6,644 gallons of 35% hydrazine solution is analyzed for potential explosive vapor threats.

The ALOHA analysis indicates that the vapor cloud does not reach the LFL and, therefore, does not explode. Since there is no explosion, the safety related structures and operators working in the Units 3 and 4 control rooms are not impacted.

Hazard from a Tank Explosion

The methodology presented below is for a confined explosion occurring within some form of a storage container (i.e. tank). Since only vapor will burn or explode, the methodology employed considers the maximum vapor within the hydrazine storage tank as explosive (equivalent TNT). For atmospheric liquid storage, this maximum vapor would involve the container to be completely empty of liquid and filled only with air and chemical vapor at UFL conditions (NUREG-1805). Due to complete confinement and the use of only the UFL vapor mass, a 100% yield factor is attributed to the explosion (NUREG-1805). The equivalent mass of TNT is calculated by taking into account the product of the vapor mass (within the flammable range), heat of combustion, and the explosion yield factor. Once the equivalent mass of TNT is calculated, a radial distance generating a 1 psi peak incident pressure ("safe distance") is calculated by taking the product of the factor 45 and the cube root of the equivalent mass of TNT (Regulatory Guide 1.91).

The evaluation is based on a vapor-filled 6,644 gallon hydrazine tank. For the assumed atmospheric conditions, a heat of combustion of 8,345 Btu/lb, and a vapor specific gravity of 1.1, the mass of flammable hydrazine in the tank is 79 pounds. The resulting equivalent mass of TNT is calculated to be 330 pounds, and the resulting "safe distance" is 311 feet.

Results from the TNT equivalency method indicate that there are no potential explosive vapor threats from hydrazine storage tanks to safety related structures or operators in the Units 3 and 4 control rooms.

As shown in Table 2.2-205, some chemicals previously used for Units 1 and 2 have recently been replaced. Phosphoric acid (Nalco 3DT177) is one of the new chemicals used for the existing Units 1 and 2 that was identified to be toxic. This material is stored in a 5050-gallon tank located between the two existing cooling towers at a distance of approximately 3,200 feet from the air intake for the Unit 3 control room (the closest of the new control rooms to the chemical source). An analysis has shown that under stable atmospheric conditions (F stability) the phosphoric acid concentration outside the new control room air intake would be 94 $\mu g/m^3$, which is much lower than the 8-hour TLV of 1 mg/m^3 and the short term exposure limit of 3 mg/m^3 (Reference 211) following an accidental release. Since this material is not flammable, the explosion effect was not evaluated. Another chemical shown in Table 2.2-205, that was evaluated for Units 1 and 2 is methoxypropylamine (MPA). This chemical is stored in a tank outside the turbine building and in a smaller tank inside the turbine building. The evaluation for Units 1 and 2 considered the failure of the smaller tank, inside the turbine building, due to its proximity to the control room air intake. For that evaluation, the failure assumed a 400 gallon release, 59 meters away from the control room air intake. For a wind speed of 2.5 m/s and a G stability class, the concentration outside the control room intake was calculated to be 1.5 ppm. The STEL for this chemical is 15 ppm. Due to the distance between the new Units 3 and 4 and the existing Units 1 and 2, the effects of accidental MPA release at Units 1 and 2 will be expected to be less than that for the existing Units 1 and 2.

2.2.3.2.3.2 Other Chemical Hazards from Onsite Storage Tanks

Table 6.4-201 provides specific information about the chemicals described in Table 6.4-1. This includes chemical names or limiting types and quantities. Except as noted, these chemicals have been suggested by Westinghouse for use in the AP1000 and have been evaluated in conjunction with AP1000 standard design and found not to present a hazard to the control room operators or to safety-related systems, structures, or components. In some instances, alternative chemicals to those proposed by Westinghouse have been suggested. These chemicals are comparable in function to those proposed by Westinghouse and are the same as those already in use for similar applications in VEGP Units 1 and 2. These chemicals also have been evaluated and found not to present a hazard to the control room operators or to safety-related systems, structures, or components. Therefore, no further analysis is required.

2.2.3.3 Fires

In the vicinity of the VEGP site, the following potential fire hazards exist:

a. Fire due to a transportation accident

b. Forest fire

c. Fire due to an accident at offsite industrial storage facilities

d. Fire due to an onsite storage tank spill

An analysis was performed for VEGP Units 1 and 2 which evaluated the potential fire hazards identified above. Items a, c and d above have been addressed in previous sections. For each event, the analysis concluded that combustion products would not reach concentrations in the VEGP Unit 1 and 2 control room that approached toxicity limits.

An analysis of a postulated forest fire indicates that toxic chemicals (such as CO, NO_2 and CH_4) emitted from the forest fire, located approximately 1,800 feet from the Units 1 and 2 control room, produce negligible concentrations outside the Units 1 and 2 control room air intakes due to the relatively high buoyancy of the plume. In addition, due to the long distance separating the tree line from the control room, the analysis indicates that there would not be any adverse heat impact in the form of heat flux from the forest fire. The temperature rise for each event was calculated to be insignificant when compared with fuel oil fires for causing thermal damage to any safety-related structures at VEGP Units 1 and 2. For all of the fire events evaluated, the location of the new AP1000 units on the VEGP site is the same distance from the source of the fire as the existing VEGP Units 1 and 2, or is further removed, and therefore the same conclusions concerning impact may be made. In addition the design of the control room HVAC for the AP1000 includes smoke detectors. Any smoke detected from an onsite or offsite fire would initiate isolation of the control room HVAC prior to toxicity limits being exceeded.

The specific application to Units 3 and 4 of these forest and industrial fire evaluations is further described below.

2.2.3.3.1 Forest Fires

The surrounding plant terrain is characterized by gently rolling hills and is approximately 30-percent farmland, with the remainder primarily wooded areas. The nearest forest to the Units 1 and 2 control room is the Sandhill-Upland hardwood pine forest with an assumed total area of approximately 3,169 acres and an assumed distance of 1,836 feet away. Based on historical data on forest fires from the state of Georgia, the average size of a forest fire typically is approximately 11.4 acres. The rate of spread is conservatively assumed to be 8 feet per minute with a duration of 4 hours.

The toxic chemicals emitted from a forest fire are CO, NO_2, and CH_4. The emission concentrations in the control room air intake were calculated using the infinite line source diffusion equation with the wind direction perpendicular to the line source and blowing directly toward the control room intake, and the Briggs plume rise equation, which accounts for the buoyancy effect from the heat of the fire. For Units 1 and 2, calculations were performed to demonstrate that the pollutant concentrations outside the control room air intake for a variety of wind speeds (from 0.25 to 10 m/sec) and the Pasquill stability category G are effectively zero. Therefore, the release of toxic combustion products from the onsite forest fire did not pose a hazard to the Units 1 and 2 control room operators.

Using the methodology described in NUREG/CR-1748, the heat flux and resultant temperature rise on plant structures due to a forest fire were also evaluated for Units 1 and 2. The calculated temperature rise (~46.5°C) is less than the allowable temperature rise (bulk 194°C and local 361°C). Therefore, a forest fire will not cause thermal damage to VEGP safety-related structures, based on the distance from the forest.

The centerline of VEGP Units 3 and 4 is approximately 2,100 feet west and 400 feet south of the center of the Unit 2 containment building. The Unit 4 containment is approximately 800 feet west of the Unit 3 containment. It is assumed that the distance from the nearest forest to VEGP Units 3 and 4 is the same as that from the forest to VEGP Units 1 and 2. Since Units 3 and 4 are approximately adjacent to Units 1 and 2 and the vegetation in the vicinity remains the same even after revegetation of the Units 3 and 4 construction site, the toxic chemicals emitted from a forest fire and the emission concentrations in the control room would have the same effect for Units 3 and 4. Therefore, the release of toxic combustion products from the onsite forest fire does not pose a hazard to the Units 3 and 4 control room operators.

2.2.3.3.2 Fire Due to an Accident at Offsite Industrial Storage Facility

Georgia Power Company's combustion turbine plant (Plant Wilson) is located approximately 1,350 meters from the VEGP Units 1 and 2 control room. Of the chemicals and toxic substances stored at this location, diesel fuel oil and miscellaneous oils are flammable. Based on a previous evaluation, a diesel fuel oil fire at Plant Wilson bounds the impacts from any fires of miscellaneous oils stored at Plant Wilson. One of the three tanks containing no. 2 diesel fuel oil is assumed to burn. The entire tank volume of 3×10^6 gallons is spilled into a dike area of 8,756 m^2.

The primary products of combustion emitted from a diesel fuel oil fire at Plant Wilson are CO, CO_2, CH_4, NO_2, SO_2, and SO_3. The toxicity limits in ppm for these constituents are 50 (CO), 5,000 (CO_2), 1.43×10^5 (CH_4), 2 (SO_2 and SO_3), and 3 (NO_2). Using the Briggs plume rise equations and by assuming the maximum burning rate of 0.12 inches/min, the maximum emission rate, duration of fire (8 hours), class A stability, and wind speeds (0.25-10 m/s), it was determined that the resulting concentrations of the primary products of combustion outside the Units 1 and 2 control room air intakes would not approach the above listed toxicity limits.

Using the methodology described in NUREG/CR-1748, the heat flux and resultant temperature rise on the VEGP structures due to a diesel fuel oil fire at Plant Wilson were also evaluated for the Units 1 and 2 control rooms. The calculated temperature rise (115°C) is less than the maximum allowable temperature rise (bulk 194°C and local 361°C). Since a fire at Plant Wilson is limiting (the largest source at the closest distance to the VEGP site), it is concluded that source fires and vapor cloud fires resulting from a delayed ignition at nearby industrial facilities will not cause thermal damage to safety-related structures at VEGP Units 1 and 2.

Units 3 and 4 are located at a farther distance from Plant Wilson than Units 1 and 2. Drawing from the conclusion based on the previous evaluation of Units 1 and 2, any industrial fire due to diesel oil or miscellaneous oils stored at Plant Wilson would not have an impact on control room habitability or cause thermal damage to safety-related structures at Units 3 and 4.

2.2.3.4 Radiological Hazards

The hazard due to the release of radioactive material from either VEGP Units 1 and 2 or the facilities at SRS, as a result of normal operations or an unanticipated event, would not threaten safety of the new units. Smoke detectors, radiation detectors, and associated control equipment are installed at various plant locations as necessary to provide the appropriate operation of the systems. Radiation monitoring of the main control room environment is provided by the radiation monitoring system (RMS). The habitability systems for the AP1000 are capable of maintaining the main control room environment suitable for prolong occupancy throughout the duration of the postulated accidents that require protection from external fire, smoke and airborne radioactivity. Automatic actuation of the individual systems that perform a habitability systems function is provided. In addition, safety related structures, systems, and components for the AP1000 have been designed to withstand the effects of radiological events and the consequential releases which would bound the contamination from a release from either of these potential sources. (Reference 229)

The effect on the control rooms of VEGP Unit 3 and 4 of a postulated design basis accident (DBA) in Unit 1 or 2 was evaluated based on a LOCA in Unit 1 or 2, at uprated conditions, using the releases produced from the alternate source term (AST) methodology. The dose at the Unit 3 and 4 control rooms were determined considering the time-dependent source terms, the atmospheric dispersion factors (χ/Q values), the assumed occupancy rates, the volume of the control room, the HVAC filtration and flow rates, and the operator breathing rates. The χ/Q values from the containment of Unit 2 to the Units 3 and 4 control room air intakes were conservatively calculated using the same methodology and meteorology as was used to calculate the control room χ/Q values presented in Subsection 2.3.4. Breathing rates were assumed to be constant for the control room operators for the duration of the period evaluated. The occupancy rate in the control room was assumed to be 100 percent for the first 24 hours and then decreasing to 60 percent for the next 3 days and then to 40 percent over the remainder of the 30 day period. The resultant dose from this analysis is comparable to the dose reported in Table 15.6.5-3 for a postulated LOCA in the AP1000 and is less than the GDC 19 limits.

2.2.4 Combined License Information for Identification of Site-Specific Potential Hazards

Site-specific information related to the identification of potential hazards within the site vicinity is addressed in Subsections 2.2.3.2.3.1, 2.2.3.2.3.2, 2.2.3.3, and 2.2.3.4.

2.2.5 References

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Table 2.2-1
AP1000 OnSite Explosion Safe Distances

Material	Explosion Minimum Safe Distance(1) (feet)	Flammable Vapor Cloud Safe Distance(1) (feet)	AP1000 Distance to SSC (feet)
Liquid Hydrogen, H_2	577	175	635
Pressurized Gaseous Hydrogen, H_2 (PGS Hydrogen Switchover Station)	10.5	Not Applicable	24
Pressurized Gaseous Hydrogen, H_2 (Spare Cylinders)	6	Not Applicable	10
Hydrazine, N_2H_4	45	Not Applicable	123
Ethanolamine, C_2H_7NO 3-Methoxy propylamine, (MOPA), $C_4H_{11}NO$ Morpholine, $O(CH_2CH_2)_2NH$	56	Not Applicable	123
No. 2 Diesel Fuel Oil	119	Not Applicable	313
Waste Oil	102	Not Applicable	201
Liquid Propane	1440	565	3280

Note:
1. Safe distance is to nearest point of nuclear island SSC.

Table 2.2-201
Nearby Largest Employers

Burke County, GA	Aiken County, SC	Barnwell County, SC
Burke County Hospital	Westinghouse Savannah River	Dixie Narco Inc.
Kwikset Corporation	Aiken County Board of Education	Barnwell School District #45
Management Analysis & Utilization Inc.	Bechtel Savannah River Company	Ness Motley Loadholt Richardson
Samson Manufacturing Inc.	Avondale Mills Inc.	Sara Lee Sock Company Inc.
Southern Nuclear Operating Co. Inc.	Kimberly-Clark Corporation	Excel Comfort Systems Inc.

Table 2.2-202
Description of Products and Materials: Chem-Nuclear Systems, Inc.

Products or Materials	Status	Annual Amounts	Shipment
Isotopes – Including Co-60 (by far largest quantity), Fe-55, and Ni-63	Stored	0.50×10^6 ft^3 (7/1/04-6/30/05) 0.45×10^6 ft^3 (7/1/05-6/30/06) 0.40×10^6 ft^3 (7/1/06-6/30/07) 0.35×10^6 ft^3 (7/1/07-7/30/08)	400/year; average volume - 150 ft^3; largest volume for a single shipment - 8,000 ft^3

Note: The above materials are transported via highway.

Table 2.2-203
Burke County, Georgia, Transportation Accident Data Within 5 Miles of the VEGP Site

	1999	2000	2001	2002	2003
State Route 80					
Accidents					
Injuries	5	0	10	3	3
Fatalities	0	0	0	0	0
State Route 23					
Accidents					
Injuries	14	3	9	15	12
Fatalities	3	0	0	0	0
State Route 56C					
Accidents					
Injuries	0	0	0	0	0
Fatalities	0	0	0	0	0

Table 2.2-204
Bush Field (Augusta) Terminal Area Forecast Fiscal Years 1990–2025 Total Flights

Year	Total[a]
1990	47981
1991	38455
1992	37682
1993	36246
1994	33057
1995	34008
1996	33346
1997	34459
1998	34428
1999	37631
2000	36961
2001	35222
2002	34617
2003	33916
2004	35561
2005	27917
2006	28330
2007	28753
2008	29184
2009	29625
2010	30074
2011	30532
2012	31001
2013	31479
2014	31967
2015	32305
2016	32647
2017	32995
2018	33347
2019	33703
2020	34065
2021	34430
2022	34801
2023	35178
2024	35558
2025	35945

[a] Itinerant Operations (air taxi + commercial air carrier + general aviation + military)

Table 2.2-205
VEGP Units 1 and 2 Onsite Chemical Storage

Material	Quantity	Location
Kitchen Grease	550 gallons	Underground tank east of service building
No. 2 Diesel Fuel	1,500 gallons	South of PESB
No. 2 Diesel Fuel	160,000 gallons*	East of U1 diesel generator building
No. 2 Diesel Fuel	160,000 gallons*	West of U2 diesel generator building
Hydrazine	6,000 gallons	East of turbine building
Methoxypropylamine	400 gallons	Turbine building
Methoxypropylamine	12,780 gallons	East of turbine building
Clean Lube Oil	30,000 gallons	East of turbine building
Dirty Lube Oil	30,000 gallons	East of turbine building
No. 2 Diesel Fuel	100,000 gallons	East of turbine building
No. 2 Diesel Fuel	560 gallons	Fire protection pumphouse
No. 2 Diesel Fuel	560 gallons	Fire protection pumphouse
Main Turbine Lube Oil	12,800 gallons	Turbine building
Main Turbine Lube Oil	12,800 gallons	Turbine building
SGFP Lube Oil	2,800 gallons	Turbine building
SGFP Lube Oil	2,800 gallons	Turbine building
EHC Fluid	1,600 gallons	Turbine building
EHC Fluid	1,600 gallons	Turbine building
No. 2 Diesel Fuel	1,250 gallons	U1 diesel generator building
No. 2 Diesel Fuel	1,250 gallons	U1 diesel generator building
No. 2 Diesel Fuel	1,250 gallons	U2 diesel generator building
No. 2 Diesel Fuel	1,250 gallons	U2 diesel generator building
Unleaded Gasoline	6,000 gallons	East of receiving warehouse
No. 2 Diesel Fuel	3,000 gallons	East of receiving warehouse
Sodium Hypochlorite	6,700 gallons	Main Cooling towers
Dispersant**	4,400 gallons	Main Cooling towers
MS Corrosion Inhibitor***	5,050 gallons	Main Cooling towers
Copper Corrosion Inhibitor****	2,200 gallons	Main Cooling towers
Kerosene	7,000 gallons	Fire training area
Sodium Hypochlorite	250 gallons	East of plant potable water storage tank
Boric Acid	46,000 gallons	U1 aux building
Boric Acid	46,000 gallons	U2 aux building
Used Oil	4,000 gallons	NW of admin support building
Used Oil	5,000 gallons	NW of admin support building
Sodium Bromide	4,000 gallons	Main Cooling towers
Nalco STABREX	6,700 gallons	Main Cooling towers
Sodium Hypochlorite	200 gallons	Plant potable water building
Sodium Phosphate, Tribasic	200 gallons	Plant potable water building
Copper Corrosion Inhibitor****	200 gallons	U1 NSCW tower chemical addition building
Copper Corrosion Inhibitor****	200 gallons	U2 NSCW tower chemical addition building
Ammonium Bisulfite	200 gallons	Circulating water dechlorination building
Liquid Propane	2000 gallons	Fire Training Facility
Liquid Propane	2000 gallons	Fire Training Facility
Liquid Propane	2000 gallons	Fire Training Facility

* Actually two 80,000 gallon tanks that are interconnected and function as one tank.
** Currently using Nalco 3DT102, swapping to Nalco 3DT190 during summer 2006.
*** Currently using Nalco 73297, swapping to Nalco 3DT177 during summer 2006.
**** Currently using Nalco 1336.

Table 2.2-206
Not Used

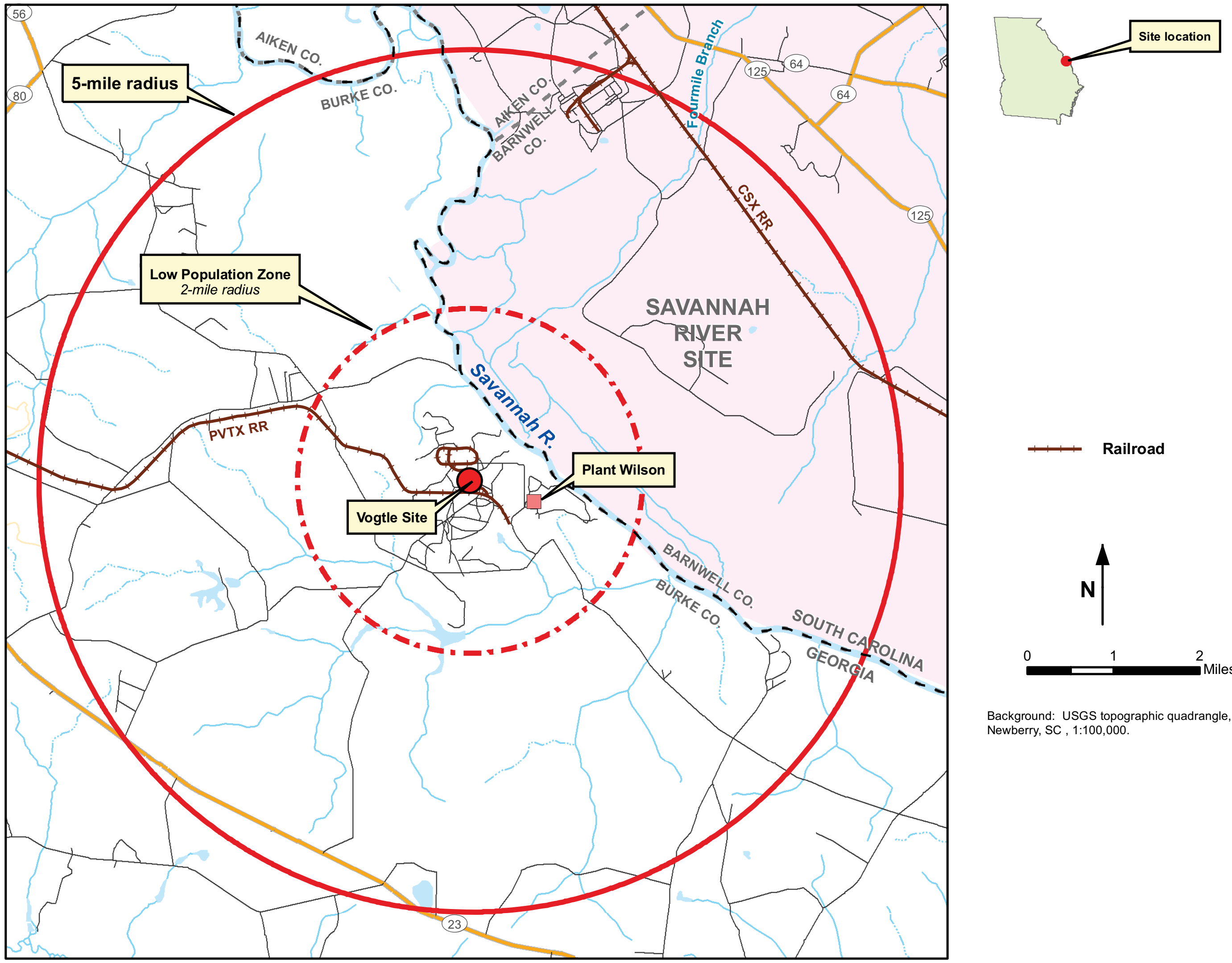

Figure 2.2-201
Site Vicinity Map

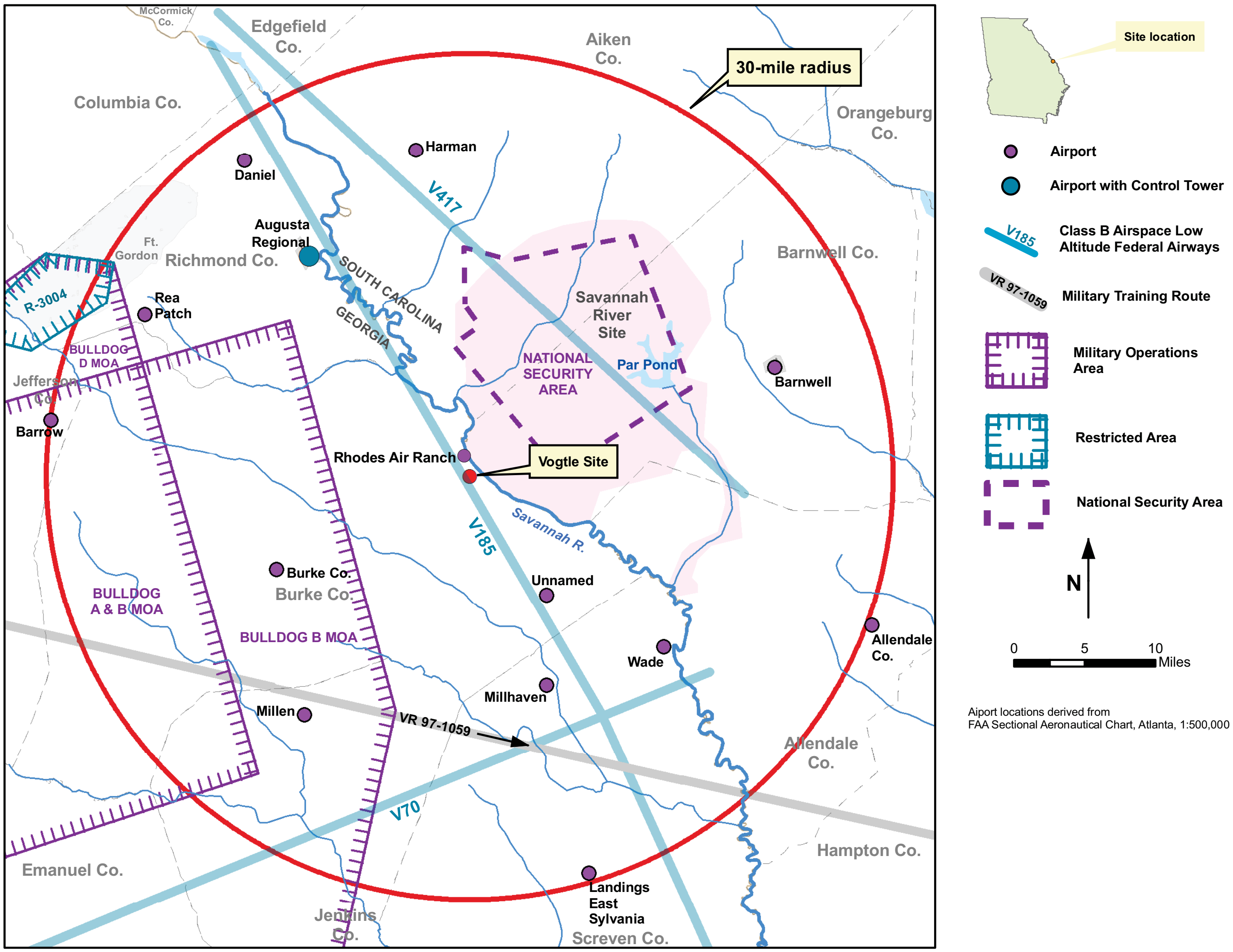

Figure 2.2-202
Airports Within 30 Miles of VEGP

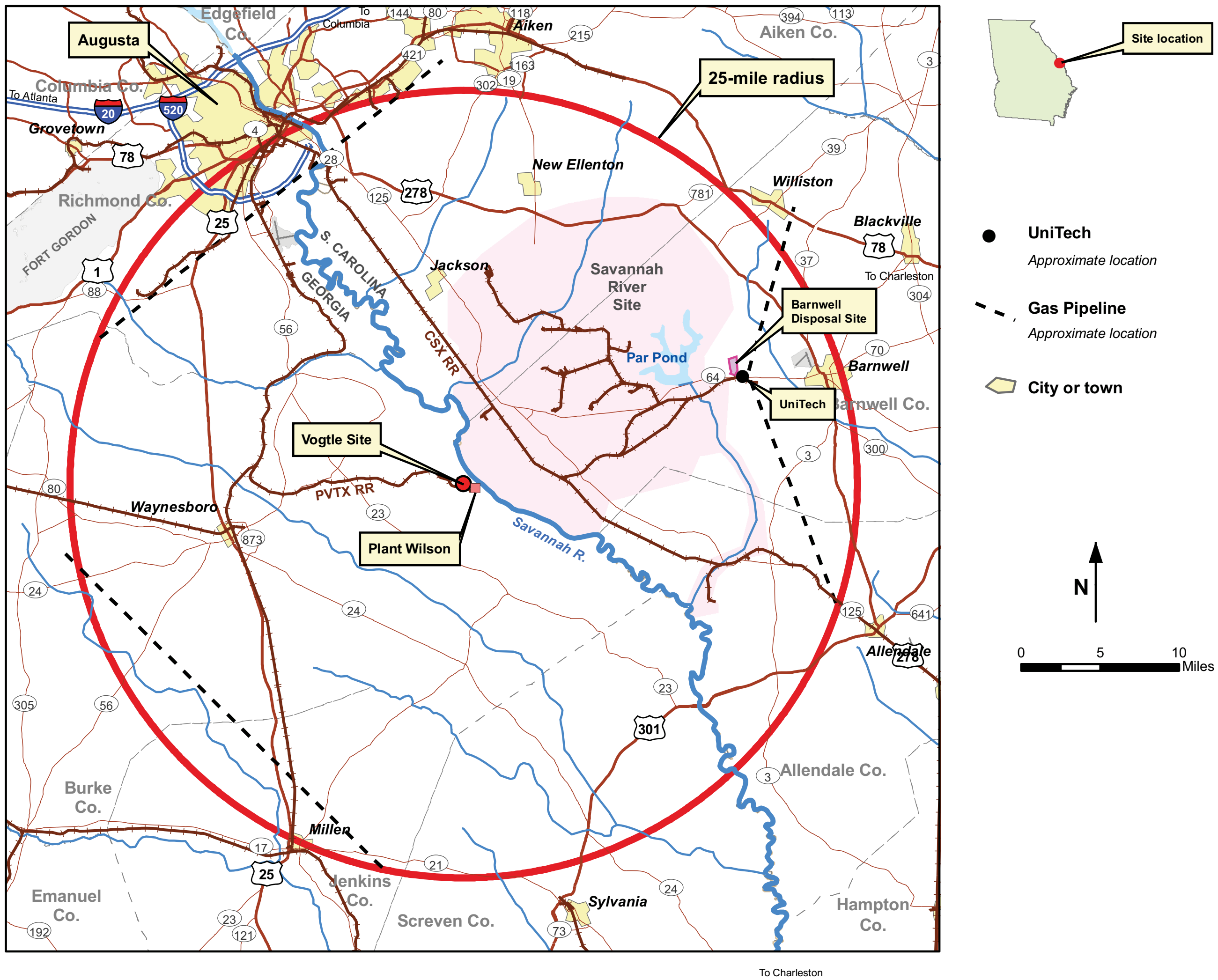

Figure 2.2-203
Industrial Facilities Within 25 Miles of VEGP

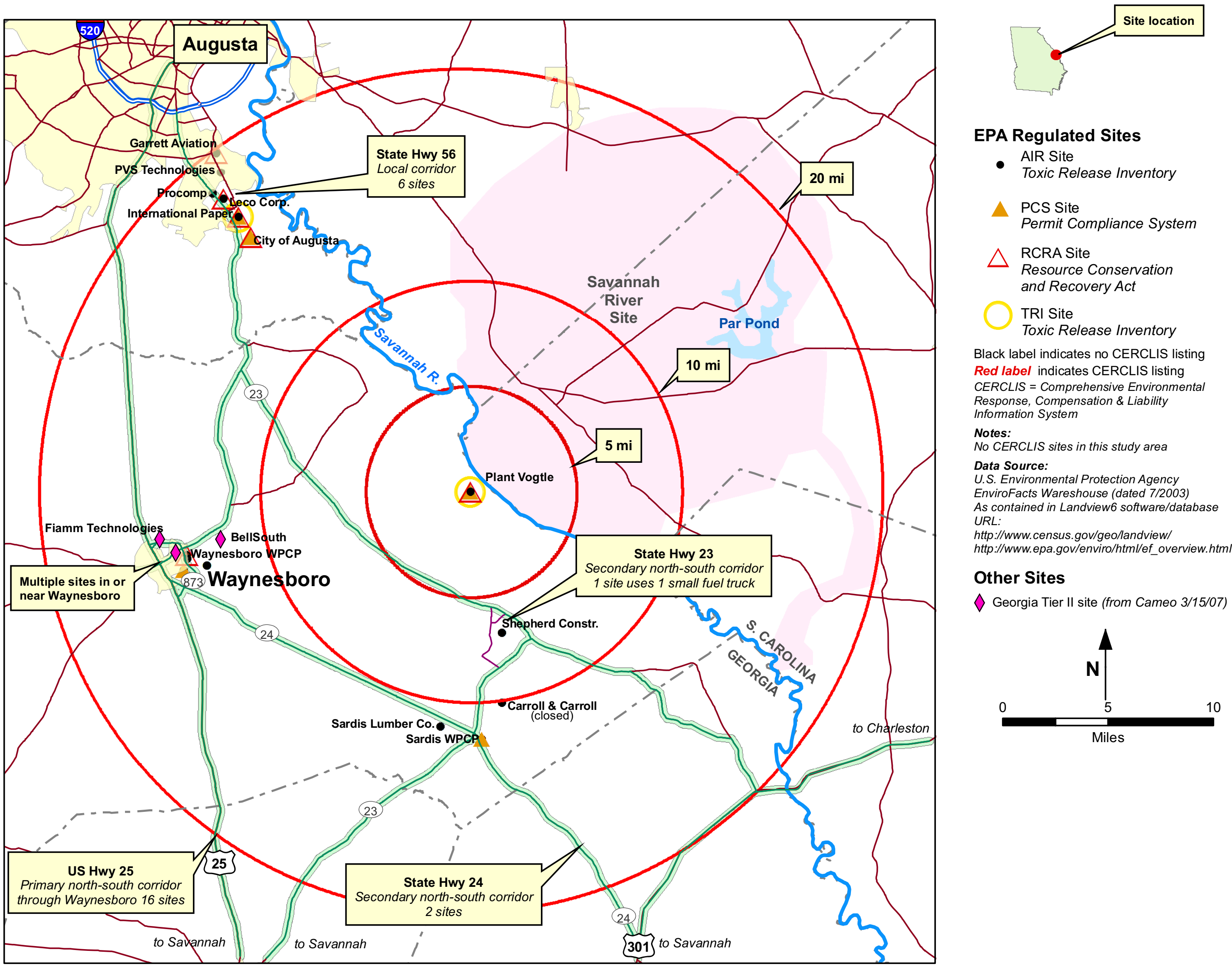

Figure 2.2-204
Corridor Analysis

2.3 Meteorology

The AP1000 is designed for air temperatures, humidity, precipitation, snow, wind, and tornado conditions as specified in Table 2.0-201. The design wind is specified as a basic wind speed of 145 mph with an annual probability of occurrence of 0.02. Wind loads are calculated for exposure C, which is applicable to shorelines in hurricane prone areas. The VEGP site parameters for the design wind are demonstrated to be acceptable by comparison of the wind loads on the structures. Refer to Subsection 2.3.1.3.

This section describes the regional and local climatological and meteorological characteristics applicable to the VEGP site for consideration in the design and operating bases of safety- and/or non-safety related structures, systems and components for proposed VEGP Units 3 and 4. This section also provides site-specific meteorological information for use in evaluating construction-related, routine operational, and hypothetical accidental releases to the atmosphere.

2.3.1 Regional Climatology

The VEGP site is located in the region known as the Upper Coastal Plain, lying between the Appalachian Mountains and the Atlantic Ocean, just south of the Fall Line that separates the Piedmont from the Coastal Plain. Elevation is generally 150 to 250 ft above sea level in this region, which is cut by the valley of the Savannah River. The river valley ranges from 2 to 5 mi wide near the VEGP site.

2.3.1.1 Data Sources

SNC used several sources of data to characterize regional climatological conditions pertinent to the VEGP site. The National Climatic Data Center (NCDC) compiled data from the first-order National Weather Service (NWS) station in Augusta, Georgia, and from nine other nearby locations in its network of cooperative observer stations.

These climatological observing stations are located in Burke, Richmond, Jenkins, Screven, and Jefferson Counties, Georgia, and in Aiken, Barnwell, Orangeburg, and Bamberg Counties, South Carolina. Table 2.3-203 identifies the specific stations and lists their approximate distance and direction from the existing reactors at the VEGP site. Figure 2.3-201 illustrates these station locations relative to the VEGP site.

The objective of selecting nearby, off-site climatological monitoring stations is to demonstrate that the mean and extreme values measured at those locations are reasonably representative of conditions that might be expected to be observed at the VEGP site. The 50-mi radius circle shown in Figure 2.3-201 provides a relative indication of the distance between the climate observing stations and the VEGP site.

However, a 50-km (about 31-mi) grid spacing is considered to be a reasonable fine mesh grid in current regional climate modeling, and this distance was used as a nominal radius for the station selection process. The identification of stations to be included was based on the following considerations:

- Proximity to the site (i.e., within the nominal 50-km radius indicated above, to the extent practicable)
- Coverage in all directions surrounding the site (to the extent possible)

- Where more than one station exists for a given direction relative to the site, a station was chosen if it contributed one or more extreme conditions (e.g., rainfall, snowfall, maximum and/or minimum temperatures) for that general direction.

Nevertheless, if an overall extreme precipitation or temperature condition was identified for a station located within a reasonable distance beyond the nominal 50-km radius and that event was considered to be reasonably representative for the site area, such stations were also included, regardless of directional coverage.

Normals (i.e., 30-year averages), means, and extremes of temperature, rainfall, and snowfall are based on the:

- *2004 Local Climatological Data, Annual Summary with Comparative Data for Augusta, Georgia* (Reference 221)
- *Climatography of the United States, No. 20, 1971-2000, Monthly Station Climate Summaries* (Reference 222)
- *Climatography of the United States, No. 81, 1971-2000, U.S. Monthly Climate Normals* (Reference 211)
- Southeast Regional Climate Center (SERCC), *Historical Climate Summaries and Normals for the Southeast* (Reference 230).
- *Cooperative Summary of the Day, TD3200, Period of Record Through 2001, for the Eastern United States, Puerto Rico and the Virgin Islands* (Reference 213).

First-order NWS stations also record measurements, typically on an hourly basis, of other weather elements, including winds, several indicators of atmospheric moisture content (i.e., relative humidity, dew point, and wet-bulb temperatures), and barometric pressure, as well as other observations when those conditions occur (e.g., fog, thunderstorms). Table 2.3-204, excerpted from the 2004 local climatological data (LCD) summary for the Augusta NWS Station, presents the long-term characteristics of these parameters.

The following data sources were also used in describing climatological characteristics of the VEGP site area and region:

- *Solar and Meteorological Surface Observation Network, 1961-1990, Volume 1, Eastern U.S.* (Reference 227)
- *Hourly United States Weather Observations, 1990-1995* (Reference 210)
- *Integrated Surface Hourly Observations*, 1995-1999 (Reference 215), 2000 (Reference 216), 2001 (Reference 217), 2002 (Reference 218), 2003 (Reference 220), 2004 (Reference 223), 2005 (Reference 226)
- *International Station Meteorological Climate Summary* (Reference 232)
- *Engineering Weather Data, 2000 Interactive Edition, Version 1.0* (Reference 202)
- *Minimum Design Loads for Buildings and Other Structures* (Reference 204)

- *Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian,* Hydrometeorological Report No. 53, June 1980 **(NUREG/CR-1486)**
- *Storm Events for Georgia and South Carolina*, Tornado Event Summaries, accessed July 2005 and January 2006 (Reference 224)
- *Historical Hurricane Tracks Storm Query*, 1851 through 2004 (Reference 228)
- *The Climate Atlas of the United States* (Reference 212)
- *Storm Events for Georgia and South Carolina*, Hail Event and Snow and Ice Event Summaries for Burke, Jenkins, Richmond, and Screven Counties in Georgia, and Aiken, Allendale, and Barnwell Counties in South Carolina (Reference 225)
- *Storm Data (and Unusual Weather Phenomena with Late Reports and Corrections)*, January 1959 (Volume 1, Number 1) to January 2004 (Volume 42, Number 1) (Reference 219)
- *Air Stagnation Climatology for the United States (1948-1998)* (Reference 233)
- *Mixing Heights, Wind Speeds, and Potential for Urban Air Pollution Throughout the Contiguous United States* (Reference 209)
- *Climatography of the United States, No. 85, Divisional Normals and Standard Deviations of Temperature, Precipitation, and Heating and Cooling Degree Days 1971-2000 (and previous normals periods)* (Reference 214)

2.3.1.2 General Climate

The general climate in this region is characterized by mild, short winters; long periods of mild sunny weather in the autumn; somewhat more windy but mild weather in spring; and long, hot summers.

The regional climate is predominately influenced by the Azores high-pressure system. Due to the clockwise circulation around the western extent of the Azores High, maritime tropical air mass characteristics prevail much of the year, especially during the summer with the establishment of the Bermuda High and the Gulf High. Together, these systems govern Georgia’s summertime temperature and precipitation patterns. This macro-circulation feature also has an effect on the frequency of high air pollution potential in the VEGP site region. These characteristics and their relationship to the Bermuda High, especially in the late summer and autumn, are addressed in Subsection 2.3.1.6.

This macro-scale circulation feature continues during the transitional seasons and winter months; however, it is regularly disrupted by the passage of synoptic- and meso-scale weather systems. During winter, cold air masses may briefly intrude into the region with the cyclonic (i.e., counter-clockwise) northerly flow that follows the passage of low-pressure systems. These systems frequently originate in the continental interior around Colorado, pick up moisture-laden air due to southwesterly through southeasterly airflow in advance of the system, and result in a variety of precipitation events that include rain, snow, sleet, and freezing rain or mixtures, depending on the temperature characteristics of the weather system itself and the temperature of the underlying air (see Subsection 2.3.1.3.5). Similar cold air intrusion and precipitation patterns may also be associated with secondary low-pressure systems that form in the eastern Gulf of Mexico or along the Atlantic Coast and move northeastward along the coast (also referred to as “nor’easters”).

Larger and relatively more persistent outbreaks of very cold, dry air associated with massive high-pressure systems that move southeastward out of Canada also periodically affect the VEGP site region. These weather conditions are moderated by the Appalachian Mountains to the northwest, which shelter the region in winter from these cold air masses that sweep down through the continental interior. In general, the cold air that does reach the VEGP site area is warmed by its descent to the relatively lower elevations of the region, as well as by modification due to heating as it passes over the land.

Monthly precipitation exhibits a cyclical pattern, with one maximum during the winter into early spring and a second maximum during late spring into summer (see Table 2.3-204). The winter and early spring maximum is associated with low-pressure systems moving eastward and northward through the Gulf States and up the Atlantic Coast, drawing in warm, moist air from the Gulf of Mexico and the Atlantic Ocean. These air masses receive little modification as they move into the region. The late spring and summer maximum is due to thunderstorm activity. Heavy precipitation associated with late summer and early autumn tropical cyclones, as discussed in Subsection 2.3.1.3.3, is not uncommon. The VEGP site is located far enough inland that the strong winds associated with tropical cyclones are much reduced by the time that such systems affect the site area.

2.3.1.3 Severe Weather

2.3.1.3.1 Extreme Winds

Estimating the wind loading on plant structures for design and operating bases considers the “basic” wind speed, which is the “3-second gust speed at 33 ft (10 m) above the ground in Exposure Category C,” as defined in Sections 6.2 and 6.3 of the ASCE-SEI design standard, *Minimum Design Loads for Buildings and Other Structures* (Reference 204).

The basic wind speed for the VEGP site is about 97 mph, as estimated by linear interpolation from the plot of basic wind speeds in Figure 6-1 of ASCE (2002) for that portion of the U.S. that includes the VEGP site (Reference 204). This interpolated value is about 7.5 percent higher than the basic wind speed reported in the Engineering Weather Data summary for the Augusta (Bush Field) NWS Station (i.e., 90 mph) (Reference 202), which is located about 20 mi northwest of the VEGP site. The former value is, therefore, considered to be a reasonably conservative indicator of the basic wind speed.

From a probabilistic standpoint, these values are associated with a mean recurrence interval of 50 years. Section C6.0 of the ASCE-SEI design standard provides conversion factors for estimating 3-second-gust wind speeds for other recurrence intervals (Reference 204). Based on this guidance, the 100-year return period value is determined by multiplying the 50-year return period value by a scaling factor of 1.07, which yields a 100-year return period 3-second-gust wind speed for the VEGP site of about 104 mph.

2.3.1.3.2 Tornadoes

The design-basis tornado (DBT) characteristics applicable to structures, systems, and components important to safety at the proposed VEGP site include the following parameters as identified in Draft Regulatory Guide DG-1143, *Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants, Proposed Revision 1 of NRC Regulatory Guide 1.76 (dated April 1974)*, January 2006 (DG-1143) and the predecessor US Atomic Energy Commission (USAEC) guidance document WASH-1300, *Technical Basis for Interim Regional Tornado Criteria* (Reference 231)**,** on which the original version of Regulatory Guide 1.76 is based:

- Tornado strike probability

- Maximum wind speed
- Translational speed
- Maximum rotational wind speed
- Radius of maximum rotational speed
- Pressure drop
- Rate of pressure drop

The tornado strike probability is determined by evaluating certain characteristics of tornadoes that have been observed within a 2-degree latitude and longitude square centered on the VEGP site. These characteristics include the Fujita-scale wind speed classification (or “F-scale”) and the Pearson-scale path length and path width classification (or “P-scale”). As tornado intensity increases, so does the magnitude or the dimensions of these parameters along with the assigned numerical classification, which ranges from 0 to 5.

The 2-degree square area was assumed to be centered on the VEGP Unit 1 reactor, adjacent to the new unit footprint, and located at the following coordinates:

Latitude = 33° 08’ 30” N; Longitude = 81° 45’ 44” W

A searchable database of tornado occurrences by location, date, and time; starting and ending coordinates; F-scale classification; P-scale dimensions; and damage statistics has been compiled by the NCDC beginning with January 1950 (Reference 224). The 2-degree square area for this evaluation includes all or portions of 30 counties in Georgia and all or portions of 18 counties in South Carolina.

Through the nearly 55-year period ending April 30, 2005, the records in the database indicate that a total of 348 tornadoes or portions of a tornado path passed within the 2-degree square area centered on the VEGP site. Tornado F-scale classifications (with corresponding wind speed range) and respective frequencies of occurrence are as follows:

- F5 (wind speed > 117 m/sec) = 0
- F4 (wind speed 93 to 116 m/sec) = 1
- F3 (wind speed 70 to 92 m/sec) = 18
- F2 (wind speed 50 to 69 m/sec) = 62
- F1 (wind speed 33 to 49 m/sec) = 151
- F0 (wind speed 18 to 32 m/sec) = 116

Following the WASH-1300 methodology, the probability that a tornado will strike a particular location during any one year is given as:

PS = n (a / A)

where:

P_S = mean tornado strike probability per year
n = average number of tornadoes per year in the area being considered
a = average individual tornado area
A = total area being considered (i.e., the 2-degree square area)

Based on an average occurrence of 6.29 tornadoes per year (i.e., 348 tornadoes over a 55.33-year period of record), an average individual tornado area of 0.197 sq mi (i.e., an average tornado path length of 3.3 mi and an average tornado path width of 105.3 yds), and a total area of 16,010 sq mi for the 2-degree square under consideration, the tornado strike probability (P_S) for the VEGP site area is estimated to be about 774 x 10^{-7} (about 0.0000774 per year), or a recurrence interval of once every 12,920 years.

WASH-1300 indicates that determination of the DBT characteristics is based on the premise that the probability of occurrence of a tornado that exceeds the DBT should be on the order of 10^{-7} per year per nuclear power plant. DG-1143 retains that threshold criterion.

The estimated recurrence interval for the VEGP site area exceeds this threshold; therefore, it is necessary to determine the DBT parameters listed at the beginning of this section. These parameters are able to be calculated from the area-specific database used to determine PS. However, DG-1143 also provides DBT characteristics for three tornado intensity regions, each with a 10^{-7} probability of occurrence, that are acceptable to the agency.

As indicated in DG-1143, Figure 1, the VEGP site is adjacent to Tornado Intensity Regions I and II. The more conservative DBT parameters for Region I will be used for the design of structures, systems, and components that are important to safety that must take DBT characteristics into account. DG-1143, Table 1, provides the following DBT parameter values for Tornado Intensity Region I:

- Maximum wind speed = 300 mph
- Translational speed = 60 mph
- Maximum rotational wind speed = 240 mph
- Radius of maximum rotational speed = 150 ft
- Pressure drop = 2.0 psi
- Rate of pressure drop = 1.2 psi/sec

2.3.1.3.3 Tropical Cyclones

Tropical cyclones include not only hurricanes and tropical storms, but systems classified as tropical depressions, sub-tropical depressions, and extra-tropical storms, among others. This characterization considers all “tropical cyclones” (rather than systems classified only as hurricanes and tropical storms) because storm classifications are generally downgraded once landfall occurs and the systems weaken, although they may still result in significant rainfall events as they travel through the site region.

NOAA’s Coastal Services Center (NOAA-CSC) provides a comprehensive historical database, extending from 1851 through 2004, of tropical cyclone tracks based on information compiled by the National Hurricane Center. This database indicates that a total of 102 tropical cyclone centers or storm tracks have passed within a 100-nautical mile radius of the VEGP site during this historical period (Reference 228). Storm classifications and respective frequencies of occurrence over this 154-year period of record are as follows:

- Hurricanes – Category 3 (5), Category 2 (4), Category 1 (16)
- Tropical storms – 46
- Tropical depressions – 23
- Sub-tropical storms – 1
- Sub-tropical depressions – 2
- Extra-tropical storms – 5

Tropical cyclones within this 100-nautical-mile radius have occurred as early as May and as late as November, with the highest frequency (36 out of 102 events) recorded during September, including all classifications except sub-tropical depressions. August and October account for 21 and 20 events, respectively, indicating that 75 percent of the tropical cyclones that affect the VEGP site area occur from mid-summer to early autumn. Three of the five Category 3 hurricanes occurred in September, and the other two occurred in August.

Tropical cyclones are responsible for at least 12 separate rainfall records at 8 NWS cooperative observer network stations in the VEGP site area – eight 24-hour (daily) rainfall totals and 3 monthly rainfall totals (see Table 2.3-205). In October 1990, rainfall associated with Tropical Depression Marco (along with a slow-moving cold frontal system) resulted in historical daily maximum totals of 8.60 in. at the Louisville 1E Station, 8.19 in. at the Midville Experiment Station, and 5.50 in. at the Newington 2NE Station, all located in Georgia. Two daily records were established due to Hurricane Gracie in September 1959, at the Blackville 3W (7.53 in.) and Springfield (7.10 in.) stations in South Carolina. In August 1964, a 24-hour rainfall total of 8.02 in. was recorded at the Millen 4N Station (in Georgia) due to Tropical Storm Cleo; and in September 2000, Tropical Depression Helene produced 8.02 in. of rain in a 24-hour period at the Bamberg, South Carolina, observing station. A daily maximum total of 7.30 in. was measured at the Augusta Weather Service Office (WSO) (also in Georgia) in September 1998 during the passage of Tropical Storm Earl (Reference 219, Reference 225; Reference 230).

Monthly station records were established due to contributions from the following tropical cyclones: Tropical Depression Marco in October 1990 (14.82 in. at Augusta WSO and 14.67 in. at Blackville 3W); Tropical Storm Cleo in August 1964 (13.45 in. at Millen 4N); and to some extent, Tropical Depression Jerry in August 1995 (15.26 in. at Bamberg) (Reference 215, Reference 219, Reference 225).

2.3.1.3.4 Precipitation Extremes

Because precipitation is a point measurement, mean and extreme statistics, such as individual storm event, or daily or cumulative monthly totals typically vary from station to station. Assessing the variability of precipitation extremes over the VEGP site area, in an effort to evaluate whether the available long-term data are representative of conditions at the site, is largely dependent on station coverage.

Historical precipitation extremes (rainfall and snowfall) are presented in Table 2.3-205 for the ten nearby climatological observing stations listed in Table 2.3-203. Based on the similarity of the maximum recorded 24-hour and monthly totals among these stations and the areal distribution of these stations around the VEGP site, the data suggest that these statistics are reasonably representative of precipitation extremes that might be expected to be observed at the site.

As indicated in Subsection 2.3.1.3.3, most of the individual station 24-hour rainfall records (and to a lesser extent the monthly record totals) were established as a result of precipitation associated with tropical cyclones that passed within a 100-nautical-mile radius of the VEGP site.

However, the overall highest 24-hour rainfall total in the VEGP site area — 9.68 in. on April 16, 1969, at the Aiken 4NE Station in South Carolina (Reference 222), about 25 mi north-northeast of the VEGP site—was not associated with a low-pressure system or other well-defined synoptic-scale feature. Rather, this appears to have been an embedded, localized event in an otherwise widespread area of disturbed weather that brought precipitation to the entire East Coast (Reference 208).

Similarly, the overall highest monthly rainfall total recorded in the VEGP site area —17.32 in. during June 1973 at the cooperative observing station in Springfield, South Carolina **(**Reference 230**;** Reference 213**)**, 37 mi northeast of the VEGP site — represents the accumulation of 21 days of measurable precipitation during that month (Reference 213) due to both synoptic-scale weather features (e.g., stationary frontal boundaries and stalled low-pressure areas off the Carolina coast) and more regional- to local-scale events (i.e., thunderstorms).

For the most part, when daily or monthly rainfall records were established at a given station, regardless of their cause(s), significant amounts of precipitation were usually measured at the other stations in the VEGP site area (Reference 213).

Although the disruptive effects of any winter storm accompanied by frozen precipitation can be significant in the Upper Coastal Plain of Georgia and South Carolina, storms that produce large measurable amounts of snow occur infrequently. With one exception, all of the 24-hour and monthly record snowfall totals listed in Table 2.3-205 were established during the storm of early February 1973, the highest 24-hour and monthly totals (19.0 and 22.0 in., respectively) being recorded at the Bamberg Station in South Carolina, about 44 mi east-northeast of the VEGP site. Similar amounts, ranging from 14.0 to 17.0 in., were recorded at most of the other stations **(**Reference 222**;** Reference 230**)**.

The stations with lower maximum 24-hour snowfall totals — 8.0 in. at the Augusta WSO on February 9 and 5.0 in. at Newington 2NE on February 10 (both in Georgia) **(**Reference 222**;** Reference 230**)**, and 8.0 in. at Springfield, South Carolina, on February 11 (Reference 230**;**Reference 213) — recorded a comparable amount of snowfall on the preceding or following day, making the 2-day totals for these stations similar to the single-day records at the other stations (except at the Newington 2NE station, the lowest of all the station records).

The record monthly snowfall total at the Millen 4N Station (15.0 in. in February 1968) represents the cumulative amount from two smaller snow events that occurred around February 8 and from February 22 to 24. A review of the daily records for the other stations indicates that except for the Augusta (Georgia) and Blackville 3W (South Carolina) stations, the data are missing for these time periods. (Reference 213)

Estimating the design basis snow load on the roofs of safety-related structures considers two climate-related components: the weight of the 100-year return period ground-level snowpack, and the weight of the 48-hour probable maximum winter precipitation (PMWP). From a probabilistic standpoint, the estimated weight of the 100-year return period ground-level snowpack for the VEGP site area is about 10 lb/ft^2, as determined in accordance with the guidance in Section C7.0 of the ASCE-SEI design standard, *Minimum Design Loads for Buildings and Other Structures* (Reference 204).

The 48-hour PMWP component is derived from plots of 24- and 72-hour, 10-sq mi area, monthly probable maximum precipitation (PMP) as presented in NUREG/CR-1486, *Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Extremes, United States East of the 105th Meridian,* NOAA Hydrometeorological Report No. 53, June 1980 (NUREG/CR-1486). The highest winter season (i.e., December through February) PMP values for the VEGP site area occur in December. The 48-hour PMWP value is determined by linear interpolation between the 24- and 72-hour PMP values for that month (Figures 35 and 45 of NUREG/CR-1486) and result in a value of 28.3 in. One inch of liquid water is equivalent to 5.2 lb/ft^2; therefore, the estimated weight of the 48-hour PMWP is about 147 lb/ft^2.

The AP1000 safety-related roofs are sloped and designed to handle winter snowpack with margin to handle rainfall on top of the snowpack. The AP1000 design basis snow load of 75 psf (ground) and 63 psf (roof) has sufficient margin to include the weight of rain water adding to a pre-existing snow pack. Using ASCE 7-98 the design snow load 50 psf (ground) converts to 42 psf (roof). Therefore, the AP1000 design includes a 21 psf (63 - 42) margin above the design ASCE 7-98 requirement. This margin could accommodate the equivalent weight of 4" of water within the snow on the roof.

Winter PMP loads in excess of this loading are not considered credible based on the design of the roof. The safety related roofs are constructed of 15" thick reinforced concrete supported by steel beams. The roofs will not deflect enough to hold water under the snow load; therefore, ponding of rain water with pre-existing snow pack conditions will not occur. The physical arrangement of the AP1000 sloped roof is designed such that the 100-year snow pack will not prevent the winter PMP water from draining off the sloped roof system.

In addition the AP1000 roof includes R10 insulation that assures uniform temperatures on the roof surface. This minimizes the potential for ice dams that are typically formed across roofs with a temperature differential.

For the VEGP site, the 100 year snow load is 10 psf which is well within the 63 psf design basis snow load of the AP1000. Thus, for the VEGP site, a 53 psf margin is available to accommodate winter PMP water that may be impounded in the 100-year snow pack as the water flows off of the roof.

2.3.1.3.5 Hail, Snowstorms, and Ice Storms

Frozen precipitation typically occurs in the form of hail, snow, sleet, and freezing rain. The frequency of occurrence of these types of weather events in the VEGP site area is based on the latest version of *The Climate Atlas of the United States* (Reference 212), which has been developed from observations made over the 30-year period of record from 1961 to 1990.

Though hail can occur at any time of the year and is associated with well-developed thunderstorms, it has been observed primarily during the spring and early summer months and least often during the late summer and autumn months. The Climate Atlas indicates that Burke County, Georgia, and adjacent Barnwell County, South Carolina, can expect, on average, hail with diameters 0.75 in. or greater about 1 day per year. The occurrence of hailstorms with hail greater than or equal to 1.0 in. in diameter averages less than 1 day per year in Burke County.

However, the annual mean number of days with hail 0.75 in. and 1.0 in. or greater is slightly higher in nearby Richmond and Columbia Counties, Georgia (just to the northwest of the VEGP site), and in Aiken and Edgefield Counties, South Carolina (just to the north and north-northwest of the VEGP site), ranging from 1 to 2 days per year (0.75 in. diameter or greater) and up to 1 day per year (1.0 in. diameter or greater).

NCDC cautions that hailstorm events are point observations and somewhat dependent on population density. While no hailstorms of note have been recorded in some years, multiple events have been observed in other years, including 16 events on 9 separate dates in 1998 and 8 events on 8 separate dates during 1999 in Aiken County, and 8 events on 6 separate dates during 1998 in Richmond County (Reference 225). Therefore, the slightly higher annual mean number of hail days may be a more representative indicator of frequency for the relatively less-populated VEGP site area.

Despite these long-term statistics, golfball-size hail (about 1.75 in. in diameter) is not a rare occurrence (Reference 219, Reference 225). However, in terms of extreme hailstorm events, the NCDC publication *Storm Data* indicates that baseball-size hail (about 2.75 in. in diameter) was observed at one location in the general VEGP site area (Reference 219) on May 21, 1964, at Hampton, South Carolina, about 43 mi southeast of the VEGP site.

Snow is infrequent in the Upper Coastal Plain of Georgia and South Carolina, where the VEGP site is located, but can occur when a source of moist air from the Atlantic Ocean or the Gulf of Mexico interacts with a very cold air mass that penetrates across the otherwise protective Appalachian mountain range in northern Georgia and northwestern South Carolina. The Climate Atlas (Reference 212) indicates that the occurrence of snowfalls 1 in. or greater in the VEGP site area averages less than 1 day per year.

Heavy snow is a rarity. The greatest snowfall on record in the VEGP site area occurred between February 9 and 11, 1973, depending on the cooperative observing station records. Snowfall totals for the overall event typically ranged between 14 and 22 in., the highest single-day total recorded at the Bamberg Station (19.0 in.) on February 10, which contributed to the highest cumulative monthly total for that station and for the site area. Single-day and cumulative monthly record snowfall totals were also set at nearly all of the other nearby cooperative observing stations as a result of this event. Additional details were given previously in Subsection 2.3.1.3.4 and Table 2.3-205.

Depending on the temperature characteristics of the air mass, snow events are often accompanied by or alternate between sleet and freezing rain as the weather system traverses the VEGP region. The Climate Atlas (Reference 212) indicates that, on average, freezing precipitation occurs only about 1 or 2 days per year in the VEGP site area.

However, the site area appears to be in a transition zone for frequency of occurrence, with the eastern two-thirds of Aiken and Barnwell Counties and all of Allendale County (immediately to the northeast, east, and southeast in South Carolina) and the northeastern quadrant of Screven County, Georgia (just to the southeast of the VEGP site in northeastern Burke County), showing an average frequency of 3 to 5 days of freezing precipitation per year (Reference 212). Therefore, it is not unreasonable to expect a slightly higher annual frequency of occurrence of freezing precipitation events at the VEGP site.

Storm event records from the winters of 2000 through 2005 for the seven-county area surrounding the VEGP site note that ice accumulations of up to 1 in. have occurred, although it is typically less than this thickness (Reference 225).

2.3.1.3.6 Thunderstorms

Thunderstorms can occur in the VEGP site area at any time during the year. Based on a 54-year period of record, Augusta, Georgia, averages about 52 thunderstorm-days (i.e., days on which thunder is heard at an observing station) per year. On average, July has the highest monthly frequency of occurrence — about 12 days. On an annual basis, nearly 60 percent of thunderstorm-days are recorded between late spring and mid-summer (i.e., from June through August). From October through January, a thunderstorm might be expected to occur about 1 day per month. (Reference 221)

The mean frequency of lightning strikes to earth can be estimated using a method attributed to the Electric Power Research Institute, as reported by the US Department of Agriculture Rural Utilities Service in the publication entitled Summary of Items of Engineering Interest Reference 206. This methodology assumes a relationship between the average number of thunderstorm-days per year (T) and the number of lightning strikes to earth per square mile per year (N), where:

$$\boldsymbol{N} = 0.31\boldsymbol{T}$$

Based on the average number of thunderstorm-days per year at Augusta, Georgia (i.e., 52; see Table 2.3-204), the frequency of lightning strokes to earth per square mile is about 16 per year for the VEGP site area. This frequency is essentially equivalent to the mean of the 5-year (1996 to 2000) flash density for the area that includes the VEGP site, as reported by the NWS—4 to 8 flashes per square kilometer per year Reference 229—and, therefore, a reasonable indicator.

The potential reactor area for VEGP Units 3 and 4 is represented in Figure 1.1-202 as an area bounded by a 775-ft-radius circle (or approximately 0.068 mi^2). Given the estimated annual average frequency of lightning strokes to earth in the VEGP site area, the frequency of lightning strokes in the reactor area can be calculated as follows:

(16 lightning strokes/mi^2/year) X (0.068 mi^2) = 1.09 lightning strokes/year
or about once each year in the reactor area.

2.3.1.4 Meteorological Data for Evaluating the Ultimate Heat Sink

Unlike the Vogtle 1 and 2 design, the AP1000 design does not use a cooling tower to release heat to the atmosphere following a Loss-of-Coolant Accident (LOCA). Instead, the AP1000 design uses a passive containment cooling system (PCS) to provide the safety-related ultimate heat sink (UHS) for the plant (Reference 234). The PCS uses a high-strength steel containment vessel inside a concrete shield building. The steel containment vessel provides the heat transfer surface that removes heat from inside the containment and transfers it to the atmosphere.

Heat is removed from the containment vessel by continuous, natural circulation of air. In the event of a LOCA, a high-pressure signal activates valves, allowing water to drain by gravity from a storage tank installed on top of the shield building. An air flow path is formed between the shield building and the containment vessel to aid in the evaporation and is exhausted through a chimney at the top of the shield building (Reference 205).

The use of the PCS in the AP1000 design is not significantly influenced by local weather conditions. Therefore, the identification of meteorological conditions that are associated with maximum evaporation and drift loss of water, as well as minimum cooling by the UHS (i.e., periods of maximum wet-bulb temperatures) is not necessary.

A reactor design has been chosen as specified in Section 1.1 that does not use an ultimate heat sink cooling tower to release heat to the atmosphere following a loss of coolant accident; therefore, evaluation of meteorological site characteristics such as maximum evaporation and drift loss and minimum water cooling conditions used to evaluate this design is not necessary.

2.3.1.5 Design Basis Dry- and Wet-Bulb Temperatures

Long-term, engineering-related climatological data summaries, prepared by the AFCCC and the NCDC for the nearby Augusta NWS Station (Reference 202) are used to characterize typical design basis dry- and wet-bulb temperatures for the VEGP site. These characteristics include:

- Maximum ambient threshold dry-bulb (DB) temperatures at annual exceedance probabilities of 2.0 and 0.4 percent, along with the mean coincident wet-bulb (MCWB) temperatures at those values.

- Minimum ambient threshold DB temperatures at annual exceedance probabilities of 1.0 and 0.4 percent.

- Maximum ambient threshold wet-bulb temperature with an annual exceedance probability of 0.4 percent.

Based on the 24-year period of record from 1973 to 1996 for Augusta, Georgia, the maximum DB temperature with a 2.0 percent annual exceedance probability is 92°F, with a MCWB temperature of 75°F. The maximum DB temperature with a 0.4 percent annual exceedance probability is 97°F with a corresponding MCWB temperature value of 76°F. (Reference 202)

For the same period of record, the minimum DB temperatures with 1.0 and 0.4 percent annual exceedance probabilities are 25°F and 21°F, respectively. The maximum wet-bulb temperature with a 0.4 percent annual exceedance probability is 79°F. (Reference 202)

The AFCCC-NCDC data summaries, from which the dry-bulb and mean coincident wet-bulb temperatures, presented above, were obtained, do not include values that represent return intervals of 100 years. Maximum dry-bulb, minimum dry-bulb, and maximum wet-bulb temperatures corresponding to a 100-year return period were derived through linear regression using individual daily maximum and minimum dry-bulb temperatures and maximum daily wet-bulb temperatures for each year over a 30-year period of record from 1966 through 1995 at the Augusta, Georgia, NWS station (Reference 227; Reference 210).

Based on the linear regression analyses of these data sets for a 100-year return period, the maximum dry-bulb temperature is estimated to be about 115°F, the minimum dry-bulb temperature is estimated to be about -8°F, and the maximum wet-bulb temperature is estimated to be about 88°F.

The Westinghouse basis for the determination of maximum design-basis dry- and wet-bulb (WB) temperature values reflected in the AP1000 design (Reference 234, Reference 235) is summarized below:

- <u>Maximum Safety Dry-Bulb and Coincident Wet-Bulb Temperatures</u>. These site parameter values represent a maximum DB temperature that exists for 2 hours or more, combined with the maximum WB temperature that exists in that population of dry-bulb temperatures. Note that this coincident WB temperature is not defined in the same way as the MCWB values presented previously.

- <u>Maximum Safety Wet-Bulb Temperature (Non-Coincident)</u>. This site parameter value represents a maximum WB temperature that exists within a set of hourly data for a duration of 2 hours or more.

- <u>Maximum Normal Dry-Bulb and Coincident Wet-Bulb Temperatures</u>. The DB temperature component of this site parameter pair is represented by a maximum DB temperature that exists for 2 hours or more, excluding the highest 1 percent of the values in an hourly data set. The WB temperature component is similarly represented by the highest WB temperature excluding the highest 1 percent of the data, although there is no minimum 2-hour persistence criterion associated with this WB temperature. The coincident WB temperature is not defined in the same way as the MCWB values presented previously.

- Maximum Normal Wet-Bulb Temperature (Non-Coincident). This site parameter value represents a maximum WB temperature, excluding the highest 1 percent of the values in an hourly data set (i.e., a 1 percent exceedance), that exists for 2 hours or more.

Site characteristic values for the Maximum Safety Dry-Bulb and Coincident Wet-Bulb Temperatures, and the Maximum Safety Wet-Bulb Temperature (Non-Coincident) were estimated, as discussed below, using a conservative approach that reflects 100-year return intervals for these values.

The dry-bulb temperature component of the Maximum Safety Dry-Bulb and Coincident Wet-Bulb Temperature site characteristic pair is represented by the 100-year return period maximum dry-bulb value (i.e., 115°F) reported earlier. Because this 100-year return period dry-bulb value is extrapolated from a regression curve on a single parameter, there is no corresponding MCWB temperature. As a result, the coincident wet-bulb temperature component had to be derived based on a characteristic relationship between concurrent dry- and wet-bulb temperatures—that is, as dry-bulb temperature continues to increase, there is a point at which the concurrent wet-bulb temperature reaches a maximum and thereafter changes little or even decreases. This characteristic is not unique to this location or climatological setting.

This relationship is exhibited by the annual percent frequency distribution of wet-bulb temperature depression for the Augusta, Georgia, NWS station, as reported in the International Station Meteorological Climate Summary (Reference 232), over the 47-year period from 1949 through 1995. This type of summary is a bivariate distribution of dry-bulb temperatures in 2-degree ranges by wet-bulb depression (i.e., the difference between concurrent dry- and wet-bulb observations), also in 2-degree ranges.

For the Augusta NWS station, this threshold dry-bulb temperature occurs at about 85°F. A cubic polynomial curve was fit to the concurrent maximum dry-bulb and maximum wet-bulb temperature pairs extracted from this bivariate distribution at and above this threshold dry-bulb value. The equation of the curve is an estimation of the trend where the maximum coincident wet-bulb temperature can then be determined as a function of the maximum dry-bulb temperature in this upper range of dry-bulb values. Based on a 100-year return period maximum dry-bulb temperature of 115°F, the corresponding wet-bulb temperature is estimated to be 77.7°F. Therefore, this pair of values is used to represent the Maximum Safety Dry-Bulb and Coincident Wet-Bulb Temperature site characteristic values, respectively, for the VEGP Units 3 & 4 site.

The Maximum Safety Wet-Bulb Temperature (Non-Coincident) site characteristic value was developed in a manner similar to the previously reported 100-year return period maximum and minimum dry-bulb temperatures and the maximum wet-bulb temperature in that a regression equation was used to extrapolate the available data to that return interval. However, the wet-bulb temperature data were filtered to include only observed periods of persistence of two hours or more, consistent with the Westinghouse basis.

This persistence criterion introduced the constraint of only being able to analyze data sets with sequential hourly wet-bulb observations. As a result, the period of record utilized to estimate the Maximum Safety Wet-Bulb Temperature (Non-Coincident) associated with a 100-year return period was different than the 100-year return period maximum wet-bulb temperature reported above. A 30-year period of record from 1975 through 2005 (except 1980) for the Augusta NWS station was used to identify the maximum wet-bulb temperature for each year (References 215, 216, 217, 218, 220, 223, and 226).

When applied to the equation of the curve defined by these maximum yearly values, the wet-bulb temperature associated with a return period of 100 years was estimated to be 83.9°F. Therefore, this value is used to represent the Maximum Safety Wet-Bulb Temperature (Non-Coincident) site characteristic for the VEGP Units 3 & 4 site.

The AP1000 DCD maximum and minimum normal temperature site characteristics are 1-percent (99-percent) seasonal exceedance values. According to the ASHRAE 2001 Fundamentals Handbook, these are approximately equivalent to the annual 0.4-percent (99.6-percent) annual exceedance values. Thus, the maximum normal dry bulb temperature (1% seasonal exceedance) is 97° F with a coincident maximum normal wet bulb temperature of 76°F. The maximum normal non-coincident wet bulb temperature is 79°F. Additionally, the minimum normal dry bulb temperature (99% seasonal exceedance) is 21°F.

These values are summarized in Table 2.0-203, *Site Characteristics, Design Parameters, and Site Interface Values*.

2.3.1.6 Restrictive Dispersion Conditions

Atmospheric dispersion can be described as the horizontal and vertical transport and diffusion of pollutants released into the atmosphere. Horizontal and along-wind dispersion is controlled primarily by wind direction variation and wind speed. Subsection 2.3.2.2.1 addresses wind characteristics for the VEGP site vicinity based on measurements from the existing meteorological monitoring program at the VEGP site. The persistence of those wind conditions is also discussed in Subsection 2.3.2.2.1.

In general, lower wind speeds represent less turbulent air flow, which is restrictive to horizontal and vertical dispersion. And, although wind direction tends to be more variable under lower wind speed conditions (which increases horizontal transport), air parcels containing pollutants often re-circulate within a limited area, thereby increasing cumulative exposure.

Major air pollution episodes are usually related to the presence of stagnating high-pressure weather systems (or anti-cyclones) that influence a region with light and variable wind conditions for 4 days or more. An updated air stagnation climatology is available for the continental US based on over 50 years of observations from 1948 through 1998. Although inter-annual frequency varies, the data in Figures 1 and 2 of that report indicate that, on average, the VEGP site area can expect about 20 days per year with stagnation conditions, or about 4 cases per year with the mean duration of each case lasting about 5 days. (Reference 233)

Air stagnation conditions primarily occur during an “extended” summer season that runs from May through October. This is a result of the weaker pressure and temperature gradients, and therefore weaker wind circulations, during this period (as opposed to the winter season). Based on the *Air Stagnation Climatology for the United States (1948-1998)*, Figures 17 to 67, the highest incidence is recorded in the latter half of that period between August and October, typically reaching its peak in September. As the LCD summary for Augusta, Georgia, in Table 2.3-204 indicates, this 3-month period coincides with the lowest monthly mean wind speeds during the year. Within this “extended” summer season, air stagnation is at a relative minimum during July due to the influence of the Bermuda High pressure system. (Reference 233)

The mixing height (or depth) is defined as the height above the surface through which relatively vigorous vertical mixing takes place. Lower mixing heights (and wind speeds), therefore, are a relative indicator of more restrictive dispersion conditions. Holzworth (1972) reports mean seasonal and annual morning and afternoon mixing heights and wind speeds for the contiguous US based on observations over the 5-year period from 1960 to 1964. Out of the network of 62 NWS stations in the 48 contiguous US at which daily surface and upper air sounding measurements were routinely made, one station was located in Athens, Georgia, about 105 mi northwest of the VEGP site. The information in that report indicates that the results from that station should be reasonably representative of conditions at the VEGP site.

Table 2.3-206 summarizes the mean seasonal and annual morning and afternoon mixing heights and wind speeds for Athens, Georgia (Reference 209). From a climatological standpoint, considering all weather conditions, the lowest morning mixing heights occur in the autumn and are highest during the winter although, on average, morning mixing heights are only slightly lower in the spring and summer months than during the winter. Conversely, afternoon mixing heights reach a seasonal minimum in the winter and a maximum during the summer, as might be expected due to more intense summertime heating.

The wind speeds listed in Table 2.3-206 for Athens, Georgia, are consistent with the LCD summary for Augusta, Georgia, in Table 2.3-204 in that the lowest mean wind speeds are shown to occur during summer and autumn. This period of minimum wind speeds likewise coincides with the “extended” summer season described by Wang and Angell (1999) that is characterized by relatively higher air stagnation conditions.

2.3.1.7 Climate Changes

It is a given that climatic conditions change over time and that such changes are cyclical in nature on various time and spatial scales. The timing, magnitude, relative contributions to, and implications of these changes are generally more speculative, even more so for specific areas or locations.

With regard to the expected 40-year operating life for proposed VEGP Units 3 and 4, which could extend until the year 2070 based on a start-up year of 2030 (see Subsection 2.3.1.6), it is reasonable to evaluate the record of readily-available and well-documented climatological observations of temperature and rainfall (normals, means, and extremes) as they have varied over time (i.e., the last 60 to 70 years or so), and the occurrences of severe weather events, in the context of the plant’s design bases.

Trends of temperature and rainfall normals and standard deviations are identified over a 70-year period for successive 30-year intervals, updated every 10 years, beginning in 1931 (e.g., 1931–1960, 1941–1970, etc.) through the most recent normal period (i.e., 1971–2000) in the NCDC publication Climatography of the United States, No. 85 (Reference 214). The report summarizes these observations for the 344 climate divisions in the 48 contiguous states.

A climate division represents a region within a state that is as climatically homogeneous as possible. Division boundaries generally coincide with county boundaries except in the Western US. In Georgia, the VEGP site is located within Climate Division GA-06 (East Central). In South Carolina, Climate Division SC-05 (West Central), whose southern extent includes Aiken County, is nearly adjacent to the VEGP site.

Summaries of successive annual temperature and rainfall normals as well as the composite 70-year average are provided below for these climate divisions (Reference 214).

	Temperature (°F)		Rainfall (inches)	
Period	**GA-06**	**SC-05**	**GA-06**	**SC-05**
1931-2000	64.3	62.2	45.60	46.99
1931-1960	65.0	62.9	43.42	44.88
1941-1970	64.3	62.3	45.35	46.46
1951-1980	63.8	61.8	45.95	47.53
1961-1990	63.6	61.6	46.61	48.46
1971-2000	63.9	61.8	47.06	48.36

These data indicate a slight cooling trend over most of the 70-year period, with a slight increase of about 0.2 to 0.3°F during the most recent normal period. In general, total annual rainfall has increased slightly in these divisions over the period by about 1.5 inches. Similar trends are observable for all of the other climate divisions in Georgia and South Carolina (Reference 214).

The preceding values represent variations of “average” temperature and rainfall conditions over time. The occurrence of extreme temperature and precipitation (rainfall and snowfall) events does not necessarily follow the same trends. However, characteristics about the occurrence of such events over time are indicated by the summaries for observed extremes of temperature and rainfall and snowfall totals recorded in the VEGP site area (see Table 2.3-205).

The data summarized in Table 2.3-205 show that individual station records for maximum temperature have been set between 1952 (including the overall highest value for the site area) and 1999, i.e., there is no discernible trend for these extremes in the site area. Similarly, record-setting 24-hour rainfall totals were established between 1959 and 2000, with station records for total monthly rainfall between 1964 and 1995 – again, no clear trend. Cold air outbreaks that result in overall extreme low temperature records occur infrequently; record-setting snowfalls are even more rare events. The almost singular dates of their occurrence (in 1985 and 1973, respectively) are indicative of this characteristic. Nevertheless, records of these types for individual calendar days span a range of years similar to the maximum temperature, and the maximum 24-hour and monthly total rainfall records (Reference 230).

Characteristics and/or effects of other types of severe weather phenomena have been discussed previously, including tornadoes (see Subsection 2.3.1.3.2 and tropical cyclones (see Subsection 2.3.1.3.3).

The number of recorded tornado events has increased, in general, since detailed records were routinely documented beginning around 1950. However, some of this increase is attributable to a growing population, greater public awareness and interest, and technological advances in detection. These changes are superimposed on normal year-to-year variations. Consequently, the number of observations recorded within a 2-degree latitude and longitude square centered on the VEGP site reflect these effects.

As the frequency distribution in Subsection 2.3.1.3.2 indicates, the most intense tornado recorded in this study area was classified as an “F4” storm. The event occurred in 1973 and is the only tornado classified as such based on the nearly 55-year period of record evaluated. All of the tornadoes classified as “F3” storms (a total of 18) were recorded since 1972. Tornadoes with lower intensity classifications are much more numerous and have been identified throughout the available period of record (Reference 224).

The occurrence of all tropical cyclones within a 100-nautical mile radius of the VEGP site has been fairly steady since about 1950 when considered on a decadal (i.e., 10-year) basis or in terms of 30-year intervals similar to the “normal” periods used to evaluate temperature and rainfall data. Both the frequency and intensity of hurricanes passing within 100 nautical miles of the site have generally decreased over the available 154-year period of record, reaching a peak more than a hundred years ago around the turn of the last century. The frequency of tropical depressions has shown some increase in the last 30 years – storms of this classification have been associated with many of the 24-hour and monthly total rainfall records identified in Table 2.3-205 and discussed in Subsection 2.3.1.3.3 (Reference 228).

Nevertheless, the regulatory guidance for evaluating the climatological characteristics of a site from a design basis standpoint is not event specific, but rather is statistically based and for several parameters includes expected return periods of 100 years or more and probable maximum event concepts. These return periods exceed the design life of the proposed units. The design-basis characteristics determined previously under Subsection 2.3.1.3 are developed consistent with the intent of that guidance and incorporate the readily-available, historical data records for locations considered to be representative of the site for VEGP Units 3 and 4. These site characteristic values are summarized and compared in Table 2.0-203, *Site Characteristics, Design Parameters, and Site Interface Values*.

2.3.2 Local Meteorology

The potential influence of the construction and operation of VEGP Units 3 and 4 are evaluated using meteorological data representative of local conditions as described below.

2.3.2.1 Data Sources

The primary sources of data used to characterize local meteorological and climatological conditions representative of the VEGP site include summaries for the first-order NWS station at Augusta, Georgia (Bush Field) and nine other nearby cooperative network observing stations, and measurements from the existing VEGP onsite meteorological monitoring program. Table 2.3-203 identifies the offsite observing stations and provides the approximate distance and relative direction of each station to the VEGP site; their locations are shown in Figure 2.3-201. The onsite meteorological tower is located about 1 mi south-southwest of the Units 1 and 2 Containment Buildings and about 0.9 mi south of VEGP Units 3 and 4 as shown on Figure 1.1-202.

The NWS and cooperative observing station summaries were used to characterize climatological normals, period-of-record means, and extremes of temperature, rainfall, and snowfall in the vicinity of the VEGP site. In addition, first-order NWS stations also record measurements, typically on an hourly basis, of other weather elements, including winds, relative humidity, dew point, and wet-bulb temperatures, as well as other observations (e.g., fog, thunderstorms). This information was based on the following resources:

- *2004 Local Climatological Data, Annual Summary with Comparative Data for Augusta, Georgia* (Reference 221)
- *Climatography of the United States, No. 20, 1971-2000, Monthly Station Climate Summaries* (Reference 222)
- *Climatography of the United States, No. 81, 1971-2000, U.S. Monthly Climate Normals* (Reference 211)
- *SERCC, Historical Climate Summaries and Normals for the Southeast* (Reference 230)
- *Cooperative Summary of the Day, TD3200, Period of Record through 2001 for the Eastern United States, Puerto Rico and the Virgin Islands* (Reference 213)

Wind speed, wind direction, and atmospheric stability data based on the VEGP meteorological monitoring program form the basis for determining and characterizing atmospheric dispersion conditions in the vicinity of the site. These data include measurements taken over the 5-year period of record from 1998 through 2002.

2.3.2.2 Normal, Mean, and Extreme Values of Meteorological Parameters

Historical extremes of temperature, rainfall, and snowfall are listed in Table 2.3-205 for the 10 NWS and cooperative observing stations in the VEGP site area. The normals, means, and extremes of the more extensive set of measurements and observations made at the Augusta NWS Station are summarized in Table 2.3-204. Finally, Table 2.3-207 compares the annual normal (i.e., 30-year average) daily maximum, minimum, and mean temperatures, as well as the normal annual rainfall and snowfall totals for these stations.

2.3.2.2.1 Wind

Average Wind Direction and Wind Speed Conditions

The distribution of wind direction and wind speed is an important consideration when characterizing the dispersion climatology of a site. Long-term average wind motions at the macro- and synoptic scales (i.e., on the order of several thousand down to several hundred kilometers) are influenced by the general circulation patterns of the atmosphere at the macro-scale and by large-scale topographic features (e.g., mountain ranges, land-water interfaces such as coastal areas). These characteristics are addressed in Subsection 2.3.1.2.

Site-specific or micro-scale (i.e., on the order of 2 km or less) wind conditions, while reflecting these larger-scale circulation effects, are influenced primarily by local and, to a lesser extent (generally), by meso- or regional-scale (i.e., up to about 200 km) topographic features. Wind measurements at these smaller scales are available from the existing meteorological monitoring program at the VEGP site and from data recorded at the nearby Augusta NWS Station.

Subsection 2.3.3 provides a summary description of the onsite meteorological monitoring program at the VEGP site. In its current configuration, wind direction and wind speed measurements are made at two levels on an instrumented 60-m tower (i.e., the lower level at 10 m and the upper level at 60 m).

Figures 2.3-202 through 2.3-206 present annual and seasonal wind rose plots (i.e., graphical distributions of the direction from which the wind is blowing and wind speeds for each of sixteen 22.5-degree compass sectors centered on north, north-northeast, northeast, etc.) for the 10-m level based on measurements at the VEGP site over the composite 5-year period from 1998 through 2002.

For the VEGP site, the wind direction distribution at the 10-m level generally follows a southwest-northeast orientation on an annual basis (see Figure 2.3-202). The prevailing wind (i.e., defined as the direction from which the wind blows most often) is from the southwest, with nearly 25 percent of the winds blowing from the southwest through west sectors. Conversely, winds from the northeast through east sectors occur about 20 percent of the time. On a seasonal basis, winds from the southwest quadrant predominate during the spring and summer months (see Figures 2.3-204 and 2.3-205). This is also the case during the winter, although westerly winds prevail and the relative frequency of west-northwest winds during this season is greater (see Figure 2.3-203) due to increased cold frontal passages. Winds from the northeast quadrant predominate during the autumn months (see Figure 2.3-206). Plots of individual monthly wind roses at the 10-m measurement level are presented in Figure 2.3-207 (Sheets 1 to 12).

Wind rose plots based on measurements at the 60-m level are shown in Figures 2.3-208 through 2.3-213. By comparison, wind direction distributions for the 60-m level are fairly similar to the 10-m level wind roses on a composite annual (see Figure 2.3-208) and seasonal basis (see Figures 2.3-209 through 2.3-212). Plots of individual monthly wind roses at the 60-m measurement level are presented in Figure 2.3-213 (Sheets 1 to 12).

Wind information summarized in the LCD for the Augusta NWS Station (see Table 2.3-204) indicates a prevailing west-southwesterly wind direction (Reference 221) that appears to be similar to the 10-m level wind flow at the VEGP site, at least on an annual basis (see Figure 2.3-202).

Table 2.3-208 summarizes seasonal and annual mean wind speeds based on measurements from the upper and lower levels of the existing VEGP site meteorological tower (1998–2002) and from wind instrumentation at the Augusta NWS Station (1971–2000 station normals) (Reference 221). The elevation of the wind instruments at the Augusta NWS Station is nominally 20 ft (about 6.1 m) (Reference 221), comparable to the lower (10-m) level measurements at the VEGP site.

On an annual basis, mean wind speeds at the 10- and 60-m levels are 2.5 m/sec and 4.6 m/sec, respectively, at the VEGP site. The annual mean wind speed at Augusta (i.e., 2.7 m/sec) is similar to the 10-m level at the VEGP site, differing by only 0.2 m/sec; seasonal average wind speeds at Augusta are likewise slightly higher. Seasonal mean wind speeds for both measurement levels at the VEGP site follow the same pattern discussed in Subsection 2.3.1.6 for Augusta and Athens, Georgia, and their relationship to the seasonal variation of relatively higher air stagnation and restrictive dispersion conditions in the site region.

Based on the joint frequency distributions of wind speed and wind direction by atmospheric stability class (see Subsection 2.3.2.2.2), the annual frequencies of calm wind conditions are 0.35 and 0.05 percent of the time for the 10-m and 60-m tower levels, respectively, at the VEGP site.

Wind Direction Persistence

Wind direction persistence is a relative indicator of the duration of atmospheric transport from a specific sector-width to a corresponding downwind sector-width that is 180 degrees opposite. Atmospheric dilution is directly proportional to the wind speed (other factors remaining constant). When combined with wind speed, a wind direction persistence/wind speed distribution further indicates the downwind sectors with relatively more or less dilution potential (i.e., higher or lower wind speeds, respectively) associated with a given transport wind direction.

Tables 2.3-207 and 2.3-208 present wind direction persistence/wind speed distributions based on measurements at the VEGP site for the 5-year period of record from 1998 through 2002. The distributions account for durations ranging from 1 to 48 hours for wind directions from 22.5-degree and 67.5-degree upwind sectors centered on each of the 16 standard compass radials (i.e., north, north-northeast, northeast, etc.). Further, the distributions are provided for wind measurements made at the lower (10-m) and the upper (60-m) tower levels, respectively.

2.3.2.2.2 Atmospheric Stability

Atmospheric stability is a relative indicator for the potential diffusion of pollutants released into the ambient air. Atmospheric stability, as discussed in this SSAR, is determined by the delta-temperature (Δ**T**) method as defined in Table 1 of Proposed Revision 1 to Regulatory Guide 1.23, *Meteorological Programs in Support of Nuclear Power Plants,* September 1980.

The approach classifies stability based on the temperature change with height (i.e., the difference in °C per 100 m). Stability classifications are assigned according to the following criteria:

- Extremely Unstable (Class A) —$\Delta T/\Delta Z \leq$ -1.9°C
- Moderately Unstable (Class B) — -1.9°C < $\Delta T/\Delta Z \leq$ -1.7°C
- Slightly Unstable (Class C) — -1.7°C < $\Delta T/\Delta Z \leq$ -1.5°C
- Neutral Stability (Class D) — -1.5°C < $\Delta T/\Delta Z \leq$ -0.5°C
- Slightly Stable (Class E) — -0.5°C < $\Delta T/\Delta Z \leq$ +1.5°C
- Moderately Stable (Class F) — +1.5°C < $\Delta T/\Delta Z \leq$ +4.0°C
- Extremely Stable (Class G) — +4.0°C < $\Delta T/\Delta Z$

The diffusion capacity is greatest for extremely unstable conditions and decreases progressively through the remaining unstable, neutral stability, and stable classifications.

During the 1998 through 2002 time period at the VEGP site, Δ**T** was determined from the difference between temperature measurements made at the 10-m and 60-m tower levels. Seasonal and annual frequencies of atmospheric stability class and associated 10-m level mean wind speeds for this period of record are presented in Table 2.3-211.

The data indicate a predominance of slightly stable (Class E) and neutral stability (Class D) conditions, ranging from about 50 to 60 percent of the time on a seasonal and annual basis. Extremely unstable conditions (Class A) are more frequent during the spring and summer months due to greater solar insolation. Extremely stable conditions (Class G) are most frequent during the fall and winter months, owing in part to increased radiational cooling at night.

Joint frequency distributions (JFDs) of wind speed and wind direction by atmospheric stability class and for all stability classes combined for the 10-m and 60-m wind measurement levels at the VEGP site are presented in Tables 2.3-210 and 2.3-211, respectively, for the 5-year period of record from 1998 through 2002. The 10-m level JFDs are used to evaluate short-term dispersion estimates for accidental atmospheric releases (see Subsection 2.3.4) and long-term diffusion estimates of routine releases (see Subsection 2.3.5).

2.3.2.2.3 Temperature

Extreme maximum temperatures recorded in the vicinity of the VEGP site have ranged from 105°F to 112°F, with the highest reading observed at the Louisville 1E Station on July 24, 1952. The station record high temperature for the Midville Experiment Station (i.e., 105°F) has been reached on four separate occasions. As Table 2.3-205 shows, individual station extreme maximum temperature records were set at multiple locations on the same or adjacent dates (i.e., Waynesboro 2NE, Louisville 1E, and Millen 4N; Augusta, Midville Experiment Station, and Aiken 4NE; and Waynesboro 2NE, Midville Experiment Station, and Newington 2NE) (Reference 222; Reference 230).

Extreme minimum temperatures in the vicinity of the VEGP site have ranged from 2°F to -4°F, with the lowest reading on record observed at the Aiken 4NE Station on January 21, 1985, the same date on which the record low temperature was set at the nine other nearby stations (Reference 222; Reference 230).

The extreme maximum and minimum temperature data indicate that synoptic-scale conditions responsible for periods of record-setting excessive heat as well as significant cold air outbreaks tend to affect the overall VEGP site area. The similarity of the respective extremes suggests that these statistics are reasonably representative of the temperature extremes that might be expected to be observed at the VEGP site.

Daily mean temperatures (which are based on the average of the daily mean maximum and minimum temperature values) for these stations are similar, ranging from 63.1°F at Waynesboro 2NE to 65.0°F at the Midville Experiment Station (Reference 211). Likewise, the diurnal (day-to-night) temperature ranges, as indicated by the differences between the daily mean maximum and minimum temperatures, are fairly comparable, ranging from 21.9°F at Bamberg to 26.3°F at Aiken 4NE (Reference 211).

2.3.2.2.4 Water Vapor

Based on a 49-year period of record, the LCD summary for the Augusta, Georgia NWS Station (see Table 2.3-204) indicates that the mean annual wet-bulb temperature is 56.7°F, with a seasonal maximum during the summer months (June through August) and a seasonal minimum during the winter months (December through February). The highest monthly mean wet-bulb temperature is 72.7°F in July (only slightly less during August); the lowest monthly mean value (40.3°F) occurs during January. (Reference 221) Wet-bulb temperature characteristics are addressed in Subsection 2.3.1.5 from a design-basis standpoint.

The LCD summary shows a mean annual dew point temperature of 51.9°F, also reaching its seasonal maximum and minimum during the summer and winter, respectively. The highest monthly mean dew point temperature is 69.7°F in July; again, only slightly less during August. The lowest monthly mean dew point temperature (34.4°F) occurs during January. (Reference 221)

The 30-year normal daily relative humidity averages 72 percent on an annual basis, typically reaching its diurnal maximum in the early morning (around 0700 hours) and its diurnal minimum during the early afternoon (around 1300 hours). There is less variability in this day-to-night pattern with the passage of weather systems, persistent cloud cover, and precipitation. Nevertheless, this diurnal pattern is evident throughout the year. The LCD summary shows that average early morning relative humidity levels exceed 90 percent during August, September, and October. (Reference 221)

2.3.2.2.5 Precipitation

With the exception of the Aiken 4NE Station, normal annual rainfall totals are similar for the nine other nearby observing stations listed in Table 2.3-207, differing by only about 4.7 in. (or about 10 percent) and ranging from 43.85 to 48.57 in. The current 30-year average for the Aiken 4NE Station is somewhat higher at 52.43 in. Snowfall is an infrequent occurrence, as discussed in Subsection 2.3.1, with normal annual totals of only 0.1 to 1.4 in. (References 211, 222; 230).

2.3.2.2.6 Fog

The closest station to the VEGP site at which observations of fog are made and routinely recorded is the Augusta NWS Station about 20 mi to the northwest. The 2004 LCD summary for this station (Table 2.3-204) indicates an average of 35.1 days per year of heavy fog conditions based on a 54-year period of record. The NWS defines heavy fog as fog that reduces visibility to 1/4 mi or less.

The frequency of fog conditions at the VEGP site would be expected to be similar to that of Augusta because of their proximity to one another and because of the similarity of topographic features at both locations (i.e., gently rolling terrain, adjacent to the Savannah River, and location within that broad river valley).

2.3.2.3 Potential Influence of the Plant and Related Facilities on Meteorology

The dimensions and operating characteristics of VEGP Units 3 and 4 and existing VEGP Units 1 and 2 facilities and the associated paved, concrete, or other improved surfaces are considered to be insufficient to generate discernible, long-term effects to local- or micro-scale meteorological conditions.

Wind flow may be altered in areas immediately adjacent to and downwind of larger site structures. However, these effects will likely dissipate within ten structure heights downwind of the intervening structure(s). Similarly, while ambient temperatures immediately above any improved surfaces could increase, these temperature effects will be too limited in their vertical profile and horizontal extent to alter local- or regional-scale ambient temperature patterns.

Units 1 and 2 at the VEGP site use two 550-ft-high natural-draft cooling towers as a means of heat dissipation. Depending on local meteorological conditions, plume rise ranges from 500 to 1,000 ft above those 550-ft-high towers. Because of the elevated release point and plume rise, there is minimal effect on local meteorology or the plant.

Two 600-ft-high natural-draft cooling towers provide cooling for VEGP Units 3 and 4. Because the release height of the thermal/water vapor plumes from these cooling towers is even higher than that of the VEGP Units 1 and 2 cooling towers, minimal effect on local meteorology or the plant is expected.

While there is excavation, landscaping, site leveling, and clearing associated with the construction of the new units, these alterations to the site terrain would be localized and would not represent a significant alteration to the flat-to-gently-rolling topographic character of the area and region around the site. Therefore, the overall meteorological characteristics of the site will not be affected.

2.3.2.4 Current and Projected Site Air Quality

The VEGP site is located within the Augusta (Georgia) – Aiken (South Carolina) Interstate Air Quality Control Region (40 CFR 81.114). The counties within this region are designated as being in attainment or unclassified for all criteria air pollutants (40 CFR 81.311; 40 CFR 81.341). Attainment areas are areas where the ambient air quality levels are better than the EPA-promulgated National Ambient Air Quality Standards (NAAQS). Criteria pollutants are those for which NAAQS have been established: sulfur dioxide, particulate matter (i.e., PM_{10} and $PM_{2.5}$ – particles with nominal aerodynamic diameters less than or equal to 10.0 and 2.5 micons, respectively), carbon monoxide, nitrogen dioxide, ozone, and lead (40 CFR Part 50).

Four pristine areas in the States of Georgia and South Carolina are designated as “Mandatory Class I Federal Areas Where Visibility is an Important Value.” They include the Cohutta Wilderness Area, the Okefenokee Wilderness Area, and the Wolf Island Wilderness Area in Georgia (40 CFR 81.408), and the Cape Romain Wilderness Area in South Carolina (40 CFR 81.426). The two closest of these Class I areas are both about 130 mi away from the VEGP site—the Wolf Island Wilderness Area to the south-southeast and the Cape Romain Wilderness Area to the east-southeast.

The nuclear steam supply system and other related radiological systems are not sources of criteria pollutants or other air toxics. Supporting equipment (e.g., diesel generators, fire pump engines) and other non-radiological emission-generating sources (e.g., storage tanks and related equipment) or activities are not a significant source of criteria pollutant emissions.

Emergency equipment is only operated on an intermittent test or emergency-use basis. Therefore, these emission sources are not expected to significantly impact ambient air quality levels in the vicinity of the VEGP site, nor be a significant factor in the design and operating bases of VEGP Units 3 and 4. Likewise, because of the relatively long distance of separation from the VEGP site, visibility at any of these Class I Federal Areas are not significantly impacted by project construction and facility operations.

Nevertheless, these non-radiological emission sources are regulated by the Georgia Department of Natural Resources (DNR) under the Georgia Rules for Air Quality Control (Chapter 391-3-1) and permitted under the State’s Title V Operating Permit Program implemented by the Georgia DNR pursuant to 40 CFR Part 70 either as a separate facility or via a revision to the then current Title V Operating Permit for the existing VEGP site.

2.3.2.5 Topographic Description

The VEGP site (approximately 3,169 acres) is located in Burke County, Georgia, along (west of) the Savannah River. Topographic features within a 5-mi radius of the VEGP site are shown in Figure 2.3-214. Terrain elevation profiles along each of the 16 standard 22.5-degree compass radials out to a distance of 50 mi from the VEGP site are illustrated in Figure 2.3-215 (Sheets 1 through 4).

These profiles indicate that the terrain in the VEGP site area is flat to gently rolling. The only other nearby topographic feature of note is the Savannah River, located adjacent to the VEGP site; the broad river valley represents a depression running northwest to southeast.

2.3.3 Onsite Meteorological Measurements Program

2.3.3.1 Onsite Meteorological Measurements Program

SNC uses measurement data from the VEGP onsite meteorological monitoring program to support operation of VEGP Units 3 and 4.

2.3.3.2 General Program Description

The VEGP onsite meteorological measurements program commenced operation in April 1972. Instruments for measuring pertinent meteorological parameters were installed on a 45-m tower located in a cleared area at site coordinates N 3260 and E 8040. This location is about 3,840 ft (1,170 m) south of the 775-ft-radius circle that encloses the VEGP Units 3 and 4 power block area (see Figure 1.1-202 for general location). The base of the tower is at approximately plant grade.

The onsite meteorological measurements program and equipment were updated in the first quarter of 1984 to meet the intent of NUREG-0654 (*Criteria for Preparation and Evaluation of Radiological Emergency Response Plans and Preparedness in Support of Nuclear Power Plants,* FEMA-REP-1, Revision 1, November 1980). A new meteorological data collection center (MDCC) included a 60-m tower located at site coordinates N 3100 and E 7940 with permanent instrumentation at the 10- and 60-m elevations. The 60-m tower is located about 3,960 ft (1,207 m) south of the 775-ft-radius circle that encloses the VEGP Units 3 and 4 power block area (see Figure 1.1-202 for general location). A 2-kVA uninterruptible power supply was also installed to prevent the loss of meteorological data collection in the event that offsite power is interrupted.

The onsite meteorological measurements program and equipment were upgraded in the second quarter of 2015 to replace obsolete equipment and to improve data recovery values. The upgrade replaced the 60-m tower's existing 10-m and 60-m instruments with redundant instruments providing primary and secondary (backup) data. The use of the backup 45-m tower and its instruments was discontinued. Table 2.3-214 presents instrument descriptions and accuracies for the meteorological monitoring systems. Measurement system accuracies are in conformance with Regulatory Guide 1.23.

The instruments are monitored at least once a week by SNC personnel. Preventive maintenance is performed by SNC personnel in accordance with the instrument manuals and is intended to maintain 90 percent data recovery.

Data collection for the MDCC consists of redundant data loggers and workstations, both located in the meteorological tower equipment building. These data are transmitted via redundant fiber optic cables. The fiber optic cables provide instrument data to the Control Rooms for Units 1, 2, 3 and 4, Technical Support Center, and Emergency Operations Facility. The collected data are compiled in accordance with Regulatory Guide 1.23 and are summarized and edited to provide averages representative of each hour of measurements.

The annual and/or seasonal summaries of onsite meteorological data presented in this section are based on hourly-averaged measurements from instrumentation mounted on the 60-m tower taken over the 5-year period of record from 1998 through 2002. These data were used to determine the wind roses and joint frequency distributions of wind speed and wind direction by atmospheric stability class presented and discussed in Subsection 2.3.2.

A year-by-year summary of the percent data recoveries for each parameter is shown in Table 2.3-215. Composite data recoveries of 94 percent or greater were achieved in each of those 5 years for the dispersion modeling-related parameters of wind speed and wind direction from the 10-m and 60-m levels, and vertical stability based on the delta-temperature between the 60-m and 10-m levels. The only parameters with annual data recoveries less than the 90 percent target recovery level are dew point temperature (i.e., 89.6 percent) and rainfall (i.e., 78.8 percent) during 2002.

2.3.3.3 Location, Elevation, and Exposure of Instruments

The general location of the meteorological tower is shown in Figure 1.1-202.

The nearest major structures are VEGP Units 3 and 4 reactors and their associated natural-draft cooling towers located, respectively, about 4,525 ft (mid-point between the two units) and about 3,025 ft (closest point on the Unit 3 cooling tower) to the north of the meteorological tower. Regulatory Guide 1.23 indicates that a meteorological tower located at 10-building-heights horizontal distance downwind will not have adverse building wake effects exerted by the structure. Since the height of the AP1000 units is about 234 ft above grade, the zone of turbulent flow created by the reactor buildings is limited to about 2,340 ft (or 10 building heights) downwind. Thus, the reactors do not adversely affect the measurements taken at the meteorological tower.

The 10-building-height distance of separation guidance is usually applied to square- or rectangular-shaped structures or objects. A round structure will produce a downwind wake zone that is shorter than a square or rectangular structure or object. The downwind region of adverse influence of a hyperbolically-shaped, natural-draft cooling tower is estimated to be about five times the width of the tower at the top of the structure (Reference 207).

The natural-draft cooling towers are about 600 ft high, with a base diameter of 550 ft, and a diameter of 330 ft at the top. Based on the EPA guidance for this type of structure and the diameter at its top, the outermost boundary of influence that is exerted by the cooling towers is estimated to be about 1,650 ft. This distance is much shorter than the physical separation of the cooling towers from the meteorological tower (i.e., about 3,025 ft). Therefore, the natural-draft cooling towers do not adversely affect measurements made at the meteorological tower. Similarly, minor structures in the vicinity of the meteorological tower have been evaluated as having no adverse effect on the measurements taken at that tower.

2.3.3.4 VEGP Meteorological Monitoring Program Compliance

The meteorological monitoring program operated in support of VEGP Units 1 and 2 will also support the operation of VEGP Units 3 and 4. Characteristics of this monitoring program, include:

- siting of the meteorological tower with respect to potential obstructions to air flow (e.g., containment structures, cooling towers, tree lines),
- descriptions of the meteorological instrumentation (e.g., performance specifications, methods and equipment for recording sensor output, QA program for sensors and recorders, and data acquisition and reduction procedures), and
- operation, maintenance, and calibration procedures.

The NRC evaluated the meteorological monitoring program as part of the ESPA SSAR safety evaluation site audit on December 6, 2006 and through their review of Subsection 2.3.3.

The current monitoring program and its implementation were determined to meet the guidance in Proposed Revision 1 to Regulatory Guide 1.23 and found to provide an acceptable basis for estimating atmospheric dispersion conditions for accidental and routine releases of radioactive material to the atmosphere.

2.3.4 Short-Term (Accident) Diffusion Estimates

In the absence of a specific site for use in determining values for short-term diffusion, a study was performed to determine the atmospheric dispersion factors (χ/Q values) that would envelope most current plant sites and that could be used to calculate the radiological consequences of design basis accidents. The χ/Q values thus derived for offsite are provided in Table 2.0-201.

This set of offsite χ/Q values is representative of potential sites for construction of the AP1000. The values are appropriate for analyses to determine the radiological consequences of accidents. These values were selected to bound 70 to 80 percent of U.S. sites.

The χ/Q values for the control room air intake or the door leading to the control room are dependent not only on the site meteorology but also on the plant design and layout. These χ/Q values are addressed in Appendix 15A. Separate sets of χ/Q values are identified for each combination of activity release location and receptor location.

This subsection addresses the determination of conservative, short-term atmospheric dispersion estimates due to postulated design-basis, accidental releases of radioactive material to the ambient air for receptors located:

- on the Exclusion Area Boundary (EAB) and the outer boundary of the Low Population Zone (LPZ) (Subsections 2.3.4.1 and 2.3.4.2) to support the evaluation of offsite radiological consequences; and
- at air intake points to the control room (Subsection 2.3.4.3) to support the evaluation of personnel exposures inside the control room and the design of the control room habitability system.

This subsection also briefly addresses the determination of accident-related concentrations at the control room due to onsite and/or offsite airborne releases of hazardous materials such as flammable vapor clouds, toxic chemicals, and smoke from fires (Subsection 2.3.4.4).

In the AP1000 reactor DCD, the terms “site boundary” and “exclusion area boundary” are used interchangeably. Thus, the χ/Q value specified for the site boundary applies whenever a discussion in the DCD refers to the exclusion area boundary. In the Subsections 2.3.4.1 and 2.3.4.2 site specificχ/Q calculations, the term “Dose Calculation EAB” is equivalent to the DCD term “EAB”.

Short-term, dispersion-related site parameters at the site boundary and the LPZ boundary, on which the AP1000 design is based, are identified in DCD Tier 1, Table 5.0-1, Table 2.0-201, and Table 15A-5. As indicated above, site-specific dispersion characteristics that correspond to these site parameters are presented in Subsections 2.3.4.1 and 2.3.4.2.

Short-term, dispersion-related site parameters at the control room, also incorporated in the AP1000 design, are identified in DCD Tier 1, Table 5.0-1, Table 2.0-201, and Table 15A-6. Site-specific dispersion characteristics that correspond to these site parameters are presented in Subsection 2.3.4.3.

Tables 2.0-201 and 2.0-202 compare the applicable site parameters and corresponding site-specific characteristic values.

2.3.4.1 Basis

To evaluate potential health effects for Westinghouse AP1000 design-basis accidents, a hypothetical accident is postulated to predict upper-limit concentrations and doses that might occur in the event of a containment release to the atmosphere.

Regulatory Guide 4.7, *General Site Suitability Criteria for Nuclear Power Stations*, Revision 2, April 1998, states that for site approval, each applicant should collect at least 1 year of meteorological information that is representative of the site conditions for calculating radiation doses resulting from the release of fission products as a consequence of a postulated accident. Site-specific meteorological data covering the 5-year period of record from 1998 through 2002 (see Subsection 2.3.2.2.2) have been used to quantitatively evaluate such a hypothetical accident at the VEGP site. Onsite data provide representative measurements of local dispersion conditions appropriate to the VEGP site and a 5-year period is considered to be reasonably representative of long-term conditions.

According to 10 CFR Part 100, it is necessary to consider the doses for various time periods immediately following the onset of a postulated containment release at the exclusion distance and for the duration of exposure for the low population zone and population center distances. The relative air concentrations (χ/Qs) are estimated for various time periods ranging from 2 hours to 30 days.

Meteorological data have been used to determine various postulated accident conditions as specified in Regulatory Guide 1.145, *Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants*, Revision 1, November 1982 (Re-issued February 1983). Compared to an elevated release, a ground-level release usually results in higher ground-level concentrations at downwind receptors due to less dilution from shorter traveling distances. Since the ground-level release scenario provides a bounding case, elevated releases are not considered in this ESP application.

The NRC-sponsored PAVAN computer code (NUREG/CR-2858, *PAVAN: An Atmospheric Dispersion Program for Evaluating Design Basis Accidental Releases of Radioactive Materials from Nuclear Power Stations*, PNL-4413, November 1982 [NUREG/CR-2858]) has been used to estimate ground-level χ/Qs at the Exclusion Area Boundary (EAB) and Low Population Zone (LPZ) for potential accidental releases of radioactive material to the atmosphere. Such an assessment is required by 10 CFR Part 100.

As discussed in Subsection 2.1.1.3, the EAB for VEGP Units 3 and 4 is the same as the exclusion area for the existing VEGP units. For the purposes of determining χ/Qs and subsequent radiation dose analyses, an effective EAB, hereafter referred to as the Dose Calculation EAB, was developed for the proposed units. The AP1000 units will be located within the power block area, shown in Figure 1.1-202, which is the perimeter of a 775-ft-radius circle with the centroid at a point between the two AP1000 units. The Dose Calculation EAB is a circle that extends 1/2 mi beyond the power block area (i.e., a circle with a 3,415-ft radius with its centroid at the centroid of the power block circle). The Dose Calculation EAB is completely within the actual plant EAB and, thus, the χ/Qs and the subsequent radiation doses are conservatively higher.

The PAVAN program implements the guidance provided in Regulatory Guide 1.145. Mainly, the code computes χ/Qs at the EAB and LPZ for each combination of wind speed and atmospheric stability class for each of 16 downwind direction sectors (i.e., north, north-northeast, northeast, etc.). The χ/Q values calculated for each direction sector are then ranked in descending order, and an associated cumulative frequency distribution is derived based on the frequency distribution of wind speeds and stabilities for the complementary upwind direction sector. The χ/Q value that is equaled or exceeded 0.5 percent of the total time becomes the maximum sector-dependent χ/Q value.

The χ/Q values calculated above are also ranked independently of wind direction into a cumulative frequency distribution for the entire site. The PAVAN program then selects the χ/Qs that are equaled or exceeded 5 percent of the total time.

The larger of the two values (i.e., the maximum sector-dependent 0.5 percent χ/Q or the overall site 5 percent χ/Q value) is used to represent the χ/Q value for a 0- to 2-hour time period. To determine χ/Qs for longer time periods, the program calculates an annual average χ/Q value using the procedure described in Regulatory Guide 1.111, *Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors*, Revision 1, July 1977. The program then uses logarithmic interpolation between the 0- to 2-hour χ/Qs for each sector and the corresponding annual average χ/Qs to calculate the values for intermediate time periods (i.e., 8 hours, 16 hours, 72 hours, and 624 hours). As suggested in NUREG/CR-2858, each of the sector-specific 0- to 2-hour χ/Qs provided in the PAVAN output file has been examined for “reasonability” by comparing them with the ordered χ/Qs also presented in the model output.

The PAVAN model has been configured to calculate offsite χ/Q values assuming both wake-credit allowed and wake-credit not allowed. The entire Dose Calculation EAB is located beyond the wake influence zone induced by the Reactor Building. And, because the LPZ is located farther away from the plant site than the Dose Calculation EAB (i.e., a 2-mi-radius [3,218 m] circle centered at the midpoint of the existing reactors bounds the LPZ), the “wake-credit not allowed” scenario of the PAVAN results has been used for the χ/Q analyses at both the Dose Calculation EAB and the LPZ.

The PAVAN model input data are presented below:

- Meteorological data: 5-year (January 1, 1998 to December 31, 2002) composite onsite JFD of wind speed, wind direction, and atmospheric stability.
- Type of release: Ground-level.
- Wind sensor height: 10 m.
- Vertical temperature difference: (10 m-60 m).
- Number of wind speed categories: 11.
- Release height: 10 m (default height).
- Distances from release point to Dose Calculation EAB: 800 m, for all downwind sectors.
- Distances from release point to LPZ: 2,304 m, for all downwind sectors.

The PAVAN model uses building cross-sectional area and containment height to estimate wake-related χ/Q values. Since the Dose Calculation EAB and the LPZ are both located beyond the building wake influence zone, these two input parameters have no effect in calculating the non-wake χ/Q values.

To be conservative, the 1/2 mi (or approximately 800 m) distance between the VEGP Units 3 and 4 power block area circle and the Dose Calculation EAB has been entered as input for each downwind sector to calculate the χ/Q values at the Dose Calculation EAB. Similarly, the shortest distance from the power block area circle to the LPZ has been input for all direction sectors to calculate the χ/Q values at the LPZ. The distance from the center-point of the existing units to the western perimeter of the power block area is about 914 m. Therefore, the minimum distance from the power block area circle to the LPZ is about 2,304 m (or about 1.4 mi).

2.3.4.2 PAVAN Modeling Results

As presented in Table 2.3-216, the maximum 0- to 2-hour, 0.5 percentile, direction-dependent χ/Q value (3.14 x 10^{-4} sec/m^3) is less than the corresponding 5 percentile overall site χ/Q value (3.49 x 10^{-4} sec/m^3) at the Dose Calculation EAB. Therefore, the 5 percentile overall site χ/Qs should be used as the proper χ/Qs at the Dose Calculation EAB.

Similarly, Table 2.3-217 shows that the maximum 0- to 2-hour, 0.5 percentile, direction-dependent χ/Q value (1.17 x 10^{-4} sec/m^3) is less than the corresponding 5 percentile overall site χ/Q value (1.27 x 10^{-4} sec/m^3) at the LPZ. Therefore, the 5 percentile overall site χ/Qs should be used as the proper χ/Qs at the LPZ.

The maximum χ/Qs presented in Tables 2.3-214 and 2.3-215 for the Dose Calculation EAB and the LPZ, respectively, are summarized below for the 0- to 2-hour time period, the annual average time period, and other intermediate time intervals evaluated by the PAVAN model.

Summary of PAVAN χ/Q Results (5% Limiting Case), 1998–2002 Meteorological Data

Source Location	Receptor Location	0-2 hr (Dir, Dist)	0-8 hr (Dir, Dist)	8-24 hr (Dir, Dist)	1-4 days (Dir, Dist)	4-30 days (Dir, Dist)	Annual (Dir, Dist)
ESP PBAC[(a)]	Dose Calculation EAB	3.49E-04	2.41E-04	2.00E-04	1.34E-04	7.56E-05	3.74E-05
ESP PBAC[a]	LPZ	1.27E-04 [(b)]	7.04E-05	5.25E-05	2.77E-05	1.11E-05	3.63E-06

(a) PBAC = Power Block Area Circle
(b) The 0-2 hour χ/Q values are reported here for reference only (not required based on Regulatory Guide 1.145).

Using the same assumptions and methodology as described in Subsection 2.3.4.1 (which relied on DCD Revision 15), the short-term (accidental release) dispersion estimates at the EAB and the LPZ boundary were evaluated using the revised building dimensions provided in DCD Revision 17. That evaluation confirmed that the χ/Q values for the EAB and LPZ remain the same. This result is reasonable given that the designated receptor points at the EAB and the LPZ boundary are beyond the distance that would be influenced by building wake.

2.3.4.3 Radiological Accident Dispersion Estimates at the Control Room

Subsection 2.3.4.3.1 describes the dispersion modeling analysis used to determine short-term, relative concentration estimates associated with a postulated design-basis, accidental release of radioactive material to the atmosphere. The results of this dispersion analysis for receptors at air intake points to the control room are summarized in Subsection 2.3.4.3.2.

2.3.4.3.1 Regulatory Basis and Technical Approach

General Design Criterion 19 (*Control Room*) under 10 CFR Part 50, Appendix A, requires that the control room remain functional so that actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain the plant in a safe state under accident conditions.

Regulatory Guide 1.194, *Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants*, June 2003, provides guidance on utilizing the ARCON96 dispersion model to characterize atmospheric dispersion conditions (χ/Q values) that are input to the evaluation of the consequences of accidental airborne radiological releases on control room habitability. The ARCON96 dispersion model is described in NUREG/CR-6331 (*Atmospheric Relative Concentrations in Building Wakes*, PNNL-10521, Revision 1, May 1997). [Reference 201]

Five consecutive calendar years (from 1998 through 2002) of sequential hourly meteorological data, from the onsite monitoring program operated in support of VEGP Units 1 & 2, were input to ARCON96 in model-required format. As such, the estimated χ/Q values represent the composite 5-year period of record. Wind data from both the 10- and 60-m measurement levels were included. Wind speed units of measure were in meters per second.

Joint data recovery of atmospheric stability class and 10-m level wind speed and wind direction was greater than 94 percent for each of the five years. Data recoveries for 60-m level wind data exceeded 95 percent for wind speed during each year, and ranged from about 93 to 97 percent for wind direction for all years except 1998 (at slightly more than 88 percent). Subsections 2.3.2 and 2.3.3 establish that these data are representative of site dispersion characteristics.

χ/Q values were estimated at two air intake points leading to the control room—at the Heating Ventilation and Air Conditioning (HVAC) system intake and at the annex building access door (i.e., the pathway for outside air to the control room is that due to building ingress/egress). These two air intake points, designated as Receptors 1 and 2, respectively, are illustrated in Figure 15A-1.

These receptors may be contaminated by accidental radiological releases from any of eight potential sources (the two- or four-letter Source Indicator is included in the ARCON96 model):

- plant vent (Source Indicator - PV);
- passive containment cooling system (PCS) air diffuser (Source Indicator - AD);
- auxiliary building fuel handling area blowout panel (Source Indicator - BP);
- radwaste building truck staging area door (Source Indicator - BD);
- a steam vent (or line) break (Source Indicator - SV);
- Power Operated Relief Valves (PORV) and safety valves (Source Indicator - PORV);
- condenser air removal stack (Source Indicator - AR); and
- the containment shell (Source Indicator - CS).

These potential release points, designated as Sources 1 to 8, respectively, are also illustrated in Figure 15A-1. Note that Source 4, the fuel building rail bay door in the list above, is referred to as the “Radwaste Building Truck Staging Area Door” in Figure 15A-1.

The receptor locations are also reflected in the ARCON96 model and may be distinguished by the respective two-letter indicators “CR” (i.e., control room HVAC intake) and “AN” (annex building access door).

The release types used in the ARCON96 modeling analyses follow those specified in Chapter 15, Appendix 15A. Figure 15A-1 shows that among the potential release sources, the containment shell is considered to be a diffuse area source. All other releases are considered to be point sources.

The Regulatory Position in Section 3.2.2 of Regulatory Guide 1.194 specifies that the stack release mode in ARCON96 is appropriate for releases from a freestanding, vertical, uncapped stack that is outside the directionally dependent zone of influence of adjacent structures. Furthermore, Regulatory Guide 1.194 states that such a stack should be more than 2-1/2 times the height of adjacent structures. From Table 15A-7, the height of the plant vent is 55.7 m above grade; the condenser air removal stack only 49.5 m above grade. Given that the PCS air diffuser sits atop the containment shield building at an elevation of 69.8 m above grade, the vertical criterion for stack releases is not met. Therefore, modeling these sources in stack release mode was not considered.

The Regulatory Position in Section 3.2.3 of Regulatory Guide 1.194 states that modeling sources using the vent release mode “may not be sufficiently conservative for accident evaluations” and so “should not be used in design basis assessments”. As neither a release from the condenser air removal stack nor the plant vent can be represented as stack releases, both potential sources were considered to be ground-level releases in the ARCON96 modeling analyses.

Different building cross-sectional areas were input to the model depending on the receptor being evaluated. For the annex building access door, a building cross-sectional area of 2,636 m^2 was used. This receptor, at an assumed elevation of 1.5 m, is located in a region where the air flow is under the influence of the combined structural wakes generated by the entire containment shield building, the auxiliary building, and the annex building. However, for this modeling analysis, the wake effects induced by the auxiliary building and the annex building were not considered. By excluding these two structures, the total building cross-sectional area is reduced, which is a relatively conservative assumption in that a smaller cross-sectional area results in higher χ/Q values.

The 2,636 m^2 cross-sectional area is based on an assumed diameter of the containment shield building of 43.3 m and an effective structural height of 60.9 m. The assumed diameter of the containment is slightly smaller than the actual diameter and is conservative since the smaller diameter results in a higher χ/Q. The effective structural height takes into account the fact that the containment shield building is a tapered structure beginning at elevation 170.84 ft above grade. The overall height of this building is 228.75 ft above grade. The effective structural height is taken, then, as the mid-point between the start of the taper and the overall building height—that is, 199.8 ft or 60.9 m.

For the receptor at the control room HVAC system intake, a cross-sectional area of 1,805 m^2 was assumed. This receptor, at an elevation of 19.7 m above grade, is located within the wake generated by that portion of the containment shield building that extends above the roof of the auxiliary building where this receptor is situated. The difference between the effective structural height of the containment shield building (i.e., 60.9 m, as discussed above) and the roof height of this part of the auxiliary building (i.e., 19.2 m above grade) is multiplied by the diameter of the containment shield building (i.e., 43.3 m) to yield the cross-sectional area input to the ARCON96 model for estimating χ/Q values at this receptor.

Specification of initial diffusion coefficients is only applicable to a hypothetical release from the containment shell which was modeled as a diffuse area source, as indicated previously. The Regulatory Positions in Sections 3.2.4.4 and 3.2.4.5 of Regulatory Guide 1.194 indicate that in the absence of site-specific empirical data, as is the case here, the initial horizontal and vertical diffusion coefficients may be estimated as follows:

- Sigma-y_o = Area Source Width ÷ 6; and
- Sigma-z_o = Area Source Height ÷ 6.

Consistent with those regulatory positions, the area source width and height are based on the horizontal and vertical dimensions used to determine the building cross-sectional areas input to the ARCON96 modeling analyses. For the receptor at the annex building access door, Sigma-y_o and Sigma-z_o are estimated to be 7.2 m (i.e., 43.3 m ÷ 6) and 10.2 m (i.e., 60.9 m ÷ 6), respectively. For the receptor at the control room HVAC intake, Sigma-y_o and Sigma-z_o are estimated to be 7.2 m (i.e., 43.3 m ÷ 6) and 7.0 m (i.e., 41.7 m ÷ 6), respectively.

Other parameters input to ARCON96 that are based on the recommendations in Regulatory Guide 1.194, Table A-2 (which are different, in some cases, than the default values in the model user's guidance, Reference 201) include:

- Surface Roughness Length = 0.2 (rather than the model default value of 0.1);
- Averaging Sector Width Constant = 4.3 (rather than the model default value of 4.0);
- Vertical Velocity, Stack Radius, and Stack Flow = 0 (all sources are assumed to be ground-level releases and so vertical velocity and stack radius are not used; stack flow during the course of an accident cannot be demonstrated with reasonable assurance);
- Release Height Elevation Difference = 0 (differences in grade elevations between all sources and receptors are only a few feet or less); and
- Wind Direction Window = 90 (default value in both Regulatory Guide 1.194 and Reference 201).

Finally, Table 15A-7 lists the heights of the two modeled receptors and the eight potential sources of radioactive releases, the straight-line distances between these sources and the respective receptors.

2.3.4.3.2 ARCON96 Modeling Results

The χ/Qs determined by the ARCON96 dispersion model represent 95th-percentile values based on all of the hourly relative concentrations calculated using the 5-year meteorological data set input to the model. χ/Q values at the control room HVAC intake and at the annex building access door for time averaging intervals of 0-2 hours, 2-8 hours, 8-24 hours, 1-4 days, and 4-30 days are summarized in Tables 2.3-201 and 2.3-202, respectively.

2.3.4.4 Dispersion Estimates Associated with Accidental Onsite and Offsite Hazardous Material Releases

Potential control room habitability effects and personnel exposures at VEGP Units 3 & 4 due to:

- postulated accidental releases of chemicals and other hazardous materials stored onsite, and at offsite locations within 5 miles of the units;
- for toxic or flammable materials carried over nearby transportation routes (e.g., roadways, railways, and waterways); and
- explosions

were addressed in Subsection 2.2.3 and in Section 2.2.

Concentrations at the control room HVAC intake and at the annex building access door due to accidental hazardous chemical releases were determined and evaluated in consideration of the guidance in Regulatory Guide 1.78, *Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release*, Revision 1, December 2001.

2.3.5 Long-Term (Routine) Diffusion Estimates

The long-term diffusion estimates are site specific. The site boundary annual average χ/Q shown in Table 2.0-201 is used to calculate release concentrations at the site boundary for comparison with the activity release limits defined in 10 CFR 20. The value specified is expected to bound atmospheric conditions at most U.S. sites. If a selected site has a χ/Q value that exceeds this reference site value, the release concentrations reported in Section 11.3 would be adjusted proportionate to the change in χ/Q.

2.3.5.1 Basis

The NRC-sponsored XOQDOQ computer program (NUREG/CR-2919, *XOQDOQ: Computer Program for the Meteorological Evaluation of Routine Effluent Releases at Nuclear Power Stations*, PNL-4380, September 1982 [NUREG/CR-2919]) was used to estimate χ/Q values due to routine releases of gaseous effluents to the atmosphere. The XOQDOQ computer code has the primary function of calculating annual average χ/Q values and annual average relative deposition (D/Q) values at receptors of interest (e.g., the Dose Calculation EAB and the LPZ boundaries, the nearest milk cow, residence, garden, meat animal). χ/Q and D/Q values due to intermittent releases, which occur during routine operation, may also be evaluated using the XOQDOQ model.

The XOQDOQ dispersion model implements the assumptions outlined in Regulatory Guide 1.111. The program assumes that the material released to the atmosphere follows a Gaussian distribution around the plume centerline. In estimating concentrations for longer time periods, the Gaussian distribution is assumed to be evenly distributed within a given directional sector. A straight-line trajectory is assumed between the release point and all receptors.

The following input data and assumptions have been used in the XOQDOQ modeling analysis:

- Meteorological Data: 5-year (January 1, 1998 to December 31, 2002) composite onsite JFD of wind speed, wind direction, and atmospheric stability.
- Type of release: Ground-level.
- Wind sensor height: 10 m.
- Vertical temperature difference: (10 m – 60 m).
- Number of wind speed categories: 11.
- Release height: 10 m (default height).
- Minimum building cross-sectional area: 2,926 m^2.
- Containment structure equivalent height: 65.6 m.
- Distances from the release point to the nearest residence, nearest site boundary, vegetable garden, and meat animal.

The AP1000 reactor design has been used to calculate the minimum building cross-sectional area as called for in NUREG/CR-2919 for evaluating building downwash effects on dispersion. The containment building minimum cross-sectional area contains two parts: the reactor enclosure building plus a PCS water tank on the top of that structure. The height of the entire contiguous building is assumed to be 234.4 ft (71.4 m), while the bottom (W_B) and the top (W_T) widths of the building are 146.3 ft (44.6 m) and 89 ft (27.1 m), respectively. The height of the PCS is 39.1 ft (11.9 m).

The total calculated cross-sectional area of the structure (A_T) is 31,498 ft^2 (2,926 m^2). Using this total area, and dividing by the actual width of the bottom of the reactor enclosure building (i.e., 146.3 ft), the equivalent structural height is calculated ($H_e = A_T / W_B$) to be 215.2 ft (65.6 m), which assumes that the structure width is uniform in the vertical direction.

These calculated values were input into the XOQDOQ model to predict the required annual average χ/Q and D/Q values.

Distances from the midpoint between the VEGP Unit 1 and Unit 2 reactors to various receptors of interest (i.e., nearest residence, meat animal, site boundary, and vegetable garden) for each directional sector are provided in AREOR (2004). The distance to the nearest residence (i.e., 0.67 mi) was conservatively used in all the directional sectors for all types of sensitive receptors (meat animal, vegetable garden, and residence). The results are presented in Table 2.3-218.

2.3.5.2 XOQDOQ Modeling Results

Table 2.3-219 summarizes the maximum relative concentration and relative deposition (i.e., χ/Q and D/Q values predicted by the XOQDOQ model for identified sensitive receptors in the vicinity of the VEGP site due to routine releases of gaseous effluents. The listed maximum χ/Q values reflect several plume depletion scenarios that account for radioactive decay (i.e., no decay, and the default half-life decay periods of 2.26 and 8 days).

The overall maximum annual average χ/Q value (with no decay) is 5.5 x 10^{-6} sec/m^3 and occurs at the Dose Calculation EAB at a distance of 0.5 mi to the northeast of the VEGP site. The maximum annual average χ/Q values (along with the direction and distance of the receptor locations relative to the VEGP site) for the other sensitive receptor types are:

- 3.4 x 10^{-6} sec/m^3 for the nearest residence occurring in the northeast sector at a distance of 0.67 mi.
- Because the same shortest distance (0.67 mi) was used to estimate χ/Q values for the nearest vegetable garden and meat animal, the same χ/Q value (3.4 x 10^{-6} sec/m^3) was obtained at these receptors.

Finally, Table 2.3-220 summarizes annual average χ/Q values (for no decay) and D/Q values at the XOQDOQ model's 22 standard radial distances between 0.25 and 50 mi and for the model's 10 distance-segment boundaries between 0.5 and 50 mi downwind along each of the 16 standard direction radials (i.e., separated by 22.5 degrees).

In the AP1000 reactor DCD, the terms "site boundary" and "exclusion area boundary" (EAB) are used interchangeably. Thus, the χ/Q specified for the site boundary applies whenever a discussion in the DCD refers to the exclusion area boundary. In Subsection 2.3.5 site specific χ/Q calculations, the term "Dose Calculation EAB" is equivalent to the DCD term "EAB".

Using the same assumptions and methodology as described earlier in this subsection (which relied on DCD Revision 15), along with the building dimensions provided in DCD Revision 17, the long-term (routine release) dispersion and deposition estimates were evaluated at the Dose Calculation EAB and at the various receptor locations. This evaluation confirmed that the χ/Q values for the EAB and the various receptor locations are within approximately 3.3% of those previously calculated. This result is reasonable given that the designated receptor points at the EAB and the various receptor locations are beyond the distance that would be appreciably influenced by building wake.

2.3.6 Combined License Information

2.3.6.1 Regional Climatology

Site-specific information related to regional climatology is addressed in Subsection 2.3.1.

2.3.6.2 Local Meteorology

Site-specific local meteorology information is addressed in Subsection 2.3.2.

2.3.6.3 Onsite Meteorological Measurements Program

The site-specific onsite meteorological measurements program is addressed in Subsections 2.3.3.4 and 2.3.3.

2.3.6.4 Short-Term Diffusion Estimates

Site-specific χ/Q values are addressed in Subsections 2.3.4, 15.6.5.3.7.3, and Appendix 15A.3.3.

2.3.6.5 Long-Term Diffusion Estimates

Long-term diffusion estimates and χ/Q values are addressed in Subsection 2.3.5.

2.3.7 References

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224. **(NCDC 2006a)** National Climatic Data Center, *Storm Events for Georgia and South Carolina,* Tornado Event summaries, NCDC, NOAA, http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwEvent~Storms, accessed July 2005 and January 2006.

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234. **(Westinghouse 2005)** AP1000 Design Control Document, Revision 15, Westinghouse Electric Company, 2005.

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Table 2.3-201
ARCON96 X/Q Values at the Control Room HVAC Intake

Release Point	0 – 2 hours	2 – 8 hours	8 – 24 hours	1 – 4 days	4 – 30 days
Plant Vent	2.27E-03	1.86E-03	7.36E-04	5.99E-04	4.31E-04
PCS Air Diffuser	1.71E-03	1.32E-03	5.56E-04	4.63E-04	3.43E-04
Auxiliary Building Fuel Handling Area Blowout Panel	1.57E-03	1.15E-03	4.62E-04	3.72E-04	2.68E-04
Radwaste Building Truck Staging Area Door	1.30E-03	9.36E-04	3.78E-04	2.98E-04	2.09E-04
Steam Line Break	1.87E-02	1.51E-02	6.79E-03	4.94E-03	4.14E-03
PORV & Safety Valves	1.77E-02	1.41E-02	6.25E-03	4.61E-03	3.87E-03
Condenser Air Removal Stack	6.60E-04	4.83E-04	2.17E-04	1.57E-04	1.17E-04
Containment Shell (As Diffuse Area Source)	2.93E-03	1.75E-03	7.78E-04	6.81E-04	5.30E-04

Table 2.3-202
ARCON96 X/Q Values at the Annex Building Access Door

Release Point	0 – 2 hours	2 – 8 hours	8 – 24 hours	1 – 4 days	4 – 30 days
Plant Vent	5.02E-04	3.94E-04	1.61E-04	1.29E-04	9.63E-05
PCS Air Diffuser	4.62E-04	3.55E-04	1.49E-04	1.23E-04	9.12E-05
Auxiliary Building Fuel Handling Area Blowout Panel	3.99E-04	3.00E-04	1.22E-04	1.00E-04	7.23E-05
Radwaste Building Truck Staging Area Door	3.83E-04	2.88E-04	1.21E-04	9.58E-05	6.78E-05
Steam Line Break	1.00E-03	7.97E-04	3.25E-04	2.58E-04	1.91E-04
PORV & Safety Valves	1.13E-03	8.98E-04	3.69E-04	2.92E-04	2.19E-04
Condenser Air Removal Stack	1.72E-03	1.12E-03	4.50E-04	3.17E-04	2.60E-04
Containment Shell (As Diffuse Area Source)	3.97E-04	3.26E-04	1.34E-04	1.10E-04	8.32E-05

Table 2.3-203
NWS and Cooperative Observing Stations Near the VEGP Site

Station[a]	State	County	Approximate Distance (miles)	Direction Relative to Site	Elevation (feet)
Waynesboro 2NE	GA	Burke	16	WSW	270
Augusta WSO (Bush Field)	GA	Richmond	20	NW	132
Millen 4N	GA	Jenkins	22	SSW	195
Midville Experiment Station	GA	Burke	32	SW	280
Louisville 1E	GA	Jefferson	37	WSW	322
Newington 2NE	GA	Screven	41	SSE	209
Aiken 4NE	SC	Aiken	25	NNE	502
Blackville 3W	SC	Barnwell	29	ENE	324
Springfield	SC	Orangeburg	37	NE	300
Bamberg	SC	Bamberg	44	ENE	165

Notes:

a – Numeric and letter designators following a station name (e.g., Waynesboro 2NE) indicate the station's approximate distance in miles (e.g., 2) and direction (e.g., northeast) relative to the place name (e.g., Waynesboro)

Table 2.3-204
Local Climatological Data Summary for Augusta, Georgia

NORMALS, MEANS, AND EXTREMES

AUGUSTA, GA (AGS)

LATITUDE: 33° 22' 11" N **LONGITUDE:** 81° 57' 53" W **ELEVATION (FT):** GRND: 160 BARO: 163 **TIME ZONE:** EASTERN (UTC +5) **WBAN:** 03820

	ELEMENT	POR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
TEMPERATURE °F	NORMAL DAILY MAXIMUM	30	56.5	61.3	69.2	76.7	83.9	89.6	92.0	90.2	85.3	76.5	67.8	59.1	75.7
	MEAN DAILY MAXIMUM	48	56.4	60.6	68.3	76.8	84.0	89.4	91.9	90.6	85.6	76.9	68.3	59.1	75.7
	HIGHEST DAILY MAXIMUM	54	82	86	89	96	99	105	107	108	101	97	90	82	108
	YEAR OF OCCURRENCE		2002	1962	1995	1986	2000	1952	1980	1983	1999	1954	1961	1998	AUG 1983
	MEAN OF EXTREME MAXS.	56	74.4	76.0	80.7	88.8	93.4	98.1	99.0	97.9	94.5	88.3	81.5	76.1	87.4
	NORMAL DAILY MINIMUM	30	33.1	35.5	42.5	48.1	57.2	65.4	69.6	68.4	62.4	49.6	40.9	34.7	50.6
	MEAN DAILY MINIMUM	48	32.7	34.7	40.4	48.9	58.0	66.0	70.1	69.1	63.3	50.7	41.5	34.3	50.8
	LOWEST DAILY MINIMUM	54	-1	0	0	26	35	47	55	52	36	22	15	5	-1
	YEAR OF OCCURRENCE		1985	1998	1998	1982	1971	1984	1951	2004	1967	1952	1970	1981	JAN 1985
	MEAN OF EXTREME MINS.	56	16.6	19.0	25.0	33.4	43.5	54.7	62.5	60.4	49.7	34.4	24.9	18.5	36.9
	NORMAL DRY BULB	30	44.8	48.4	55.9	62.4	70.5	77.5	80.8	79.3	73.8	63.1	54.4	46.9	63.1
	MEAN DRY BULB	56	45.2	48.4	55.3	63.0	71.2	77.9	81.0	80.1	74.6	64.1	54.5	46.9	63.5
	MEAN WET BULB	49	40.3	42.8	48.4	55.5	63.4	69.8	72.7	72.3	67.4	57.4	48.5	41.7	56.7
	MEAN DEW POINT	49	34.4	36.0	41.5	49.4	58.9	66.0	69.7	69.4	64.3	53.4	43.2	36.1	51.9
	NORMAL NO. DAYS WITH:														
	MAXIMUM ≥ 90°	30	0.0	0.0	0.0	0.6	5.9	16.0	23.5	19.4	9.4	0.6	0.0	0.0	75.4
	MAXIMUM ≤ 32°	30	0.4	0.2	*	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.7
	MINIMUM ≤ 32°	30	15.0	11.5	4.6	0.9	0.0	0.0	0.0	0.0	0.0	0.6	6.5	13.1	52.2
	MINIMUM ≤ 0°	30	*	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
H/C	NORMAL HEATING DEG. DAYS	30	617	469	301	129	21	1	0	0	5	118	317	547	2525
	NORMAL COLLING DEG. DAYS	30	1	2	15	52	191	385	506	459	285	74	15	1	1986
RH	NORMAL (PERCENT)	30	70	67	66	66	70	72	74	77	77	75	74	72	72
	HOUR 01 LST	30	80	77	77	80	86	87	88	91	90	89	86	82	84
	HOUR 07 LST	30	84	84	85	86	87	87	89	92	92	91	89	85	88
	HOUR 13 LST	30	55	50	48	45	48	52	54	56	55	50	51	54	52
	HOUR 19 LST	30	68	61	57	55	60	63	67	72	77	78	74	71	67
S	PERCENT POSSIBLE SUNSHINE														
W/O	MEAN NO. DAYS WITH:														
	HEAVY FOG (VISBY ≤ 1/4 MI)	54	3.5	2.7	2.1	2.5	2.5	1.4	1.6	3.1	3.8	3.9	4.0	4.0	35.1
	THUNDERSTORMS	54	0.9	1.8	2.7	3.6	6.0	9.4	11.9	9.3	3.4	1.3	0.8	0.7	51.8
CLOUDINESS	MEAN:														
	SUNRISE-SUNSET (OKTAS)	1			7.2		3.2	4.0	5.6	4.8		5.6		4.0	
	MIDNIGHT-MIDNIGHT (OKTAS)	1			6.4		4.0	4.0	4.8	4.0					
	MEAN NO. DAYS WITH:														
	CLEAR	1	2.0	2.0	9.0		14.0	7.0	2.0	6.0	3.0	7.0	5.0	10.0	
	PARTLY CLOUDY	1		2.0	1.0		2.0	8.0	2.0	2.0	2.0	4.0	1.0	1.0	
	CLOUDY	1	2.5	3.0	12.0		3.0	4.0		6.0	7.0	3.0	1.0	7.0	
PR	MEAN STATION PRESSURE (IN)	31	29.97	29.93	29.89	29.86	29.83	29.84	29.87	29.88	29.89	29.93	29.96	29.98	29.90
	MEAN SEA-LEVEL PRES. (IN)	47	30.14	30.09	30.04	30.02	30.00	29.99	30.03	30.01	30.04	30.08	30.11	30.15	30.06
WINDS	MEAN SPEED (MPH)	28	6.7	7.1	7.4	6.9	6.1	5.7	5.6	5.0	5.3	5.2	5.5	6.2	6.1
	PREVAIL. DIR (TENS OF DEGS)	29	27	29	29	18	14	14	24	14	04	04	29	29	24
	MAXIMUM 2-MINUTE:														
	SPEED (MPH)	10	40	37	40	37	49	45	36	38	36	38	38	35	49
	DIR. (TENS OF DEGREES)		26	30	29	28	23	34	21	01	02	34	18	28	23
	YEAR OF OCCURRENCE		1997	2003	1999	2001	2004	1998	1995	2002	1997	1995	2001	2000	MAY 2004
	MAXIMUM 5-SECOND:														
	SPEED (MPH)	10	54	45	51	55	74	53	47	49	45	52	49	43	74
	DIR. (TENS OF DEGREES)		25	31	29	34	23	33	21	01	01	33	03	28	23
	YEAR OF OCCURRENCE		1997	2003	1999	1997	2004	1998	1998	2002	1997	1995	1995	2000	MAY 2004
PRECIPITATION	NORMAL (IN)	30	4.50	4.11	4.61	2.94	3.07	4.19	4.07	4.48	3.59	3.20	2.68	3.14	44.58
	MAXIMUM MONTHLY (IN)	54	8.91	7.67	11.92	8.43	9.61	10.57	11.43	11.34	9.51	14.82	7.76	8.65	14.82
	YEAR OF OCCURRENCE		1987	1961	1980	1961	1979	2004	1967	1986	1975	1990	1985	1981	OCT 1990
	MINIMUM MONTHLY (IN)	54	0.75	0.69	0.88	0.60	0.36	0.68	1.02	0.65	0.31	T	0.09	0.32	T
	YEAR OF OCCURRENCE		1981	1968	1968	1970	2000	1984	1987	1980	1984	1953	1960	1955	OCT 1953
	MAXIMUM IN 24 HOURS (IN)	54	3.61	3.69	5.31	3.96	4.44	5.08	3.71	5.98	7.30	8.57	3.82	3.12	8.57
	YEAR OF OCCURRENCE		1960	1985	1967	1955	1981	1981	1979	1964	1998	1990	1985	1970	OCT 1990
	NORMAL NO. DAYS WITH:														
	PRECIPITATION ≥ 0.01	30	11.0	8.7	9.8	7.4	9.0	10.1	11.2	10.9	7.8	6.2	7.2	9.5	108.8
	PRECIPITATION ≥ 1.00	30	1.2	1.2	1.3	0.8	0.8	1.4	1.1	1.4	0.9	1.0	0.8	0.7	12.6
SNOWFALL	NORMAL (IN)	30	0.3	1.0	0.*	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	1.4
	MAXIMUM MONTHLY (IN)	50	2.6	14.0	1.1	T	0.0	T	0.0	0.0	0.0	0.0	T	1.0	14.0
	YEAR OF OCCURRENCE		1992	1973	1980	1992		1994					1968	1993	FEB 1973
	MAXIMUM IN 24 HOURS (IN)	50	2.6	13.7	1.1	T	0.0	T	0.0	0.0	0.0	0.0	T	1.0	13.7
	YEAR OF OCCURRENCE		1992	1973	1980	1992		1994					1968	1993	FEB 1973
	MAXIMUM SNOW DEPTH (IN)	48	2	13	1	0	0	0	0	0	0	0	0	1	13
	YEAR OF OCCURRENCE		1988	1973	1980									1958	FEB 1973
	NORMAL NO. DAYS WITH:														
	SNOWFALL ≥ 1.0	30	0.1	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3

Note: Source: NCDC, 2005a

Table 2.3-205
Climatological Extremes at Selected NWS and Cooperative Observing Stations in the VEGP Site Area

Parameter	Waynesboro 2NE	Augusta WSO	Millen 4N	Midville Exp Station	Louisville 1E	Newington 2NE	Aiken 4NE	Blackville 3W	Springfield	Bamberg
Maximum Temperature	108 °F [a, b] (7/25/52); (7/14/80)	108 °F [a] (8/21/83)	109 °F [b] (7/24/52)	105 °F [a, b] (7/13/80); (8/21/83) (7/19/86); (7/21/86)	112 °F [a] (7/24/52)	110 °F [a] (7/13/80)	109 °F [a] (8/22/83)	108 °F [a] (8/1/99)	NA [d]	109 °F [a] (7/24/52)
Minimum Temperature	-1 °F [a, b] (1/20/85); (1/21/85)	-1 °F [a] (1/21/85)	0 °F [b] (1/21/85)	-1 °F [a] (1/21/85)	-2 °F [a] (1/21/85)	-1 °F [a] (1/21/85)	-4 °F [a] (1/21/85)	-1 °F [a] (1/21/85)	NA [d]	2 °F [a] (1/21/85)
Maximum 24-hr Rainfall	7.40 in. [a] (10/3/94)	7.30 in. [a] (9/3/98)	8.02 in. [b] (8/29/64)	8.19 in. [a] (10/12/90)	8.60 in. [a] (10/12/90)	5.50 in. [a] (10/10/90)	9.68 in. [a] (4/16/69)	7.53 in. [a] (9/30/59)	7.10 in. [b, c] (9/30/59)	8.02 in. [a, c] (9/23/00)
Maximum Monthly Rainfall	16.99 in. [a, b] (10/94)	14.82 in. [a, b] (10/90)	13.45 in. [b] (8/64)	15.97 in. [b, c] (8/70)	14.76 in. [b, c] (8/91)	15.29 in. [a, b] (7/89)	14.45 in. [a, b] (3/80)	14.67 in. [a, b] (10/90)	17.32 in. [b, c] (6/73)	15.26 in. [a, b] (8/95)
Maximum 24-hr Snowfall	16.0 in. [a, b] (2/10/73)	8.0 in. [a, b] (2/9/73)	14.0 in. [b] (2/10/73)	14.0 in. [b, c] (2/10/73)	14.8 in. [a, b] (2/10/73)	5.0 in. [a, b] (2/10/73)	15.0 in. [a, b] (2/10/73)	17.0 in. [b, c] (2/10/73)	8.0 in. [b, c] (2/11/73)	19.0 in. [a, b] (2/10/73)
Maximum Monthly Snowfall	16.0 in. [a, b] (2/73)	14.0 in. [a, b] (2/73)	15.0 in. [b] (2/68)	14.0 in. [b, c] (2/73)	14.8 in. [a, b] (2/73)	8.0 in. [a, b] (2/73)	15.0 in. [a, b] (2/73)	17.0 in. [b, c] (2/73)	15.0 in. [b, c] (2/73)	22.0 in. [a, b] (2/73)

Sources:
a – NCDC 2005b
b – SERCC 2006
c – NCDC 2002c
d – NA = Measurements not made at this station

Table 2.3-206
Mean Seasonal and Annual Morning and Afternoon Mixing Heights and Wind Speeds for Athens, Georgia

Parameter	Winter	Spring	Summer	Autumn	Annual
Mixing Height – AM (m)	407	383	390	314	374
Wind Speed – AM (m/sec)	6.0	5.3	3.8	4.4	4.9
Mixing Height – PM (m)	1042	1754	1918	1455	1542
Wind Speed – PM (m/sec)	7.0	7.2	4.9	5.7	6.2

Note: Mean wind speed values represent the arithmetic average of speeds observed at the surface and aloft within the mixed layer.
Source: Reference 209

Table 2.3-207
Climatological Normals (Means) at Selected NWS and Cooperative Observing Stations in the VEGP Site Area

Station	Normal Annual Temperatures (°F)[a]			Normal Annual Precipitation	
	Daily Maximum	Daily Minimum	Daily Mean	Rainfall[a] (inches)	Snowfall (inches)
Waynesboro 2NE	75.2	51.0	63.1	47.20	1.0 [a]
Augusta	75.7	50.6	63.2	44.58	1.4 [b]
Millen 4N	76.1	50.6	63.4	43.85	0.5 [c]
Midville Exp Station	76.9	52.9	65.0	44.90	0.1 [b]
Louisville 1E	75.6	51.7	63.7	45.92	0.9 [b]
Newington 2NE	76.2	52.5	64.4	47.81	0.8 [b]
Aiken 4NE	77.2	50.9	64.1	52.43	1.4 [b]
Blackville 3W	77.6	51.6	64.6	47.23	0.7 [b]
Springfield	NA [e]	NA [e]	NA [e]	46.28	0.7 [d]
Bamberg	75.0	53.1	64.1	48.57	1.3 [b]

Sources:
a – Reference 211
b – Reference 212
c – Reference 230, based on available Period of Record (1930-1998)
d – Reference 230, based on available Period of Record (1948-2005)
e – NA = Measurements not made at this station

Table 2.3-208
Seasonal and Annual Mean Wind Speeds for the VEGP Site (1998–2002) and the Augusta, Georgia, NWS Station (1971–2000, Normals)

60-Meter Tower Tower Elevation	Location	Winter	Spring	Summer	Autumn	Annual
Upper Level (60 m) (m/sec)	Plant Vogtle	5.0	5.0	4.1	4.4	4.6
Lower Level (10 m) (m/sec)	Plant Vogtle	2.6	2.8	2.4	2.3	2.5
Single Level (6.1 m) (m/sec)	Augusta WSO [a]	3.0	3.0	2.4	2.4	2.7

Notes:
Winter = December, January, February
Spring = March, April, May
Summer = June, July, August
Autumn = September, October, November
Source: a - Reference 221

Table 2.3-209 (Sheet 1 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

1998 TO 2002 WIND PERSISTENCE
VEGP METEOROLOGICAL TOWER – 10-M LEVEL
22.5° SECTOR WIDTH
START AND END OF PERIOD 98010101 - 02123124

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 5.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1180	1133	1919	2028	1392	822	948	863	906	1298	1541	1478	1804	1444	856	894
2	439	376	919	983	538	231	353	294	305	493	621	526	830	639	266	310
4	99	75	343	326	139	27	88	58	56	102	164	105	246	197	51	52
8	6	4	97	65	13	4	5	2	3	4	14	4	28	30	3	0
12	0	0	36	10	0	0	0	0	0	0	0	0	2	9	0	0
18	0	0	9	0	0	0	0	0	0	0	0	0	0	3	0	0
24	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 10.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	136	126	323	415	149	58	116	85	74	167	246	250	362	361	150	59
2	42	51	129	197	39	16	37	27	24	57	106	91	156	167	46	22
4	7	9	40	63	5	3	8	5	3	9	25	21	47	45	11	6
8	0	0	11	7	0	0	0	0	0	0	0	1	4	5	0	0
12	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-209 (Sheet 2 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 15.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	3	9	13	25	8	1	6	3	4	14	21	17	40	43	19	2
2	0	3	2	10	0	0	0	0	0	4	6	5	13	14	5	1
4	0	0	0	5	0	0	0	0	0	0	3	0	0	2	0	0
8	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 20.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1	0	0	4	0	0	0	0	0	2	0	1	3	5	0	0
2	0	0	0	3	0	0	0	0	0	0	0	0	0	1	0	0
4	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-209 (Sheet 3 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 25.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-209 (Sheet 4 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

1998 TO 2002 WIND PERSISTENCE
VEGP METEOROLOGICAL TOWER - 10-M LEVEL
67.5° SECTOR WIDTH

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 5.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	3207	4232	5080	5339	4242	3162	2633	2717	3067	3745	4317	4823	4726	4104	3194	2930
2	1885	2649	3569	3875	2751	1762	1438	1539	1694	2224	2686	3187	3226	2738	1881	1630
4	901	1461	2358	2587	1495	830	666	740	733	1031	1363	1765	1941	1635	908	738
8	310	653	1331	1443	570	271	219	248	208	250	455	623	824	749	297	216
12	129	358	828	880	237	96	78	116	68	73	168	209	361	376	119	80
18	54	187	466	471	87	23	19	29	4	15	57	64	134	148	41	20
24	32	107	283	287	32	0	3	6	0	3	20	15	52	67	17	2
30	17	69	164	178	2	0	0	0	0	0	6	2	22	33	2	0
36	11	48	96	117	0	0	0	0	0	0	0	0	4	20	0	0
48	0	27	33	38	0	0	0	0	0	0	0	0	0	8	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 10.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	321	585	864	887	622	323	259	275	326	487	663	858	973	873	570	345
2	160	271	484	515	328	114	115	127	143	243	354	489	592	549	332	143
4	74	115	212	243	128	26	42	49	40	71	135	218	299	313	168	59
8	33	44	69	74	24	0	12	15	0	2	15	36	81	115	55	16
12	19	21	26	20	4	0	2	3	0	0	0	2	30	43	18	4
18	5	6	3	1	0	0	0	0	0	0	0	0	6	13	4	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-209 (Sheet 5 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 15.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	14	25	47	46	34	15	10	13	21	39	52	78	100	102	64	24
2	5	6	20	17	10	0	0	0	4	14	23	29	49	56	29	7
4	0	0	7	7	5	0	0	0	0	5	5	6	16	21	9	0
8	0	0	1	1	1	0	0	0	0	0	0	0	3	3	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 20.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1	1	4	4	4	0	0	0	2	2	3	4	9	8	5	1
2	0	0	3	3	3	0	0	0	0	0	0	0	3	3	1	0
4	0	0	1	1	1	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-209 (Sheet 6 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 10-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 25.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-210 (Sheet 1 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

1998 TO 2002 WIND PERSISTENCE
VEGP METEOROLOGICAL TOWER - 60-M LEVEL
22.5° SECTOR WIDTH

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 5.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	1610	1940	3083	2713	2037	1558	1645	2015	2294	2694	3397	3268	3052	2001	1615	1488
2	641	889	1687	1343	946	666	734	986	1057	1266	1739	1594	1576	910	663	575
4	168	245	736	446	273	167	218	319	290	346	569	492	586	293	146	131
8	20	33	192	70	43	19	20	56	35	27	73	51	122	67	6	3
12	4	7	67	7	15	1	4	15	0	0	5	13	17	16	0	0
18	0	0	20	0	5	0	0	0	0	0	0	0	0	0	0	0
24	0	0	13	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	7	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 10.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	616	954	1922	1457	984	747	802	713	1006	1597	2138	2098	2036	1247	775	615
2	240	435	1107	710	442	303	339	305	433	750	1106	1066	1106	619	322	231
4	68	116	515	219	114	77	100	82	118	207	366	359	444	233	73	59
8	14	16	161	33	23	10	13	6	12	13	43	44	101	60	4	2
12	4	6	63	5	12	0	1	0	0	0	3	13	13	15	0	0
18	0	0	20	0	2	0	0	0	0	0	0	0	0	0	0	0
24	0	0	13	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	7	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-210 (Sheet 2 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 15.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	131	211	522	254	106	66	112	75	171	364	628	721	732	436	147	123
2	53	87	264	94	31	11	33	15	52	123	277	314	362	211	49	39
4	23	27	117	29	6	0	10	2	8	26	81	94	140	89	15	9
8	12	10	44	8	0	0	3	0	0	0	3	9	34	21	2	1
12	4	6	24	4	0	0	0	0	0	0	0	3	1	2	0	0
18	0	0	17	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	11	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	5	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 20.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	24	21	44	25	12	8	16	4	19	48	97	135	183	118	36	12
2	13	6	20	10	4	0	5	0	3	14	21	48	87	54	16	4
4	7	1	7	5	0	0	3	0	0	2	0	12	30	19	7	2
8	3	0	0	1	0	0	0	0	0	0	0	0	6	1	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-210 (Sheet 3 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 22.5 DEGREES)

Speed Greater than or Equal to 25.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	2	0	5	5	1	0	0	0	2	6	15	26	37	21	5	3
2	0	0	1	3	0	0	0	0	0	2	2	12	16	7	1	2
4	0	0	0	1	0	0	0	0	0	0	0	6	6	2	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-210 (Sheet 4 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

1998 TO 2002 WIND PERSISTENCE
VEGP METEOROLOGICAL TOWER - 60-M LEVEL
67.5° SECTOR WIDTH

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 5.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	5038	6633	7736	7833	6308	5240	5218	5954	7003	8385	9359	9717	8321	6668	5104	4713
2	3401	4871	6139	6199	4565	3663	3670	4240	5098	6291	7318	7740	6402	4858	3475	3173
4	1887	3216	4448	4396	2827	2165	2126	2561	3130	4099	5024	5525	4399	3100	1942	1745
8	842	1778	2685	2516	1215	905	847	1122	1331	1939	2694	3133	2539	1549	726	666
12	459	1095	1746	1561	527	398	376	556	576	953	1523	1874	1606	876	295	286
18	225	581	1046	836	152	127	134	198	184	370	671	934	842	425	112	121
24	123	355	665	449	61	52	44	77	69	151	331	511	460	223	51	71
30	82	241	417	251	19	28	14	46	24	57	146	308	217	110	17	49
36	52	162	253	145	11	16	4	28	5	13	58	186	84	54	3	38
48	18	66	95	49	0	0	0	1	0	0	4	80	9	11	0	26

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 10.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	2185	3492	4333	4363	3188	2533	2262	2521	3316	4741	5833	6272	5381	4058	2637	2006
2	1281	2389	3217	3156	2011	1548	1344	1406	2029	3291	4248	4711	4052	2884	1663	1170
4	627	1465	2159	1982	998	757	620	620	959	1932	2698	3182	2793	1848	876	557
8	245	751	1218	993	313	228	183	188	223	775	1306	1701	1607	984	325	207
12	139	460	754	570	119	74	69	76	50	330	700	985	1007	555	125	109
18	84	230	449	296	26	8	14	21	0	118	275	496	503	264	24	52
24	45	131	285	165	5	0	1	4	0	48	104	273	252	130	2	32
30	26	76	176	97	0	0	0	0	0	19	30	170	108	56	0	20
36	12	45	108	62	0	0	0	0	0	1	6	106	35	29	0	14
48	0	13	44	19	0	0	0	0	0	0	0	41	0	10	0	2

Table 2.3-210 (Sheet 5 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 15.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	465	864	987	882	426	284	253	358	610	1163	1713	2081	1889	1315	706	401
2	223	470	549	462	163	104	90	126	243	606	1005	1322	1217	814	369	189
4	100	233	276	207	46	32	29	32	63	239	482	731	711	456	173	92
8	51	106	121	79	11	5	8	8	1	41	111	257	304	206	59	48
12	37	59	75	47	7	0	0	0	0	8	31	97	145	95	8	33
18	19	24	44	29	1	0	0	0	0	0	5	12	37	40	0	21
24	10	11	26	19	0	0	0	0	0	0	0	0	18	25	0	10
30	4	5	14	13	0	0	0	0	0	0	0	0	11	18	0	4
36	0	0	6	7	0	0	0	0	0	0	0	0	5	12	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 20.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	57	89	90	81	45	36	28	39	71	164	280	415	436	337	166	72
2	26	43	44	38	15	10	5	9	21	55	122	210	240	194	88	38
4	14	16	17	15	5	3	3	3	3	12	38	82	107	92	41	23
8	5	3	1	1	1	0	0	0	0	1	2	16	18	19	7	10
12	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	2
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-210 (Sheet 6 of 6)
Wind Direction Persistence/Wind Speed Distributions for the VEGP Site – 60-m Level

PERSISTENCIES FROM 98010101 TO 02123124 (SECTOR WIDTH = 67.5 DEGREES)

Speed Greater than or Equal to 25.0 mph

Hours	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1	5	7	10	11	6	1	0	2	8	23	47	78	84	63	29	10
2	2	1	5	5	3	0	0	0	2	6	21	36	40	30	12	3
4	0	0	1	1	1	0	0	0	0	1	9	16	17	12	3	0
8	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-211
Seasonal and Annual Vertical Stability Class and Mean 10-Meter Level Wind Speed Distributions for the VEGP Site (1998–2002)

	Vertical Stability Categories[a]						
Period	**A**	**B**	**C**	**D**	**E**	**F**	**G**
Winter							
Frequency (%)	2.23	2.94	6.40	31.25	28.96	14.06	14.14
Wind Speed (m/sec)	3.4	3.9	3.6	3.1	2.6	1.7	1.4
Spring							
Frequency (%)	11.49	5.29	7.04	25.18	27.10	13.94	9.95
Wind Speed (m/sec)	3.6	3.7	3.6	3.3	2.5	1.8	1.4
Summer							
Frequency (%)	8.27	6.12	7.60	24.73	33.00	14.22	6.04
Wind Speed (m/sec)	3.4	3.1	2.9	2.7	2.1	1.5	1.4
Autumn							
Frequency (%)	3.76	3.79	8.36	28.90	26.92	13.65	14.62
Wind Speed (m/sec)	3.2	3.3	3.2	2.8	2.2	1.7	1.2
Annual							
Frequency (%)	6.48	4.54	7.34	27.50	28.99	13.97	11.17
Wind Speed (m/sec)	3.5	3.5	3.3	3.0	2.4	1.7	1.3

Note: a – Vertical stability based on temperature difference (DT) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 1 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: A Delta Temperature Extremely Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	0	0	3	7	38	63	4	0	0	0	0	115
NNE	0	0	2	3	17	48	33	13	0	0	0	0	116
NE	0	0	0	7	6	36	79	17	0	0	0	0	145
ENE	0	0	1	3	13	75	127	30	0	0	0	0	249
E	0	0	0	5	15	77	133	10	0	0	0	0	240
ESE	0	0	1	4	17	66	55	0	0	0	0	0	143
SE	0	1	1	4	11	41	49	5	0	0	0	0	112
SSE	0	0	1	9	2	32	36	2	1	0	0	0	83
S	0	1	0	10	22	42	51	5	0	0	0	0	131
SSW	0	0	2	6	19	59	97	12	0	0	0	0	195
SW	0	0	2	8	18	71	117	20	3	0	0	0	239
WSW	0	0	2	6	23	74	167	26	3	0	0	0	301
W	0	2	0	4	17	79	156	26	2	0	0	0	286
WNW	0	0	0	5	9	39	88	16	3	0	0	0	160
NW	0	0	0	6	9	28	57	14	3	0	0	0	117
NNW	1	0	1	2	6	23	59	1	0	0	0	0	93
Totals	1	4	13	85	211	828	1367	201	15	0	0	0	2725

Number of Calm Hours for this Table	0
Number of Variable Direction Hours for this Table	11
Number of Invalid Hours	1633
Number of Valid Hours for this Table	2725
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 2 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: B Delta Temperature Moderately Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	0	1	3	9	39	64	5	0	1	0	0	122
NNE	0	0	0	5	13	38	36	8	2	0	0	0	102
NE	0	1	0	4	7	40	48	7	0	0	0	0	107
ENE	1	0	0	1	11	54	69	23	0	0	0	0	159
E	0	0	0	5	4	44	65	8	0	0	0	0	126
ESE	0	0	1	6	6	31	22	3	0	0	0	0	69
SE	0	0	4	7	8	23	22	1	0	0	0	0	65
SSE	0	0	0	7	14	21	18	1	0	0	0	0	61
S	0	1	0	2	12	30	27	4	0	0	0	0	76
SSW	0	0	0	3	17	53	51	5	2	0	0	0	131
SW	0	0	1	9	18	51	75	19	2	0	0	0	175
WSW	0	0	0	4	7	58	64	18	1	0	0	0	152
W	0	0	0	2	8	60	96	22	3	0	0	0	191
WNW	0	0	0	2	7	37	75	28	4	1	0	0	154
NW	0	0	0	3	5	33	42	12	2	0	0	0	97
NNW	0	0	0	1	11	37	70	4	0	0	0	0	123
Totals	1	2	7	64	157	649	844	168	16	2	0	0	1910

Number of Calm Hours for this Table	1
Number of Variable Direction Hours for this Table	44
Number of Invalid Hours	1633
Number of Valid Hours for this Table	1910
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 3 of 8) Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: C Delta Temperature Slightly Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	1	1	8	24	81	84	5	1	0	0	0	205
NNE	0	0	4	6	17	72	72	3	0	0	0	0	174
NE	0	0	0	5	15	60	72	13	0	0	0	0	165
ENE	0	0	3	6	19	74	115	17	0	0	0	0	234
E	0	0	1	9	21	58	105	1	1	0	0	0	196
ESE	0	0	2	9	15	52	44	1	0	0	0	0	123
SE	0	1	2	11	19	43	35	5	1	0	0	0	117
SSE	0	0	2	10	9	28	45	10	1	0	0	0	105
S	0	0	3	8	29	70	47	4	0	0	0	0	161
SSW	0	1	0	7	26	70	84	8	1	0	0	0	197
SW	0	0	0	11	22	74	127	21	3	0	0	0	258
WSW	0	1	0	11	24	94	101	23	1	0	0	0	255
W	0	0	3	10	27	110	138	41	5	0	0	0	334
WNW	0	0	0	8	22	53	71	43	7	0	0	0	204
NW	0	2	1	3	24	68	66	14	4	0	0	0	182
NNW	2	1	2	4	20	81	67	1	0	0	0	0	178
Totals	2	7	24	126	333	1088	1273	210	25	0	0	0	3088

Number of Calm Hours for this Table	1
Number of Variable Direction Hours for this Table	114
Number of Invalid Hours	1633
Number of Valid Hours for this Table	3088
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 4 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: D Delta Temperature Neutral

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	7	13	78	137	345	215	29	0	0	0	0	824
NNE	2	6	8	72	106	278	209	32	2	0	0	0	715
NE	3	4	15	57	99	342	507	75	1	0	0	0	1103
ENE	1	2	12	61	95	303	454	87	4	1	0	0	1020
E	1	10	18	67	114	268	215	21	3	0	0	0	717
ESE	3	5	14	49	71	165	124	9	0	0	0	0	440
SE	1	16	9	48	80	138	149	39	2	0	0	0	482
SSE	4	9	17	65	96	186	152	18	0	0	0	0	547
S	2	9	14	78	114	240	125	10	0	0	0	0	592
SSW	1	9	21	47	96	229	219	38	3	0	0	0	663
SW	3	3	14	83	117	269	238	40	7	0	0	0	774
WSW	1	8	18	68	141	294	246	53	2	1	0	0	832
W	1	4	11	72	123	269	334	81	16	0	0	0	911
WNW	6	3	19	59	109	222	287	83	14	0	0	0	802
NW	2	4	11	69	97	212	123	31	4	0	0	0	553
NNW	0	3	12	60	98	244	154	17	0	0	0	0	588
Totals	31	102	226	1033	1693	4004	3751	663	58	2	0	0	11563

Number of Calm Hours for this Table	4
Number of Variable Direction Hours for this Table	543
Number of Invalid Hours	1633
Number of Valid Hours for this Table	11563
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 5 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: E Delta Temperature Slightly Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	9	16	26	87	94	154	108	12	1	0	0	0	507
NNE	9	11	37	89	93	224	112	13	1	0	0	0	589
NE	9	20	26	88	124	338	272	23	3	0	0	0	903
ENE	12	14	33	94	149	327	206	29	6	1	0	0	871
E	7	23	38	95	164	330	114	19	2	0	0	0	792
ESE	12	8	50	123	184	246	86	14	0	0	0	0	723
SE	13	21	45	110	184	293	160	9	0	0	0	0	835
SSE	13	25	47	167	250	322	101	8	0	0	0	0	933
S	10	21	60	239	233	271	76	9	1	0	0	0	920
SSW	3	21	43	151	200	272	135	17	1	0	0	0	843
SW	8	18	53	167	245	335	170	13	1	0	0	0	1010
WSW	9	18	40	191	223	266	82	10	1	0	0	0	840
W	5	13	59	127	156	281	169	15	0	0	0	0	825
WNW	9	11	22	113	122	216	185	29	1	0	0	0	708
NW	8	14	27	102	107	147	84	9	1	0	0	0	499
NNW	7	8	21	57	85	128	75	6	2	0	0	0	389
Totals	143	262	627	2000	2613	4150	2135	235	21	1	0	0	12187

Number of Calm Hours for this Table	35
Number of Variable Direction Hours for this Table	396
Number of Invalid Hours	1633
Number of Valid Hours for this Table	12187
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 6 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: F Delta Temperature Moderately Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	11	20	20	56	61	65	15	0	0	0	0	0	248
NNE	16	21	30	62	44	61	25	0	0	0	0	0	259
NE	22	15	24	70	71	97	19	0	0	0	0	0	318
ENE	17	29	27	77	86	162	24	1	0	0	0	0	423
E	16	28	45	103	128	117	5	0	0	0	0	0	442
ESE	16	25	37	94	112	69	2	0	0	0	0	0	355
SE	21	17	35	85	112	52	6	0	0	0	0	0	328
SSE	15	28	30	88	106	65	7	0	0	0	0	0	339
S	12	22	47	143	111	55	0	1	0	0	0	0	391
SSW	20	14	36	138	135	88	10	0	0	0	0	0	441
SW	19	24	36	148	224	99	7	0	0	0	0	0	557
WSW	12	19	47	183	228	110	1	0	0	0	0	0	600
W	10	18	50	169	129	64	9	1	0	0	0	0	450
WNW	10	24	30	103	110	45	11	3	0	0	0	0	336
NW	6	16	21	66	57	34	3	0	0	0	0	0	203
NNW	12	14	18	44	49	38	7	0	0	0	0	0	182
Totals	235	334	533	1629	1763	1221	151	6	0	0	0	0	5872

Number of Calm Hours for this Table	39
Number of Variable Direction Hours for this Table	230
Number of Invalid Hours	1633
Number of Valid Hours for this Table	5872
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 7 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Stability Class: G Delta Temperature Extremely Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	26	31	49	75	46	18	5	0	0	0	0	0	250
NNE	25	26	34	33	13	16	1	0	1	0	0	0	149
NE	45	30	35	58	24	16	0	0	0	0	0	0	208
ENE	29	26	42	73	61	36	2	0	0	0	0	0	269
E	28	33	55	101	78	30	3	0	0	0	0	0	328
ESE	28	33	56	110	40	17	1	0	0	0	0	0	285
SE	21	31	39	48	48	20	3	0	0	0	0	0	210
SSE	20	34	43	46	36	14	2	0	0	0	0	0	195
S	15	20	41	58	47	22	1	0	1	0	0	0	205
SSW	24	22	56	104	111	49	5	0	0	0	0	0	371
SW	32	34	56	150	203	68	2	0	0	0	0	0	545
WSW	19	38	61	207	170	50	2	0	0	0	0	0	547
W	25	36	78	178	133	42	0	0	0	0	0	0	492
WNW	26	34	43	83	56	14	2	1	0	0	0	0	259
NW	35	32	32	41	21	6	0	0	0	0	0	0	167
NNW	22	25	45	81	28	16	1	0	0	0	0	0	218
Totals	420	486	765	1446	1115	434	30	1	2	0	0	0	4698

Number of Calm Hours for this Table	67
Number of Variable Direction Hours for this Table	432
Number of Invalid Hours	1633
Number of Valid Hours for this Table	4698
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-212 (Sheet 8 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (10-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP10M **Direction:** DI10M **Lapse:** DT60M

Summary of All Stability Classes Delta Temperature

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	46	75	110	310	378	740	554	55	2	1	0	0	2271
NNE	52	64	115	270	303	737	488	69	6	0	0	0	2104
NE	79	70	100	289	346	929	997	135	4	0	0	0	2949
ENE	60	71	118	315	434	1031	997	187	10	2	0	0	3225
E	52	94	157	385	524	924	640	59	6	0	0	0	2841
ESE	59	71	161	395	445	646	334	27	0	0	0	0	2138
SE	56	87	135	313	462	610	424	59	3	0	0	0	2149
SSE	52	96	140	392	513	668	361	39	2	0	0	0	2263
S	39	74	165	538	568	730	327	33	2	0	0	0	2476
SSW	48	67	158	456	604	820	601	80	7	0	0	0	2841
SW	62	79	162	576	847	967	736	113	16	0	0	0	3558
WSW	41	84	168	670	816	946	663	130	8	1	0	0	3527
W	41	73	201	562	593	905	902	186	26	0	0	0	3489
WNW	51	72	114	373	435	626	719	203	29	1	0	0	2623
NW	51	68	92	290	320	528	375	80	14	0	0	0	1818
NNW	44	51	99	249	297	567	433	29	2	0	0	0	1771
Totals	833	1196	2195	6383	7885	12374	9551	1484	137	5	0	0	42043

Number of Calm Hours for this Table	147
Number of Variable Direction Hours for this Table	1770
Number of Invalid Hours	1633
Number of Valid Hours for this Table	42043
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 1 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: A Delta Temperature Extremely Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	0	0	4	5	22	36	33	6	0	0	0	106
NNE	0	0	0	1	6	24	34	21	9	1	0	0	96
NE	0	0	0	0	4	23	84	88	28	0	0	0	227
ENE	0	0	1	3	7	35	141	71	15	1	0	0	274
E	0	0	0	1	2	31	86	26	2	0	0	0	148
ESE	1	0	0	4	3	19	52	21	1	0	0	0	101
SE	0	0	0	2	2	10	31	7	0	0	0	0	52
SSE	0	0	1	2	4	27	49	14	1	0	0	0	98
S	0	0	2	4	6	15	51	32	8	0	0	0	118
SSW	0	0	0	2	11	27	80	51	23	3	0	0	197
SW	0	0	0	3	14	33	98	110	60	13	0	0	331
WSW	0	1	1	2	9	26	96	104	76	15	5	9	335
W	0	1	0	2	9	34	57	48	46	5	0	0	202
WNW	0	0	1	2	1	12	37	37	12	7	0	0	109
NW	0	0	0	2	10	19	46	30	4	1	2	0	114
NNW	0	0	1	0	5	22	47	33	2	0	0	0	110
Totals	1	2	7	34	98	379	1025	726	293	46	7	0	2618

Number of Calm Hours for this Table	0
Number of Variable Direction Hours for this Table	6
Number of Invalid Hours	3217
Number of Valid Hours for this Table	2618
Total Hours for the Period	43823

Note: Stability class based on temperature difference (∆T or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 2 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: B Delta Temperature Moderately Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	0	1	4	4	17	48	20	8	0	0	0	102
NNE	0	0	0	1	5	15	33	22	5	0	0	0	81
NE	0	1	0	4	1	20	60	46	12	0	0	0	144
ENE	0	0	0	2	3	23	67	35	4	0	0	0	134
E	0	0	0	2	3	18	43	21	1	0	0	0	88
ESE	0	0	0	1	2	18	27	10	0	0	0	0	58
SE	0	0	1	0	3	12	20	10	0	0	0	0	46
SSE	0	0	0	3	1	15	19	5	0	0	0	0	43
S	0	0	0	1	4	15	29	11	8	0	0	0	68
SSW	0	0	1	1	1	17	48	22	18	1	1	0	110
SW	0	0	0	0	8	28	80	46	35	4	1	0	202
WSW	0	0	0	1	6	26	73	49	35	7	1	0	198
W	0	0	0	1	6	17	67	48	29	12	0	0	180
WNW	0	0	0	0	3	14	45	26	17	7	2	0	115
NW	0	0	0	2	4	17	52	27	8	1	0	0	111
NNW	0	0	0	0	5	18	53	28	2	0	0	0	106
Totals	0	1	3	23	59	290	765	426	182	32	5	0	1786

Number of Calm Hours for this Table	0
Number of Variable Direction Hours for this Table	26
Number of Invalid Hours	3217
Number of Valid Hours for this Table	1786
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 3 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: C Delta Temperature Slightly Unstable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	0	0	0	2	12	54	80	27	4	0	0	0	179
NNE	1	1	1	3	10	34	62	24	8	0	0	0	144
NE	0	2	0	6	7	36	99	48	6	0	0	0	204
ENE	0	0	2	5	8	45	97	49	8	0	0	0	214
E	0	0	0	6	11	44	100	16	2	1	0	0	180
ESE	0	0	1	6	5	18	34	11	0	1	0	0	76
SE	0	0	1	1	7	19	41	14	2	0	0	0	85
SSE	0	0	0	6	5	26	51	13	6	1	0	0	108
S	0	0	0	4	13	38	63	21	10	0	0	0	149
SSW	0	0	0	4	9	37	85	38	13	3	0	0	189
SW	0	0	2	3	4	49	102	73	34	7	0	0	274
WSW	0	1	0	5	9	52	122	60	41	6	1	0	297
W	0	1	1	1	12	47	111	54	44	11	1	0	283
WNW	0	0	0	4	5	34	69	43	26	12	2	0	195
NW	0	0	1	5	12	40	92	30	5	2	0	0	187
NNW	0	1	3	5	4	46	89	22	5	0	0	0	175
Totals	1	6	12	66	133	619	1297	543	214	44	4	0	2939

Number of Calm Hours for this Table	0
Number of Variable Direction Hours for this Table	60
Number of Invalid Hours	3217
Number of Valid Hours for this Table	2939
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 4 of 8) Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: D Delta Temperature Neutral

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	2	2	1	24	47	152	291	114	39	2	0	0	674
NNE	0	4	9	24	49	129	319	182	62	2	0	0	780
NE	0	3	5	25	42	147	425	382	125	1	0	0	1155
ENE	1	1	8	27	59	158	352	199	47	3	2	0	857
E	1	4	6	24	40	115	237	91	27	1	0	0	546
ESE	2	0	6	21	32	76	134	50	12	2	0	0	335
SE	2	2	9	20	38	72	170	100	41	1	0	0	455
SSE	1	5	7	23	43	114	210	109	22	0	0	0	534
S	1	4	4	29	59	148	233	100	22	3	0	0	603
SSW	2	3	7	19	36	102	231	138	57	12	1	0	608
SW	1	3	6	22	48	135	307	186	111	13	1	0	833
WSW	2	3	6	23	37	149	299	253	155	22	2	0	951
W	0	4	9	24	45	143	286	212	166	46	8	0	943
WNW	0	5	6	26	33	93	189	139	93	21	0	0	605
NW	0	2	11	18	34	122	206	109	31	5	0	0	538
NNW	2	2	5	22	42	158	258	109	45	1	0	0	644
Totals	17	47	105	371	684	2013	4147	2473	1055	135	14	0	11061

Number of Calm Hours for this Table	0
Number of Variable Direction Hours for this Table	257
Number of Invalid Hours	3217
Number of Valid Hours for this Table	11061
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 5 of 8) Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: E Delta Temperature Slightly Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	3	2	7	18	17	91	205	107	21	8	0	0	479
NNE	0	0	3	20	25	93	248	212	58	0	0	0	659
NE	2	1	4	12	32	87	331	373	122	4	0	0	968
ENE	1	1	4	19	31	89	347	277	50	4	3	0	826
E	1	2	4	15	21	82	312	204	27	3	0	0	671
ESE	1	2	6	16	24	71	289	221	24	1	0	0	655
SE	0	1	6	9	16	81	345	215	18	0	0	0	691
SSE	0	4	6	31	48	196	513	163	11	1	0	0	973
S	0	3	5	25	41	179	421	222	29	2	1	0	928
SSW	1	3	6	13	21	90	371	336	57	3	0	0	901
SW	1	4	3	18	27	71	419	368	98	7	0	0	1016
WSW	2	2	2	11	25	64	310	288	106	9	0	0	819
W	3	3	5	13	26	48	253	364	146	10	1	0	872
WNW	5	1	6	11	15	61	170	204	112	9	0	0	591
NW	1	3	3	16	14	60	169	147	41	2	0	0	456
NNW	1	0	8	15	25	61	131	91	17	3	1	0	353
Totals	19	32	78	262	408	1424	4834	3792	937	66	6	0	11858

Number of Calm Hours for this Table	8
Number of Variable Direction Hours for this Table	83
Number of Invalid Hours	3217
Number of Valid Hours for this Table	11858
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 6 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: F Delta Temperature Moderately Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	1	3	1	8	9	39	78	43	8	0	0	0	190
NNE	0	0	1	3	13	39	117	68	15	0	0	0	256
NE	1	2	0	8	9	39	100	156	33	0	0	0	348
ENE	2	1	1	8	16	27	150	174	26	0	0	0	405
E	1	1	2	8	7	30	163	142	2	0	0	0	356
ESE	3	2	1	13	14	44	157	89	3	0	0	0	326
SE	1	1	3	6	15	41	157	85	6	0	0	0	315
SSE	1	2	4	18	27	94	142	94	5	0	0	0	387
S	1	1	11	25	30	80	156	149	8	0	0	0	461
SSW	1	5	3	4	8	47	187	212	28	0	0	0	495
SW	3	1	5	10	15	40	156	280	44	0	0	0	554
WSW	0	0	3	8	11	26	150	242	37	1	0	0	478
W	2	1	4	6	14	29	133	216	49	0	0	0	454
WNW	1	0	2	7	13	31	89	142	31	0	0	0	316
NW	0	0	3	5	8	30	87	80	5	0	0	0	218
NNW	2	2	2	4	9	27	75	51	7	0	0	0	179
Totals	20	22	46	141	218	663	2097	2223	307	1	0	0	5738

Number of Calm Hours for this Table	4
Number of Variable Direction Hours for this Table	14
Number of Invalid Hours	3217
Number of Valid Hours for this Table	5738
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 7 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Stability Class: G Delta Temperature Extremely Stable

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	2	2	4	10	16	32	69	17	0	0	1	0	153
NNE	2	2	2	12	15	56	86	17	1	0	0	0	193
NE	1	1	7	15	22	37	90	55	7	0	0	0	235
ENE	0	3	8	13	12	40	118	88	20	0	0	0	302
E	0	4	3	9	13	24	123	97	10	0	0	0	283
ESE	2	2	5	7	8	28	111	72	1	0	0	0	236
SE	1	1	4	9	20	38	90	43	2	0	0	0	208
SSE	1	2	7	17	29	76	82	39	4	0	0	0	257
S	1	1	7	18	33	70	113	94	27	0	0	0	364
SSW	1	3	5	13	12	34	135	172	45	0	0	0	420
SW	1	0	2	9	13	43	147	171	58	0	0	0	444
WSW	4	1	2	7	15	41	103	216	37	0	0	0	426
W	4	5	3	12	15	47	126	159	33	0	0	0	404
WNW	1	3	3	8	10	41	102	90	11	0	0	0	269
NW	1	1	6	11	12	47	98	50	4	0	0	0	230
NNW	0	0	3	8	16	44	57	31	2	0	0	0	161
Totals	22	31	71	178	261	698	1650	1411	262	0	1	0	4585

Number of Calm Hours for this Table	9
Number of Variable Direction Hours for this Table	42
Number of Invalid Hours	3217
Number of Valid Hours for this Table	4585
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-213 (Sheet 8 of 8)
Joint Frequency Distribution of Wind Speed and Wind Direction (60-m Level) by Atmospheric Stability Class for the VEGP Site (1998–2002)

Hours at Each Wind Speed and Direction

Period of Record: 01/01/98 1:00 - 12/31/02 23:00 Total Period

Elevation: **Speed:** SP60M **Direction:** DI60M **Lapse:** DT60M

Summary of All Stability Classes Delta Temperature

Wind Speed (m/s)

Wind Direction (from)	0.23 - 0.50	0.51 - 0.75	0.76 - 1.0	1.1 - 1.5	1.6 - 2.0	2.1 - 3.0	3.1 - 5.0	5.1 - 7.0	7.1 - 10.0	10.1 - 13.0	13.1 - 18.0	> 18.0	Total
N	8	9	14	70	110	407	807	361	86	10	1	0	1883
NNE	3	7	16	64	123	390	899	546	158	3	0	0	2209
NE	4	10	16	70	117	389	1189	1148	333	5	0	0	3281
ENE	4	6	24	77	136	417	1272	893	170	8	5	0	3012
E	3	11	15	65	97	344	1064	597	71	5	0	0	2272
ESE	9	6	19	68	88	274	804	474	41	4	0	0	1787
SE	4	5	24	47	101	273	854	474	69	1	0	0	1852
SSE	3	13	25	100	157	548	1066	437	49	2	0	0	2400
S	3	9	29	106	186	545	1066	629	112	5	1	0	2691
SSW	5	14	22	56	98	354	1137	969	241	22	2	0	2920
SW	6	8	18	65	129	399	1309	1234	440	44	2	0	3654
WSW	8	8	14	57	112	384	1153	1212	487	60	9	0	3504
W	9	15	22	59	127	365	1033	1101	513	84	10	0	3338
WNW	4	9	18	58	80	286	702	681	302	56	4	0	2200
NW	2	6	24	59	94	335	750	473	98	11	2	0	1854
NNW	5	5	22	54	106	376	710	365	80	4	1	0	1728
Totals	80	141	322	1075	1861	6086	15815	11594	3250	324	37	0	40585

Number of Calm Hours for this Table	21
Number of Variable Direction Hours for this Table	488
Number of Invalid Hours	3217
Number of Valid Hours for this Table	40585
Total Hours for the Period	43823

Note: Stability class based on temperature difference (ΔT or lapse) between 10-m and 60-m measurement levels.

Table 2.3-214
VEGP Onsite Weather Instruments

60-m Tower Instruments						
Sensed Parameter	**Range**	**System Accuracy**	**Starting Threshold**	**Distance Constant**	**Damping Ratio**	**Elevation**
Wind Speed	0-112 mph (0-50 m/sec)	≤ ±5%	1.0 mph (0.45 m/sec)	2 m	-----	10 m; 60 m
Wind Direction	0°-360°	≤ ±5°	1.0 mph (0.45 m/sec)	-----	0.4-0.6 with deflection of 10° and delay distance of ≤ 2 m	10 m; 60 m
Ambient Temperature	-58°F to +122°F (-50°C to +50°C)	≤ ±0.9°F (±0.5°C)	-----	-----	-----	10 m; 60 m
Differential Temperature	-10°F to +20°F (-23.3°C to -6.6°C)	≤ ±0.18°F (±0.1°C)	-----	-----	-----	10 m – 60 m
Relative Humidity	0-100%	≤ ±4%	-----	-----	-----	10 m
Dew Point	40°F to 120°F (40°C to 49°C)	≤ ±2.7°F (±1.5°C)	-----	-----	-----	10 m
Precipitation	0-100 in. reset daily	±10% of the total accumulated catch	Resolution of 0.01 in. (0.25 mm)	-----	-----	Tower base
Sigma-Theta	0°-100°	-----	-----	-----	See wind direction	10 m; 60 m

Note: Redundant primary and secondary instruments are installed for each sensed parameter.

Table 2.3-215
Annual Data Recovery Statistics - VEGP 60-Meter Meteorological Tower (1998-2002)

Parameter	**1998**	**1999**	**2000**	**2001**	**2002**
Wind Speed (10m)	99.0	99.0	97.8	95.1	97.1
Wind Speed (60 m)	98.4	98.1	97.7	95.2	96.7
Wind Direction (10 m)	99.1	98.9	98.4	95.2	96.4
Wind Direction (60 m)	88.2	93.3	96.6	95.3	97.6
Δ -Temperature (60m – 10m)[a]	96.6	98.6	97.2	94.9	99.3 [b]
Temperature (10 m)	99.2	98.9	97.8	95.0	97.6 [b]
Dewpoint (10 m)	99.0	98.3	85.5	95.1	89.6
Rainfall	99.5	99.3	99.1	96.3	78.8
Composite Parameters					
WS/WD (10m), ΔT (60m-10m) [a]	96.4	98.3	96.5	94.9	95.3
WS/WD (60m), ΔT (60m-10m) [a]	85.6	91.9	94.8	94.9	96.1

Notes:

a – Temperature difference (ΔT) between 10-m and 60-m levels.

b – Data recovery for Δ-Temperature is greater than the 10-m temperature parameter recovery rate due to data substitution by SNC in the 2002 data set for the ΔT parameter only.

Table 2.3-216
PAVAN Output – χ/Q Values at the Dose Calculation EAB

/ PLANT NAME: Vogtle COL
DATA PERIOD: 1998–2002 JFD
TYPE OF RELEASE: Ground-Level Release
SOURCE OF DATA: Onsite
COMMENTS: Accidental Releases
PROGRAM: PAVAN, 10/76, 8/79 REVISION, IMPLEMENTATION OF REGULATORY GUIDE 1.145

METEOROLOGICAL INSTRUMENTATION
WIND SENSORS HEIGHT: 10 m
DELTA-T HEIGHTS: 10 m – 60 m

0 RELATIVE CONCENTRATION (X/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0–2 HOURS	0–8 HOURS	8–24 HOURS	1–4 DAYS	4–30 DAYS	ANNUAL AVERAGE	HOURS PER YEAR MAX 0–2 HR X/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR
S	800.	2.53E–04	1.69E–04	1.38E–04	8.89E–05	4.73E–05	2.19E–05	29.9	S
SSW	800.	2.22E–04	1.49E–04	1.21E–04	7.84E–05	4.18E–05	1.94E–05	530.6	SSW
SW	800.	2.59E–04	1.77E–04	1.46E–04	9.61E–05	5.29E–05	2.54E–05	34.4	SW
WSW	800.	2.67E–04	1.82E–04	1.50E–04	9.95E–05	5.49E–05	2.65E–05	32.5	WSW
W	800.	2.88E–04	1.97E–04	1.63E–04	1.08E–04	5.94E–05	2.88E–05	36.9	W
WNW	800.	2.85E–04	1.92E–04	1.57E–04	1.02E–04	5.52E–05	2.59E–05	36.4	WNW
NW	800.	2.47E–04	1.67E–04	1.37E–04	8.97E–05	4.87E–05	2.30E–05	30.0	NW
NNW	800.	2.45E–04	1.67E–04	1.38E–04	9.16E–05	5.06E–05	2.45E–05	29.3	NNW
N	800.	2.42E–04	1.67E–04	1.39E–04	9.26E–05	5.20E–05	2.57E–05	25.6	N
NNE	800.	2.78E–04	1.92E–04	1.59E–04	1.06E–04	5.92E–05	2.91E–05	34.1	NNE
NE	800.	3.14E–04	2.21E–04	1.85E–04	1.26E–04	7.30E–05	3.73E–05	43.7	NE
ENE	800.	2.95E–04	2.10E–04	1.77E–04	1.22E–04	7.18E–05	3.74E–05	36.7	ENE
E	800.	3.03E–04	2.11E–04	1.77E–04	1.19E–04	6.82E–05	3.43E–05	40.1	E
ESE	800.	2.59E–04	1.75E–04	1.44E–04	9.45E–05	5.14E–05	2.44E–05	31.2	ESE
SE	800.	2.11E–04	1.42E–04	1.16E–04	7.51E–05	4.03E–05	1.88E–05	26.5	SE
SSE	800.	2.39E–04	1.56E–04	1.26E–04	7.91E–05	4.07E–05	1.80E–05	26.6	SSE
MAX X/Q		3.14E–04				TOTAL HOURS AROUND SITE:		*****	
SRP 2.3.4	800.	1.86E–03	9.75E–04	7.06E–04	3.50E–04	1.28E–04	3.74E–05		
SITE LIMIT		3.49E–04	2.41E–04	2.00E–04	1.34E–04	7.56E–05	3.74E–05		

0THE FIVE-PERCENT-FOR-THE-ENTIRE-SITE X/Q IS LIMITING.

Table 2.3-217
PAVAN Output – χ/Q Values at the LPZ

/ PLANT NAME: Vogtle COL
DATA PERIOD: 1998–2002 JFD
TYPE OF RELEASE: Ground-Level Release
SOURCE OF DATA: Onsite
COMMENTS: Accidental Release

METEOROLOGICAL INSTRUMENTATION
WIND SENSORS HEIGHT: 10 m
DELTA-T HEIGHTS: 10 m – 60 m

PROGRAM: PAVAN, 10/76, 8/79 REVISION, IMPLEMENTATION OF REGULATORY GUIDE 1.145

0 RELATIVE CONCENTRATION (X/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0–2 HOURS	0–8 HOURS	8–24 HOURS	1–4 DAYS	4–30 DAYS	ANNUAL AVERAGE	HOURS PER YEAR MAX 0–2 HR X/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR
S	2304.	8.95E–05	4.82E–05	3.54E–05	1.81E–05	6.89E–06	2.12E–06	29.6	S
SSW	2304.	7.42E–05	4.04E–05	2.98E–05	1.54E–05	5.97E–06	1.87E–06	502.4	SSW
SW	2304.	8.94E–05	4.94E–05	3.67E–05	1.93E–05	7.64E–06	2.47E–06	32.2	SW
WSW	2304.	9.17E–05	5.08E–05	3.78E–05	1.99E–05	7.92E–06	2.57E–06	29.5	WSW
W	2304.	1.02E–04	5.61E–05	4.17E–05	2.19E–05	8.66E–06	2.79E–06	34.5	W
WNW	2304.	1.03E–04	5.56E–05	4.09E–05	2.10E–05	8.10E–06	2.52E–06	34.9	WNW
NW	2304.	8.52E–05	4.67E–05	3.45E–05	1.80E–05	7.04E–06	2.24E–06	28.3	NW
NNW	2304.	8.53E–05	4.72E–05	3.51E–05	1.85E–05	7.36E–06	2.38E–06	28.5	NNW
N	2304.	8.32E–05	4.66E–05	3.48E–05	1.86E–05	7.52E–06	2.49E–06	24.1	N
NNE	2304.	1.00E–04	5.55E–05	4.13E–05	2.18E–05	8.68E–06	2.82E–06	32.8	NNE
NE	2304.	1.11E–04	6.28E–05	4.73E–05	2.56E–05	1.06E–05	3.61E–06	39.5	NE
ENE	2304.	1.17E–04	6.59E–05	4.95E–05	2.65E–05	1.08E–05	3.63E–06	43.7	ENE
E	2304.	1.15E–04	6.40E–05	4.78E–05	2.53E–05	1.01E–05	3.32E–06	42.3	E
ESE	2304.	9.13E–05	4.99E–05	3.69E–05	1.92E–05	7.48E–06	2.36E–06	29.4	ESE
SE	2304.	7.34E–05	3.98E–05	2.94E–05	1.51E–05	5.83E–06	1.82E–06	25.8	SE
SSE	2304.	8.49E–05	4.47E–05	3.24E–05	1.61E–05	5.92E–06	1.74E–06	26.6	SSE
MAX X/Q		1.17E–04				TOTAL HOURS AROUND SITE:		984.3	
SRP 2.3.4	2304.	3.47E–04	1.63E–04	1.12E–04	4.94E–05	1.53E–05	3.63E–06		
SITE LIMIT		1.27E–04	7.04E–05	5.25E–05	2.77E–05	1.11E–05	3.63E–06		

0THE FIVE-PERCENT-FOR-THE-ENTIRE-SITE X/Q IS LIMITING.

Table 2.3-218
Shortest Distances Between the VEGP Units 3 and 4 Power Block Area and Receptors of Interest by Downwind Direction Sector[a]

Direction	Meat Animal	Residence	Vegetable Garden	EAB[b]
N	1,071	1,071	1,071	800
NNE	1,071	1,071	1,071	800
NE	1,071	1,071	1,071	800
ENE	1,071	1,071	1,071	800
E	1,071	1,071	1,071	800
ESE	1,071	1,071	1,071	800
SE	1,071	1,071	1,071	800
SSE	1,071	1,071	1,071	800
S	1,071	1,071	1,071	800
SSW	1,071	1,071	1,071	800
SW	1,071	1,071	1,071	800
WSW	1,071	1,071	1,071	800
W	1,071	1,071	1,071	800
WNW	1,071	1,071	1,071	800
NW	1,071	1,071	1,071	800
NNW	1,071	1,071	1,071	800

Notes:
a – Distances shown are in meters
b – EAB = Exclusion Area Boundary
c – There are no milk-giving animals (i.e., cows, goats) within a 5-mile radius of the VEGP Units 3 and 4 Site.

Table 2.3-219
XOQDOQ-Predicted Maximum χ/Q and D/Q Values at Receptors of Interest

Type of Location	Direction from Site	Distance meters/ (miles)	χ/Q (sec/m^3) (No Decay) (Undepleted)	χ/Q (sec/m^3) (2.26 Day Decay) (Undepleted)	χ/Q (sec/m^3) (8 Day Decay) (Depleted)	D/Q ($1/m^2$)
Residence	NE	1,071 (0.67)	3.4E-06	3.4E-06	3.0E-06	1.0E-08[a]
Dose Calculation EAB	NE	800 (0.5)	5.5E-06	5.5E-06	5.0E-06	1.7E-08[b]
Meat Animal	NE	1,071 (0.67)	3.4E-06	3.4E-06	3.0E-06	1.0E-08[a]
Vegetable Garden	NE	1,071 (0.67)	3.4E-06	3.4E-06	3.0E-06	1.0E-08[a]

Notes:
a – NE, ENE, and E
b – NE and ENE

Table 2.3-220 (Sheet 1 of 4)
XOQDOQ-Predicted Annual Average χ/Q and D/Q Values at the Standard Radial Distances and Distance-Segment Boundaries

No Decay X/Qs at Various Distances

EXIT ONE - GROUND LEVEL RELEASE - NO PURGE RELEASES
NO DECAY, UNDEPLETED

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED) — DISTANCE IN MILES FROM THE SITE

SECTOR	0.250	0.500	0.750	1.000	1.500	2.000	2.500	3.000	3.500	4.000	4.500
S	1.097E-05	3.306E-06	1.697E-06	1.088E-06	6.032E-07	3.998E-07	2.971E-07	2.339E-07	1.912E-07	1.606E-07	1.377E-07
SSW	9.903E-06	2.986E-06	1.546E-06	9.958E-07	5.570E-07	3.707E-07	2.750E-07	2.160E-07	1.762E-07	1.478E-07	1.265E-07
SW	1.326E-05	3.993E-06	2.063E-06	1.328E-06	7.408E-07	4.926E-07	3.660E-07	2.881E-07	2.353E-07	1.976E-07	1.694E-07
WSW	1.342E-05	4.026E-06	2.076E-06	1.336E-06	7.479E-07	4.982E-07	3.702E-07	2.912E-07	2.378E-07	1.996E-07	1.711E-07
W	1.421E-05	4.237E-06	2.168E-06	1.392E-06	7.796E-07	5.201E-07	3.877E-07	3.059E-07	2.504E-07	2.106E-07	1.808E-07
WNW	1.282E-05	3.803E-06	1.947E-06	1.251E-06	7.014E-07	4.684E-07	3.498E-07	2.764E-07	2.266E-07	1.908E-07	1.639E-07
NW	1.157E-05	3.450E-06	1.790E-06	1.156E-06	6.516E-07	4.357E-07	3.241E-07	2.552E-07	2.086E-07	1.751E-07	1.502E-07
NNW	1.210E-05	3.626E-06	1.899E-06	1.231E-06	6.940E-07	4.637E-07	3.443E-07	2.706E-07	2.208E-07	1.852E-07	1.586E-07
N	1.239E-05	3.719E-06	1.951E-06	1.266E-06	7.147E-07	4.779E-07	3.543E-07	2.781E-07	2.266E-07	1.898E-07	1.624E-07
NNE	1.424E-05	4.240E-06	2.171E-06	1.395E-06	7.821E-07	5.221E-07	3.892E-07	3.071E-07	2.515E-07	2.115E-07	1.816E-07
NE	1.832E-05	5.438E-06	2.773E-06	1.778E-06	9.945E-07	6.633E-07	4.952E-07	3.914E-07	3.208E-07	2.702E-07	2.322E-07
ENE	1.781E-05	5.295E-06	2.696E-06	1.728E-06	9.670E-07	6.451E-07	4.816E-07	3.805E-07	3.119E-07	2.626E-07	2.257E-07
E	1.645E-05	4.895E-06	2.488E-06	1.591E-06	8.856E-07	5.890E-07	4.395E-07	3.473E-07	2.847E-07	2.397E-07	2.060E-07
ESE	1.211E-05	3.630E-06	1.865E-06	1.198E-06	6.685E-07	4.449E-07	3.310E-07	2.607E-07	2.132E-07	1.791E-07	1.537E-07
SE	9.657E-06	2.893E-06	1.486E-06	9.531E-07	5.289E-07	3.509E-07	2.611E-07	2.058E-07	1.684E-07	1.415E-07	1.215E-07
SSE	9.037E-06	2.711E-06	1.382E-06	8.836E-07	4.892E-07	3.242E-07	2.413E-07	1.903E-07	1.558E-07	1.310E-07	1.125E-07

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED) — DISTANCE IN MILES FROM THE SITE

SECTOR	5.000	7.500	10.000	15.000	20.000	25.000	30.000	35.000	40.000	45.000	50.000
S	1.201E-07	7.112E-08	4.917E-08	2.936E-08	2.045E-08	1.546E-08	1.232E-08	1.018E-08	8.626E-09	7.459E-09	6.552E-09
SSW	1.102E-07	6.491E-08	4.471E-08	2.655E-08	1.841E-08	1.388E-08	1.103E-08	9.093E-09	7.694E-09	6.642E-09	5.826E-09
SW	1.477E-07	8.727E-08	6.025E-08	3.589E-08	2.493E-08	1.883E-08	1.498E-08	1.236E-08	1.046E-08	9.039E-09	7.932E-09
WSW	1.492E-07	8.812E-08	6.081E-08	3.621E-08	2.515E-08	1.899E-08	1.511E-08	1.246E-08	1.055E-08	9.113E-09	7.996E-09
W	1.579E-07	9.376E-08	6.494E-08	3.885E-08	2.707E-08	2.048E-08	1.632E-08	1.348E-08	1.143E-08	9.884E-09	8.682E-09
WNW	1.432E-07	8.529E-08	5.918E-08	3.548E-08	2.475E-08	1.875E-08	1.495E-08	1.236E-08	1.048E-08	9.067E-09	7.967E-09
NW	1.309E-07	7.737E-08	5.339E-08	3.178E-08	2.206E-08	1.664E-08	1.323E-08	1.091E-08	9.232E-09	7.971E-09	6.992E-09
NNW	1.381E-07	8.131E-08	5.597E-08	3.318E-08	2.297E-08	1.730E-08	1.373E-08	1.130E-08	9.553E-09	8.239E-09	7.221E-09
N	1.413E-07	8.295E-08	5.697E-08	3.369E-08	2.328E-08	1.751E-08	1.388E-08	1.142E-08	9.644E-09	8.313E-09	7.281E-09
NNE	1.585E-07	9.419E-08	6.524E-08	3.904E-08	2.720E-08	2.058E-08	1.640E-08	1.355E-08	1.149E-08	9.932E-09	8.724E-09
NE	2.029E-07	1.209E-07	8.394E-08	5.038E-08	3.517E-08	2.666E-08	2.127E-08	1.759E-08	1.492E-08	1.291E-08	1.135E-08
ENE	1.971E-07	1.174E-07	8.150E-08	4.889E-08	3.413E-08	2.586E-08	2.064E-08	1.706E-08	1.447E-08	1.253E-08	1.101E-08
E	1.800E-07	1.073E-07	7.453E-08	4.477E-08	3.129E-08	2.373E-08	1.895E-08	1.568E-08	1.331E-08	1.152E-08	1.013E-08
ESE	1.341E-07	7.943E-08	5.492E-08	3.279E-08	2.282E-08	1.725E-08	1.374E-08	1.134E-08	9.613E-09	8.310E-09	7.297E-09
SE	1.060E-07	6.292E-08	4.357E-08	2.607E-08	1.818E-08	1.376E-08	1.097E-08	9.066E-09	7.689E-09	6.652E-09	5.845E-09
SSE	9.818E-08	5.836E-08	4.046E-08	2.425E-08	1.693E-08	1.283E-08	1.024E-08	8.467E-09	7.186E-09	6.220E-09	5.468E-09

Table 2.3-220 (Sheet 2 of 4)
XOQDOQ-Predicted Annual Average χ/Q and D/Q Values at the Standard Radial Distances and Distance-Segment Boundaries

No Decay X/Qs at Various Segments

EXIT ONE - GROUND LEVEL RELEASE - NO PURGE RELEASES
NO DECAY, UNDEPLETED
CHI/Q (SEC/METER CUBED) FOR EACH SEGMENT

	SEGMENT BOUNDARIES IN MILES FROM THE SITE									
DIRECTION FROM SITE	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	1.784E-06	6.205E-07	2.992E-07	1.917E-07	1.380E-07	7.225E-08	2.980E-08	1.554E-08	1.020E-08	7.469E-09
SSW	1.621E-06	5.717E-07	2.769E-07	1.767E-07	1.268E-07	6.600E-08	2.697E-08	1.395E-08	9.115E-09	6.651E-09
SW	2.165E-06	7.609E-07	3.686E-07	2.360E-07	1.697E-07	8.868E-08	3.643E-08	1.892E-08	1.238E-08	9.051E-09
WSW	2.181E-06	7.677E-07	3.727E-07	2.385E-07	1.714E-07	8.955E-08	3.676E-08	1.908E-08	1.249E-08	9.125E-09
W	2.283E-06	8.003E-07	3.903E-07	2.511E-07	1.812E-07	9.519E-08	3.941E-08	2.057E-08	1.351E-08	9.897E-09
WNW	2.050E-06	7.200E-07	3.521E-07	2.272E-07	1.642E-07	8.656E-08	3.598E-08	1.883E-08	1.238E-08	9.079E-09
NW	1.877E-06	6.678E-07	3.263E-07	2.092E-07	1.504E-07	7.861E-08	3.226E-08	1.672E-08	1.093E-08	7.982E-09
NNW	1.986E-06	7.111E-07	3.467E-07	2.215E-07	1.589E-07	8.267E-08	3.371E-08	1.738E-08	1.133E-08	8.251E-09
N	2.039E-06	7.319E-07	3.568E-07	2.273E-07	1.627E-07	8.438E-08	3.424E-08	1.760E-08	1.145E-08	8.325E-09
NNE	2.286E-06	8.027E-07	3.918E-07	2.521E-07	1.819E-07	9.562E-08	3.960E-08	2.068E-08	1.358E-08	9.945E-09
NE	2.923E-06	1.021E-06	4.985E-07	3.217E-07	2.326E-07	1.227E-07	5.108E-08	2.677E-08	1.763E-08	1.293E-08
ENE	2.843E-06	9.930E-07	4.847E-07	3.127E-07	2.260E-07	1.192E-07	4.958E-08	2.598E-08	1.710E-08	1.254E-08
E	2.624E-06	9.106E-07	4.425E-07	2.854E-07	2.064E-07	1.089E-07	4.539E-08	2.383E-08	1.571E-08	1.154E-08
ESE	1.961E-06	6.867E-07	3.333E-07	2.138E-07	1.540E-07	8.068E-08	3.328E-08	1.733E-08	1.137E-08	8.321E-09
SE	1.562E-06	5.440E-07	2.629E-07	1.688E-07	1.217E-07	6.390E-08	2.645E-08	1.382E-08	9.086E-09	6.660E-09
SSE	1.456E-06	5.035E-07	2.430E-07	1.562E-07	1.127E-07	5.925E-08	2.460E-08	1.289E-08	8.486E-09	6.228E-09

Table 2.3-220 (Sheet 3 of 4)
XOQDOQ-Predicted Annual Average χ/Q and D/Q Values at the Standard Radial Distances and Distance-Segment Boundaries

D/Qs at Various Distances

EXIT ONE - GROUND LEVEL RELEASE - NO PURGE RELEASES

******************** RELATIVE DEPOSITION PER UNIT AREA (M**-2) AT FIXED POINTS BY DOWNWIND SECTORS ********************

DIRECTION	DISTANCES IN MILES										
FROM SITE	0.25	0.50	0.75	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50
S	3.128E-08	1.058E-08	5.431E-09	3.335E-09	1.663E-09	1.008E-09	6.817E-10	4.940E-10	3.756E-10	2.959E-10	2.396E-10
SSW	2.900E-08	9.807E-09	5.035E-09	3.092E-09	1.541E-09	9.348E-10	6.321E-10	4.580E-10	3.483E-10	2.744E-10	2.221E-10
SW	4.066E-08	1.375E-08	7.059E-09	4.334E-09	2.161E-09	1.311E-09	8.861E-10	6.421E-10	4.882E-10	3.847E-10	3.114E-10
WSW	4.440E-08	1.502E-08	7.710E-09	4.734E-09	2.360E-09	1.431E-09	9.678E-10	7.013E-10	5.333E-10	4.201E-10	3.401E-10
W	3.911E-08	1.323E-08	6.791E-09	4.170E-09	2.079E-09	1.261E-09	8.525E-10	6.177E-10	4.697E-10	3.701E-10	2.996E-10
WNW	2.948E-08	9.971E-09	5.119E-09	3.143E-09	1.567E-09	9.505E-10	6.426E-10	4.657E-10	3.541E-10	2.790E-10	2.258E-10
NW	2.963E-08	1.002E-08	5.145E-09	3.159E-09	1.575E-09	9.552E-10	6.458E-10	4.680E-10	3.559E-10	2.804E-10	2.270E-10
NNW	3.119E-08	1.055E-08	5.415E-09	3.325E-09	1.658E-09	1.005E-09	6.797E-10	4.925E-10	3.745E-10	2.951E-10	2.389E-10
N	3.408E-08	1.152E-08	5.917E-09	3.633E-09	1.811E-09	1.099E-09	7.427E-10	5.382E-10	4.092E-10	3.224E-10	2.610E-10
NNE	3.910E-08	1.322E-08	6.789E-09	4.169E-09	2.078E-09	1.260E-09	8.522E-10	6.175E-10	4.696E-10	3.699E-10	2.995E-10
NE	4.897E-08	1.656E-08	8.503E-09	5.221E-09	2.603E-09	1.579E-09	1.067E-09	7.735E-10	5.882E-10	4.634E-10	3.751E-10
ENE	4.850E-08	1.640E-08	8.422E-09	5.171E-09	2.578E-09	1.564E-09	1.057E-09	7.661E-10	5.825E-10	4.589E-10	3.715E-10
E	4.798E-08	1.622E-08	8.330E-09	5.115E-09	2.550E-09	1.547E-09	1.046E-09	7.578E-10	5.762E-10	4.539E-10	3.675E-10
ESE	3.612E-08	1.221E-08	6.271E-09	3.851E-09	1.920E-09	1.164E-09	7.872E-10	5.704E-10	4.338E-10	3.417E-10	2.766E-10
SE	2.507E-08	8.478E-09	4.353E-09	2.673E-09	1.333E-09	8.082E-10	5.464E-10	3.960E-10	3.011E-10	2.372E-10	1.920E-10
SSE	2.440E-08	8.252E-09	4.237E-09	2.602E-09	1.297E-09	7.867E-10	5.319E-10	3.854E-10	2.931E-10	2.309E-10	1.869E-10

DIRECTION	DISTANCES IN MILES										
FROM SITE	5.00	7.50	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00
S	1.982E-10	9.712E-11	6.094E-11	3.080E-11	1.864E-11	1.250E-11	8.956E-12	6.725E-12	5.229E-12	4.177E-12	3.409E-12
SSW	1.837E-10	9.004E-11	5.650E-11	2.856E-11	1.728E-11	1.159E-11	8.304E-12	6.235E-12	4.848E-12	3.873E-12	3.161E-12
SW	2.576E-10	1.262E-10	7.920E-11	4.003E-11	2.423E-11	1.625E-11	1.164E-11	8.741E-12	6.796E-12	5.429E-12	4.431E-12
WSW	2.813E-10	1.379E-10	8.651E-11	4.372E-11	2.646E-11	1.774E-11	1.271E-11	9.547E-12	7.423E-12	5.930E-12	4.840E-12
W	2.478E-10	1.214E-10	7.620E-11	3.851E-11	2.331E-11	1.563E-11	1.120E-11	8.409E-12	6.538E-12	5.223E-12	4.263E-12
WNW	1.868E-10	9.155E-11	5.744E-11	2.903E-11	1.757E-11	1.178E-11	8.442E-12	6.339E-12	4.929E-12	3.937E-12	3.214E-12
NW	1.877E-10	9.200E-11	5.773E-11	2.918E-11	1.766E-11	1.184E-11	8.484E-12	6.371E-12	4.954E-12	3.957E-12	3.230E-12
NNW	1.976E-10	9.683E-11	6.075E-11	3.071E-11	1.859E-11	1.246E-11	8.929E-12	6.705E-12	5.213E-12	4.164E-12	3.399E-12
N	2.159E-10	1.058E-10	6.639E-11	3.356E-11	2.031E-11	1.362E-11	9.757E-12	7.327E-12	5.697E-12	4.551E-12	3.714E-12
NNE	2.477E-10	1.214E-10	7.617E-11	3.850E-11	2.330E-11	1.562E-11	1.120E-11	8.406E-12	6.536E-12	5.221E-12	4.262E-12
NE	3.103E-10	1.521E-10	9.541E-11	4.823E-11	2.919E-11	1.957E-11	1.402E-11	1.053E-11	8.187E-12	6.540E-12	5.338E-12
ENE	3.073E-10	1.506E-10	9.450E-11	4.776E-11	2.891E-11	1.938E-11	1.389E-11	1.043E-11	8.109E-12	6.477E-12	5.287E-12
E	3.040E-10	1.490E-10	9.347E-11	4.724E-11	2.859E-11	1.917E-11	1.374E-11	1.032E-11	8.021E-12	6.407E-12	5.229E-12
ESE	2.288E-10	1.121E-10	7.036E-11	3.557E-11	2.153E-11	1.443E-11	1.034E-11	7.766E-12	6.038E-12	4.823E-12	3.937E-12
SE	1.588E-10	7.784E-11	4.884E-11	2.469E-11	1.494E-11	1.002E-11	7.178E-12	5.390E-12	4.191E-12	3.348E-12	2.733E-12
SSE	1.546E-10	7.577E-11	4.754E-11	2.403E-11	1.454E-11	9.752E-12	6.988E-12	5.247E-12	4.080E-12	3.259E-12	2.660E-12

Table 2.3-220 (Sheet 4 of 4)
XOQDOQ-Predicted Annual Average χ/Q and D/Q Values at the Standard Radial Distances and Distance-Segment Boundaries

D/Qs at Various Segments

EXIT ONE - GROUND LEVEL RELEASE - NO PURGE RELEASES

************************ RELATIVE DEPOSITION PER UNIT AREA (M**-2) BY DOWNWIND SECTORS ************************

	SEGMENT BOUNDARIES IN MILES									
DIRECTION FROM SITE	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	5.643E-09	1.743E-09	6.937E-10	3.791E-10	2.409E-10	1.035E-10	3.209E-11	1.272E-11	6.793E-12	4.204E-12
SSW	5.232E-09	1.616E-09	6.432E-10	3.515E-10	2.234E-10	9.595E-11	2.975E-11	1.179E-11	6.298E-12	3.898E-12
SW	7.335E-09	2.266E-09	9.017E-10	4.927E-10	3.132E-10	1.345E-10	4.171E-11	1.653E-11	8.829E-12	5.465E-12
WSW	8.011E-09	2.475E-09	9.848E-10	5.382E-10	3.420E-10	1.469E-10	4.556E-11	1.806E-11	9.643E-12	5.968E-12
W	7.056E-09	2.180E-09	8.675E-10	4.740E-10	3.013E-10	1.294E-10	4.013E-11	1.591E-11	8.494E-12	5.257E-12
WNW	5.319E-09	1.643E-09	6.539E-10	3.574E-10	2.271E-10	9.756E-11	3.025E-11	1.199E-11	6.403E-12	3.963E-12
NW	5.346E-09	1.652E-09	6.572E-10	3.591E-10	2.283E-10	9.804E-11	3.040E-11	1.205E-11	6.435E-12	3.983E-12
NNW	5.626E-09	1.738E-09	6.917E-10	3.780E-10	2.402E-10	1.032E-10	3.200E-11	1.268E-11	6.772E-12	4.192E-12
N	6.148E-09	1.899E-09	7.558E-10	4.130E-10	2.625E-10	1.128E-10	3.496E-11	1.386E-11	7.400E-12	4.580E-12
NNE	7.054E-09	2.179E-09	8.672E-10	4.739E-10	3.012E-10	1.294E-10	4.012E-11	1.590E-11	8.491E-12	5.255E-12
NE	8.835E-09	2.730E-09	1.086E-09	5.936E-10	3.773E-10	1.620E-10	5.025E-11	1.992E-11	1.064E-11	6.583E-12
ENE	8.751E-09	2.703E-09	1.076E-09	5.879E-10	3.736E-10	1.605E-10	4.977E-11	1.972E-11	1.053E-11	6.520E-12
E	8.656E-09	2.674E-09	1.064E-09	5.815E-10	3.696E-10	1.587E-10	4.923E-11	1.951E-11	1.042E-11	6.449E-12
ESE	6.516E-09	2.013E-09	8.011E-10	4.377E-10	2.782E-10	1.195E-10	3.706E-11	1.469E-11	7.843E-12	4.855E-12
SE	4.523E-09	1.397E-09	5.560E-10	3.039E-10	1.931E-10	8.295E-11	2.572E-11	1.020E-11	5.444E-12	3.370E-12
SSE	4.403E-09	1.360E-09	5.413E-10	2.958E-10	1.880E-10	8.075E-11	2.504E-11	9.924E-12	5.300E-12	3.280E-12

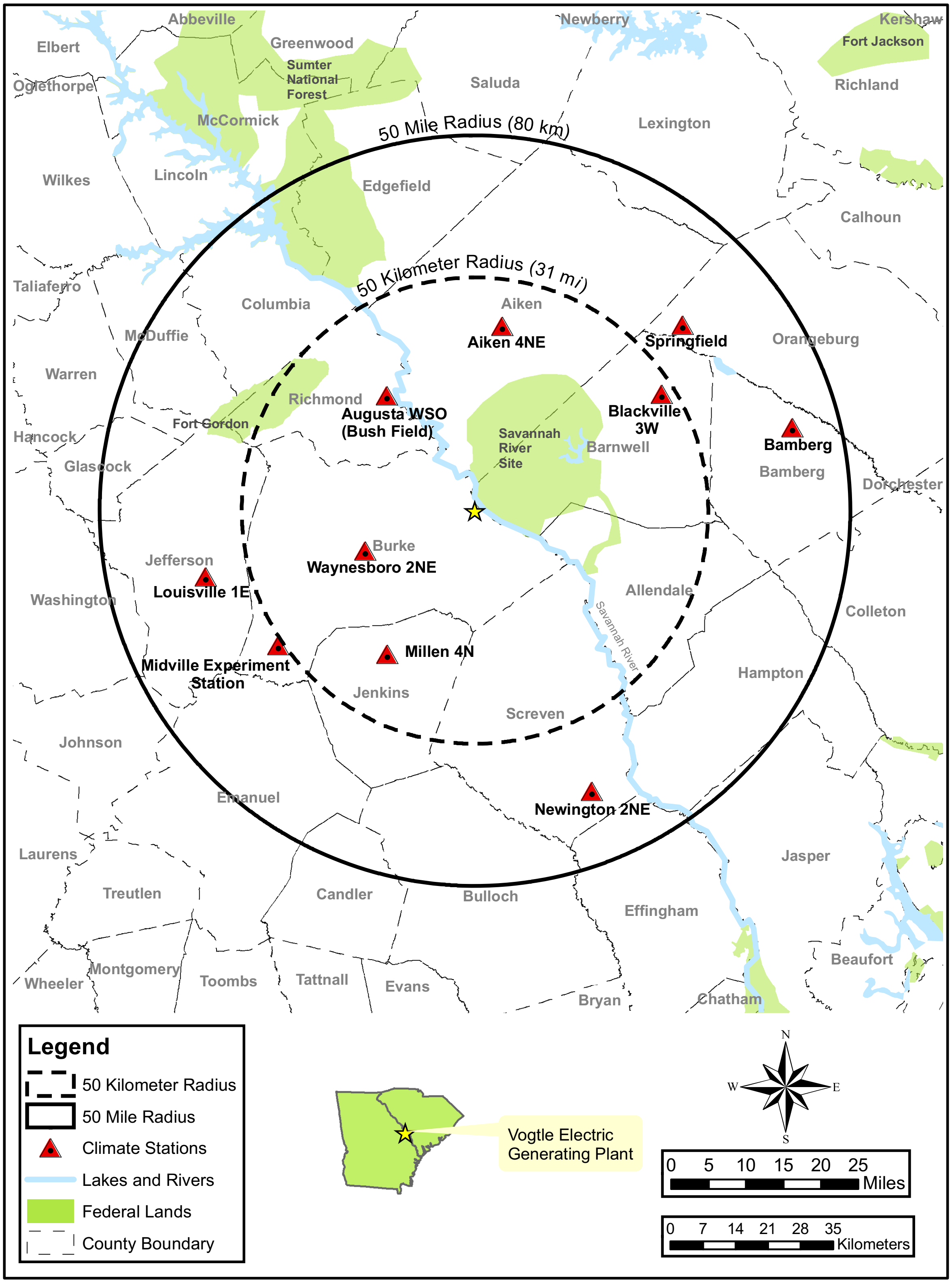

Figure 2.3-201
Climatological Observing Stations Near the VEGP Site

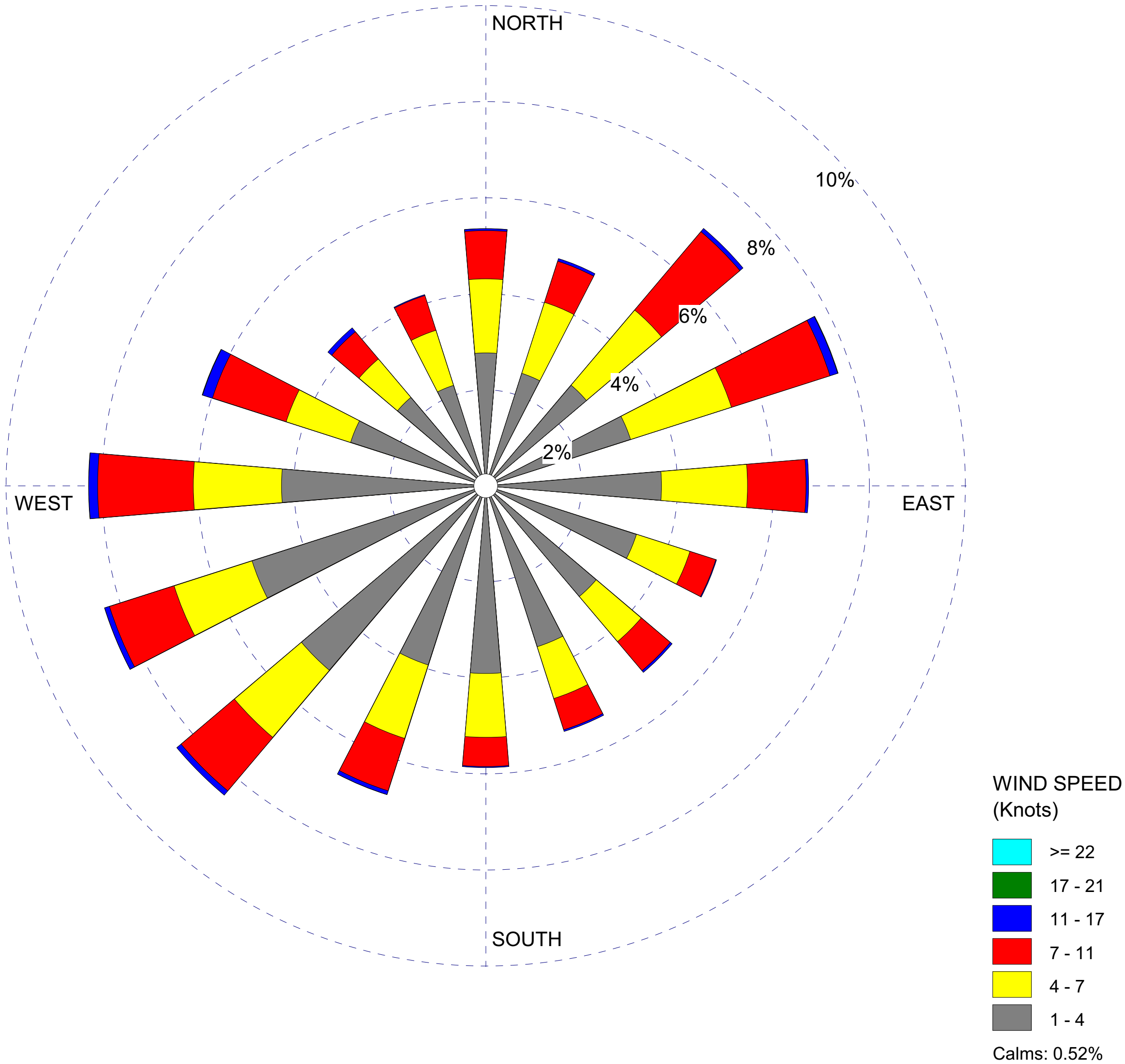

Figure 2.3-202
VEGP 10-m Level Annual Wind Rose (1998-2002)

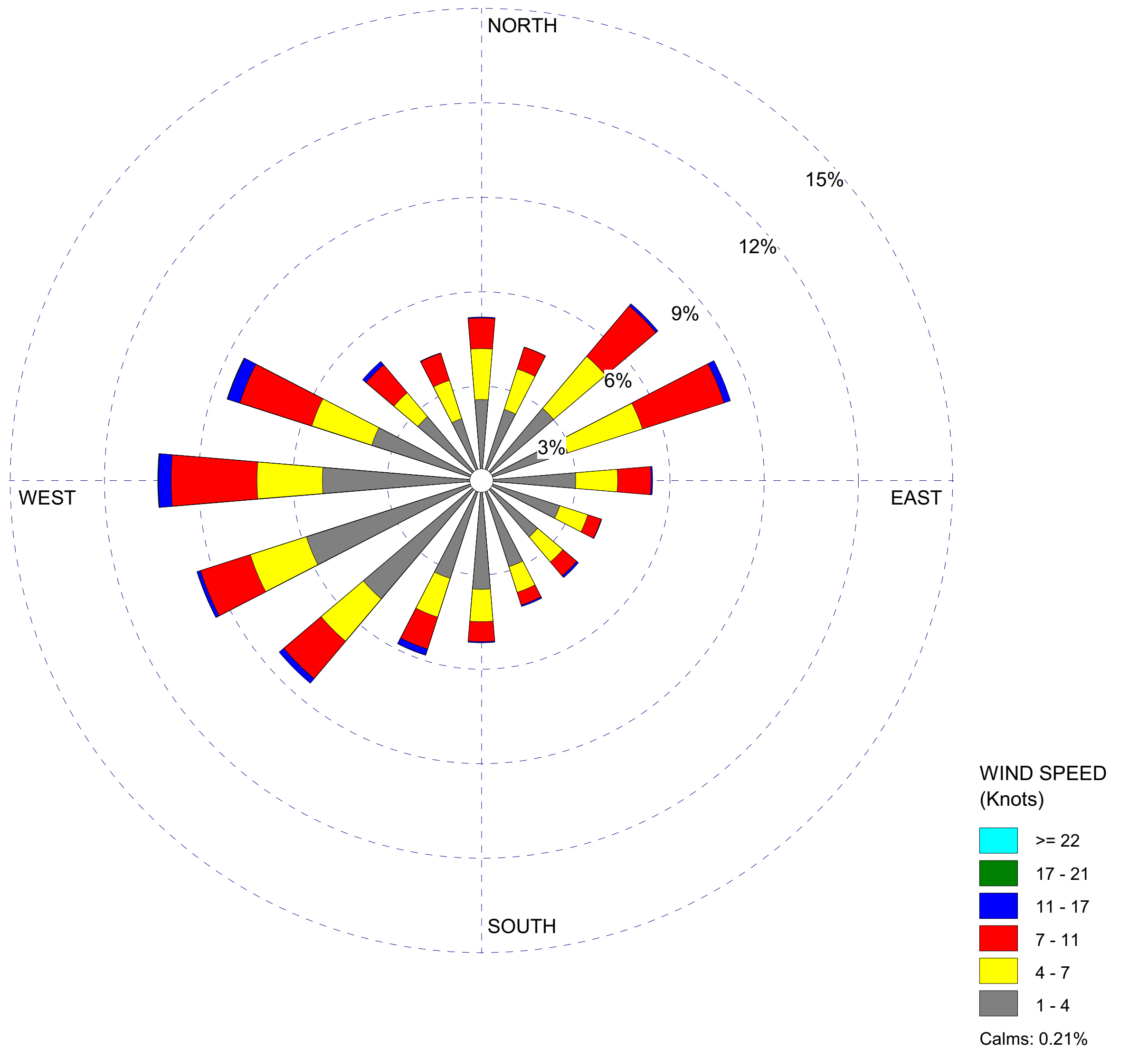

Figure 2.3-203
VEGP 10-m Level Winter Wind Rose (1998-2002)

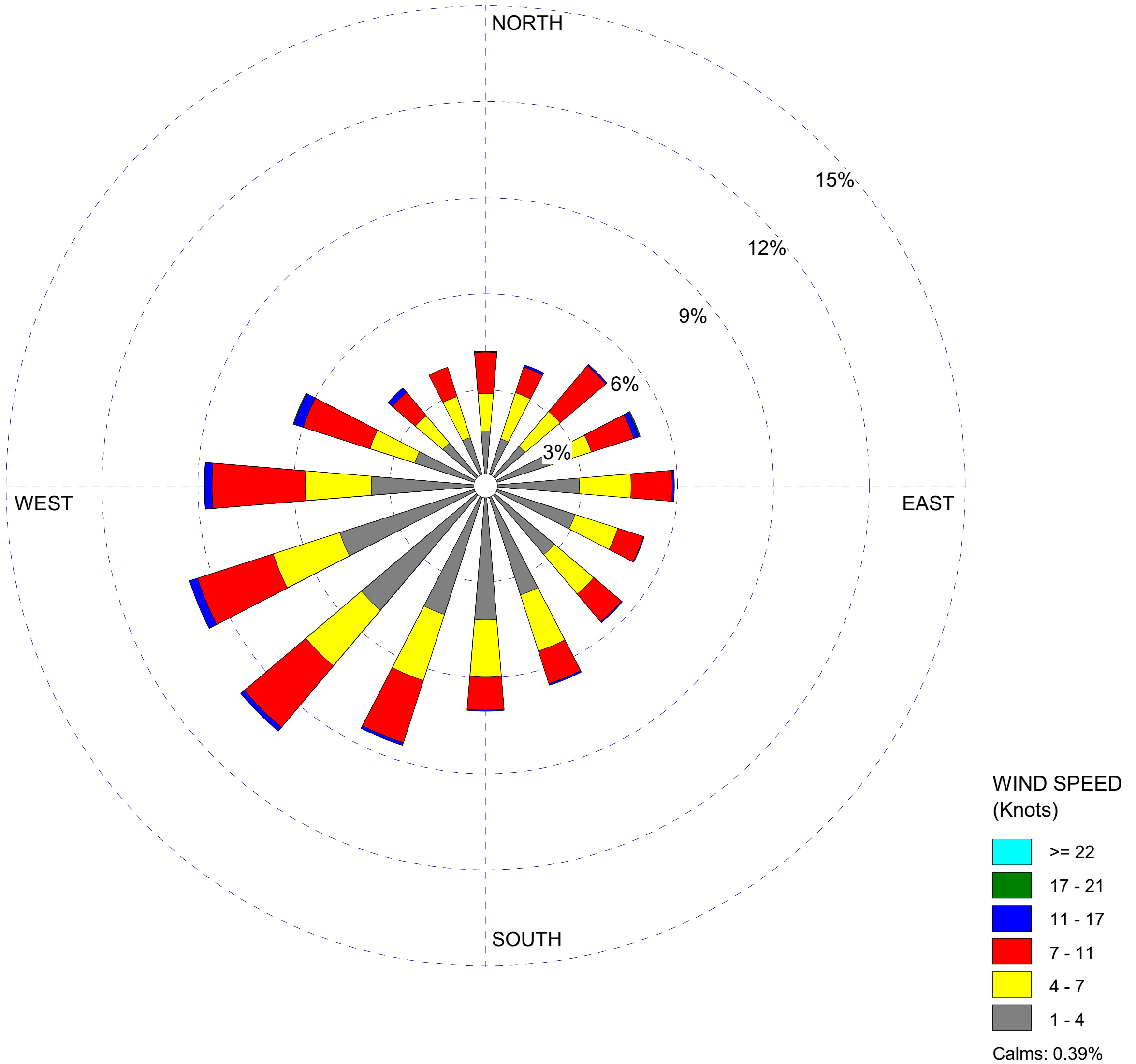

Figure 2.3-204
VEGP 10-m Level Spring Wind Rose (1998-2002)

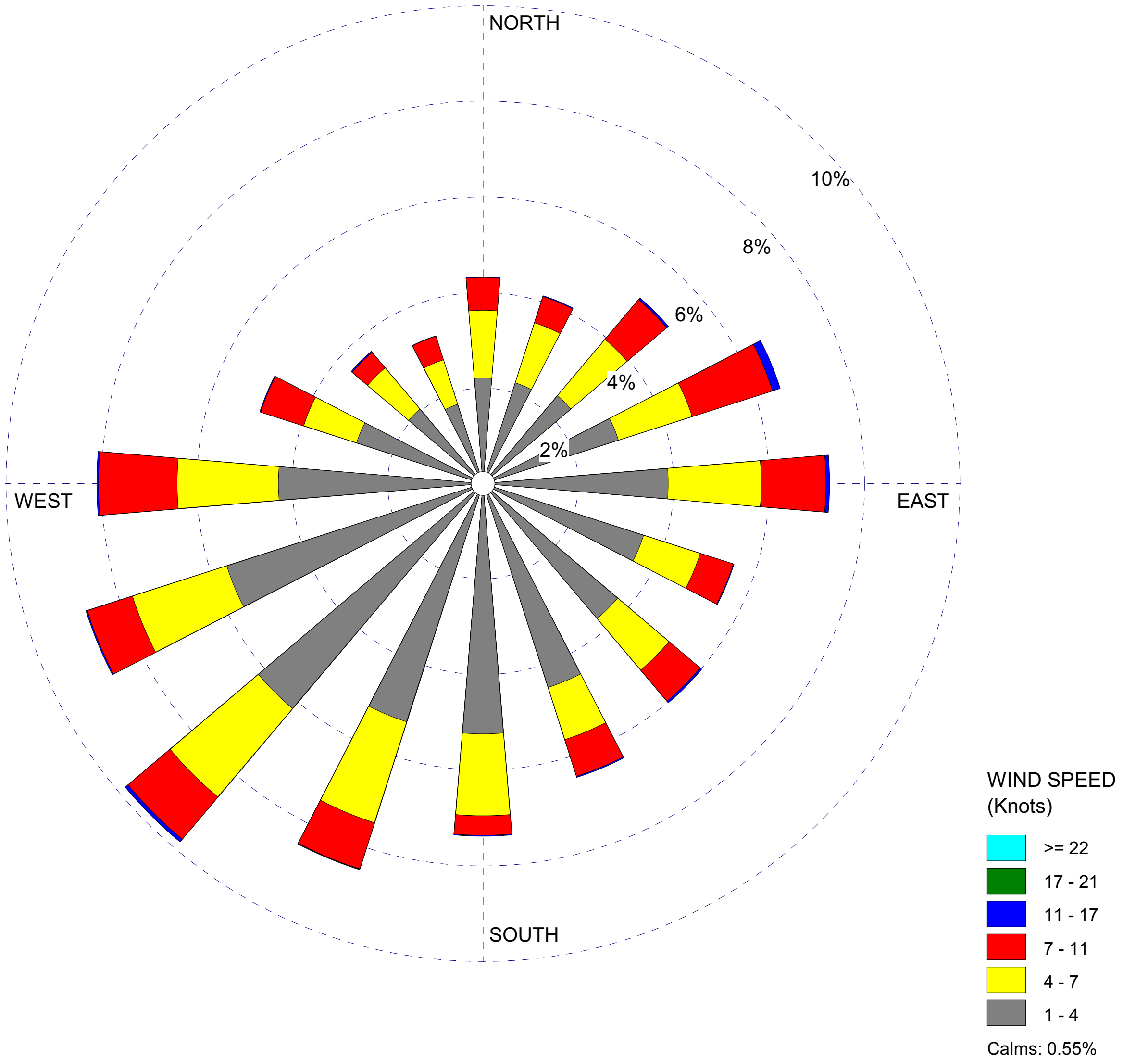

Figure 2.3-205
VEGP 10-m Level Summer Wind Rose (1998-2002)

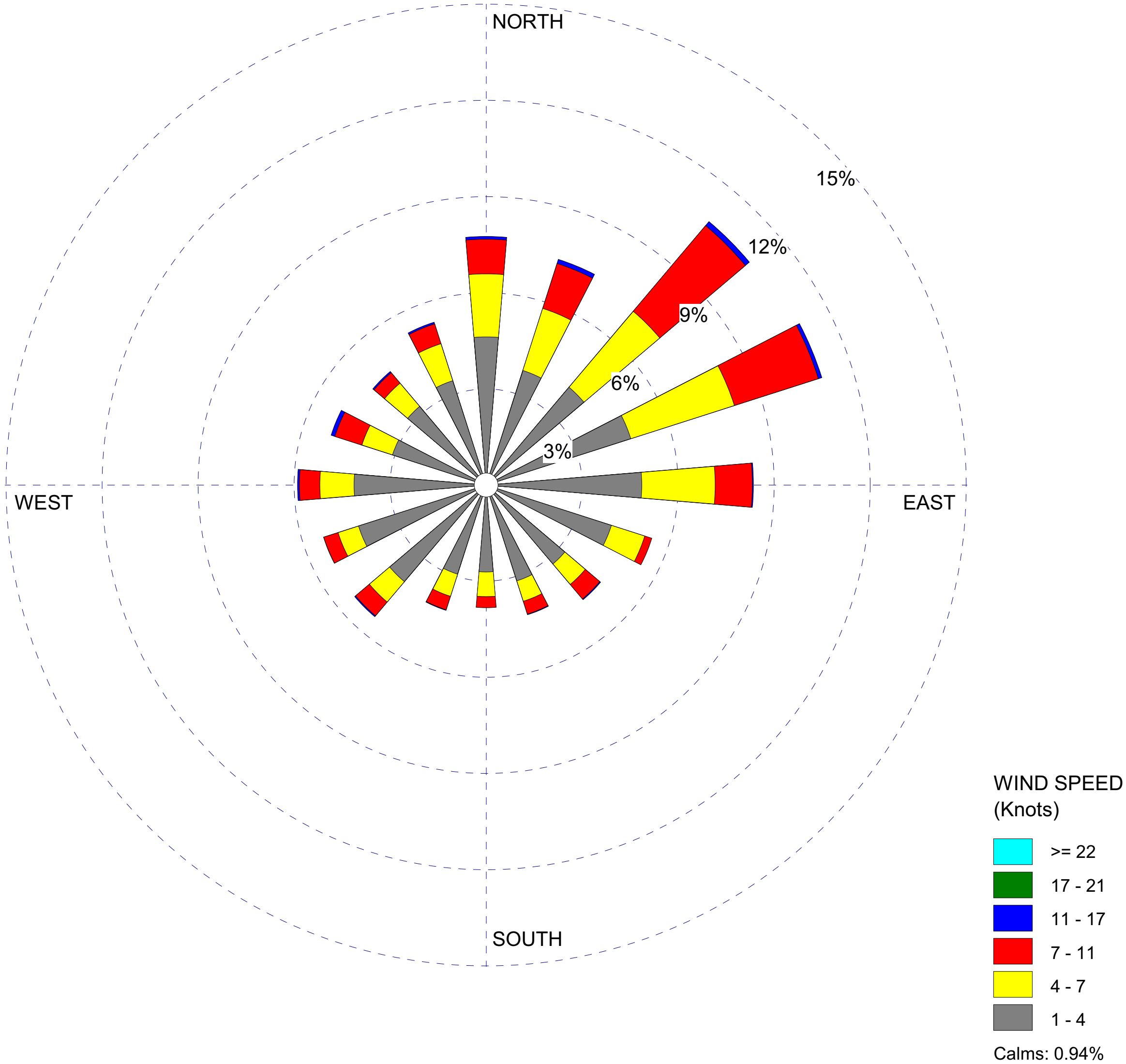

Figure 2.3-206
VEGP 10-m Level Autumn Wind Rose (1998-2002)

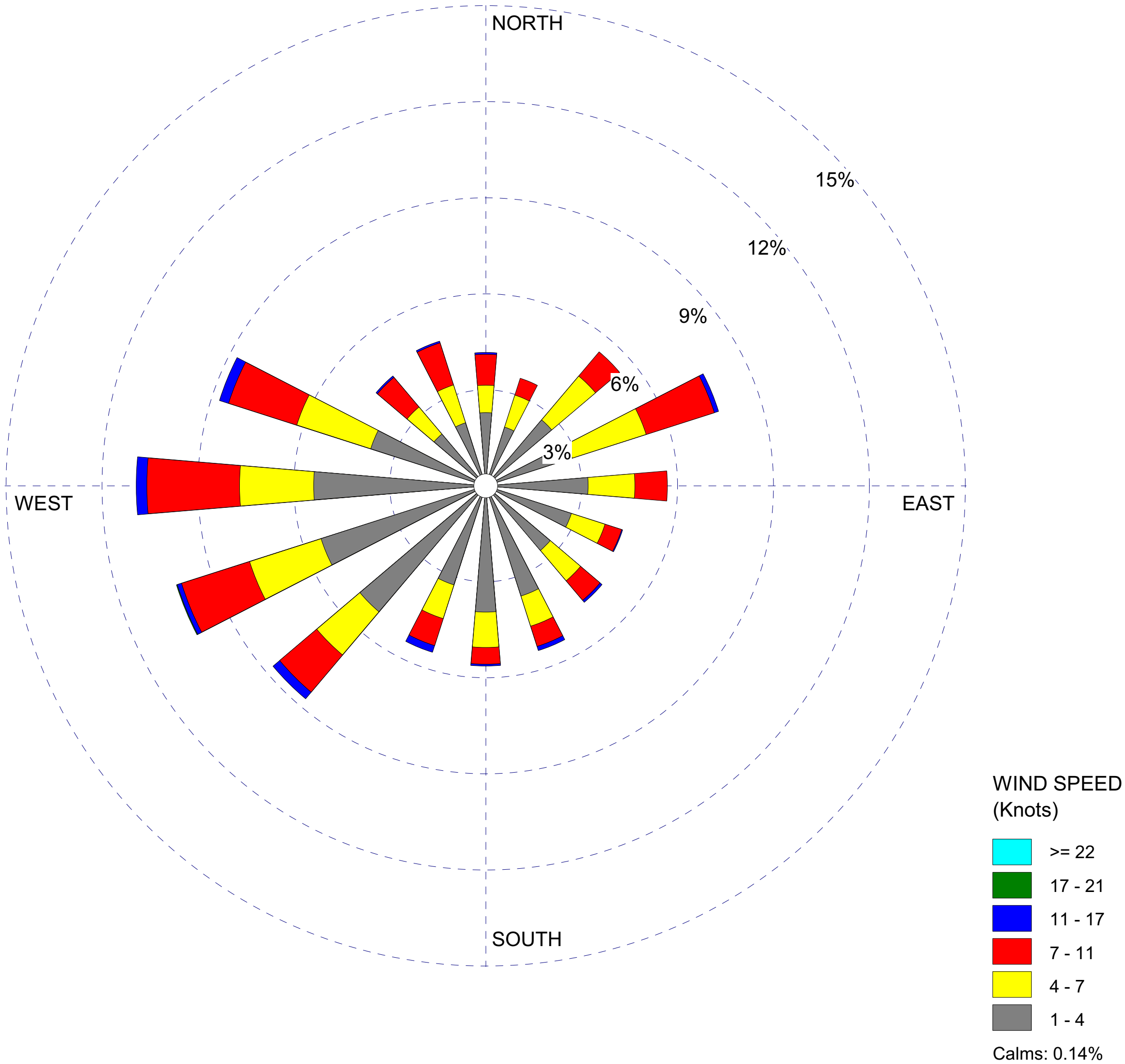

Figure 2.3-207 (Sheet 1 of 12)
VEGP 10-m Level January Wind Rose (1998–2002)

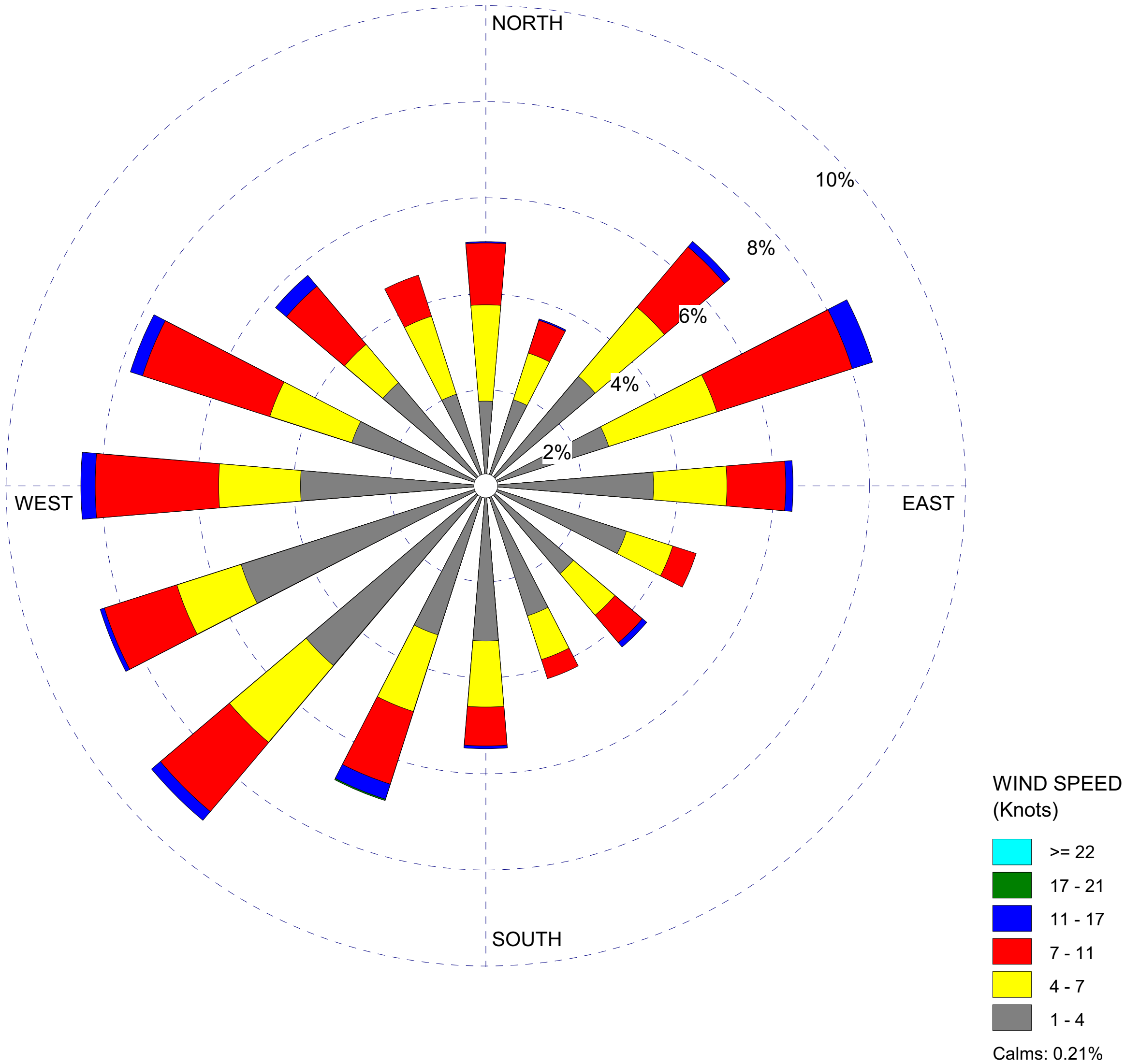

Figure 2.3-207 (Sheet 2 of 12)
VEGP 10-m Level February Wind Rose (1998–2002)

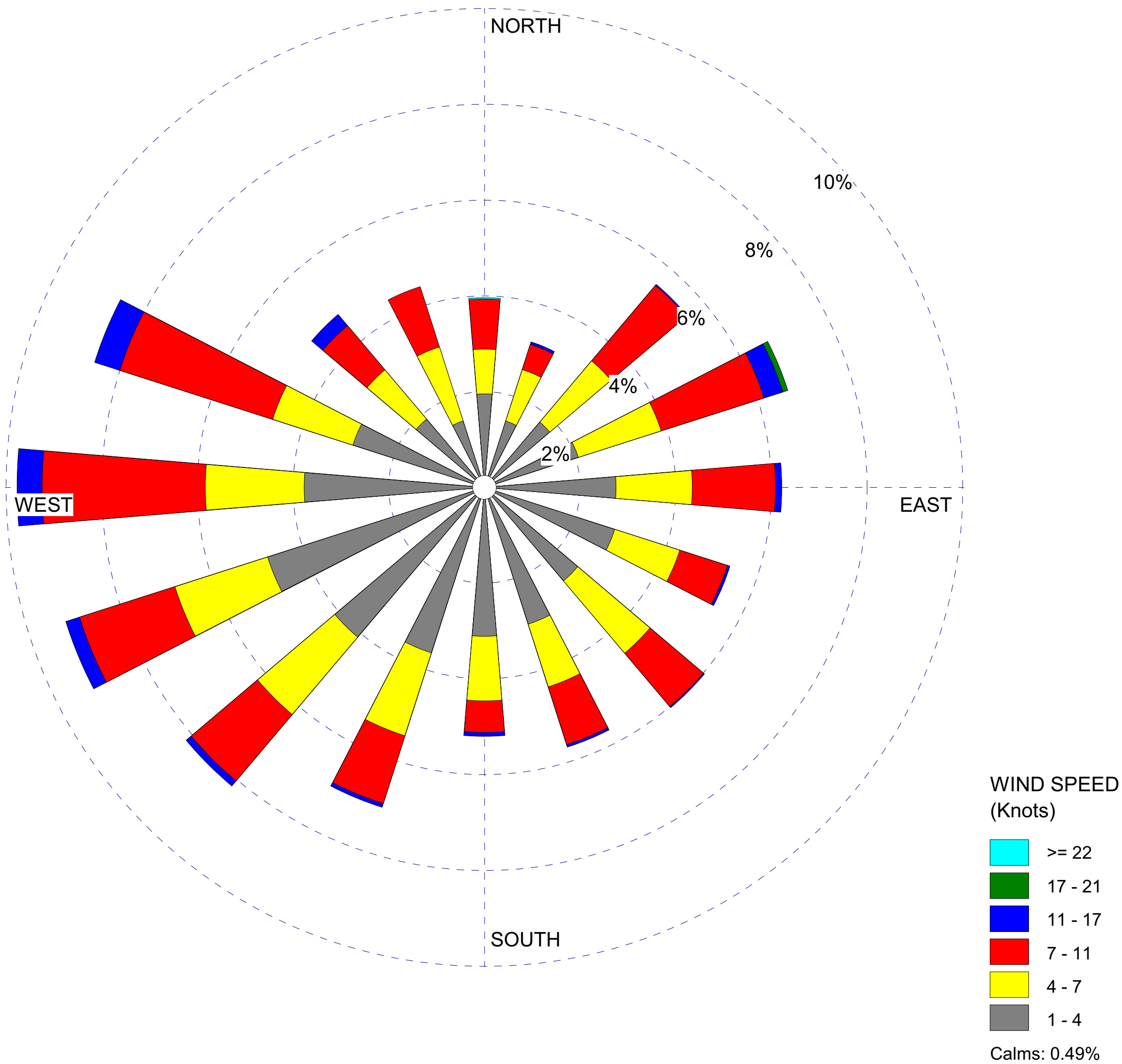

Figure 2.3-207 (Sheet 3 of 12)
VEGP 10-m Level March Wind Rose (1998–2002)

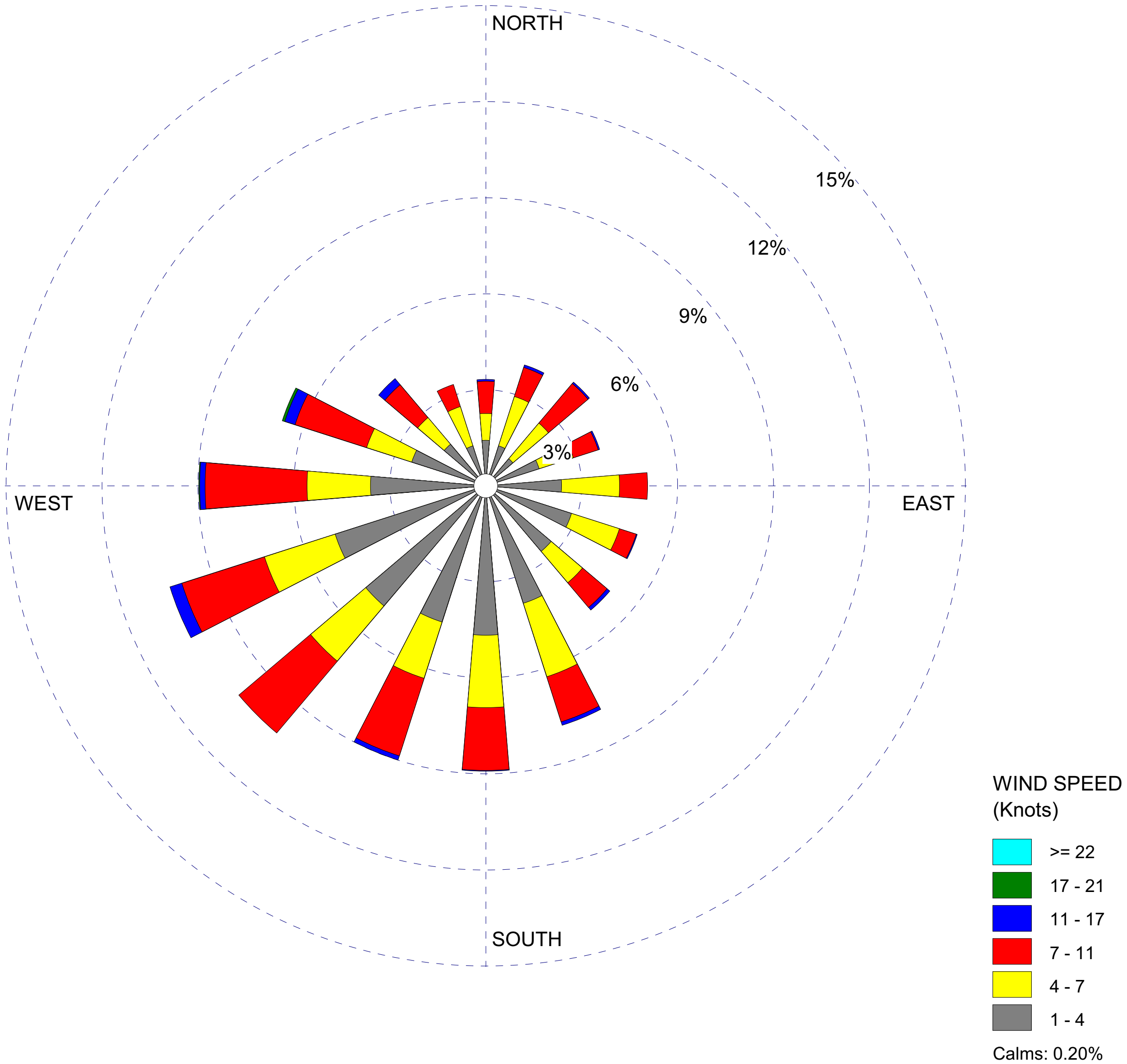

Figure 2.3-207 (Sheet 4 of 12)
VEGP 10-m Level April Wind Rose (1998–2002)

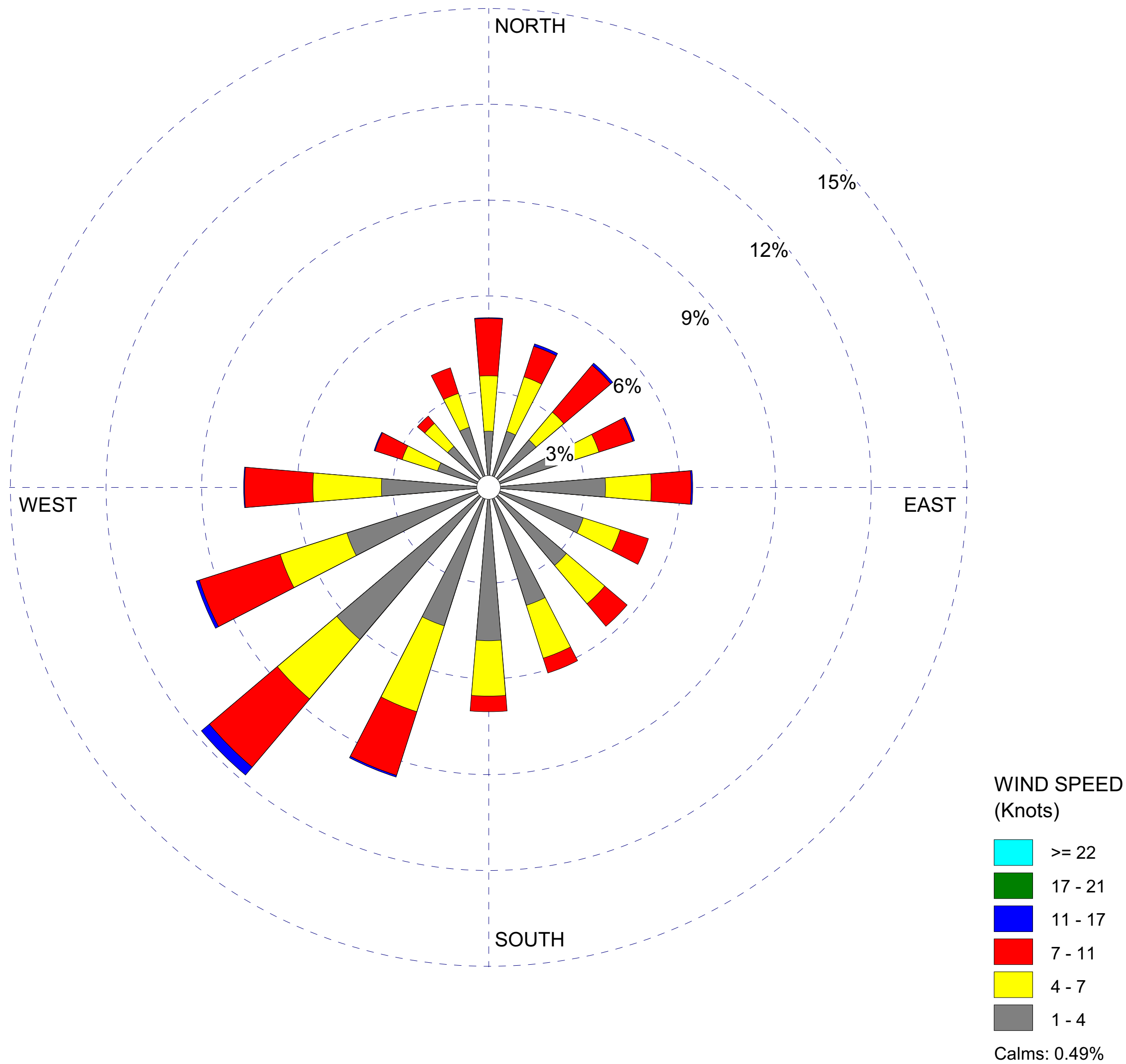

Figure 2.3-207 (Sheet 5 of 12)
VEGP 10-m Level May Wind Rose (1998–2002)

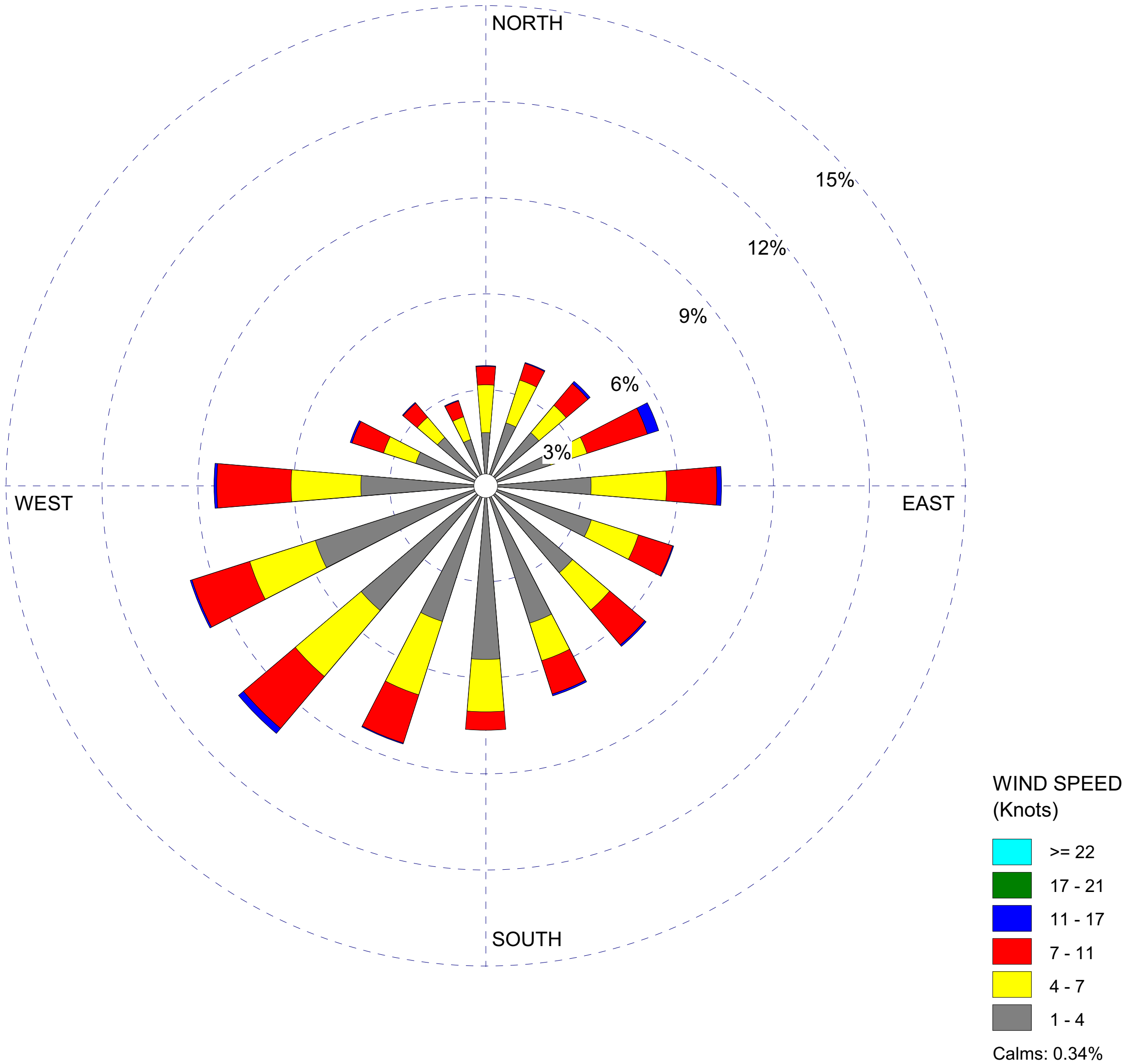

Figure 2.3-207 (Sheet 6 of 12)
VEGP 10-m Level June Wind Rose (1998–2002)

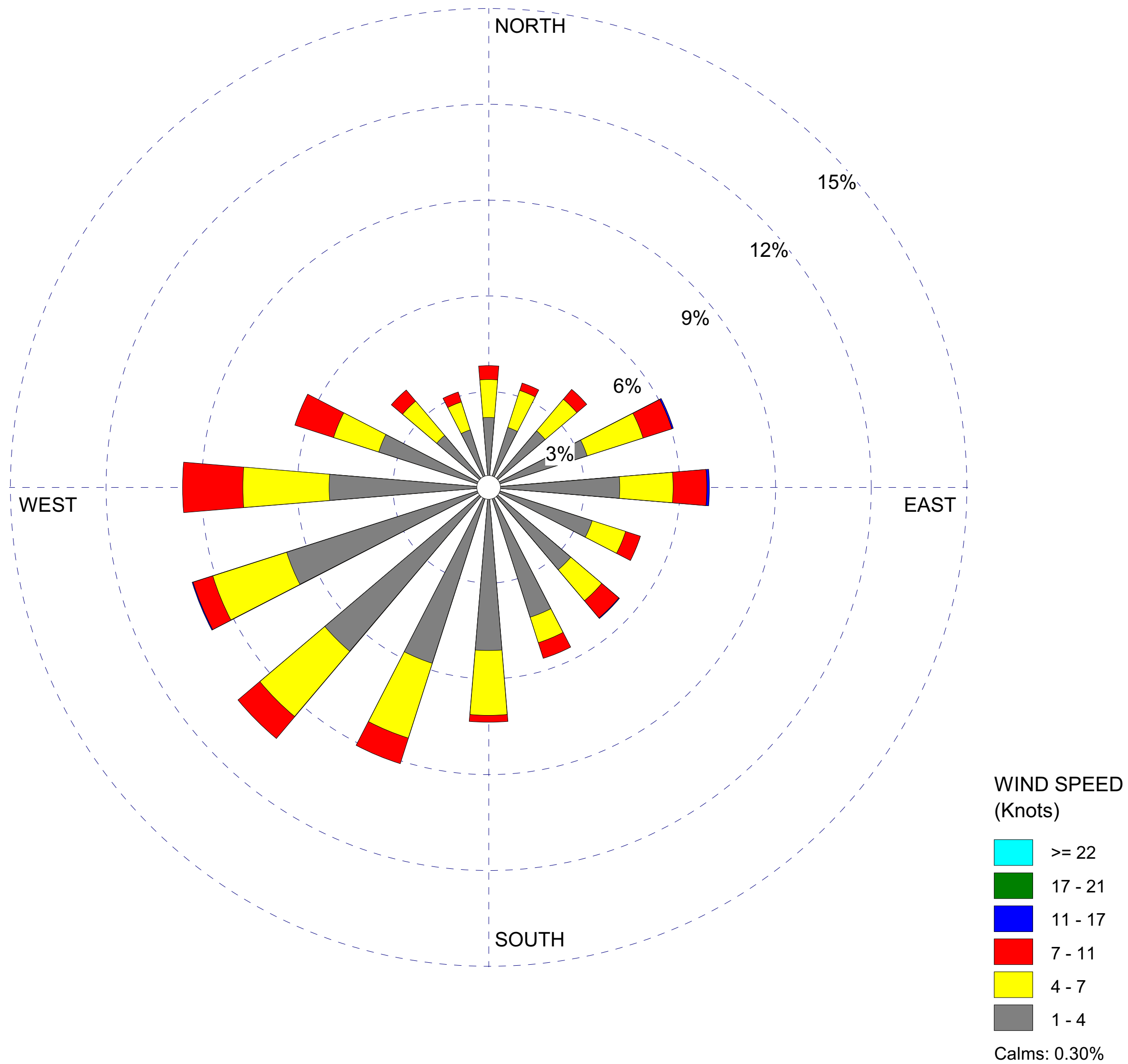

Figure 2.3-207 (Sheet 7 of 12)
VEGP 10-m Level July Wind Rose (1998–2002)

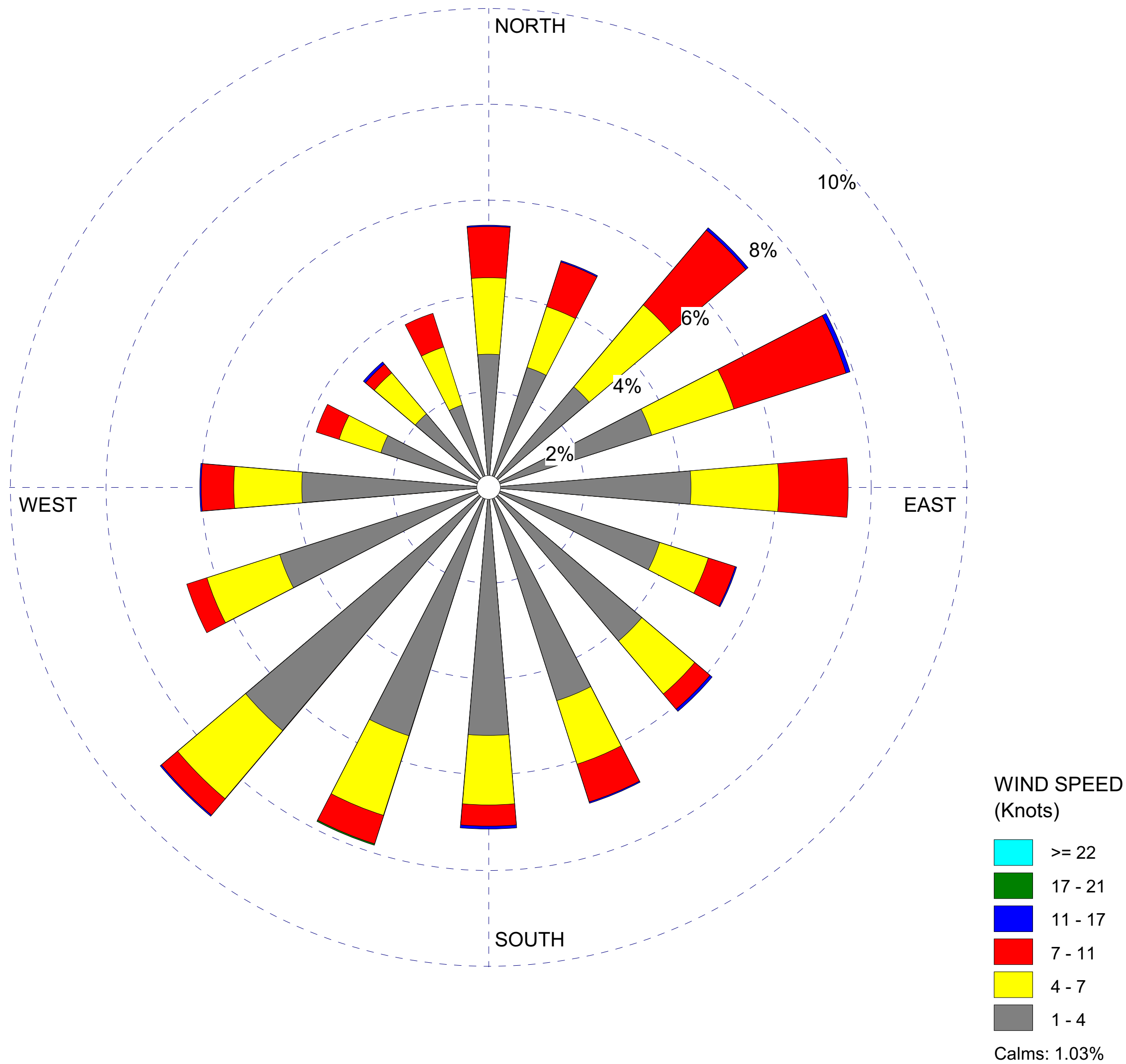

Figure 2.3-207 (Sheet 8 of 12)
VEGP 10-m Level August Wind Rose (1998–2002)

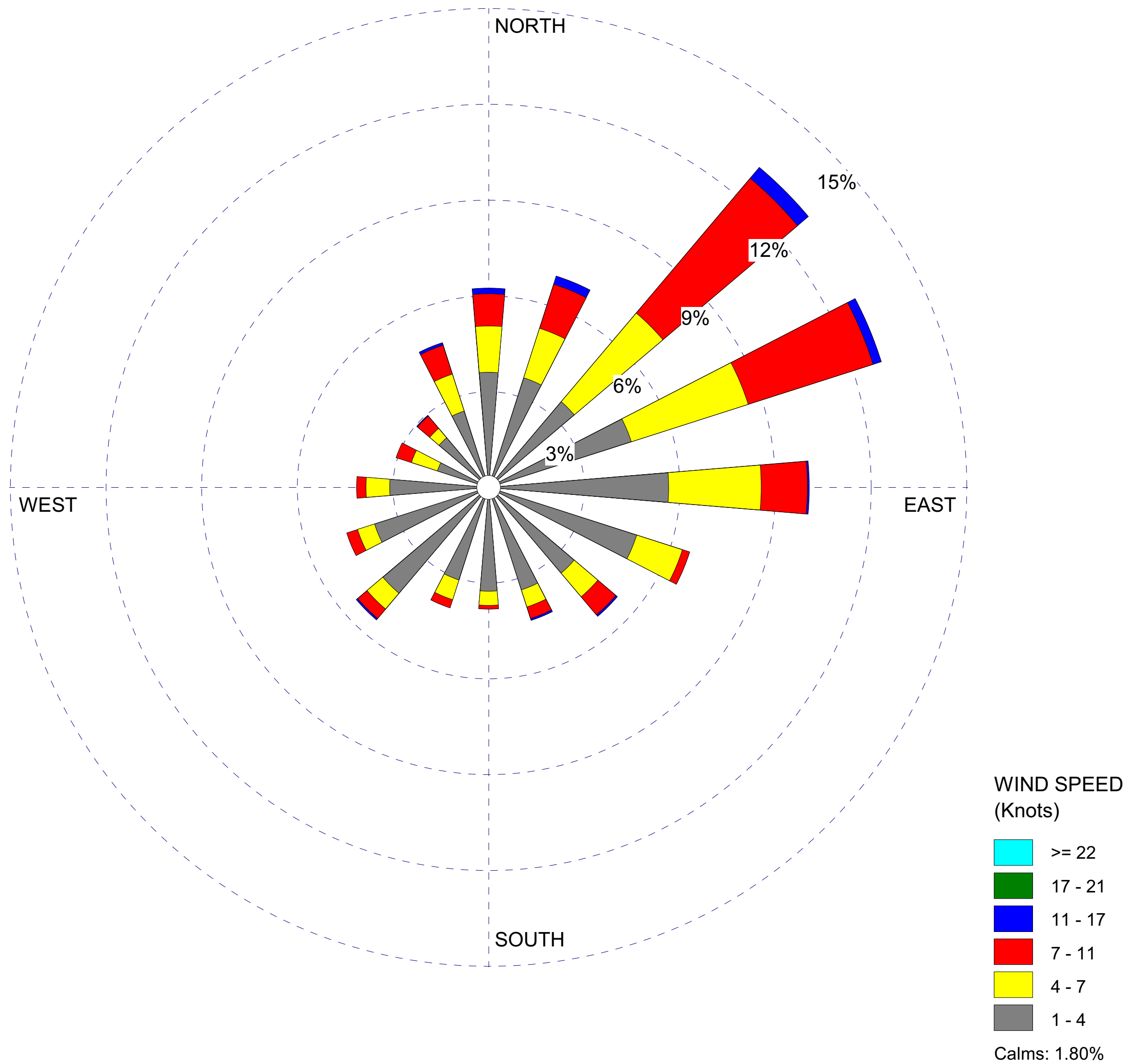

Figure 2.3-207 (Sheet 9 of 12)
VEGP 10-m Level September Wind Rose (1998–2002)

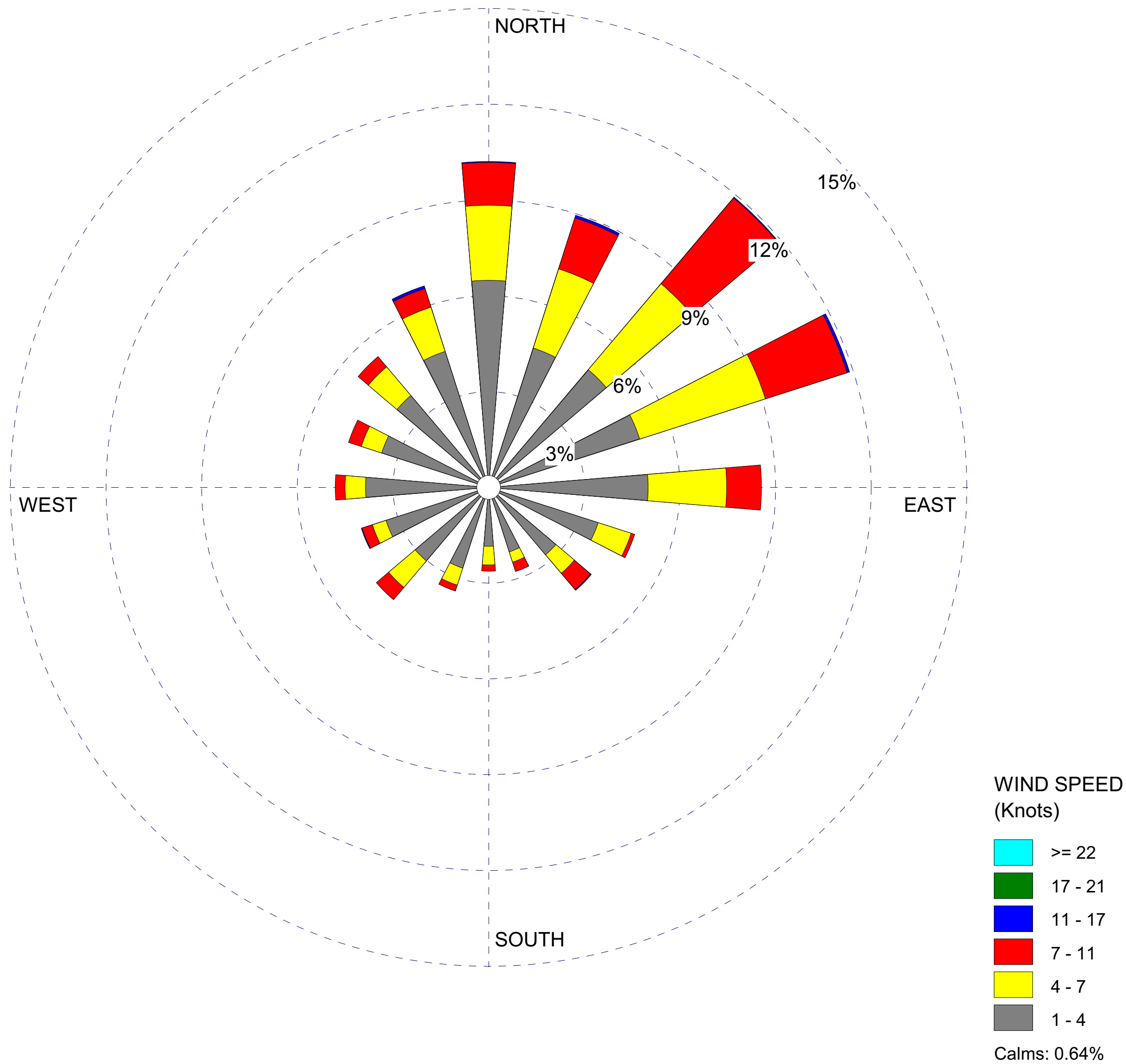

Figure 2.3-207 (Sheet 10 of 12)
VEGP 10-m Level October Wind Rose (1998–2002)

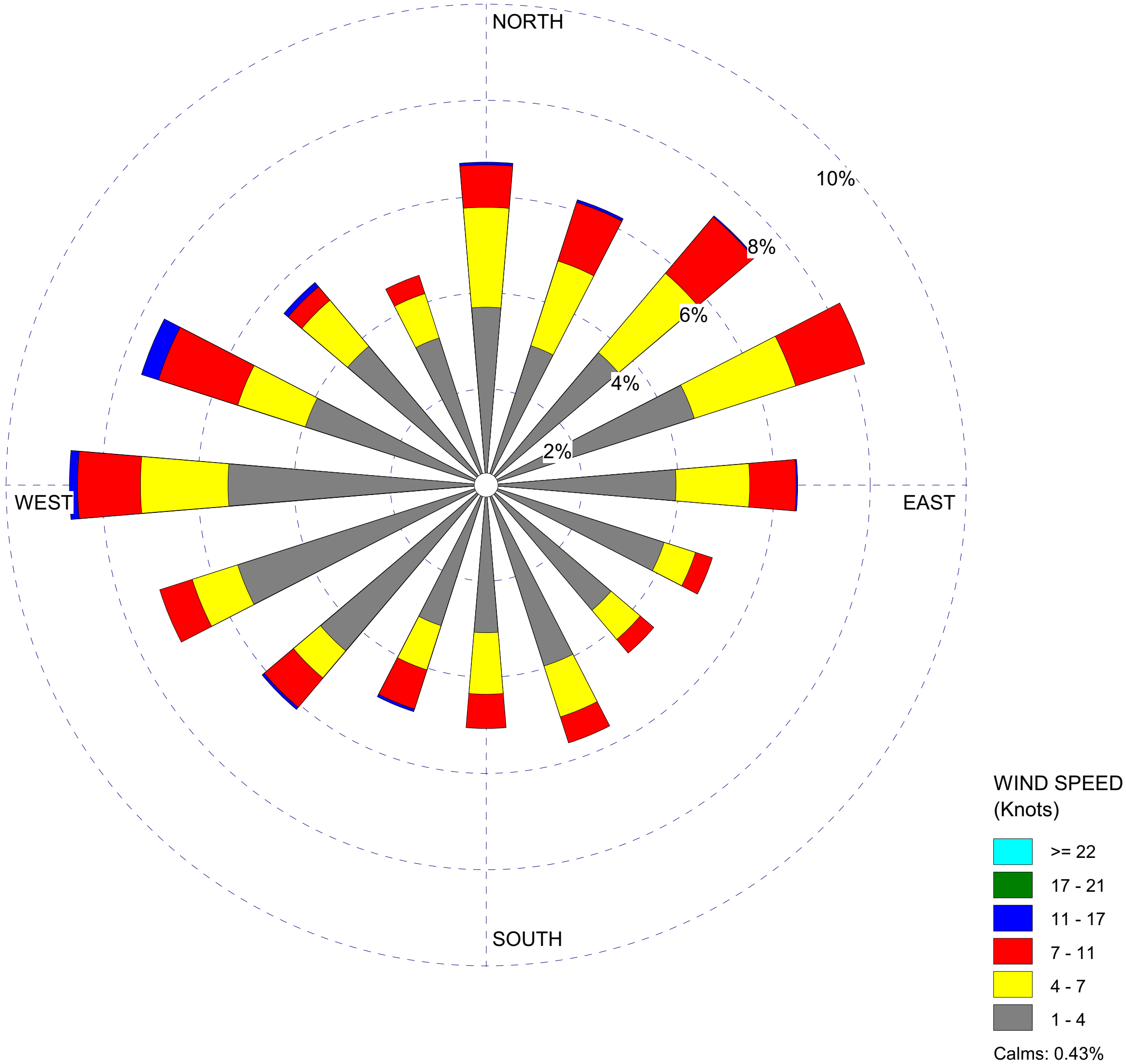

Figure 2.3-207 (Sheet 11 of 12)
VEGP 10-m Level November Wind Rose (1998–2002)

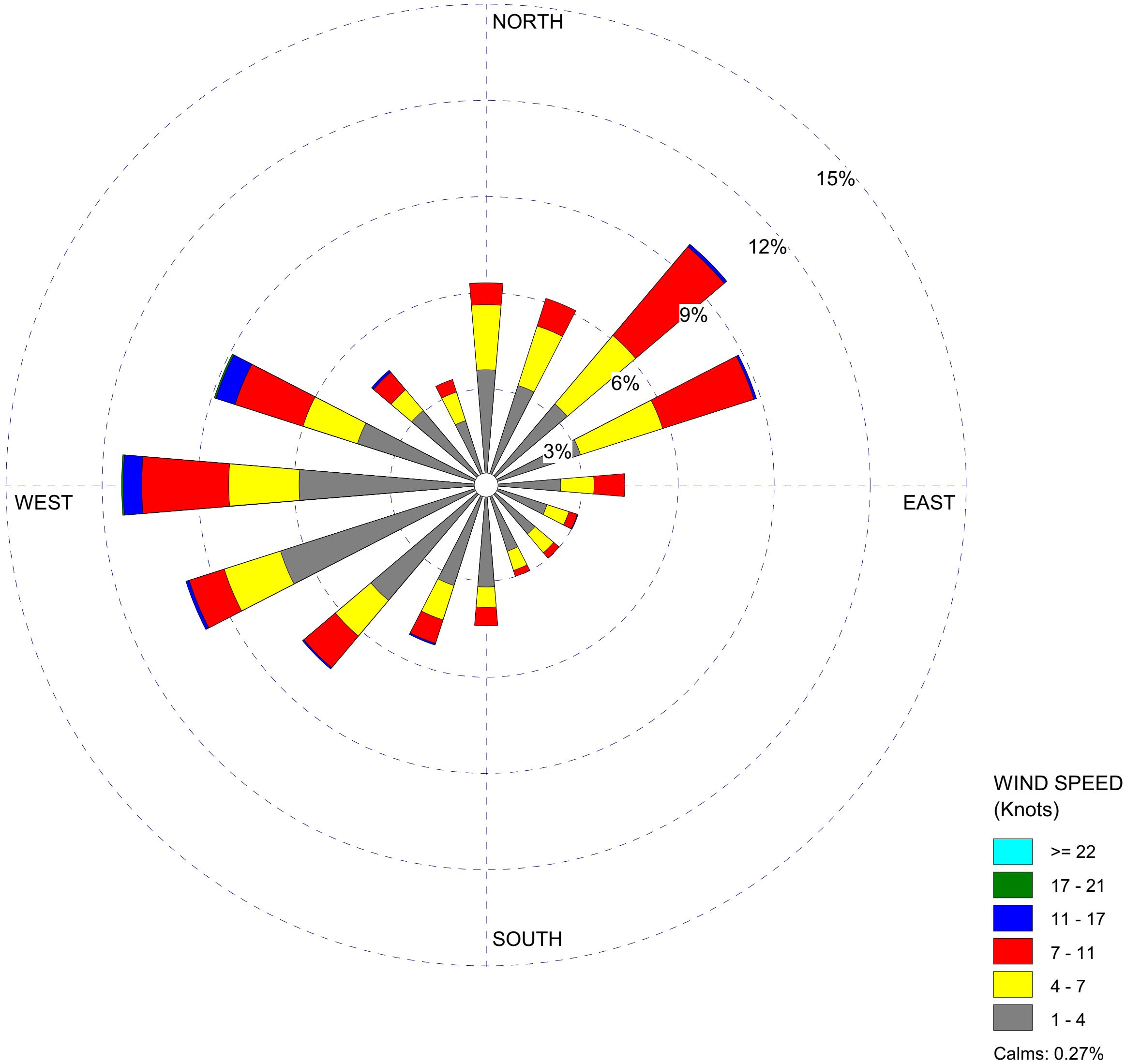

Figure 2.3-207 (Sheet 12 of 12)
VEGP 10-m Level December Wind Rose (1998–2002)

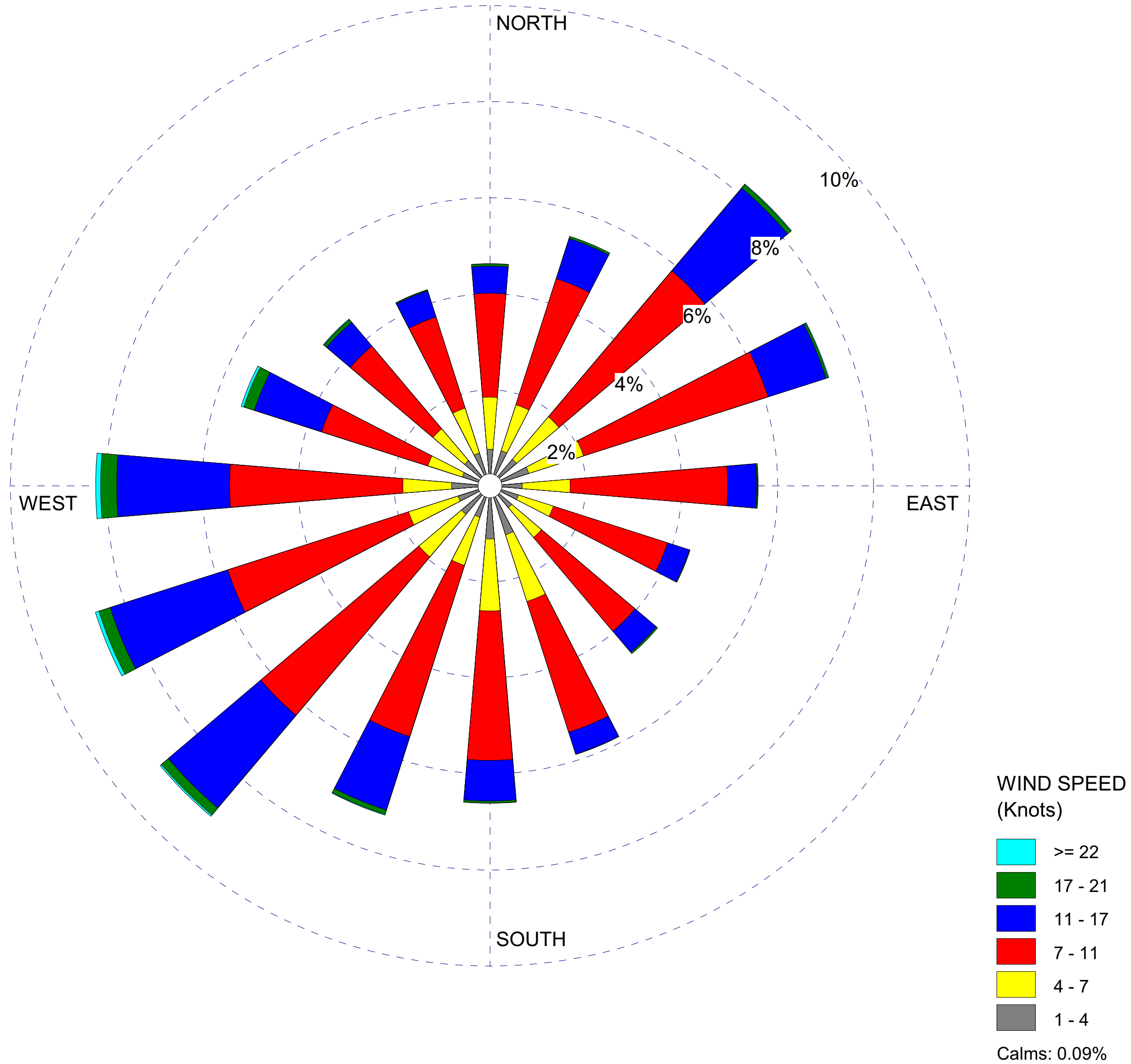

Figure 2.3-208
VEGP 60-m Level Annual Wind Rose (1998-2002)

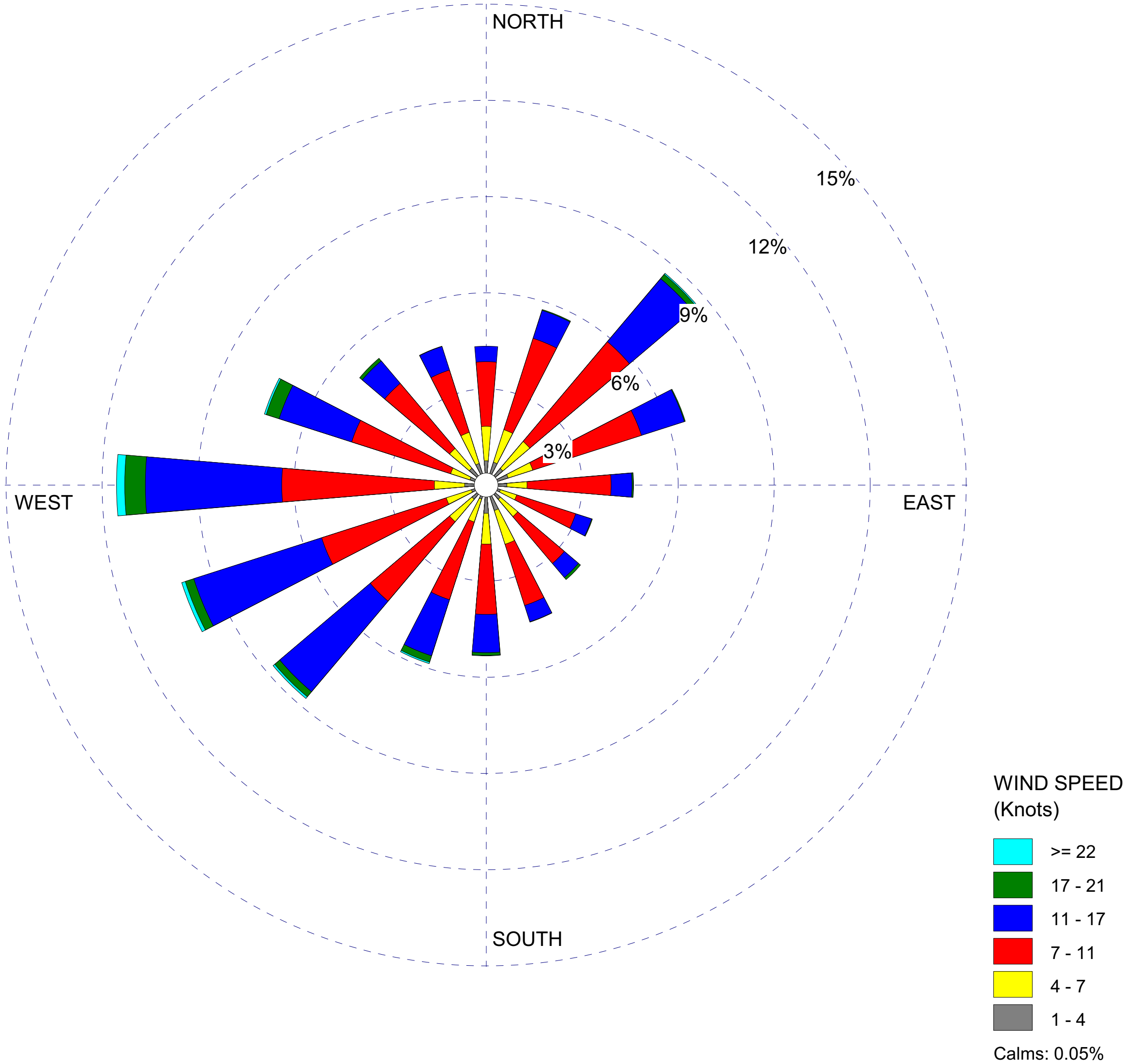

Figure 2.3-209
VEGP 60-m Level Winter Wind Rose (1998-2002)

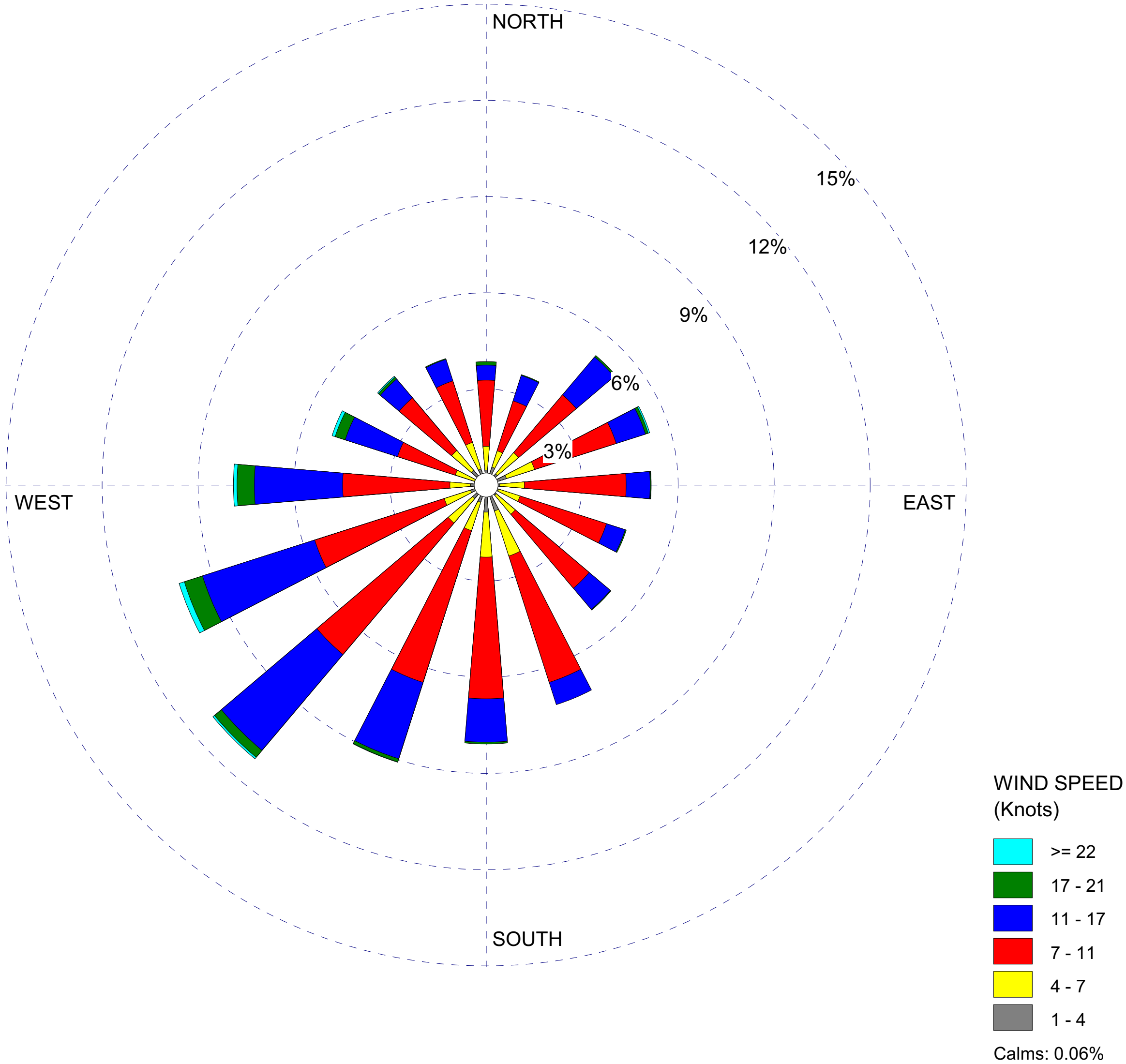

Figure 2.3-210
VEGP 60-m Level Spring Wind Rose (1998-2002)

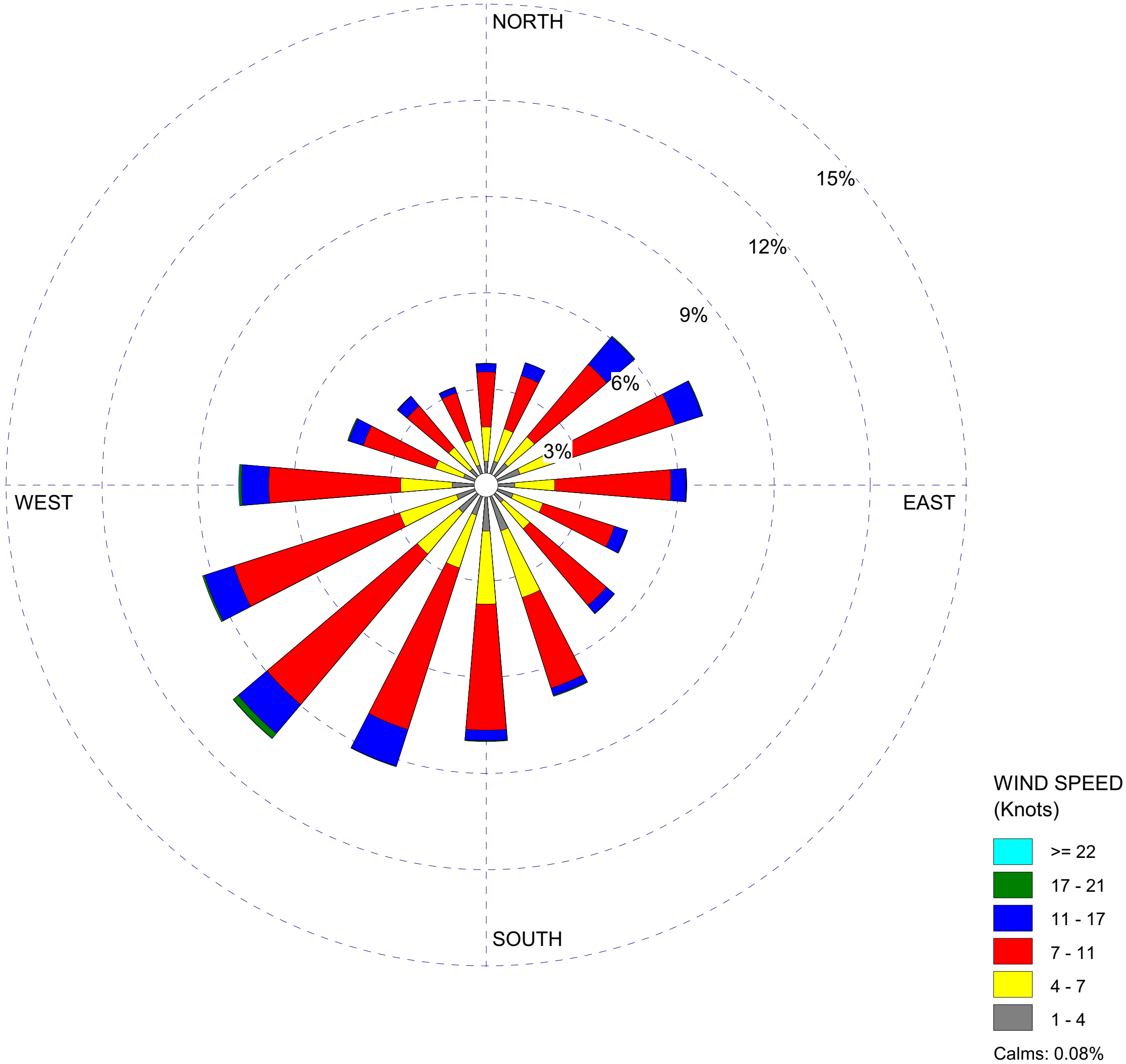

Figure 2.3-211
VEGP 60-m Level Summer Wind Rose (1998-2002)

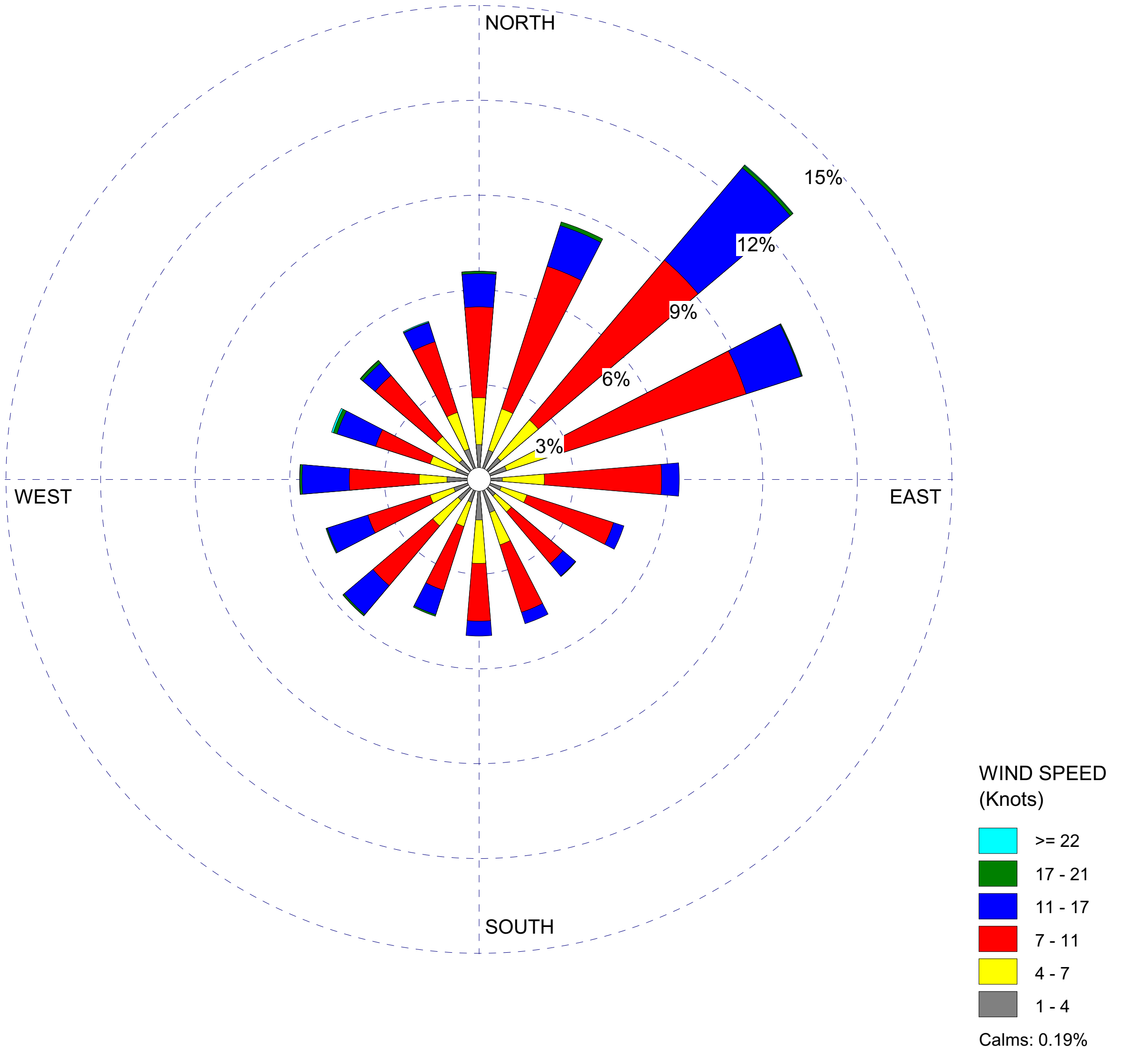

Figure 2.3-212
VEGP 60-m Level Autumn Wind Rose (1998-2002)

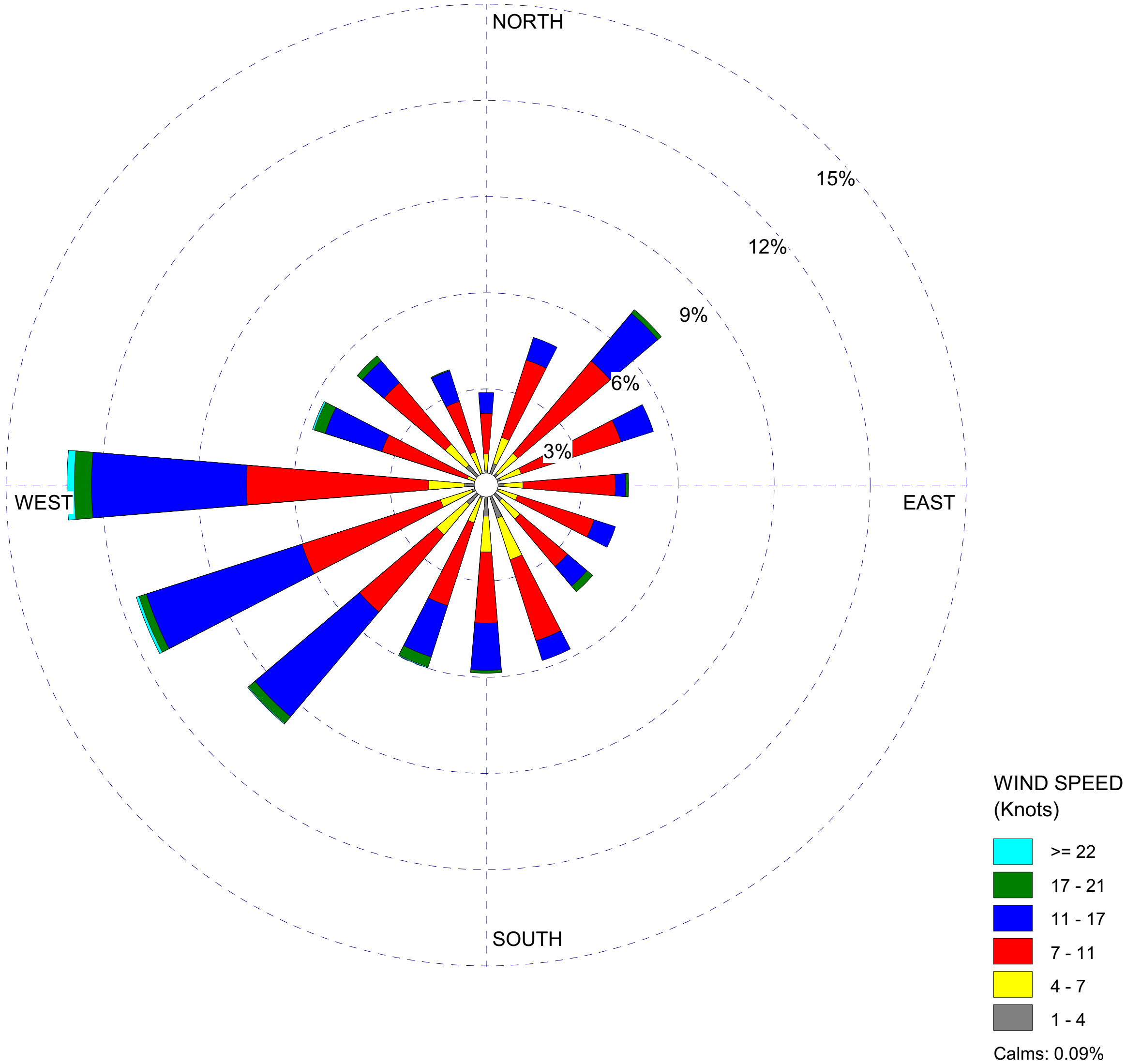

Figure 2.3-213 (Sheet 1 of 12)
VEGP 60-m Level January Wind Rose (1998–2002)

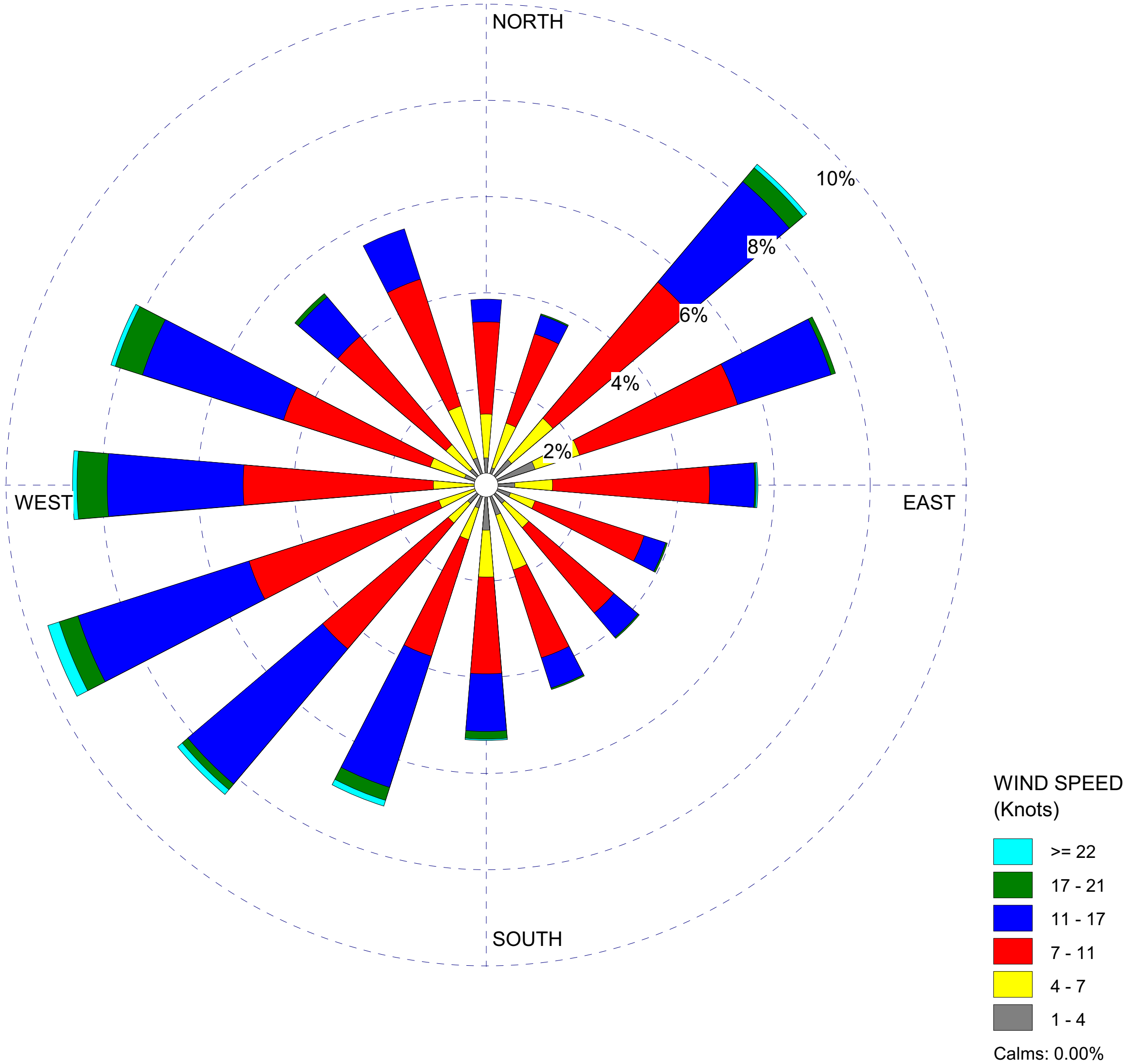

Figure 2.3-213 (Sheet 2 of 12)
VEGP 60-m Level February Wind Rose (1998–2002)

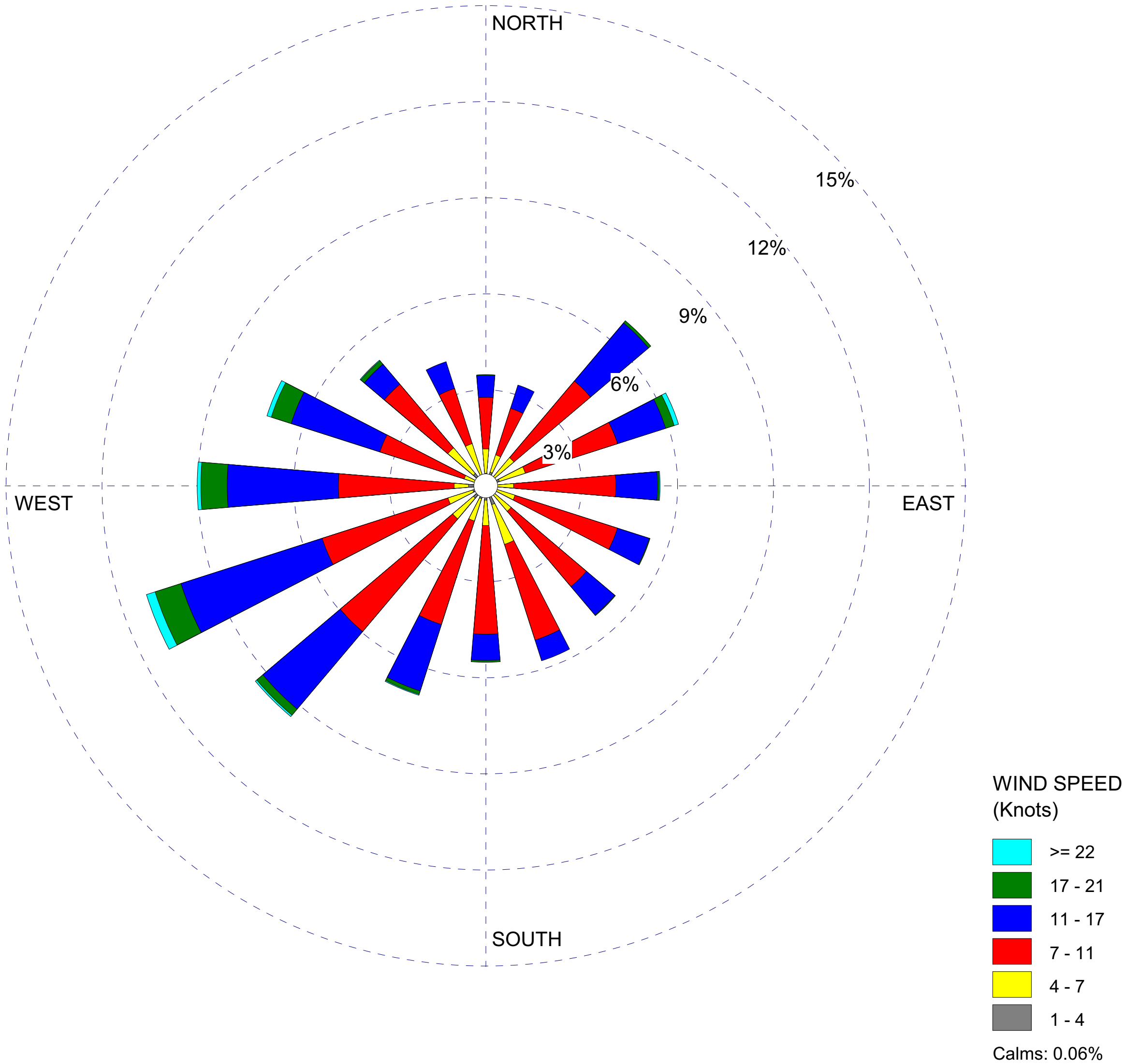

Figure 2.3-213 (Sheet 3 of 12)
VEGP 60-m Level March Wind Rose (1998–2002)

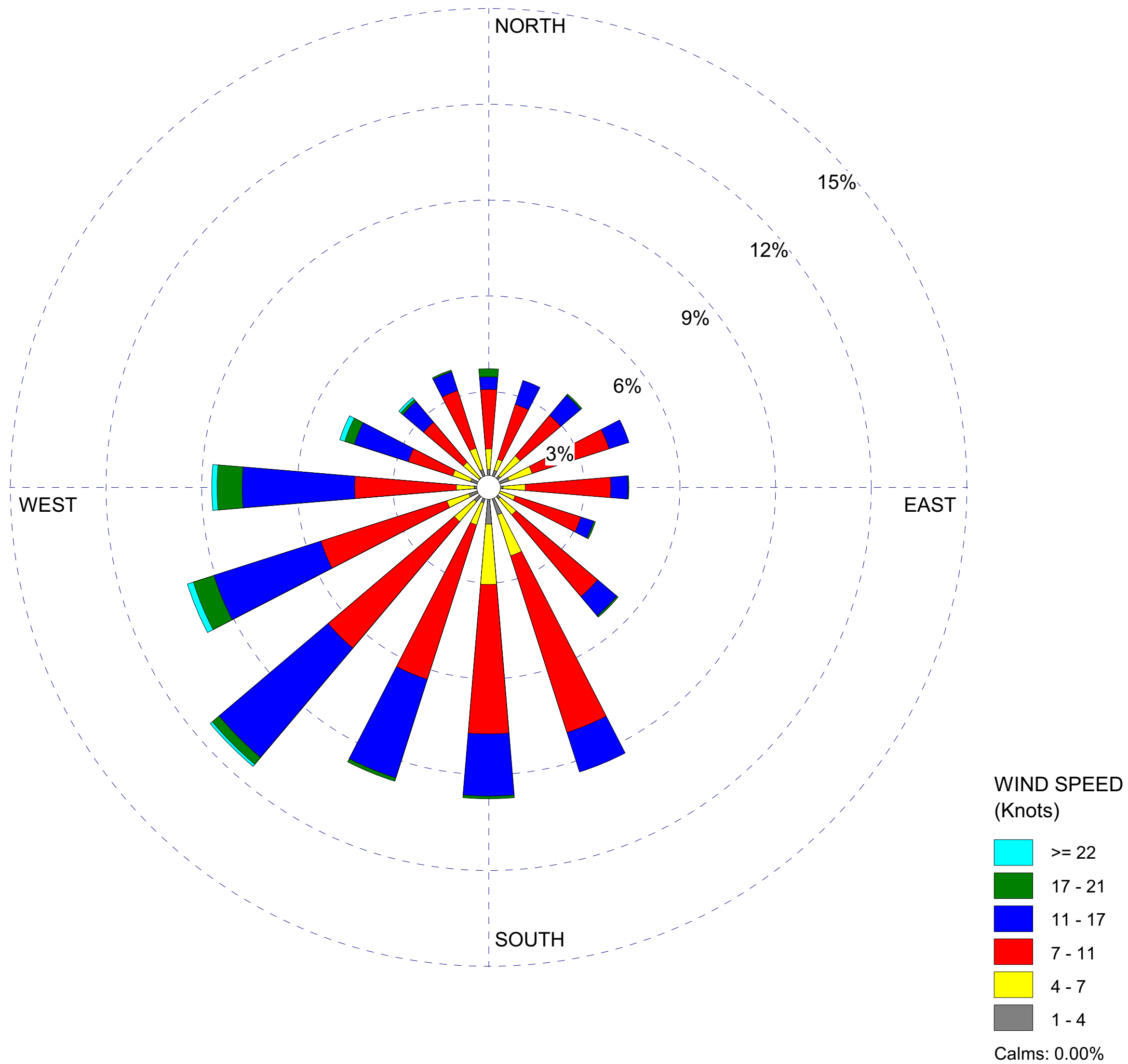

Figure 2.3-213 (Sheet 4 of 12)
VEGP 60-m Level April Wind Rose (1998–2002)

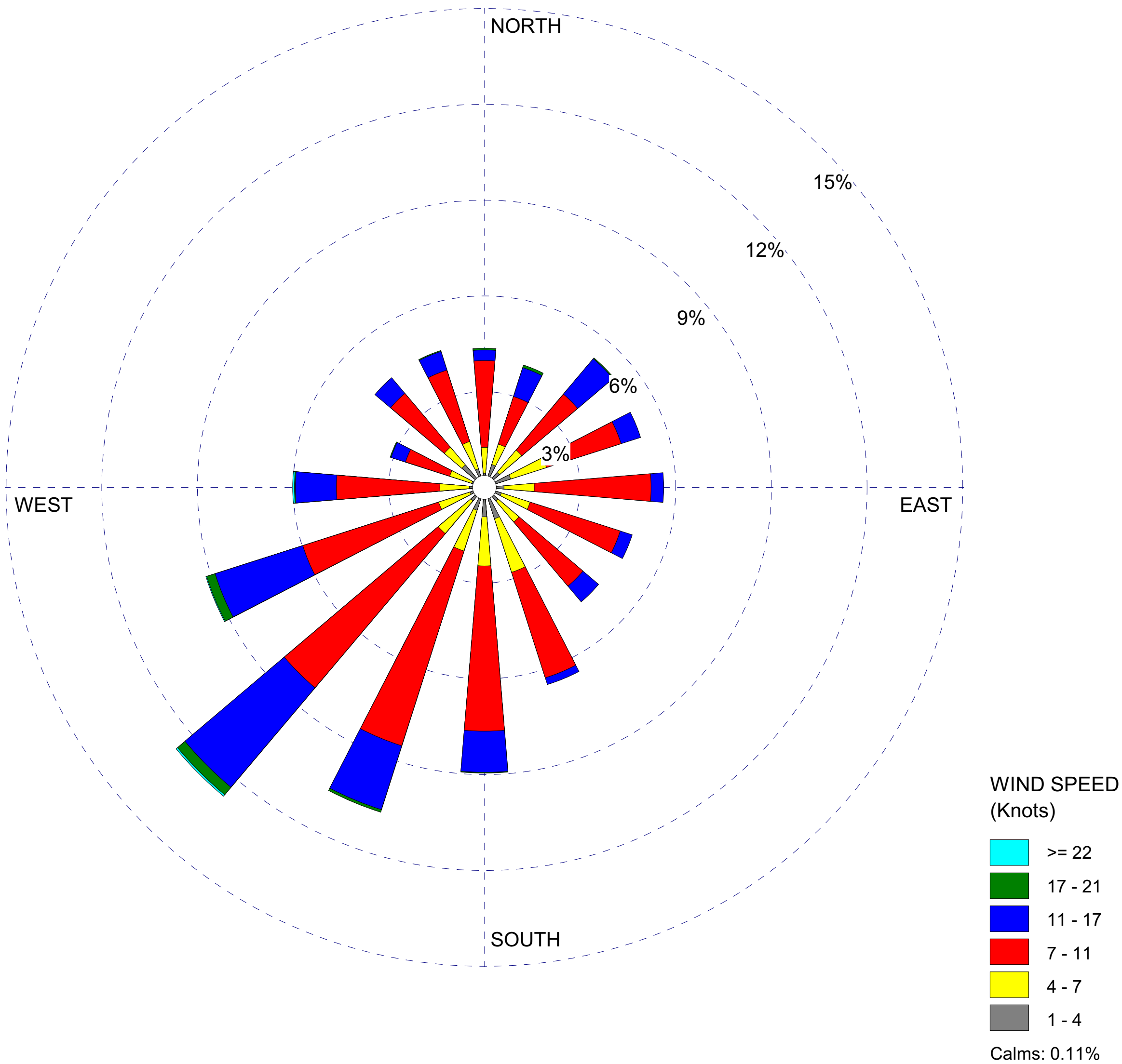

Figure 2.3-213 (Sheet 5 of 12)
VEGP 60-m Level May Wind Rose (1998–2002)

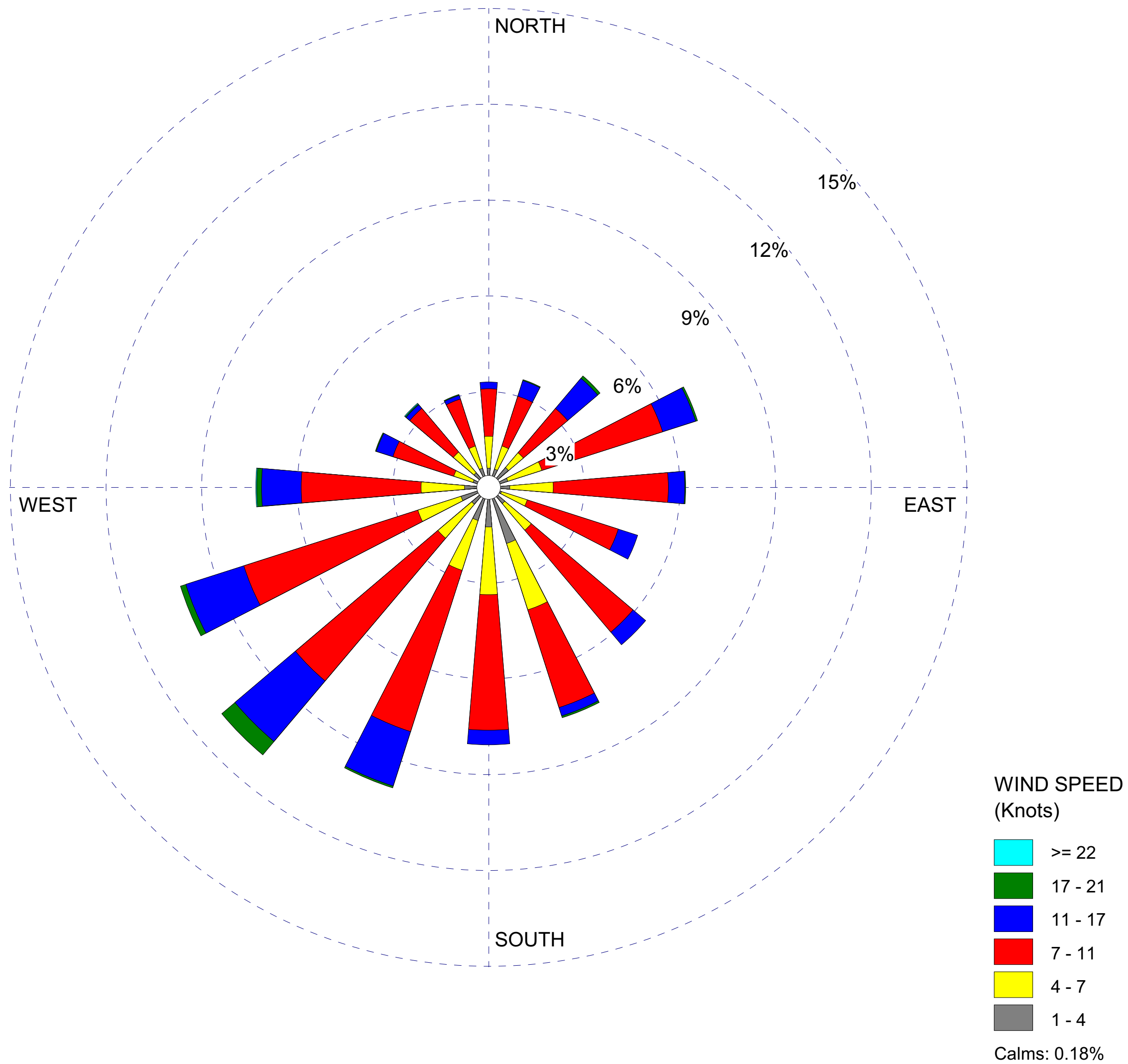

Figure 2.3-213 (Sheet 6 of 12)
VEGP 60-m Level June Wind Rose (1998–2002)

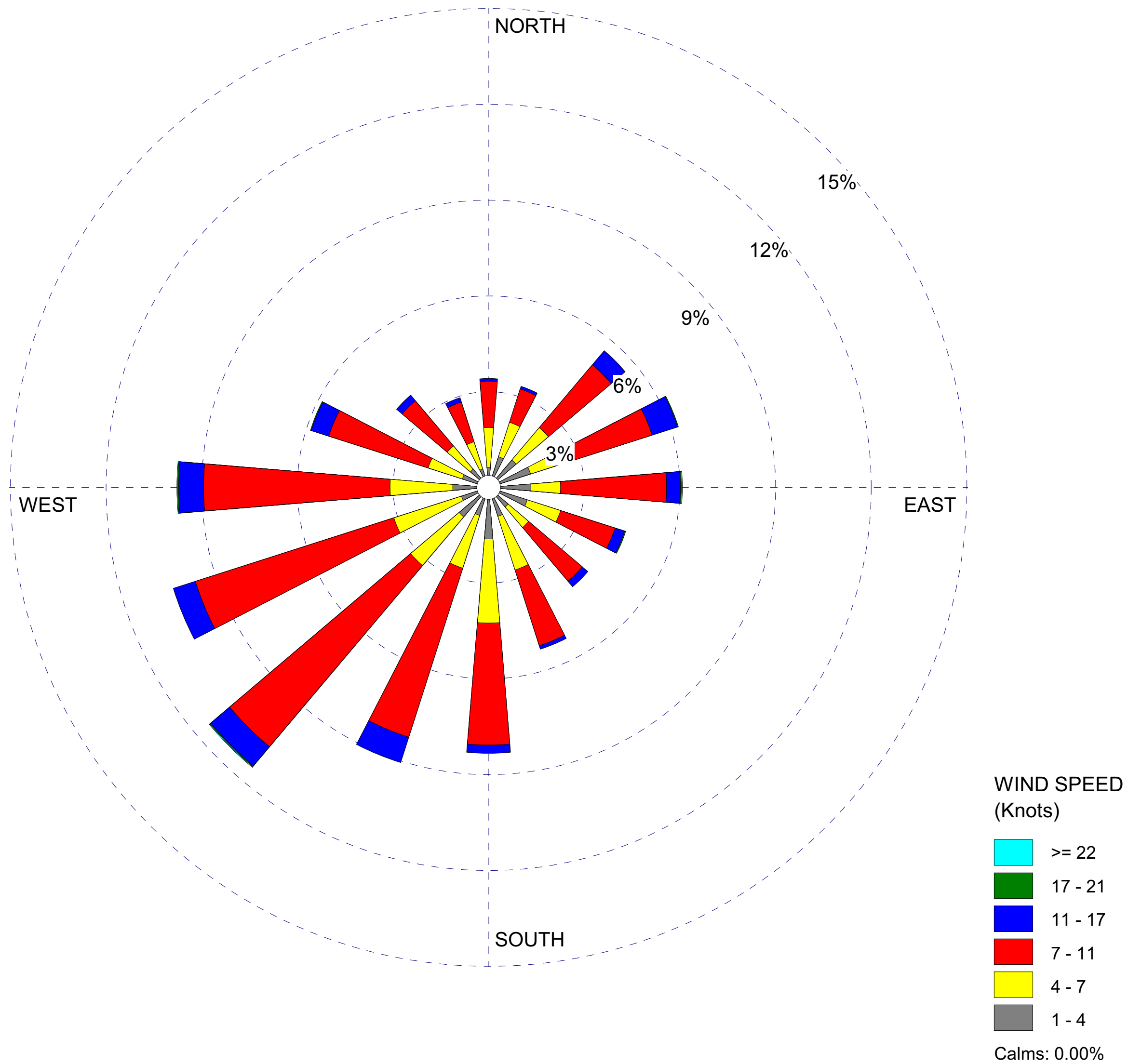

Figure 2.3-213 (Sheet 7 of 12)
VEGP 60-m Level July Wind Rose (1998–2002)

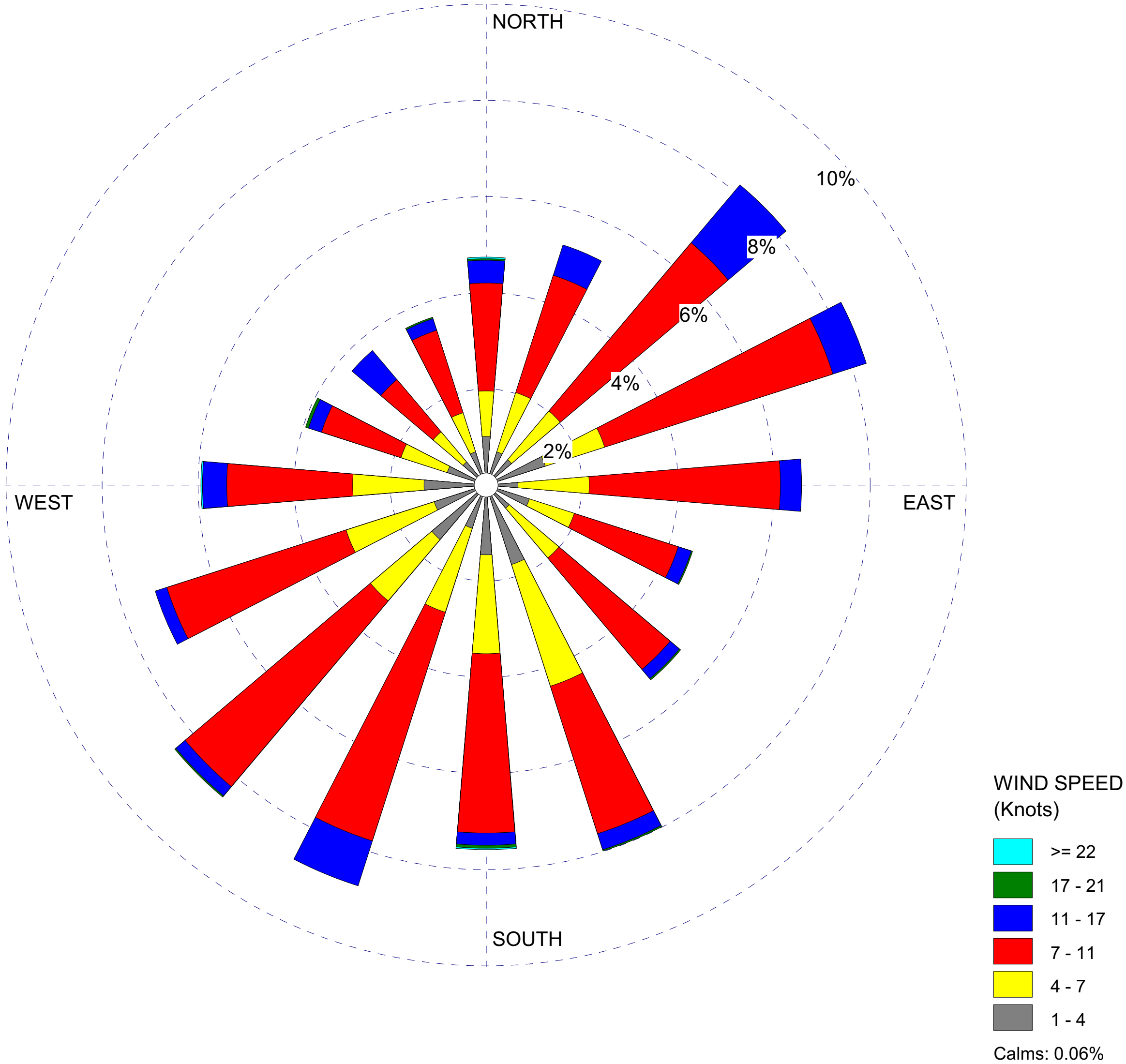

Figure 2.3-213 (Sheet 8 of 12)
VEGP 60-m Level August Wind Rose (1998–2002)

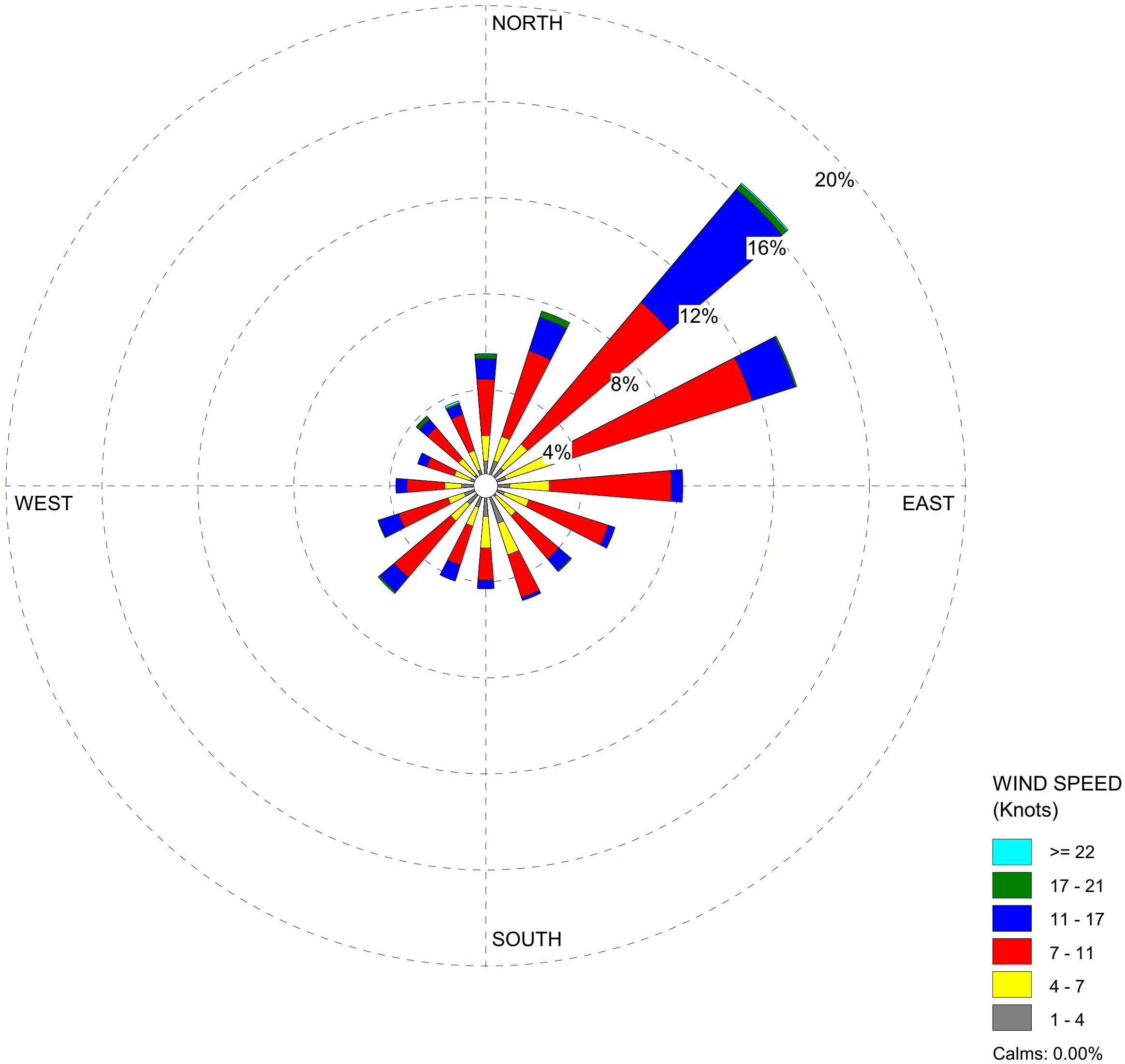

Figure 2.3-213 (Sheet 9 of 12)
VEGP 60-m Level September Wind Rose (1998–2002)

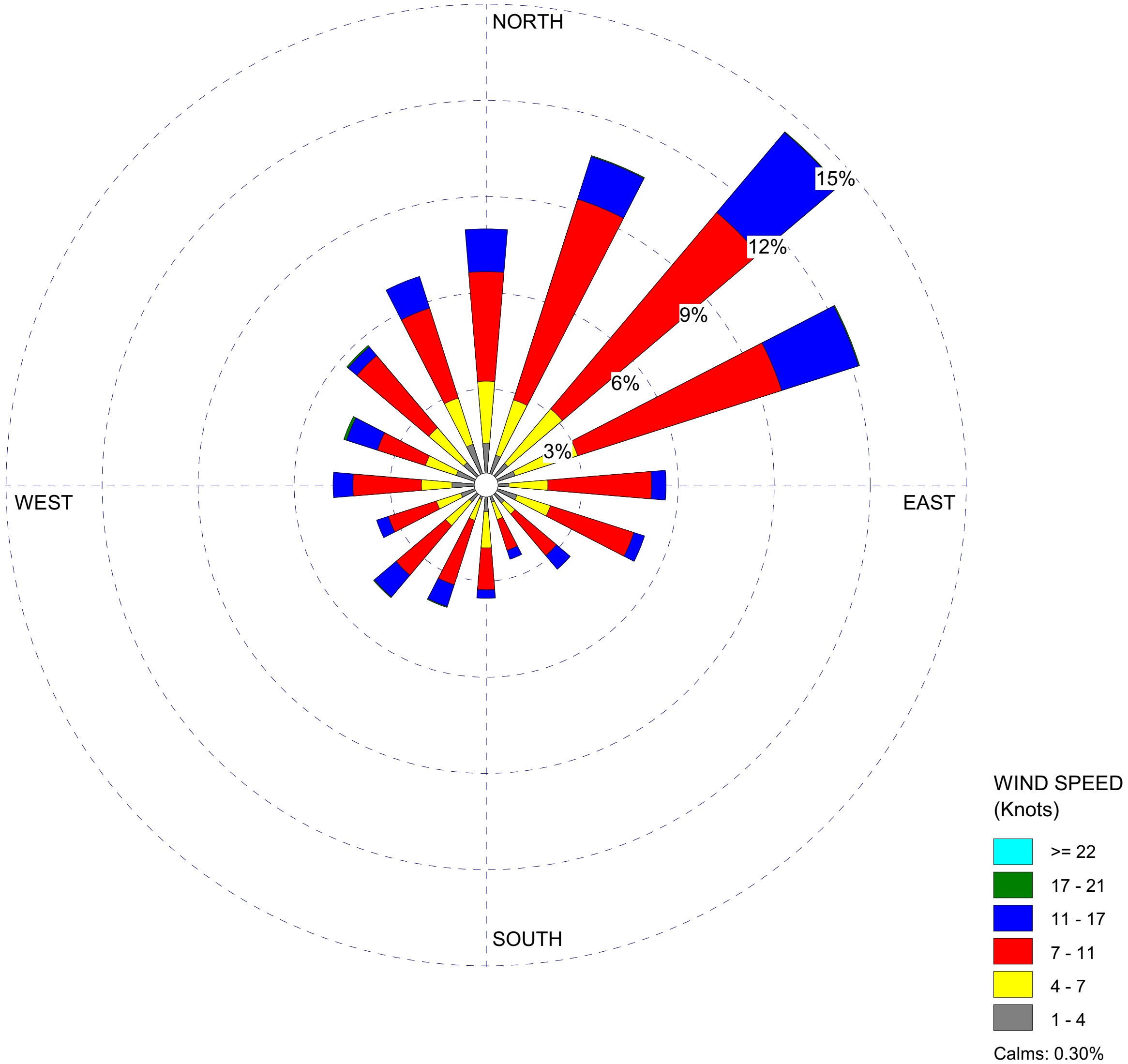

Figure 2.3-213 (Sheet 10 of 12)
VEGP 60-m Level October Wind Rose (1998–2002)

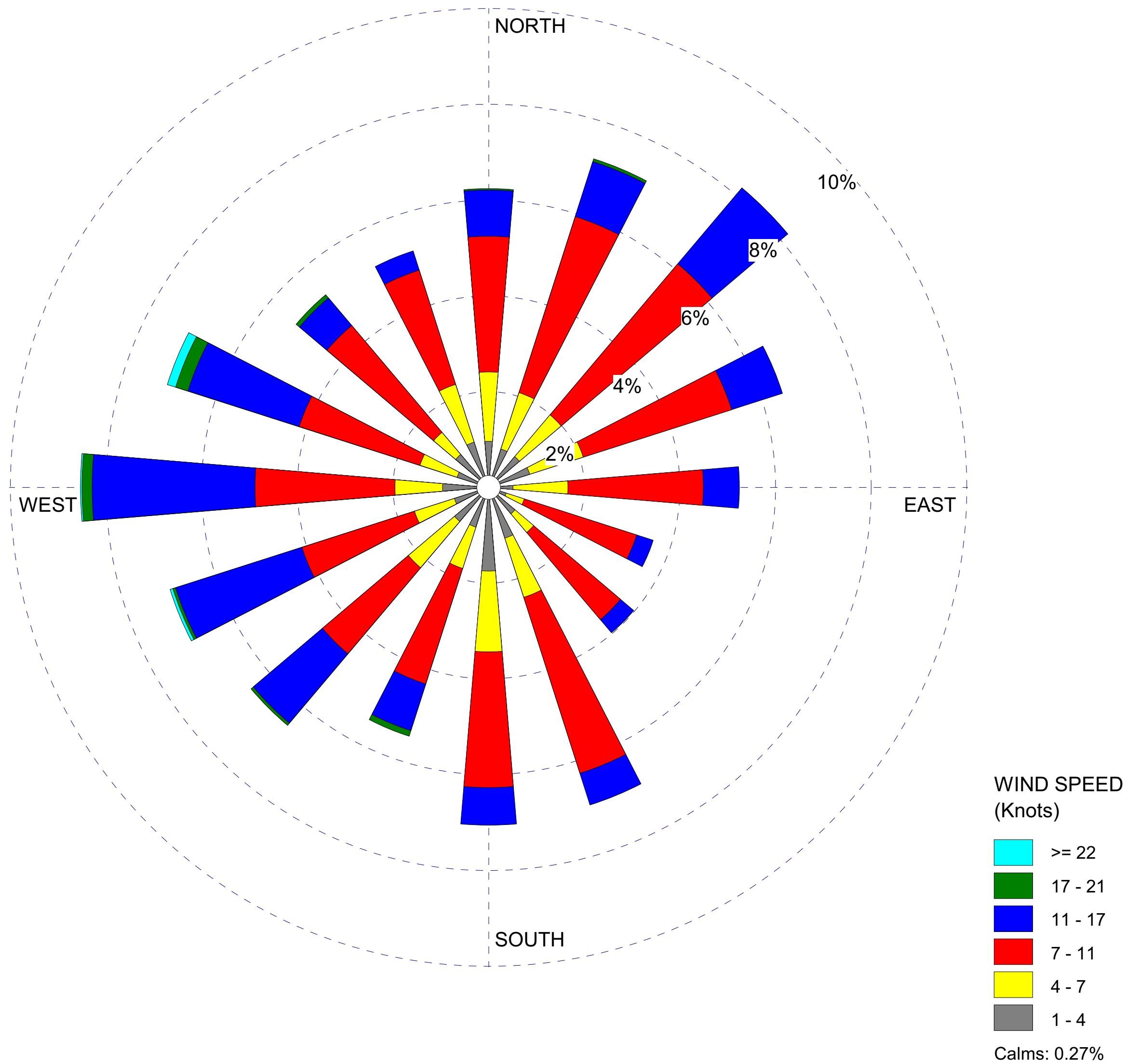

Figure 2.3-213 (Sheet 11 of 12)
VEGP 60-m Level November Wind Rose (1998–2002)

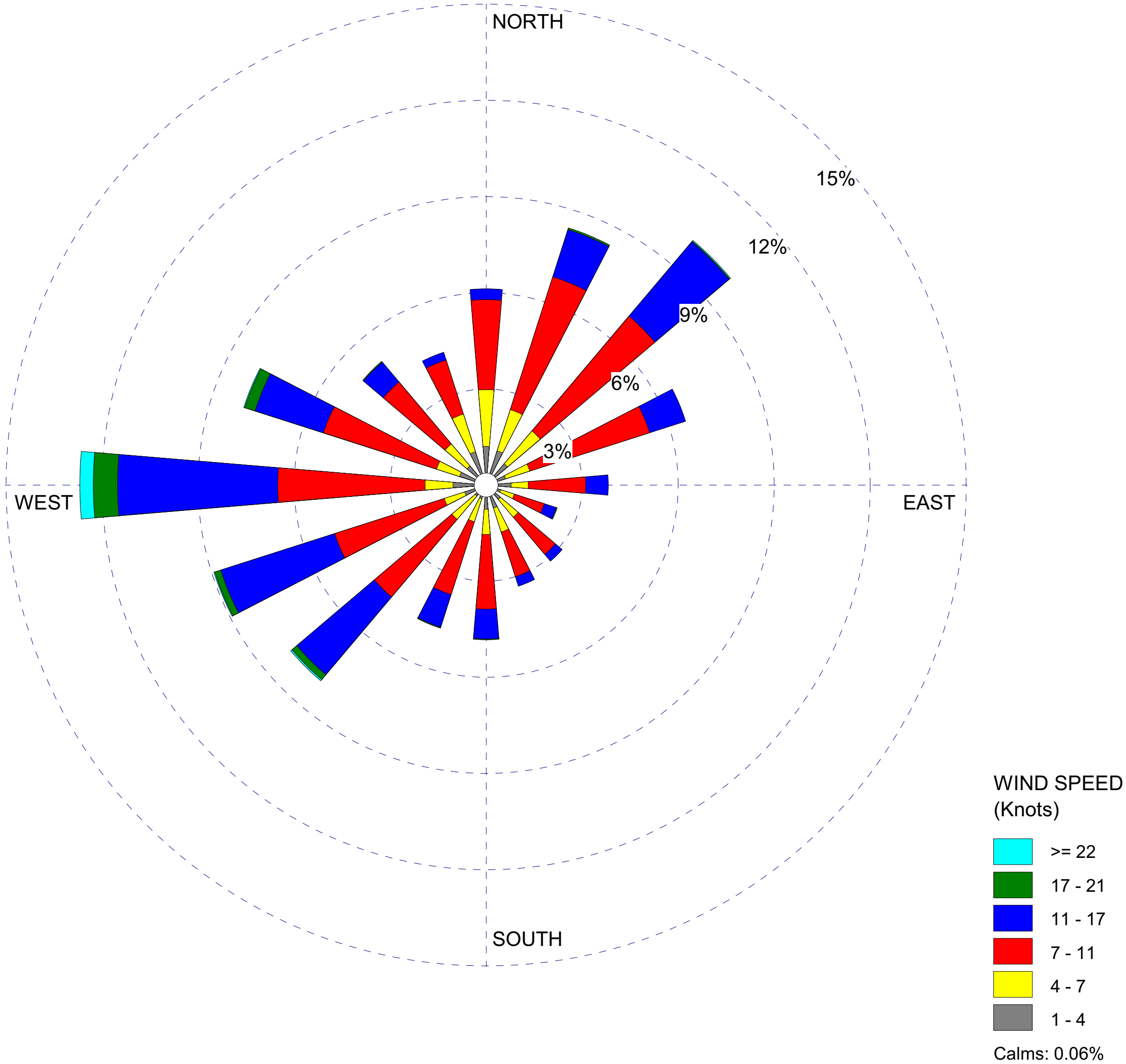

Figure 2.3-213 (Sheet 12 of 12)
VEGP 60-m Level December Wind Rose (1998–2002)

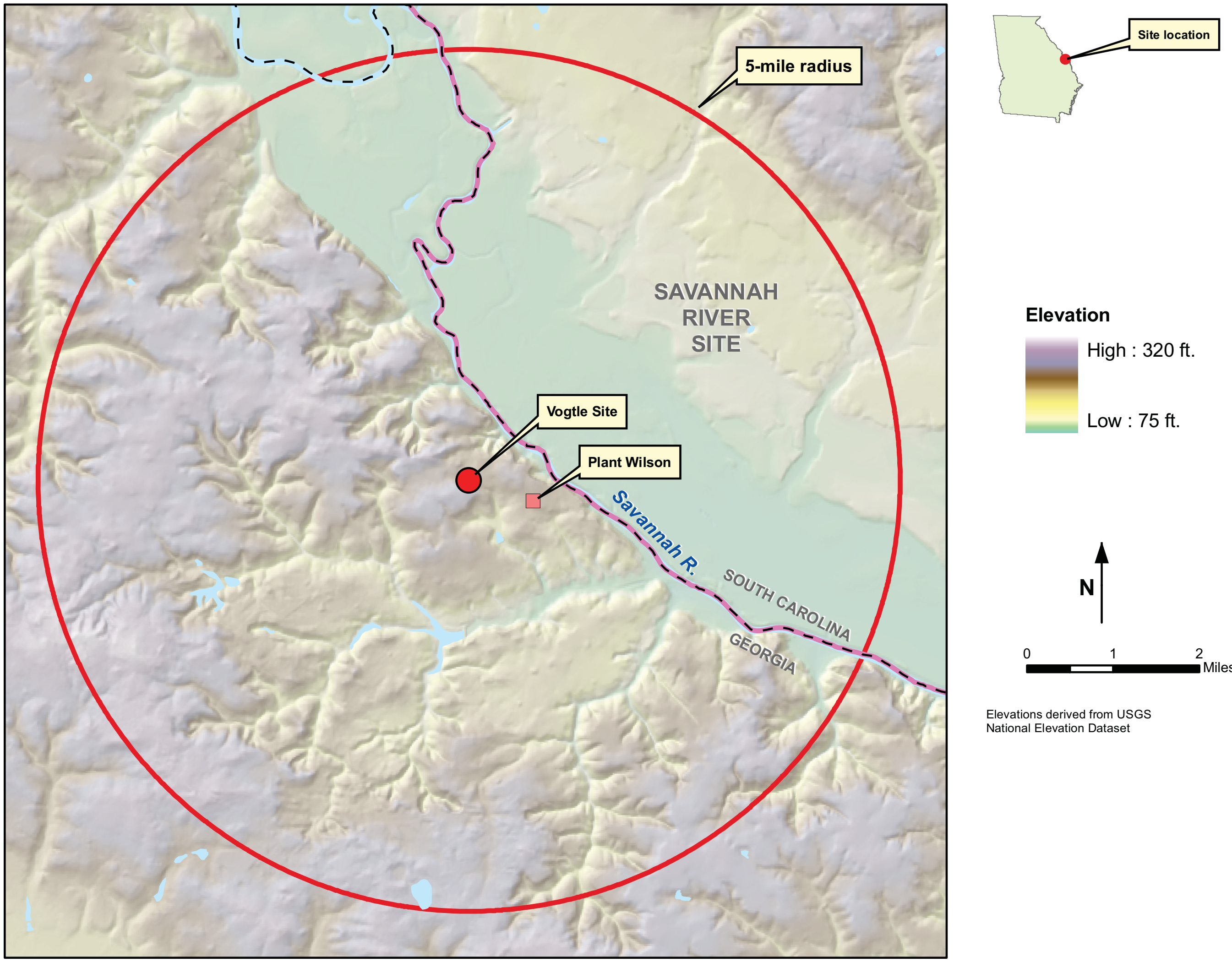

Figure 2.3-214
Topographic Features Within a 5-Mile Radius of the VEGP Site

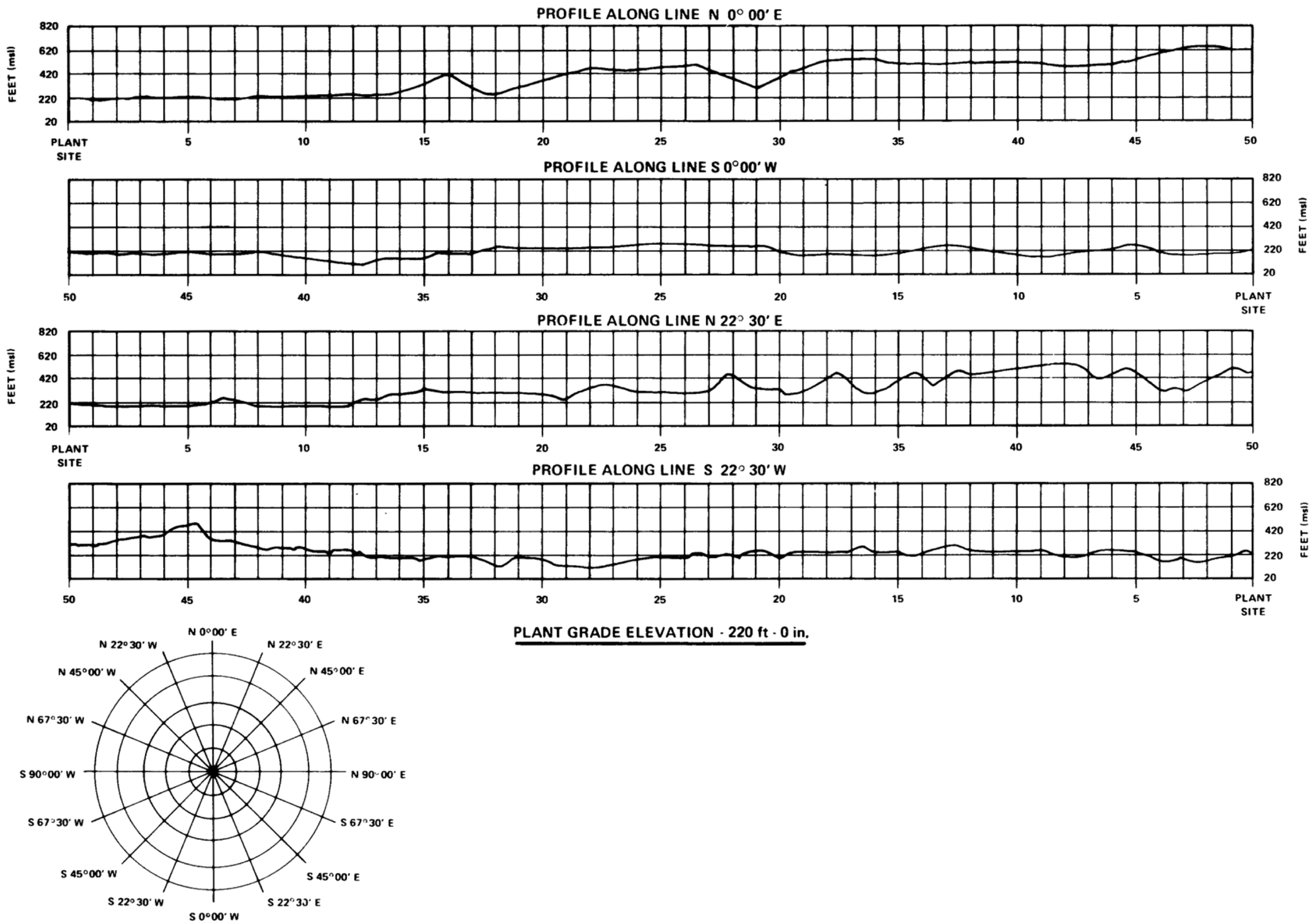

Figure 2.3-215 (Sheet 1 of 4)
Terrain Elevation Profiles Within 50 Miles of the VEGP Site

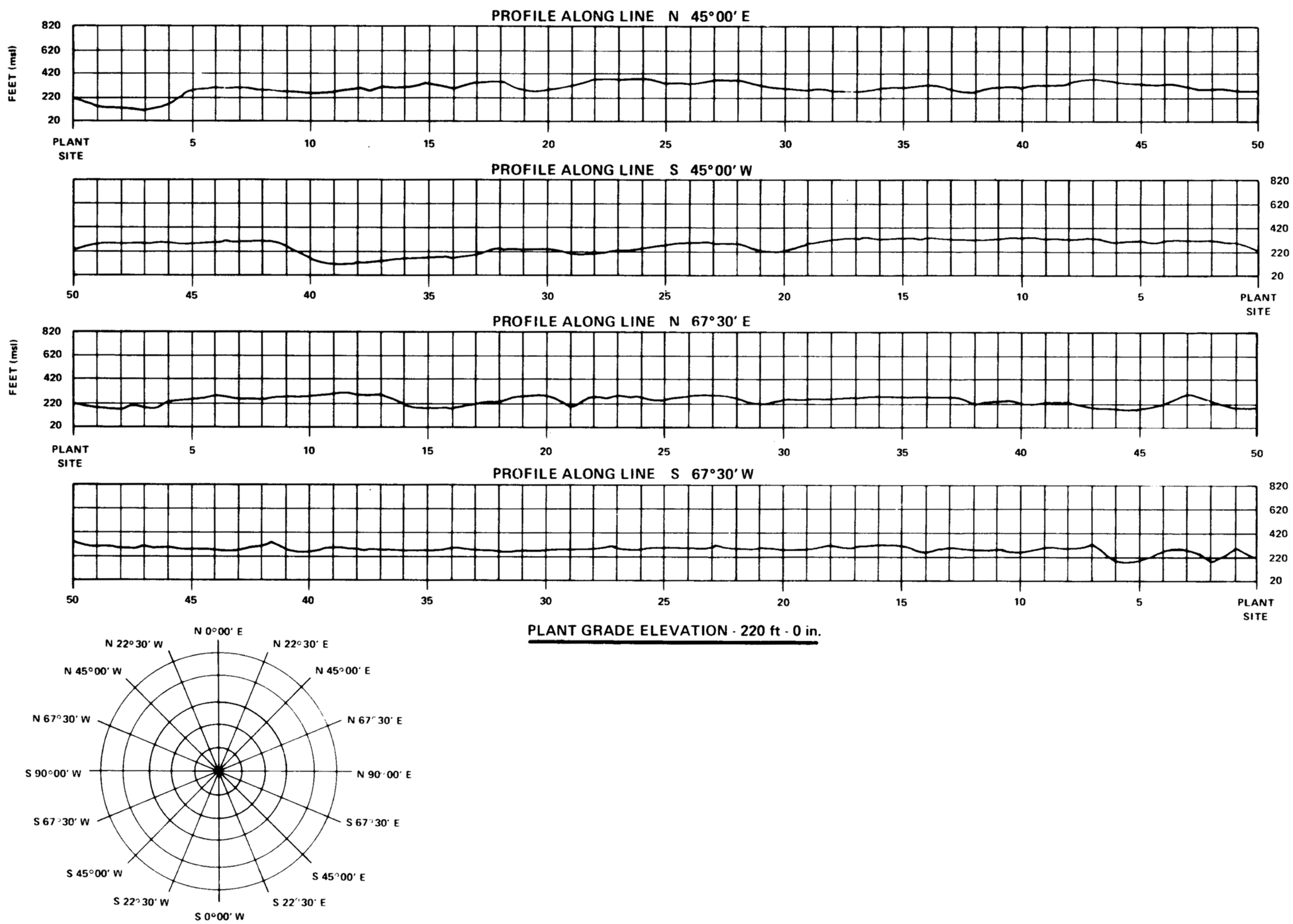

Figure 2.3-215 (Sheet 2 of 4)
Terrain Elevation Profiles Within 50 Miles of the VEGP Site

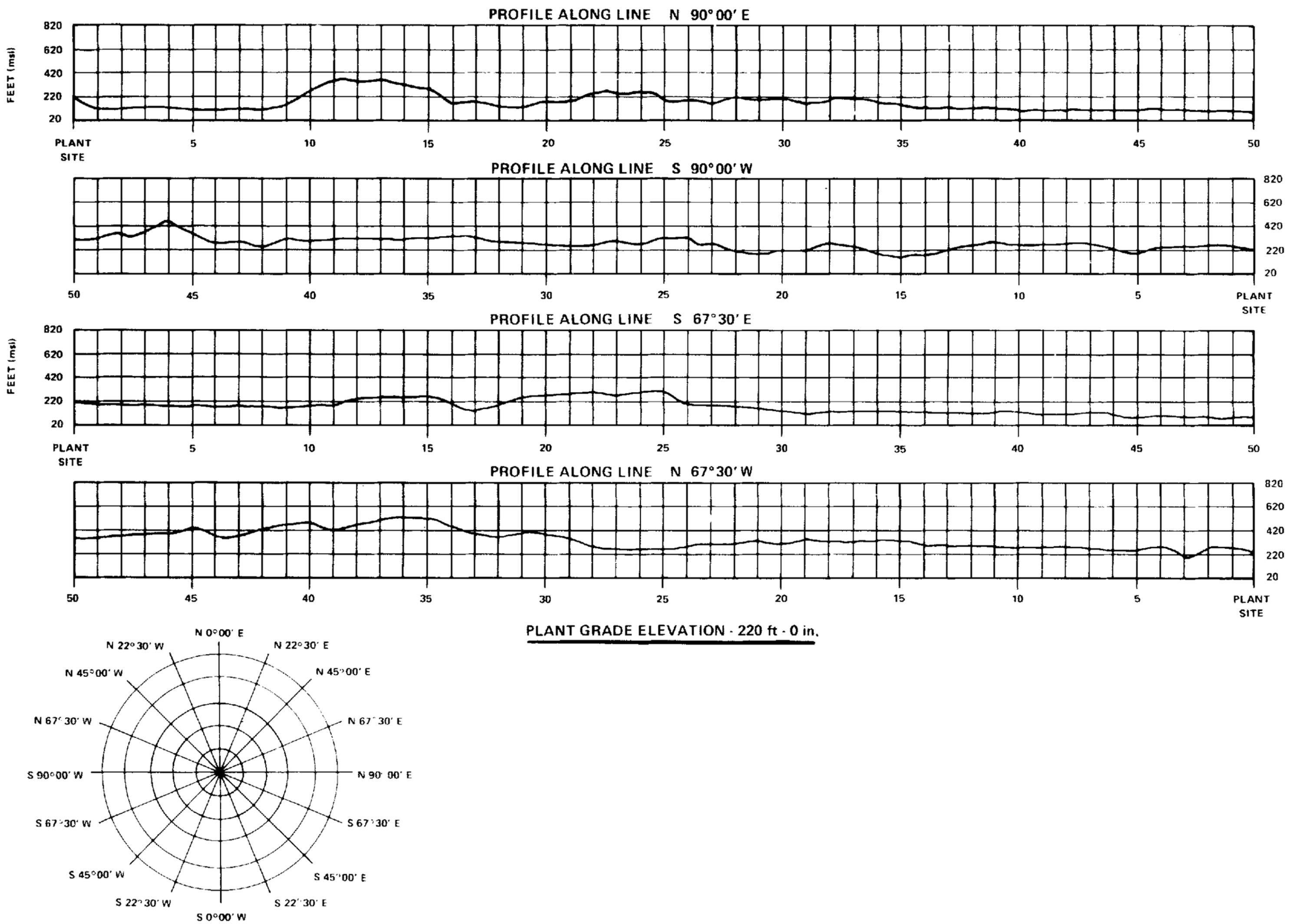

Figure 2.3-215 (Sheet 3 of 4)
Terrain Elevation Profiles Within 50 Miles of the VEGP Site

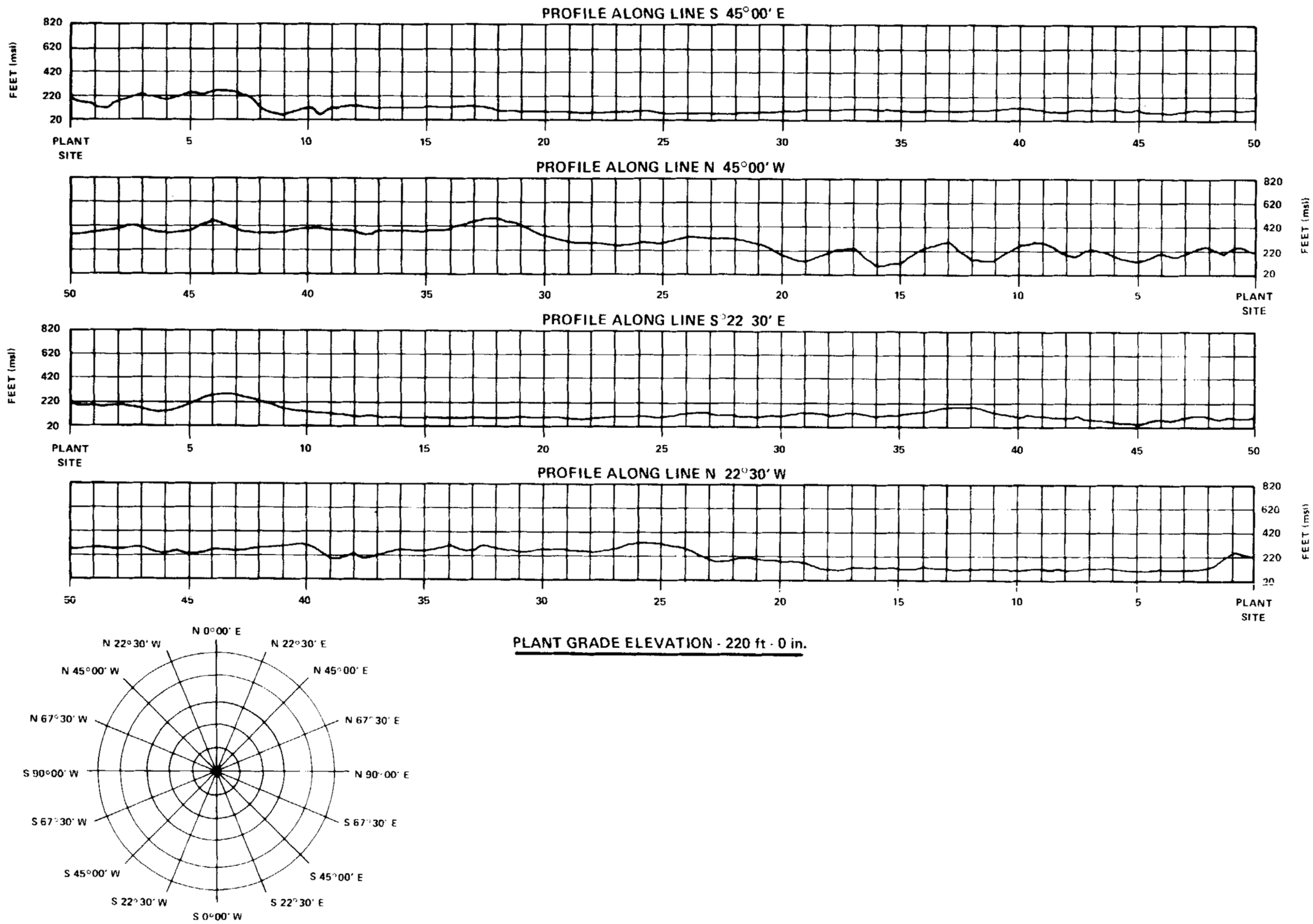

Figure 2.3-215 (Sheet 4 of 4)
Terrain Elevation Profiles Within 50 Miles of the VEGP Site

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 (Subsection 2.4.1)
- Floods (Subsection 2.4.2)
- Probable Maximum Flood on Streams and Rivers (Subsection 2.4.3)
- Potential Dam Failures (Subsection 2.4.4)
- Probable Maximum Surge and Seiche Flooding (Subsection 2.4.5)
- Probable Maximum Tsunami Flooding (Subsection 2.4.6)
- Ice Effects (Subsection 2.4.7)
- Cooling Water Canals and Reservoirs (Subsection 2.4.8)
- Channel Diversions (Subsection 2.4.9)
- Flood Protection Requirements (Subsection 2.4.10)
- Low Water Considerations (Subsection 2.4.11)
- Groundwater (Subsection 2.4.12)
- Accidental Releases of Liquid Effluents in Ground and Surface Waters (Subsection 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 (Reference 208), and further subdivided into USGS Hydrologic Unit Code (HUC-12) subbasins (Reference 219), are shown in Figure 2.4-204. The drainage areas of the NWS subbasins are given in Table 2.4-208.

Two Westinghouse pressurized water reactors (PWRs), rated at 3,625.6 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).** The AP1000 units, referred to as Units 3 and 4, are located west of and adjacent to existing Units 1 and 2 as shown in Figure 1.1-202. 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 is 220 feet msl. An extensive site storm water drainage system was developed during construction of Units 1 and 2 and is 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 (Reference 209):

- 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 (Reference 211).

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 (Reference 211).

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 (Reference 211):

- 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 (Reference 211).

The following principal streams make up the Savannah River stream system (Reference 211):

- 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 (Reference 211). Table 2.4-209 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 60ºF. January, which is usually the coldest month of the year, frequently has night temperatures of 20ºF or lower. July and August, the hottest months of the year, have many days with temperatures over 90ºF. In the lower section of the basin, the winters are milder and the summer temperatures higher (Reference 211).

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 (Reference 211).

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 2.4-210 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 Subsection 2.4.2; daily discharge data for these gages are used in Subsection 2.4.11.3. Summary statistics characterizing the seasonal flow variability are discussed below.

As indicated in Table 2.4-209, 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 2.4-213, 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 2.4-205, based on USGS records for the years of record of each gage, without accounting for the impact of changes in upstream regulation. Tables 2.4-211 through 2.4-213 show the mean daily discharge for the years of record for each of the three gages presented in Figure 2.4-205.

2.4.1.2.2 Local Site Drainage

Local drainage is shown in Figure 2.4-206, 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 2.4-206.

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 2.4-214.

2.4.1.2.3 Dams and Reservoirs

There are a number of water control structures on the Savannah River and its major tributaries (Reference 213, Reference 210, and Reference 211). Table 2.4-215 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 (Reference 211):

<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 (Reference 211).

Each project is described briefly in the following paragraphs (Reference 211).

The Hartwell Lake and Dam 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.

The Richard B. Russell Lake and Dam 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.

The J. Strom Thurmond Lake and Dam 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 (Reference 212).

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 (Reference 209) 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 (Reference 209).

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 2.4-216 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 Subsection 2.4.12, while Subsection 2.4.13 discusses the consequences of liquid effluent releases to surface waters.

2.4.1.2.6 Water Consumption

The 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 29,125 gpm, with a maximum of 30,585 gpm. Groundwater consumptive use is 752 gpm on average, with a maximum of 2,797 gpm. During normal operation, approximately 305 gpm of groundwater is returned as surface water to the Savannah River. Table 2.4-217 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 29,125 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 2.4-217 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.

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 Subsections 2.4.5 and 2.4.6).

Table 2.4-218 (Reference 214) 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 (Reference 214) to 360,000 cfs for a reported maximum flood stage of 40 ft (Reference 213). 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 2.4-218).

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 (Reference 211). In Figure 2.4-207 (Reference 213), 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 2.4-209), has a record length significantly shorter than that of the Augusta gage and contains no observations before upstream dams were closed. Table 2.4-219 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 2.4-219, 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 2.4-208, 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 2.4-209 (Reference 213). 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 (Reference 213).

Figure 2.4-209 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 Subsection 2.4.4.

2.4.2.2 Flood Design Considerations

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

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

The 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 1.1-202. The site is located on a high bluff on the west bank of the Savannah River. The site grade for the new units is El. 220 ft msl, similar to the existing VEGP units, well above the probable maximum flood stage of the Savannah River, as discussed in Subsection 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 Subsection 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* (Reference 220), as detailed in Subsections 2.4.3 through 2.4.7.

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

The controlling event for the VEGP site was determined to be from the breach of the upstream dams, estimated as described in Subsection 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 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" (Reference 222). 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 (Reference 221).

As can be seen in Figure 2.4-206, 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 (Reference 222). 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 2.4-220 and the estimated curve is plotted in Figure 2.4-210.

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.

In general, the storm water management system developed for Units 3 and 4 has been integrated with the existing facilities as possible; runoff from Units 3 and 4 is directed away from Unit 1 and 2 structures, to outfall to the west and south of the VEGP site.

The storm drain system has been designed in accordance with good engineering practice, following all applicable federal, state, and local storm water management regulations. In addition, site grading has been 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. The storm drain system is visually inspected to verify the flow path is unobstructed. The system is observed under simulated or actual precipitation events to verify that the runoff from roof drains and the plant site and adjacent areas does not result in unacceptable soil erosion adjacent to, or flooding of, Seismic Category I structures.

The design elements of the VEGP Units 3 and 4 storm water management system pertaining to the local PMP flood event are described below.

As shown in Figure 2.4-201, the VEGP Units 3 and 4 power block is graded to direct runoff east and west to three north-south ditches which will outfall to the concrete-lined main ditch, running east and west for 2,000 feet along the south side of the power block. The trapezoidal ditch cross section has a 10-foot bottom width with 2:1 side slopes, sized to provide adequate conveyance for PMP discharges. At the southwest corner of the power block, the main ditch turns due south, and the bottom width is increased to 14 feet. From the west, it intercepts runoff from the construction laydown area; from the east it intercepts discharge from three ditches draining the cooling tower block.

The main ditch has a mild slope (0.22%) for its first 3,800 feet, at which point the slope increases to over 5% before outfalling about 4,500 feet from its upstream end into Debris Basin No. 2, which drains to an unnamed tributary of Daniels Branch, about 2,500 feet upstream of Telfair Pond.

The main ditch drains runoff from a total area of about 473 acres during the PMP event, including about 80 acres from the VEGP Units 3 and 4 power block, 97 acres from the cooling tower area, 56 acres south of the cooling tower, and 82 acres from the laydown area. An additional 133 acres from an area north of the haul road and 25 acres from VEGP Units 1 and 2 power block are assumed to drain to the main ditch for the PMP design rainfall event due to blocked culverts.

The local PMP event was modeled in HEC-HMS, which is an industry standard program for this application. For inputs of rainfall and drainage basin characteristics, the program outputs stream flow hydrographs at selected locations within the drainage basin (Reference 203).

The design rainfall hyetograph was developed in HEC-HMS utilizing the frequency storm option in the Meteorologic Models module (Reference 204). This option requires the input of PMP point depths for durations of 5, 15, 60, 120, 180, and 360 minutes.

Based on the logarithmic fit to the data shown in Table 2.4-220, a PMP total depth was estimated for the missing durations, as indicated in Table 2.4-201. An intensity position of 50 percent was selected for the HEC-HMS calculation, consistent with the alternating block pattern used in standard analysis (Reference 201) The rainfall hyetograph developed from the data is shown in Figure 2.4-202.

Elements within the HEC-HMS basin model include subbasins, reaches, and junctions. Runoff hydrographs were developed for subbasins and were routed through the channel system along reaches connected by junctions (Reference 204).

This calculation utilized the SCS Hydrograph Methodology (Reference 204), which requires the following parameters for each subbasin:

- Drainage Area, in square miles
- Runoff Curve Number and Initial Abstractions
- Lag Time, in minutes
- Base flow, in cfs

Drainage areas were delineated and measured for each subbasin shown in Figure 2.4-201.

The runoff curve number (CN) was selected as 98 for all types of cover to provide a conservative estimate of runoff volume and peak discharges and to account for nonlinear basin response to extreme rainfall events. Under normal flood conditions, the area-weighted average for each subbasin could be expected to vary between 50 and 75, while a CN value of 98 is typically used for impervious areas.

The lag time was estimated as 60 percent of the time of concentration, which is the time required for all areas of the drainage basin to be contributing to outflow. It was calculated for each subbasin as the sum of the overland, shallow concentrated, and channel flow times along the critical flow path through the basin using standardized equations (Reference 204).

An assumption of the PMP design storm is that a 50-percent PMP storm has occurred 3 days prior to the start of the rainfall associated with the actual PMP event, so some flow in the drainage ditches would be expected as the result of interflow draining from the pervious areas of the upstream watershed, although it would not be a significant quantity for this site, considering the limited drainage area. For this site, base flow is taken as zero for subbasins that are completely paved. Base flow is estimated on a 100 cfs per square mile basis for subbasins with uncovered ground.

The SCS unit hydrograph parameters calculated for each of the subbasins in the HEC-HMS models are provided in Table 2.4-202.

The subbasin hydrographs are added at junctions and routed through channel reaches. Straight lag time was used for the smaller reaches; the kinematic wave routing option for most of the main channel reaches. The routing parameters are shown in Table 2.4-203.

Peak discharges from all subbasins, at all junctions, and at the downstream end of each of the routing reaches resulting from modeling the PMP rainfall event in HEC-HMS are summarized in Table 2.4-204. The highlighted entries indicate junctions along the main ditch. The hydrographs simulated for these junctions are shown in Figure 2.4-203.

The backwater analysis for the PMP drainage network was developed in HEC-RAS (Reference 205). Cross sections were developed for the main drainage ditch and feeder channels with topographic data for the overbank area, using the proposed geometric configuration for the channels. The locations of the cross sections used in the HEC-RAS model are shown in Figure 2.4-201a.

The assumptions made and the data utilized in the development of the hydraulic model are as follows:

- All channels are concrete lined, so no local scoured-out cross sections are utilized in the model.
- All culverts in the model are assumed to be 100% blocked by debris collected from the catchment.
- The blocked culverts within the power block area are modeled as in-line weirs in HEC-RAS following common hydraulic engineering practice (Reference 205). The effect of the blocked culvert in Feeder Ditch 4 is accounted for by adjusting cross section geometry to indicate the ditch is filled in at the culvert location.
- Peak discharges from the HEC-HMS model were used at all sections in a steady-state calculation. Based on the close coincidence in time of peak discharges along the main channel and in the contributing subbasins, as shown in Table 2.4-204 and Figure 2.4-202, this was considered to be a reasonable simplification.

Peak PMP discharges simulated in HEC-HMS at eight locations along the main channel (nodes M1 through M8, as shown in Figure 2.4-201) were utilized in HEC-RAS at the cross sections indicated in Table 2.4-205.

In HEC-HMS, discharge was calculated at two points along each of the feeder ditches 1, 2, and 3, within the power block area. To better represent the lateral inflow to the feeder ditches along their entire length, the discharge from the two HEC-HMS nodes for each ditch were distributed linearly to each section in the models of the respective ditches to better represent lateral inflow, as summarized in Table 2.4-206.

The model was run with the mixed flow regime option with the downstream boundary condition taken as normal depth at Section 45+00, with the energy slope equal to the channel slope at that point of 5 percent (section stationing is shown at 500-foot intervals in Figure 2.4-201). The upstream boundary conditions were also taken as normal depth with an energy slope of 0.0001 to account for the severe backwater effect at the upstream ends of the branches of the drainage system.

The Manning's n roughness values used in the model were selected for standard conservative assumptions (Reference 202) as follows:

- concrete feeder ditches (assumed to be well maintained) with n = .014 and overbank areas assumed to be gravel bottom and concrete curbs with n = .020
- all other ditches assumed to be float-finished concrete lining with n = .015 and overbank areas assumed to be short grass with n = .030

The results of the mixed-flow regime back water calculation for PMP discharges in the drainage network are presented in Table 2.4-207. Flow is supercritical in the steep reach of the main ditch from the downstream section up to section 37+00, with control (Froude No. = 1) at section 38+00, with a velocity of 16.6 fps and a depth of 14.14 feet. Velocities decrease and depths generally increase in the mild-sloped (S = .0022) reach upstream of that section to 3.7 fps and 15.98 feet respectively at section 20+00, and 0.9 fps and 11.98 feet respectively at section 1+00.

The feeder ditches draining the power block area are subject to high tailwater conditions in the main ditch for the PMP runoff event. The HEC-RAS output indicates that the maximum floodwater surface elevation would be between 219.28 ft msl in the SW corner and 219.47 ft msl in the NE corner of the VEGP Units 3 and 4 power block. As all safety-related facilities have entry elevations at or above 220 ft msl, it has been determined that the maximum local PMP flood elevation is at least 0.53 ft below any entry to any safety related facility, and the flooding of safety-related facilities due to this PMP event does not occur. Configuration control of the plant layout, as assumed in the hydraulic model described above, is governed by applicable plant procedures.

In summary, the main ditch system has been designed to convey the peak discharge of the PMP flood event safely offsite. In addition, site grading is sufficiently sloped to convey runoff overland from the local PMP event away from all buildings and safety-related equipment, without flooding.

The required maintenance for the drainage ditches and overbank areas will be determined during the quarterly walk-through inspections of the drainage features (main drainage and feeder ditches and their overbank areas) in the Units 3 and 4 portion of the protected area and from the protected area fence through the Units 3 and 4 cooling tower area.

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 Subsection 2.4.2, or riverine flooding, as described below.

The location of VEGP Units 3 and 4 is adjacent to and generally to the west of the existing VEGP units, as illustrated in Figure 1.1-202. The site is located on a high bluff on the west bank of the Savannah River. The site grade for the new units is established as 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" (Reference 220).

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 Subsection 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 Subsection 2.4.3.2, and the calculation of the associated flood stage using a steady-state hydraulic model and wave run-up, reported in Subsection 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 2.4-221 and are described in more detail below.

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 2.4-208 and are used for the PMF approximation described in Subsection 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 in Table 2.4-222.

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 2.4-222.

A logarithmic plot of the power curve fit to these values is presented in Figure 2.4-211. Based on the curve fit to the data and the currently estimated drainage area of 8,304 sq mi (as discussed in Subsection 2.4.1), the estimated PMF for the VEGP site is about 920,000 cfs. This point is located on the curve in Figure 2.4-211, along with a data point for VEGP (reported as Alvin W. Vogtle), presented on page 4 of 17 in Table B.1 of Regulatory Guide 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 Regulatory Guide 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 (Reference 224).

The steady-state model was adapted from the dynamic model used for the analysis of the dam-break scenario described in Subsection 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 (Reference 225).

The results for the cross-section nearest to the VEGP site (River Mile 150.906 in the model) are shown in Table 2.4-223.

The longitudinal profile output for the Savannah River for this model is reproduced as Figure 2.4-212. The cross section developed for the VEGP site is shown in Figure 2.4-213.

The estimated maximum stages at the VEGP site for the PMF estimated per the approximate method outlined in Regulatory Guide 1.59 are shown in Table 2.4-224.

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 setup, 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 Subsection 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 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.

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 2.4-216 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 2.4-214 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 2.4-216 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 (Reference 211). 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 (Reference 211). 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 2.4-216, the most severe failure mode is estimated. The combined normal pool storage volumes behind the dams in each mode are shown in Table 2.4-225.

Table 2.4-225 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) (Reference 224) 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* (Reference 211) 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 (Reference 228). Area-capacity curves for each of the five reservoirs are shown on Figures 2.4-215 through 2.4-219, 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 (Reference 231). 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 2.4-226. 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 2.4-227 through 2.4-230.

For the Thurmond Dam, the FERC (1987) equation from Table 2.4-226, 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 (Reference 231), 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 2.4-226 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 (Reference 231) 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 2.4-228 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 2.4-220, the total dam width at the top of the dam is about 5,700 ft (Reference 211). 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 (Reference 211). 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 2.4-220 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 2.4-230 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 2.4-221, the total dam width at the top of the dam is about 4,500 ft. (Reference 211). 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 (Reference 211). 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 2.4-221 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 (Reference 229). 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 (Reference 232). 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

(Reference 232) 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* (Reference 211). 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 (Reference 232). The below-water portions of the cross-section data were obtained from fishing maps with depth contours (Reference 226; Reference 227).

Roughness coefficients (Manning's n) were estimated using procedures developed by the US Geological Survey (Reference 233). 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 (Reference 211), 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 (Reference 211). 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 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 2.4-222. 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 2.4-223 through 2.4-225, 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 (Reference 220). The *Coastal Engineering Manual* (Reference 230) 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 2.4-226.

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 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.

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 (Reference 234). 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 (Reference 235).

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 2.4-231.

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 (Reference 220). 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 2.4-231.

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.

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 (Reference 236). Computer models suggested a wave height of 10 ft along the east coast of the US (Reference 237) 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.

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 Subsection 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) (Reference 239). 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 2.3-205). 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 2.4-227). 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 2.4-232). 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 (Reference 238) are presented in Table 2.4-233. 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 (Reference 240).

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 2.4-233, 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 are 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, are located upstream from the existing river intake structure for the VEGP Units 1 and 2. Makeup water from the Savannah River are 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 is supplied from site groundwater wells or a site water storage tank. Consequently, no water is 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 does not have any impact on safety-related UHS or non-safety-related SWS of the proposed AP1000 units.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 Cooling Water Canals

The VEGP Units 3 and 4 use a closed cycle cooling system for condenser heat rejection and use wet, natural-draft, cooling towers for circulating water system cooling. Makeup water from the Savannah River is required to replace evaporative water losses, drift losses, and blowdown discharge. The river intake for VEGP Units 3 and 4 withdraws makeup water from the Savannah River at a maximum rate of approximately 61,145 gpm (136.2 cfs). The intake system is located upstream of the river intake of the existing VEGP units. The makeup water is 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 is not part of the safety-related facilities for VEGP Units 3 and 4, and the river intake canal and structure 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 is supplied from site groundwater wells; therefore, the SWS does not depend on the river intake system.

The river intake system for VEGP Units 3 and 4 consists of an intake canal and an intake structure. The design details of the river intake system have been established. An overview of the conceptual design is provided below.

The river intake canal is approximately 320 ft long and 120 ft wide at the intake structure entrance and approximately 170 ft wide at the entrance to the river, with a bottom elevation of about El. 70 ft msl. The canal upstream of the intake structure apron is unpaved. The river intake canal acts as a siltation basin and incorporates 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 is about 0.1 fps, calculated based on a maximum makeup water demand of 136.2 cfs. Because the river intake canal also acts as the siltation basin, maintenance dredging may be necessary to maintain the canal invert elevation. Also, the canal embankment slopes and benches are protected by earthen berms armored with rip-rap of appropriate design specifications on the exterior (river-side) slopes of the berms.

The intake structure, located at the end of the river intake canal, houses 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 are 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 do not rely on the Savannah River for safe shutdown, a minimum river water level is not necessary for safety-related cooling water supply.

2.4.8.2 Reservoirs

VEGP Units 3 and 4 do 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 2.4-206.

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 (Reference 241; Reference 242; Reference 243) 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 2.4-228. 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 AP1000 reactor plants do not use water from the river intake. Hence, the river intake is not 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.

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 Subsection 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 are 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 has been 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 2.4-206. Thus, the topography of the proposed site facilitates drainage of intense rainfall events. Drainage facilities for the VEGP Units 3 and 4 site have been 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 also assumes that all drainage structures (e.g., culverts, storm drains, and bridges) are blocked during the PMP event. The safety-related structures, systems, and components are still safe from resulting flood hazards.

Additionally, the design of the drainage facilities and the development of construction and operation plans incorporates 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 is directed away from the existing drainage facilities of VEGP Units 1 and 2. Hence, drainage from the VEGP Units 3 and 4 site does not interfere with the safety-related structures, systems, and components of VEGP Units 1 and 2.

The roofs of all safety-related structures have been 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 Subsection 2.3.1.3.4.

Although the river intake is not a safety-related facility, rip-rap protection of embankment slopes has been 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 have been followed in the design of the drainage facilities.

The maximum flood elevation in the Savannah River at the VEGP site is El. 178.10 ft msl, resulting from the cascading failure of upstream dams including wind setup and wave run-up, as discussed in Subsection 2.4.4. This elevation is well below the VEGP site grade at El. 220 ft msl.

Subsection 2.4.2 subsequently considered the flooding effects of local intense precipitation (also termed as the local probable maximum precipitation or local PMP) on the Units 3 and 4 safety-related structures at the VEGP site. A local PMP drainage analysis was performed by conservatively assuming that all underground storm drains and culverts were clogged. Details of the local PMP analysis and the resulting flood levels are presented in Subsection 2.4.2. As indicated in Subsection 2.4.2, the maximum water level in the Units 3 and 4 power block area due to the local PMP flood event is calculated to be at El. 219.47 ft msl. The entrances and openings for all safety-related facilities are located at or above the VEGP site grade of EL. 220 ft msl.

Thus, none of the VEGP Units 3 and 4 safety-related structures will be adversely affected by any flood event. Consequently, no flood protection measures are required for VEGP Units 3 and 4. Additionally, no technical specifications or emergency procedures to implement flood protection activities are required.

Furthermore, the design of VEGP Units 3 and 4 drainage facilities incorporates measures to ensure that 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 portion of the site during construction and operation of Units 3 and 4 is directed away from the drainage facilities of Units 1 and 2. Hence, drainage from the VEGP Units 3 and 4 site area do not affect the safety-related structures, systems, and components of VEGP Units 1 and 2.

As discussed in Subsection 3.4.1.1, the roofs of all safety-related structures are designed to prevent flooding of, or leakage into, safety-related structures, systems, and components as a result of the 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 Subsection 2.3.1.3.4.

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 are Westinghouse AP1000 reactors that do not require a conventional ultimate heat sink to provide safety-related cooling during emergency shutdown. Consequently, river water is not necessary to achieve safe shutdown of the units. The only use of water from the Savannah River for the reactor units is for the circulating water system/turbine plant cooling water system makeup, where river water is 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 (Reference 249). The contingency plan was developed in 1989 during one of the most severe droughts in the region in recent history. The objectives (Reference 249) 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 2.4-234. 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 (Reference 223). 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 Subsection 2.4.11.1.4. Details of gage locations and data availability are shown in Table 2.4-235.

Annual minimum daily-mean stream flow data from the three gages are shown in Figure 2.4-229 and Table 2.4-236. 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 2.4-230 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 2.4-236). 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 (Reference 249) and 1998–2003 (Reference 250; Reference 244).

American National Standard ANSI/ANS-2.13-1979, *Evaluation of Surface-Water Supplies for Nuclear Power Sites* (Reference 245), 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 ^{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 2.4-237. 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 2.4-231. 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 2.4-232 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 ^{2} goodness-of-fit test (Table 2.4-238).

Table 2.4-238 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 2.4-238. 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 2.4-238.

The corresponding low flow for a 100-year return period at Jackson (3,746 cfs) is also presented in Table 2.4-238 to facilitate a comparison. Figure 2.4-233 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 2.4-234.

2.4.11.1.3 Probable Minimum Flow

Because the river water is not 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 2.4-238. 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 2.4-235.

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 (Reference 251). Details of these flow measurements and corresponding river stages are shown in Table 2.4-239. 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 2.4-239), 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 2.4-235. 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 2.4-235. 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 2.4-236.

Using the stage-discharge relationship developed in Figure 2.4-235, 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 Subsection 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 2.4-236 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 Subsection 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 (Reference 246). 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 (Reference 248). 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 2.4-240.

2.4.11.5 Plant Requirements

VEGP Units 3 and 4 are 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 is for the circulating water system/turbine plant cooling water system makeup, where river water is 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 is 82.9 cfs (37,212 gpm). The maximum water requirement for plant operation is 136.2 cfs (61,145 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 is not necessary to achieve safe shutdown of the units.

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 2.5-204). The proposed AP1000 units referred to as VEGP Units 3 and 4 have a finished grade level elevation of 220 ft msl. The bottom of the foundation slab for the safety related AP1000 containment structure is 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 provide make-up water for the service water system (SWS). The wells 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 is 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 (Reference 292).

In constructing the new units, the site was excavated approximately 80 to 90 ft below existing grade to remove the in situ soil down to the principal bearing strata, the Blue Bluff Marl. The in situ soil was replaced with Seismic Category 1 and 2 fill material as described in Subsection 2.5.4. Foundations for the new units are poured on this new backfill material and the fill material is placed around the structures and continues 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)
- Data Report of Geotechnical Investigation and Laboratory Testing MACTEC Engineering and Consulting Inc., November 2007. (Appendix 2.5C)
- 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. (Reference 282)
- 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. (Reference 276)
- 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. (Reference 289)
- 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. (Reference 262)
- 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. (Reference 263)

- 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. (Reference 261)

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 2.5-204 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 (Reference 282). 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. (Reference 282)

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. (Reference 282)

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 (Reference 282).

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 (Reference 282).

Coastal Plain sediments comprise three aquifer systems consisting of seven aquifers that are separated hydraulically by confining units. As presented by (Reference 262), 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 (Reference 262) for geologic and hydrogeologic units differs from the (Reference 276) nomenclature used in Subsection 2.4.12.1.2 to describe the local hydrogeologic units. In this document, 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 2.4-237 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. (Reference 262)

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 (Reference 278) 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. (Reference 262)

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 (Reference 262) and numerical simulation (Reference 261). 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 Subsection 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 ft^3/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.

A two-dimensional, site specific, single layer numerical groundwater model has been developed to predict the effects of VEGP Units 3 and 4 construction on the Water Table aquifer flow regime (Appendix 2.4B). Aquifer recharge was varied across the model domain to account for variations and post-construction changes in surficial geology, vegetative cover, and local land use patterns. Net recharge and hydraulic conductivity values are varied across the model domain based on observed hydrogeologic conditions in order to calibrate the model to observed Water Table aquifer groundwater levels. The results of this modeling yield a recharge rate ranging from 0.0 to 10.0 inches per year depending on surficial conditions (Appendix 2.4B, Table 8). These values are in general agreement with the recharge rates of Clarke and West (1997).

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 (Reference 263; Reference 261). 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. Subsection 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. Figure 2.5-245 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 (Figure 2.5-242). 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 (Subsection 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 provide information on the VEGP site from the Triassic Basin rock to the ground surface. The geotechnical logs are provided in Appendices 2.5A and 2.5C and further discussed in Subsection 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 (Reference 282) and was not encountered in the investigations performed.

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 2.4-237 illustrates the hydrostratigraphic column for the VEGP site and surrounding area, identifying geologic units, confining units, and aquifers. Figures 2.4-238 and 2.4-239 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 (Reference 276). The elevations, thicknesses, and descriptions of these geologic formations, as determined from VEGP geotechnical boring B-1003 (Appendix 2.5A), 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 (Appendix 2.5A), 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 54 ft, with the top of the formation being approximately El. 50 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 fossiliferous limestone layer not encountered in borings at the VEGP Units 3 and 4 site. The formation has a thickness of approximately 63 ft, and the top of the formation is at approximately El. 130 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. VEGP geotechnical and hydrogeological boring data have been incorporated into the Blue Bluff Marl dataset. These data indicate that the base of the Blue Bluff Marl ranges in elevation between 21 ft msl and 83 ft msl. Where fully penetrated, the marl thickness ranges from a minimum of 5.0 feet where it has been scoured by the Savannah River to a maximum of approximately 95 feet. Where the marl is fully intact, its mean thickness is approximately 63 feet. Blue Bluff Marl structure contour and isopach maps have been prepared to include this new data. These are included as Figures 2.5-250 and 2.5-254, respectively.

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 geotechnical and hydrogeological borings and are described below (Appendices 2.4A, 2.5A, and 2.5C).

- 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 many 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, 2.5A, and 2.5C. 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 2.4-253.

  The data presented in Table 2.4-253 indicate that 27 of 189 borings were terminated above the elevation where the Utley limestone would be expected to be encountered. An additional 10 borings were advanced at locations where the ground surface is below the elevation where the Utley limestone would be expected to be encountered. Of the remaining 152 soil borings, the Utley Limestone is absent in 54 borings, or 36 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 and south towards the VEGP Units 3 and 4 cooling towers. Where present, the base of the Utley Limestone ranges in elevation from approximately 96 ft msl to 152 ft msl. The Utley Limestone isopach map presented in Figure 2.5-255 indicates that the limestone is a linear feature in its areal extent with the axis of maximum thickness roughly extending north-northeast from the VEGP Units 3 and 4 cooling towers to a location approximately 1200 feet east of Mallard Pond. The limestone is absent along the flanks of this feature and increases in thickness to a maximum of approximately

25 ft to 38 ft along its axis. Total thickness varies considerably, and the Utley Limestone is absent in some places within its general area of extent.

- Overlying the Utley limestone are undifferentiated sands, clays, and silts of the Barnwell Group. The thickness of this group is variable and ranges from approximately 26 to 162 ft in borings where the undifferentiated sediments of the Barnwell Group are fully penetrated. The top of the group extends to the ground surface and ranges from approximately El. 164 ft msl to 280 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 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 2.4-254.

Groundwater level elevations in OW-1001 measured between the period June 2005 and July 2007 (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. A 26-month data set representing June 2005 through July 2007 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 2.4-240. Table 2.4-241 lists the observation wells currently being used to monitor the Water Table aquifer, while Table 2.4-242 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 2.4-241 and 2.4-242, Figures 2.4-241 through 2.4-245, Figures 2.4-247 through 2.4-251, Figures 2.4-254 through 2.4-259, and Figures 2.4-261 through 2.4-264.

Water Table Aquifer

Groundwater level data for the Water Table aquifer available for the 1979 through 2007 period are provided in Figure 2.4-254. Table 2.4-255 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 Millen 4N climate stations. Precipitation data were obtained from the South Carolina Department of Natural Resources website (Reference 286). In addition, the Palmer Drought Severity Index (PDSI) and Palmer Hydrological Drought Index (PHDI) are plotted on Figure 2.4-255 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 (Reference 283). 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 2.4-254 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 2.4-256 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 July 2007 for the Water Table aquifer are summarized in Table 2.4-241 and shown in Figure 2.4-256. During the 26-month monitoring period, groundwater elevations ranged from about 132 to 165.5 ft msl with seasonal fluctuations averaging about 1.7 feet. 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 2.4-241 were used to develop groundwater surface elevation contour maps for the Water Table aquifer on a quarterly basis. These maps are presented in Figures 2.4-241 through 2.4-245 for June 2005 through June 2006, Figure 2.4-257 for November 2006, and Figures 2.4-261 to 2.4-262 for March and June 2007. Note that a contour map for November 2006 was not 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. 165.5 ft msl in the vicinity of well OW-1013 to a low of approximately El. 132 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 observed horizontal hydraulic gradient across the site for the Water Table aquifer is relatively consistent between the seven 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 2.4-246.

Recent groundwater elevation data from June 2005 to July 2007 for the Tertiary aquifer are summarized in Table 2.4-242 and shown in Figure 2.4-258. Groundwater elevations for this 26-month monitoring period range from about 81 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. A decline in piezometric head is observed from February 2006 through November 2006 followed by rising levels through February 2007. Groundwater elevations decreased from March 2007 through July 2007, reaching the lowest levels seen during the 26-month observation period. 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 Savannah River.

The groundwater elevation data summarized in Table 2.4-242 were used to develop piezometric surface maps for the Tertiary aquifer. The Tertiary aquifer piezometric surface is presented in Figures 2.4-247 through 2.4-251 for June 2005 through June 2006, Figure 2.4-259 for November 2006, and Figures 2.4-263 to 2.4-264 for March and June 2007. 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 by an average of approximately 8.7 ft across the VEGP site during the 26-month observation period.

The horizontal hydraulic gradient across the site for the Tertiary aquifer is relatively consistent among the seven figures and is approximately 0.005 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 63 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) (Reference 285). 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 to determine in situ hydraulic conductivity values for the Water Table and Tertiary aquifers. Table 2.4-243 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 2.4-244 through 2.4-246. 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 member of the Barnwell Group 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.

Hydraulic conductivities were reported from the site characterization investigations for VEGP Units 3 and 4. 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 2.4-243). A two-dimensional, site specific, single layer numerical groundwater model has been developed to predict the effects of VEGP Units 3 and 4 construction on the Water Table aquifer flow regime (Appendix 2.4B). Hydraulic conductivity was varied across the model domain to account for lateral variations in surficial geology and locations of construction fill materials. Horizontal hydraulic conductivity values were varied across the model domain based on observed hydrogeologic conditions in order to calibrate the model to recently observed Water Table aquifer groundwater levels. The results of this modeling yield horizontal hydraulic conductivity values ranging from 8.0 to 32 ft/day for the areas outside of the immediate VEGP Units 3 and 4 Power Block area. These values lie within the range of Barnwell Group hydraulic conductivity values cited above and are considered representative of the horizontal hydraulic conductivity for the Water Table aquifer.

VEGP Units 1 and 2 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. Table 2.4-244 summarizes the laboratory test results for geotechnical samples collected below the capillary fringe in the Barnwell Formation, which were at depths ranging from El. 108 to 160 ft msl. Sand and clay make up the majority of samples. Measured moisture contents, by weight percent, range from 19.7 to 47.0 percent and have a median value of 27.6 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 27.6 percent and a value of 2.66 for the specific gravity, the void ratio is estimated to be about 0.73. A total porosity of 42 percent is calculated from this void ratio (Reference 264), and an effective porosity of about 34 percent is estimated based on 80 percent of the total porosity (Reference 280). 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 (Reference 281). The effective porosity of the backfill is assumed to be 0.34 as established during site characterization investigations for VEGP Units 1 and 2.

Post-Construction Groundwater Model

A two-dimensional single layer groundwater model was developed to simulate post-construction groundwater flow in the Water Table aquifer at the VEGP site (Appendix 2.4B). A conceptual representation of the groundwater model developed for the VEGP site is shown on Appendix 2.4B, Figure 18. Appendix 2.4B, Figure 19 shows the numerical representation of the groundwater model including the horizontal grid formulation. The grid spacing surrounding the existing (Units 1 and 2) and proposed (Units 3 and 4) plant areas is set at 100 ft by 100 ft, whereas for the remaining area, the grid spacing is set at 200 ft by 200 ft.

Topographic and surficial geology maps were used to delineate the vertical extent of the Water Table aquifer in this single layer groundwater model. The top elevation of the groundwater model is the ground surface elevation. The ground surface elevation data were obtained from USGS 1:24,000 quadrangle maps. The base elevation of the groundwater model (i.e. top of Blue Bluff Marl) is defined by geotechnical and hydrogeological boring data. The base layer of the model is the low permeability Blue Bluff Marl which hydraulically isolates the Water Table aquifer from the underlying Tertiary aquifer (Appendix 2.4B, Figure 5). The model domain covers approximately six square miles.

The boundaries of the model domain were selected to coincide with key physical features of the model area that are shown in Appendix 2.4B, Figure 18. These key physical features are numerically represented as drain boundaries and no flow boundaries.

Hydraulic conductivity and net recharge values were allowed to vary in order to calibrate the model to the observed groundwater levels (March 2006). The model calibration suggested that a much higher hydraulic conductivity value should be used for the area surrounding Mallard Pond (100 ft/day) in order to match the observed water table elevations in the area near Mallard Pond. Hydraulic conductivities for the Barnwell Group in the primary model domain area were varied to account for the presence/absence of the more permeable Utley Limestone member. During construction, fill material will be used around and beneath the power block and auxiliary buildings. It is necessary to account for this fill material in the post-construction model. The hydraulic conductivity of the fill material for the construction of Units 3 & 4 is assumed to be the same as that of the fill material used for Units 1 & 2. As discussed in Appendix 2.4B, Section 2.7, the geometric mean of four slug tests conducted in the structural fill material for Units 1 & 2 was 2.3 ft/day. The hydraulic conductivity values from these tests ranged from1.3 to 3.3 ft/day. As a conservative assumption it is assumed that the hydraulic conductivity of the fill material is equal to the maximum measured value, i.e. 3.3 ft/day (Appendix 2.4B, Table 4). The aquifer recharge rate was varied across the model domain to account for variations in surficial geology, vegetative cover, and local land use patterns. For paved areas, the net recharge rate in the model was set equal to zero.

By executing a series of seven model runs with different combinations of hydraulic conductivity and recharge values, the best performing model run was identified as Model 7. These seven model runs represent alternative conceptual models, i.e. different sets of assumptions, for the site. The key input parameters used for these model runs are described in Appendix 2.4B, Section 4.4 and summarized in Appendix 2.4B, Table 8. Simulated post-construction groundwater levels generated using Model 7 are presented in Appendix 2.4B, Figure 74. The post-construction groundwater levels at observation well OW-1003 (Unit 3 location) are approximately 0.5 feet higher than the calibrated pre-construction groundwater levels.

Post-construction release points, groundwater pathways and discharge points were evaluated using particle tracking for the selected model run (Model 7). In each case, particles are released from the perimeter of the 775-ft radius circle defining the area surrounding the nuclear island auxiliary buildings of Units 3 & 4 and tracked to their potential discharge points. As seen from Appendix 2.4B, Figure 77, the potential particle tracking path line from the various discharge points is always directed towards Mallard Pond. This implies that all releases from any point inside the 1200-ft diameter circle around the power block area will also discharge to Mallard Pond. The particle tracking path line from the specific release point in the Unit 4 Auxiliary Building is shown in Appendix 2.4B, Figure 78.

Groundwater Travel Time

The groundwater travel time has been estimated by considering the locations of the effluent holdup tanks (the initial release location), observed hydraulic conductivities of the backfill, and estimates of the hydraulic gradients, and hydraulic conductivities, and travel path lengths through the native materials comprising the Water Table aquifer based on the results of the post-construction groundwater modeling. The total saturated zone travel time is the sum of three components: (1) travel time in the backfill; (2) travel time in the Water Table aquifer in the area between the backfill and the area near OW-1005; and (3) travel time between OW-1005 and Mallard Pond. The travel time in each is a function of the travel distance, hydraulic conductivity, effective porosity, and hydraulic gradient. The basis for estimating the travel time in each of these three segments is described below.

The travel distance in the backfill represents the curvilinear distance along the predicted particle flow track between the release point in the northwestern portion of the Unit 4 auxiliary building potentially flooded by a tank rupture and the southwestern extent of the power block excavation at an elevation of approximately 158 ft msl, where groundwater would flow from backfill to native material (Appendix 2.4B, Figure 78). As indicated previously, a hydraulic conductivity of 3.3 ft/day was assigned to the backfill. The effective porosity of the backfill was taken to be 0.34 as established during site characterization investigations for VEGP Units 1 and 2. Because the backfill for Units 3 and 4 will be obtained from the borrow areas used for Units 1 and 2 and compacted to the same criteria, the hydraulic conductivity and porosity values observed for Units 1 and 2 should be representative of Units 3 and 4. Based on the aforementioned parameters, the groundwater travel time in the backfill was calculated to be 2.4 years (Appendix 2.4B, Table 15) (Reference 275).

The travel distance through the native material between the power block area and well OW-1005 lies along the predicted particle flow track between the location on southwestern side of the power block excavation where groundwater flow enters native material and the area near observation well OW-1005 where higher permeability alluvial material is modeled to be encountered (Appendix 2.4B, Figure 78). A hydraulic conductivity of 32 ft/day, based on the groundwater model calibration results, is used in this analysis. The effective porosity of the Water Table aquifer has been estimated to be

0.34 based on site-specific investigation measurements. Using the parameters described above, a groundwater travel time of 3.2 years is estimated for this segment (Appendix 2.4B, Table 15).

The predicted groundwater travel time along the particle flow track between the modeled boundary of the alluvial materials near observation well OW-1005 and Mallard Pond (Appendix 2.4B, Figure 78) is approximately 1.1 years (Appendix 2.4B, Table 15). A hydraulic conductivity of 100 ft/day, based on the groundwater model calibration results, and the same effective porosity cited above for the native materials (0.34) is used in this analysis.

Summing the above travel times, the total travel time for this analysis is 6.7 years (Appendix 2.4B, Table 15).

The geotechnical boring logs contained in Appendix 2.4A, 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 Subsection 2.5.4.5, construction of the new VEGP Units 3 and 4 required a substantial amount of excavation and backfill. The excavation was 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 ranged from approximately 80 to 90 ft below existing grade. Backfilling was 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. Filling continued up around these structures to final grade. The fill primarily consisted 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 was 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 was compacted to a minimum of 97 percent of the maximum dry density.

Excavating existing soils and replacing these soils with structural fill altered the hydrogeologic characteristics of the subsurface materials within the footprint of VEGP Units 3 and 4. 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 lower than that of the in situ soils.

Development of VEGP Units 3 and 4 also increased 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) are used to convey runoff from precipitation offsite. The increased impervious area and use of storm-water management facilities 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 entails the placement of relatively large and impermeable structures below grade. The base elevations of the major structures (containment and auxiliary buildings) are at about El. 186.5 ft msl. This elevation is at least 25 to 35 ft above the water table. Because these structures do not extend below the water table, they do 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 2.4-245 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 2.4-245), 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 2.4-243). 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 geotechnical samples collected in the Tertiary aquifer are presented in Table 2.4-246. Sample elevations range from El. -273 ft msl to 75 ft msl, with the samples consisting mainly of sand and fine particles, with some gravel. Moisture content ranges from 16.5 to 40.7 percent, with specific gravity values varying from 2.62 to 2.69. Using the median moisture content of 23.6 percent and a value of 2.67 for the specific gravity, the void ratio of the Tertiary aquifer is estimated to be about 0.63. A total porosity of 38.7 percent is calculated from this void ratio (Reference 264), and an effective porosity of about 30.9 percent is estimated based on 80 percent of the total porosity (Reference 280). 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 (81.01 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 30.9 percent (Reference 275). Using a distance of 5,600 ft from the 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 1142 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 (Reference 268). 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 2.4-247 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 2.4-248 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 2.4-249 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 2.4-252. 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 (Reference 267). Projections for

Burke County total water use in 2050 are provided in the Comprehensive Water Supply Management Plan for Burke County and its Municipalities (Reference 284). 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 (Reference 284).

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 2.4-250 lists these wells, while Figure 2.4-253 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 (Reference 287). 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 2.4-251.

Table 2.4-252 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 2.4-260. SNC's groundwater use permit (Reference 287) 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 VEGP Units 3 and 4 COL. Current groundwater monitoring programs for the existing units are addressed in VEGP Procedure Number 30140-C, Revision 22 (Reference 290). The results of these programs are reported semiannually.

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

The existing SNC groundwater monitoring programs are evaluated with respect to placement of the new units in Subsection 2.4.12.3.1 below.

2.4.12.3.1 Long Term Groundwater Level Monitoring

Subsection 2.4.12.3 indicates that the existing groundwater monitoring programs would be evaluated to determine if any additional monitoring of existing observation wells or construction of new observation wells would be required to adequately monitor the impact on groundwater. The results of the evaluation indicate that the long term collection of Units 3 and 4 water table level data is appropriate to confirm the direction of groundwater flow in the vicinity of the power blocks of Units 3 and 4.

Groundwater level data will be collected from a network of observation wells similar to that utilized for the ESP phase (June 2005 through July 2007) to ensure the data is comparable to that of the ESP phase. Most of the active Units 1 and 2 observation wells that were included in the ESPA data will not be impacted by the earth moving activities for Units 3 and 4. However, most of the remainder of the ESPA observation wells will be impacted by these earthwork activities. The number and location of the replacement wells to be installed for long term monitoring will be determined after the earthwork activities are complete and heavy construction is well underway. Some of these observation wells will be installed in the Units 3 and 4 power block areas.

Groundwater level monitoring will be initiated prior to commercial operation of Unit 3 and revised as needed based upon the review and evaluation of the observed data.

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 Subsection 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 2.4-241 through 2.4-245, Figure 2.4-257, and Figures 2.4-261 and 2.4-262.

A two-dimensional, site specific, single layer numerical groundwater model was developed to predict the effects of VEGP Units 3 and 4 construction on the Water Table aquifer flow regime. To predict post-construction groundwater flow conditions, the model accounts for the different hydraulic conductivity value of the fill material associated with the excavated areas for Units 3 & 4, as well as changes in groundwater recharge due to building and parking lot construction, regrading, and assumed changes in vegetative cover patterns. The results of this model indicate that the post-construction Water Table aquifer elevation in the power block area at OW-1003 is approximately 0.5 feet higher than calibrated pre-construction levels, or approximately 157.9 ft msl. This elevation is approximately 22 ft below the bottom elevation of the containment building slab. 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.

2.4.13 Accidental Releases of Liquid Effluents in Ground and Surface Waters

2.4.13.1 Groundwater

This section provides a conservative analysis of a postulated, accidental liquid release of effluents to the groundwater at the VEGP site. The accident scenario is described. The conceptual model used to evaluate radionuclide transport is presented, along with potential pathways of contamination to water users. The radionuclide transport analysis is described, and the results are summarized. The radionuclide concentrations to which a water user might be exposed are compared against the regulatory limits.

Results are considered acceptable if the concentrations are less than the effluent concentration limits (ECLs) included in 10 CFR Part 20, Appendix B, Table 2, Column 2. Because the identity and concentration of each radionuclide in the mixture are known, the ratio present in the mixture and the concentration otherwise established in 10 CFR Part 20, Appendix B, for the specific radionuclide not in a mixture must also be determined. The sum of such ratios for all of the radionuclides in the mixture may not exceed "1" (i.e., "unity"). These criteria apply to the nearest potable water supply in an unrestricted area.

2.4.13.1.1 Accident Scenario

The accident scenario has been selected based on information developed by Westinghouse to assist AP1000 COL applicants in evaluating the accidental liquid release of effluents (Reference 301). The accident scenario assumes an instantaneous release from one of the two effluent holdup tanks located in the lowest level of the AP1000 auxiliary building.

There are two effluent holdup tanks, each with a capacity of 28,000 gal., for each AP1000 unit. These tanks have both the highest potential radionuclide concentrations and the largest volume. Therefore, they have been selected by Westinghouse as the limiting tanks for evaluating an accidental release of liquid effluents that could lead to the most adverse contamination of groundwater or surface water, via the groundwater pathway.

Westinghouse estimated the radionuclide concentrations of the effluent holdup tanks to be 101 percent of the reactor coolant. Westinghouse determined the radionuclide concentrations in reactor coolant itself to be as follows:

- For tritium (H-3), a coolant concentration of 1.0 μCi/g should be used.
- Corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) should be taken directly from the AP1000 DCD, Table 11.1-2, *Design Basis Reactor Coolant Activity*.
- Other radionuclides should be based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25 to adjust the failed fuel rate from the design basis to a conservatively bounding value for this analysis.

Based on these recommendations, the expected radionuclide concentrations in the effluent holdup tanks have been calculated, and the results are summarized in Table 2.4-257.

2.4.13.1.2 Conceptual Model

Figures 2.4-265 and 2.4-266 illustrate the conceptual models used to evaluate an accidental liquid release of effluent to groundwater, or to surface water via the groundwater pathway. The key elements and assumptions embodied in the conceptual model are described and discussed below.

2.4.13.1.2.1 Water Table Aquifer

As indicated in Subsection 2.4.13.1.1, the effluent holdup tanks are assumed to be the source of the release, with each tank having a volume of 28,000 gal. and the radionuclide concentrations as summarized in Table 2.4-257. These tanks are located at the lowest level of the auxiliary building, which has a floor elevation of approximately 186.5 ft msl and is approximately 25 to 35 ft above the water table, based on water table contour plots presented on Figures 2.4-241 through 2.4-245, Figure 2.4-257, and Figures 2.4-261 to 2.4-262. One of these tanks is postulated to rupture, and 80 percent of the liquid volume (22,400 gal.) is assumed to be released in accordance with Branch Technical Position 11-6 of NUREG-0800. Flow from a tank rupture would initially flood the tank room and begin to flow to the auxiliary building radiologically controlled area sump via floor drains as described in Subsection 3.4.1.2.2.2 of the AP1000 DCD. It is assumed that sump pumps are inoperable. According to the AP1000 DCD, this would result in the 22,400 gal. release flooding the balance of level 1 of the auxiliary building via the interconnecting floor drains. Once level 1 is flooded, it is assumed that a pathway is created that would allow the entire 22,400 gal. to enter the groundwater (Water Table aquifer) instantaneously. This assumption is very conservative because it requires failure of the floor drain system, plus it ignores the barriers presented by the 6-ft-thick basemat and the sealed, 3-ft-thick exterior walls of the AP1000 auxiliary building. Furthermore, there is a minimum of 20 ft of unsaturated zone beneath the basemat. Radionuclide concentrations would be attenuated during unsaturated zone transport as a consequence of adsorption, dispersion, and radioactive decay, which is not considered in this conservative analysis.

With the postulated instantaneous release of the contents of an effluent holdup tank to groundwater, radionuclides would enter the Water Table aquifer and migrate with the groundwater in the direction of decreasing hydraulic head. Hydraulic head contour maps for the Water Table aquifer presented in Figures 2.4-241 through 2.4-245, Figure 2.4-257, and Figures 2.4-261 to 2.4-262 indicate that the pre-construction groundwater pathway from a point of release in either of the AP1000 auxiliary buildings would be northward to Mallard Pond, a groundwater discharge area, as discussed in Subsection 2.4.12.1.3. Because the underlying Blue Bluff Marl has a very low vertical permeability, as is described in Subsection 2.4.12, groundwater flow in the Water Table aquifer is predominantly horizontal. Since VEGP Unit 4 is closer to Mallard Pond, it is selected as the release location for this evaluation. Post-construction groundwater modeling described in Appendix 2.4B was conducted to reflect the hydrologic alterations associated with the construction of VEGP Units 3 and 4. Modeling results indicate that the groundwater pathway will still be northward toward Mallard Pond after construction of the new units. Particle tracking results show the flow path to be curvilinear between the VEGP Unit 4 auxiliary building and the south side of Mallard Pond. During saturated zone transport, radionuclide concentrations of the liquid released to the water table would be reduced by the processes of adsorption, hydrodynamic dispersion, and radioactive decay. There are no existing water-supply wells between the postulated release points and Mallard Pond that withdraw water from the Water Table aquifer. Based on the data in Table 2.4-250, all water-supply wells for the existing VEGP plant withdraw their water from the deeper, confined Tertiary and Cretaceous aquifers. Figure 2.4-237 illustrates the conceptual model for evaluating radionuclide transport in the Water Table aquifer.

Mallard Pond serves as a groundwater discharge area for the Water Table aquifer. The radionuclides associated with a liquid release would enter the surface water system via Mallard Pond. Radionuclide concentrations would be diluted in the pond and in the stream running from the pond to the Savannah River. Groundwater flow into Mallard Pond is continuous, and the pond level is controlled by a spillway. Measurements of stream flow discharge from Mallard Pond and at points downstream indicate that flow increases progressively in magnitude before discharging to the Savannah River (Reference 294). Upon discharge to the Savannah River, the stream flow would mix with the Savannah River flow, resulting in significantly further dilution prior to withdrawal by the nearest surface water user. As noted in Subsection 2.4.1, the nearest downstream industrial surface water users include the Fort James Operating Company and the Georgia Power Company. Both companies operate river intakes that withdraw water from the Savannah River near River Mile 45, which is about 106 miles downstream of the VEGP site. The City of Savannah Municipal and Industrial Plant, and the Beaufort-Jasper County Water and Sewer Authority are the nearest downstream municipal water users. The City of Savannah obtains water from Abercorn Creek where it enters the Savannah River near River Mile 29, which is about 122 miles downstream from the VEGP site. Beaufort-Jasper County withdraws water from the Savannah River via an 18-mile canal.

2.4.13.1.2.2 Tertiary Aquifer

An alternative, less likely, conceptual model is also considered in this analysis. This model considers groundwater flow in the deeper Tertiary aquifer with eventual direct discharge to the Savannah River (Figure 2.4-266). Based on Table 2.4-250 and Figure 2.4-253, there are no existing VEGP plant Tertiary aquifer potable water-supply wells located downgradient of the postulated accidental release or any potable wells potentially impacted by such a release. The release mechanism is the same as that described in Subsection 2.4.13.1.2.1 for the Water Table aquifer with the exception that the accidental release is assumed to enter the Tertiary aquifer instantaneously. This conceptual model is conservative because the low permeability Blue Bluff Marl hydraulically isolates the overlying Water Table aquifer from the underlying Tertiary aquifer. The flow path is assumed to be a straight line between the center of the power block area for the new AP1000 units downgradient to the closest point of the Savannah River, a distance of approximately 5600 ft. Upon discharge to the Savannah River, contaminated groundwater would mix with the Savannah River flow, resulting in significant dilution prior to withdrawal by the nearest surface water user described in Subsection 2.4.13.1.2.1. Figure 2.4-266 illustrates the conceptual model for evaluating radionuclide transport in the Tertiary aquifer.

2.4.13.1.3 Radionuclide Transport Analysis

A radionuclide transport analysis has been conducted to estimate the radionuclide concentrations that might expose existing and future water users based on an instantaneous release of the radioactive liquid of an AP1000 effluent holdup tank. Analysis of liquid effluent release commenced with the simplest of models, using demonstratively conservative assumptions and coefficients. Radionuclide concentrations resulting from the preliminary analysis were then compared against the ECLs identified in 10 CFR Part 20, Appendix B, Table 2, Column 2, to determine acceptability. Further analysis, using progressively more realistic and less conservative assumptions and modeling techniques, was conducted when the preliminary results do not meet 10 CFR Part 20 requirements.

Radionuclide transport along a groundwater path line is governed by the advection-dispersion-reaction equation (Reference 295), which is given as

$$R\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} - v\frac{\partial C}{\partial x} - \lambda RC \qquad \text{(Equation 2.4.13-1)}$$

where: C = radionuclide concentration; R = retardation factor; D = coefficient of longitudinal hydrodynamic dispersion; v = average linear velocity; and λ = radioactive decay constant. The retardation factor is defined from the relationship

$$R = 1 + \frac{\rho_b K_d}{n_e} \qquad \text{(Equation 2.4.13-2)}$$

where: ρ_b = bulk density; K_d = distribution coefficient; and n_e = effective porosity. The average linear velocity is determined using Darcy's law, which is

$$v = -\frac{K}{n_e}\frac{dh}{dx} \qquad \text{(Equation 2.4.13-3)}$$

where: K = hydraulic conductivity; and dh/dx = hydraulic gradient. The radioactive decay constant can be written as

$$\lambda = \frac{\ln 2}{t_{1/2}} \qquad \text{(Equation 2.4.13-4)}$$

where $t_{1/2}$ = radionuclide half-life. Conservatively neglecting hydrodynamic dispersion, Equation 2.4.13-1 can be integrated to yield

$$C = C_0 \exp(-\lambda t) \qquad \text{(Equation 2.4.13-5)}$$

where: C = radionuclide concentration; C_0 = initial radionuclide concentration; $t = LR/v$ = radionuclide travel time; and L = groundwater path line length.

To estimate the radionuclide concentrations in the Water Table aquifer, groundwater discharging to Mallard Pond, Equation 2.4.13-5 was applied along the groundwater path line that would originate at the liquid effluent release points beneath the AP1000 auxiliary building at VEGP Unit 4 and terminate at Mallard Pond. For the Tertiary aquifer, Equation 2.4.13-5 was similarly applied along the groundwater path line from the center of the power block area for the new AP1000 units downgradient to the discharge point in the Savannah River. These analyses were performed sequentially as described below.

2.4.13.1.3.1 Water Table Aquifer

Transport Considering Radioactive Decay Only

An initial screening analysis was performed considering radioactive decay only. This analysis assumed that all radionuclides migrate at the same rate as groundwater and considered no adsorption and retardation, which would otherwise result in a longer travel time and more radioactive decay. The concentrations of the radionuclides appearing in Table 2.4-257 were decayed for a period equal to the groundwater travel time from the point of release to Mallard Pond, using Equation 2.4.13-5 with R = 1. Radionuclides having concentrations less than 1 percent of their respective ECLs were eliminated from consideration because their concentrations would be well below their regulatory limits. Any radionuclides having a concentration greater than or equal to 1 percent of their ECL were retained for further evaluation. Evaluating transport considering radioactive decay only requires an estimate of the groundwater travel time.

The groundwater travel time from the VEGP Unit 4 Auxiliary Building to Mallard Pond has been estimated using a two-dimensional groundwater flow model (Appendix 2.4B) considering the locations of the effluent holdup tanks, and modeled estimates of hydraulic gradients and hydraulic conductivities of the Water Table aquifer and construction backfill material. The total saturated zone travel time was determined to be 6.7 years. The travel times in the various hydrogeologic units encountered along the groundwater pathway are as follows: 2.4 years in the backfill; 3.2 years in the Water Table aquifer in the area between the backfill and a point near OW-1005 where more permeable sediments are present; and 1.1 years in the more permeable sediments present between OW-1005 and Mallard Pond.

Using Equation 2.4.13-5, the initial concentrations given in Table 2.4-257 were decayed for a period of 6.7 years. Table 2.4-258 summarizes the results considering only radioactive decay and identifies those radionuclides that would exceed 1 percent of their ECL. These include H-3, Mn-54, Fe-55, Co-60, Sr-90, Ag-110m, I-129, Cs-134, Cs-137, and Ce-144.

Transport Considering Radioactive Decay and Adsorption

The H-3, Mn-54, Fe-55, Co-60, Sr-90, Ag-110m, I-129, Cs-134, Cs-137, and Ce-144 retained from the radioactive decay screening analysis were further evaluated considering adsorption and retardation in addition to radioactive decay. Distribution coefficient values for Co, Sr, and Cs were determined based on laboratory analyses of soil samples obtained from the VEGP site (Reference 296; Reference 298) and are shown in Table 2.4-259. Sixteen soil samples were taken from shallow test pits located in potential borrow source areas for backfill that will be required for the new AP1000 units. Laboratory testing of these backfill samples yielded distribution coefficients that range from 1.4 to 15.3 mL/g for Co, 6.0 to 51.7 mL/g for Sr, and 3.5 to 56.2 mL/g for Cs. Three additional soil samples were obtained from a vibratory boring located near B-1003. The samples acquired from the vibratory boring represent the Barnwell Group sediments based on the boring log for B-1003. Testing of the Barnwell Group sediment samples resulted in distribution coefficients that range from 3.9 to 21.3 mL/g for Co, 14.4 to 17.4 mL/g for Sr, and 22.7 to 33.2 mL/g for Cs.

Distribution coefficients for Co, Sr, and Cs in the backfill were conservatively assigned the minimum value determined from the 16 samples (1.4 mL/g for Co, 6.0 mL/g for Sr, and 3.5 mL/g for Cs). Distribution coefficients for Co, Sr, and Cs in the Barnwell Group sediments were conservatively assigned the minimum value observed for the three vibratory boring samples (3.9 mL/g for Co, 14.4 mL/g for Sr, and 22.7 mL/g for Cs). Distribution coefficients for H-3 and I-129, which have no or little tendency for adsorption, were taken to be zero for both the backfill and Barnwell Group sediments. Distribution coefficients for Mn-54, Fe-55, Ag-110m, and Ce-144 were conservatively assumed to be zero in both the backfill and the native Barnwell Group sediments. Distribution coefficients for the more permeable material near Mallard Pond were taken to be the same as those used for the native material in the Barnwell Group.

Retardation factors were calculated using Equation 2.4.13-2 with the distribution coefficients as stated above, effective porosities of 0.34 for the backfill, Barnwell Group sediments, and the permeable Mallard Pond materials, and a bulk density of 1.54 g/cm^3 for all materials. The bulk density was calculated using a total porosity value of 0.42 and a specific gravity of 2.66 as provided in Subsection 2.4.12.1.4. Total radionuclide travel times were calculated by summing the radionuclide travel times in the backfill, Barnwell Group, and permeable Mallard Pond materials described above. Radionuclide concentrations were then determined at the point of discharge to Mallard Pond using Equation 2.4.13-5 and the appropriate initial concentration, decay rate, and total travel time. Results are summarized in Table 2.4-260 and indicate that H-3, Mn-54, Fe-55, Sr-90, Ag-110m, I-129, Cs-137, and Ce-144 would exceed 1 percent of their respective ECL.

Transport Considering Radioactive Decay, Adsorption, and Dilution

The radionuclides retained after screening for the effects of radioactive decay and adsorption in groundwater would discharge to surface water (Mallard Pond) and mix with other, uncontaminated surface water. A dilution factor was estimated to account for the mixing and dilution of contaminated groundwater with uncontaminated surface waters. For the Water Table aquifer, the dilution factor is the ratio of the rate at which the postulated release would discharge to surface water (Mallard Pond) as contaminated groundwater to the total rate of groundwater discharge to Mallard Pond, which would include both uncontaminated and contaminated groundwater. The magnitude of the dilution factor was estimated as described below.

The rate at which a release from an effluent holdup tank discharges to surface water (Mallard Pond) is determined by the transport characteristics of the Water Table aquifer. A release from an effluent holdup tank would undergo unsaturated zone transport beneath the auxiliary building, followed by saturated zone transport first through the backfill and then through the Barnwell Group and more permeable Mallard Pond materials, and would finally discharge to Mallard Pond from the permeable Mallard Pond materials. The discharge rate itself is a function of the Darcy velocity, and the assumed volume and dimensions of the resulting contaminant slug. The mean Darcy velocity in the backfill was determined to be 0.043 ft/day based on a hydraulic conductivity of 3.3 ft/day and a hydraulic gradient of 0.013 ft/ft estimated from Appendix 2.4B, Figure 78. The volume of the liquid release has been assumed to be 22,400 gal. (2995 ft^3), which represents 80 percent of the 28,000 gal. capacity of one effluent holdup tank (NUREG-0800, Branch Technical Position 11-6 recommends that 80 percent of the liquid volume be considered in this analysis). Considering the effective porosity of the backfill (0.34), the release would occupy about 8810 ft^3 of the saturated backfill. The shape of the resulting contaminant slug is assumed to be square in plan view and extend vertically throughout the entire saturated thickness of the backfill. Using 20 ft as a representative saturated thickness (water table to top of Blue Bluff Marl), the slug would have an area of about 440 ft^2 in plan view and a width of about 21 ft. The cross-sectional area of the contaminant slug normal to the groundwater flow direction would therefore be 20 ft by 21 ft or about 420 ft^2. The discharge rate of the contaminant slug is then the product of the Darcy velocity (0.043 ft/day) and the cross-sectional area (420 ft^2) or 18 ft^3/day (0.094 gpm). The rate of total groundwater discharge to surface water has been estimated as 1125 gpm at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. This value is the result of stream flow measurements that were taken in the months of June and July to support the licensing of VEGP Units 1 and 2 (Reference 294). Because the stream discharging from Mallard Pond and its west branch are both perennial streams, the stream flow measurements would represent the groundwater discharge. The resulting dilution factor at this location is calculated as the ratio of 0.094 gpm to 1125 gpm, or 8.3E-5.

This dilution factor is applied to the H-3, Mn-54, Fe-55, Sr-90, Ag-110m, I-129, Cs-137, and Ce-144 concentrations reported in Table 2.4-260 to account for dilution in addition to radioactive decay and adsorption. Table 2.4-261 summarizes the resulting concentrations, which would represent the concentrations in the surface water at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. It is seen that the concentrations of each of these radionuclides are below their respective ECLs.

2.4.13.1.3.2 Tertiary Aquifer

Transport Considering Radioactive Decay Only

As indicated in Subsection 2.4.12.1.4, the horizontal hydraulic gradient of the Tertiary aquifer is approximately 0.005 ft/ft, based on the water levels observed at well OW-1008 and Well 27, and the distance between the two observation wells. Tertiary aquifer travel time was calculated using a hydraulic conductivity of 0.83 ft/day as reported in Subsection 2.4.12.1.4 and Table 2.4-243, a hydraulic gradient of 0.005 ft/ft, and an effective porosity of 0.309 based on the site-specific investigation measurements presented in Subsections 2.4.12.1.3 and 2.4.12.1.4. Using a distance of 5600 ft from the 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 1142 years. Using Equation 2.4.12-5, the initial concentrations given in Table 2.4-257 were decayed for a period of 1142 years. Table 2.4-262 summarizes the results and identifies those radionuclides that would exceed 1 percent of their ECL. The only radionuclide exceeding 1 percent of its ECL is I-129. As with the Water Table aquifer, the distribution coefficient of I-129 is taken to be zero because it has little to no tendency for adsorption. Therefore, no credit is taken for I-129 adsorption, and the I-129 concentration discharging to the Savannah River from the Tertiary aquifer remains unchanged from that calculated in the radioactive decay screening analysis. As seen in Table 2.4-262, the calculated concentration is well below the ECL.

2.4.13.1.4 Compliance with 10 CFR Part 20

The radionuclide transport analysis presented in Subsection 2.4.13.1.3 demonstrates that all of the radionuclides that could be accidentally released to groundwater would be individually below their ECLs. However, 10 CFR Part 20, Appendix B, Table 2, imposes additional requirements when the identity and concentration of each radionuclide in a mixture are known. In this case, the ratio present in the mixture and the concentration otherwise established in 10 CFR Part 20 Appendix B for the specific radionuclide not in a mixture must be determined. The sum of such ratios for all of the radionuclides in the mixture may not exceed “1” (i.e., “unity”) as indicated by Note 4 in Appendix B, 10 CFR Part 20.

This sum of fractions approach was applied to the radionuclide concentrations conservatively estimated in Subsection 2.4.13.1.3. Results are summarized in Table 2.4-263 for the Water Table aquifer and Table 2.4-264 for the Tertiary aquifer. The ratios for the mixture sum to 0.058 for the

Water Table aquifer, which demonstrates that an accidental liquid release of effluents in groundwater would not exceed 10 CFR Part 20 limits in the Mallard Pond stream before reaching the VEGP site property (EAB). Compliance with 10 CFR Part 20 is further assured considering that the point at which compliance has been demonstrated is within the restricted area and not a potable water source. The stream discharging from Mallard Pond is a gaining stream that discharges to, and mixes with, the Savannah River. Nearly the entire reach of this stream, about 1.0 mi. in length, is within the restricted area and not a potable water supply. Downstream of the point where compliance has been demonstrated, the stream appears to enter the adjacent Hancock Landing property for a short distance before re-entering the VEGP site property and discharging into the Savannah River. The nearest potable water supply in an unrestricted area to which the 10 CFR Part 20 requirements would apply is the Savannah River. Mixing of the tributary stream flow with the Savannah River flow would dilute radionuclide concentrations further. The magnitude of this additional dilution can be estimated from the ratio of the tributary stream flow rate (1125 gpm) to the Savannah River flow rate. Using the 100-year drought flow, given as 3298 ft^3/sec (1,480,000 gpm) in Subsection 2.4.11, to conservatively represent the Savannah River flow rate, a dilution factor of 7.6E-04 is calculated. Accounting for this additional dilution would further reduce radionuclide concentrations by a factor of about 1,000. Consequently, the ratios for the mixture would sum to a value much less than unity and well below the compliance limit.

Considering radioactive decay only, the ratios for the mixture sum to 0.036 for the Tertiary aquifer prior to discharge in the Savannah River indicating compliance with 10 CFR Part 20 limits (Table 2.4-264). Mixing of the Tertiary aquifer discharge with the Savannah River flow would significantly dilute radionuclide concentrations further.

2.4.13.2 Surface Water

No outdoor tanks contain radioactivity in the Westinghouse AP1000 design (Reference 301). In particular, the AP1000 design does not require boron changes for load follow and does not recycle boric acid or reactor coolant water, so the boric acid tank is not radioactive. Because no outdoor tanks contain radioactivity, no accident scenario could result in the release of liquid effluent directly to the surface water.

2.4.14 Technical Specifications and Emergency Operation Requirements

The plant elevation (220 ft MSL) of the VEGP is above the design basis river flood elevation and the probable maximum precipitation flood elevation; therefore, due to design there are no requirements for emergency protective measures designed to minimize the impact of hydrology-related events on safety-related facilities, and none are incorporated into the technical specifications or emergency procedures.

2.4.15 Combined License Information

2.4.15.1 Hydrological Description

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

2.4.15.2 Floods

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

2.4.15.3 Cooling Water Supply

The water supply sources to provide makeup water to the service water system cooling tower are addressed in Subsection 2.4.12.

2.4.15.4 Groundwater

Site-specific information on groundwater is addressed in Subsection 2.4.12.

2.4.15.5 Accidental Release of Liquid Effluents in Ground and Surface Water

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

2.4.15.6 Emergency Operation Requirement

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

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Table 2.4-201
Rainfall Depths Used as Input for Frequency Storm HEC-HMS Module

Duration, Minutes	Depth, Inches
5	6.20[a]
15	9.80[a]
60	19.20[a]
120	23.52[b]
180	25.95[b]
360	31.00[a]

a) Calculated with HMR51/52
b) Calculated by curve fitting

Table 2.4-202
Subbasin Parameters for Entry in Unit 3 & 4 Drainage System HEC-HMS Model

Subbasin	Subbasin Area sq. ft	acres	sq. mi	Base flow? 0:no, 1: yes	Baseflow, cfs	Tc, min	Lag, min
FD1W	222,801	5.11	0.0080	0	0.0	13.9	8.4
FD2E	373,473	8.57	0.0134	0	0.0	26.4	15.8
FD2W	265,336	6.09	0.0095	0	0.0	24.8	14.9
FD3E	387,040	8.89	0.0139	0	0.0	10.8	6.5
FD3W	576,756	13.24	0.0207	1	2.1	22.2	13.3
FD5aN	555,561	12.75	0.0199	0	0.0	9.9	5.9
FD6aE	407,411	9.35	0.0146	1	1.5	18.7	11.2
FD6bE	872,411	20.03	0.0313	1	3.1	22.8	13.7
LD2	885,606	20.33	0.0318	1	3.2	13.7	8.2
LD3	295,463	6.78	0.0106	1	1.1	12.9	7.7
LD4	302,610	6.95	0.0109	1	1.1	12.0	7.2
LD5	133,962	3.08	0.0048	1	0.5	9.8	5.9
M1S	245,755	5.64	0.0088	0	0.0	9.1	5.5
M1W	210,975	4.84	0.0076	0	0.0	8.7	5.2
M2E	354,572	8.14	0.0127	0	0.0	10.6	6.3
M2S	650,674	14.94	0.0233	0	0.0	21.4	12.8
M2W	252,841	5.80	0.0091	0	0.0	8.9	5.4
M3E	406,105	9.32	0.0146	0	0.0	8.1	4.9
M3S	821,527	18.86	0.0295	0	0.0	15.6	9.3
M3W	578,010	13.27	0.0207	1	2.1	18.7	11.2
M4W	94,250	2.16	0.0034	1	0.3	15.3	9.2
M5W	289,232	6.64	0.0104	1	1.0	15.9	9.5
M6E	744,743	17.10	0.0267	0	0.0	11.3	6.8
M6W	394,537	9.06	0.0142	1	1.4	14.5	8.7
M7E	544,496	12.50	0.0195	1	2.0	20.1	12.1
M7W	290,945	6.68	0.0104	1	1.0	14.2	8.5
M8Cat	1,831,895	42.05	0.0657	1	6.6	37.6	22.6
M8W	326,470	7.49	0.0117	1	1.2	14.3	8.6
OF1	4,204,636	96.53	0.1508	1	15.1	45.5	27.3
OF2	1,597,617	36.68	0.0573	1	5.7	32.0	19.2
UN12-N	762,505	17.50	0.0274	0	0.0	25.1	15.1
UN12-S	758,482	17.41	0.0272	0	0.0	25.1	15.1
Totals:	20,638,697	473.80	0.7403	0	48.9		

Table 2.4-203
Routing Parameters for Reaches in South-Side Drainage System Model

Direct Lag Reaches:

Reach	Length, ft.	vel., fps	lag. min.
FD1-M1	600	2	5.0
FD2-M2	600	1	10.0
FD3-M3	700	3.5	3.3
FD5a-M6	1150	3.5	5.5
FD6a-M7	1150	3.5	5.5
FD6b-a	900	3.5	4.3
M3-M4	200	5	0.7
OF1-FD3	N/A	*	23.9
OF2-FD3	N/A	**	10.0
RROF2-M3	775	2.1	6.2
RROF3-M5	600	2.1	4.8
RROF4-M6	600	2.1	4.8
RROF5-M7	600	2.1	4.8

* Use the time of concentration for subbasin FD3W for lag time.

** Use shallow concentrated flow velocity from FD3W for 350 feet and total ditch flow time.

Kinematic Wave Reaches:

Reach	Length, ft.	slope ft./ft.	n	# Subreaches	Shape	W, ft.	XH:1V
M1-M2	900	0.0022	0.015	2	TRAP	10	2
M2-M3	1,100	0.0022	0.015	2	TRAP	10	2
M4-M5	500	0.0022	0.015	2	TRAP	14	2
M5-M6	600	0.0022	0.015	2	TRAP	14	2
M6-M7	450	0.0022	0.015	2	TRAP	14	2
M7-M8	670	0.0526	0.015	2	TRAP	14	2

Table 2.4-204 (Sheet 1 of 2)
Summary Results of HEC-HMS Model of Unit 3 & 4 PMP Drainage System

Element	Area, mi2	Qpeak, cfs	Time of peak *	Runoff	
				inches	ac-ft
FD1	0.0354	698	01Jan2007, 03:15	30.68	57.9
FD1-M1	0.0354	698	01Jan2007, 03:20	30.68	57.9
FD1W	0.0080	200	01Jan2007, 03:10	30.68	13.1
FD2	0.0229	434	01Jan2007, 03:20	30.68	37.5
FD2E	0.0134	253	01Jan2007, 03:20	30.68	21.9
FD2-M2	0.0229	434	01Jan2007, 03:30	30.68	37.5
FD2W	0.0095	183	01Jan2007, 03:15	30.68	15.5
FD3	0.2427	2,924	01Jan2007, 03:45	34.19	442.5
FD3E	0.0139	380	01Jan2007, 03:10	30.68	22.7
FD3-M3	0.2427	2,924	01Jan2007, 03:50	34.19	442.5
FD3W	0.0207	426	01Jan2007, 03:15	34.45	38
FD5a	0.0199	547	01Jan2007, 03:10	30.68	32.6
FD5a-M6	0.0199	547	01Jan2007, 03:15	30.68	32.6
FD5aN	0.0199	547	01Jan2007, 03:10	30.68	32.6
FD6a	0.0459	902	01Jan2007, 03:20	34.41	84.2
FD6aE	0.0146	321	01Jan2007, 03:15	34.5	26.9
FD6a-M7	0.0459	902	01Jan2007, 03:25	34.41	84.2
FD6b	0.0313	635	01Jan2007, 03:15	34.36	57.4
FD6b-a	0.0313	629	01Jan2007, 03:20	34.36	57.4
FD6bE	0.0313	635	01Jan2007, 03:15	34.36	57.4
LD2	0.0318	808	01Jan2007, 03:10	34.42	58.4
LD3	0.0106	277	01Jan2007, 03:10	34.54	19.5
LD4	0.0109	291	01Jan2007, 03:10	34.43	20
LD5	0.0048	133	01Jan2007, 03:10	34.55	8.8
M1	0.0790	1,433	01Jan2007, 03:15	30.68	129.3
M1-M2	0.0790	1,420	01Jan2007, 03:20	30.7	129.3
M1S	0.0088	241	01Jan2007, 03:10	30.68	14.4
M1W	0.0076	207	01Jan2007, 03:10	30.68	12.4

* for an assumed start of storm time of 01 Jan 2007, 0:00 hours

Table 2.4-204 (Sheet 2 of 2)
Summary Results of HEC-HMS Model of Unit 3 & 4 PMP Drainage System

Element	Area, mi2	Qpeak, cfs	Time of peak *	Runoff	
				inches	ac-ft
M2	0.1470	2,546	01Jan2007, 03:15	30.69	240.6
M2E	0.0127	349	01Jan2007, 03:10	30.68	20.8
M2-M3	0.1470	2,530	01Jan2007, 03:15	30.7	240.7
M2S	0.0233	486	01Jan2007, 03:15	30.68	38.1
M2W	0.0091	249	01Jan2007, 03:10	30.68	14.9
M3	0.4863	6,291	01Jan2007, 03:15	32.84	851.8
M3E	0.0146	403	01Jan2007, 03:05	30.68	23.9
M3-M4	0.4863	6,291	01Jan2007, 03:15	32.84	851.8
M3S	0.0295	699	01Jan2007, 03:10	30.68	48.3
M3W	0.0207	455	01Jan2007, 03:15	34.45	38
M4	0.4897	6,367	01Jan2007, 03:15	32.85	858
M4-M5	0.4897	6,336	01Jan2007, 03:15	32.86	858.1
M4W	0.0034	81	01Jan2007, 03:10	33.96	6.2
M5	0.5107	6,835	01Jan2007, 03:15	32.92	896.7
M5-M6	0.5107	6,790	01Jan2007, 03:15	32.93	896.8
M5W	0.0104	244	01Jan2007, 03:10	34.25	19
M6	0.5824	8,463	01Jan2007, 03:15	32.81	1019.1
M6E	0.0267	723	01Jan2007, 03:10	30.68	43.7
M6-M7	0.5824	8,431	01Jan2007, 03:15	32.82	1019.3
M6W	0.0142	351	01Jan2007, 03:10	34.35	26
M7	0.6630	9,919	01Jan2007, 03:15	33.01	1167.3
M7E	0.0195	418	01Jan2007, 03:15	34.49	35.9
M7-M8	0.6630	9,896	01Jan2007, 03:15	33.01	1167.4
M7W	0.0104	260	01Jan2007, 03:10	34.25	19
M8	0.7404	11,021	01Jan2007, 03:15	33.16	1309.5
M8Cat	0.0657	1,058	01Jan2007, 03:25	34.41	120.6
M8W	0.0117	291	01Jan2007, 03:10	34.49	21.5
OF1	0.1508	2,203	01Jan2007, 03:30	34.4	276.7
OF1-FD3	0.1508	2,178	01Jan2007, 03:55	34.4	276.7
OF2	0.0573	993	01Jan2007, 03:20	34.38	105.1
OF2-FD3	0.0573	993	01Jan2007, 03:30	34.38	105.1
RROF2	0.0318	808	01Jan2007, 03:10	34.42	58.4
RROF2-M3	0.0318	763	01Jan2007, 03:15	34.42	58.4
RROF3	0.0106	277	01Jan2007, 03:10	34.54	19.5
RROF3-M5	0.0106	267	01Jan2007, 03:15	34.54	19.5
RROF4	0.0109	291	01Jan2007, 03:10	34.43	20
RROF4-M6	0.0109	278	01Jan2007, 03:15	34.43	20
RROF5	0.0048	133	01Jan2007, 03:10	34.55	8.8
RROF5-M7	0.0048	124	01Jan2007, 03:15	34.55	8.8
UN12-N	0.0274	522	01Jan2007, 03:15	30.68	44.8
UN12-S	0.0272	518	01Jan2007, 03:15	30.68	44.5

* for an assumed start of storm time of 01 Jan 2007, 0:00 hours

Table 2.4-205
Location of Main Channel Discharge Points in HEC-RAS Model

Main Ditch Section	HEC-RAS Section	PMP Q, cfs	Comment	HEC-HMS node
0+00	46	759	Upstream end of model	M1S+UN12-S
0+60	45.1	1433	Confluence of Feeder Ditch 1	M1
10+00	36	2546	Confluence of Feeder Ditch 2	M2
20+00	26	6291	Confluence of Feeder Ditch 3	M3
22+00	24	6367	Confluence of Feeder Ditch 4	M4
27+00	19	6835	Local Inflow	M5
33+00	13	8463	Confluence of Feeder Ditch 5	M6
38+00	8	9919	Confluence of ditch (not modeled)	M7
45+00	1	11021	Local Inflow	M8

Table 2.4-206
Location of Main Channel Discharge Points in HEC-RAS Model

Feeder Ditch	HEC-RAS Section	PMP Q, cfs	Comment	HEC-HMS node
1	83	103	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
1	82	206		
1	81	309		
1	80	413		
1	79	516		
1	78	619		
1	77	722	Q = FD1W + UN12-N	FD1
1	76	843	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
1	75	964		
1	74	1085		
1	73	1205		
1	72	1326		
1	71	1447	Q = M1W + UN12-S + FD1W + UN12-N	U/S of M1
2	83	62	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
2	82	124		
2	81	186		
2	80	249		
2	79	311		
2	78	373		
2	77	435	Q = FD2W + FD2E	FD2
2	76	535	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
2	75	634		
2	74	734		
2	73	834		
2	72	933		
2	71	1033	Q = M2W + M2E + FD2W + FD2E	U/S of M2
3	83	568	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
3	82	1136		
3	81	1705		
3	80	2273		
3	79	2841		
3	78	3409		
3	77	3978	Q = FD3W + FD3E + OF1-FD3 + OF2-FD3	FD3
3	76	4172	Distribute discretized HEC-HMS node flow between all HEC-RAS sections	
3	75	4366		
3	74	4561		
3	73	4755		
3	72	4949		
3	71	5144	Q = Sum(FD3)+ M3E + RROF2-M3	U/S of M3

Table 2.4-207 (Sheet 1 of 3)
Summary of HEC-RAS output for PMP Profile

HEC-RAS identification		Section stationing	Q Total	Min Ch El	Bottom Width, ft	W.S. Elev	Crit W.S.	E.G. Elev	Vel Chnl	Froude # Chl
Channel	Station		(cfs)	(ft)		(ft)	(ft)	(ft)	(ft/s)	
Main Ditch	46	00+00	759	207.66	10.00	219.39	213.16	219.39	0.5	0.03
Main Ditch	45.5	00+30	759	207.66	10.00	219.39	213.16	219.39	0.5	0.03
Main Ditch	45.1	00+60	1,433	207.56	10.00	219.39	214.47	219.39	0.9	0.05
Main Ditch	45	01+00	1,433	207.41	10.00	219.39	213.84	219.39	0.9	0.05
Main Ditch	44	02+00	1,433	207.19	10.00	219.39	213.68	219.39	0.9	0.05
Main Ditch	43	03+00	1,433	206.98	10.00	219.39	213.54	219.39	0.9	0.05
Main Ditch	42	04+00	1,433	206.76	10.00	219.39		219.39	0.8	0.05
Main Ditch	41	05+00	1,433	206.54	10.00	219.39	212.99	219.39	0.9	0.05
Main Ditch	40	06+00	1,433	206.32	10.00	219.38	212.32	219.39	1.1	0.06
Main Ditch	39	07+00	1,433	206.11	10.00	219.38	212.09	219.39	0.9	0.05
Main Ditch	38	08+00	1,433	205.89	10.00	219.38	211.83	219.39	0.9	0.05
Main Ditch	37	09+00	1,433	205.67	10.00	219.38	211.58	219.39	0.9	0.05
Main Ditch	36	10+00	2,546	205.45	10.00	219.37	213.59	219.39	1.6	0.09
Main Ditch	35	11+00	2,546	205.24	10.00	219.37	213.31	219.38	1.6	0.09
Main Ditch	34	12+00	2,546	205.02	10.00	219.37	213.03	219.38	1.5	0.09
Main Ditch	33	13+00	2,546	204.80	10.00	219.36	212.28	219.38	1.6	0.08
Main Ditch	32	14+00	2,546	204.58	10.00	219.36	212.02	219.38	1.4	0.08
Main Ditch	31	15+00	2,546	204.37	10.00	219.36	211.75	219.38	1.5	0.08
Main Ditch	30	16+00	2,546	204.15	10.00	219.36	211.49	219.38	1.5	0.08
Main Ditch	29	17+00	2,546	203.93	10.00	219.35	211.22	219.38	1.7	0.09
Main Ditch	28	18+00	2,546	203.71	10.00	219.35	210.96	219.37	1.6	0.08
Main Ditch	27	19+00	2,546	203.50	10.00	219.35	210.69	219.37	1.6	0.08
Main Ditch	26.95	19+05	2,546	203.49	10.00	219.35	211.37	219.37	1.7	0.09
Main Ditch	26	20+00	6,291	203.28	14.00	219.26		219.36	3.7	0.19
Main Ditch	25	21+00	6,291	203.06	14.00	219.17		219.35	4.5	0.23
Main Ditch	24.44	21+56	6,291	202.94	14.00	219.17		219.34	4.4	0.22
Main Ditch	24.39	21+61	6,291	202.94	14.00	219.20		219.32	3.9	0.20
Main Ditch	24	22+00	6,367	202.84	14.00	218.90		219.29	6.0	0.32
Main Ditch	23	23+00	6,367	202.63	14.00	218.84		219.27	6.1	0.33
Main Ditch	22	24+00	6,367	202.41	14.00	218.42		219.21	7.6	0.42
Main Ditch	21	25+00	6,367	202.19	14.00	217.94	213.62	219.13	8.8	0.50
Main Ditch	20	26+00	6,367	201.97	14.00	217.92		219.08	8.7	0.49
Main Ditch	19	27+00	6,835	201.88	14.00	217.59		219.01	9.6	0.55
Main Ditch	18	28+00	6,835	201.54	14.00	217.61		218.93	9.2	0.53
Main Ditch	17	29+00	6,835	201.32	14.00	217.60		218.86	9.0	0.51
Main Ditch	16	30+00	6,835	201.00	14.00	217.67		218.78	8.5	0.47
Main Ditch	15	31+00	6,835	200.89	14.00	217.72		218.71	8.2	0.44
Main Ditch	14	32+00	6,835	200.67	14.00	217.80		218.63	7.6	0.41
Main Ditch	13	33+00	8,463	200.45	14.00	216.97		218.51	10.3	0.57
Main Ditch	12	34+00	8,463	200.23	14.00	216.54	213.37	218.42	11.0	0.63
Main Ditch	11	35+00	8,463	200.02	14.00	216.57		218.31	10.7	0.60
Main Ditch	10	36+00	8,463	199.80	14.00	216.54		218.23	10.5	0.59
Main Ditch	9	37+00	8,463	199.58	14.00	216.58		218.14	10.1	0.56
Main Ditch	8	38+00	9,919	199.36	14.00	213.50	213.50	217.77	16.6	1.00
Main Ditch	7	39+00	9,919	198.68	14.00	211.54	213.15	217.39	19.4	1.22
Main Ditch	6	40+00	9,919	198.00	14.00	209.75	211.96	216.92	21.7	1.49
Main Ditch	5	41+00	9,919	196.51	14.00	207.34	210.13	216.27	24.4	1.66
Main Ditch	4	42+00	9,919	192.00	14.00	198.94	202.49	214.63	36.3	3.15
Main Ditch	3	43+00	9,919	183.22	14.00	189.47	193.41	211.29	41.9	3.90
Main Ditch	2	44+00	9,919	172.00	14.00	179.37	179.37	181.90	17.3	1.24
Main Ditch	1	45+00	11,021	161.21	14.00	167.41	170.79	180.33	30.7	2.39

Table 2.4-207 (Sheet 2 of 3)
Summary of HEC-RAS output for PMP Profile

HEC-RAS identification		Section stationing	Q Total	Min Ch El	Bottom Width, ft	W.S. Elev	Crit W.S.	E.G. Elev	Vel Chnl	Froude # Chl
Channel	Station		(cfs)	(ft)		(ft)	(ft)	(ft)	(ft/s)	
Feeder Ditch 3	83	83+00	568	211.70	5.00	219.43	214.83	219.44	0.4	0.02
Feeder Ditch 3	82	82+00	1,136	211.20	5.00	219.43		219.43	0.8	0.05
Feeder Ditch 3	81.9	81+90	1,136	211.15	5.00	219.43	215.22	219.43	0.7	0.05
Feeder Ditch 3	81.75	81+75	Inl Struct							
Feeder Ditch 3	81.6	81+60	1,136	211.00	5.00	219.42		219.42	0.7	0.04
Feeder Ditch 3	81	81+00	1,705	210.70	5.00	219.41		219.42	1.5	0.10
Feeder Ditch 3	80	80+00	2,273	210.20	5.00	219.40		219.42	2.0	0.12
Feeder Ditch 3	79	79+00	2,841	209.70	5.00	219.38		219.42	2.5	0.16
Feeder Ditch 3	78	78+00	3,409	209.20	5.00	219.36		219.41	3.0	0.19
Feeder Ditch 3	77	77+00	3,978	208.70	5.00	219.33		219.40	3.4	0.21
Feeder Ditch 3	76	76+00	4,172	208.20	5.00	219.34		219.39	3.0	0.18
Feeder Ditch 3	75	75+00	4,366	207.70	5.00	219.34		219.39	2.9	0.17
Feeder Ditch 3	74	74+00	4,561	207.20	5.00	219.32		219.38	3.1	0.18
Feeder Ditch 3	73	73+00	4,755	206.70	5.00	219.31		219.37	3.2	0.18
Feeder Ditch 3	72	72+00	4,949	206.20	5.00	219.30		219.37	3.3	0.19
Feeder Ditch 3	71	71+00	5,144	205.70	5.00	219.28		219.36	3.4	0.19
Feeder Ditch 2	83	83+00	62	214.26	5.00	219.42	214.99	219.42	0.1	0.00
Feeder Ditch 2	82	82+00	124	212.76	5.00	219.42		219.42	0.1	0.01
Feeder Ditch 2	81.95	81+95	124	212.74	5.00	219.42	214.85	219.42	0.1	0.01
Feeder Ditch 2	81.65	81+65	Inl Struct							
Feeder Ditch 2	81.35	81+35	124	212.44	5.00	219.41		219.41	0.1	0.01
Feeder Ditch 2	81	81+00	186	212.26	5.00	219.41		219.41	0.2	0.01
Feeder Ditch 2	80.65	80+65	186	212.06	5.00	219.41	214.65	219.41	0.2	0.01
Feeder Ditch 2	80.3	80+30	Inl Struct							
Feeder Ditch 2	80	80+00	249	211.76	5.00	219.41		219.41	0.2	0.02
Feeder Ditch 2	79.95	79+95	249	211.74	5.00	219.41		219.41	0.2	0.02
Feeder Ditch 2	79	79+00	311	211.26	5.00	219.41		219.41	0.3	0.02
Feeder Ditch 2	78.45	78+45	311	210.98	5.00	219.41	213.92	219.41	0.3	0.02
Feeder Ditch 2	78.2	78+20	Inl Struct							
Feeder Ditch 2	78	78+00	373	210.76	5.00	219.40		219.40	0.3	0.02
Feeder Ditch 2	77.95	77+95	373	210.74	5.00	219.40		219.40	0.3	0.02
Feeder Ditch 2	77	77+00	435	210.26	5.00	219.40		219.40	0.5	0.03
Feeder Ditch 2	76	76+00	535	209.76	5.00	219.40		219.40	0.8	0.05
Feeder Ditch 2	75	75+00	634	209.26	5.00	219.40		219.40	0.6	0.04
Feeder Ditch 2	74.95	74+95	634	209.24	5.00	219.40	213.35	219.40	0.6	0.04
Feeder Ditch 2	74.7	74+70	Inl Struct							
Feeder Ditch 2	74.45	74+45	634	208.99	5.00	219.39		219.39	0.6	0.04
Feeder Ditch 2	74.15	74+15	734	208.33	5.00	219.39	212.75	219.39	0.6	0.04
Feeder Ditch 2	74.1	74+10	Inl Struct							
Feeder Ditch 2	73.35	73+35	734	208.43	5.00	219.39		219.39	0.7	0.05
Feeder Ditch 2	73	73+00	834	208.26	5.00	219.38		219.39	0.8	0.05
Feeder Ditch 2	72	72+00	933	207.76	5.00	219.38		219.39	0.8	0.05
Feeder Ditch 2	71	71+00	1,033	207.26	5.00	219.38		219.39	0.9	0.06
Feeder Ditch 1	83	83+00	103	216.00	5.00	219.47	216.65	219.47	0.2	0.02
Feeder Ditch 1	82	82+00	206	215.70	5.00	219.47	216.85	219.47	0.4	0.03

Table 2.4-207 (Sheet 3 of 3)
Summary of HEC-RAS output for PMP Profile

HEC-RAS identification		Section stationing	Q Total	Min Ch El	Bottom Width, ft	W.S. Elev	Crit W.S.	E.G. Elev	Vel Chnl	Froude # Chl
Channel	Station		(cfs)	(ft)		(ft)	(ft)	(ft)	(ft/s)	
Feeder Ditch 1	81.9	81+90	206	215.65	5.00	219.47	216.81	219.47	0.4	0.03
Feeder Ditch 1	81.75	81+75	Inl Struct							
Feeder Ditch 1	81.6	81+60	206	215.50	5.00	219.46	216.67	219.46	0.4	0.03
Feeder Ditch 1	81	81+00	309	215.20	5.00	219.46	217.02	219.46	0.6	0.05
Feeder Ditch 1	80	80+00	413	214.70	5.00	219.46	217.15	219.46	0.8	0.07
Feeder Ditch 1	79	79+00	516	214.20	5.00	219.46	217.26	219.46	1.0	0.08
Feeder Ditch 1	78	78+00	619	213.70	5.00	219.45	217.35	219.46	1.2	0.10
Feeder Ditch 1	77	77+00	722	213.20	5.00	219.44	217.40	219.46	1.5	0.12
Feeder Ditch 1	76	76+00	843	212.70	5.00	219.43	217.43	219.45	1.8	0.14
Feeder Ditch 1	75.1	75+10	843	212.25	5.00	219.43	217.30	219.45	1.7	0.12
Feeder Ditch 1	75	75+00	964	212.20	5.00	219.43	217.41	219.45	1.9	0.14
Feeder Ditch 1	74.85	74+85	Inl Struct							
Feeder Ditch 1	74.7	74+70	964	212.05	5.00	219.42	217.26	219.44	1.7	0.13
Feeder Ditch 1	74	74+00	1,085	211.70	5.00	219.39	217.38	219.43	2.4	0.18
Feeder Ditch 1	73	73+00	1,205	211.20	5.00	219.38	217.29	219.42	2.3	0.17
Feeder Ditch 1	72	72+00	1,326	210.70	5.00	219.37	217.13	219.42	2.5	0.18
Feeder Ditch 1	71	71+00	1,447	210.20	5.00	219.35	216.22	219.41	2.7	0.18

Table 2.4-208
Savannah River Subbasins and Drainage Areas above VEGP Site

NWS Subbasin			Drainage Area, mi^2	
No.	I.D.	NWS Subbasin Name	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 2.4-209
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
Confluence 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 2.4-210
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 2.4-211
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 ft^3/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.

Table 2.4-212
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.

Table 2.4-213
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 2.4-214
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	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 2.4-206
** from outfall to end of longest tributary

Table 2.4-215
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 2.4-216
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 2.4-217 (Sheet 1 of 2)
Plant Water Use

Stream Description	Normal Case[a] gpm	Maximum Case[a,b] gpm	Comments
Groundwater (Well) Streams:			
Plant Well Water Demand	752	2,797	
Well Water for Service Water System Makeup	537	1,600	
• Service Water System Consumptive Use	403	1,100	
– Service Water System Evaporation	402	1,099	
– Service Water System Drift	1	1	c
• Service Water System Blowdown	134	500	d
Well Water for Power Plant Makeup/Use	215	1,197	
• Demineralized Water System Feed	150	1,080	
– Plant System Makeup/Processes	109	999	
– Misc. Consumptive Use	41	81	
• Potable Water Feed	42	70	
• 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 Makeup	38,825	61,145	
• Circulating Water/Turbine Plant Cooling Water System Consumptive Use	29,125	30,585	
Circulating Water/Turbine Plant Cooling Water System Evaporation	29,100	30,560	
Circulating Water/Turbine Plant Cooling Water System Drift	25	25	c
• Circulating Water/Turbine Plant Cooling Water System Blowdown	9,700	30,560	d

Table 2.4-217 (Sheet 2 of 2) 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.

Table 2.4-218
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 2.4-219
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 2.4-220
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

Table 2.4-221
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

Table 2.4-222
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

Table 2.4-223
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

Table 2.4-224
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

Table 2.4-225
Normal Pool Storage Volumes

Dam	Mode 1 Reservoir Storage Volume (1,000 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 2.4-226
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 $\leq d_{ovtop}$ 0.61 m 0.25 hr $\leq t_f \leq$ 1.0 hr
MacDonald & Langridge-Monopolis (1984)	42	Earthfill dams: $V_{er} = 0.0261\ (V_{out} * h_w)^{0.769}$ [best-fit] $T_f = 0.0179(V_{er})^{0.364}$ [upper envelope] Non-earthfill dams: $V_{er} = 0.00348(V_{out} * h_w)^{0.852}$ [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 2.4-227
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*		2.58
V_{er}		15085176.57 m^3
K_o		1.4
K_c		0.6

Table 2.4-228
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)	422.7	1387	0.9	
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 2.4-229
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*		2.95
V_{er}		7274160.639 m^3
K_o		1.4
K_c		0.6

Table 2.4-230
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)	320.8	1053	1.0	
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

Table 2.4-231
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	20.6 3	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)

Table 2.4-232
Variation in Lowest Average Daily Temperatures and Number of Days with Average Daily Temperature Below Freezing

Year	Lowest Average Daily Temp °F (°C)		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 2.4-233
Variation in the Minimum Water Temperatures at Five Locations on the Savannah River

USGS Station No.	Location	River Mile	Data Period	Observed Minimum Temperature, °F (°C)											
				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

Table 2.4-234
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 2.4-235
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 (mi^2)	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 2.4-236 (Sheet 1 of 4)
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			
1934	1,950			

Table 2.4-236 (Sheet 2 of 4)
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
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	
1962	4,760		5,700	
1963	5,130		6,260	
1964	6,120		6,900	

Table 2.4-236 (Sheet 3 of 4)
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
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	
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	

Table 2.4-236 (Sheet 4 of 4)
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
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 2.4-237
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 2.4-238
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 2.4-239
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 Subsection 2.4.11.1.4
Source: USGS 2006j

Table 2.4-240
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

Table 2.4-241
Monthly Groundwater Level Elevations in the Water Table Aquifer (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	Dec-06	Jan-07	Feb-07	Mar-07	Apr-07	May-07	Jun-07	Jul-07
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	154.03	154.00	153.97	153.93	153.75	153.59	153.61	153.59
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	148.78	148.57	148.89	148.51	148.45	148.40	148.40	148.44
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	157.37	157.24	157.19	157.67	156.92	156.80	156.79	156.75
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	159.50	159.25	159.30	159.25	158.94	158.80	158.80	158.78
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	163.19	162.95	162.98	163.09	162.47	162.59	162.70	162.82
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	158.01	158.77	157.67	157.69	157.40	157.31	157.27	157.29
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	155.21	155.06	155.10	155.09	154.89	154.71	154.72	154.69
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	158.40	158.00	157.96	158.17	158.01	158.06	158.02	157.93
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	152.62	152.63	152.65	152.62	152.37	152.30	152.32	152.3
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	154.63	154.51	154.33	154.35	154.28	153.98	153.98	153.9
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	154.01	153.96	153.68	153.70	153.68	153.24	153.36	153.32
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	157.79	157.77	157.48	157.60	157.53	156.95	157.19	157.05
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	155.75	155.63	155.41	155.55	155.30	155.05	155.10	155.05
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	155.70	155.60	155.91	155.70	155.46	155.34	155.35	155.3
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	132.39	132.32	132.51	132.25	132.18	132.07	132.04	132.14
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	146.26	146.25	146.47	146.10	145.98	145.60	145.70	145.58
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	149.94	150.06	150.24	150.26	150.14	149.96	149.86	149.7
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	161.95	161.74	161.89	161.80	161.65	161.54	161.53	161.42
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	162.33	162.24	162.23	162.40	162.45	162.33	162.25	161.89
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	160.31	160.19	160.26	160.23	160.15	159.95	159.95	159.8
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	164.01	163.76	163.94	163.75	163.68	163.49	163.49	163.39
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	158.83	158.58	158.63	158.52	158.24	158.07	158.04	157.94

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).**

Groundwater level data for the period between December 2006 and July 2007 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00050, SNC ALWR ESP Project. **(Bechtel Power Corporation, August 2007)**.

Table 2.4-242
Monthly Groundwater Level Elevations in the Tertiary Aquifer (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	Dec-06	Jan-07	Feb-07	Mar-07	Apr-07	May-07	Jun-07	Jul-07
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	81.68	85.32	83.04	82.84	83.49	82.90	81.64	81.01
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	90.35	92.84	92.53	92.54	92.24	91.49	90.69	89.50
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	98.21	100.01	100.60	100.50	99.88	99.19	98.66	97.06
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	108.71	110.36	111.20	111.07	110.56	109.66	108.91	106.31
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	109.14	110.96	111.40	111.44	111.14	109.55	108.44	105.79
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	101.55	103.35	104.09	103.98	103.32	102.63	102.10	100.40
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	100.35	101.88	102.85	102.72	102.05	101.44	100.90	99.30
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	94.35	97.34	96.64	96.74	96.48	95.57	94.60	92.92
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	109.44	110.43	110.86	110.95	110.57	109.04	107.94	105.32
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	115.52	117.20	117.74	117.97	117.52	116.56	115.80	112.68
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	101.79	103.32	104.35	104.22	103.67	102.95	102.49	100.87
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	121.26	122.86	123.51	123.41	123.17	122.22	121.19	117.63
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	117.51	119.20	119.82	119.85	119.54	118.16	117.04	113.77
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	105.04	106.63	107.51	107.45	107.03	106.16	105.59	103.63

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).

Groundwater level data for the period between December 2006 and July 2007 provided Request For Information (RFI) Number 25144-000-GRI-GEX-00050, SNC ALWR ESP Project. (Bechtel Power Corporation, August 2007).

Table 2.4-243 (Sheet 1 of 2)
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 2.4-243 (Sheet 2 of 2)
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 2.4-244 (Sheet 1 of 2)
Summary of Laboratory Test Results on Grain Size, Moisture Content and Specific Gravity for the Barnwell Formation

ESP / COL	Borehole / Well No.	Sample Elevation	Water Table Elevation1	Grain Size Distribution			Moisture Content	Specific Gravity
				Gravel	Sand	Clay/Silt		
		(ft msl)	(ft msl)	(%)	(%)	(%)	(%)	
ESP	OW-1003	144.5	156.0	0.0	65.1	34.9	ND	2.69
ESP	OW-1003	139.5	156.0	31.1	50.0	18.4	ND	2.68
ESP	OW-1005	115.9	132.0	8.9	57.0	34.1	ND	2.63
ESP	OW-1005	110.9	132.0	18.2	47.6	34.3	ND	2.61
ESP	OW-1006	113.6	146.0	7.0	61.1	31.9	ND	2.67
ESP	OW-1006	108.6	146.0	3.6	74.4	22.0	ND	2.59
ESP	OW-1007	113.4	152.0	0.0	85.0	15.0	ND	2.65
ESP	OW-1007	108.4	152.0	0.0	85.0	18.1	ND	2.66
ESP	OW-1009	135.9	162.0	2.7	74.6	22.7	ND	2.61
ESP	OW-1009	130.9	162.0	34.7	45.9	19.2	ND	2.75
ESP	OW-1010	143.4	163.0	0.0	89.3	10.7	ND	2.67
ESP	OW-1010	138.4	163.0	0.0	63.5	36.5	ND	2.63
ESP	OW-1012	131.9	162.0	0.0	76.1	23.9	ND	2.66
ESP	OW-1012	126.9	162.0	0.0	14.1	85.9	ND	2.66
ESP	OW-1013	132.9	164.0	0.0	91.1	8.9	ND	2.65
ESP	OW-1013	122.9	164.0	0.0	91.1	8.9	ND	2.65
ESP	OW-1015	126.9	160.0	0.0	97.7	2.8	ND	2.63
ESP	OW-1015	125.4	160.0	0.0	93.2	6.8	ND	2.67
ESP	B-1002	148.5	150.0	0.4	89.6	10.0	24.5	ND
ESP	B-1002	138.5	150.0	0.0	93.9	6.1	27.6	ND
ESP	B-1003	148.2	156.0	0.0	91.8	8.2	32.3	ND
ESP	B-1004	126.3	144.0	48.6	32.2	19.2	19.7	ND
ESP	B-1010	160.1	164.0	0.0	86.7	13.3	27.3	ND
COL	B-3001	159.9	159.0	0.0	93.6	6.4	ND	ND
COL	B-3002	155.4	159.0	0.0	84.3	15.7	47.0	ND
COL	B-3003	159.8	159.0	ND	ND	15.4	ND	ND
COL	B-3004	160.0	159.0	ND	ND	5.3	ND	ND
COL	B-3008	159.4	159.0	0.0	84.4	15.6	31.4	ND
COL	B-3024	156.7	155.0	ND	ND	8.4	ND	ND
COL	B-3036	149.4	159.0	0.0	76.5	23.5	ND	ND

Table 2.4-244 (Sheet 2 of 2)
Summary of Laboratory Test Results on Grain Size, Moisture Content and Specific Gravity for the Barnwell Formation

ESP / COL	Borehole / Well No.	Sample Elevation	Water Table Elevation1	Grain Size Distribution			Moisture Content	Specific Gravity
				Gravel	Sand	Clay/Silt		
		(ft msl)	(ft msl)	(%)	(%)	(%)	(%)	
COL	B-4002	145.6	155.0	ND	ND	6.2	ND	ND
COL	B-4004	155.0	157.0	10.5	70.9	18.6	31.5	ND
COL	B-4004	145.0	157.0	2.1	81.4	16.5	24.2	ND
COL	B-4007	159.4	158.0	ND	ND	12.5	ND	ND
COL	B-4009	154.4	157.0	0.0	90.2	9.8	ND	ND
						Median	27.6	2.66

Note.
[1] Elevation at time of sample collection (October 2005 for ESP samples, February 2007 for COL samples)
ND - Not Determined
OW-series data are provided in Appendix 2.4A
B-series ESP data are provided in Appendix 2.5A
B-series COL data are provided in Appendix 2.5C
Moisture content is by weight percent.

Table 2.4-245
Summary of Laboratory Test Results on Grain Size, Moisture Content, and Porosity for the Lisbon Formation

Borehole/Well	Sample	Grain Size Distribution			Moisture	D50	Porosity
No.	Elevation	Gravel	Sand	Clay/Silt	Content		
	(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 2.4-246 (Sheet 1 of 2)
Summary of Laboratory Test Results on Grain Size, Moisture Content, and Specific Gravity for the Still Branch and Congaree Formations

ESP / COL	Borehole / Well No.	Sample Elevation (ft msl)	Grain Size Distribution Gravel (%)	Sand (%)	Clay/Silt (%)	Moisture Content (%)	Specific Gravity
ESP	OW-1002	8.9	0.2	79.6	20.2	ND	2.65
ESP	OW-1002	-9.6	0.0	1.4	90.6	ND	2.62
ESP	OW-1004	69.4	0.1	89.7	10.2	ND	2.69
ESP	OW-1004	64.4	0.0	93.4	6.6	ND	2.67
ESP	OW-1008	-11.9	0.0	83.2	16.8	ND	2.69
ESP	OW-1008	-21.9	2.2	67.9	20.3	ND	2.68
ESP	OW-1011	12.3	0.0	88.9	10.8	ND	2.67
ESP	OW-1011	-2.7	4.5	89.6	5.9	ND	2.66
ESP	OW-1014	37.4	0.0	87.8	12.2	ND	2.69
ESP	OW-1014	32.4	0.0	89.6	10.4	ND	2.66
ESP	B-1002	68.5	20.0	40.6	39.4	23.3	ND
ESP	B-1002	33.5	0.0	93.4	6.6	40.7	ND
ESP	B-1002	16.5	3.1	84.6	12.3	18.5	ND
ESP	B-1003	57.5	0.0	94.6	5.4	23.6	ND
ESP	B-1003	37.5	0.9	82.7	16.4	32.3	ND
ESP	B-1003	17.5	1.4	77.2	21.4	39.3	ND
ESP	B-1003	-17.5	0.0	89.1	10.9	23.2	ND
ESP	B-1003	-57.5	0.3	85.5	14.2	23.2	ND
ESP	B-1003	-92.5	70.7	26.0	3.3	32.7	ND
ESP	B-1003	-127.5	0.0	21.5	78.5	21.3	ND
ESP	B-1003	-177.5	0.3	83.9	15.8	18.9	ND
ESP	B-1003	-227.5	0.0	84.1	15.9	28.6	ND
ESP	B-1003	-273.5	0.0	86.8	13.2	26.4	ND
COL	B-3001	44.9	ND	ND	11.6	24.2	2.65
COL	B-3001	24.9	ND	ND	ND	28.1	ND
COL	B-3001	9.9	ND	ND	17.0	21.4	ND
COL	B-3003	55.3	ND	ND	20.8	20.1	ND
COL	B-3021	74.7	0.5	91.6	7.9	ND	ND
COL	B-3021	69.7	0.0	87.5	12.5	19.3	ND
COL	B-3023	74.8	ND	ND	ND	24.5	ND

Table 2.4-246 (Sheet 2 of 2)
Summary of Laboratory Test Results on Grain Size, Moisture Content, and Specific Gravity for the Still Branch and Congaree Formations

ESP / COL	Borehole / Well No.	Sample Elevation (ft msl)	Grain Size Distribution			Moisture Content (%)	Specific Gravity
			Gravel (%)	Sand (%)	Clay/Silt (%)		
COL	B-3036	64.4	9.4	81.1	9.5	20.0	ND
COL	B-4001	40.4	0.9	73.9	25.2	32.7	ND
COL	B-4001	20.4	ND	ND	19.7	ND	ND
COL	B-4001	-19.6	ND	ND	69.6	18.6	ND
COL	B-4001	-29.6	ND	ND	ND	16.5	2.65
COL	B-4001	-79.6	ND	ND	13.7	ND	ND
COL	B-4001	-89.6	ND	ND	67.4	16.7	2.68
COL	B-4002	45.6	ND	ND	5.4	17.9	ND
COL	B-4002	-14.4	ND	ND	ND	27.7	ND
COL	B-4003	18.0	ND	ND	18.3	ND	ND
COL	B-4003	-29.5	ND	ND	17.8	ND	ND
COL	B-6027	63.2	0.0	78.2	21.8	17.3	ND
COL	B-6027	58.2	0.0	93.3	6.7	23.8	ND
COL	B-6027	48.2	0.5	90.8	8.7	29.5	ND
COL	B-6027	43.2	0.0	77.5	22.5	23.8	ND
COL	B-6027	38.2	0.0	78.8	21.2	32.4	ND
COL	B-6027	33.2	0.0	91.2	8.8	24.8	ND
COL	B-6028	67.2	0.3	97.3	2.4	25.6	ND
COL	B-6028	57.2	0.0	98.4	1.6	19.8	ND
COL	B-6028	47.2	4.9	88.3	6.8	19.6	ND
COL	B-6029	46.9	0.0	89.1	10.9	23.4	ND
COL	B-6030	44.9	0.0	44.5	55.5	35.9	ND
					Median	23.6	2.67

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

Table 2.4-247
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 2.4-252 using the listed Well IDs.
Southern Nuclear Operating Co. well locations are shown on Figure 2.4-253.

Table 2.4-248 (Sheet 1 of 2)
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

Table 2.4-248 (Sheet 2 of 2)
Georgia EPD Permitted Agricultural Groundwater Users within 25 miles of the VEGP Site

Well ID	Permit Holder	County	Depth (ft)	Permit (gpm)
A-44	RAYMOND NEIL	Burke	430	1350
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 2.4-252 using the listed Well IDs.

Table 2.4-249
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 2.4-252 using the listed Well IDs.
Southern Nuclear Operating Co. well locations are shown on Figure 2.4-253.

Table 2.4-250
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) (SNC 2005b)
SEC well data (SNC 2005a)
Well locations, excluding Well REC, are shown on Figure 2.4-253. Well REC is located approximately 9300 ft southwest from Well IW-4.

Table 2.4-251
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 2.4-252
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 2.4-253 (Sheet 1 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
COL Boring Data			
B-1105	1144168.4	620002.8	Absent
B-1107	1144153.8	620916.1	Present
B-1108	1144214.1	621273.0	Present
B-1109	1144180.5	621580.6	Present
B-1110	1144170.9	622011.3	Present
B-1111	1144212.6	622333.8	Present
B-1112A	1144219.4	622561.5	Absent
B-1113	1143901.4	620217.2	Present
B-1116	1143894.1	621264.7	Present
B-1117	1143890.8	621628.4	Absent
B-1118	1143885.9	622008.0	Absent
B-1119	1143888.3	622333.8	Present
B-1120	1143893.1	622558.5	Present
B-1121	1143575.6	620216.3	Present
B-1123	1143575.4	620922.0	Present
B-1124	1143627.6	621421.6	Absent
B-1125	1143586.8	621628.2	Present
B-1126	1143567.7	621980.4	Absent
B-1127	1143573.3	622332.3	Absent
B-1128	1143572.7	622682.4	Absent
B-1129	1143278.2	621893.7	Present
B-1130	1142482.8	622250.0	Present
B-1131	1143173.0	621823.1	Present
B-1132	1142614.2	621450.1	Present
B-1133	1142968.9	621451.2	Absent
B-1134	1143282.9	621104.3	Present
B-1136	1143178.1	621023.0	Absent
B-1138	1143469.7	619192.8	NE
B-1139	1142289.9	621026.8	Present
B-1140	1142290.2	621823.6	Present
B-1142	1144416.6	620649.6	NE
B-1146	1145428.4	622272.1	Present
B-1148	1145537.8	623236.5	Absent
B-1150	1145467.3	624235.3	Absent
B-1152	1145581.7	625227.3	Absent

Table 2.4-253 (Sheet 2 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
B-1153	1145569.0	625673.5	Absent
B-1154	1145664.2	626216.1	Absent[1]
B-1155	1147390.3	624936.4	Absent[1]
B-1156	1147302.5	624571.7	Absent[1]
B-1157	1147209.6	625062.2	Absent
B-1158	1145194.9	626669.1	Absent[1]
B-1159	1147285.8	624954.5	Absent
B-1161	1147363.4	624862.1	Absent[1]
B-1162	1147234.9	624815.0	Absent[1]
B-1163	1147170.6	624938.8	Absent
B-1164	1146994.8	624518.6	Present
B-1166	1147453.0	623961.6	Absent
B-1168	1147688.5	623467.8	Absent
B-1170	1147423.9	622953.7	NE
B-1172	1146983.4	622538.7	NE
B-1174	1146476.1	622228.1	NE
B-1176	1145876.3	622195.2	NE
B-1176A	1145878.8	622196.8	Present
B-1185	1144716.6	622232.2	Present
B-1186	1144711.9	618818.9	NE
B-1187	1144710.2	619259.6	NE
B-1189	1144459.7	618997.5	NE
B-1191	1144301.6	619490.8	NE
B-1192	1144217.4	618840.9	Absent
B-1193	1144091.5	619277.8	Absent
B-1194	1147504.7	621630.2	NE
B-1195	1147574.8	622478.4	NE
B-1196	1147286.6	622017.5	NE
B-1197	1146874.7	622003.8	NE
B-3001	1142599.5	621799.6	Present
B-3002	1142600.0	621872.5	Present
B-3003	1142599.9	621727.3	Present
B-3004	1142447.4	621867.1	Present
B-3005	1142717.6	621749.1	Present
B-3006	1142425.6	621925.0	Present
B-3007	1142718.5	621876.7	Present
B-3008	1142425.4	621773.0	Absent
B-3009	1142484.5	621956.6	Present

Table 2.4-253 (Sheet 3 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
B-3010	1142634.9	622025.0	Present
B-3011	1142776.7	622024.9	Present
B-3012	1142772.5	621911.9	Absent
B-3013 (c)	1142842.9	621825.4	Present
B-3014	1142799.4	621748.6	Present
B-3015	1142956.9	621824.0	Present
B-3016	1142978.4	621913.4	Present
B-3017	1143034.4	621749.9	Present
B-3018	1142738.1	622115.8	Present
B-3019	1142977.4	622167.5	Present
B-3020	1142977.9	622074.8	Present
B-3021	1143070.2	622033.2	Present
B-3022	1143069.8	621873.4	Present
B-3023	1143061.1	621679.9	Present
B-3024	1142905.8	621399.7	Absent
B-3025	1142460.4	621425.3	Present
B-3026	1142290.2	621403.7	Present
B-3027	1142058.7	621423.3	Present
B-3028	1141867.3	621408.8	Present
B-3029	1141881.5	621803.9	Present
B-3030	1141699.9	621799.7	Present
B-3031	1141398.7	622042.0	Present
B-3032	1141158.2	621709.5	Present
B-3033	1141405.3	621715.2	Absent
B-3034	1141399.8	621914.7	Present
B-3035	1142729.2	621675.4	Present
B-3036	1142441.6	621676.0	Present
B-3037	1143057.4	621768.9	Present
B-3038	1141883.0	621543.2	Present
B-3039	1142917.7	621753.5	Present
B-4001(DH)	1142599.5	621000.2	Absent
B-4002(DH)	1142600.2	621072.2	Absent
B-4003(DH)	1142599.9	620927.1	Absent
B-4004	1142459.7	621046.6	Present
B-4005	1142715.0	620948.7	Absent
B-4006	1142719.6	621076.4	Absent
B-4007	1142426.2	621125.3	Present
B-4008	1142424.2	620973.8	Present

Table 2.4-253 (Sheet 4 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
B-4009	1142486.1	621156.9	Present
B-4010	1142667.6	621249.0	Present
B-4011	1142773.1	621236.4	Absent
B-4013 (c)	1142842.7	621020.3	Absent
B-4014	1142832.0	620950.2	Present
B-4015	1142773.0	621115.2	Absent
B-4016	1142996.4	621112.9	Absent
B-4017	1143034.8	620949.9	Present
B-4018	1142735.5	621315.5	Present
B-4019	1142975.9	621371.4	Present
B-4020	1142969.4	621280.0	Present
B-4020A	1142973.7	621280.3	Present
B-4021	1143092.6	621247.4	Present
B-4022	1143081.3	621073.5	Present
B-4023	1143062.4	620879.8	Absent
B-4024	1142904.8	620601.8	Present
B-4025	1142510.0	620625.0	Present
B-4026	1142330.2	620597.7	Present
B-4027	1142180.1	620633.5	Present
B-4028	1141984.2	620587.8	Present
B-4029	1141874.9	620700.0	Absent
B-4030	1141676.7	620698.5	Absent
B-4031	1141399.8	620975.0	Absent
B-4032	1141118.5	620794.6	NE
B-4032A	1141123.7	620794.7	Present
B-4033	1141398.1	620348.8	Present
B-4034	1141375.7	620795.4	Absent
B-4035	1142729.1	620876.3	Present
B-4036	1142457.2	620876.3	Present
B-5001	1146177.1	621807.7	Absent
B-5002	1146339.8	621808.3	Absent
B-5003	1146386.6	621574.7	Absent
B-5004	1146547.8	621568.4	Present
B-6002	1144134.1	619626.9	NE
B-6003	1143925.0	619422.8	Absent
B-6004	1143718.2	619473.3	Absent
B-6005	1143718.0	619873.8	Absent
B-6006	1143069.8	620301.8	NE

Table 2.4-253 (Sheet 5 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
B-6007	1142730.7	620301.8	NE
B-6008	1145443.8	622676.4	Absent
B-6009	1144773.7	621748.2	Present
B-6010	1143893.3	621059.2	Present
B-6011	1144557.9	621261.7	Present
B-6012	1144256.7	620480.5	Present
B-6013	1143169.5	617234.9	NE
B-6014	1143168.2	618281.5	NE
B-6015	1143166.3	619317.9	NE
B-6018	1142909.3	618366.6	NE
B-6019	1142132.7	618344.5	NE
B-6020	1142634.0	619555.9	Present
B-6021	1142185.7	619103.4	Present
B-6022	1142224.8	620040.3	Present
B-6023	1141553.1	619177.9	NE
B-6024	1141545.9	619997.7	NE
B-6025	1140518.7	619189.7	NE
B-6026	1140537.7	619900.2	NE
B-6027	1145779.4	626145.1	Absent[1]
B-6028	1145611.4	626062.4	Absent[1]
B-6029	1147771.7	623966.6	Absent[1]
B-6030	1147588.1	624222.6	Absent[1]
ESP Boring Data			
B-1001	1142661.92	620220.42	Present
B-1002	1142998.52	620985.47	Absent
B-1003	1142974.36	621889.85	Present
B-1004	1142985.41	620131.44	Present
B-1005	1143991.57	620155.35	Present
B-1006	1143810.26	621342.9	Absent
B-1007	1142662.29	621120.13	Present
B-1008	1142670.93	621996.15	Present
B-1009	1141000.54	620361.26	Absent
B-1010	1141000.12	621279.68	Absent
B-1011	1143741.13	622378.01	Present
B-1013	1140976.08	622272.5	Absent
Observation Well Data			
OW-1006	1,143,817.85	619,179.75	Present
OW-1008	1,142,347.94	619,306.69	Present

Table 2.4-253 (Sheet 6 of 6)
Presence of Utley Limestone in the VEGP ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Limestone
	Northing	Easting	
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:
[1] Surface elevation of boring is below the elevation of the Utley Limestone.
NE = Not encountered, indicating that the boring terminated in the Barnwell sands.
COL boring data are provided in Appendix 2.5C
ESP Boring data are provided in Appendix 2.5A
OW-series data are provided in Appendix 2.4A

Table 2.4-254
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 2.4-255 (Sheet 1 of 8)
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											
10-Feb-74		166.6											
23-Mar-74		168.1											

Table 2.4-255 (Sheet 2 of 8)
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
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		
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							

Table 2.4-255 (Sheet 3 of 8)
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
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
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

Table 2.4-255 (Sheet 4 of 8)
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
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
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

Table 2.4-255 (Sheet 5 of 8)
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
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
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

Table 2.4-255 (Sheet 6 of 8)
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-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
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

Table 2.4-255 (Sheet 7 of 8)
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
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
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

Table 2.4-255 (Sheet 8 of 8)
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-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				
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									
13-Dec-06	154.03	148.78	157.37	159.50	163.19	158.01	155.21	158.40	152.62	154.63	154.01	157.79	155.75
25-Jan-07	154.00	148.57	157.24	159.25	162.95	158.77	155.06	158.00	152.63	154.51	153.96	157.77	155.63
14-Feb-07	153.97	148.89	157.19	159.30	162.98	157.67	155.10	157.96	152.65	154.33	153.68	157.48	155.41
1-Mar-07	153.93	148.51	157.67	159.25	163.09	157.69	155.09	158.17	152.62	154.35	153.70	157.60	155.55
17-Apr-07	153.75	148.45	156.92	158.94	162.47	157.40	154.89	158.01	152.37	154.28	153.68	157.53	155.30
24-May-07	153.59	148.40	156.80	158.80	162.59	157.31	154.71	158.06	152.30	153.98	153.24	156.95	155.05
7-Jun-07	153.61	148.40	156.79	158.80	162.70	157.27	154.72	158.02	152.32	153.98	153.36	157.19	155.10
16-Jul-07	153.59	148.44	156.75	158.78	162.82	157.29	154.69	157.93	152.3	153.9	153.32	157.05	155.05

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 2.4-256
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 2.4-255.

Table 2.4-257 (Sheet 1 of 2)
Radionuclide Concentrations in the AP1000 Effluent Holdup Tanks

Radionuclide	Design Basis Reactor Coolant Activity[a] (μCi/g)	Reactor Coolant Concentrations[b] ($\mu Ci/cm^3$)	Effluent Holdup Tank Concentrations[c] ($\mu Ci/cm^3$)
H-3	-	1.00E+00	1.01E+00
Cr-51	1.30E-03	1.30E-03	1.31E-03
Mn-54	6.70E-04	6.70E-04	6.77E-04
Mn-56	1.70E-01	1.70E-01	1.72E-01
Fe-55	5.00E-04	5.00E-04	5.05E-04
Fe-59	1.30E-04	1.30E-04	1.31E-04
Co-58	1.90E-03	1.90E-03	1.92E-03
Co-60	2.20E-04	2.20E-04	2.22E-04
Br-83	3.20E-02	1.54E-02	1.55E-02
Br-84	1.70E-02	8.16E-03	8.24E-03
Br-85	2.00E-03	9.60E-04	9.70E-04
Rb-88	1.50E+00	7.20E-01	7.27E-01
Rb-89	6.90E-02	3.31E-02	3.35E-02
Sr-89	1.10E-03	5.28E-04	5.33E-04
Sr-90	4.90E-05	2.35E-05	2.38E-05
Sr-91	1.70E-03	8.16E-04	8.24E-04
Sr-92	4.10E-04	1.97E-04	1.99E-04
Y-90	1.30E-05	6.24E-06	6.30E-06
Y-91m	9.20E-04	4.42E-04	4.46E-04
Y-91	1.40E-04	6.72E-05	6.79E-05
Y-92	3.40E-04	1.63E-04	1.65E-04
Y-93	1.10E-04	5.28E-05	5.33E-05
Nb-95	1.60E-04	7.68E-05	7.76E-05
Zr-95	1.60E-04	7.68E-05	7.76E-05
Mo-99	2.10E-01	1.01E-01	1.02E-01
Tc-99m	2.00E-01	9.60E-02	9.70E-02
Ru-103	1.40E-04	6.72E-05	6.79E-05
Rh-103m	1.40E-04	6.72E-05	6.79E-05
Rh-106	4.50E-05	2.16E-05	2.18E-05
Ag-110m	4.00E-04	1.92E-04	1.94E-04
Te-127m	7.60E-04	3.65E-04	3.68E-04
Te-129m	2.60E-03	1.25E-03	1.26E-03
Te-129	3.80E-03	1.82E-03	1.84E-03
Te-131m	6.70E-03	3.22E-03	3.25E-03
Te-131	4.30E-03	2.06E-03	2.08E-03
Te-132	7.90E-02	3.79E-02	3.83E-02
Te-134	1.10E-02	5.28E-03	5.33E-03

Table 2.4-257 (Sheet 2 of 2) Radionuclide Concentrations in the AP1000 Effluent Holdup Tanks

Radionuclide	Design Basis Reactor Coolant Activity[a] (μCi/g)	Reactor Coolant Concentrations[b] (μCi/cm^3)	Effluent Holdup Tank Concentrations[c] (μCi/cm^3)
I-129	1.50E-08	7.20E-09	7.27E-09
I-130	1.10E-02	5.28E-03	5.33E-03
I-131	7.10E-01	3.41E-01	3.44E-01
I-132	9.40E-01	4.51E-01	4.56E-01
I-133	1.30E+00	6.24E-01	6.30E-01
I-134	2.20E-01	1.06E-01	1.07E-01
I-135	7.80E-01	3.74E-01	3.78E-01
Cs-134	6.90E-01	3.31E-01	3.35E-01
Cs-136	1.00E+00	4.80E-01	4.85E-01
Cs-137	5.00E-01	2.40E-01	2.42E-01
Cs-138	3.70E-01	1.78E-01	1.79E-01
Ba-137m	4.70E-01	2.26E-01	2.28E-01
Ba-140	1.00E-03	4.80E-04	4.85E-04
La-140	3.10E-04	1.49E-04	1.50E-04
Ce-141	1.60E-04	7.68E-05	7.76E-05
Ce-143	1.40E-04	6.72E-05	6.79E-05
Pr-143	1.50E-04	7.20E-05	7.27E-05
Ce-144	1.20E-04	5.76E-05	5.82E-05
Pr-144	1.20E-04	5.76E-05	5.82E-05

a Values from AP1000 DCD Table 11.1-2.
b For tritium (H-3) a coolant concentration of 1.0 μCi/g is used; corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) are taken directly from the AP1000 DCD, Table 11.1-2; and other radionuclides are based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25. The density of all liquids is assumed to be 1 g/cm^3.
c Values are 101% of the reactor coolant concentrations.

Table 2.4-258 (Sheet 1 of 2)
Water Table Aquifer Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[a] (μCi/cm^3)	Half-life[b] (days)	Decay Rate[c] (days^{-1})	ECL[d] (μCi/cm^3)	Groundwater Concentration[e] (μCi/cm^3)	Groundwater Concentration / ECL
H-3	1.01E+00	4.51E+03	1.54E-04	1.00E-03	6.93E-01	6.93E+02
Cr-51	1.31E-03	2.77E+01	2.50E-02	5.00E-04	3.33E-29	6.66E-27
Mn-54	6.77E-04	3.13E+02	2.21E-03	3.00E-05	3.00E-06	1.00E-01
Mn-56	1.72E-01	1.07E-01	6.48E+00	7.00E-05	0.00E+00	0.00E+00
Fe-55	5.05E-04	9.86E+02	7.03E-04	1.00E-04	9.04E-05	9.04E-01
Fe-59	1.31E-04	4.45E+01	1.56E-02	1.00E-05	3.65E-20	3.56E-16
Co-58	1.92E-03	7.08E+01	9.79E-03	2.00E-05	7.56E-13	3.78E-09
Co-60	2.22E-04	1.93E+03	3.59E-04	3.00E-06	9.22E-05	3.07E+01
Br-83	1.55E-02	9.96E-02	6.96E+00	9.00E-04	0.00E+00	0.00E+00
Br-84	8.24E-03	2.21E-02	3.14E+01	4.00E-04	0.00E+00	0.00E+00
Br-85	9.70E-04	2.01E-03	3.44E+02	1.00E+00	0.00E+00	0.00E+00
Rb-88	7.27E-01	1.24E-02	5.59E+01	4.00E-04	0.00E+00	0.00E+00
Rb-89	3.35E-02	1.06E-02	6.54E+01	9.00E-04	0.00E+00	0.00E+00
Sr-89	5.33E-04	5.05E+01	1.37E-02	8.00E-06	1.38E-18	1.72E-13
Sr-90	2.38E-05	1.06E+04	6.54E-05	5.00E-07	2.03E-05	4.06E+01
Sr-91	8.24E-04	3.96E-01	1.75E+00	2.00E-05	0.00E+00	0.00E+00
Sr-92	1.99E-04	1.13E-01	6.16E+00	4.00E-05	0.00E+00	0.00E+00
Y-90	6.30E-06	2.67E+00	2.60E-01	7.00E-06	7.80E-282	1.11E-276
Y-91m	4.46E-04	3.45E-02	2.01E+01	2.00E-03	0.00E+00	0.00E+00
Y-91	6.79E-05	5.85E+01	1.18E-02	8.00E-06	1.73E-17	2.17E-12
Y-92	1.65E-04	1.48E-01	4.68E+00	4.00E-05	0.00E+00	0.00E+00
Y-93	5.33E-05	4.21E-01	1.65E+00	2.00E-05	0.00E+00	0.00E+00
Nb-95	7.76E-05	3.52E+01	1.97E-02	3.00E-05	9.15E-26	3.05E-21
Zr-95	7.76E-05	6.40E+01	1.08E-02	2.00E-05	2.40E-16	1.20E-11
Mo-99	1.02E-01	2.75E+00	2.52E-01	2.00E-05	1.34E-269	6.71E-265
Tc-99m	9.70E-02	2.51E-01	2.76E+00	1.00E-03	0.00E+00	0.00E+00
Ru-103	6.79E-05	3.93E+01	1.76E-02	3.00E-05	1.22E-23	4.07E-19
Rh-103m	6.79E-05	3.90E-02	1.78E+01	6.00E-03	0.00E+00	0.00E+00
Rh-106	2.18E-05	4.63E-04	1.50E+03	NA[f]	0.00E+00	
Ag-110m	1.94E-04	2.50E+02	2.77E-03	6.00E-06	2.19E-07	3.66E-02
Te-127m	3.68E-04	1.09E+02	6.36E-03	9.00E-06	6.42E-11	7.13.E-06
Te-129m	1.26E-03	3.36E+01	2.06E-02	7.00E-06	1.506E-25	2.14E-20
Te-129	1.84E-03	4.83E-02	1.44E+01	4.00E-04	0.00E+00	0.00E+00
Te-131m	3.25E-03	1.25E+00	5.55E-01	8.00E-06	0.00E+00	0.00E+00
Te-131	2.08E-03	1.74E-02	3.98E+01	8.00E-05	0.00E+00	0.00E+00
Te-132	3.83E-02	3.26E+00	2.13E-01	9.00E-06	4.07E-228	4.53E-223
Te-134	5.33E-03	2.90E-02	2.39E+01	3.00E-04	0.00E+00	0.00E+00
I-129	7.27E-09	5.73E+09	1.21E-10	2.00E-07	7.27E-09	3.63E-02
I-130	5.33E-03	5.15E-01	1.35E+00	2.00E-05	0.00E+00	0.00E+00
I-131	3.44E-01	8.04E+00	8.62E-02	1.00E-06	8.14E-93	8.14E-87
I-132	4.56E-01	9.58E-02	7.24E+00	1.00E-04	0.00E+00	0.00E+00

Table 2.4-258 (Sheet 2 of 2)
Water Table Aquifer Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[a] (μCi/cm^3)	Half-life[b] (days)	Decay Rate[c] (days^{-1})	ECL[d] (μCi/cm^3)	Groundwater Concentration[e] (μCi/cm^3)	Groundwater Concentration / ECL
I-133	6.30E-01	8.67E-01	7.99E-01	7.00E-06	0.00E+00	0.00E+00
I-134	1.07E-01	3.65E-02	1.90E+01	4.00E-04	0.00E+00	0.00E+00
I-135	3.78E-01	2.75E-01	2.52E+00	3.00E-05	0.00E+00	0.00E+00
Cs-134	3.35E-01	7.53E+02	9.21E-04	9.00E-07	3.52E-02	3.91E+04
Cs-136	4.85E-01	1.31E+01	5.29E-02	6.00E-06	2.83E-57	4.71E-52
Cs-137	2.42E-01	1.10E+04	6.30E-05	1.00E-06	2.07E-01	2.07E+05
Cs-138	1.79E-01	2.24E-02	3.09E+01	4.00E-04	0.00E+00	0.00E+00
Ba-137m	2.28E-01	1.81E-03	3.84E+02	NA[6]	0.00E+00	
Ba-140	4.85E-04	1.27E+01	5.46E-02	8.00E-06	4.79E-62	5.98E-57
La-140	1.50E-04	1.68E+00	4.13E-01	9.00E-06	0.00E+00	0.00E+00
Ce-141	7.76E-05	3.25E+01	2.13E-02	3.00E-05	1.67E-27	5.57E-23
Ce-143	6.79E-05	1.38E+00	5.02E-01	2.00E-05	0.00E+00	0.00E+00
Pr-143	7.27E-05	1.36E+01	5.10E-02	2.00E-05	4.59E-59	2.74E-54
Ce-144	5.82E-05	2.84E+02	2.44E-03	3.00E-06	1.48E-07	4.94E-02
Pr-144	5.82E-05	1.20E-02	5.78E+01	6.00E-04	0.00E+00	0.00E+00

a Values from Table 2.4-257.
b Values from NUREG/CR-5512, Table E.1 (Kennedy and Strenge 1992), and U. S. Department of Health Radiological Health Handbook (USDOH 1970) for Sr-92, Rh-106, and Ba-137m.
c Values calculated from Equation 2.4.13-4.
d Values calculated from Equation 2.4.13-4.
e Values calculated from Equation 2.4.13-5 for a travel time of 6.7 years.
f ECL is not available.

Highlighted values indicate groundwater concentration/ECL ratios exceeding 0.01

Table 2.4-259
Results of kd Analysis

Soil Sample	K_d Value (mL/g)		
	Co	Sr	Cs
Samples From Potential Borrow Sources Areas			
A-10(a)	8.1	13.2	56.2
C-7	3.9	9.0	14.8
D-10	1.7	7.8	9.9
E-7	10.1	25.7	19.9
E-12	15.3	51.7	10.7
G-9	7.9	9.8	> 25.5
J-11	13.5	9.2	> 47.4
K-10	15.2	10.0	19.3
L-7	1.7	11.4	18.8
M-5	7.3	9.3	16.8
N-3	5.8	10.7	7.8
P-8	6.5	7.0	5.3
Q-7	3.2	9.3	14.6
H-6	1.4	6.0	3.5
S-9	3.0	8.6	19.3
R-8	2.1	10.5	13.5
Samples From Barnwell Formation			
B-1003V-55-65	10.9	17.4	> 30.1
B-1003V-65-75	3.9	15.0	22.7
B-1003V-75-82	21.3	14.4	33.2

Source: Kaplan and Millings 2006

Table 2.4-260
Water Table Aquifer Results of Transport Analysis Considering Radioactive Decay and Adsorption

Radionuclide	Effluent Holdup Tank Conc[1] (mCi/cm^3)	Half-life[2] (days)	Decay Rate[3] (years^{-1})	ECL[4] (μCi/cm^3)	Backfill			Barnwell Group			Mallard Pond Material			Total Travel Time[7] (years)	Ground Water Conc[8] (μCi/cm^3)	Ground Water Conc/ECL
					Distribution Coefficient (cm^3/g)	Retard Factor[5]	Travel Time[6] (years)	Distribution Coefficient (cm^3/g)	Retard Factor[5]	Travel Time[6] (years)	Distribution Coefficient (cm^3/g)	Retard Factor[5]	Travel Time[6] (years)			
H-3	1.01E+00	4.51E+03	5.61E-02	1.00E-03	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	6.93E-01	6.93E+02
Mn-54	6.77E-04	3.13E+02	8.09E-01	3.00E-05	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	3.00E-06	1.00E-01
Fe-55	5.05E-04	9.86E+02	2.57E-01	1.00E-04	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	9.04E-05	9.04E-01
Co-60	2.22E-04	1.93E+03	1.31E-01	3.00E-06	1.4	7.3	17.62	3.9	18.8	60.06	3.9	18.8	20.65	98.33	5.56E-10	1.85E-04
Sr-90	2.38E-05	1.06E+04	2.39E-02	5.00E-07	6.0	28.2	67.62	14.4	66.6	213.15	14.4	66.6	73.27	354.04	5.06E-09	1.01E-02
Ag-110m	1.94E-04	2.50E+02	1.01E+00	6.00E-06	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	2.19E-07	3.66E-02
I-129	7.27E-09	5.73E+09	4.42E-08	2.00E-07	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	7.27E-09	3.63E-02
Cs-134	3.35E-01	7.53E+02	3.36E-01	9.00E-07	3.5	16.9	40.45	22.7	104.4	334.16	22.7	104.4	114.87	489.48	1.13E-72	1.25E-66
Cs-137	2.42E-01	1.10E+04	2.30E-02	1.00E-06	3.5	16.9	40.45	22.7	104.4	334.16	22.7	104.4	114.87	489.48	3.10E-06	3.10E+00
Ce-144	5.82E-05	2.84E+02	8.91E-01	3.00E-06	0.0	1.0	2.40	0.0	1.0	3.20	0.0	1.0	1.10	6.70	1.48E-07	4.94E+02

1 Values from Table 2.4-257.
2 Values from NUREG/CR-5512, Table E.1 (Kennedy and Strenge 1992).
3 Values calculated from Equation 2.4.13-4.
4 Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.
5 Values calculated from Equation 2.4.13-2.
6 Radionuclide travel time = retardation factor x groundwater travel time.
7 Total travel time = backfill travel time + Barnwell Group travel time + permeable Mallard Pond material travel time.
8 Values calculated from Equation 2.4.13-5.
Highlighted values indicate groundwater concentration/ECL ratios exceed 0.01.

Table 2.4-261
Results of Transport Analysis Considering Radioactive Decay, Adsorption, and Dilution

Water Table Aquifer

Radionuclide	ECL[1]	Groundwater Concentration[2] (μCi/cm^3)	Surface Water Concentration[3] (μCi/cm^3)	Surface Water Concentration / ECL
H-3	1.00E-03	6.93E-01	5.76E-05	5.76E-02
Mn-54	3.00E-05	3.00E-06	2.49E-10	8.31E-06
Fe-55	1.00E-04	9.04E-05	7.52E-09	7.52E-05
Sr-90	4.00E-05	5.06E-09	4.21E-13	1.05E-08
Ag-110m	6.00E-06	2.19E-07	1.82E-11	3.04E-06
I-129	2.00E-07	7.27E-09	6.04E-13	3.02E-06
Cs-137	1.00E-06	3.10E-06	2.58E-10	2.58E-04
Ce-144	3.00E-06	1.48E-07	1.23E-11	4.11E-06

1 Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.
2 Values from Table 2.4-260 for Water Table Aquifer.
3 Surface water concentration = groundwater concentration x dilution factor. Dilution factor = 8.3E-05 for Water Table aquifer.

Table 2.4-262 (Sheet 1 of 2)
Tertiary Aquifer Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[a] (μCi/cm^3)	Half-life[b] (days)	Decay Rate[c] (days^{-1})	ECL[d] (μCi/cm^3)	Groundwater Concentration[e] (μCi/cm^3)	Groundwater Concentration / ECL
H-3	1.01E+00	4.51E+03	1.54E-04	1.00E-03	1.49E-28	1.49E-25
Cr-51	1.31E-03	2.77E+01	2.50E-02	5.00E-04	0.00E+00	0.00E+00
Mn-54	6.77E-04	3.13E+02	2.21E-03	3.00E-05	0.00E+00	0.00E+00
Mn-56	1.72E-01	1.07E-01	6.48E+00	7.00E-05	0.00E+00	0.00E+00
Fe-55	5.05E-04	9.86E+02	7.03E-04	1.00E-04	2.53E-131	2.53E-127
Fe-59	1.31E-04	4.45E+01	1.56E-02	1.00E-05	0.00E+00	0.00E+00
Co-58	1.92E-03	7.08E+01	9.79E-03	2.00E-05	0.00E+00	0.00E+00
Co-60	2.22E-04	1.93E+03	3.59E-04	3.00E-06	2.05E-69	6.82E-64
Br-83	1.55E-02	9.96E-02	6.96E+00	9.00E-04	0.00E+00	0.00E+00
Br-84	8.24E-03	2.21E-02	3.14E+01	4.00E-04	0.00E+00	0.00E+00
Br-85	9.70E-04	2.01E-03	3.44E+02	1.00E+00	0.00E+00	0.00E+00
Rb-88	7.27E-01	1.24E-02	5.59E+01	4.00E-04	0.00E+00	0.00E+00
Rb-89	3.35E-02	1.06E-02	6.54E+01	9.00E-04	0.00E+00	0.00E+00
Sr-89	5.33E-04	5.05E+01	1.37E-02	8.00E-06	0.00E+00	0.00E+00
Sr-90	2.38E-05	1.06E+04	6.54E-05	5.00E-07	3.43E-17	6.86E-11
Sr-91	8.24E-04	3.96E-01	1.75E+00	2.00E-05	0.00E+00	0.00E+00
Sr-92	1.99E-04	1.13E-01	6.16E+00	4.00E-05	0.00E+00	0.00E+00
Y-90	6.30E-06	2.67E+00	2.60E-01	7.00E-06	0.00E+00	0.00E+00
Y-91m	4.46E-04	3.45E-02	2.01E+01	2.00E-03	0.00E+00	0.00E+00
Y-91	6.79E-05	5.85E+01	1.18E-02	8.00E-06	0.00E+00	0.00E+00
Y-92	1.65E-04	1.48E-01	4.68E+00	4.00E-05	0.00E+00	0.00E+00
Y-93	5.33E-05	4.21E-01	1.65E+00	2.00E-05	0.00E+00	0.00E+00
Nb-95	7.76E-05	3.52E+01	1.97E-02	3.00E-05	0.00E+00	0.00E+00
Zr-95	7.76E-05	6.40E+01	1.08E-02	2.00E-05	0.00E+00	0.00E+00
Mo-99	1.02E-01	2.75E+00	2.52E-01	2.00E-05	0.00E+00	0.00E+00
Tc-99m	9.70E-02	2.51E-01	2.76E+00	1.00E-03	0.00E+00	0.00E+00
Ru-103	6.79E-05	3.93E+01	1.76E-02	3.00E-05	0.00E+00	0.00E+00
Rh-103m	6.79E-05	3.90E-02	1.78E+01	6.00E-03	0.00E+00	0.00E+00
Rh-106	2.18E-05	4.63E-04	1.50E+03	NA[f]	0.00E+00	
Ag-110m	1.94E-04	2.50E+02	2.77E-03	6.00E-06	0.00E+00	0.00E+00
Te-127m	3.68E-04	1.09E+02	6.36E-03	9.00E-06	0.00E+00	0.00E+00
Te-129m	1.26E-03	3.36E+01	2.06E-02	7.00E-06	0.00E+00	0.00E+00
Te-129	1.84E-03	4.83E-02	1.44E+01	4.00E-04	0.00E+00	0.00E+00

Table 2.4-262 (Sheet 2 of 2)
Tertiary Aquifer Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[a] (µCi/cm^3)	Half-life[b] (days)	Decay Rate[c] (days^{-1})	ECL[d] (µCi/cm^3)	Groundwater Concentration[e] (µCi/cm^3)	Groundwater Concentration / ECL
Te-131m	3.25E-03	1.25E+00	5.55E-01	8.00E-06	0.00E+00	0.00E+00
Te-131	2.08E-03	1.74E-02	3.98E+01	8.00E-05	0.00E+00	0.00E+00
Te-132	3.83E-02	3.26E+00	2.13E-01	9.00E-06	0.00E+00	0.00E+00
Te-134	5.33E-03	2.90E-02	2.39E+01	3.00E-04	0.00E+00	0.00E+00
I-129	7.27E-09	5.73E+09	1.21E-10	2.00E-07	7.27E-09	3.63E-02
I-130	5.33E-03	5.15E-01	1.35E+00	2.00E-05	0.00E+00	0.00E+00
I-131	3.44E-01	8.04E+00	8.62E-02	1.00E-06	0.00E+00	0.00E+00
I-132	4.56E-01	9.58E-02	7.24E+00	1.00E-04	0.00E+00	0.00E+00
I-133	6.30E-01	8.67E-01	7.99E-01	7.00E-06	0.00E+00	0.00E+00
I-134	1.07E-01	3.65E-02	1.90E+01	4.00E-04	0.00E+00	0.00E+00
I-135	3.78E-01	2.75E-01	2.52E+00	3.00E-05	0.00E+00	0.00E+00
Cs-134	3.35E-01	7.53E+02	9.21E-04	9.00E-07	6.82E-168	7.58E-162
Cs-136	4.85E-01	1.31E+01	5.29E-02	6.00E-06	0.00E+00	0.00E+00
Cs-137	2.42E-01	1.10E+04	6.30E-05	1.00E-06	9.40E-13	9.40E-07
Cs-138	1.79E-01	2.24E-02	3.09E+01	4.00E-04	0.00E+00	0.00E+00
Ba-137m	2.28E-01	1.81E-03	3.84E+02	NA[f]	0.00E+00	
Ba-140	4.85E-04	1.27E+01	5.46E-02	8.00E-06	0.00E+00	0.00E+00
La-140	1.50E-04	1.68E+00	4.13E-01	9.00E-06	0.00E+00	0.00E+00
Ce-141	7.76E-05	3.25E+01	2.13E-02	3.00E-05	0.00E+00	0.00E+00
Ce-143	6.79E-05	1.38E+00	5.02E-01	2.00E-05	0.00E+00	0.00E+00
Pr-143	7.27E-05	1.36E+01	5.10E-02	2.00E-05	0.00E+00	0.00E+00
Ce-144	5.82E-05	2.84E+02	2.44E-03	3.00E-06	0.00E+00	0.00E+00
Pr-144	5.82E-05	1.20E-02	5.78E+01	6.00E-04	0.00E+00	0.00E+00

a Values from Table 2.4-257.
b Values from NUREG/CR-5512, Table E.1 (Kennedy and Strenge 1992), and U. S. Department of Health Radiological Health Handbook (USDOH 1970) for Sr-92, Rh-106, and Ba-137m.
c Values calculated from Equation 2.4.13-4.
d Effluent Concentration Limit (ECLs) from 10 CFR Part 20, Appendix B, Table 2, Column 2
e Values calculated from Equation 2.4.13-5 for a travel time of 1142 years.
f ECL is not available.

Highlighted values indicate groundwater concentration/ECL ratios exceeding 0.01

Table 2.4-263 (Sheet 1 of 2)
Water Table Aquifer Compliance with 10 CFR Part 20

Radionuclide	Water Table Aquifer Concentration / ECL			
	Decay[a]	Decay and Adsorption[b]	Decay, Adsorption, and Dilution[c]	Minimum
H-3	6.93E+02	6.93E+02	5.76E-02	5.76E-02
Cr-51	6.66E-26			6.66E-27
Mn-54	1.00E-01	1.00E-01	8.31E-06	8.31E-06
Mn-56	0.00E+00			0.00E+00
Fe-55	9.04E-01	9.04E-01	7.52E-05	7.52E-05
Fe-59	3.65E-16			3.65E-16
Co-58	3.78E-09			3.78E-09
Co-60	3.07E+01	1.85E-04		1.85E-04
Br-83	0.00E+00			0.00E+00
Br-84	0.00E+00			0.00E+00
Br-85	0.00E+00			0.00E+00
Rb-88	0.00E+00			0.00E+00
Rb-89	0.00E+00			0.00E+00
Sr-89	1.72E-13			1.72E-13
Sr-90	4.06E+01	1.01E-02	1.05E-08	1.05E-08
Sr-91	0.00E+00			0.00E+00
Sr-92	0.00E+00			0.00E+00
Y-90	1.11E-276			1.11E-276
Y-91m	0.00E+00			0.00E-00
Y-91	2.17E-12			2.17E-12
Y-92	0.00E+00			0.00E+00
Y-93	0.00E+00			0.00E+00
Nb-95	3.05E-21			3.05E-21
Zr-95	1.20E-11			1.20E-11
Mo-99	6.71E-265			6.71E-265
Tc-99m	0.00E+00			0.00E+00
Ru-103	4.07E-19			4.07E-19
Rh-103m	0.00E+00			0.00E+00
Rh-106[4]	0.00E+00			0.00E+00
Ag-110m	3.66E-02	3.66E-02	3.04E-06	3.04E-06
Te-127m	7.13E-06			7.13E-05
Te-129m	2.14E-20			2.14E-20
Te-129	0.00E+00			0.00E+00
Te-131m	0.00E+00			0.00E+00
Te-131	0.00E+00			0.00E+00
Te-132	4.53E-223			4.53E-223

Table 2.4-263 (Sheet 2 of 2)
Water Table Aquifer Compliance with 10 CFR Part 20

Radionuclide	Water Table Aquifer Concentration / ECL			
	Decay[a]	Decay and Adsorption[b]	Decay, Adsorption, and Dilution[c]	Minimum
Te-134	0.00E+00			0.00E+00
I-129	3.63E-02	3.63E-02	3.02E-06	3.02E-06
I-130	0.00E+00			0.00E+00
I-131	8.14E-87			8.14E-87
I-132	0.00E+00			0.00E+00
I-133	0.00E+00			0.00E+00
I-134	0.00E+00			0.00E+00
I-135	0.00E+00			0.00E+00
Cs-134	3.91E+04	1.25E-66		1.25E-66
Cs-136	4.71E-52			4.71E-52
Cs-137	2.07E+05	3.10E+00	2.58E-04	2.58E-04
Cs-138	0.00E+00			0.00E+00
Ba-137m[d]	0.00E+00			0.00E+00
Ba-140	5.98E-57			5.98E-57
La-140	0.00E+00			0.00E+00
Ce-141	5.57E-23			5.57E-23
Ce-143	0.00E+00			0.00E+00
Pr-143	2.47E-54			2.47E-54
Ce-144	4.94E-02	4.94E-02	4.11E-06	4.11E-06
Pr-144	0.00E+00			0.00E+00

Sum of Fractions = 0.058

a Table 2.4-258.
b Table 2.4-260.
c Table 2.4-261.
d No ECLs are published for Rh-106 and Ba-137m. However, the half-lives for these radionuclides are short (less than 1 day) and they decay to near zero values. Their ratios have been taken as zero.

Table 2.4-264 (Sheet 1 of 2)
Tertiary Aquifer Compliance with 10 CFR Part 20

Radionuclide	Tertiary Aquifer Concentration / ECL	Minimum
	Decay[a]	
H-3	1.49E-25	1.49E-25
Cr-51	0.00E+00	0.00E+00
Mn-54	0.00E+00	0.00E+00
Mn-56	0.00E+00	0.00E+00
Fe-55	2.53E-127	2.53E-127
Fe-59	0.00E+00	0.00E+00
Co-58	0.00E+00	0.00E+00
Co-60	6.82E-64	6.82E-64
Br-83	0.00E+00	0.00E+00
Br-84	0.00E+00	0.00E+00
Br-85	0.00E+00	0.00E+00
Rb-88	0.00E+00	0.00E+00
Rb-89	0.00E+00	0.00E+00
Sr-89	0.00E+00	0.00E+00
Sr-90	6.86E-11	6.86E-11
Sr-91	0.00E+00	0.00E+00
Sr-92	0.00E+00	0.00E+00
Y-90	0.00E+00	0.00E+00
Y-91m	0.00E+00	0.00E+00
Y-91	0.00E+00	0.00E+00
Y-92	0.00E+00	0.00E+00
Y-93	0.00E+00	0.00E+00
Nb-95	0.00E+00	0.00E+00
Zr-95	0.00E+00	0.00E+00
Mo-99	0.00E+00	0.00E+00
Tc-99m	0.00E+00	0.00E+00
Ru-103	0.00E+00	0.00E+00
Rh-103m	0.00E+00	0.00E+00
Rh-106[b]	0.00E+00	0.00E+00
Ag-110m	0.00E+00	0.00E+00
Te-127m	0.00E+00	0.00E+00
Te-129m	0.00E+00	0.00E+00
Te-129	0.00E+00	0.00E+00
Te-131m	0.00E+00	0.00E+00
Te-131	0.00E+00	0.00E+00
Te-132	0.00E+00	0.00E+00
Te-134	0.00E+00	0.00E+00
I-129	3.63E-02	3.63E-02
I-130	0.00E+00	0.00E+00
I-131	0.00E+00	0.00E+00
I-132	0.00E+00	0.00E+00
I-133	0.00E+00	0.00E+00
I-134	0.00E+00	0.00E+00
I-135	0.00E+00	0.00E+00

Table 2.4-264 (Sheet 2 of 2)
Tertiary Aquifer Compliance with 10 CFR Part 20

Radionuclide	Tertiary Aquifer Concentration / ECL	Minimum
	Decay[a]	
Cs-134	7.58E-162	7.58E-162
Cs-136	0.00E+00	0.00E+00
Cs-137	9.40E-07	9.40E-07
Cs-138	0.00E+00	0.00E+00
Ba-137m[b]	0.00E+00	0.00E+00
Ba-140	0.00E+00	0.00E+00
La-140	0.00E+00	0.00E+00
Ce-141	0.00E+00	0.00E+00
Ce-143	0.00E+00	0.00E+00
Pr-143	0.00E+00	0.00E+00
Ce-144	0.00E+00	0.00E+00
Pr-144	0.00E+00	0.00E+00

	Sum of Fractions =	0.036

a Table 2.4-262.

b No ECLs are published for Rh-106 and Ba-137m. However, the half-lives for these radionuclides are short (less than 1 day) and they decay to near zero values. Their ratios have been taken as zero.

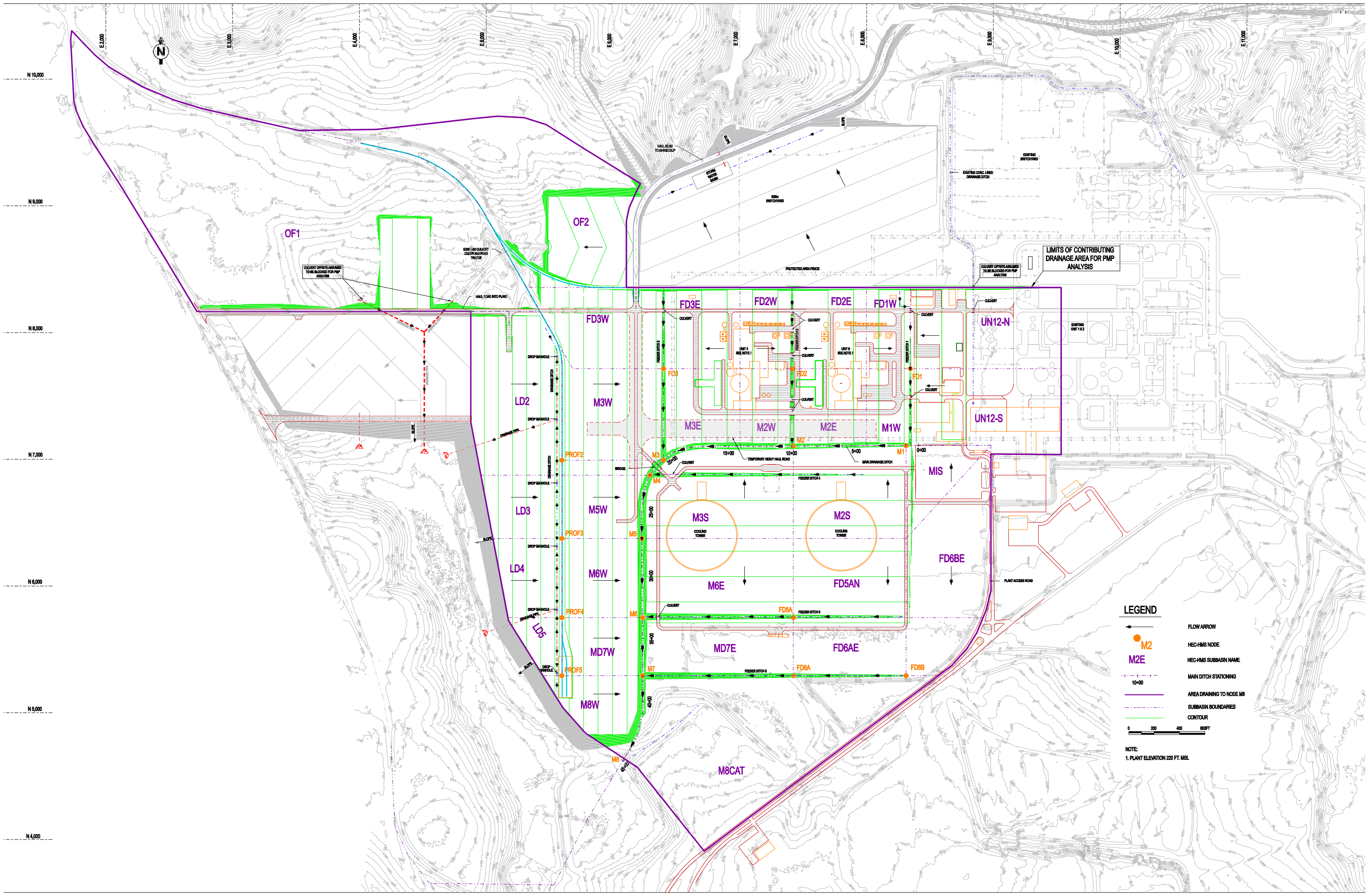


Figure 2.4-201
Site Plan with PMP Drainage Boundaries and Flow Paths

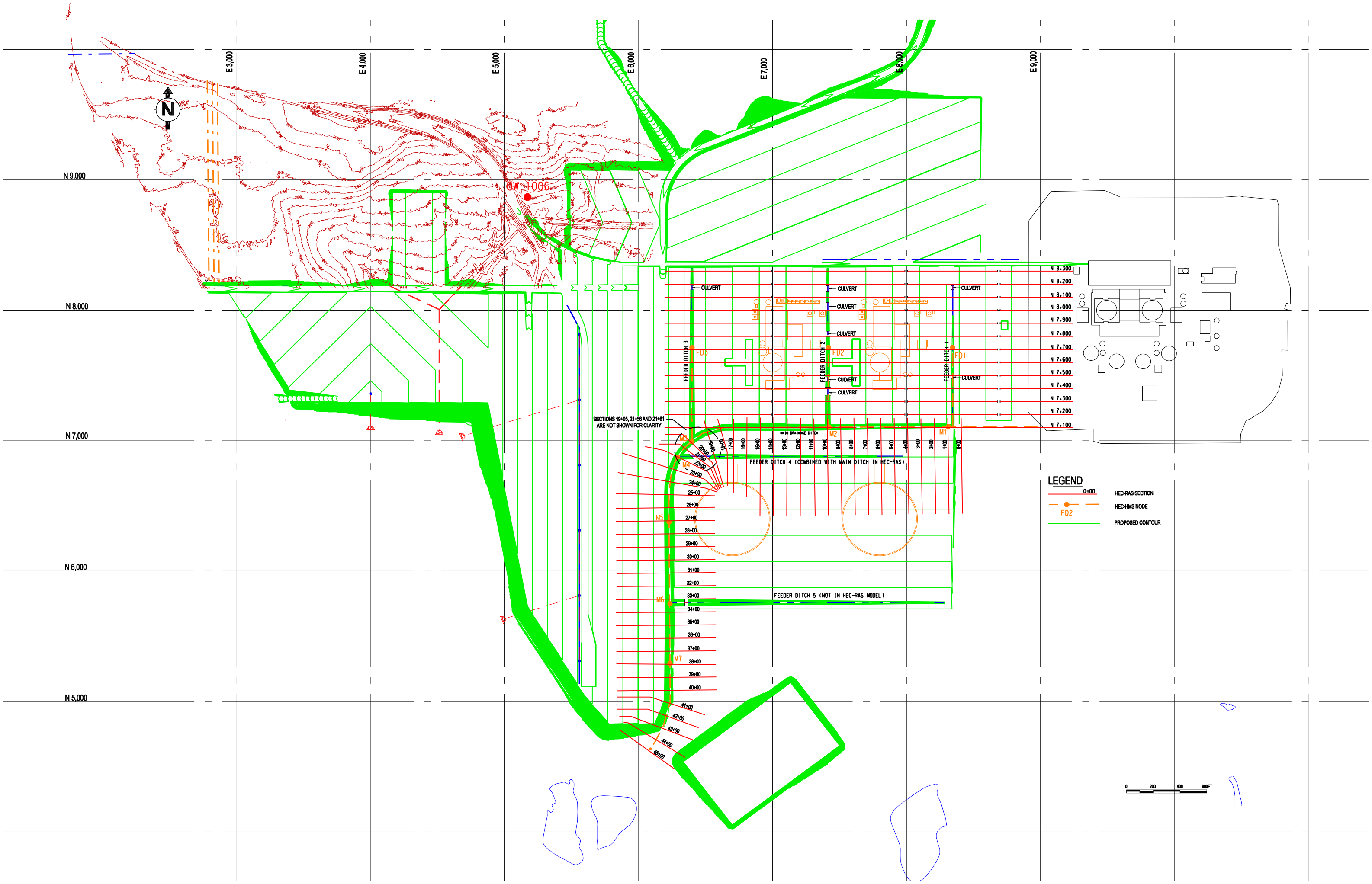


Figure 2.4-201a
Cross-Section Location Map for HEC-RAS Model of Local PMF for Units 3 and 4

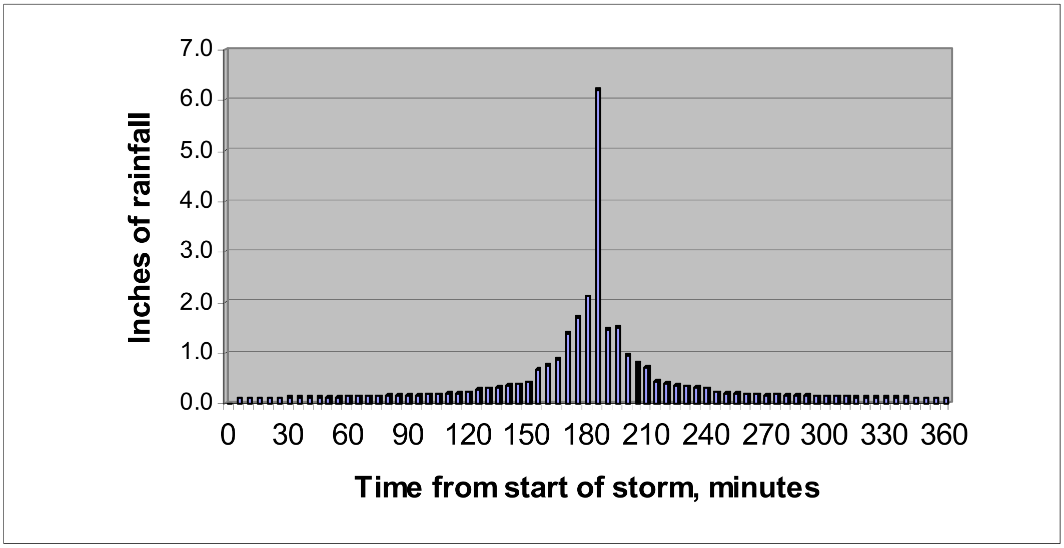


Figure 2.4-202
PMP Hyetograph Determined in Frequency Storm Module of HEC-HMS

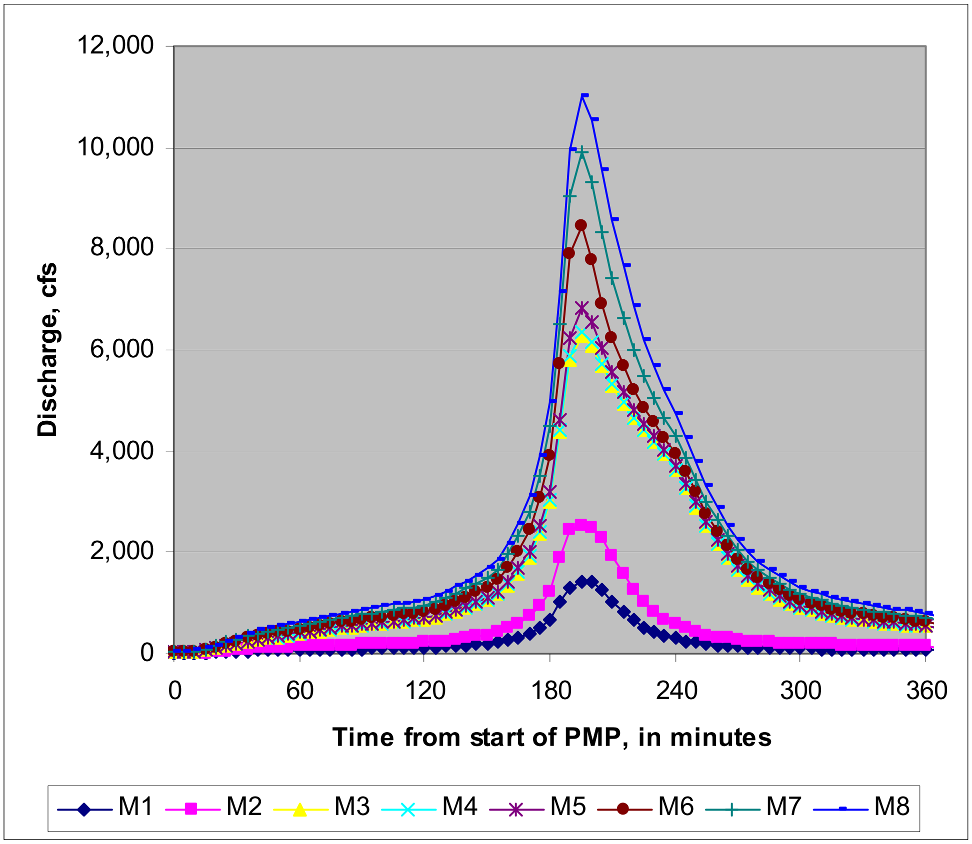
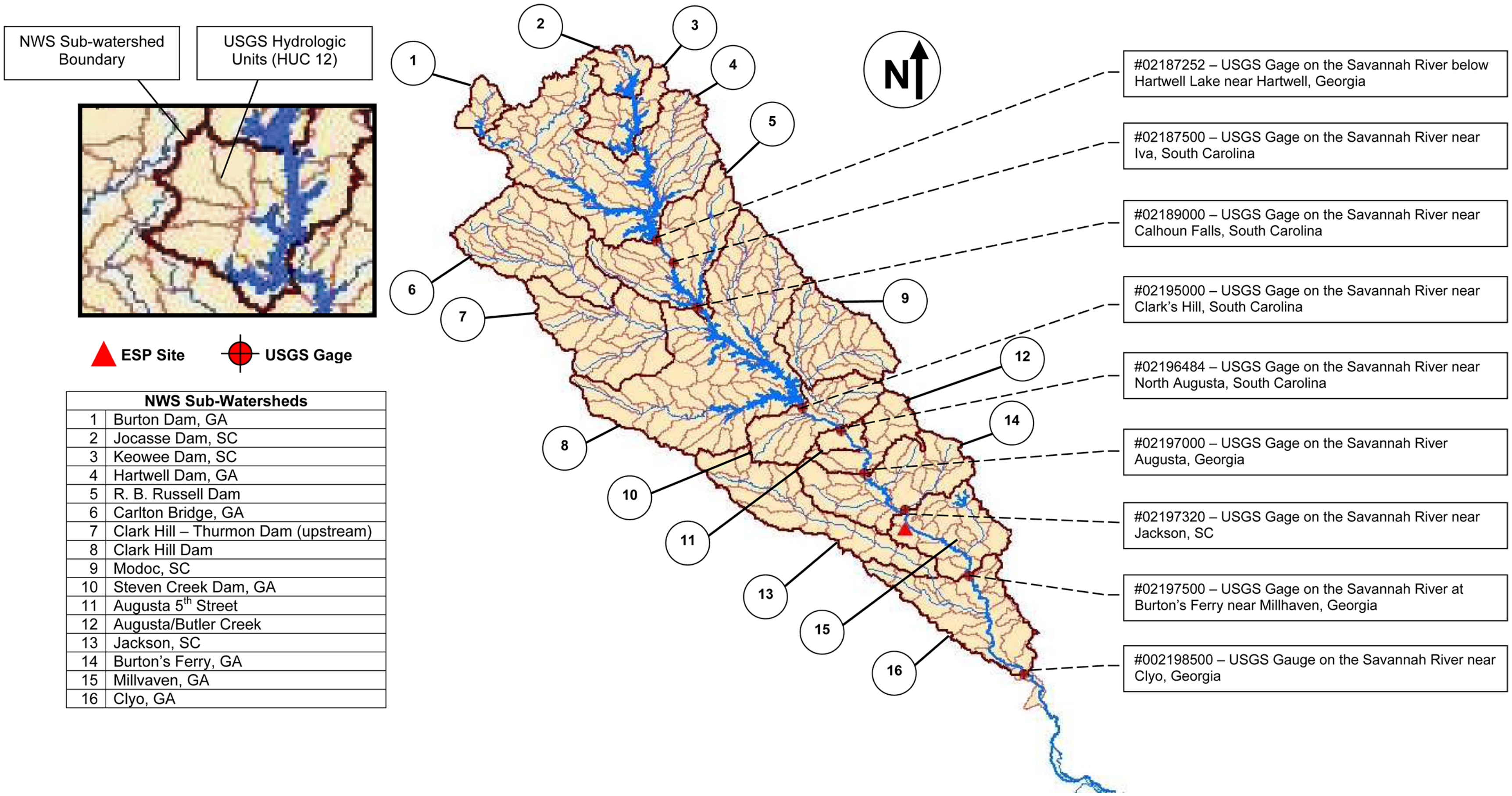


Figure 2.4-203
HEC-HMS PMP Runoff Hydrographs at Points along Main Ditch



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 2.4-204
Savannah River Watershed and HUCs (No Scale)

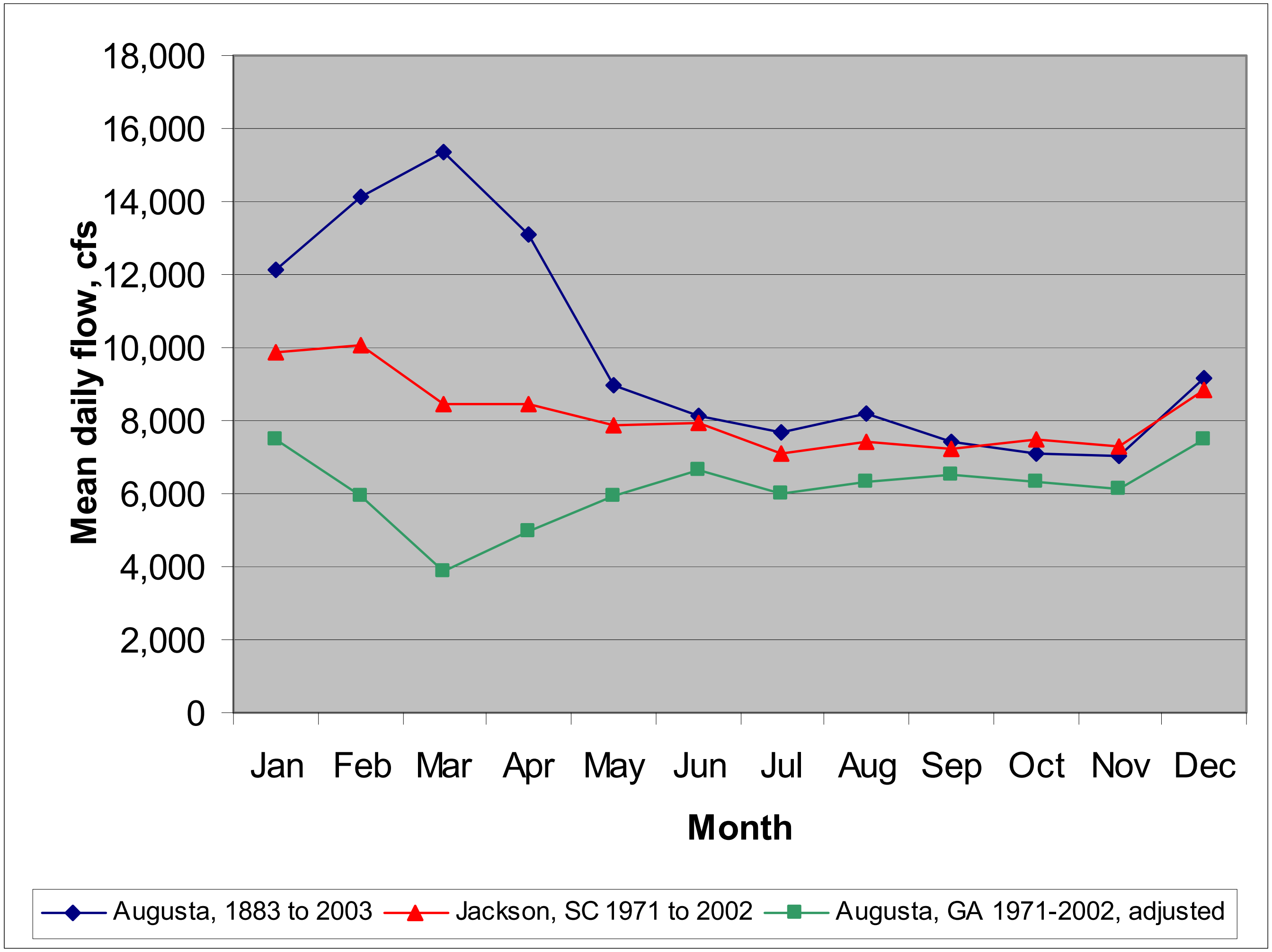
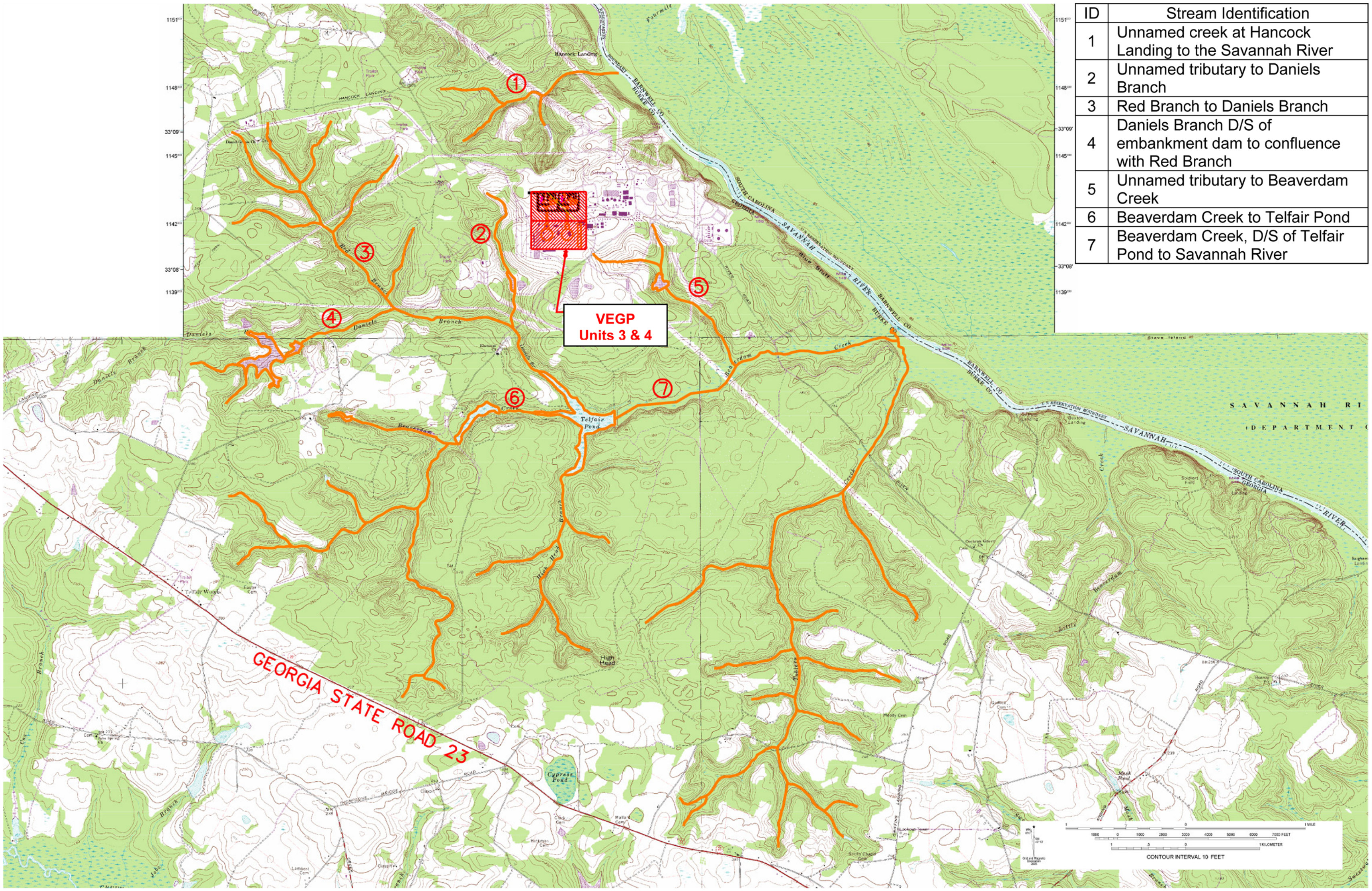
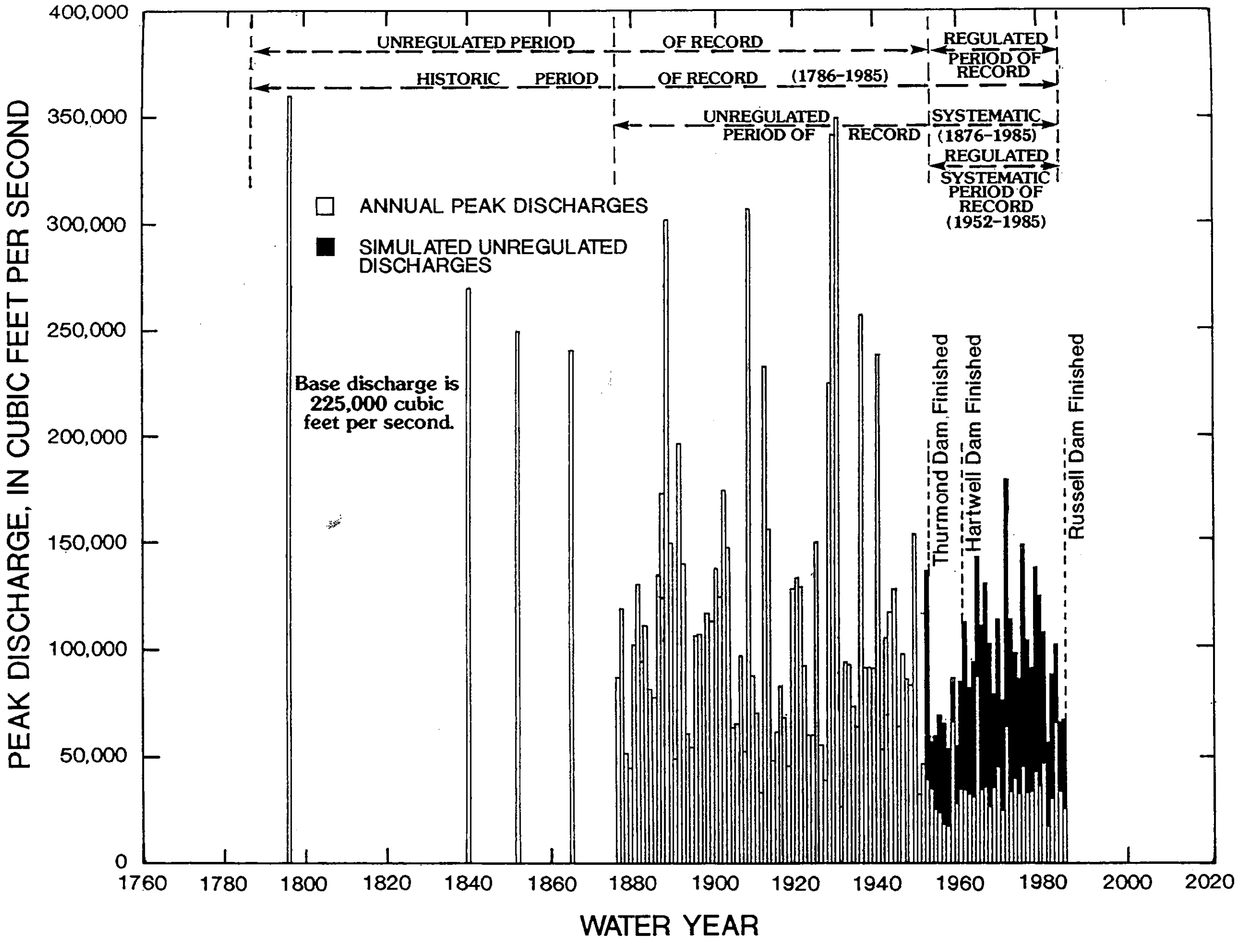


Figure 2.4-205
Mean Daily Discharge for the Year – Selected Gages of the Savannah River



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 2.4-206
Site Drainage



Source: Figure 2 from USGS 1990

Figure 2.4-207
Unregulated and Regulated Peak Discharge Frequency Curves for the Savannah River at Augusta, Georgia (02197000)

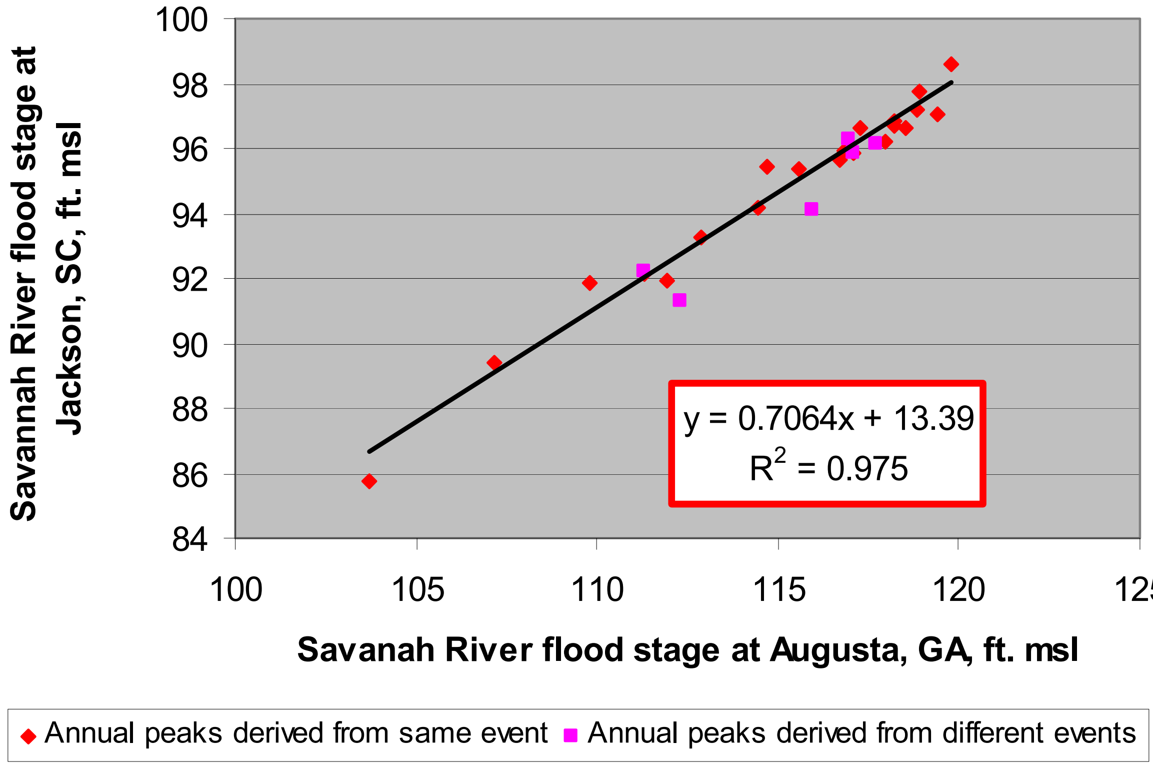
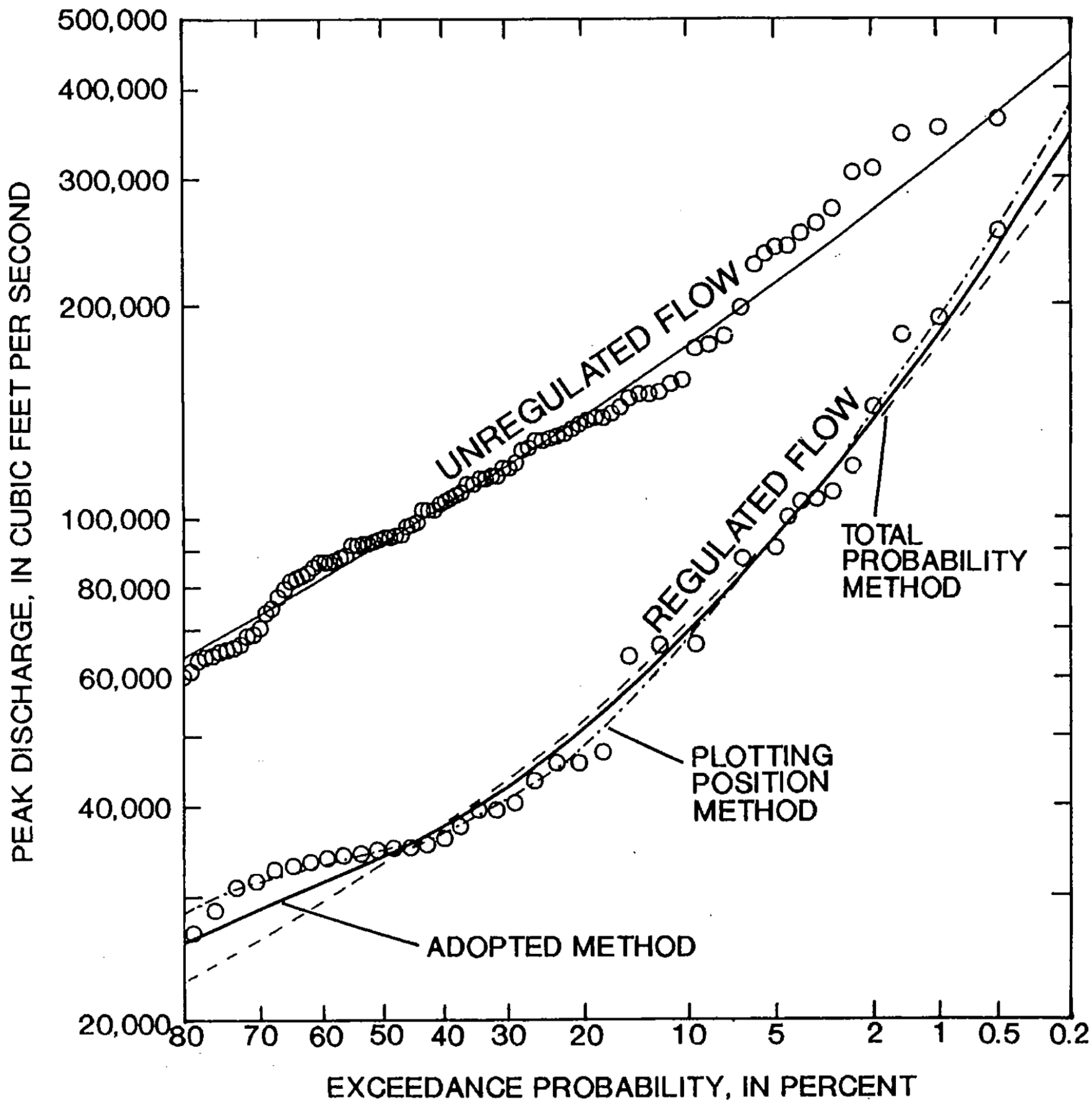


Figure 2.4-208
Correlation of Annual Peak Discharges on the Savannah River at Augusta, Georgia (02197000), and Jackson, South Carolina (2197320), for Years with Annual Peak Derived from Same Storm Event



Source: Figure 35 from USGS 1990

Figure 2.4-209
Unregulated and Regulated Annual Peak Discharge Frequency Curves for the Savannah River at Augusta, Georgia

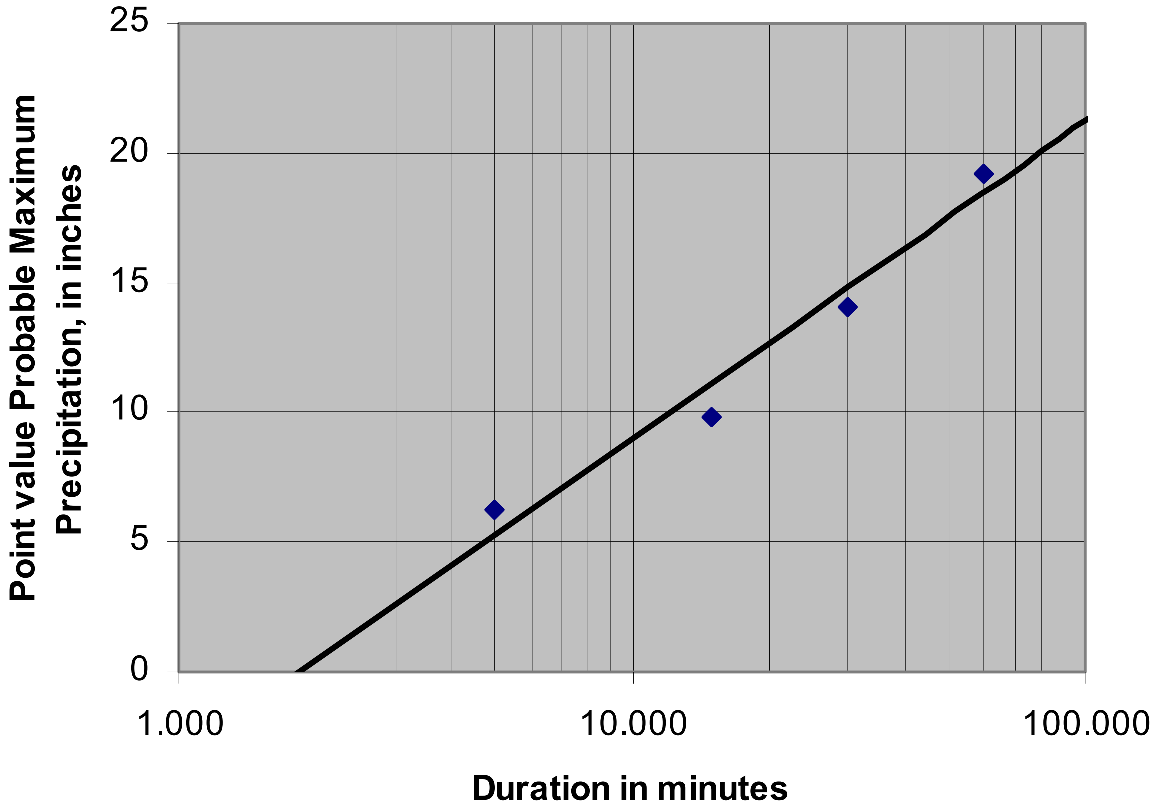


Figure 2.4-210
Probable Maximum Precipitation Values as a Function of Duration for Point Rainfall at VEGP Site

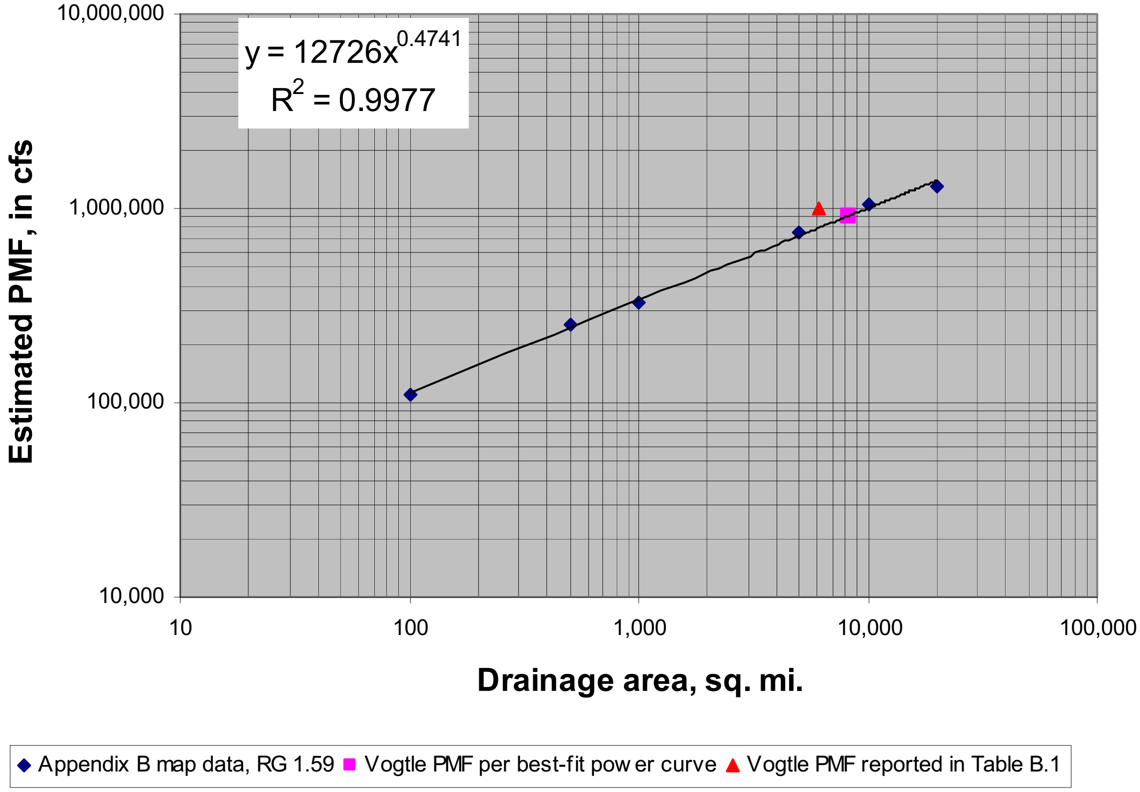


Figure 2.4-211
Area-PMF Plot for VEGP Site per Approximate Method from RG 1.59

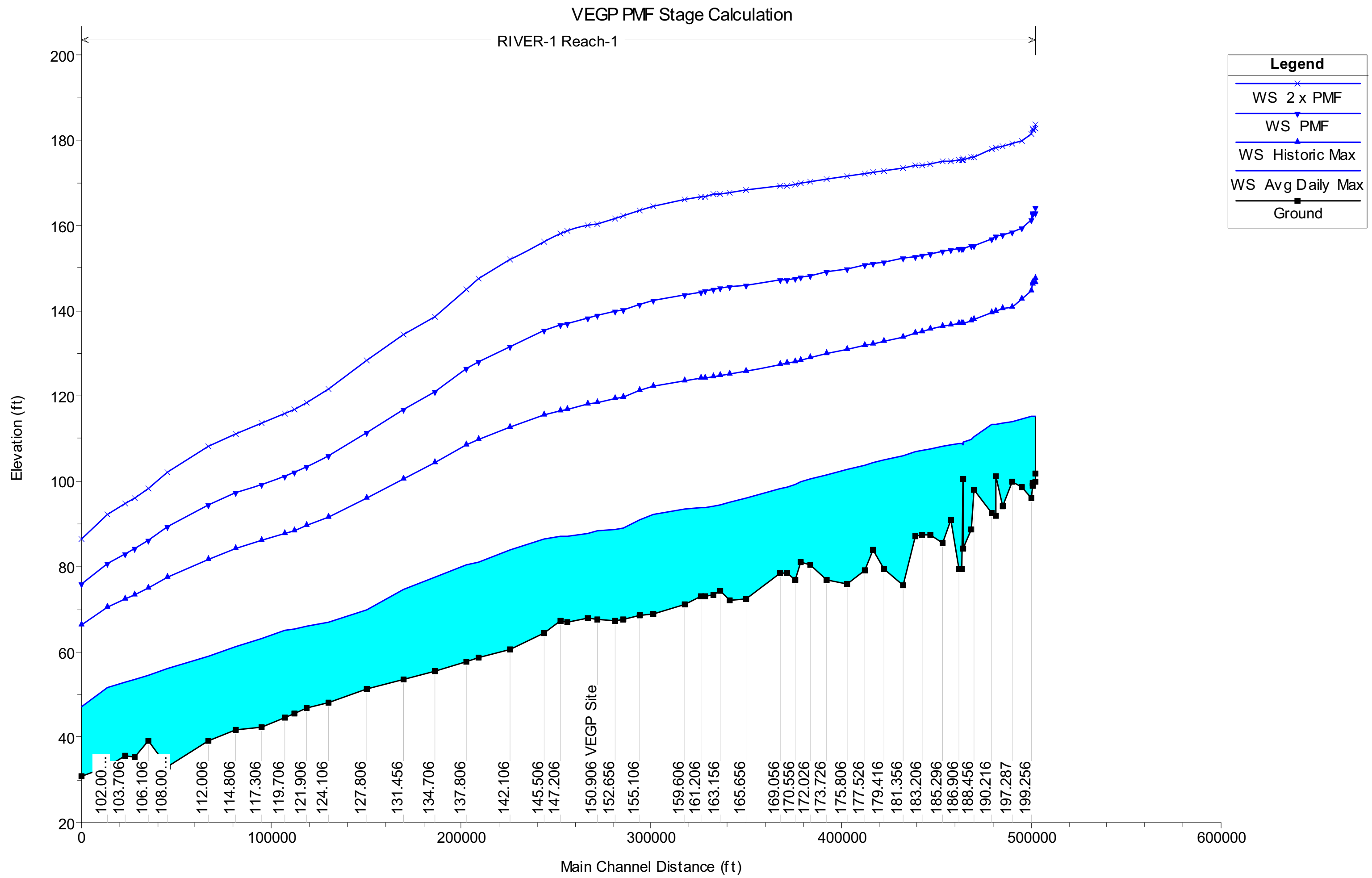


Figure 2.4-212
Longitudinal Profiles of the Savannah River from Steady-State HEC-RAS Model Run

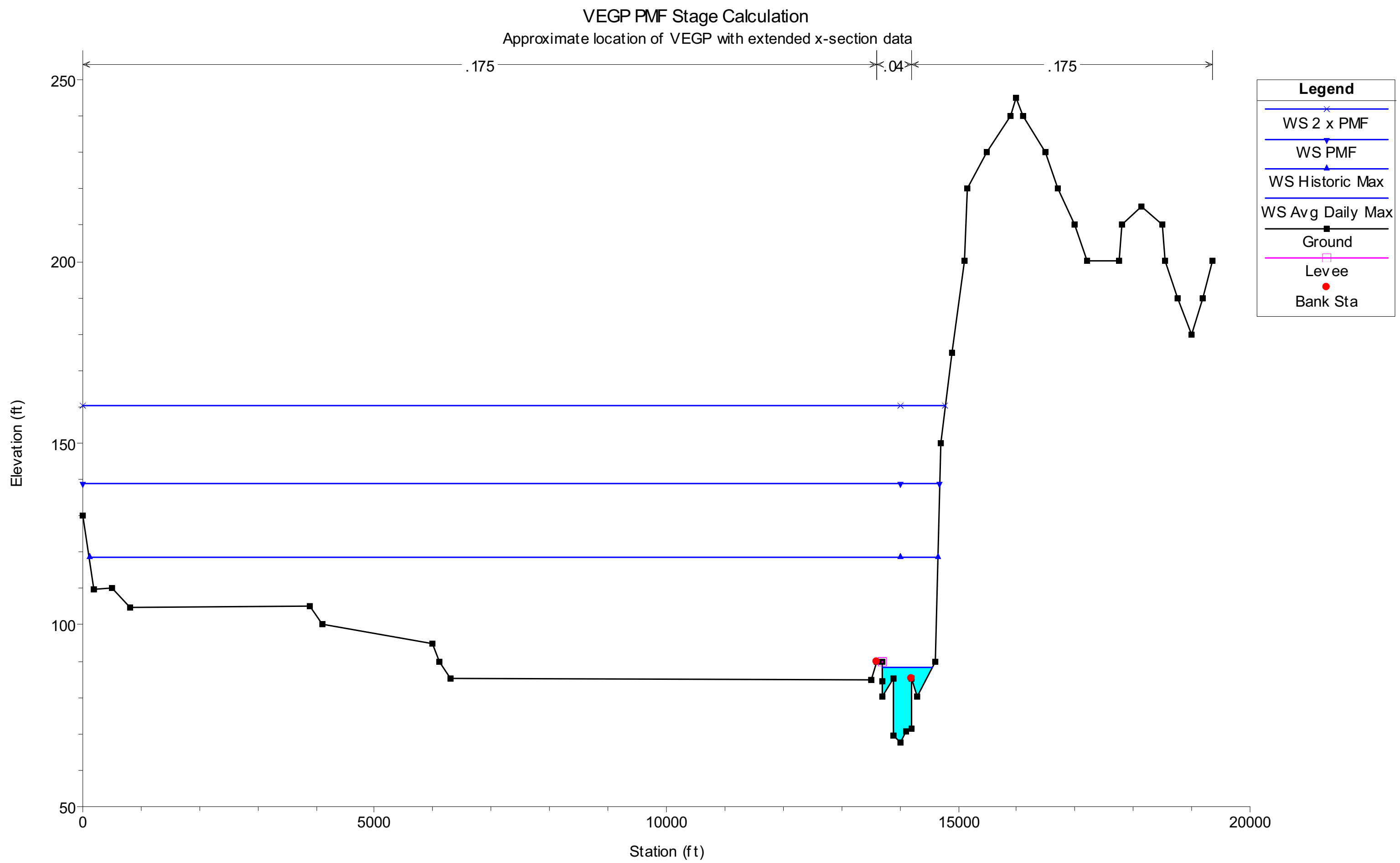


Figure 2.4-213
HEC-RAS Model Section at VEGP Site (Looking Downstream)

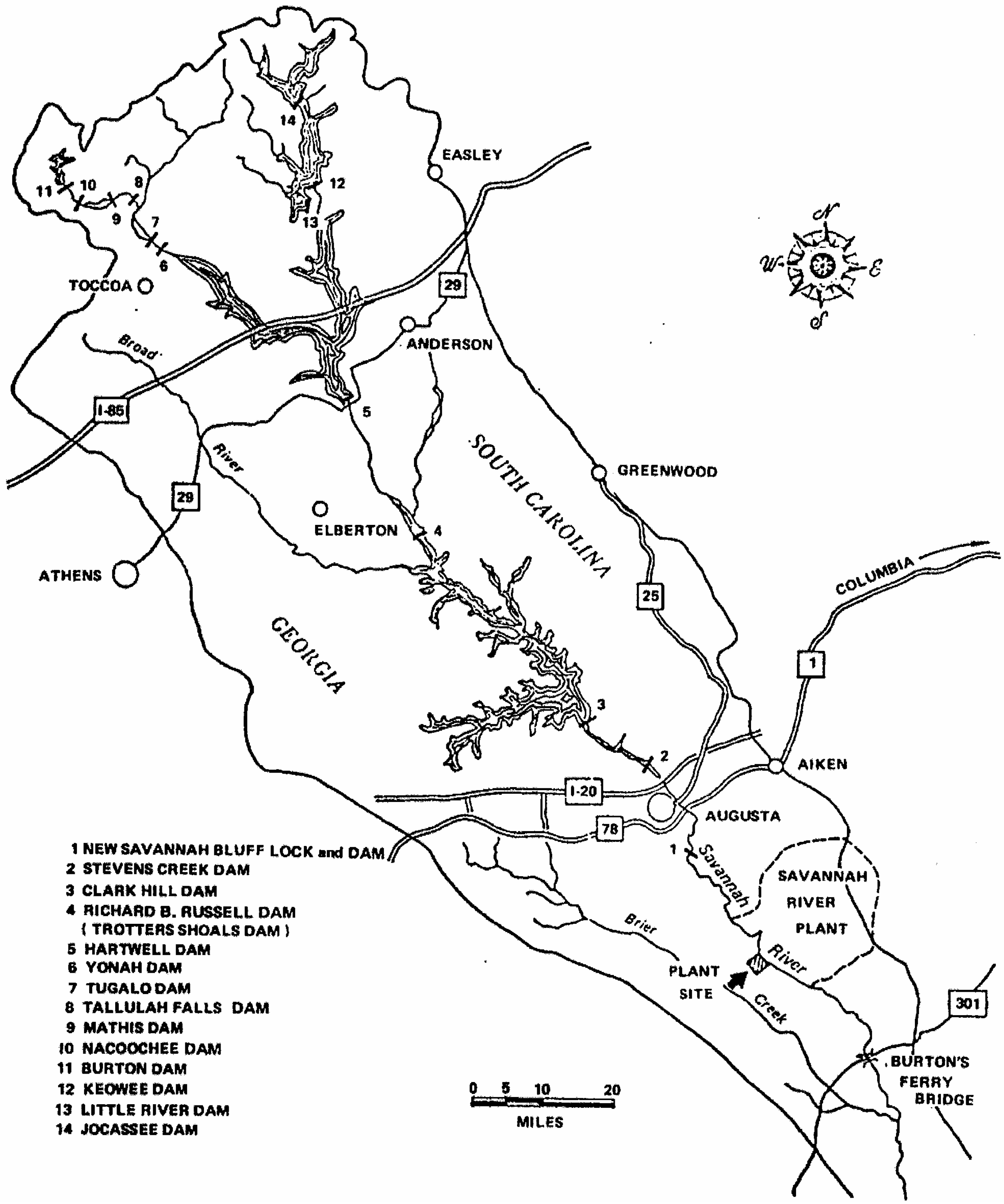
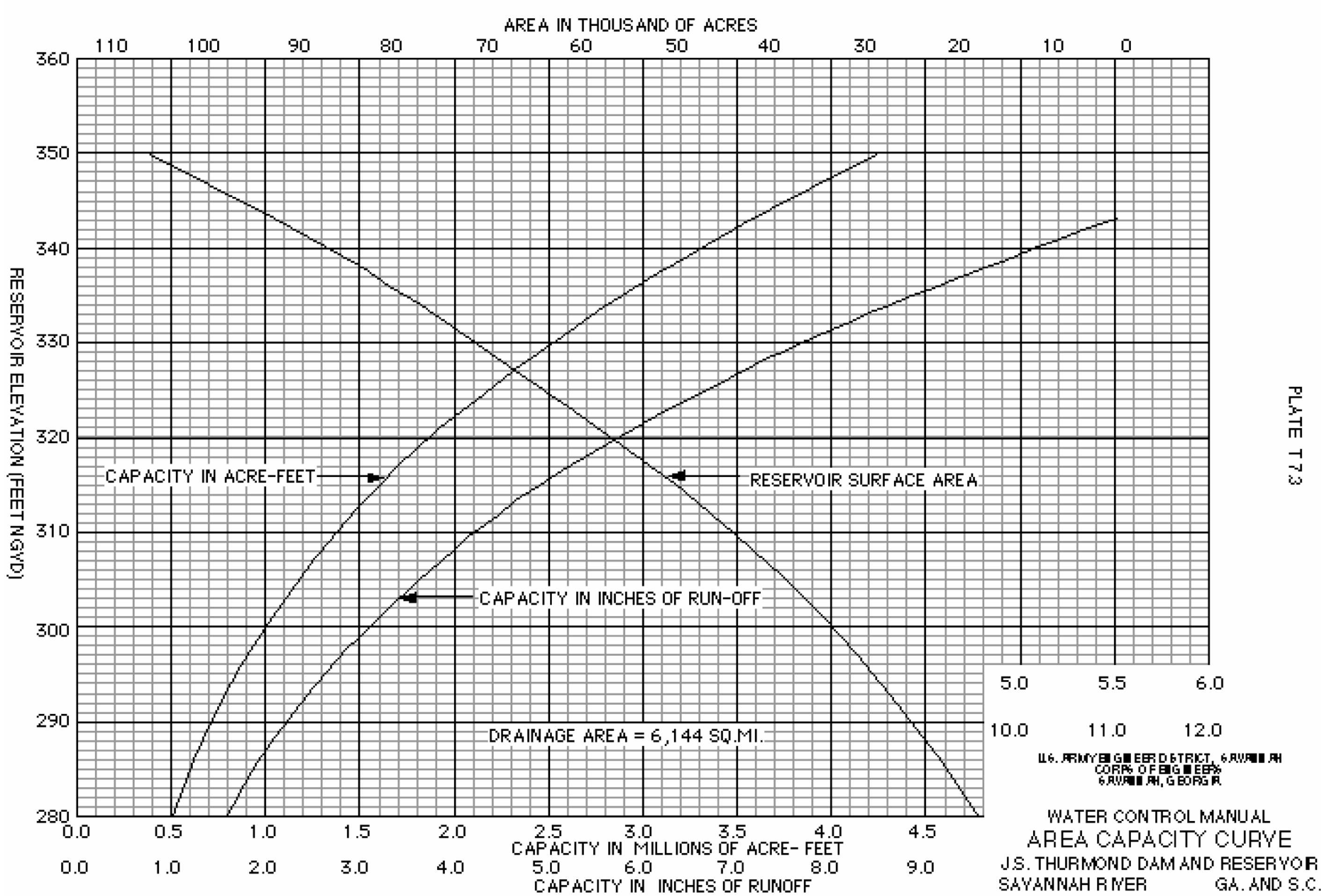
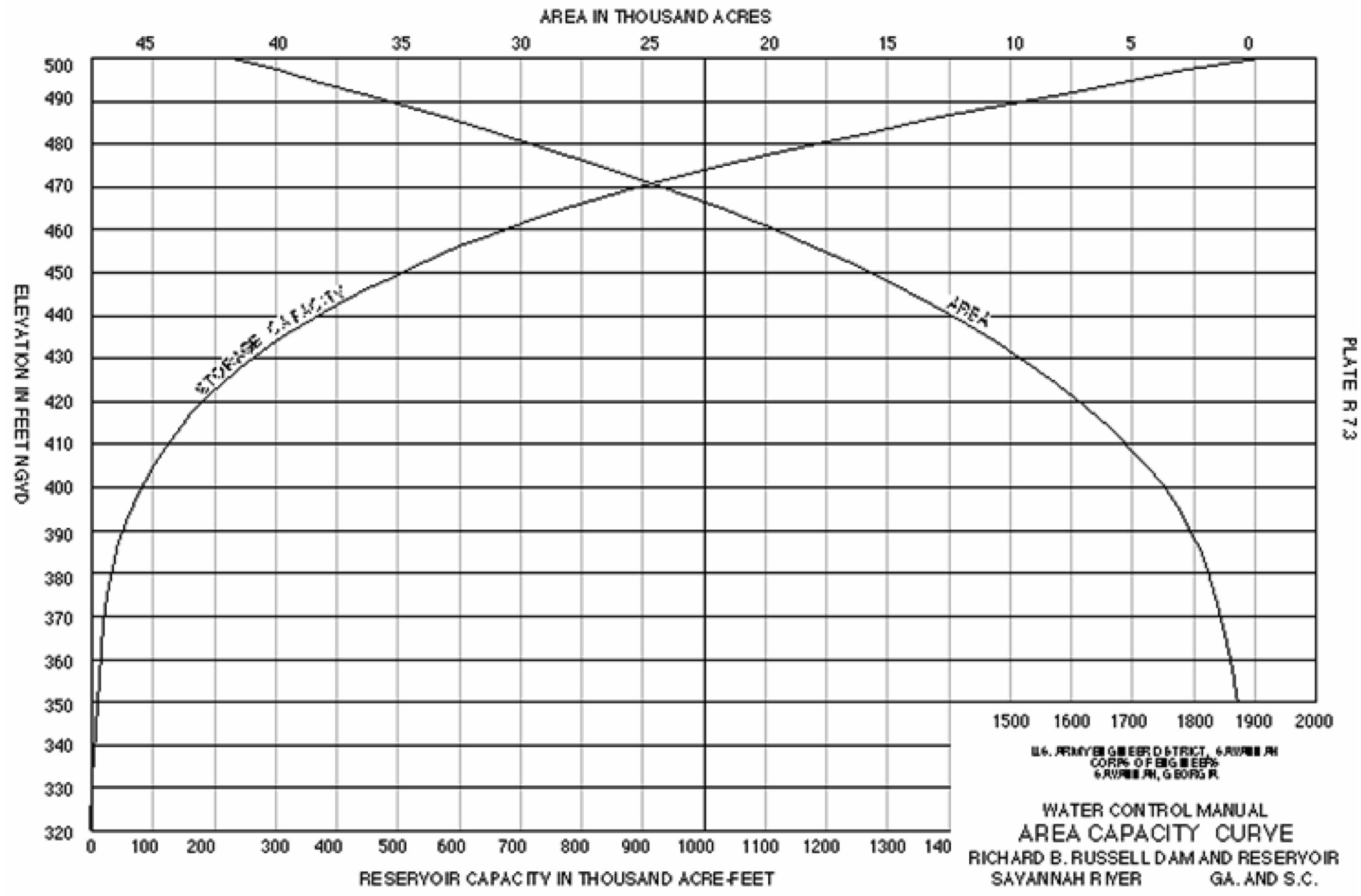


Figure 2.4-214
Savannah River Basin Dam Locations



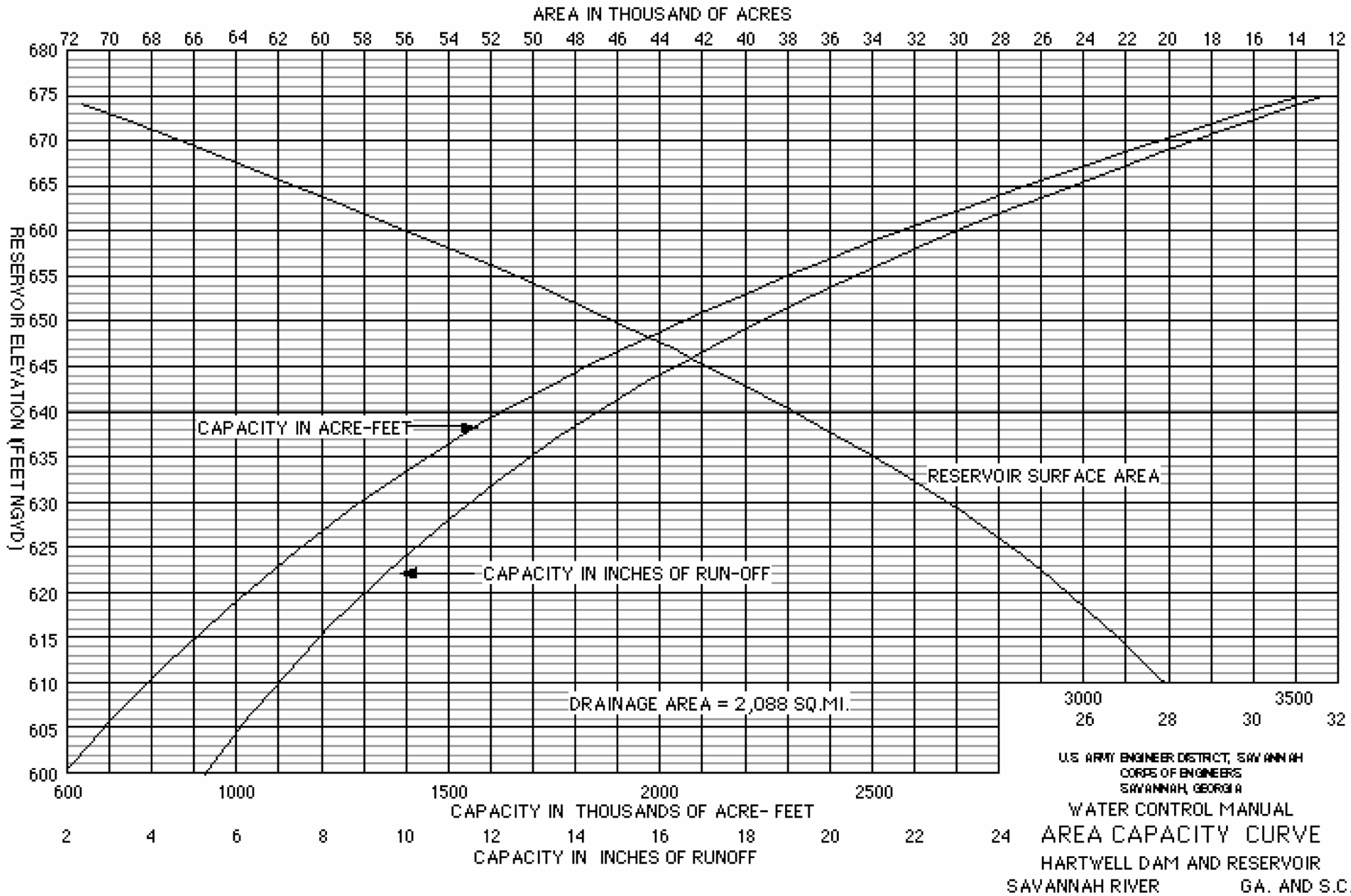
Source: USACE 1996

Figure 2.4-215
J. Strom Thurmond Area Capacity Curve



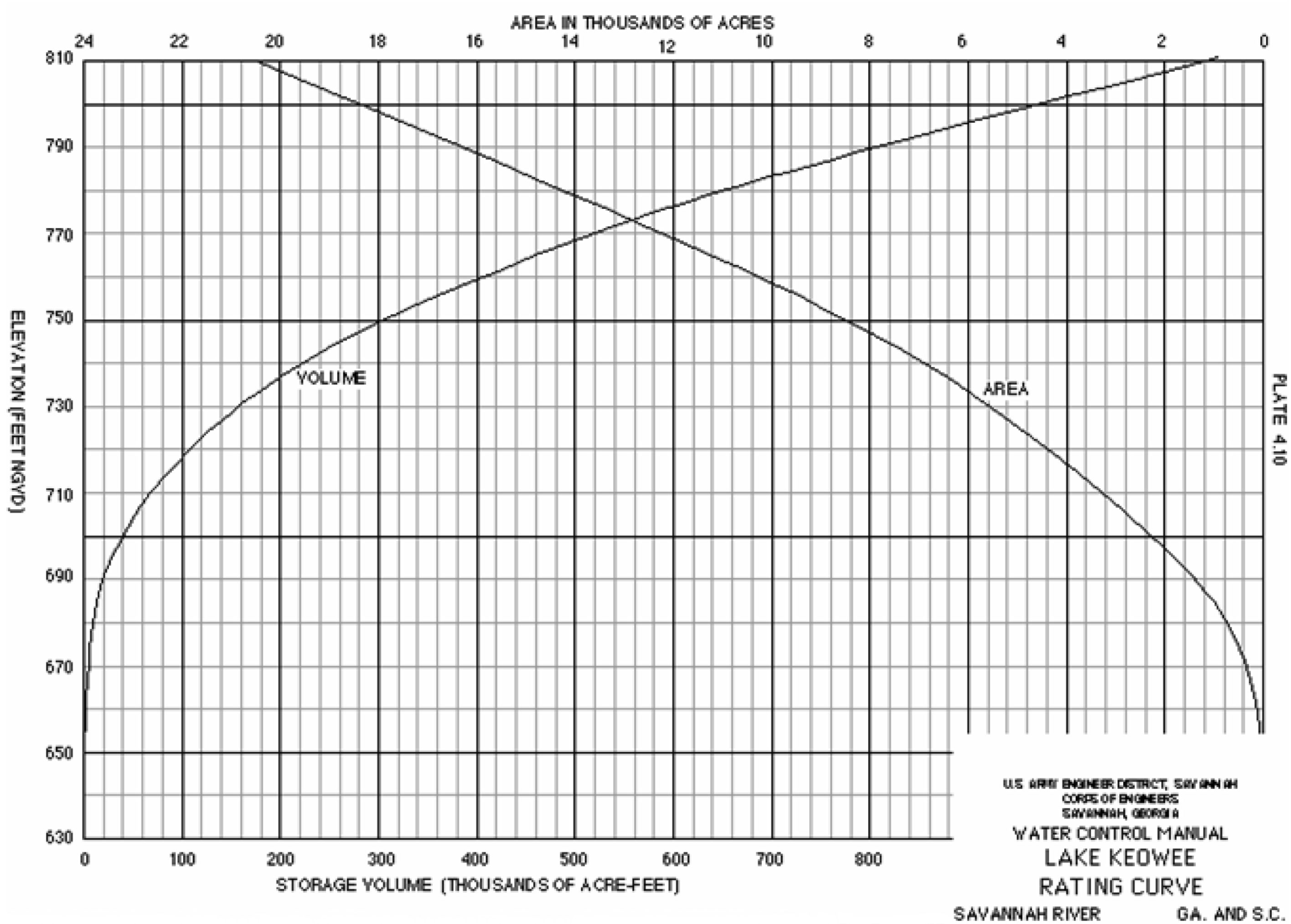
Source: USACE 1996

Figure 2.4-216
Richard B. Russell Area Capacity Curve



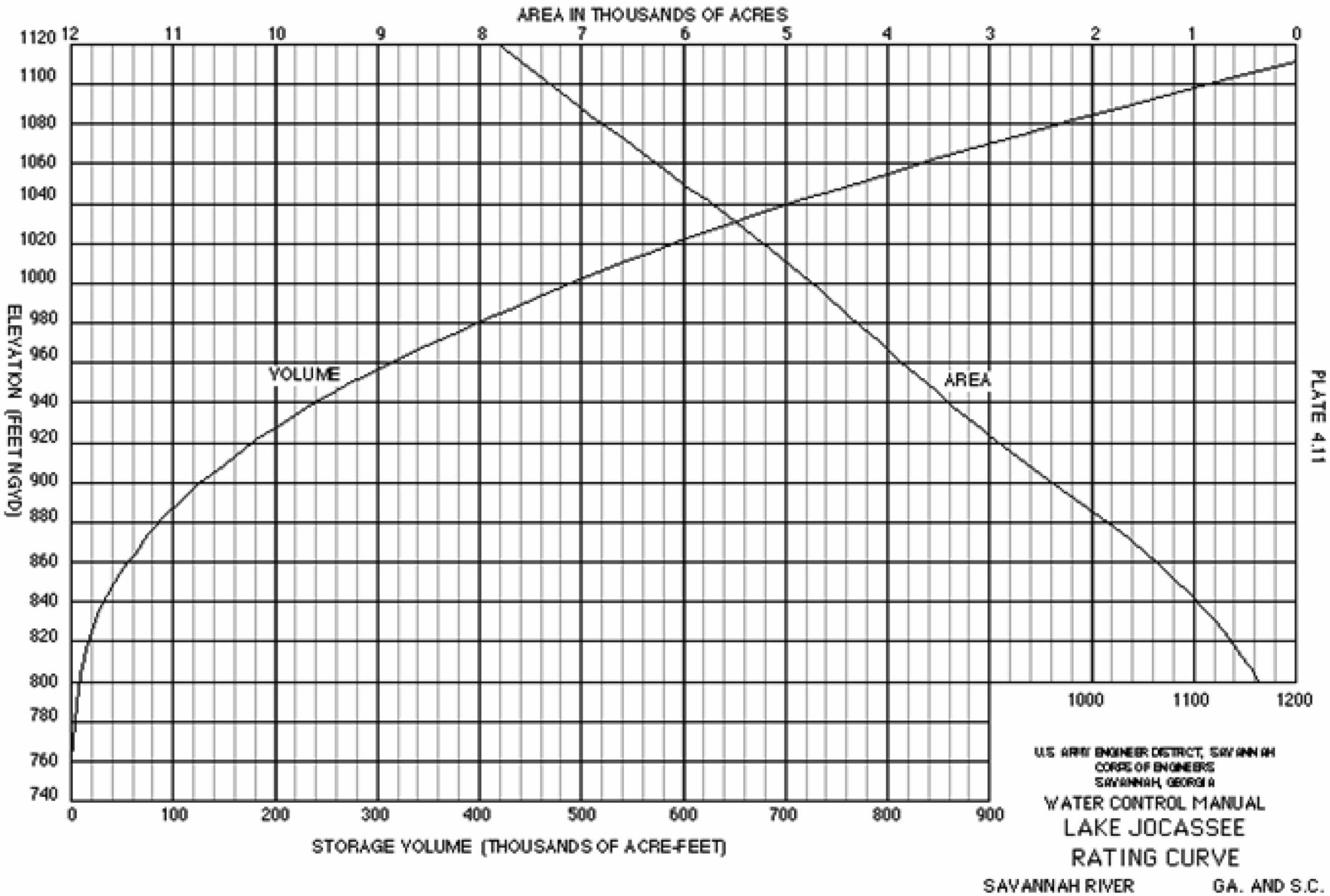
Source: USACE 1996

Figure 2.4-217
Hartwell Dam and Reservoir Area Capacity



Source: USACE 1996

Figure 2.4-218
Keowee Area Capacity Curve



Source: (USACE 1996)

Figure 2.4-219
Jocassee Area Capacity Curve

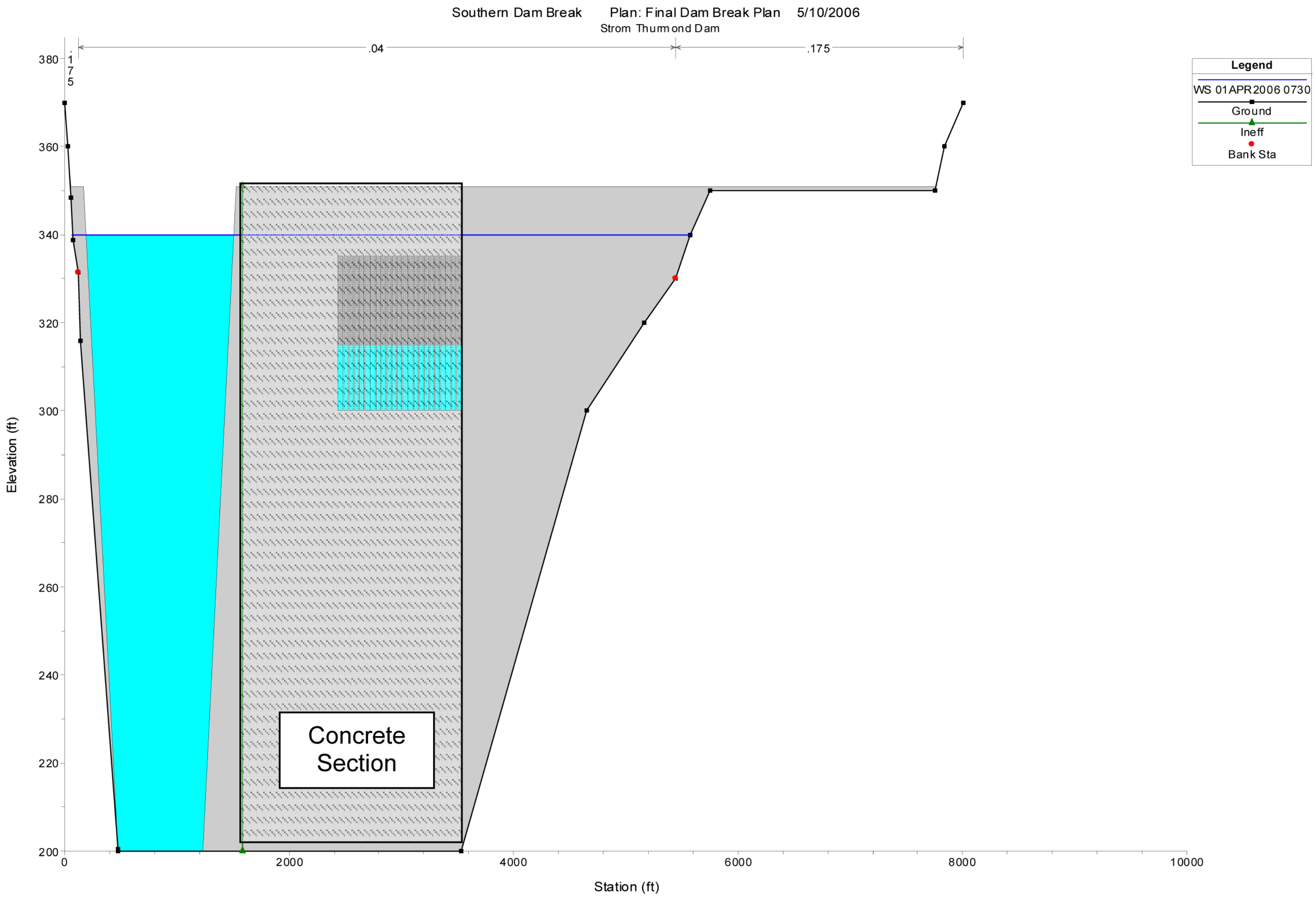


Figure 2.4-220
J. Strom Thurmond Dam Cross Section

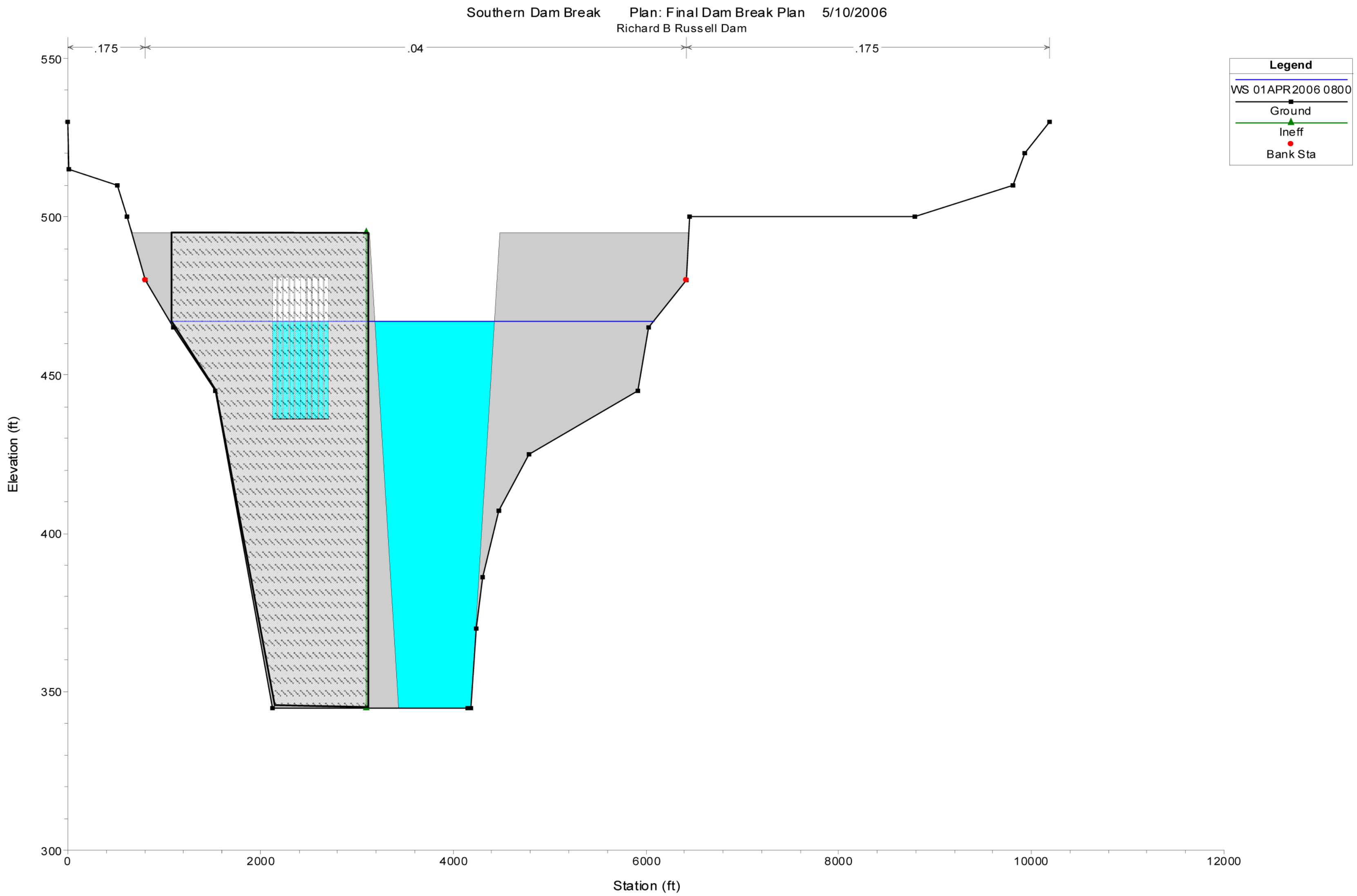


Figure 2.4-221
Richard B. Russell Dam Cross Section

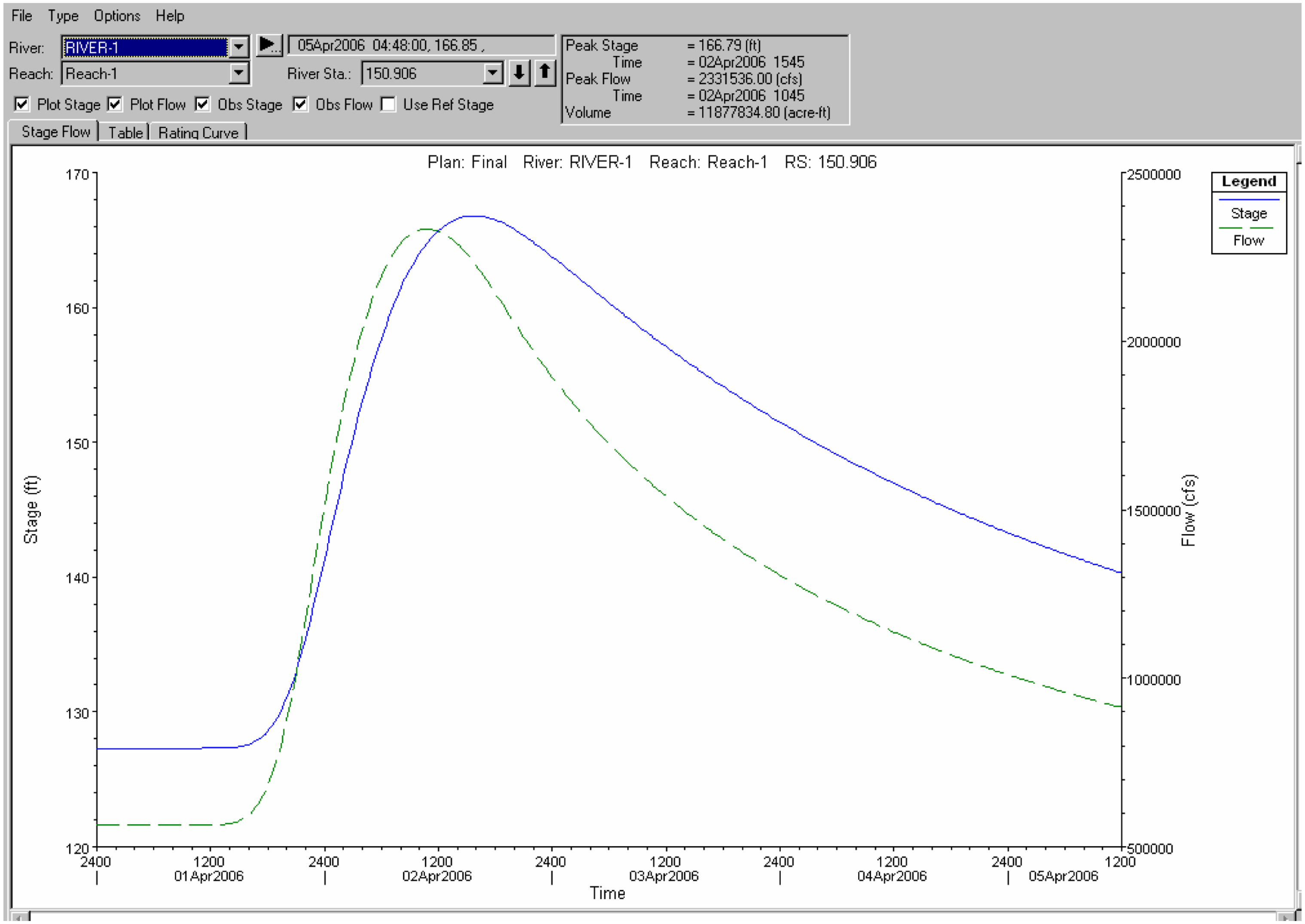


Figure 2.4-222
Dam Breach Flood Flow and Stage Hydrograph at the VEGP Site

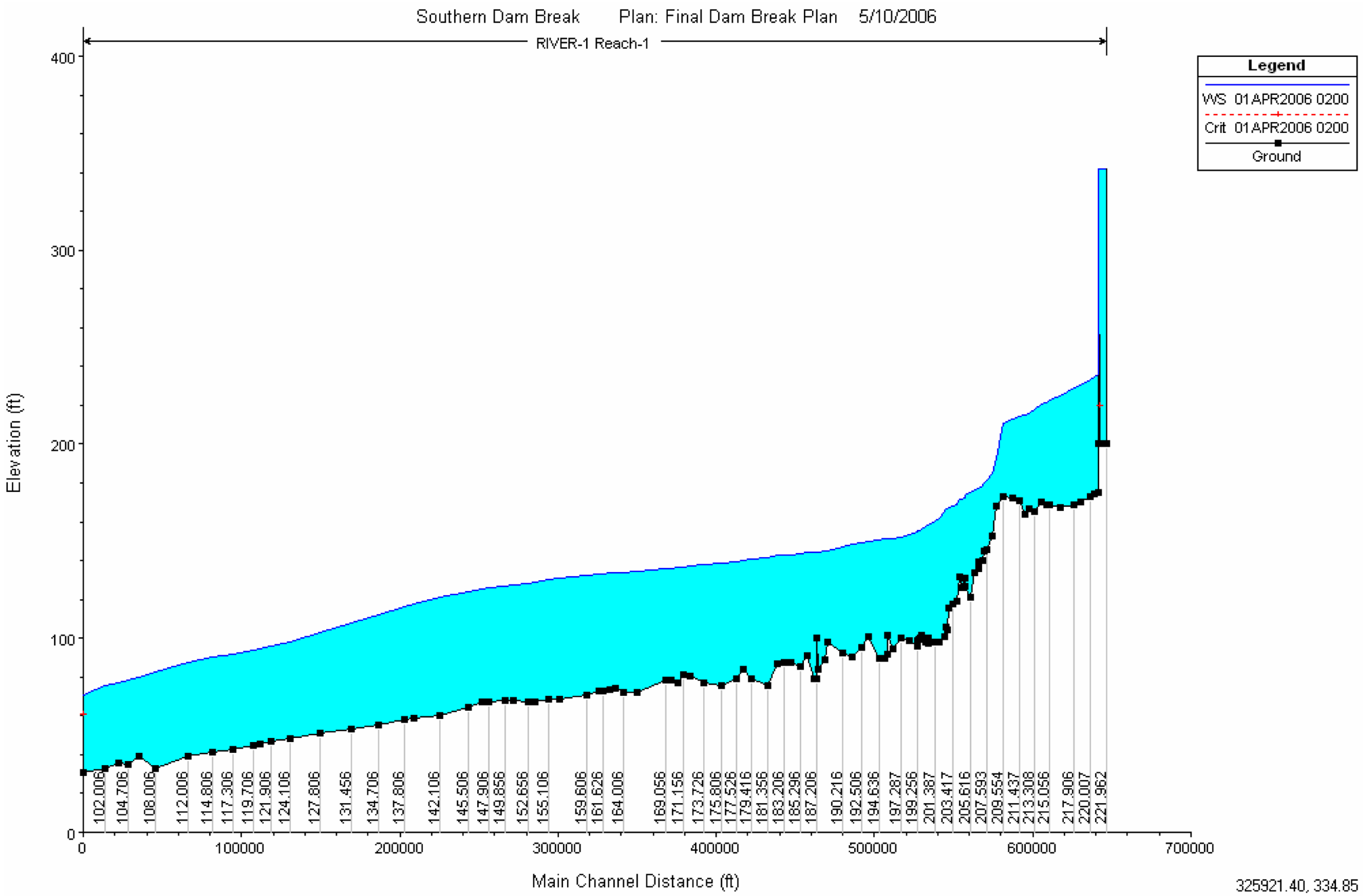


Figure 2.4-223
Savannah River SPF Water Surface Profile

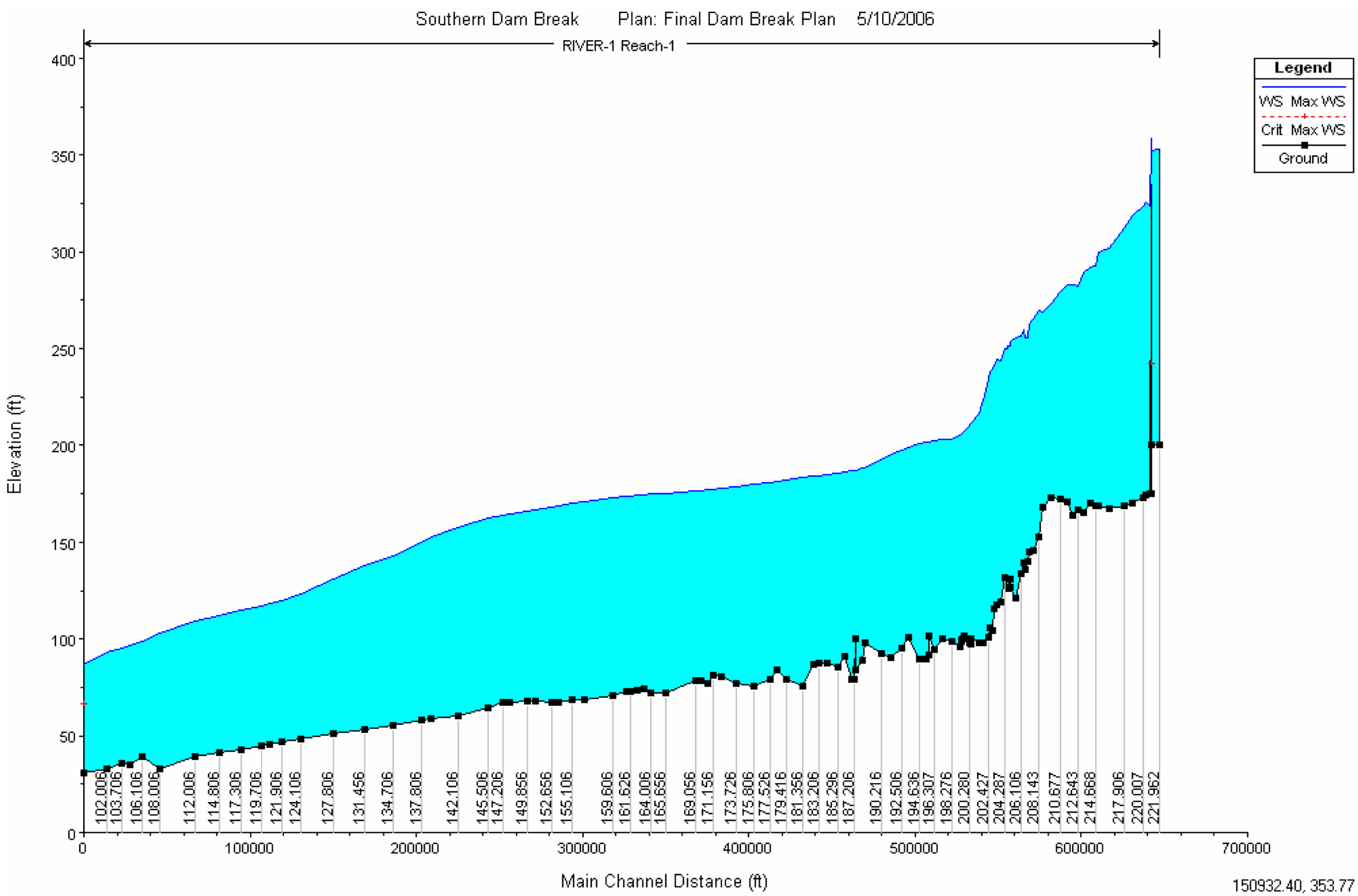


Figure 2.4-224
Savannah River Dam Breach Flood Maximum Water Surface Profile

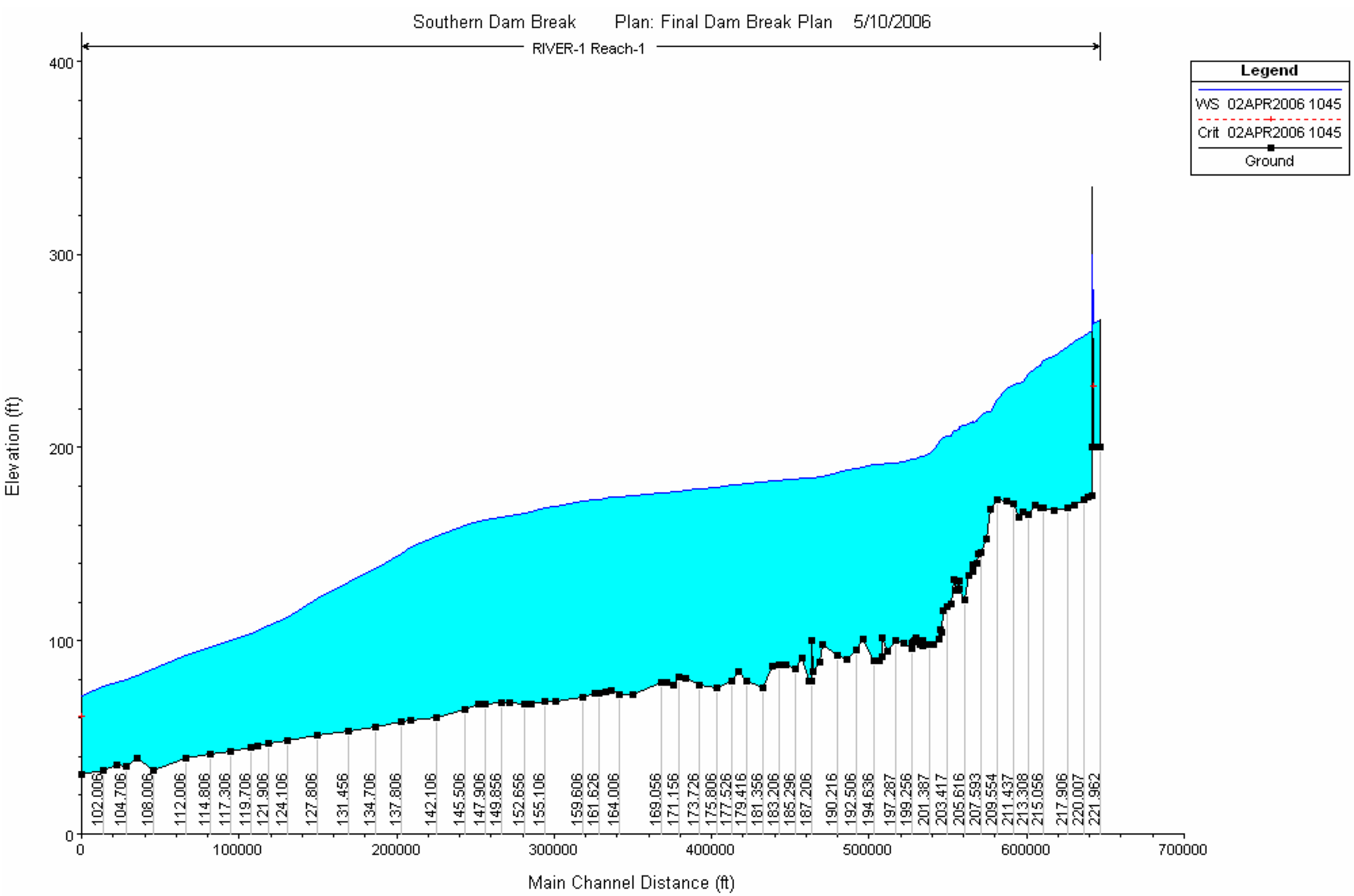


Figure 2.4-225
Savannah River Dam Breach Flood Water Surface Profile for Peak Discharge at VEGP Site

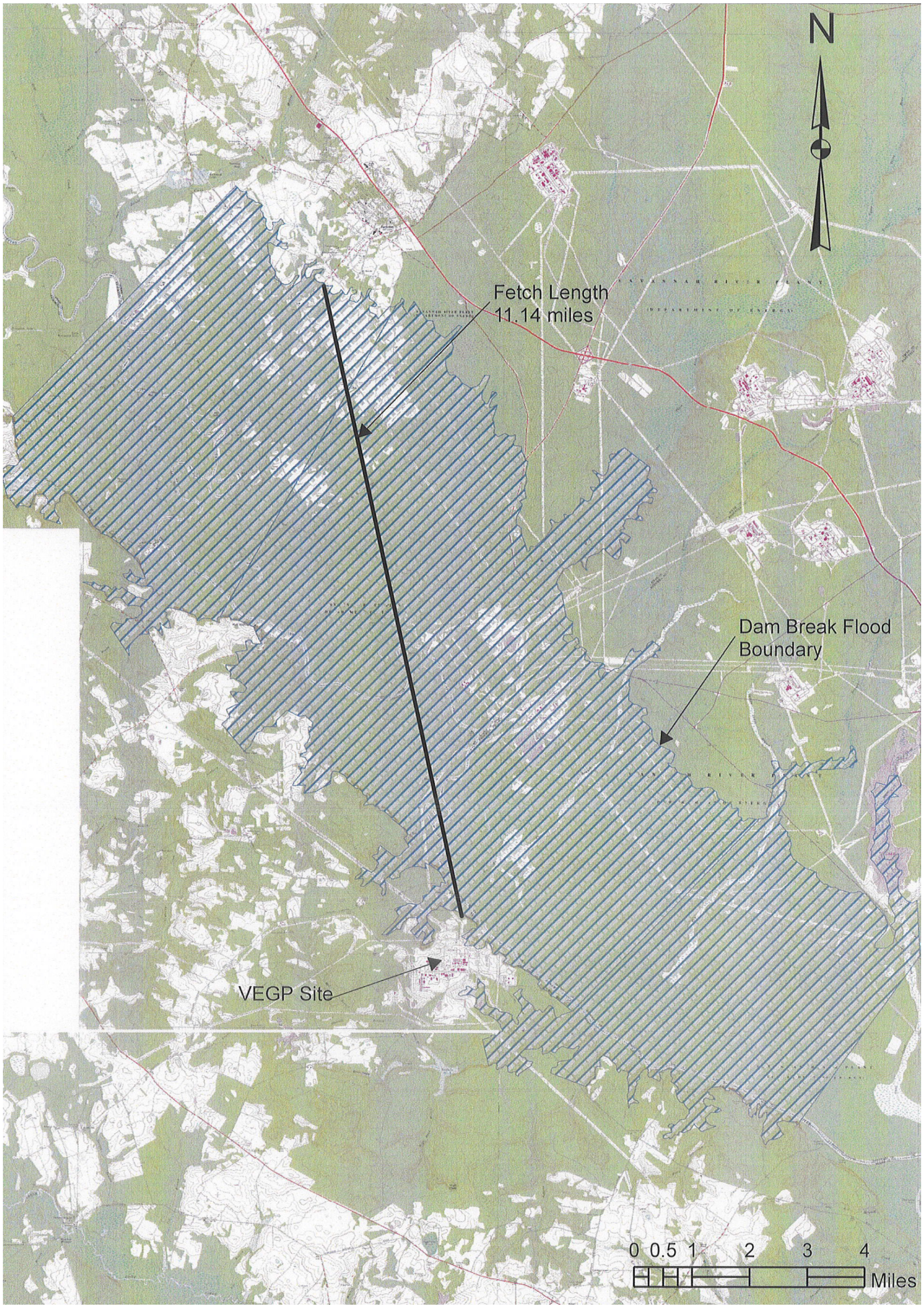
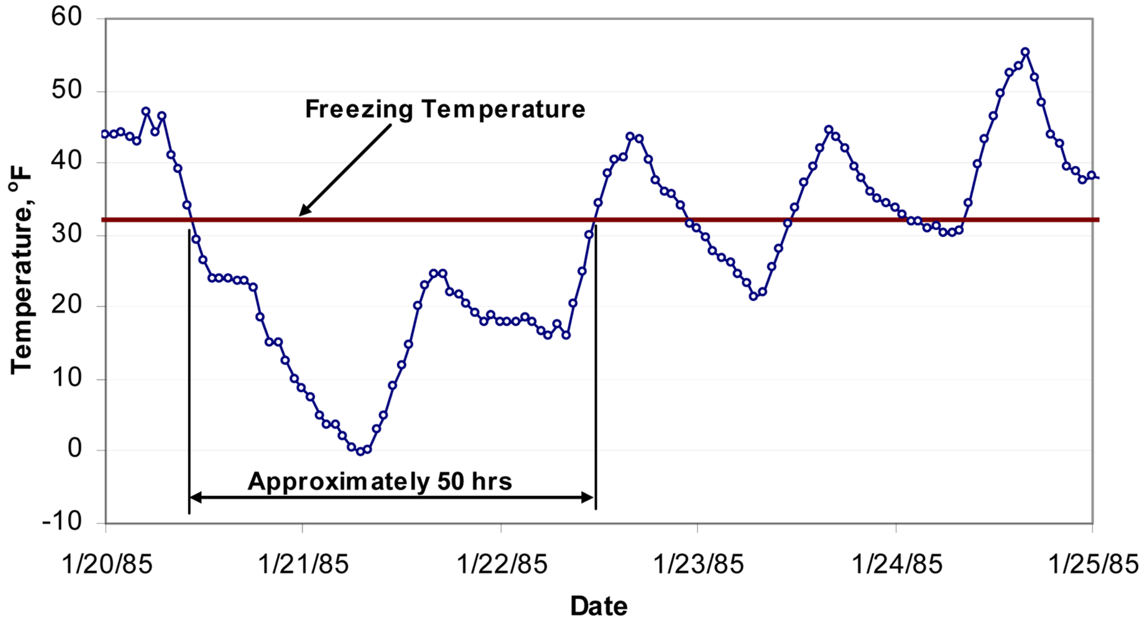
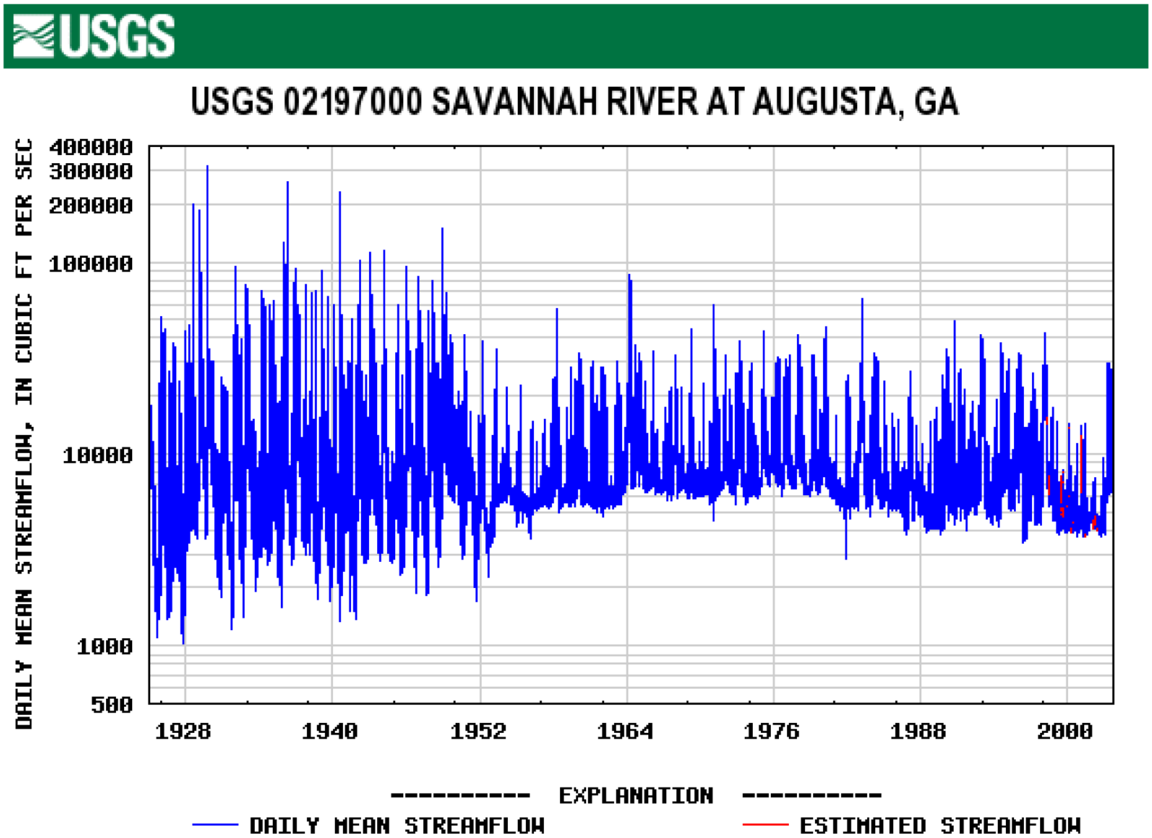


Figure 2.4-226
Maximum Fetch Length



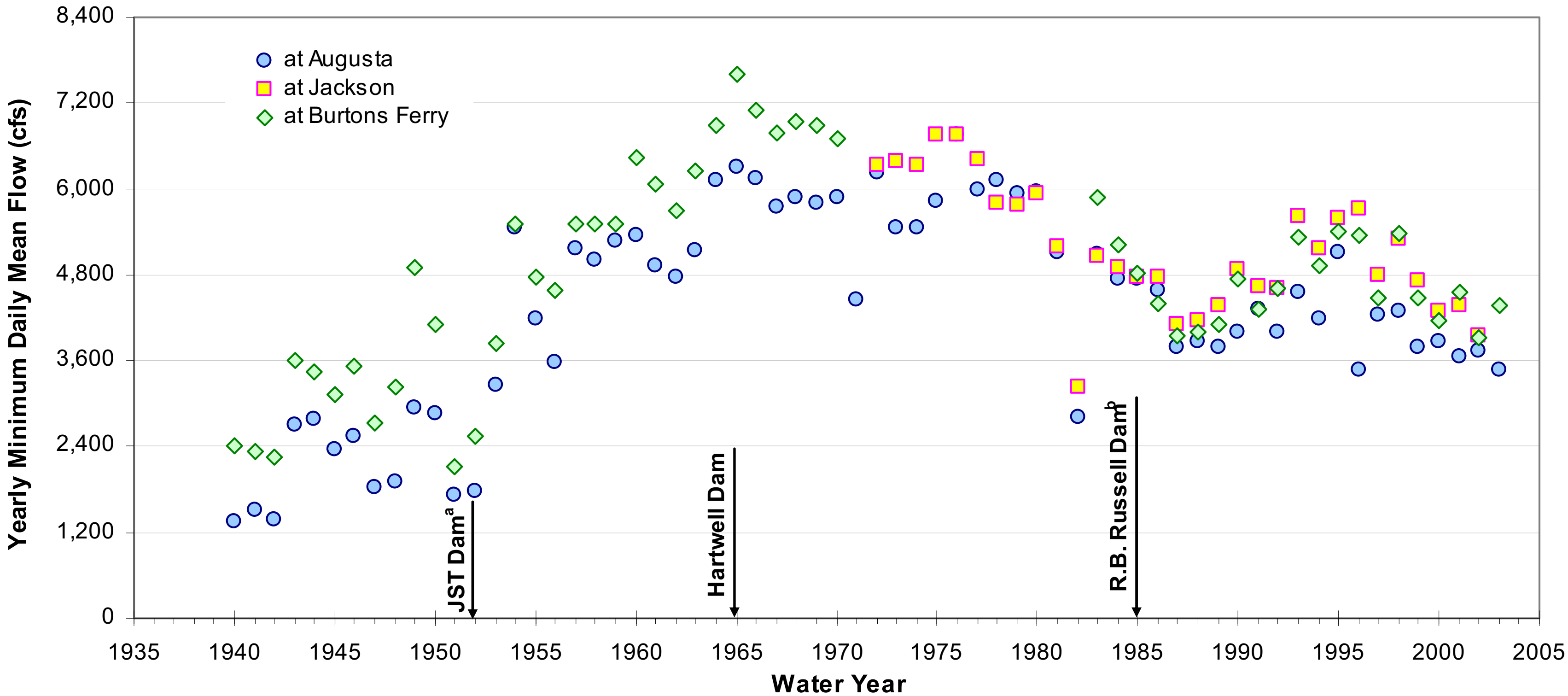
(The temperature remained below freezing for approximately 50 consecutive hours.)

Figure 2.4-227
Lowest Temperature Observed at the VEGP Site in 1985



Source: USGS 2006b

Figure 2.4-228
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

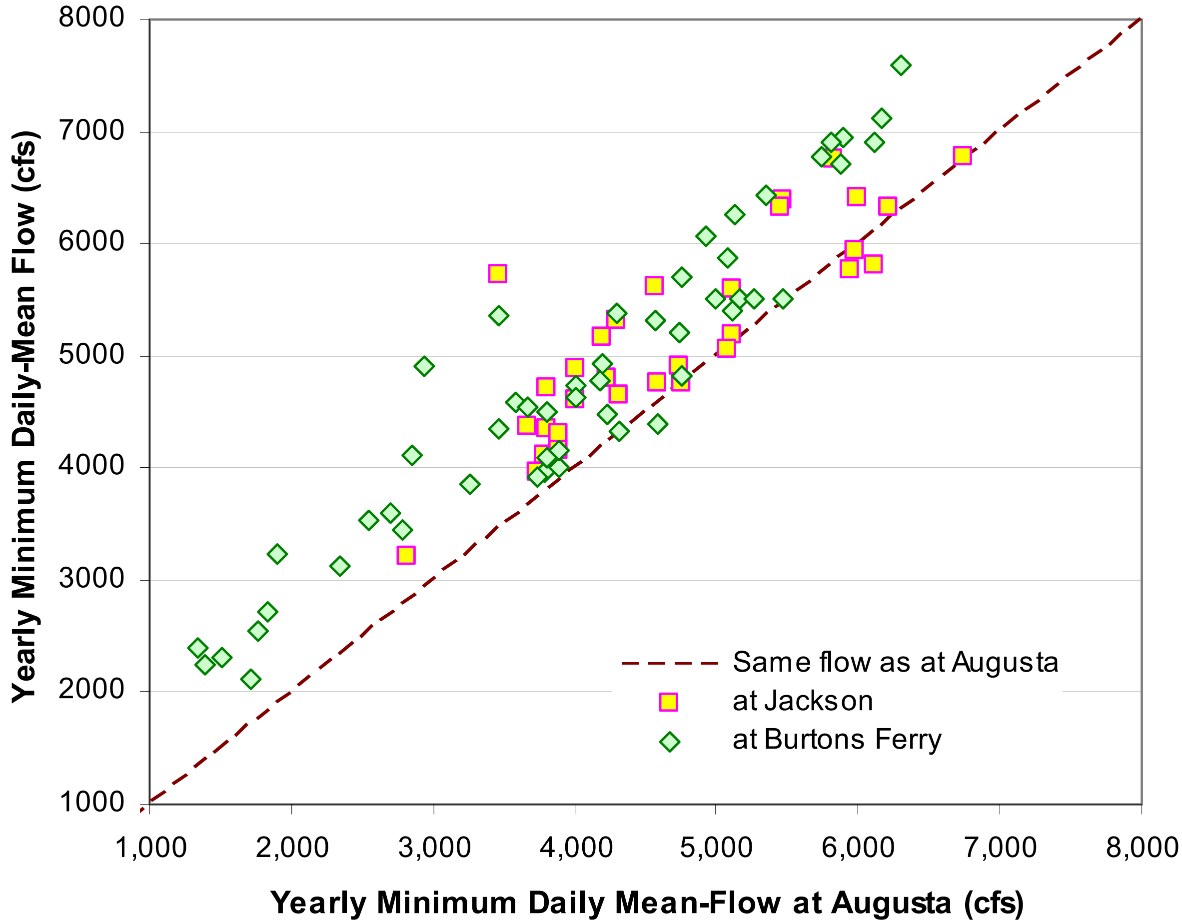


[a] J. Strom Thurmond Dam

[b] Richard B. Russell Dam

Source: USGS 2006g

Figure 2.4-229
Variation in Annual Minimum Daily-mean Stream Flow in the Savannah River at Augusta, Jackson, and Burtons Ferry Gages



Source: USGS 2006g

Figure 2.4-230
Change in Annual Minimum Daily-mean Flow at Jackson and Burtons Ferry Corresponding to that at Augusta for the Period of 1940-2003

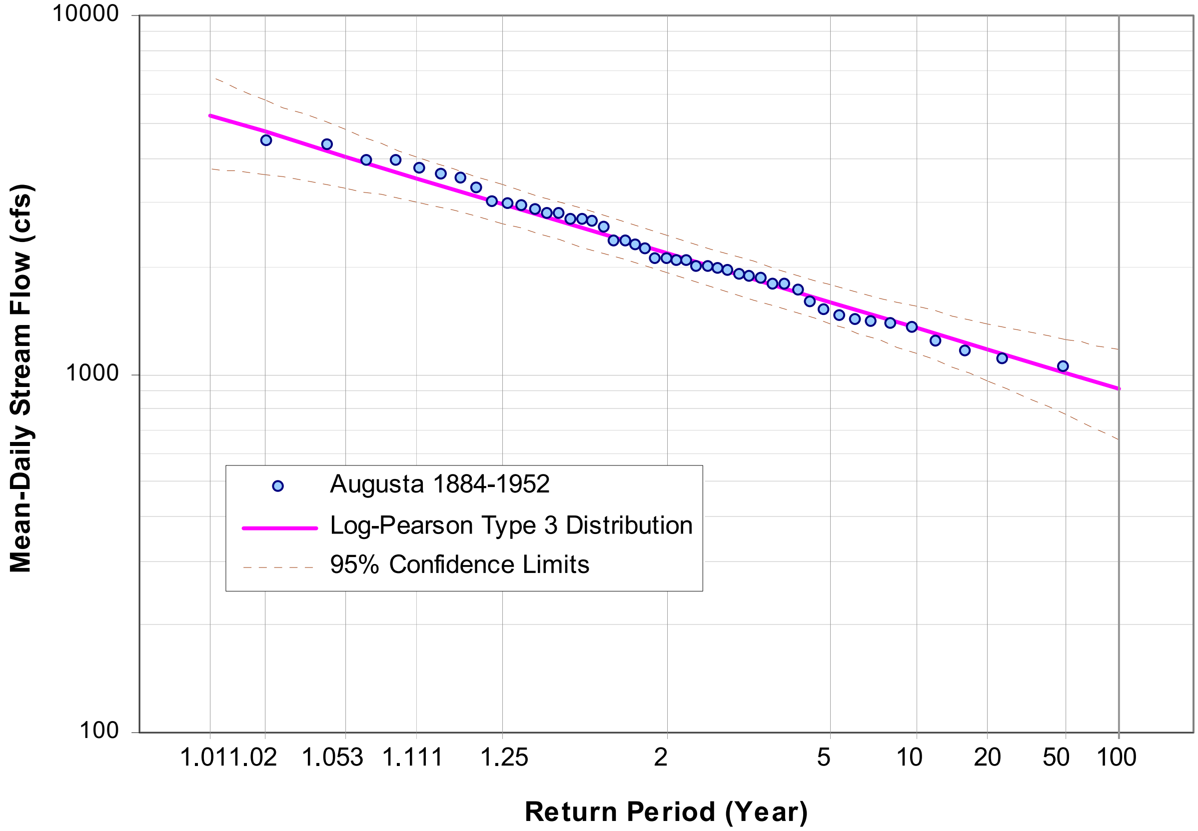


Figure 2.4-231
Log-Pearson Type 3 Distribution with Annual Minimum Daily-Mean Flow Data from Augusta for the Water Years 1884–1952

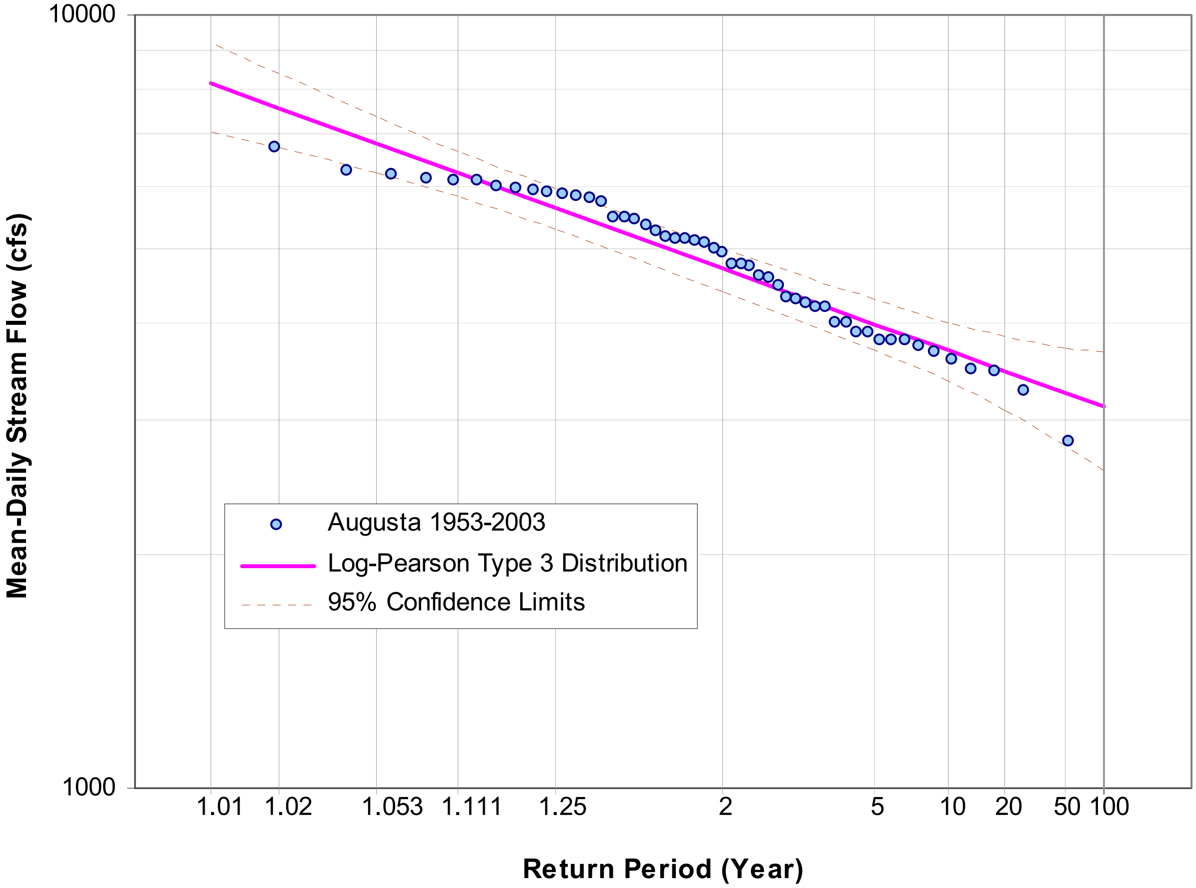


Figure 2.4-232
Log-Pearson Type 3 Distribution with Annual Minimum Daily-Mean Flow Data from Augusta for the Water Years 1953–2003

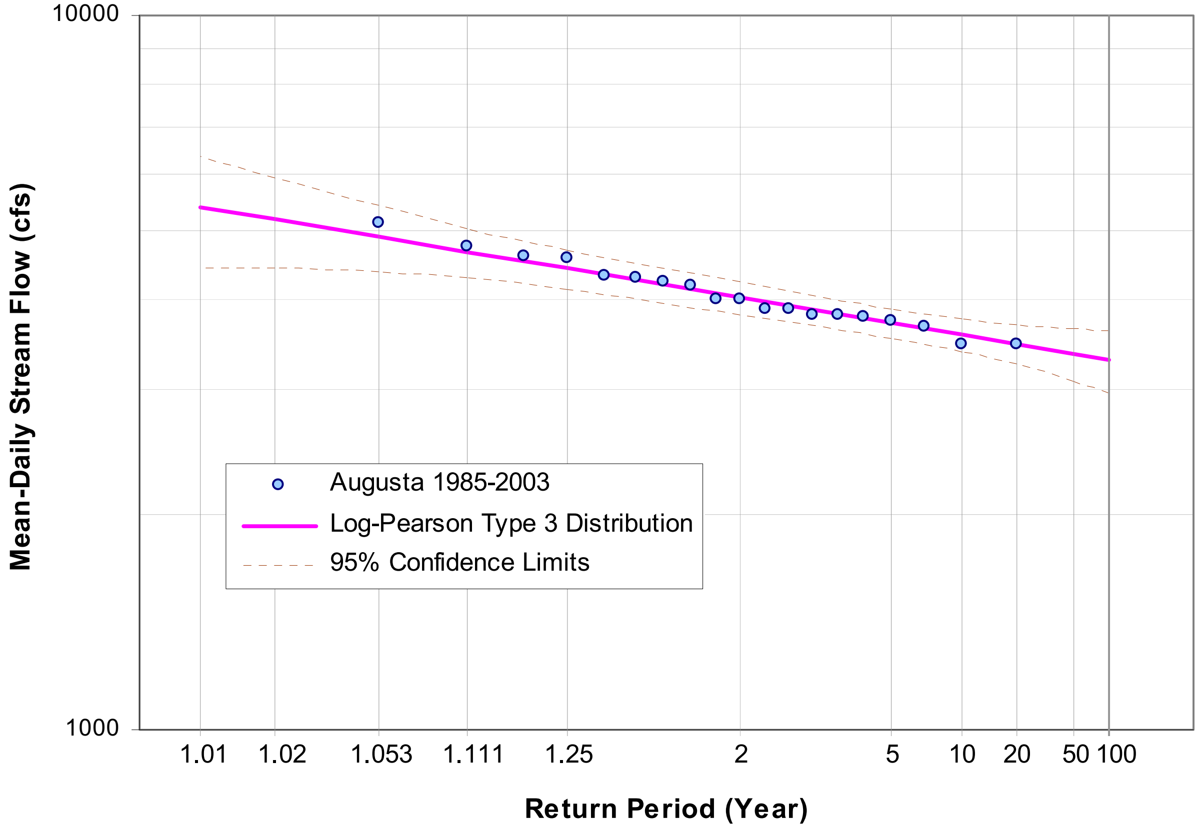


Figure 2.4-233
Log-Pearson Type 3 Distribution with Annual Minimum Daily-Mean Flow Data from Augusta for the Water Years 1985–2003

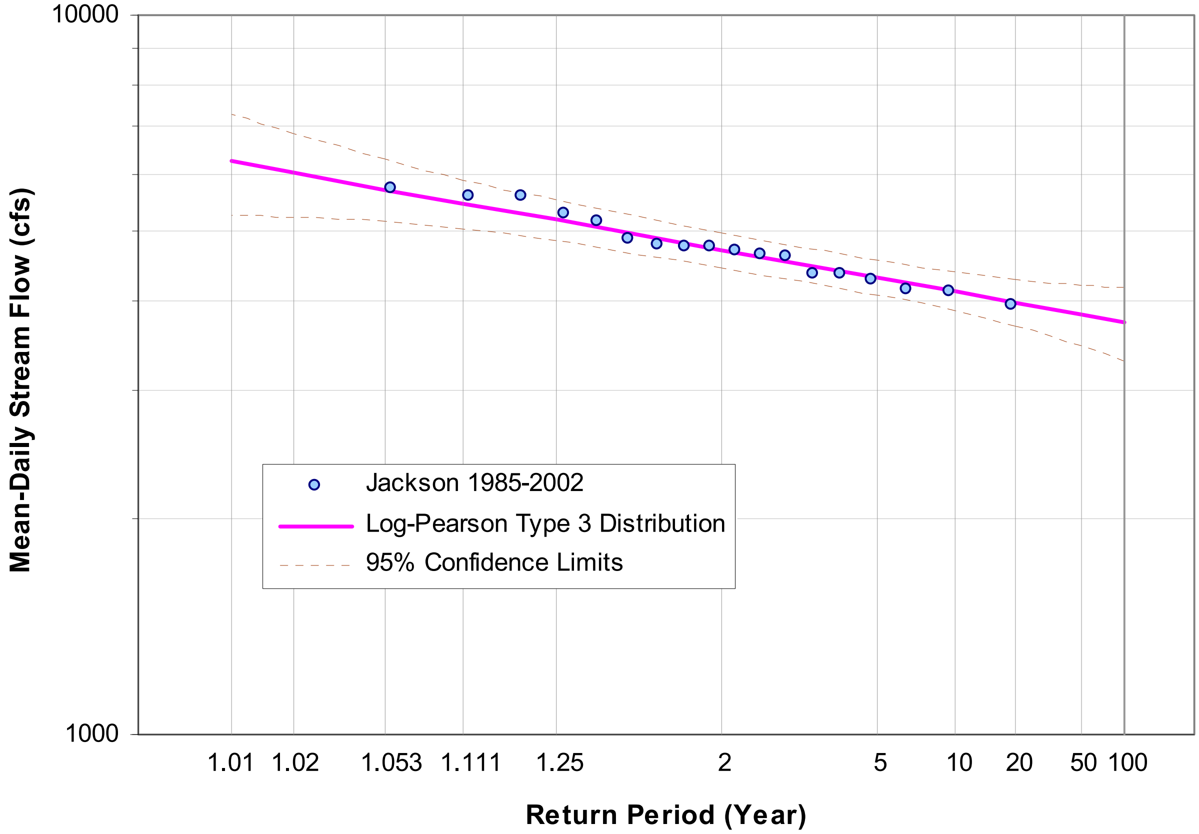
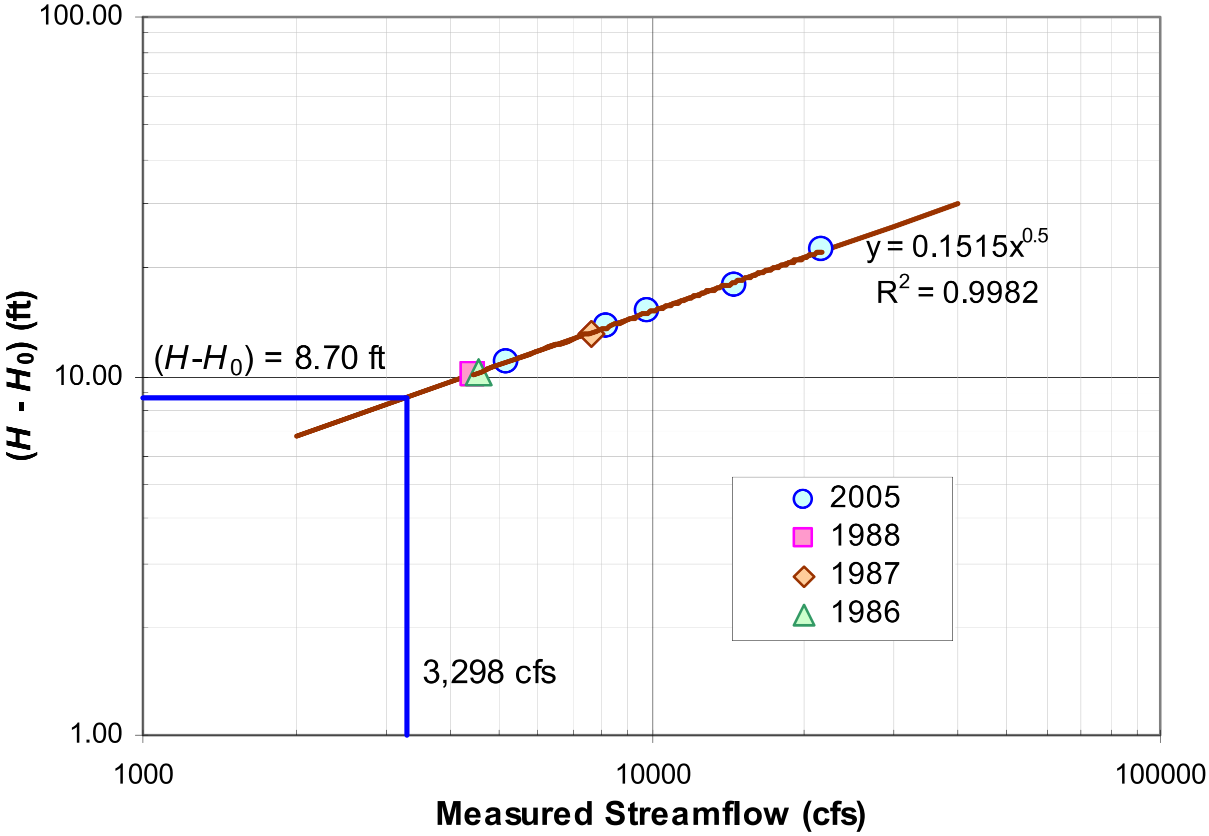


Figure 2.4-234
Log-Pearson Type 3 Distribution with Annual Minimum Daily-Mean Streamflow from Jackson for the Water Years 1985–2002



H = Water surface elevation in El. ft msl

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

Figure 2.4-235
River Stage-Discharge Rating Relationship at USGS Waynesboro Gage Station Near the VEGP Site Using Data for the Years 2005, 1988, 1987 and 1986

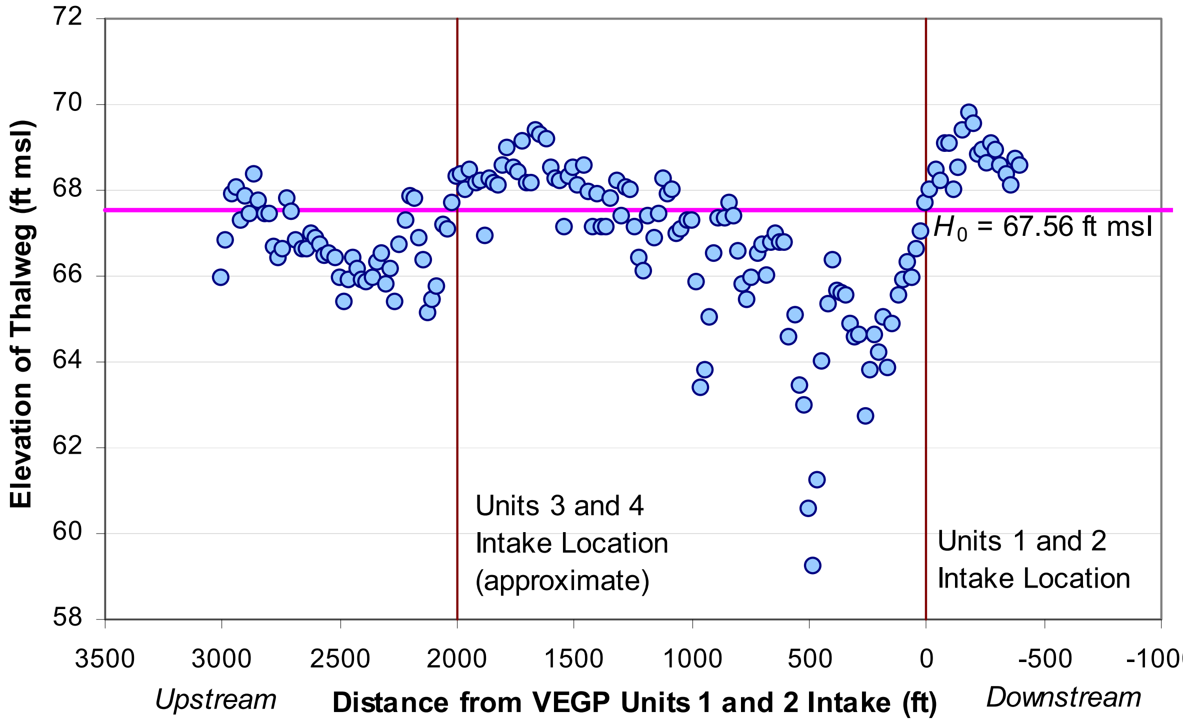


Figure 2.4-236
Comparison of Estimated River Stage Corresponding to Zero Discharge (H_0) with Measured River Thalweg Levels Near the Intake Location

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 2.4-237
Schematic Hydrostratigraphic Classification for VEGP Site

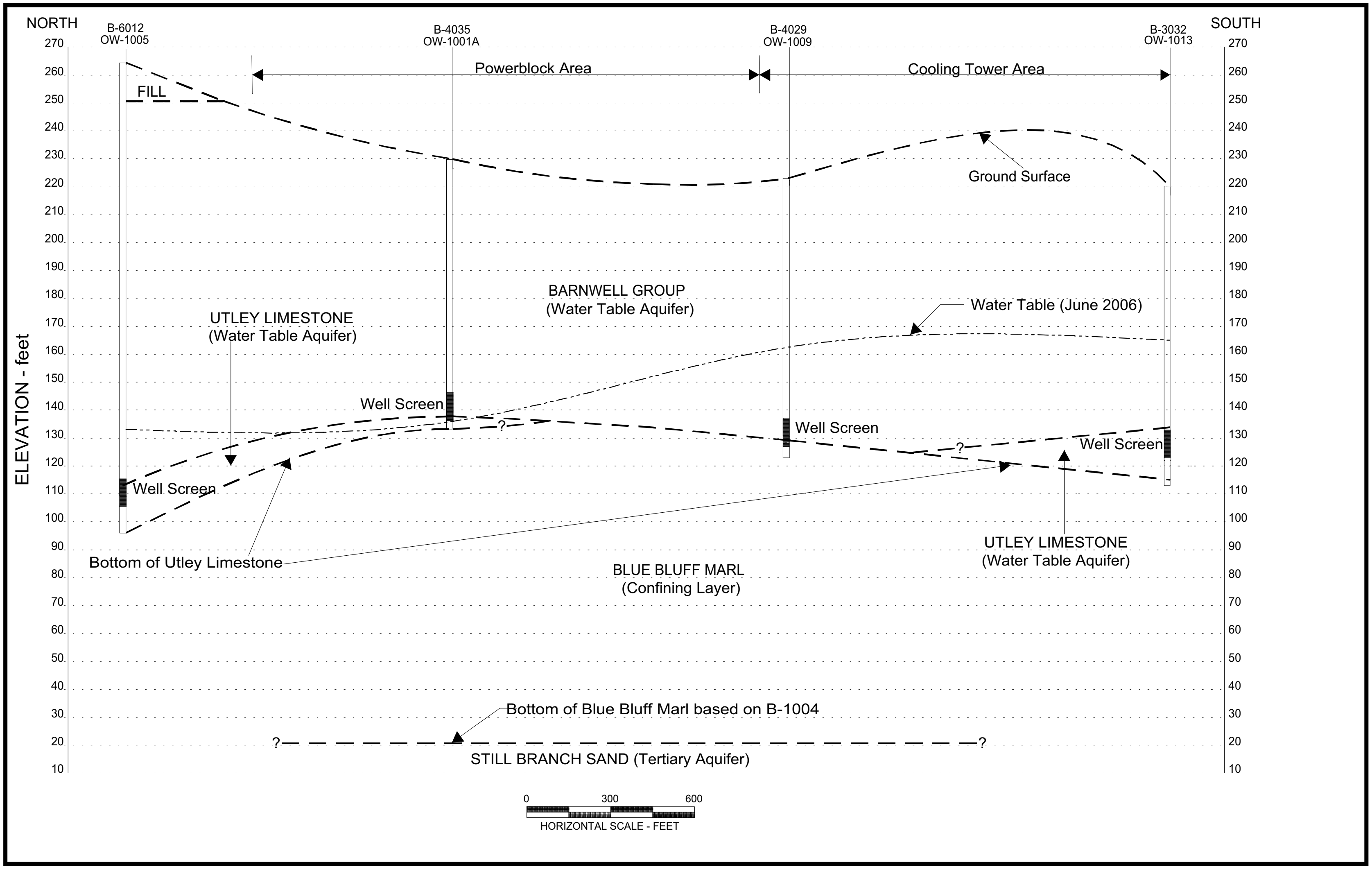


Figure 2.4-238
Hydrogeologic Cross-Section of the Water Table Aquifer at the VEGP Site

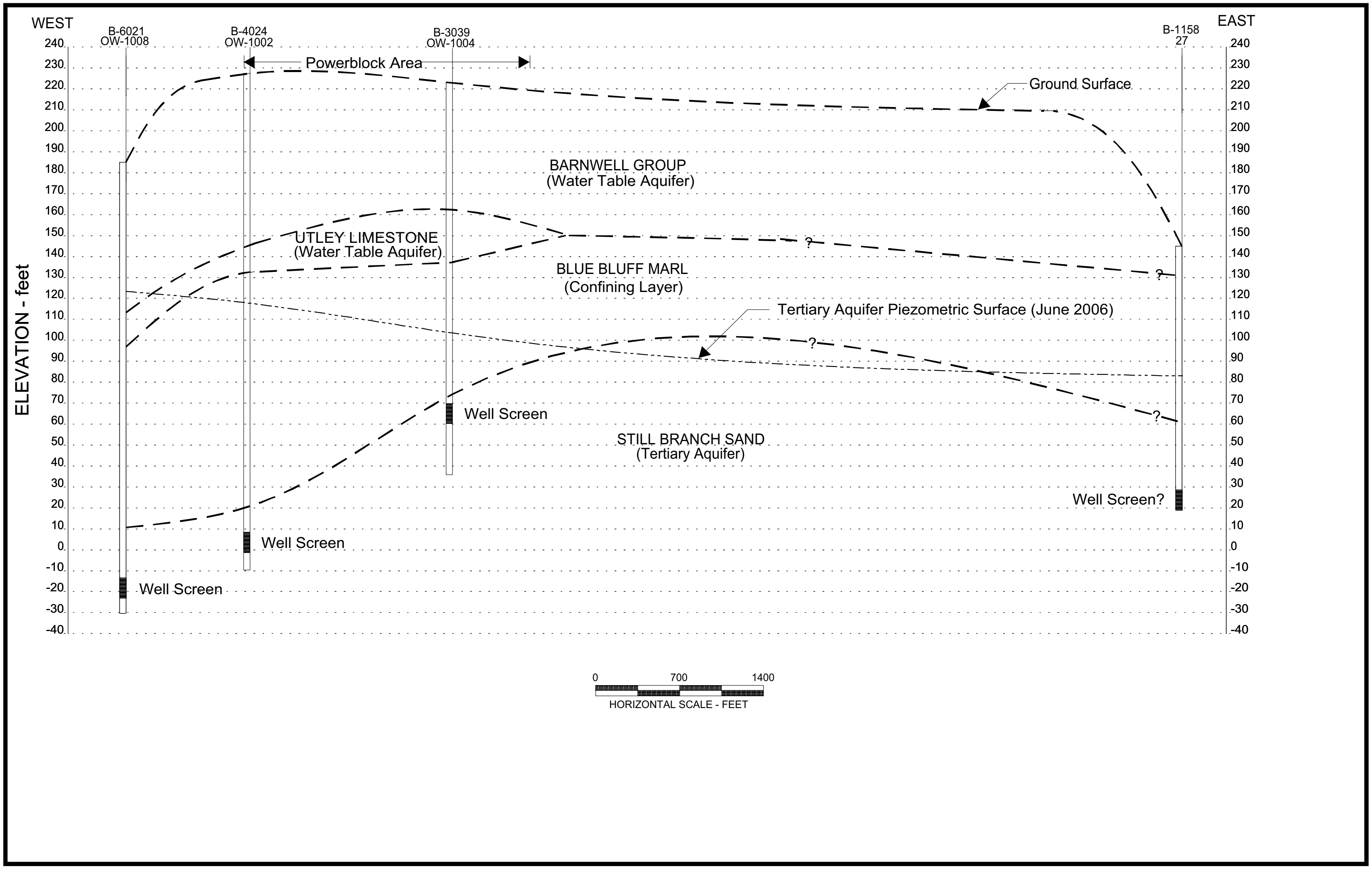


Figure 2.4-239
Hydrogeologic Cross-Section of the Tertiary Aquifer at the VEGP Site

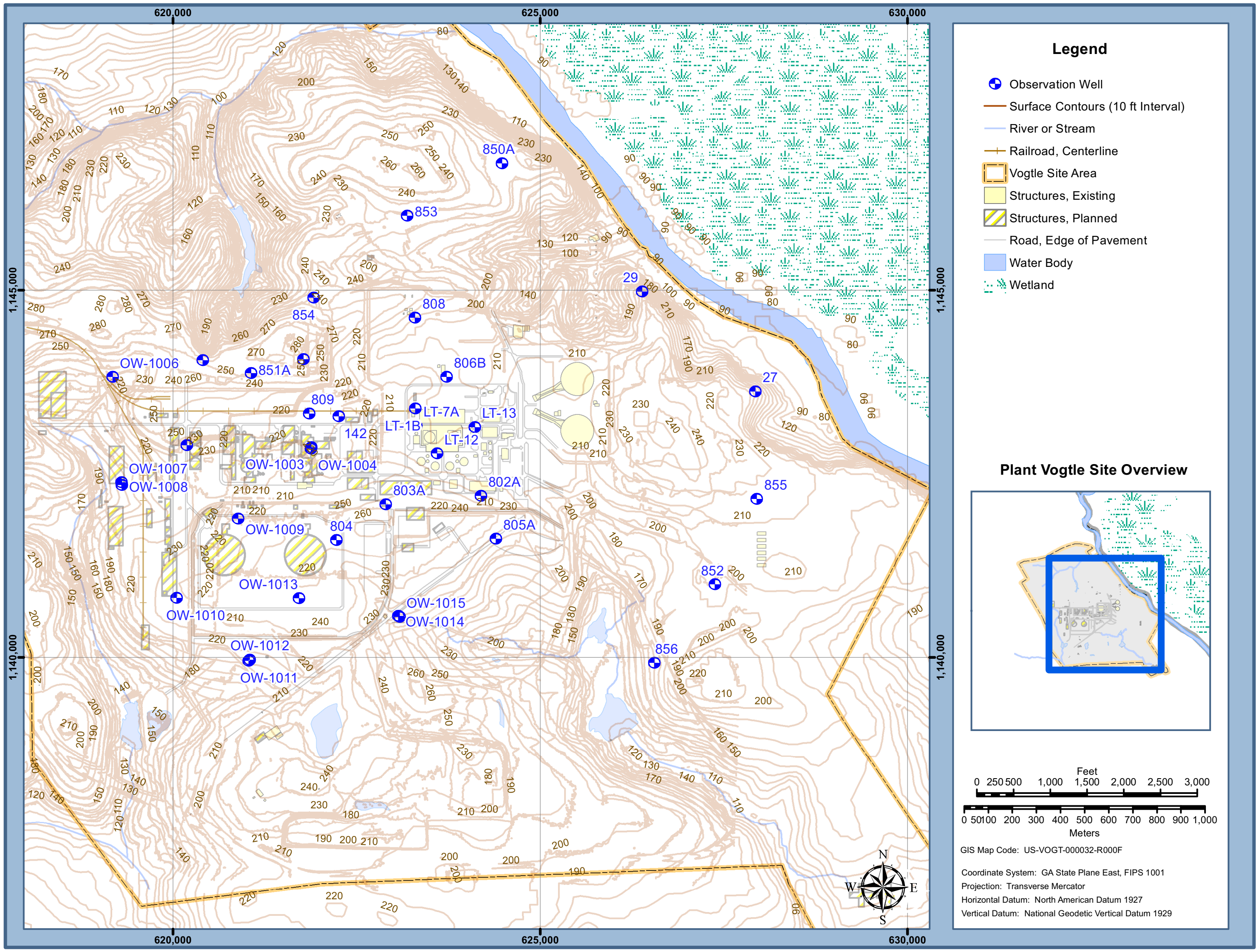


Figure 2.4-240
Observation Well Locations

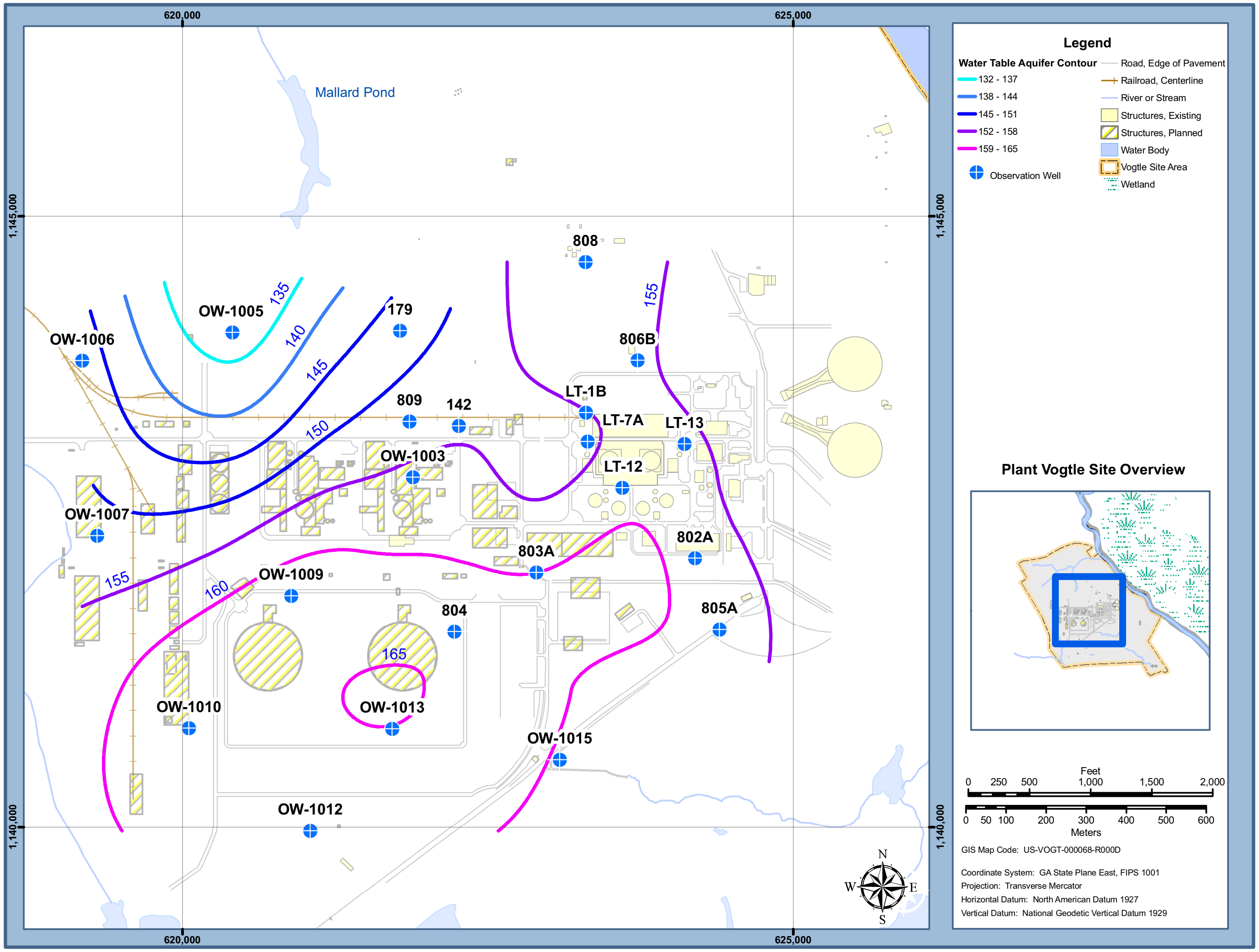


Figure 2.4-241
Water Table Aquifer: Piezometric Contour Map for June 2005

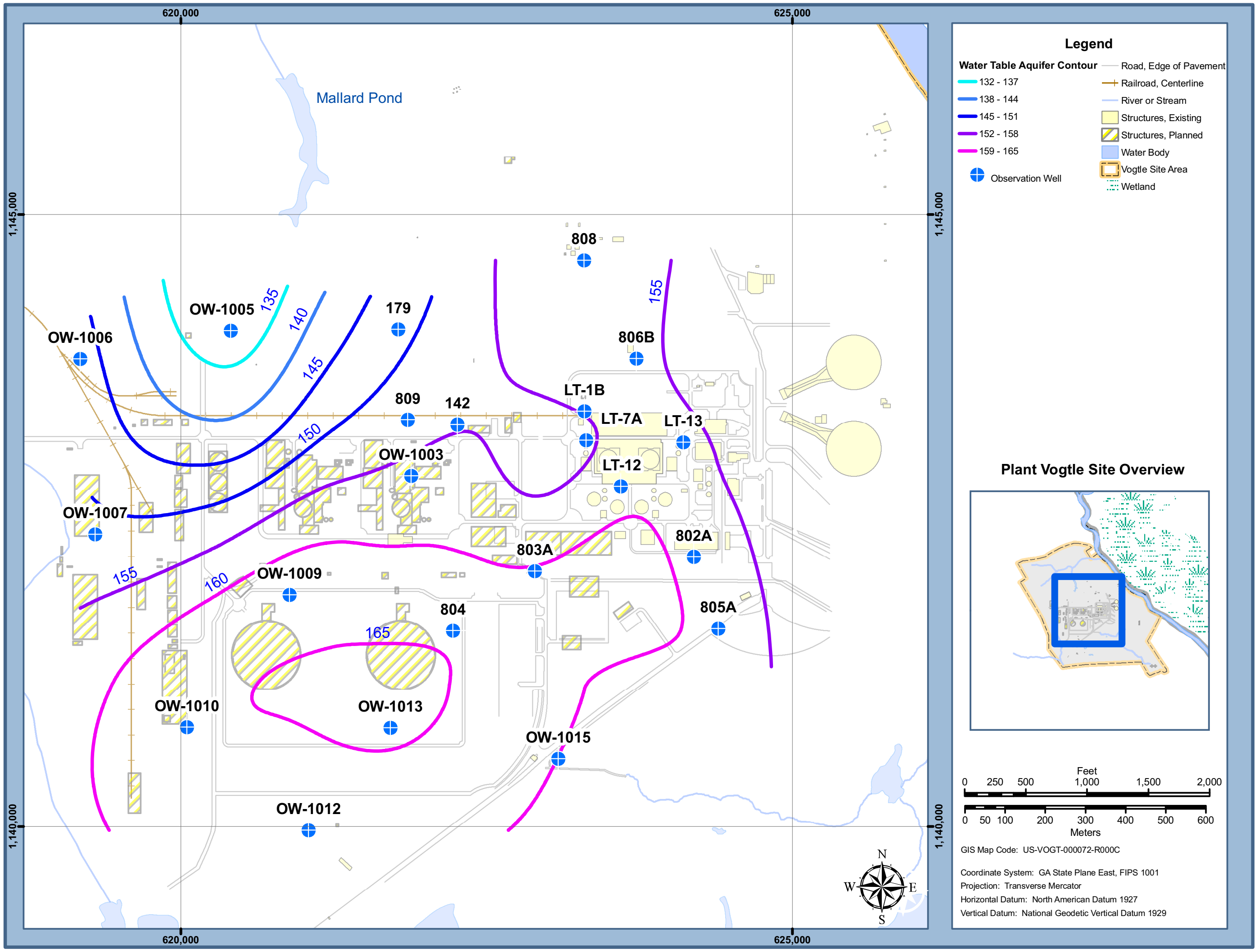


Figure 2.4-242
Water Table Aquifer: Piezometric Contour Map for October 2005

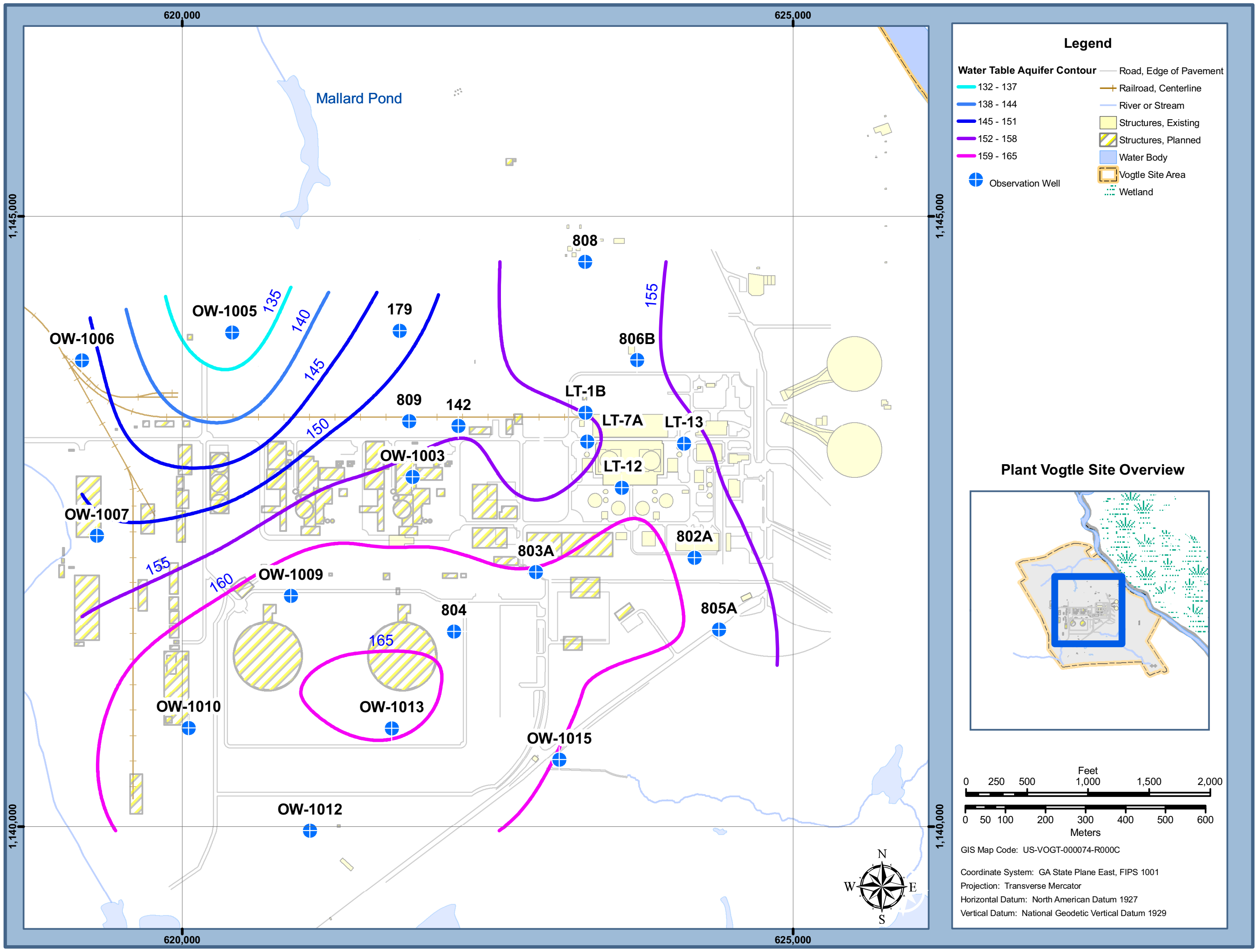


Figure 2.4-243
Water Table Aquifer: Piezometric Contour Map for December 2005

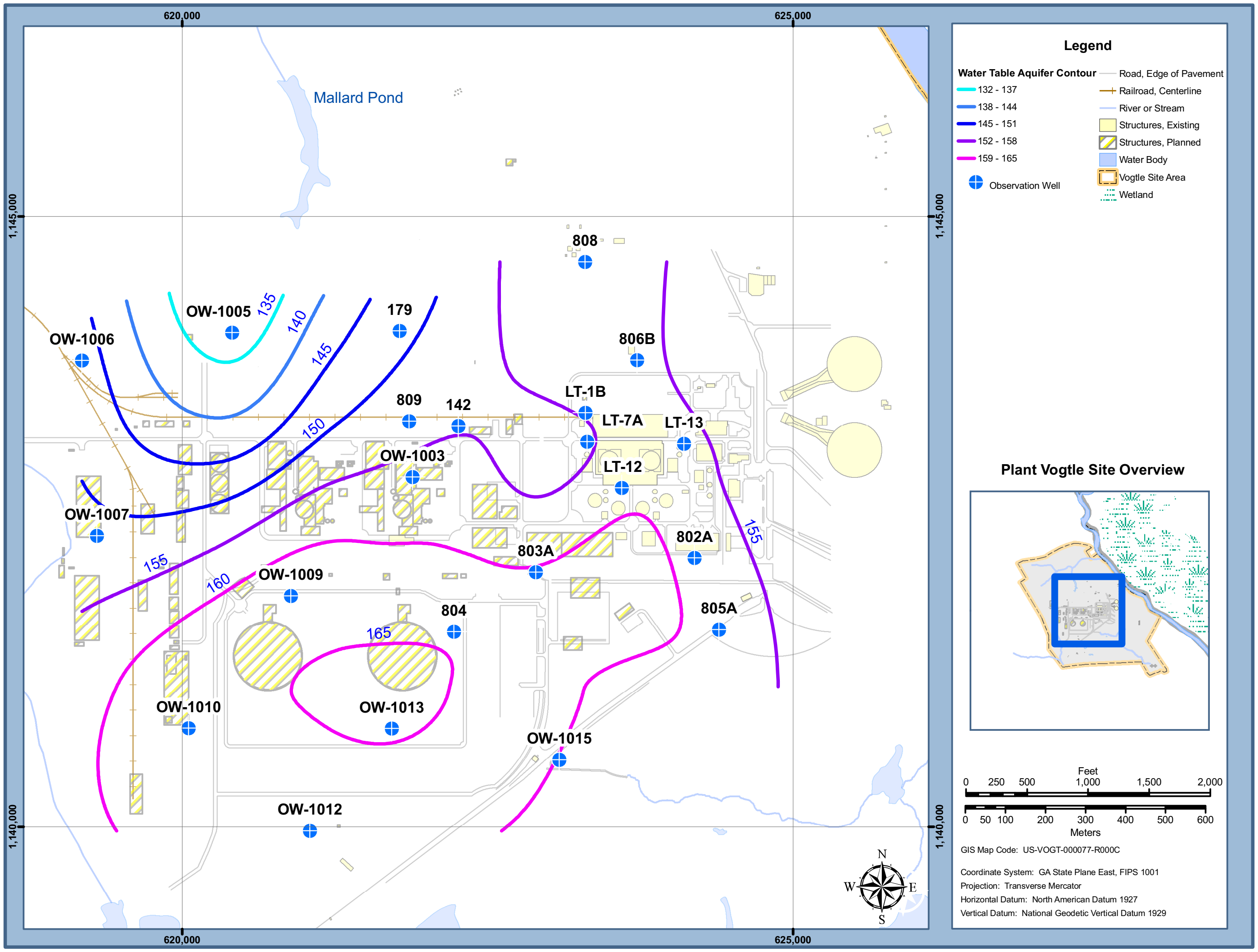


Figure 2.4-244
Water Table Aquifer: Piezometric Contour Map for March 2006

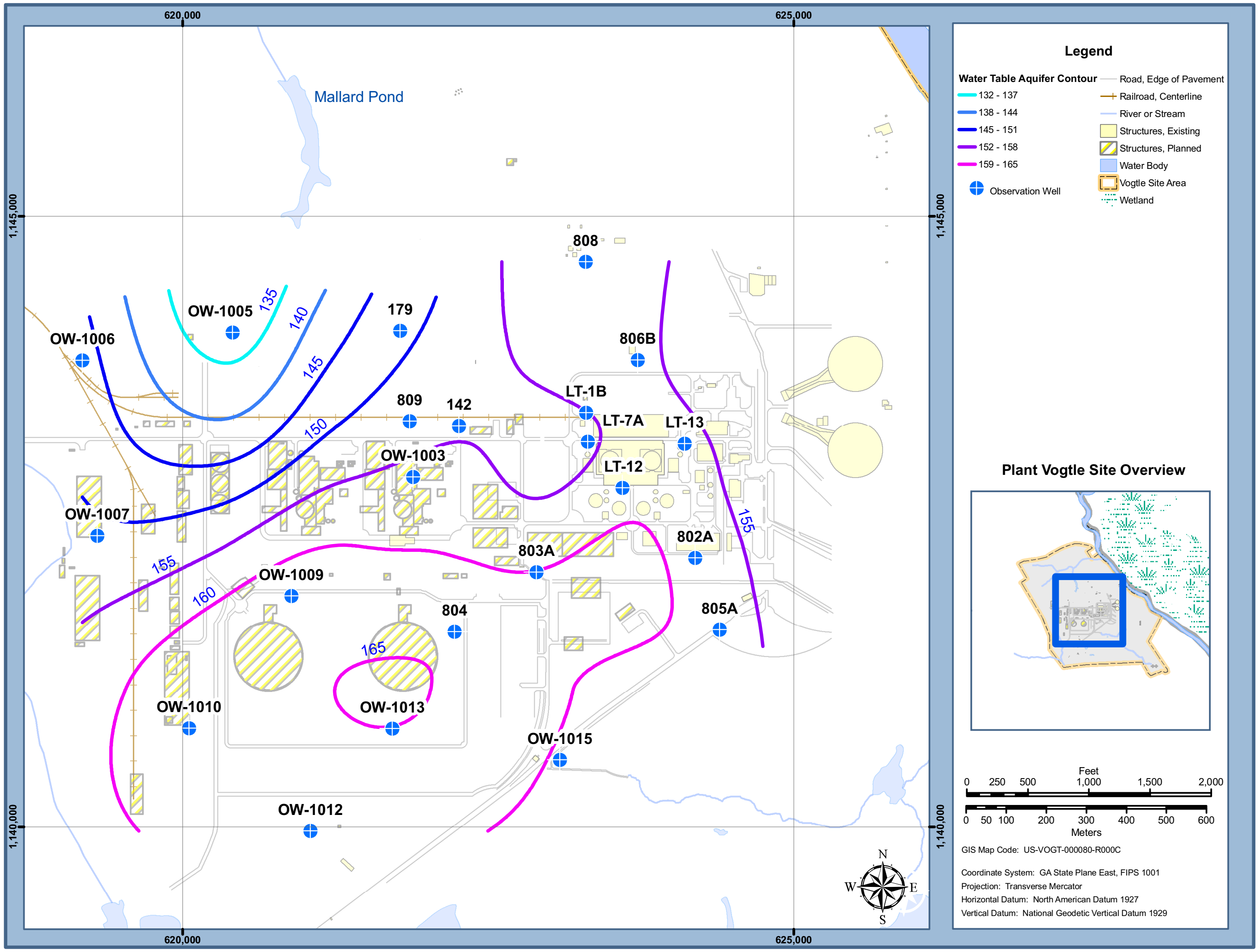


Figure 2.4-245
Water Table Aquifer: Piezometric Contour Map for June 2006

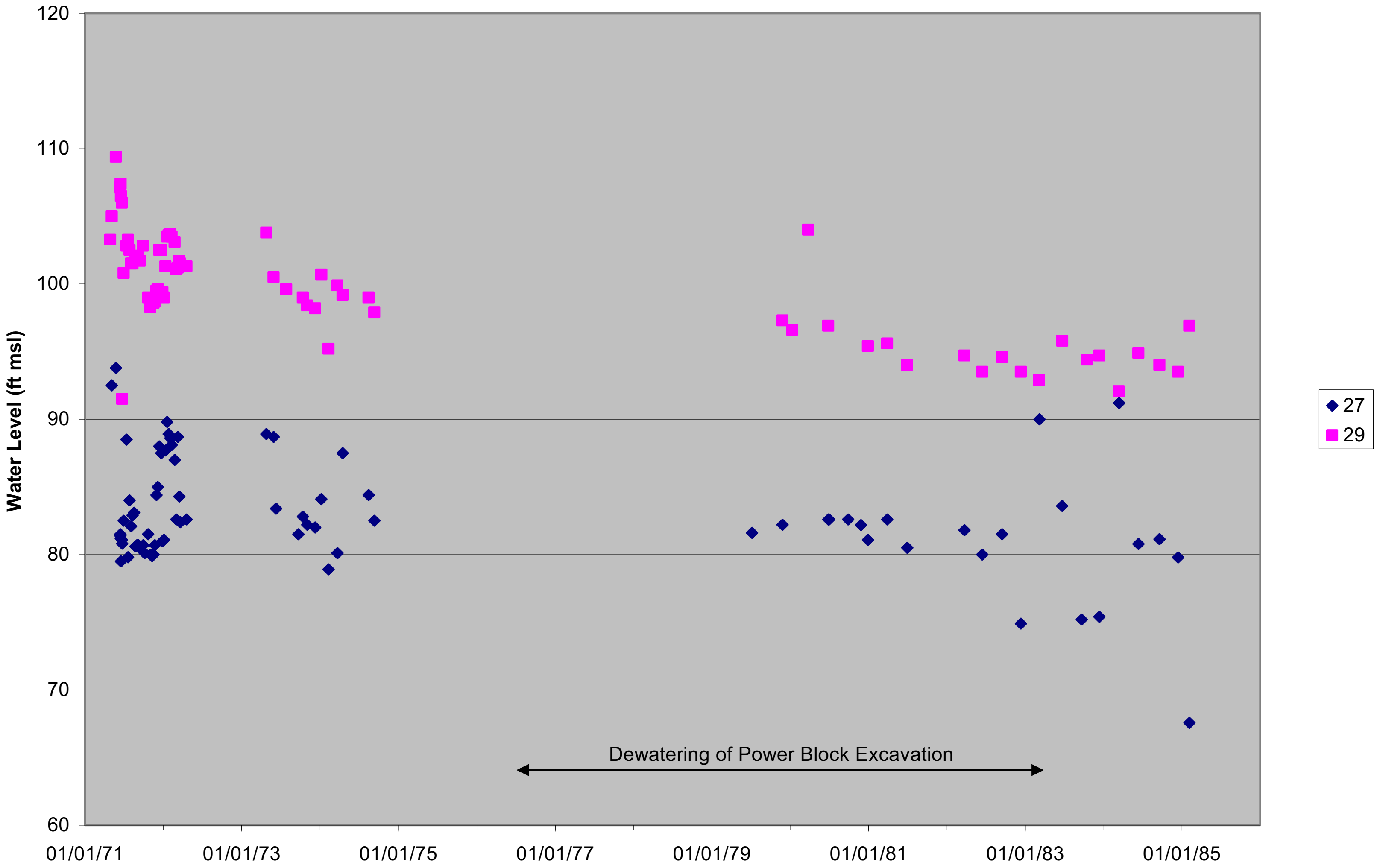


Figure 2.4-246
Tertiary Aquifer: 1971–1985 Hydrographs

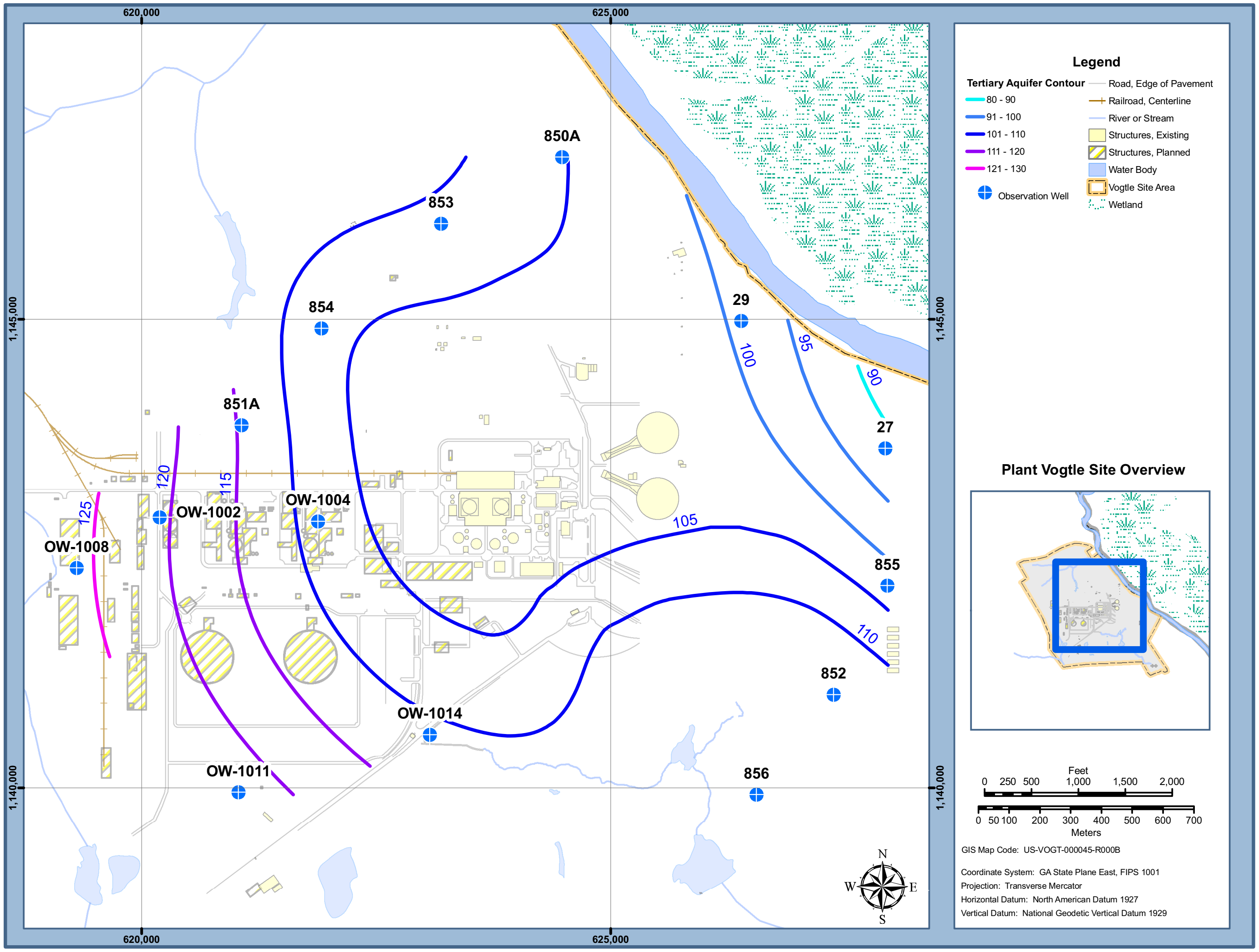


Figure 2.4-247
Tertiary Aquifer: Piezometric Contour Map for June 2005

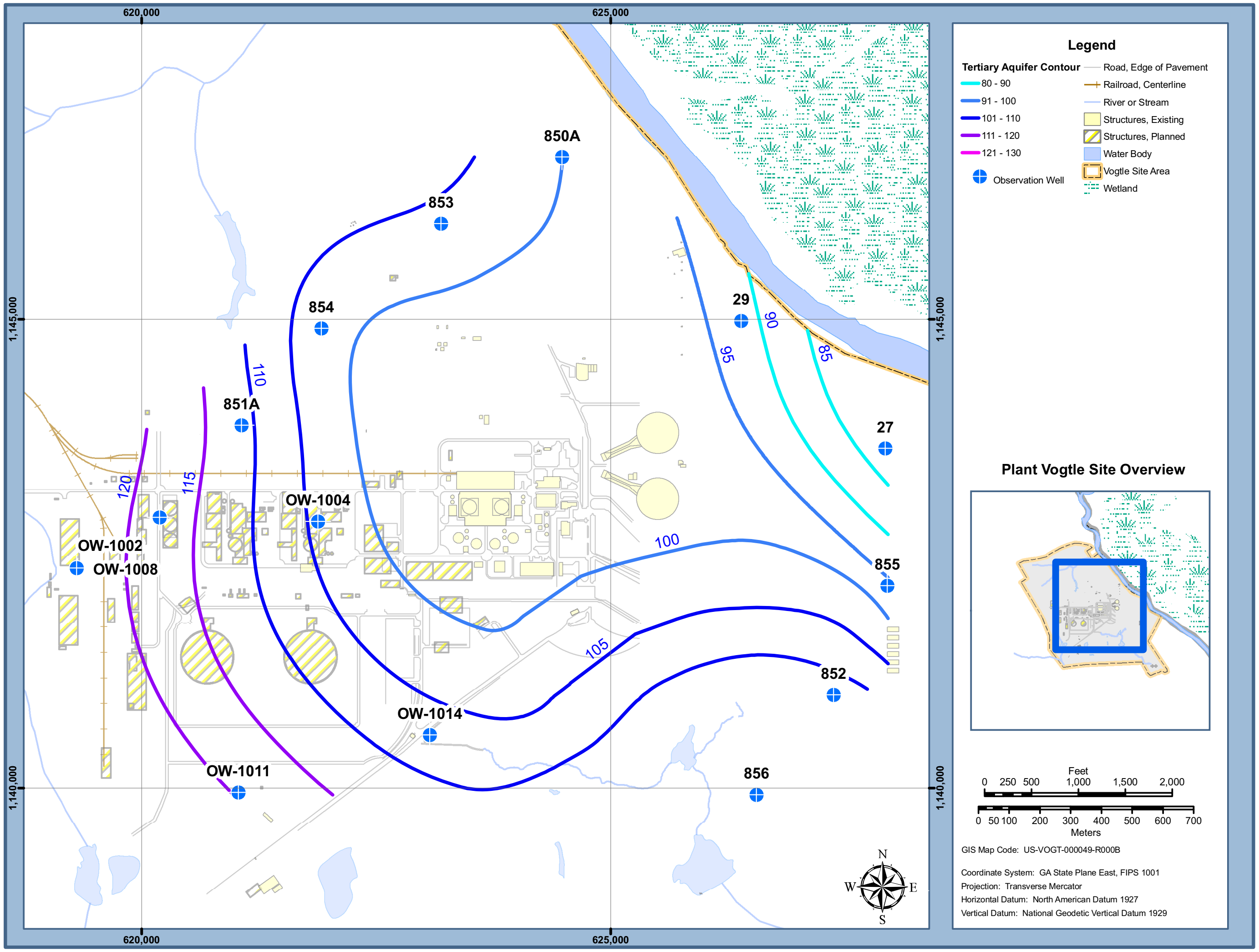


Figure 2.4-248
Tertiary Aquifer: Piezometric Contour Map for October 2005

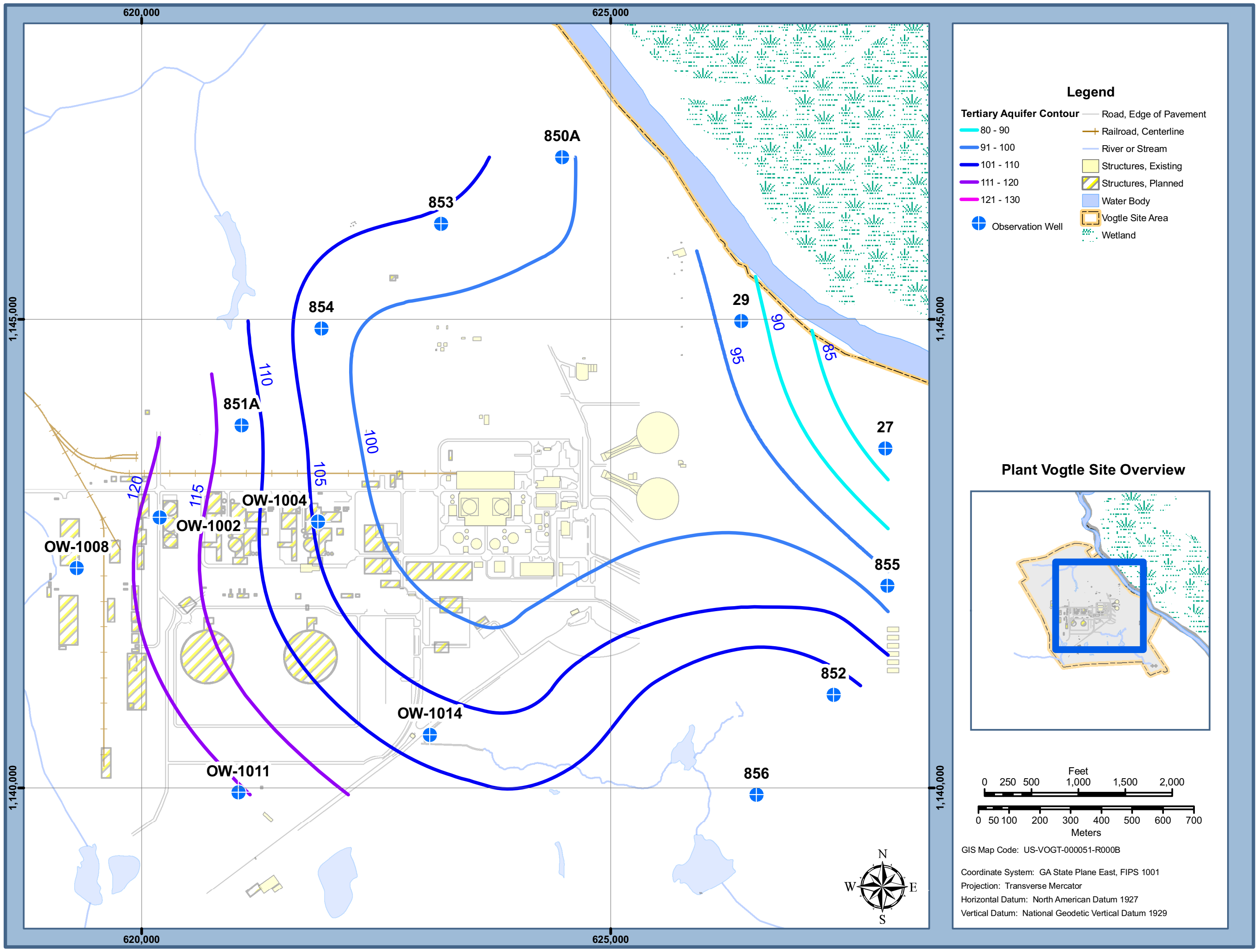


Figure 2.4-249
Tertiary Aquifer: Piezometric Contour Map for December 2005

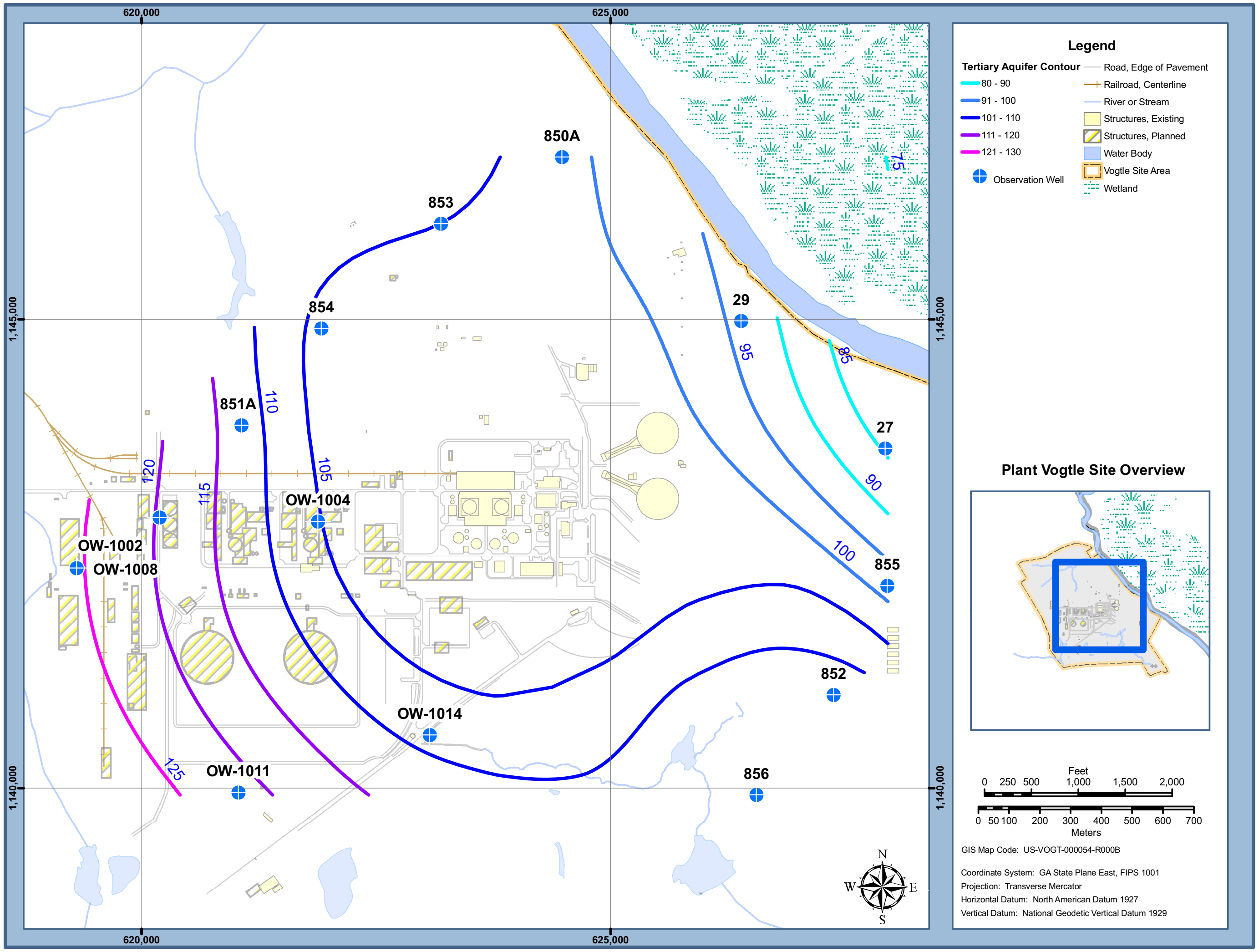


Figure 2.4-250
Tertiary Aquifer: Piezometric Contour Map for March 2006

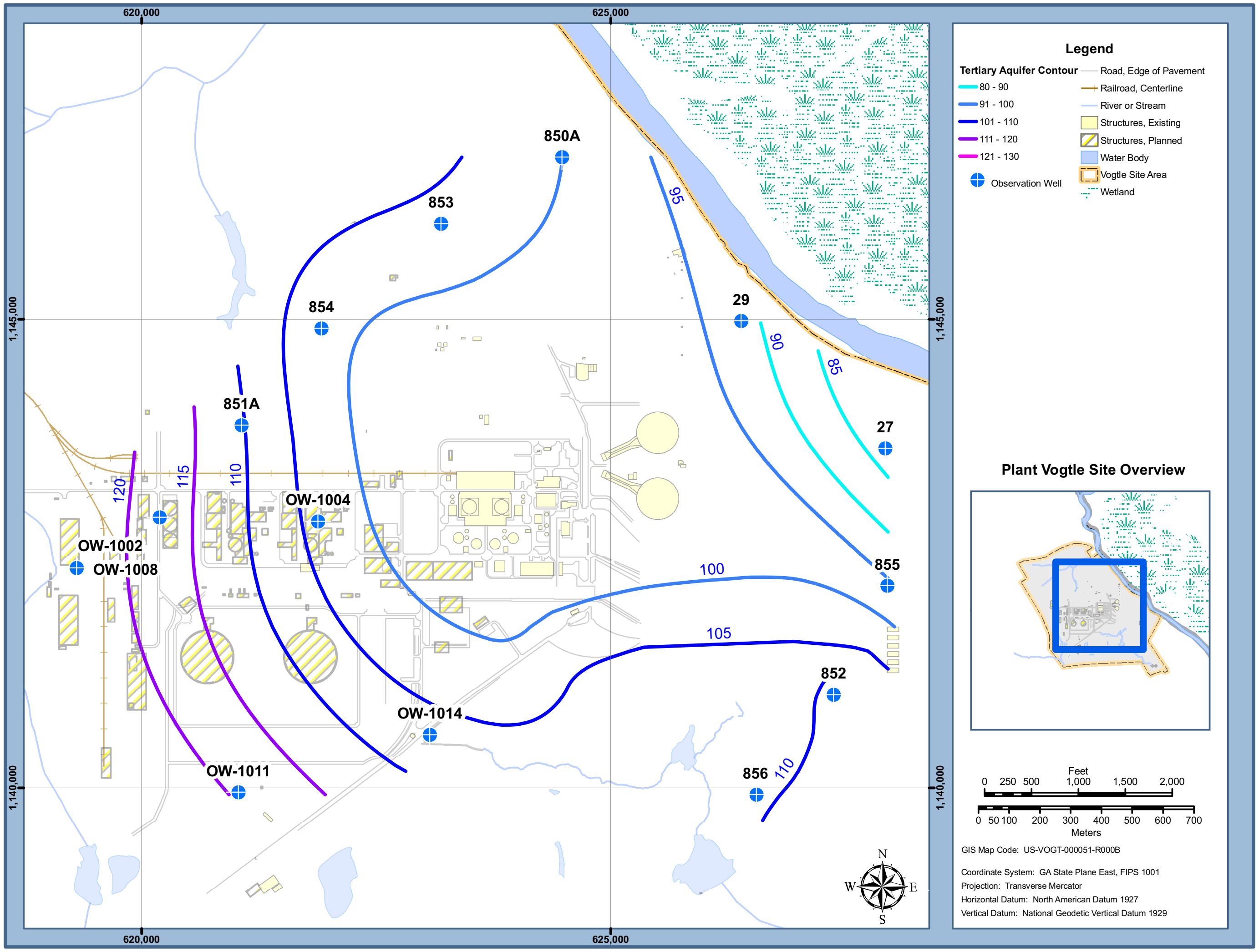


Figure 2.4-251
Tertiary Aquifer: Piezometric Contour Map for June 2006

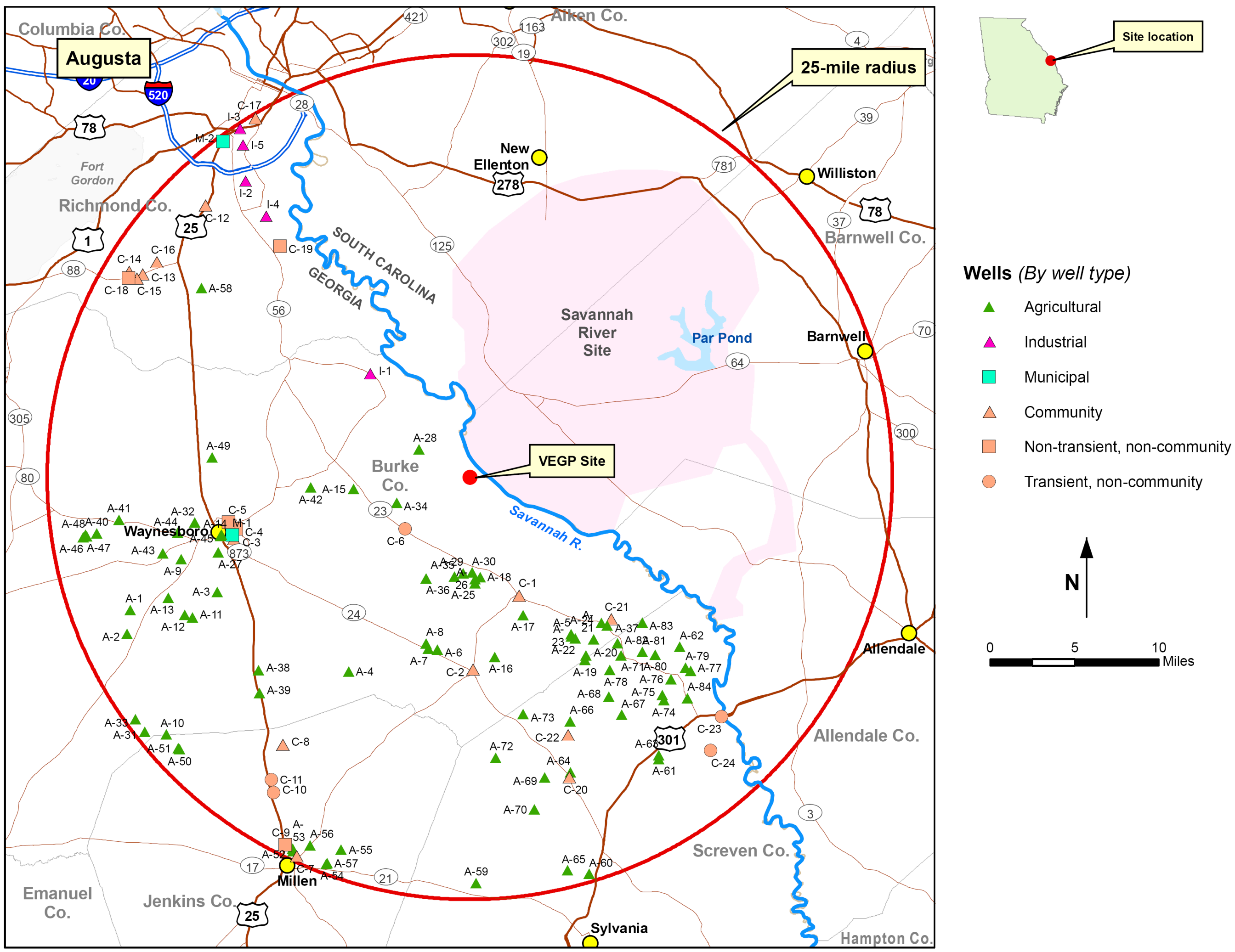


Figure 2.4-252
Locations of Agricultural, Industrial, Municipal, and Public Water Supply Wells Within 25 Miles of the VEGP Site

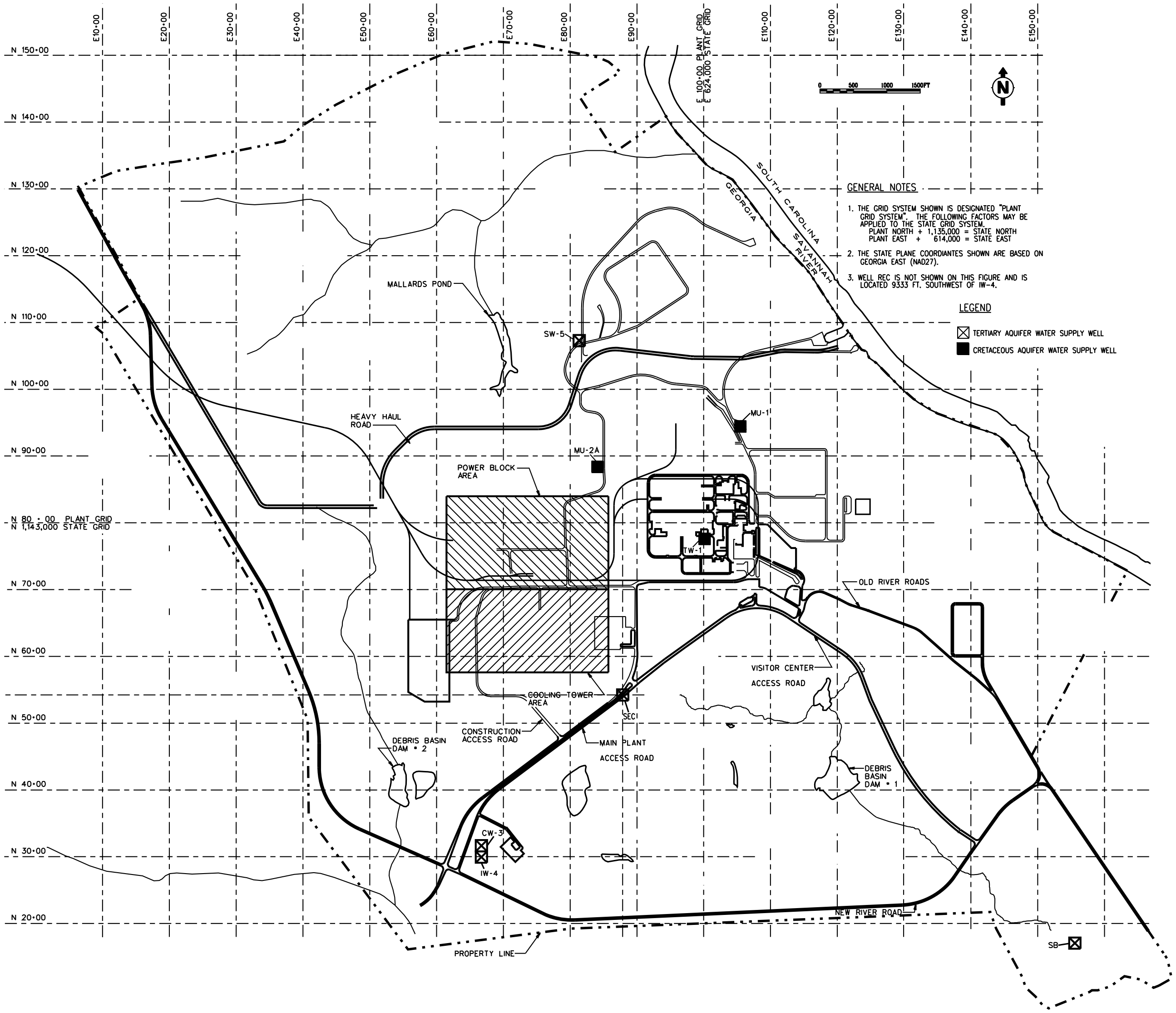


Figure 2.4-253
Locations of Existing Supply Wells at the VEGP Site

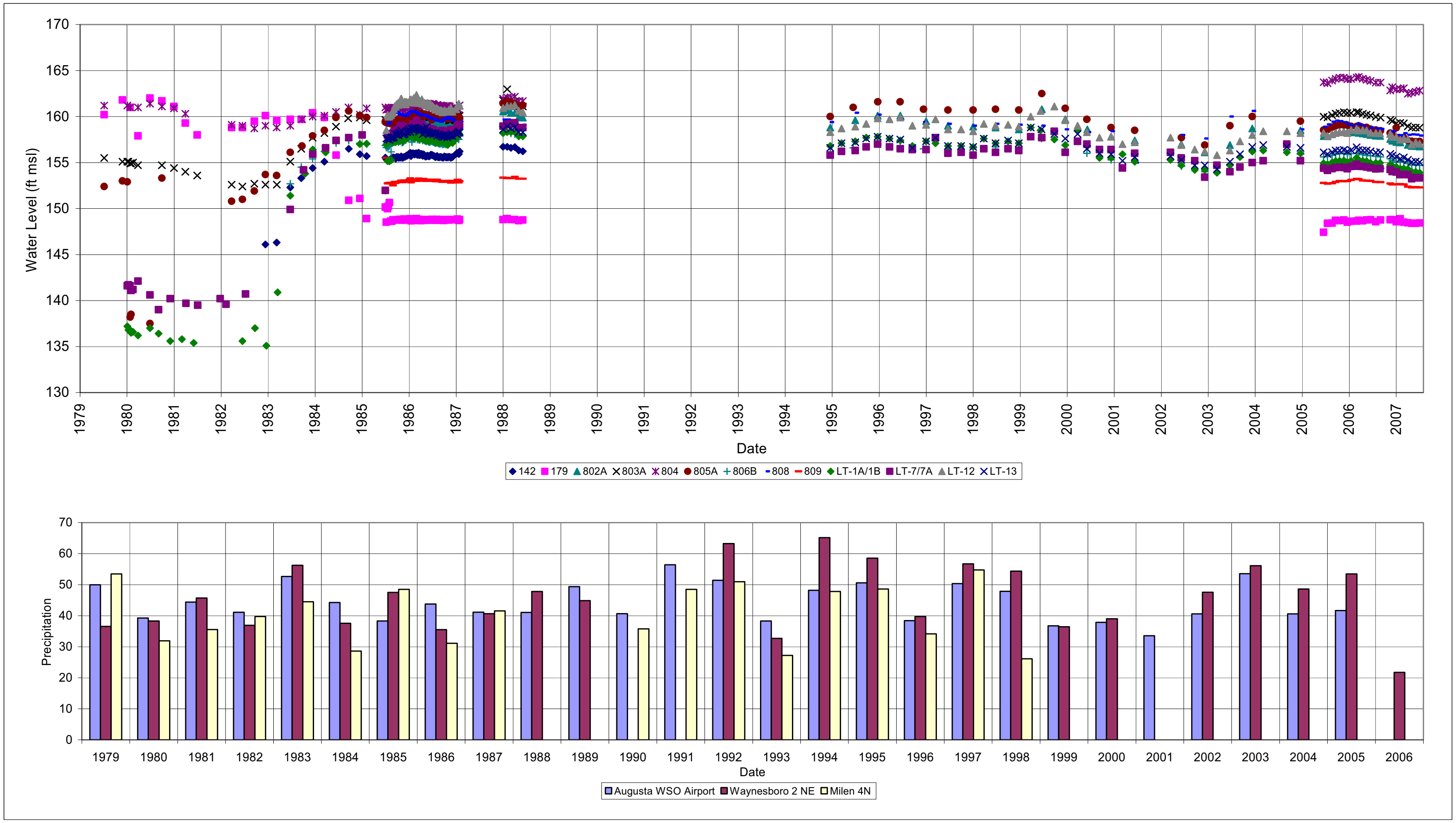


Figure 2.4-254
Water Table Aquifer: 1979–2007 Hydrographs

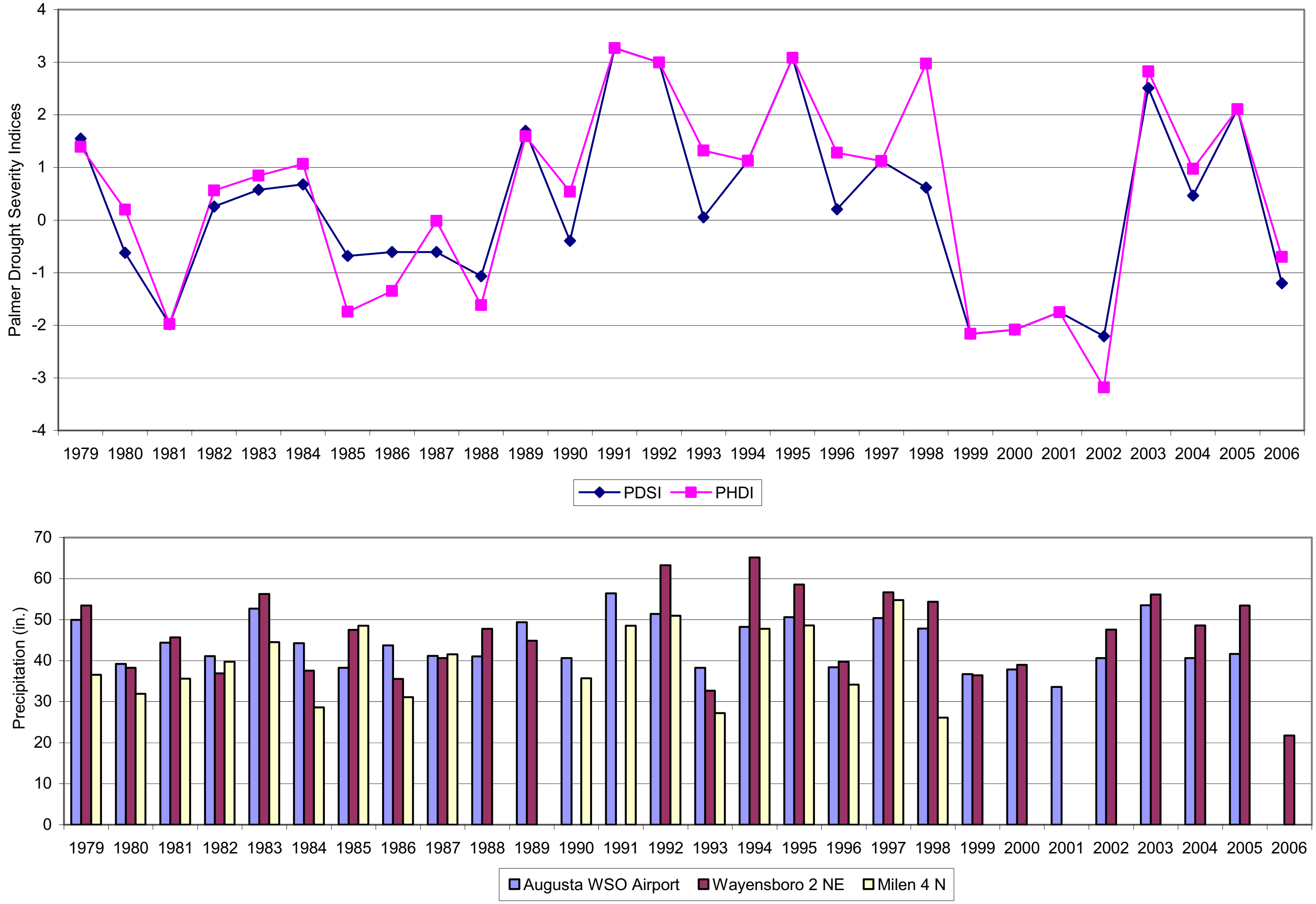


Figure 2.4-255
Average Annual PDSI and PHDI for Georgia and Total Annual Precipitation for the Period 1979–2006

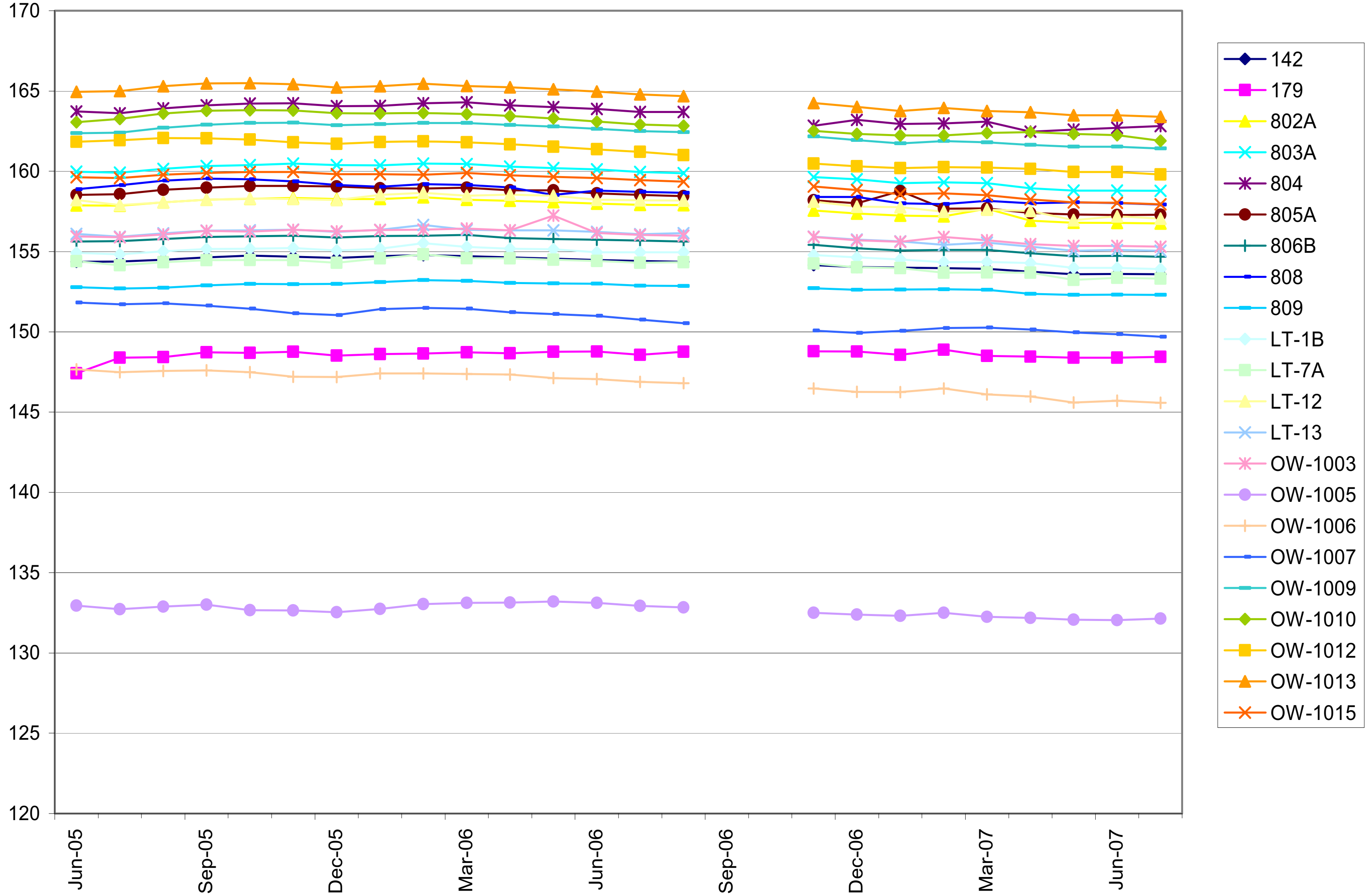


Figure 2.4-256
Water Table Aquifer: June 2005 – July 2007 Hydrographs

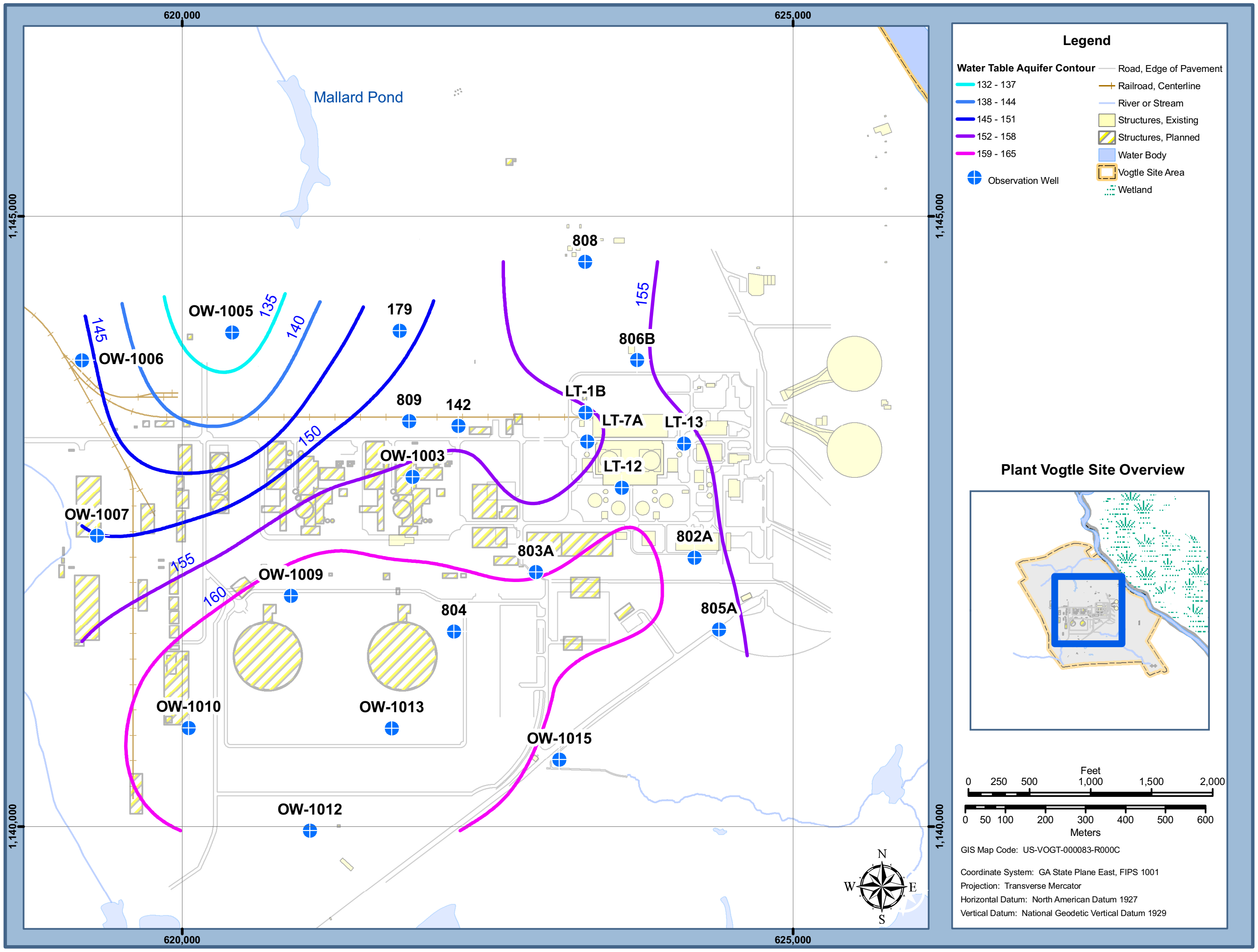


Figure 2.4-257
Water Table Aquifer: Piezometric Contour Map for November 2006

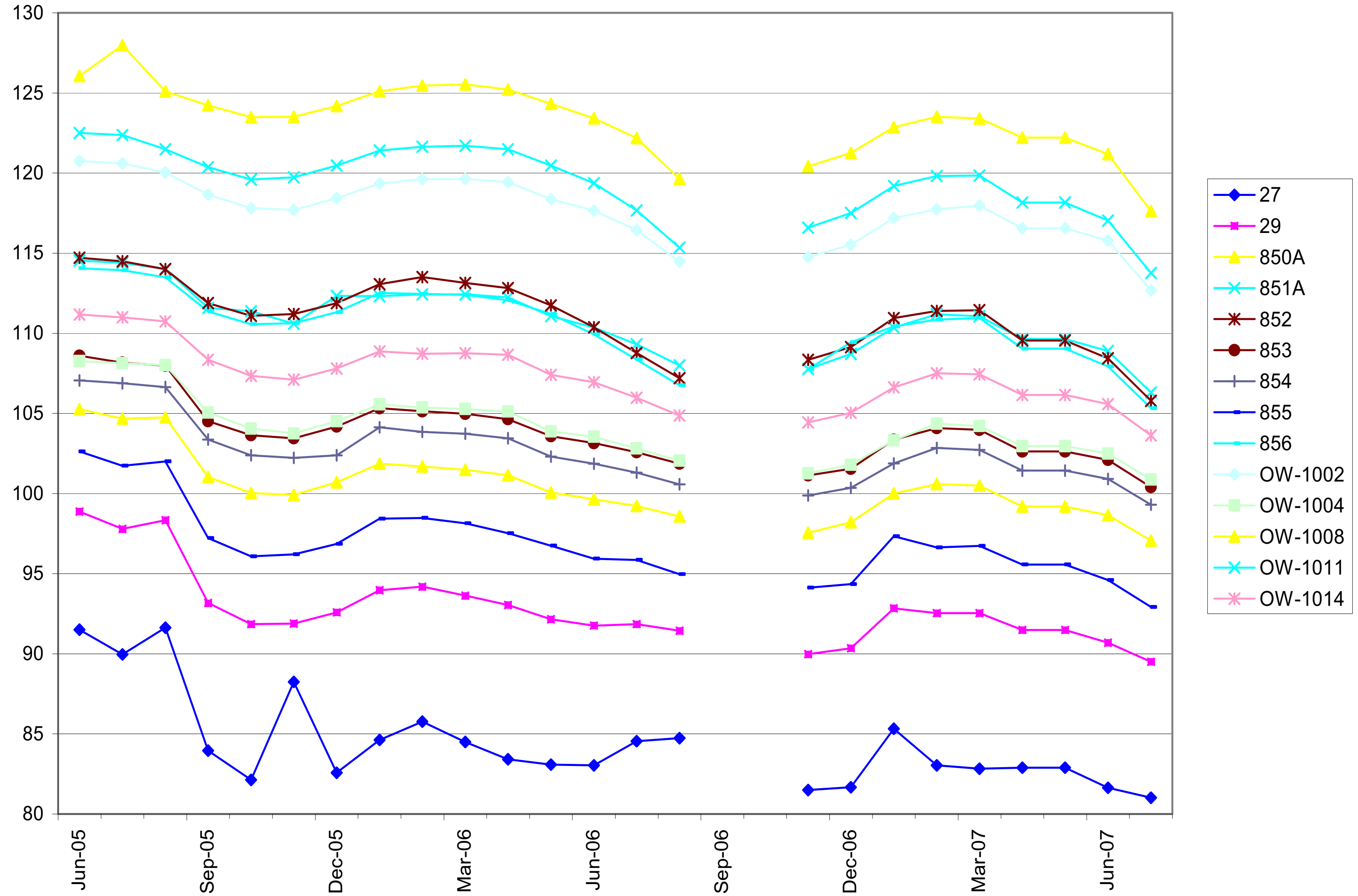


Figure 2.4-258
Tertiary Aquifer: June 2005 – July 2007 Hydrographs

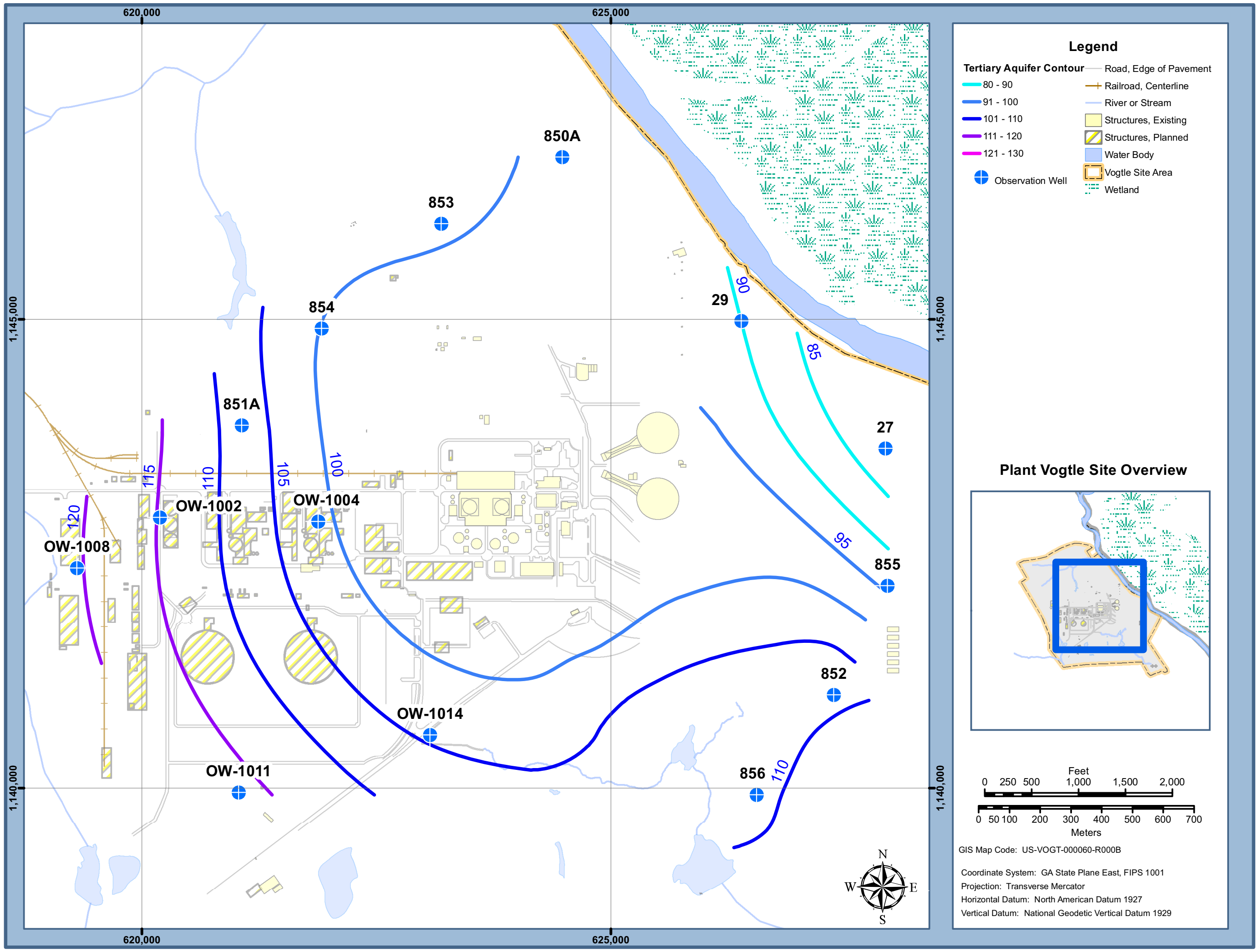


Figure 2.4-259
Tertiary Aquifer: Piezometric Contour Map for November 2006

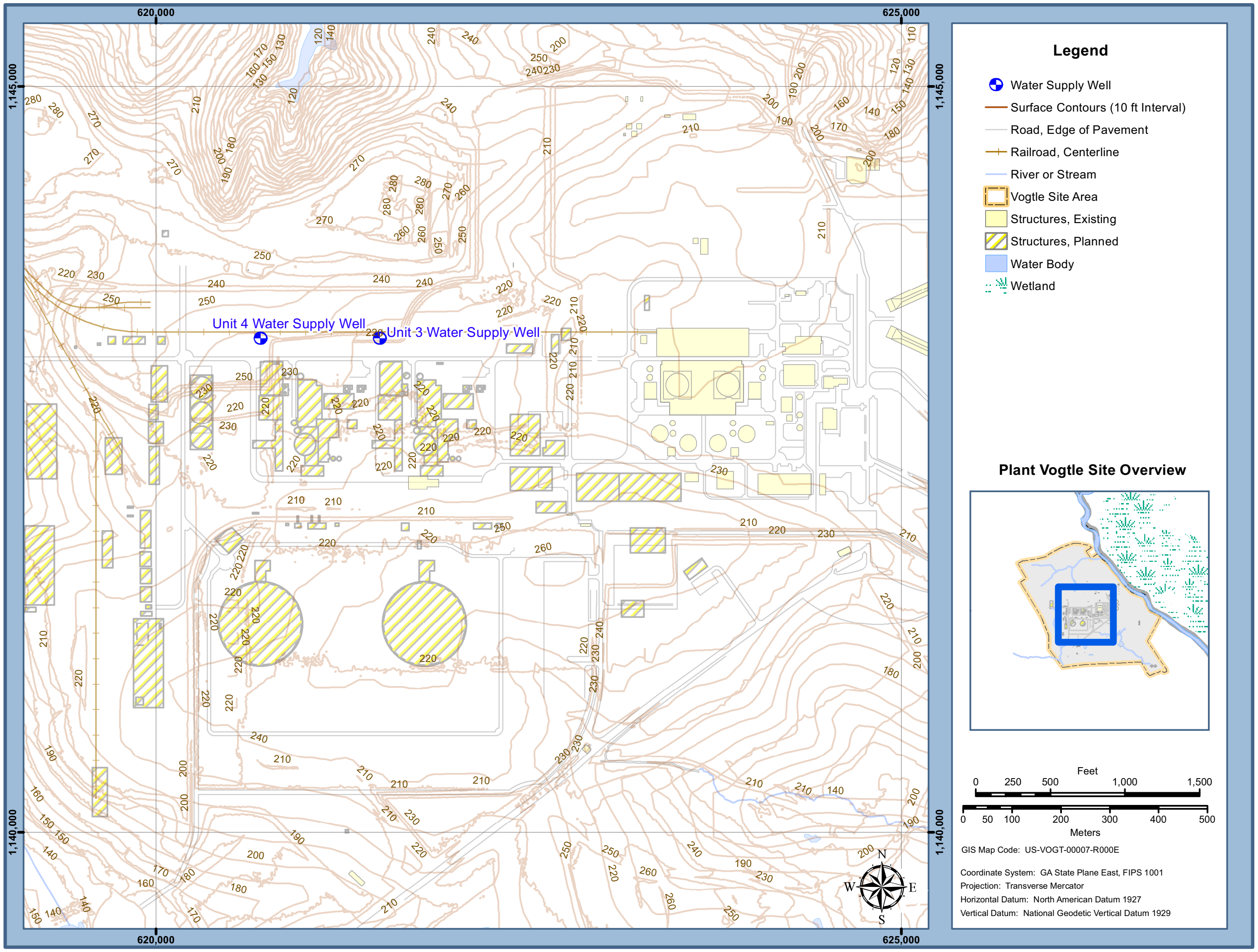


Figure 2.4-260
Proposed Locations of VEGP Units 3 and 4 Water Supply Wells

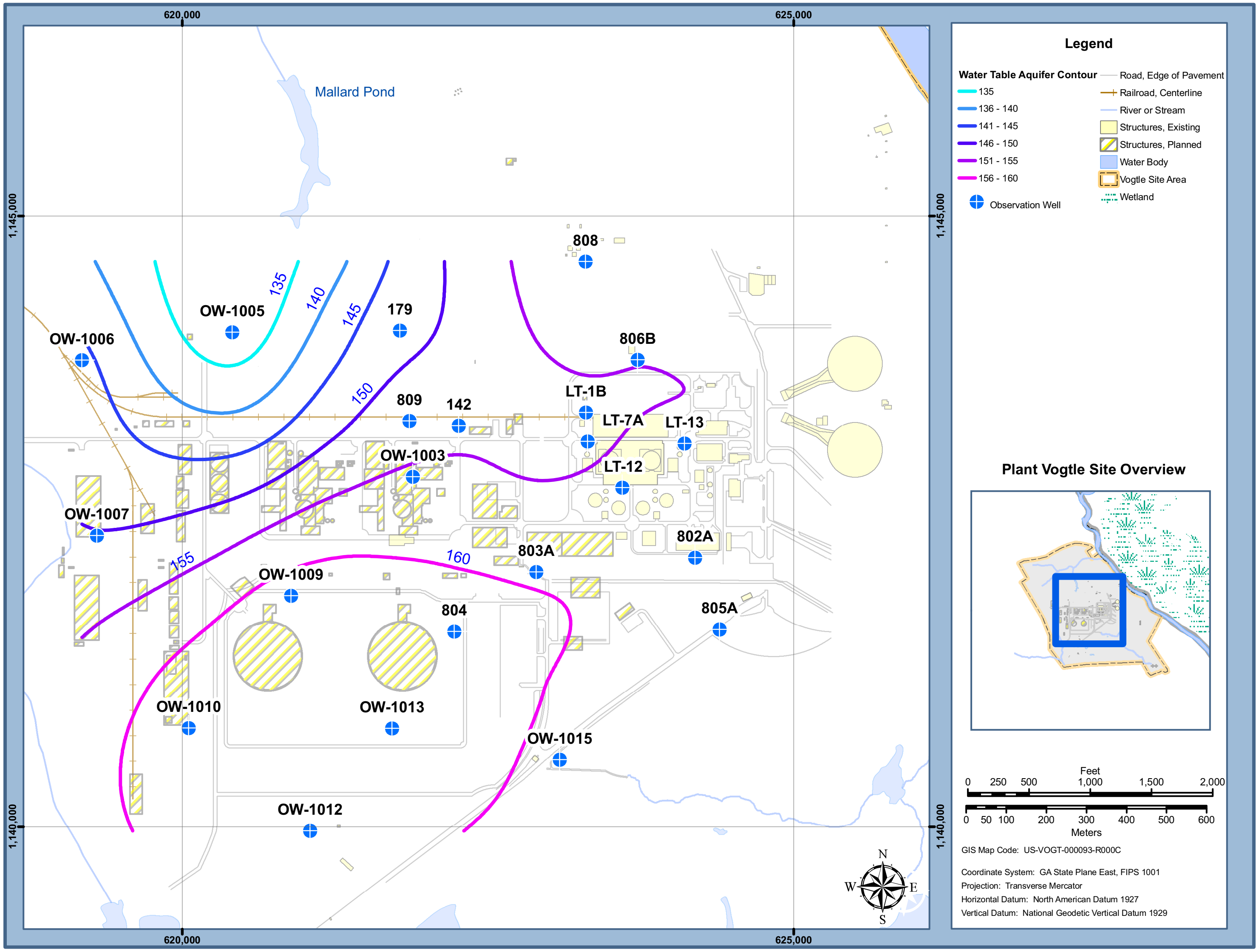


Figure 2.4-261
Water Table Aquifer — Piezometric Contour Map for March 2007

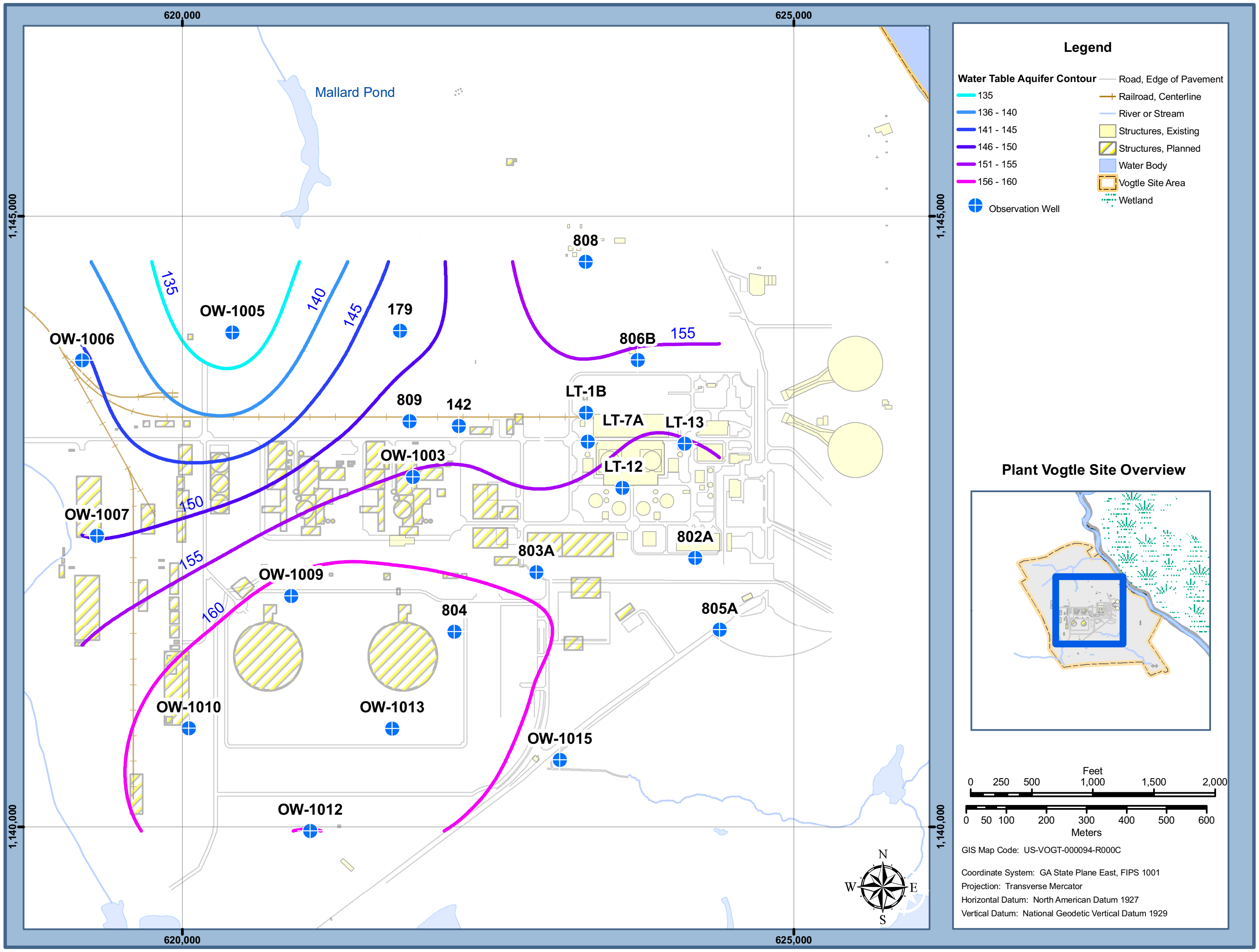


Figure 2.4-262
Water Table Aquifer — Piezometric Contour Map for June 2007

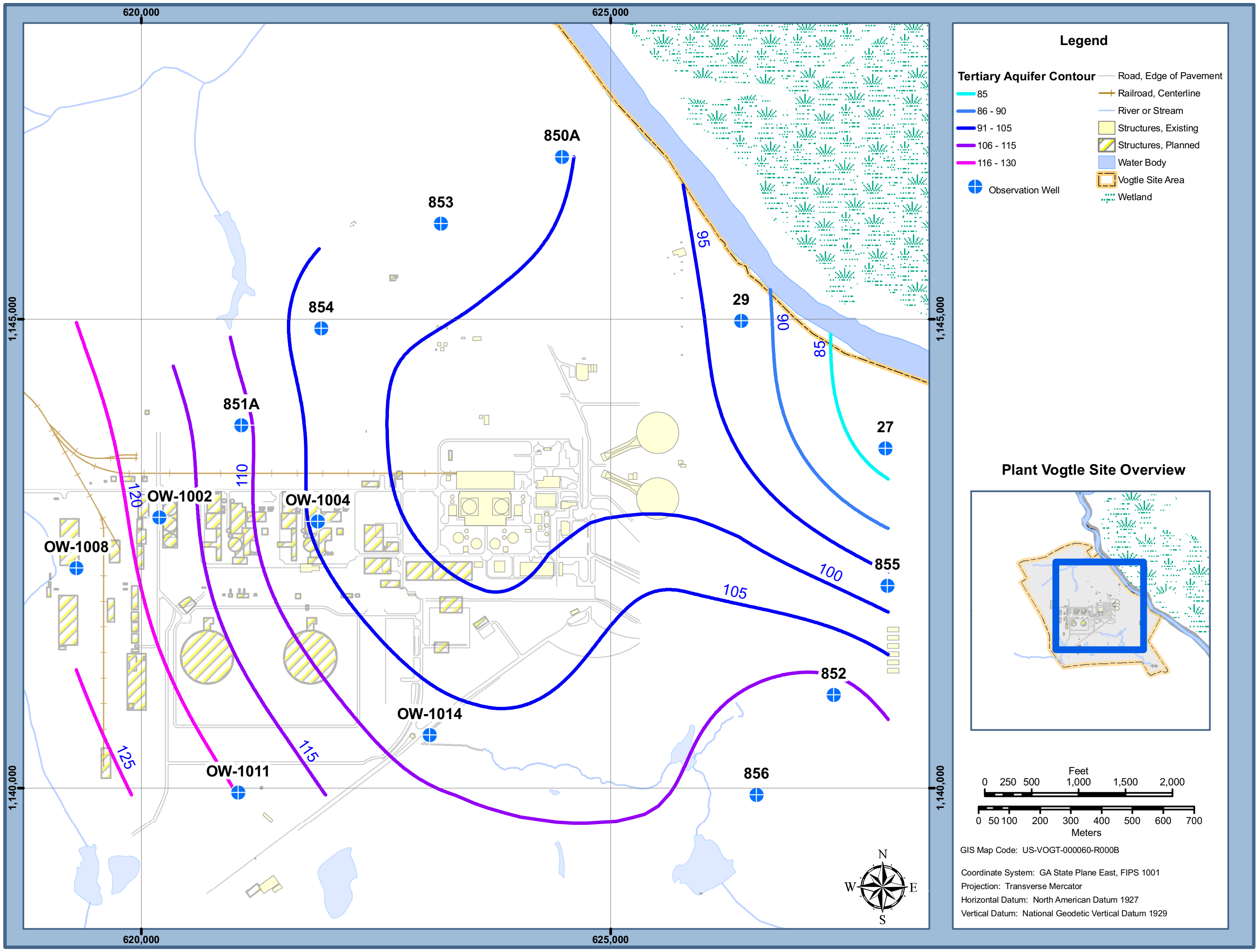


Figure 2.4-263
Tertiary Aquifer — Piezometric Contour Map for March 2007

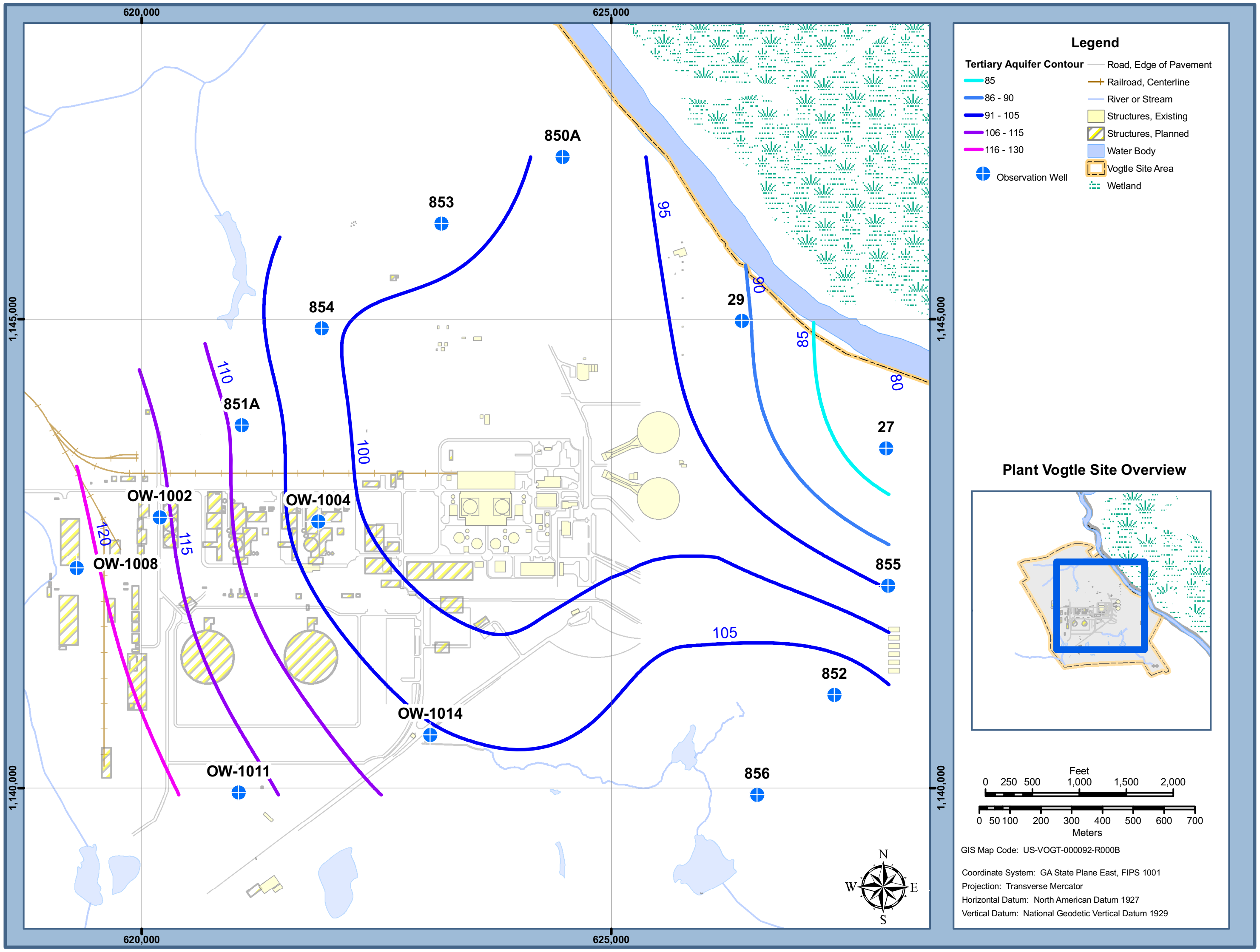


Figure 2.4-264
Tertiary Aquifer — Piezometric Contour Map for June 2007

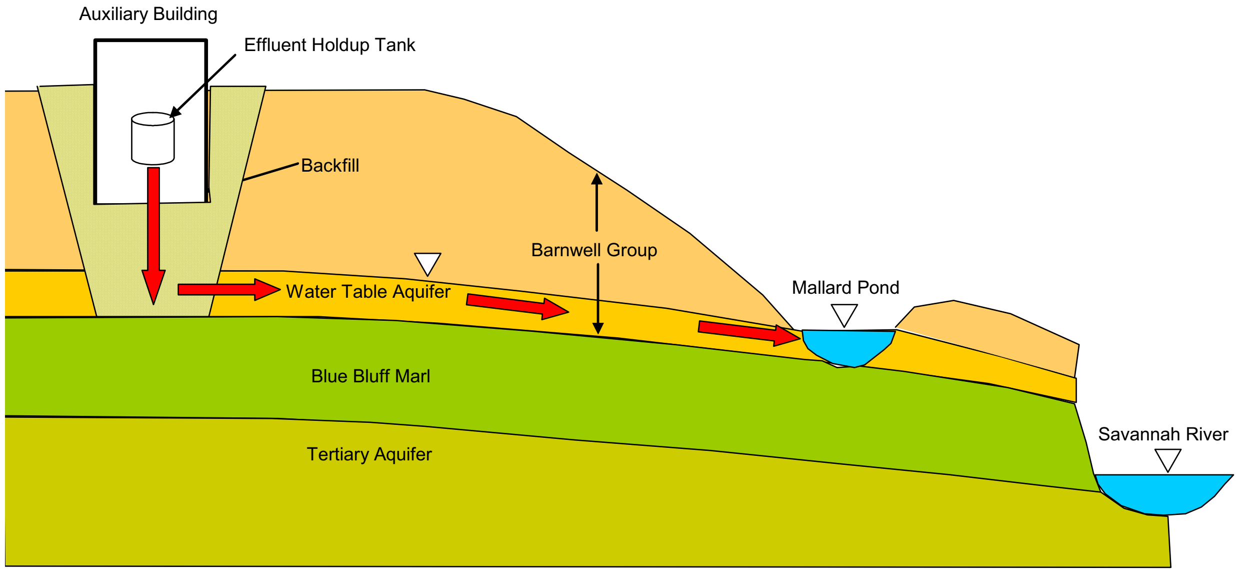


Figure 2.4-265
Conceptual Model for Evaluating Radionuclide Transport in the Water Table Aquifer

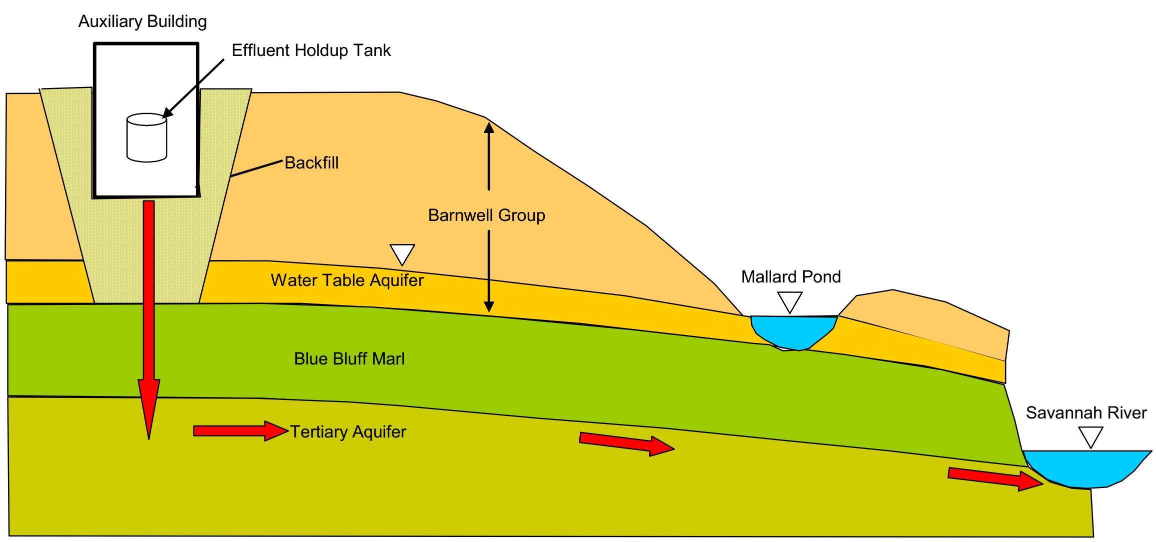


Figure 2.4-266
Conceptual Model for Evaluating Radionuclide Transport in the Tertiary Aquifer