CHAPTER 2

SITE CHARACTERISTICS

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CHAPTER 2

SITE CHARACTERISTICS

The introductory information at the beginning of Chapter 2 of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Insert the following subsection at the end of the introductory text of DCD Chapter 2, prior to DCD Section 2.1.

2.0 SITE CHARACTERISTICS

BLN SUP 2.0-1 Chapter 2 describes the characteristics and site-related design parameters of the Bellefonte Nuclear Plant, Units 3 and 4 (BLN). 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)

In this chapter, the following terms are used to describe the BLN site and surrounding area:

- BLN site – the 1600 acre site located within the BLN property line. See Figure 2.1-201.
- BLN vicinity – the area within a radius of approximately six miles around the BLN site. See Figure 2.1-202.
- BLN region – the area within a radius of approximately 50 miles around the BLN site. See Figure 2.1-203.

Table 2.0-201 provides a comparison of site-related design parameters for which the AP1000 plant is designed and site characteristics specific to BLN in support of this safety assessment. The first two columns of Table 2.0-201 are a compilation of the site parameters from DCD Table 2-1 and DCD Tier 1 Table 5.0-1. The third column of Table 2.0-201 is the corresponding site characteristic for the BLN. The fourth column denotes the place within the BLN FSAR that 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, while “No” indicates it does not. Where a “No” is indicated, justification for the exceedance is provided in the FSAR reference (fourth column of Table 2.0-201). 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.

BLN SUP 2.0-1

TABLE 2.0-201 (Sheet 1 of 5)
COMPARISON OF AP1000 DCD SITE PARAMETERS AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4 SITE CHARACTERISTICS

	AP1000 DCD Site Parameter[a]	BLN Site Characteristic	BLN FSAR Reference	BLN Within Site Parameter
Air Temperature				
Maximum Safety[b]	115°F dry bulb / 80°F coincident wet bulb	94°F dry bulb / 75°F coincident wet bulb (0.4 % exceedance)	Table 2.3-203	Yes
	85.5°F wet bulb (noncoincident)	78°F wet bulb (noncoincident) (0.4 % exceedance)	Table 2.3-203	Yes
Minimum Safety[b]	-40°F	15°F (0.4 % exceedance)	Table 2.3-203	Yes
Maximum Normal[c]	100°F dry bulb / 80.1°F coincident wet bulb	92°F dry bulb / 74°F coincident wet bulb (1% exceedance)	Table 2.3-203	Yes
	80.1°F wet bulb (noncoincident)[d]	77°F wet bulb (noncoincident) (1% exceedance)	Table 2.3-203	Yes
Minimum Normal[c]	-10°F	20°F (1% exceedance)	Table 2.3-203	Yes
Wind Speed				
Operating Basis	145 mph (3 second gust); importance factor 1.15 (safety), 1.0 (nonsafety); exposure C; topographic factor 1.0	96 mph (3 second gust); exposure C; topographic factor 1.0. (Importance factor is not a property of the wind speed.)	Subsection 2.3.1.5	Yes
Tornado	300 mph	230 mph	Subsection 2.3.1.4	Yes
	Maximum pressure differential of 2.0 lb/in^2	1.2 lb/in^2	Subsection 2.3.1.4	Yes

BLN SUP 2.0-1

TABLE 2.0-201 (Sheet 2 of 5)
COMPARISON OF AP1000 DCD SITE PARAMETERS AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4 SITE CHARACTERISTICS

	AP1000 DCD Site Parameter[(a)]	BLN Site Characteristic	BLN FSAR Reference	BLN Within Site Parameter
Seismic				
SSE	SSE free field peak ground acceleration of 0.30 g with modified Regulatory Guide 1.60 response spectra. Seismic input is defined at finished grade, except for sites where the nuclear island is founded on hard rock.	Peak ground acceleration = 0.24g High frequency exceedances of the horizontal ground motion response spectra has been evaluated by Westinghouse and these exceedances are within the seismic design margin of the AP1000 and will not adversely affect the systems, structures or components of the plant.	Subsection 3.7.1.1.1 Figure 3.7-201	Yes
Fault Displacement Potential	Negligible	Negligible.	Subsection 2.5.3.8	Yes
Soil				
Average Allowable Static Bearing Capacity	Greater than or equal to 8,600 lb/ft^2 over the footprint of the nuclear island at its excavation depth	236,000 to 251,000 lb/ft^2	Subsection 2.5.4.10.1	Yes
Maximum Allowable Dynamic Bearing Capacity for Normal Plus Safe Shutdown Earthquake (SSE)	Greater than or equal to 35,000 lb/ft^2 at the edge of the nuclear island at its excavation depth	236,000 to 251,000 lb/ft^2	Subsection 2.5.4.10.1	Yes

BLN SUP 2.0-1

TABLE 2.0-201 (Sheet 3 of 5)
COMPARISON OF AP1000 DCD SITE PARAMETERS AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4 SITE CHARACTERISTICS

	AP1000 DCD Site Parameter[a]	BLN Site Characteristic	BLN FSAR Reference	BLN Within Site Parameter
Shear Wave Velocity	Greater than or equal to 1,000 ft/sec based on low-strain best-estimate soil properties over the footprint of the nuclear island at its excavation depth	Greater than 9,200 ft/sec	Subsection 2.5.4.4.3.2	Yes
Lateral Variability	Soils supporting the nuclear island should not have extreme variations in the subgrade stiffness			
	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 in any layer.	Case 1 applies: dip not greater than 20 degrees and less than 20 percent variation in the shear wave velocity from the average shear wave velocity in any layer.	Subsections 2.5.4.1 and 2.5.4.7	Yes
	Case 2: For a layer with a low strain shear wave velocity less than 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 10 percent variation in the shear wave velocity from the average velocity in any layer (see DCD Subsection 2.5.4.5).	N/A	N/A	N/A
Liquefaction Potential	None.	None. Foundations of Seismic Category 1 structures are on rock	Subsection 2.5.4.8	Yes

TABLE 2.0-201 (Sheet 4 of 5)

BLN SUP 2.0-1

COMPARISON OF AP1000 DCD SITE PARAMETERS AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4 SITE CHARACTERISTICS

	AP1000 DCD Site Parameter[a]	BLN Site Characteristic	BLN FSAR Reference	BLN Within Site Parameter
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 DCD Section 3.5 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 maximum flood level is plant elevation 93.9 feet or 622.5 feet above mean sea level.	Subsection 2.4.2.2	Yes
Ground Water Level	Less than plant elevation 98 feet	The maximum static groundwater level in the vicinity of Units 3 and 4 power blocks is plant elevation 86 feet, or 614.6 feet mean sea level.	Subsection 2.4.12.5	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 100 feet (628.6 feet above mean sea level); the plant grade elevation is less than the plant floor elevation.	Subsection 2.4.1 Figure 2.4.2-202	Yes
Precipitation				
Rain	19.4 in/hr (6.3 in/5 min)	17.6 in/hr (3.3 in/5 min)	Table 2.4.2-206	Yes

BLN SUP 2.0-1

TABLE 2.0-201 (Sheet 5 of 5)
COMPARISON OF AP1000 DCD SITE PARAMETERS AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4 SITE CHARACTERISTICS

	AP1000 DCD Site Parameter[(a)]	BLN Site Characteristic	BLN FSAR Reference	BLN Within Site Parameter
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)	10.4 pounds per square foot	Subsection 2.3.1.2.2.3	Yes
Atmospheric Dispersion Values - $\chi/Q^{(f)}$				
Site Boundary (0-2 hr)	$\leq 1.0 \times 10^{-3}$ sec/m^3	0.585×10^{-3} sec/m^3	Table 2.3-319	Yes
Site Boundary (annual average)	$\leq 2.0 \times 10^{-5}$ sec/m^3	0.14×10^{-5} sec/m^3	Table 2.3-325	Yes
Low population zone boundary				
0 – 8 hr	$\leq 5.0 \times 10^{-4}$ sec/m^3	1.23×10^{-4} sec/m^3	Table 2.3-319	Yes
8 – 24 hr	$\leq 3.0 \times 10^{-4}$ sec/m^3	0.826×10^{-4}sec/m^3	Table 2.3-319	Yes
24 – 96 hr	$\leq 1.5 \times 10^{-4}$ sec/m^3	0.349×10^{-4} sec/m^3	Table 2.3-319	Yes
96 – 720 hr	$\leq 8.0 \times 10^{-5}$ sec/m^3	1.01×10^{-5} sec/m^3	Table 2.3-319	Yes
Control Room	See Table 2.0-202	See Table 2.0-202	See Table 2.0-202	Yes
Population Distribution				
Exclusion area (site)	0.5 mi.	The minimum distance from the effluent release boundary to the exclusion area boundary is 2805 feet (0.53 mile).	Figure 2.1-205	Yes

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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) Maximum and minimum normal values are the 1 percent exceedance magnitudes.

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 DCD 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 BLN the “site boundary” and ”exclusion area boundary” are not interchangeable. See Figures 2.1-201 and 2.1-205.

f) With ground response spectra as given in DCD Figures 3.7.1-1 and 3.7.1-2. Seismic input is defined at finished grade except for sites where the nuclear island is founded on hard rock.

BLN SUP 2.0-1

TABLE 2.0-202 (Sheet 1 of 3)
COMPARISON OF CONTROL ROOM ATMOSPHERIC DISPERSION FACTORS FOR ACCIDENT ANALYSIS FOR AP1000 DCD AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4
NOTE: Site χ/Q Values are from Table 2.3-321

	χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(a)]			χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(b)]		
	Plant Vent or PCS Air Diffuser[(c)]	Plant Vent	PCS Air Diffuser	Plant Vent or PCS Air Diffuser[(c)]	Plant Vent	PCS Air Diffuser
	DCD	FSAR	FSAR	DCD	FSAR	FSAR
0 – 2 hours	3.0E-3	2.2E-3	1.6E-3	1.0E-3	7.3E-4	6.8E-4
2 – 8 hours	2.5E-3	1.9E-3	7.8E-4	7.5E-4	6.3E-4	4.4E-4
8 – 24 hours	1.0E-3	8.6E-4	3.6E-4	3.5E-4	2.8E-4	2.0E-4
1 – 4 days	8.0E-4	6.3E-4	2.7E-4	2.8E-4	2.1E-4	1.5E-4
4 – 30 days	6.0E-4	4.8E-4	2.2E-4	2.5E-4	1.6E-4	1.4E-4

	χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(a)]			χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(b)]		
	Steam Line Break Releases	Steam Vent	Condenser Air Removal Stack	Steam Line Break Releases	Steam Vent	Condenser Air Removal Stack
	DCD	FSAR	FSAR	DCD	FSAR	FSAR
0 – 2 hours	2.4E-2	1.1E-2	1.3E-3	4.0E-3	1.7E-3	1.1E-3
2 – 8 hours	2.0E-2	3.4E-3	8.4E-4	3.2E-3	5.6E-4	4.2E-4
8 – 24 hours	7.5E-3	2.2E-3	3.3E-4	1.2E-3	3.1E-4	2.5E-4
1 – 4 days	5.5E-3	1.6E-3	2.5E-4	1.0E-3	2.5E-4	1.7E-4
4 – 30 days	5.0E-3	9.8E-4	1.9E-4	8.0E-4	1.9E-4	1.1E-4

BLN SUP 2.0-1

TABLE 2.0-202 (Sheet 2 of 3)
COMPARISON OF CONTROL ROOM ATMOSPHERIC DISPERSION FACTORS FOR ACCIDENT ANALYSIS FOR AP1000 DCD AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4
NOTE: Site χ/Q Values are from Table 2.3-321

	χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(a)]		χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(b)]	
	Ground Level Containment Release Points[(d)]		Ground Level Containment Release Points[(d)]	
	DCD	FSAR	DCD	FSAR
0 – 2 hours	6.0E-3	2.4E-3	1.0E-3	7.4E-4
2 – 8 hours	4.5E-3	1.8E-3	7.5E-4	5.8E-4
8 – 24 hours	2.0E-3	7.1E-4	3.5E-4	2.5E-4
1 – 4 days	1.8E-3	6.4E-4	2.8E-4	2.0E-4
4 – 30 days	1.5E-3	5.4E-4	2.5E-4	1.6E-4

	χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(a)]		χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(b)]	
	PORV and Safety Valve Releases[(e)]		PORV and Safety Valve Releases[(e)]	
	DCD	FSAR	DCD	FSAR
0 – 2 hours	2.0E-2	1.0E-4	4.0E-3	1.8E-3
2 – 8 hours	1.8E-2	3.8E-3	3.2E-3	6.0E-4
8 – 24 hours	7.0E-3	2.2E-3	1.2E-3	2.9E-4
1 – 4 days	5.0E-3	1.5E-3	1.0E-3	2.7E-4
4 – 30 days	4.5E-3	9.3E-4	8.0E-4	1.9E-4

BLN SUP 2.0-1

TABLE 2.0-202 (Sheet 3 of 3)
COMPARISON OF CONTROL ROOM ATMOSPHERIC DISPERSION FACTORS FOR ACCIDENT ANALYSIS FOR AP1000 DCD AND BELLEFONTE NUCLEAR PLANT UNITS 3 & 4
NOTE: Site χ/Q Values are from Table 2.3-321

	χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(a)]			χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(b)]		
	Fuel Handling Area[(f)]	Fuel Building Blowout Panel	Fuel Building Rail Bay Door	Fuel Handling Area[(f)]	Fuel Building Blowout Panel	Fuel Building Rail Bay Door
	DCD	FSAR	FSAR	DCD	FSAR	FSAR
0 – 2 hours	6.0E-3	2.2E-3	1.7E-3	6.0E-3	6.8E-4	6.4E-4
2 – 8 hours	4.0E-3	1.8E-3	1.4E-3	4.0E-3	5.7E-4	5.2E-4
8 – 24 hours	2.0E-3	8.8E-4	6.8E-4	2.0E-3	2.7E-4	2.5E-4
1 – 4 days	1.5E-3	6.8E-4	5.2E-4	1.5E-3	2.0E-4	1.8E-4
4 – 30 days	1.0E-3	4.8E-4	3.6E-4	1.0E-3	1.6E-4	1.4E-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 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.

c) 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.

d) 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.

e) The listed values bound the dispersion factors for releases from the steam line safety and power-operated relief valves and the condenser air removal stack. 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. Additionally, these dispersion coefficients are conservative for the small line break outside containment.

f) 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 building 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.

2.1 GEOGRAPHY AND DEMOGRAPHY

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Subsection 2.1.1 of the DCD is renumbered as Subsection 2.1.4 and moved to the end of Section 2.1. This is being done to accommodate the incorporation of Regulatory Guide 1.206 numbering conventions for Section 2.1.

STD DEP 1.1-1 2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

BLN COL 2.1-1 The BLN site is approximately 7 mi. northeast of downtown Scottsboro, in Jackson County Alabama. The BLN is located approximately 38 mi. east of downtown Huntsville, Alabama; 44 mi. southwest of downtown Chattanooga, Tennessee; and 48 mi. north of downtown Gadsden, Alabama. The Tennessee River borders the site from approximately Tennessee River mile (TRM) 390 to TRM 393, with the site located on the western bank (Reference 214). Figure 2.1-201 shows the BLN site plot plan and the principal structures on the site. Highway and railroad access to the site is shown, along with principal site structures. The Town Creek embayment and the Tennessee River surround the site to the north, east, and south. Figure 2.1-202 is the BLN vicinity base map, showing population centers within a radius of six miles from the center of the site. Principal highways and rail lines in the vicinity are shown. Figure 2.1-203 is the BLN regional base map, which extends to 50 miles from the site. In addition to principal highways and waterways, boundaries of the counties in the tri-state region are show, as well as the state boundaries among Alabama, Tennessee, and Georgia. Department of Defense (DOD) and Department of Energy (DOE) facilities in the region are show. Figure 2.1-204 is a USGS topographic map covering same area as Figure 2.1-201. There are military facilities located in the region but there are none in the vicinity. Figure 2.1-202 illustrates the features within a 6-mi. radius of the site center point. Detailed information regarding nearby industrial, transportation, and military facilities are presented in Section 2.2.

The BLN site lies completely within the 7.5-minute Hollywood Quadrangle. The quadrangles that bracket the site area are Wannville, Stevenson, Henagar, Sylvania, Dutton, Langston, Scottsboro, and Mud Creek (Reference 223).

The nearest population center to the BLN (as defined by 10 CFR 100.3) is Huntsville, Alabama (References 226 and 227). Huntsville's urban border, as defined by the U.S.Census Bureau, is situated 29 mi. to the west (Reference 231).

The city of Scottsboro, Alabama is the largest city whose border lies within 10 mi. of the BLN (Reference 231).

The closest communities to the BLN are the towns of Hollywood, Alabama, 3 mi. to the west, and Pisgah, Alabama, 5 mi. to the east (Reference 205). The U.S. Census Bureau estimated 2005 populations within a 10-mi. radius are shown in Table 2.1-202.

Interstate 59 connects Birmingham, Alabama, with Chattanooga, Tennessee, and its closest point to the BLN is approximately 18 mi. east to southeast. U.S. Highway 72 is located approximately 1.5 mi. northwest of the site at its closest point. In addition to U.S. 72, segments of Alabama State Highways 40 and 279 are located within an 5-mi. radius of the site center point. Jackson County Road 33 is adjacent to the western border of the site.

The coordinates of the two new reactors are given below:

LONGITUDE AND LATITUDE (degrees/minutes/seconds)

UNIT 3:	34° 42' 48.3" N	85° 55' 32.4" W
UNIT 4:	34° 42' 43.3" N	85° 55' 25.0" W

NORTHING AND EASTING IN ALABAMA MERCATOR EAST STATE PLANE PROJECTION (Feet)

	Easting	Northing
UNIT 3:	628415	1532943
UNIT 4:	629036	1532440

UNIVERSAL TRANSVERSE MERCATOR ZONE 16 (Meters)

	Easting	Northing
UNIT 3:	598376	3841787
UNIT 4:	598568	3841636

2.1.1.2 Site Area Map

The reactor buildings, turbine building, and the cooling towers are labeled in Figure 2.1-201. The auxiliary buildings are shown in the background. There are no industrial and transportation facilities, or commercial, institutional, recreational, or residential structures within the site area. Figure 2.1-202 shows greater detail of the BLN vicinity out to a radius of 6-mi. The BLN property boundary is boldly outlined and highways, railroads, and waterways that traverse or are adjacent to the site are also shown in Figure 2.1-202. The property boundary is the same as the property line, site boundary, site area, and area of control. The total area contained by the site boundary is approximately 1600 ac. of land. Figure 2.1-204 is a U.S.Geological Survey topographic map that shows prominent natural and manmade features. Figure 2.1-205 illustrates the distance from the effluent release boundary, the boundary on which limits for the release of radioactive effluents are based, to the exclusion area boundary (EAB) in each of the 22.5 degree segments centered on the 16 cardinal compass points. The shortest distance listed on this map is 2805 ft. in the northwest direction.

2.1.1.2.1 Boundary for Establishing Effluent Limits

The boundary on which limits for the release of radioactive effluents are based is the site exclusion area boundary is as shown in Figure 2.1-205. The EAB follows the site property boundary on the land-bound side, the Tennessee River side, and the lower portion of Town Creek. The EAB extends across the site property boundary to the opposite shore of Town Creek on the northwest side of the property (See Figure 2.1-205). There are no residents living in this exclusion area. No unrestricted areas within the site boundary area are accessible to members of the public. The Town Creek portion of the EAB is controlled by the TVA. Access within the site property boundary is controlled as described in Subsection 2.1.2. Section 2.3 provides details on gaseous release points and their relation to the EAB. The discussion of normal releases (gaseous and aqueous) is in Sections 11.2 and 11.3. Accidental releases are discussed in Chapter 15. Areas outside the exclusion area are unrestricted areas in the context of 10 CFR Part 20. Additionally, the guidelines provided in 10 CFR Part 50 Appendix I require that radiation exposures meet the criterion “As Low As Is Reasonably Achievable” are applied at the EAB.

Figure 2.1-204 shows the BLN property boundary. Information on how this area is controlled, including how the applicant is apprised of individuals entering the area and controls such access is discussed in Subsection 2.1.2.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

The property is clearly posted and includes actions to be taken in the event of emergency conditions at the plant. The site's physical security plan contains information on actions to be taken by security force personnel in the event of unauthorized persons crossing the EAB. The BLN EAB is greater than 0.5 mi. at its narrowest width and therefore bounds the DCD site parameter exclusion area distance identified in DCD Table 2-1.

2.1.2.1 Authority

The land and water inside the exclusion area is owned or controlled by the TVA and is in the custody of the TVA. Additionally, the TVA controls activities within the EAB including exclusion and removal of personnel and property from the area.

Mineral rights on the BLN site are owned by the TVA. There is a 30 ft. easement on either side of the road centerline along County Road 33 on the southern boundary. There are no other easements affecting the BLN site.

2.1.2.2 Control of Activities Unrelated to Plant Operation

There are no residences, commercial activities not associated with the BLN, or recreational activities within the exclusion area. No public highways or railroads traverse the exclusion area (Reference 209).

2.1.2.3 Arrangements for Traffic Control

Arrangements with Jackson County for traffic control in the event of an emergency are not required in that no publicly used transportation modes cross the EAB, except on Town Creek. Town Creek is owned and controlled by the TVA; therefore, no arrangements with Jackson County have been made.

2.1.2.4 Abandonment or Relocation of Roads

No public roads cross the exclusion area; therefore, no public roads are relocated or abandoned.

2.1.3 POPULATION DISTRIBUTION

The population distribution surrounding the BLN site, up to an 80-km (50-mi.) radius, is estimated based upon the U.S. Census Bureau 2000 census data (Reference 232). The population distribution is estimated in nine concentric bands at 0 – 2 km (0 - 1.24 mi.), 2 - 4 km (1.24 - 2.5 mi.), 4 - 6 km (2.5 - 3.7 mi.), 6 - 8 km (3.7 - 5 mi.), 8 - 10 km (5 - 6.2 mi.), 10 - 16 km (6.2 - 10 mi.), 16 - 40 km (10 - 25 mi.), 40 - 60 km (25 - 37 mi.), and 60 - 80 km (37 - 50 mi.) from the center point between the two reactors. Population sectors out to 16 km (10 mi.) are shown in Figure 2.1-206. The bands are subdivided into 16 directional sectors, each on one of the 16 compass points and consisting of 22.5 degrees.

The population projections are derived from county estimates that are based on the cohort-component method (References 203, 204, and 211). Using linear regression, an equation is derived for each county. The equation is used in conjunction with the 2000 census data to produce a growth ratio. Ratios are calculated for each county and for each year, then weighted by area and summed into sectors. The ratio set is then used to produce a sector-level population projection ratio set for the 80-km (50-mi.) region. The census population numbers are then sorted into the polar grid. In the instance that census blocks are divided by sector boundary lines, the population was weighted by area to produce proportionate data values. These values are summed and multiplied by their projection ratio to produce the final population sector map (Figure 2.1-207).

The BLN region includes all or part of the counties listed in Table 2.1-201.

2.1.3.1 Population Within 10 Miles

Figure 2.1-208 illustrates the portion of the study area within 16 km (10 mi.) of the site center point. Table 2.1-202 shows 2005 estimated populations of the towns within the 16-km (10-mi.) radius; population estimates are based on U.S. Census Bureau data.

Permanent population is projected to 40 years beyond the 2017 construction completion date for the reactors. Table 2.1-203 shows the projected permanent population for each sector, for the years 2007, 2017, 2027, 2037, 2047, and 2057. Population in the 16-km (10-mi.) radius is shown in the 'Cumulative Totals' field of Table 2.1-203 for each projected year.

2.1.3.2 Population Between 10 and 50 Miles

Figure 2.1-205 shows the region within 80 km (50 mi.) of the site center point. The map contains the sector grid, state boundaries, urban areas, and counties. The distances defining the sectors are 16 km (10 mi.), 40 km (25 mi.), 60 km (37 mi.), and 80 km (50 mi.). Huntsville, Alabama, is the largest city within the 80-km (50-mi.) area with a 2005 estimated population of 166,313 (References 205, 226, and 227). Chattanooga, Tennessee, is another large city within the 80-km (50-mi.) area with a 2005 estimated population of 154,762 (References 205, 226, and 227). Smaller cities within the 80-km (50-mi.) area include Gadsden, Alabama; Rome, Georgia; and Madison, Alabama. Based on the 2005 census estimates, their populations are 37,405; 35,816; and 35,893, respectively (References 205, 226, and 227). Several cities have 2005 estimated populations between 10,000 and 20,000 (References 205, 226, and 227). These include East Ridge, Tennessee; Tullahoma, Tennessee; Albertville, Alabama; East Brainerd, Tennessee; Fort Payne, Alabama; and Red Bank, Tennessee. Many other small towns, cities, and urban areas with populations less than 10,000 are distributed within the 80-km (50-mi.) area (References 205, 226, and 227).

Permanent population is projected to 40 years beyond the 2017 construction completion date for the reactors. Table 2.1-204 shows the projected permanent population for each sector, for the years 2007, 2017, 2027, 2037, 2047, and 2057. The number of people in the 16 – 80-km (10 –50-mi.) radius is shown in the 'Cumulative Totals' field of Table 2.1-204 for each projected year.

2.1.3.3 Transient Population

Though relatively rural in nature, the region surrounding the BLN has numerous tourist attractions that contribute moderate levels of transient population. Within a 9.7-km (6-mi.) radius of the site, the largest draw is the Unclaimed Baggage Center in Scottsboro, Alabama. More than one million visitors each year pass through this facility, which is also one of the largest retail stores in the vicinity.

The BLN region encompasses one of the most heavily visited counties in the state, Madison County. Madison County had more than 2.4 million visitors in

2005 and is the third most visited county after Baldwin County in the southwest and Jefferson County in the central portion of Alabama (Reference 202).

Transient data are gathered through personal communications with businesses, companies, and local chambers of commerce within the region. This method for collecting transient data provides a more accurate accounting of people visiting the area and a much more precise location of transient contributors than using county estimates weighted over a sector area. Data out to 24.1 km (15 mi.) are collected for the emergency plan to account for any possible emergency planning zone (EPZ) boundary and presented here in the interest of providing the most complete information possible. Major contributors to transient population in the BLN region are shown in Table 2.1-205.

Transient population is projected to 40 years beyond the 2017 construction completion date for the reactors. Table 2.1-208 illustrates the projected transient population for each sector, and projections for the years 2007, 2017, 2027, 2037, 2047, and 2057 for the non-zero sectors. The sectors that have zero values are not illustrated in this table. Peak visitor numbers are provided when available. If annual numbers are the only available data, then the average number of visitors per day is calculated from the total and taken as the peak. These peak or derived peak numbers are presented in the projected transient population.

2.1.3.3.1 Transient Population Within 10 Miles

There are numerous facilities within the 16-km (10-mi.) radius that host outdoor activities. These include Lake Guntersville Reservoir, Goose Pond Colony, and Buck’s Pocket State Park. These facilities combined have approximately 353,000 visitors each year, concentrated during the summer months of June, July, and August.

There is some overlap of transient population with U.S. census (permanent) population due to student population and a small portion of the workforce.

2.1.3.3.2 Transient Population Between 10 and 50 Miles

Within the range of 16 – 80 km (10 – 50 mi.), the bulk of transient population comes from parks and lodging within the area. The six parks and three associated lodges host more than 1.5 million visitors (including day and overnight-stay visitors) per year. From 2002 to 2005, the total number of visitors to these parks has declined by 2.54 percent.

The city of Huntsville, Alabama, located 46.7 km (29 mi.) to the west of the BLN and with a population of more than 166,000 is home to the region’s largest airport, Huntsville International Airport. In 2005, the airport handled nearly 1.3 million passengers. From 1997 to 2005 the airport experienced an average increase in passenger traffic of 3.1 percent. With passenger traffic forecast to almost double by 2025, airport authorities have embarked on an $81 million capital improvement program to provide new terminal facilities, expanded runway systems, more advanced security, and enhanced flight operations. These improvements are scheduled for completion in 2008 (References 205, 207, and 208). Lovell Field, Chattanooga's Metropolitan Airport, handles almost 480,000 passengers per year and, based on data from 2003 through 2006, the airport is experiencing an annual passenger traffic growth rate of approximately 1.6 percent (Reference 212). Average airport passenger numbers are shown in Table 2.1-206.

No passenger trains operate within a 16-km (10-mi.) radius of the BLN site. No Amtrak passenger rail lines cross the 80-km (50-mi.) radius (References 206 and 209). The nearest Amtrak stations are in Birmingham, Alabama, and Gainesville, Georgia (Reference 206).

The city of Chattanooga, Tennessee, lies on the northeast periphery of the 80-km (50-mi.) radius. There are several large attractions in the metro area, which in combination host 3.4 million visitors per year. One of the largest attractions is the Tennessee Aquarium, which along with its 3D IMAX Theater, handles more than 1.3 million visitors each year. Other attractions include the Creative Discovery Museum, Rock City Gardens, Ruby Falls, and the Tennessee Valley Railroad Museum (Chattanooga Choo Choo).

2.1.3.3.2.1 Recreational Transients

Hunting, fishing, and wildlife watching in the portions of Alabama, Georgia, and Tennessee included in the region are important recreational pastimes, as shown in Table 2.1-207. The combined wildlife-related activities attract approximately 429,728 outdoor enthusiasts per year[a] (References 228, 229, and 230).

The northern extent of Guntersville Reservoir includes an area immediately adjacent to the BLN site. Guntersville Reservoir receives more than 193,000 visitors annually with a peak visitation during each of the summer months of more than 32,500 visitors per day. Professional and amateur sport-fishing events are also held at the reservoir.

Within 24.1 km (15 mi.) of the BLN site, there are eight campgrounds with total daily peak occupancy of approximately 1350 campers. This occupancy tally includes special event counts for two of the facilities: Goose Pond Colony and Camp Jackson (Boy Scouts of America), both near Scottsboro, Alabama.

Golf courses, the closest being Goose Pond Colony, host many golfing events throughout the year. Two major events held at Goose Pond are the Spring Fling Junior College Golf Tournament (typically held the second week of March) and the National Junior College Golf Championship (typically held the third week of May). Goose Pond Colony has more than 100,000 visitors per year and represents the

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a. Visitation numbers are calculated from the 2001 U.S. Fish and Wildlife Service National Survey of Fishing, Hunting, and Wildlife using areal weighting.

second largest tourist draw in the vicinity of the BLN, the largest attraction being the Unclaimed Baggage Center (Reference 213).

There are three parks run by the Georgia State Park Division located within the 80-km (50-mi.) radius: James H. "Sloppy" Floyd State Park, Cloudland Canyon State Park, and New Echota Historic Site. These three parks account for 358,000 visitors annually. Peak seasons are spring, summer, and fall, with June and July accounting for the greatest number of visitors.

The Chattahoochee-Oconee National Forests receive more than 2.8 million visitors per year. However, only a small fraction of the total forest is within the 80-km (50-mi.) radius of the BLN, and any effect on transients is expected to be minimal. The total visitor count for the portion of the national forest within the 80-km (50-mi.) radius is just over 341,000 annually (Reference 215).

2.1.3.3.2.2 Seasonal Population

Many of the attractions within the vicinity of the BLN site are centered around outdoor activities. The peak times for these attractions, and the highest visitor counts, occur from spring through mid-fall. The lowest visitor levels occur during the winter months.

2.1.3.3.2.3 Transient Workforce

Temporary workers for construction of the new BLN facility are expected to be accommodated in Jackson and DeKalb counties, Alabama, where approximately 1197 rental properties were available in 2000 (References 201 and 224). At its peak, the temporary workforce for construction is expected to be no more than 3000 workers. Most of these workers are expected to be in-migrants to the vicinity (Reference 216).

2.1.3.3.2.4 Special Facilities (Schools, Hospitals, Nursing Homes, etc.)

The BLN region is home to 16 two-year and four-year colleges and universities. Total enrollment for these schools is more than 46,000 students[b] (References 217, 218, and 219). The two-year and four-year colleges and universities in the region are typically near peak-daily-capacity for the majority of the year, excluding the summer months (mid-May through mid-August).The majority of transient population within the 80-km (50-mi.) region visit the area for recreational purposes. Therefore, when educational institutions are at their lowest levels during the summer months, the overall transient population within the 80-km (50-mi.) region is still at its highest level.

b. The effect of on-campus faculty/staff housing is minimal: the University of Alabama in Huntsville has less than 20 faculty/staff living on campus.

Fourteen major hospitals and medical centers are situated within 80 km (50 mi.) of the BLN. These medical facilities have a combined capacity of 3200 staffed beds and discharge more than 167,000 patients per year. The two closest major medical facilities to the BLN site are Highlands Medical Center in Scottsboro, Alabama, and DeKalb Regional Medical Center in Fort Payne, Alabama. These two facilities account for 170 beds and 5145 discharges, and 103 beds and 4296 discharges, respectively. The largest medical facility within the region is Huntsville Hospital in Huntsville, Alabama, with 713 beds and more than 41,000 patient discharges annually (References 220, 221, and 222).

Three major nursing home facilities are located within the BLN region: Highlands Health & Rehab, located in Scottsboro, Alabama (50-bed capacity); Cumberland Health & Rehab, located in Bridgeport, Alabama (100-bed capacity); and Cloverdale Manor, located in Scottsboro, Alabama (141-bed capacity) (Reference 225).

2.1.3.3.3 Total Permanent and Transient Populations

The annual total of the special facilities and transient populations within the BLN region is approximately 13.3 million people. The peak transient population for the BLN region in 2007 is projected to be approximately 109,244 people (References 202, 210, 213, 215, 228, 229, and, 230). The estimated permanent population for 2007 for the BLN Region is approximately 1.2 million (Reference 232). The total population within the BLN region is calculated to be approximately 1.3 million.

2.1.3.4 Low Population Zone

At the BLN, the low population zone (LPZ) is defined as a 3.2-km (2-mi.) radius from the site center point. Using this radius, portions of Hollywood, Alabama, Town Creek, and the adjacent Tennessee River bank are incorporated into the LPZ (Figure 2.1-209).

According to the U.S. Census Bureau 2000 data, there are 344 people living within the LPZ, primarily north and west of the site around the town of Hollywood, Alabama (Table 2.1-209). There are no major contributors to the transient population in this area. This area is serviced by U.S. 72 which is routed through the LPZ. The only other transportation feature in the LPZ is the Tennessee River (Figure 2.1-209). There are no schools, hospitals, prisons, beaches, or parks in the LPZ. There are no facilities within 8 km (5 mi.) that require special consideration such as hospitals, prisons, jails, or any other facilities that involve confined populations.

The BLN operational workforce population is estimated at 850 people, causing the total daily population density within the LPZ to increase from 27.4 people per sq. mi. to 95 people per sq. mi.

At the projected end of reactor operation (2057), the permanent population of the LPZ is expected to be 504, a density value of 40.1 people per sq. mi. Combining this number with the estimated number of BLN employees, the total population is 1354, and the LPZ population density increases to 107.7 people per sq. mi.

2.1.3.5 Population Center

Using the definition of a population center found in 10 CFR 100.3, the nearest population center is the city of Huntsville, Alabama, with a 2005 estimated population of 166,313 (Figure 2.1-203) (References 231, 226, and 227). Huntsville's urban border, as defined by the U.S. Census Bureau, is situated 29 mi. to the west of the BLN (References 231 and 227).

Using county projection equations and projecting to the end of licensing (2057), Fort Payne becomes the closest population center. Its urban border, as defined by the U.S. Census Bureau, lies 18 mi. to the south east (References 231 and 227). These distances are greater than one and one-third times the distance from the reactor center point to the boundary of the low population zone as required by 10 CFR 100.21(b).

Transient population is not considered in these calculations because 10 CFR 100.3 defines a population center as "the distance from the reactor to the nearest boundary of a densely populated center containing more than about 25,000 residents." Transient populations by nature are not considered to be a part of the resident population.

2.1.3.6 Population Density

The projected permanent population of the BLN region is added to the projected transient population producing the total population. These values are plotted as a function of distance from the center point on Figures 2.1-210 and 2.1-211 for the first year of operation (2017) and about 5 years after the first year of operation (2022), respectively. Illustrated on Figures 2.1-210 and 2.1-211 is the cumulative population that would result from a uniform population density of 500 people per sq. mi. The graphs show that the total population density for both 2017 and 2022 does not exceed 500 people per sq. mi.

The projected permanent population for 2017 is approximately 1.3 million, and the projected transient population is 120,047. Transient population is projected using a ratio generated from transient sector population divided by the Census 2000 population. The projected permanent population for both 2017 and 2057 are multiplied by this ratio to calculate projected transient population. Thus, the projected total population within an 80-km (50-mi.) radius is approximately 1.4 million. The total population density for the first year of operation is 180.8 people per sq. mi.

The projected total population within an 80-km (50-mi.) radius in 2022, about 5 years after the first year of operation for the plant, is approximately 1.5 million. This includes the projected permanent population (1,345,928 people) and the projected transient population (125,455 people). The total population density is projected to be 189.5 people per sq. mi.

STD DEP 1.1-1

2.1.4 COMBINED LICENSE INFORMATION FOR GEOGRAPHY AND DEMOGRAPHY

BLN COL 2.1-1 This COL item is addressed in Section 2.1.

2.1.5 REFERENCES

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BLN COL 2.1-1

TABLE 2.1-201
COUNTIES ENTIRELY OR PARTIALLY LOCATED WITHIN THE
BLN 80-KM (50-MI.) BUFFER

Alabama Counties		Georgia Counties		Tennessee Counties	
Jackson	DeKalb	Dade	Walker	Marion	Franklin
Marshall	Madison	Chattooga	Catoosa	Lincoln	Moore
Cherokee	Etowah	Floyd	Gordon	Coffee	Grundy
Blount	Cullman	Whitfield		Sequatchie	Hamilton
Morgan	Limestone				

(Reference 214)

BLN COL 2.1-1

TABLE 2.1-202
U.S. CENSUS BUREAU ESTIMATED 2005 POPULATIONS
WITHIN THE 16-KM (10-MI.) RADIUS

Populated Places	2005 Estimated Population
Hollywood	929
Scottsboro	14,840
Pisgah	702
Section	763
Dutton	308
Henagar	2,507
Sylvania	1,238

(References 226 and 231)

BLN COL 2.1-1

TABLE 2.1-203 (Sheet 1 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	Sector 0–2 (km)	2–4 (km)	4–6 (km)	6–8 (km)	8–10 (km)	10–16 (km)	0–16 (km)
North							
2007	6	92	65	20	93	138	414
2017	7	98	70	22	100	148	445
2027	7	105	75	23	107	159	476
2037	8	112	80	25	115	169	509
2047	8	119	85	26	122	179	539
2057	8	126	90	28	129	190	571
NNE							
2007	0	77	179	192	244	457	1,149
2017	0	83	192	206	262	492	1,235
2027	0	89	206	220	280	526	1,321
2037	1	95	219	235	299	561	1,410
2047	1	101	233	249	317	595	1,496
2057	1	107	246	264	336	630	1,584
NE							
2007	0	49	15	30	43	155	292
2017	0	53	16	32	47	167	315
2027	0	56	17	34	50	178	335
2037	0	60	18	37	53	190	358
2047	0	64	19	39	56	202	380
2057	0	68	20	41	60	214	403

NOTE:
Based on 2000 Census data

TABLE 2.1-203 (Sheet 2 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	Sector 0–2 (km)	2–4 (km)	4–6 (km)	6–8 (km)	8–10 (km)	10–16 (km)	0–16 (km)
ENE							
2007	0	5	14	26	69	99	213
2017	0	6	15	28	74	106	229
2027	0	6	16	30	79	114	245
2037	0	6	17	32	84	121	260
2047	0	7	18	34	89	129	277
2057	0	7	19	36	95	136	293
EAST							
2007	0	8	48	184	202	1,058	1,500
2017	0	8	52	197	218	1,138	1,613
2027	0	9	56	211	233	1,218	1,727
2037	0	10	59	225	248	1,298	1,840
2047	0	10	63	239	264	1,378	1,954
2057	0	11	67	253	279	1,458	2,068
ESE							
2007	0	8	36	312	483	870	1,709
2017	0	9	39	336	519	936	1,839
2027	0	9	42	360	556	1,001	1,968
2037	0	10	44	383	592	1,067	2,096
2047	0	11	47	407	629	1,133	2,227
2057	0	11	50	430	665	1,199	2,355
SE							
2007	0	6	17	34	106	982	1,145

NOTE:
Based on 2000 Census data

TABLE 2.1-203 (Sheet 3 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	Sector 0–2 (km)	2–4 (km)	4–6 (km)	6–8 (km)	8–10 (km)	10–16 (km)	0–16 (km)
2017	0	7	18	37	114	1,061	1,237
2027	0	7	20	39	122	1,139	1,327
2037	0	8	21	42	130	1,218	1,419
2047	0	8	22	45	138	1,296	1,509
2057	0	9	24	47	145	1,374	1,599
SSE							
2007	0	7	13	43	162	554	779
2017	0	7	14	47	174	596	838
2027	0	8	15	50	186	639	898
2037	0	8	16	53	198	682	957
2047	0	9	17	57	211	724	1,018
2057	0	9	18	60	223	767	1,077
SOUTH							
2007	0	2	8	106	207	1,603	1,926
2017	0	2	9	114	222	1,724	2,071
2027	0	2	9	122	238	1,845	2,216
2037	0	3	10	130	253	1,966	2,362
2047	0	3	11	138	269	2,088	2,509
2057	0	3	11	146	285	2,209	2,654
SSW							
2007	0	0	25	100	104	635	864
2017	0	0	27	107	112	682	928
2027	0	0	28	115	120	730	993

NOTE:
Based on 2000 Census data

TABLE 2.1-203 (Sheet 4 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	Sector 0–2 (km)	2–4 (km)	4–6 (km)	6–8 (km)	8–10 (km)	10–16 (km)	0–16 (km)
2037	0	0	30	123	128	778	1,059
2047	0	0	32	130	136	826	1,124
2057	0	0	34	138	144	874	1,190
SW							
2007	0	5	116	340	916	3,882	5,259
2017	0	6	125	365	986	4,175	5,657
2027	0	6	133	391	1,055	4,468	6,053
2037	0	7	142	417	1,124	4,762	6,452
2047	0	7	151	442	1,193	5,055	6,848
2057	0	7	160	468	1,263	5,348	7,246
WSW							
2007	0	40	171	753	1,609	4,785	7,358
2017	0	43	184	810	1,730	5,146	7,913
2027	0	46	197	867	1,852	5,508	8,470
2037	0	49	209	924	1,973	5,869	9,024
2047	0	52	222	981	2,095	6,231	9,581
2057	0	55	235	1,038	2,216	6,593	10,137
WEST							
2007	6	79	219	210	133	477	1,124
2017	7	85	235	226	143	513	1,209
2027	7	91	252	242	153	549	1,294
2037	8	96	269	258	163	585	1,379
2047	8	102	285	274	174	621	1,464

NOTE:
Based on 2000 Census data

TABLE 2.1-203 (Sheet 5 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	Sector 0–2 (km)	2–4 (km)	4–6 (km)	6–8 (km)	8–10 (km)	10–16 (km)	0–16 (km)
2057	9	108	302	290	184	657	1,550
WNW							
2007	27	105	238	304	83	303	1,060
2017	29	113	256	327	89	326	1,140
2027	31	121	274	350	95	349	1,220
2037	33	129	292	372	102	372	1,300
2047	35	136	310	395	108	395	1,379
2057	37	144	328	418	114	417	1,458
NW							
2007	17	81	35	29	35	134	331
2017	18	87	37	31	37	145	355
2027	20	93	40	33	40	155	381
2037	21	100	43	35	42	165	406
2047	22	106	45	37	45	175	430
2057	24	112	48	39	48	185	456
NNW							
2007	15	84	26	12	50	173	360
2017	17	90	28	13	54	186	388
2027	18	97	30	14	58	199	416
2037	19	103	32	15	61	213	443
2047	20	110	34	16	65	226	471
2057	21	116	36	17	69	239	498

NOTE:
Based on 2000 Census data

TABLE 2.1-203 (Sheet 6 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
0 – 16 KM (10 MI.)

Direction / Year	**Sector 0–2 (km)**	**2–4 (km)**	**4–6 (km)**	**6–8 (km)**	**8–10 (km)**	**10–16 (km)**	**0–16 (km)**
Totals							
2007	71	648	1,225	2,695	4,539	16,305	25,483
2017	78	697	1,317	2,898	4,881	17,541	27,412
2027	83	745	1,410	3,101	5,224	18,777	29,340
2037	90	796	1,501	3,306	5,565	20,016	31,274
2047	94	845	1,594	3,509	5,911	21,253	33,206
2057	100	893	1,688	3,713	6,255	22,490	35,139

Cumulative Totals	**0–2 (km)**	**0–4 (km)**	**0–6 (km)**	**0–8 (km)**	**0–10 (km)**	**0–16 (km)**
2007	71	719	1,944	4,639	9,178	25,483
2017	78	775	2,092	4,990	9,871	27,412
2027	83	828	2,238	5,339	10,563	29,340
2037	90	886	2,387	5,693	11,258	31,274
2047	94	939	2,533	6,042	11,953	33,206
2057	100	993	2,681	6,394	12,649	35,139

NOTE:
Based on 2000 Census data

BLN COL 2.1-1

TABLE 2.1-204 (Sheet 1 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
North				
2007	867	8,858	7,260	16,985
2017	930	9,485	7,823	18,238
2027	993	10,111	8,386	19,490
2037	1,056	10,737	8,949	20,742
2047	1,118	11,363	9,511	21,992
2057	1,181	11,989	10,074	23,244
NNE				
2007	8,603	7,313	12,183	28,099
2017	9,281	7,928	13,117	30,326
2027	9,959	8,544	14,051	32,554
2037	10,638	9,159	14,985	34,782
2047	11,316	9,774	15,919	37,009
2057	11,994	10,390	16,853	39,237
NE				
2007	8,155	10,421	83,237	101,813
2017	8,800	11,401	88,415	108,616
2027	9,445	12,380	93,594	115,419
2037	10,090	13,359	98,773	122,222
2047	10,735	14,339	103,952	129,026
2057	11,380	15,318	109,131	135,829

Based on 2000 Census data

TABLE 2.1-204 (Sheet 2 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
ENE				
2007	8,308	25,226	210,418	243,952
2017	9,258	28,108	244,013	281,379
2027	10,209	30,990	277,607	318,806
2037	11,159	33,872	311,202	356,233
2047	12,110	36,753	344,797	393,660
2057	13,060	39,635	378,391	431,086
EAST				
2007	5,739	12,722	16,540	35,001
2017	6,557	13,858	18,449	38,864
2027	7,375	14,995	20,358	42,728
2037	8,194	16,131	22,267	46,592
2047	9,012	17,268	24,176	50,456
2057	9,830	18,404	26,085	54,319
ESE				
2007	6,980	19,186	12,007	38,173
2017	8,003	21,454	13,388	42,845
2027	9,026	23,721	14,769	47,516
2037	10,049	25,989	16,150	52,188
2047	11,072	28,256	17,531	56,859
2057	12,095	30,524	18,912	61,531
SE				
2007	13,642	5,479	16,407	35,528

Based on 2000 Census data

TABLE 2.1-204 (Sheet 3 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
2017	15,692	6,283	18,353	40,328
2027	17,742	7,087	20,299	45,128
2037	19,792	7,891	22,245	49,928
2047	21,841	8,695	24,190	54,726
2057	23,891	9,499	26,136	59,526
SSE				
2007	15,294	8,107	14,189	37,590
2017	17,581	9,339	16,344	43,264
2027	19,867	10,571	18,500	48,938
2037	22,154	11,804	20,656	54,614
2047	24,440	13,036	22,811	60,287
2057	26,727	14,269	24,967	65,963
SOUTH				
2007	8,552	10,860	50,008	69,420
2017	9,759	12,311	51,401	73,471
2027	10,966	13,762	52,794	77,522
2037	12,173	15,213	54,187	81,573
2047	13,380	16,664	55,580	85,624
2057	14,588	18,115	56,974	89,677
SSW				
2007	4,861	37,212	26,800	68,873
2017	5,429	42,241	28,885	76,555
2027	5,997	47,270	30,970	84,237

Based on 2000 Census data

TABLE 2.1-204 (Sheet 4 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
2037	6,566	52,299	33,056	91,921
2047	7,134	57,327	35,141	99,602
2057	7,703	62,356	37,227	107,286
SW				
2007	7,951	17,152	27,900	53,003
2017	8,835	19,381	31,893	60,109
2027	9,719	21,609	35,887	67,215
2037	10,603	23,838	39,880	74,321
2047	11,487	26,067	43,874	81,428
2057	12,370	28,296	47,867	88,533
WSW				
2007	3,698	16,148	17,391	37,237
2017	4,045	17,862	18,839	40,746
2027	4,393	19,576	20,287	44,256
2037	4,740	21,290	21,734	47,764
2047	5,087	23,005	23,182	51,274
2057	5,434	24,719	24,630	54,783
WEST				
2007	3,133	79,963	156,786	239,882
2017	3,373	87,855	172,494	263,722
2027	3,612	95,747	188,201	287,560
2037	3,851	103,639	203,908	311,398
2047	4,090	111,532	219,616	335,238

Based on 2000 Census data

TABLE 2.1-204 (Sheet 5 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
2057	4,330	119,424	235,323	359,077
WNW				
2007	2,098	16,127	35,121	53,346
2017	2,257	17,682	37,990	57,929
2027	2,416	19,236	40,858	62,510
2037	2,575	20,791	43,726	67,092
2047	2,734	22,345	46,595	71,674
2057	2,893	23,900	49,463	76,256
NW				
2007	1,587	6,063	16,328	23,978
2017	1,707	6,460	17,282	25,449
2027	1,827	6,857	18,235	26,919
2037	1,947	7,254	19,188	28,389
2047	2,066	7,650	20,142	29,858
2057	2,186	8,047	21,095	31,328
NNW				
2007	556	16,037	33,913	50,506
2017	596	17,107	36,618	54,321
2027	636	18,176	39,323	58,135
2037	676	19,245	42,028	61,949
2047	716	20,315	44,733	65,764
2057	757	21,384	47,438	69,579

Based on 2000 Census data

TABLE 2.1-204 (Sheet 6 of 6)
PROJECTED PERMANENT POPULATION FOR EACH SECTOR
16 – 80 KM (10 - 50 MI.)

Direction / Year	Sector 16–40 (km)	40–60 (km)	60–80 (km)	16–80 (km)
Totals				
2007	100,024	296,874	736,488	1,133,386
2017	112,103	328,755	815,304	1,256,162
2027	124,182	360,632	894,119	1,378,933
2037	136,263	392,511	972,934	1,501,708
2047	148,338	424,389	1,051,750	1,624,477
2057	160,419	456,269	1,130,566	1,747,254
Cummulative Totals	**16–40 (km)**	**16–60 (km)**	**16–80 (km)**	
2007	100,024	396,898	1,133,386	
2017	112,103	440,858	1,256,162	
2027	124,182	484,814	1,378,933	
2037	136,263	528,774	1,501,708	
2047	148,338	572,727	1,624,477	
2057	160,419	616,688	1,747,254	

Based on 2000 Census data

BLN COL 2.1-1

TABLE 2.1-205
MAJOR CONTRIBUTORS TO TRANSIENT POPULATION
WITHIN 80 KM (50 MI.)

Facility Name	Average Daily Transients	Peak Daily Transients
Twickenham Historic District	6,301	--
Boaz Outlet Shopping	6,250	--
Noccalula Falls Park	2,740	--
Unclaimed Baggage Center	2,740	--
Tennessee Aquarium	2,345	--
Annual Bridgeport Jubilee	--	2,000
First Monday	2,000	--
Chattanooga Choo Choo	1,623	--
IMAX 3D Theater	1,386	--
Chattahoochee-Oconee National Forests	1,266	--
Lookout Mountain Inclined Railway	1,189	--
Rock City Gardens	1,110	--
Ruby Falls	1,071	--
US Space and Rocket Center	877	--
Native American Festival	--	850
Lake Winnepesaukah Amusement	822	--
Little River Canyon Nat'l Preserve	822	--
Huntsville Botanical Garden	685	--
James H Floyd State Park	646	--
Monte Sano State Park	645	--
Lake Guntersville State Park	612	--
Creative Discovery Museum	600	--
DeSoto State Park	548	--
Jack Daniels Distillery	548	--
Madison County Nature Trail	--	500
Goose Pond Golf Colony and Plantation	450	--
Alabama Constitution Village	411	--
Town of Woodville Festival	--	400
Southern Belle Riverboat	356	--
Cloudland Canyon State Park	298	--
Huntsville Museum of Art	211	--
Gadsden Museum of Art	192	--
Tennessee Valley Railroad Museum	192	--
Burritt Museum and Park	137	--
Cathedral Caverns	102	--

(Selected References from References 202, 210, 213, 215, 228, 229, 230)

BLN COL 2.1-1

TABLE 2.1-206
DAILY AND ANNUAL PASSENGER COUNTS FOR
COMMERCIAL AIRPORTS IN THE BLN REGION

Airport Name	Avg. Daily Passenger Count	Annual Passenger Count
Huntsville International Airport	3,466	1,265,153
Chattanooga – Lovell Field	1,315	480,000

(Reference 207 and 212)

BLN COL 2.1-1

TABLE 2.1-207
HUNTING, FISHING, AND WILDLIFE WATCHING WITHIN THE BLN REGION

Alabama	Number of Visitors	Number of Visitors
Activity	State	BLN Region
Fishing	851,000	73,440
Hunting	423,000	36,504
Wildlife Watching	1,016,000	87,679
Total	2,290,000	197,623
Georgia		
Activity	State	BLN Region
Fishing	1,086,000	24,451
Hunting	417,000	9,389
Wildlife Watching	1,494,000	33,637
Total	2,997,000	67,476
Tennessee		
Activity	State	BLN Region
Fishing	903,000	44,429
Hunting	359,000	17,663
Wildlife Watching	2,084,000	102,536
Total	3,346,000	164,629
BLN Region Total		**429,728**

(References 228, 229, and 230)

BLN COL 2.1-1

TABLE 2.1-208
PROJECTED TRANSIENT POPULATION FOR EACH SECTOR
0 – 80 KM (50 MI.)

Distance(km)	**Direction**	**2007**	**2017**	**2027**	**2037**	**2047**	**2057**
8	SSW	678	725	779	834	881	935
16	WSW	5,582	6,003	6,426	6,847	7,269	7,691
40	NE	2,130	2,298	2,467	2,635	2,804	2,972
40	NNE	990	1,067	1,145	1,224	1,302	1,380
40	SE	919	1,057	1,195	1,333	1,471	1,609
40	SSE	115	132	150	167	184	201
40	SSW	87	98	108	118	128	139
40	SW	490	544	598	653	707	762
40	WSW	554	606	658	710	762	814
60	ENE	1,529	1,703	1,878	2,053	2,227	2,402
60	N	1,564	1,675	1,785	1,896	2,006	2,117
60	SSE	615	708	801	895	988	1,082
60	SSW	119	136	152	168	184	200
60	SW	773	874	974	1,075	1,175	1,276
60	W	8,575	9,422	10,268	11,115	11,961	12,807
60	WSW	542	600	658	715	773	830
80	E	440	491	542	592	643	694
80	ENE	17,563	20,368	23,172	25,976	28,780	31,584
80	ESE	1,219	1,359	1,499	1,639	1,780	1,920
80	NE	15,019	15,953	16,888	17,822	18,757	19,691
80	NNW	4,424	4,777	5,129	5,482	5,835	6,188
80	S	3,532	3,630	3,728	3,827	3,925	4,024
80	SE	2,215	2,478	2,741	3,004	3,266	3,529
80	SSE	55	63	72	80	88	97
80	SSW	8,673	9,348	10,023	10,698	11,373	12,048
80	W	30,842	33,932	37,022	40,112	43,201	46,291

BLN COL 2.1-1

TABLE 2.1-209
POPULATION DISTRIBUTION IN THE LOW POPULATION ZONE

	0–1 (mi)	1–2 (mi)	0–2 (mi)
N	2	56	58
NNE	0	22	22
NE	0	14	14
ENE	0	3	3
E	0	3	3
ESE	0	4	4
SE	0	3	3
SSE	0	3	3
S	0	0	0
SSW	0	0	0
SW	0	0	0
WSW	0	6	6
W	6	30	36
WNW	20	45	65
NW	8	48	56
NNW	9	62	71
Total	45	299	344

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

This section of the referenced DCD is incorporated by reference with the following departure(s) and/or supplement(s).

BLN COL 2.2-1 The Bellefonte Nuclear Plant, Units 3 and 4 (BLN) is located in Jackson County, Alabama. Jackson County is bordered on the west by Madison County, Alabama; on the north by Franklin and Marion counties, Tennessee; on the east by Dade County, Georgia, and DeKalb County, Alabama; and on the south by Marshall County, Alabama, as seen in Figure 2.1-203.

The BLN is accessible by road, river, and rail. Interstate 59 connects Birmingham, Alabama, with Chattanooga, Tennessee, and its closest point to the BLN is approximately 18 mi. east-southeast (Reference 223). U.S. Highway 72 runs parallel to the Tennessee River through the city of Scottsboro, Alabama, (11.3 km [7 mi.]) southwest) and the town of Hollywood, Alabama (3 mi. west) (References 214 and 223). The Tennessee River borders the site boundary from approximately Tennessee River mile (TRM) 390 to TRM 393. Norfolk Southern Railway Company (NSRC) owns and operates a railroad line that runs through the city of Scottsboro, Alabama, and the town of Hollywood, Alabama (Reference 204). The NSRC railroad is 2.7 mi. northwest of the site center point. A spur line owned and controlled by the Tennessee Valley Authority (TVA) connects the plant to the mainline (Reference 223).

This section of the safety analysis report provides information regarding the potential effects on the safe operation of the nuclear facility from industrial, transportation, mining, and military installations in the BLN area.

STD DEP 1.1-1 Subsection 2.2.1 of the DCD is renumbered as Subsection 2.2.4 and moved to the end of Section 2.2. This is being done to accomodate the incorporation of Regulatory Guide 1.206 numbering conventions for Section 2.2.

2.2.1 LOCATIONS AND ROUTES

BLN COL 2.2-1 Within a 5-mi. radius of the BLN, there are two state highways, one federal highway, one railroad, and one navigable river, all with commercial traffic (Reference 223). There are four industrial facilities, including manufacturing sites and a city landfill within 5 mi. of the center point (Reference 202). One airport is also located within 5 mi. of the site center point (References 228 and 229). Specifically, the following transportation routes and facilities are shown in Figure 2.2-201:

- City of Scottsboro Landfill.
- Great Western Products.
- Maples Industries.
- Scottsboro Coca-Cola Enterprises, Inc.
- U.S. 72.
- Alabama 40.
- Alabama 279.
- NSRC Mainline.
- Tennessee River.
- Scottsboro Municipal Airport – Word Field.

Environmental Data Resources Inc. (EDR) provided the results from a database search of petroleum storage tanks registered by the state of Alabama. State regulations for tank registrations were reported to be compliant and consistent with federal regulations. Alabama requires that above ground and underground petroleum storage tanks with a capacity greater than 110 gal. be registered. The registered tank database includes petroleum storage tanks used for bulk, retail, industrial, private, airport, and governmental purposes. Agricultural tanks with a storage capacity greater than 1100 gal. must be registered. Fuel tanks for backup generators must be registered if their storage capacity is greater than 110 gal. Alabama does not require registration of residential fuel oil storage tanks (Reference 202).

A fuel distribution center, The Fuel Center (Discus Oil Company), is located in Hollywood, Alabama, 3 mi. west of the BLN and has 14 aboveground storage tanks (ASTs) located on-site. These 14 tanks have a storage capacity of 184,500 gal. and currently contain unleaded gasoline, supreme gasoline, high-sulfur diesel, low-sulfur diesel, motor oil, and hydraulic oil. Contents of the 14 storage tanks are available in Table 2.2-201.

In addition to The Fuel Center, there are 11 other locations within a 5-mi. radius of the BLN that have registered underground storage tanks (USTs) and/or ASTs (Reference 202). These additional storage tanks are located at local convenience stores, businesses, or municipal facilities. Table 2.2-201 lists the contents and capacity of the registered storage tanks within a 5-mi. radius of the BLN (Reference 202). Figure 2.2-201 illustrates the location of the registered storage tanks within a 5-mi. radius of the BLN.

Mining and quarrying operations are discussed in Subsections 2.2.2.1.6 and 2.2.2.2.5. Oil and gas pipelines are discussed in Subsection 2.2.2.3. Military bases and missile sites are discussed in Subsections 2.2.2.1.7 and 2.2.2.2.6. None of these facilities are located within a 5-mi. radius of the BLN. Evaluations of explosions postulated to occur on transportation routes near nuclear power plants are addressed in Section 2.2.3.

AP1000 Standard Plants contain liquid hydrogen and compressed hydrogen in the amounts of 1500 gallons at 150 pounds per square inch gas (psig) and 500 scft at 6000 psig respectively. The plants do not contain liquid oxygen or propane. Both hydrogen storage tanks are located in the hydrogen storage area adjacent to the cooling towers, in the northeast corner of the site. The BLN also has quantities of liquid nitrogen and liquid carbon dioxide located in the turbine building.

2.2.2 DESCRIPTIONS

The industries within the immediate BLN area are mostly located in Scottsboro, Hollywood, and Stevenson, Alabama. Figure 2.2-201 shows the location of the industries within 5 mi. of the BLN site center point. Table 2.2-202 lists the industrial facilities near the BLN, their primary function/major products, and the number of persons employed (References 205, 206, 207 and 217).

2.2.2.1 Description of Facilities

Four major industrial facilities are located within 5 mi. of the BLN. Descriptions of these facilities are detailed in Subsections 2.2.2.1.1 to 2.2.2.1.4. Subsection 2.2.2.1.5 provides detailed information on the Widows Creek Fossil Plant, the electrical generation station closest to the BLN site. Subsection 2.2.2.1.6 details mining and quarrying activities in the area and Subsection 2.2.2.1.7 details military facilities near the site.

2.2.2.1.1 Scottsboro Landfill

The City of Scottsboro, Alabama, operates a 120- ac. landfill located 3 mi. north of the BLN.

2.2.2.1.2 Maples Industries

Maples Industries is a large manufacturing plant producing carpet and rug products. This facility is located 4.9 mi. southwest of the site center point in Scottsboro, Alabama.

2.2.2.1.3 Scottsboro Coca-Cola Enterprises, Inc

The Scottsboro Coca-Cola Enterprises, Inc., facility is a major distribution center for Coca-Cola products, located 3.8 mi. west-southwest of the site center point.

2.2.2.1.4 Great Western Products

Great Western Products manufactures snack food processing equipment, supplies, and accessories. This facility is located 2 mi. west of the site center point in Hollywood, Alabama.

2.2.2.1.5 Electrical Generation Plants

Widows Creek Fossil Plant is a coal-fired electrical generation plant operated by the TVA. The plant is located 15 river miles upriver of the site center point on the Tennessee River, between the towns of Stevenson and Bridgeport, Alabama, at TRM 408. The facility consists of eight units with a winter net dependable generating capacity of 1629 MWe. The plant consumes approximately 10,000 T. of coal per day and produces about 10 billion kWh of electricity per year (Reference 207). Table 2.2-204 lists hazardous materials reported to the EPA and their quantities. Occupational Safety and Health Administration (OSHA) permissible exposure limits for the reported materials are provided in Tables 2.2-205, 2.2-206, and 2.2-207 (References 215, 219, and 221). No nuclear electrical generation plants are located within 50 mi. of the BLN.

2.2.2.1.6 Mining and Quarrying Activities

No mining and quarrying activities are located within 5 mi. of the BLN site center point. Six permitted mines and one permitted non-fuel mine are located within Jackson County, but there are no drilling operations in the county (Reference 216).

2.2.2.1.7 Military Facilities

No military facilities lie within 5 mi. of the BLN site center point. However, two military facilities are situated within 50 mi. of the site center point: Arnold AFB, located 47 mi. north of the site, and Redstone Arsenal, located approximately 48 mi. west of the site (Reference 239).

Other than the Redstone Arsenal, there is no evidence of missile sites in the region. Redstone Arsenal includes the U.S. Army Aviation and Missile Command (AMCOM), the Space and Missile Defense Command, and major components of the Defense Intelligence Agency and the Missile Defense Agency (Reference 204).

Arnold Air Force Base operates aerodynamic and propulsion wind tunnels, rocket and turbine engine test cells, space environmental chambers, arc heaters, ballistic ranges, and other specialized units. The Arnold Engineering Development Center is an U.S. Air Force material command facility (Reference 203).

2.2.2.2 Description of Products and Materials

2.2.2.2.1 Scottsboro Landfill

This landfill is divided into two sections: a 25-ac. unlined Class C and D section, and a 55-ac. lined sanitation section. This facility is permitted to accept 190 T. per day of household and/or industrial waste from Jackson, Madison, and DeKalb counties, Alabama. This facility is not permitted to accept hazardous waste, and there are currently no plans to expand this facility.

2.2.2.2.2 Maples Industries

Products made by Maples Industries are sold in various outlets, including large retail stores such as Kohl's and Linens 'n Things. In 2004, Maples Industries completed a $6 million expansion of the Scottsboro facility, adding 48,000 sq. ft. (Reference 217). Table 2.2-203 lists hazardous materials reported to the Jackson County Emergency Management Agency stored on site.

2.2.2.2.3 Scottsboro Coca-Cola Enterprises, Inc

This major distribution center has no plans to expand this facility in the immediate future (Reference 205). According to the Jackson County Emergency Management Agency, no hazardous materials are listed as being stored at this location.

2.2.2.2.4 Great Western Products

Great Western Products has no plans to expand this manufacturing facility (Reference 206). According to the Jackson County Emergency Management Agency, no hazardous materials are listed as being stored at this location.

2.2.2.2.5 Mining and Quarrying Activities

There are no mining and quarrying activities located within 5 mi. of the site. Since there are no mines or quarrying activities located near the site, there are no explosives used in the area that would be associated with mining or quarrying activities.

2.2.2.2.6 Military Facilities

There are no military facilities, including bombing ranges and jet fuel storage facilities, located within 5 mi. of the site. There are no known transportation routes for military grade munitions or jet fuel located near the site.

2.2.2.2.7 Waterways

The nearest navigable waterway to the BLN is the Guntersville Reservoir/ Tennessee River, adjacent to the project boundary. The BLN project boundary is situated between TRM 390 and TRM 393. Table 2.2-209 lists the types and amounts of cargo shipped by barge on the Tennessee River for the year 2004 (Reference 226). Table 2.2-210 lists the type and amounts of commodities shipped past TRM 392 in 2004

The nearest major port to the BLN site is located 35 mi. south in the town of Guntersville, Alabama, in Guntersville Harbor, between TRM 358 and TRM 359. Major commodities processed at this port are grain, petroleum, and wood products (References 224 and 232). Table 2.2-211 shows the type and amount of commodities shipped between TRM 358 and TRM 363 on the Tennessee River in 2004 (Reference 225). Six items listed in Table 2.2-211 are considered hazardous cargo.

2.2.2.2.8 Highways

The nearest highway with heavy commercial traffic is U.S. 72, passing approximately 1.5 mi. to the northwest at its closest point. In addition to U.S. 72, segments of Alabama highways 40 and 279 are located within a 5-mi. radius of the site center point. Any material registered with the federal government as a hazardous material is allowed to travel along any public road in the State of Alabama, provided it is properly packaged and transported. The amount of explosives shipped along the public roads within 5 mi. of the facility is unknown. No federal, state, or local agencies are required by law to keep records of transportation of hazardous materials; therefore, no data is available

2.2.2.2.9 Railroads

Norfolk Southern Railroad Company (NSRC) owns and operates a railroad line that runs through the city of Scottsboro, Alabama, and the town of Hollywood, Alabama, approximately 3 mi. northwest of the site. Any material registered with the federal government as a hazardous material that is legally allowed to be transported via American railroads could potentially be transported at some point along the rails that are situated near the BLN site. Items that may be legally transported on the rails near the site include many types of hazardous materials and other industrial chemicals. Table 2.2-208 lists the top 25 commodities shipped through Hollywood, Alabama, between September 2005 and September 2006. OSHA permissible exposure limits for the reported materials are provided in Tables 2.2-205, 2.2-206, and 2.2-207 (Reference 219).

2.2.2.3 Description of Pipelines

No cross-county pipelines are located in the vicinity of the BLN. However, there are local residential, commercial, and industrial distribution pipelines near the site.

2.2.2.4 Description of Waterways

The BLN Units 3 and 4 are located about 3500 ft. north-northwest of the Guntersville Reservoir/Tennessee River, the closest navigable waterway.

Fifty-five river ports are located within an 50-mi. radius of the BLN. The nearest ports are Scottsboro and Fort Payne Forest Products, Mannington Bellefonte, and Tennessee Valley Port; however, these ports are currently not in operation. The nearest operating port is the Mead Corporation's Stevenson Mill Dock, located near TRM 405, and used to receive fuel oil for mill consumption (Reference 223).

The closest lock and dam is located south of the site in Guntersville, at the beginning of the Guntersville Reservoir. The Guntersville dam was completed in 1939 and is 94 ft. high and 3979 ft. long. This dam has two locks, with the larger lock completed in 1965 (Reference 224).

Different types of barges navigate the Tennessee River. Among these are dry covered barges, single-hull tank barges, double-hull tank barges, dry open barges, and deck barges. Tugboats and push boats operate on the Tennessee River, as well as personal watercraft (Reference 230).

The mean depth of the Guntersville Reservoir is 15 ft., and the average depth of the Tennessee River is a minimum of 11 ft. (References 226 and 227). The Guntersville Reservoir averages 25.7 ft. deep along the BLN site boundary in the shipping channel.

Figure 2.1-201 shows the location of the intake structure in the Guntersville Reservoir/Tennessee River for Units 3 and 4. This intake structure is located near TRM 392 at the southern end of Bellefonte Island on the western shore of the Tennessee River. The shipping channel generally follows the center of the river and the eastern fork around Bellefonte Island. Water from the Guntersville Reservoir/Tennessee River is withdrawn at this location for use as cooling tower makeup, service water cooling system makeup, and other miscellaneous water uses.

2.2.2.5 Description of Highways

As stated in Subsection 2.2.2.2.8, the nearest highway with heavy commercial traffic is U.S. 72, passing approximately 1.5 mi. to the northwest at its closest point. In addition to U.S. 72, segments of Alabama highways 40 and 279 are located within a 5-mi. radius of the site center point. Any material registered with the federal government as a hazardous material is allowed to travel along any public road in the State of Alabama, provided it is properly packaged and transported. The amount of explosives shipped along the public roads within 5 mi. of the facility is unknown since no agencies are required by law to keep records of this information.

Estimated annual average daily traffic (AADT) counts in 2005 indicate the following:

- 16,720 vehicles travel on U.S. 72 at mile 145.4 (west of the site).

- 5050 vehicles travel on Alabama 279 at mile 9 (west of the site), located before Alabama 279 merges with U.S. 72.
- 6120 vehicles travel on Alabama 40 at mile 1.7 (south of the site).
- 13,760 vehicles travel past mile 148.2 (north of the site) on U.S. 72 (Reference 220).

2.2.2.6 Description of Railroads

NSRC owns and operates a railroad line that runs through the city of Scottsboro, Alabama, and the town of Hollywood, Alabama. This railroad line is the main line in northern Alabama running from Memphis, Tennessee, through Huntsville, Alabama, to Chattanooga, Tennessee (Reference 201). At its closest point, the line runs about 3 mi. northwest of the BLN site center point.

On average, 40 trains per day pulling an average of 75 cars use this rail line and travel at speeds up to 50 mph. This line is used for freight service only; no passenger trains use this line (Reference 222).

As stated in Subsection 2.2.2.2.9, any material registered with the federal government as a hazardous material that is legally allowed to be transported via American railroads could potentially be transported at some point along the rails that are situated near the BLN site. Items that may be legally transported on the rails near the site include many types of hazardous materials and other industrial chemicals. Table 2.2-208 lists the top 25 commodities shipped through Hollywood, Alabama, between September 2005 and September 2006. OSHA permissible exposure limits for the reported materials are provided in Tables 2.2-205, 2.2-206, and 2.2-207 (Reference 219).

2.2.2.7 Description of Airports

2.2.2.7.1 Airports

One airport, Scottsboro Municipal Airport - Word Field, is situated within 5 mi. of the BLN site center point. The airport, located 4.9 mi. west to southwest, has a 5250-ft. asphalt runway oriented in a southwest to northeast direction, and is used primarily by single-engine private aircraft. There are 22 single-engine, one multi-engine, and six ultra-light aircraft based at the field. The average number of operations (landings and takeoffs are counted separately) is 21 per day. Transient general aviation accounts for 81 percent of operations and about 19 percent of operations are local general aviation (References 228 and 229). There are no designated pilot training areas near the site.

Approach and departure paths at Scottsboro Municipal Airport are not directly aligned with the BLN. On a long approach, a plane is expected to get no closer to the plant site than 2 mi., and there are no holding patterns associated with the Scottsboro Municipal Airport.

One fatal aviation accident occurred in the last 20 years within 5 mi. of the BLN. This fatal accident occurred near Scottsboro, Alabama, on June 19, 2003. Two deaths were associated with this accident. In the past 40 years, no other fatal aviation accidents occurred within 5 mi. of the BLN. During the same 40-year period, seven aviation accidents were reported in Scottsboro, Alabama; one aviation accident occurred in Section, Alabama; one in Dutton, Alabama; one in Pisgah, Alabama; and none in Hollywood, Alabama (Reference 218).

The closest commercial airport is Chattanooga Metropolitan Airport - Lovell Field, located 47 mi. northeast in Chattanooga, Tennessee. Lovell Field has two asphalt runways: one that is 5000 ft. long, and one 7400 ft. long. Federal Aviation Administration (FAA) information, effective October 25, 2006, indicates that 89 planes and one helicopter are based at this airport: 31 of these are single-engine aircraft, 32 are multi-engine aircraft, and 26 are jets. Lovell Field averages 252 aircraft operations a day. Transient general aviation accounts for 41 percent of operations, 20 percent are air taxi, 19 percent are military, 14 percent are local general aviation, and 5 percent are commercial (References 212 and 213).

Fifty-three aviation accidents or incidents have occurred since 1962 in Chattanooga, Tennessee. Of the 53 accidents, five have been fatal (Reference 211).

The next closest commercial airport is Huntsville International Airport, which is located 49 mi. west-southwest of the BLN site center point. The airport has two asphalt runways: one that is 10,006 ft. long, and one 12,600 ft. long. FAA information, effective June 7, 2006, indicates that 100 aircraft are based on the field: 69 of these are single-engine aircraft, 20 are multi-engine aircraft, nine are jet aircraft, and two are helicopters. The average number of operations is about 283 per day. Air taxis account for 32 percent of operations, 24 percent are military, 21 percent are transient general aviation, 17 percent are local general aviation, and 6 percent are commercial (References 230 and 231).

Ninety-two aviation accidents have occurred since 1962 in Huntsville, Alabama. Of the 92 accidents, nine have been fatal (Reference 210).

Historical flight data recorded prior to 2006 shows an average annual increase of 4.1 percent in the number of airline passengers at Huntsville International Airport (Table 2.2-212). Based on the data in Table 2.2-212, Table 2.2-213 provides projections for air traffic at Huntsville International Airport to fiscal year 2025. Historical passenger traffic and projected passenger traffic for Chattanooga Metropolitan Airport is not available to the public. Scottsboro Municipal Airport is currently adding additional T-Hangars, a specific type of aircraft hangar usually used for smaller aircraft. The airport has filed an application with the FAA to publish a global positioning system approach path for the airport. Scottsboro Municipal Airport is also planning to increase the maximum gross weight limit of aircraft landing at the airport by upgrading runways in the near future. The current maximum gross weight limit is 15,000 lb. per single axle. Huntsville International Airport is undergoing an $81 million expansion and renovation that is projected to be completed in 2008. This expansion and renovation is expected to meet the projected demands of the future by enlarging many existing airport facilities, including parking facilities, public screening and waiting areas, concession areas, and baggage claim carousels (Reference 233).

Approach and departure paths at Huntsville International Airport are not aligned with the BLN site. All runways, existing and proposed, are aligned north-south (Reference 233). No holding patterns are associated with Huntsville International Airport near the site. Approach and departure paths at Chattanooga Metropolitan Airport are not aligned with the BLN site. Both existing runways are aligned in a general north-south direction (Reference 213). No holding patterns are associated with Chattanooga Metropolitan Airport near the site.

2.2.2.7.2 Airways

Three low-altitude (below 18,000 ft.) federal air routes are located within 35 mi. of the BLN site as shown in Figure 2.2-202 (Reference 209). Also known as Victor air routes, these low-altitude routes are flown primarily by general aviation aircraft. They are typically 8 nautical miles wide, and they occupy the airspace between 18,000 ft. msl and the floor of controlled airspace, 700 ft. - 1200 ft. There are no military training routes within 10 mi. of the site center point. Due to the distance between these airways and the location of the BLN site, no further analysis of hazards from air traffic along the closest low-altitude airways is necessary.

Five high-altitude (18,000 ft. - 45,000 ft. msl pressure altitude) federal air routes are located within 35 mi. of the site as shown in Figure 2.2-202 (Reference 208). These high-altitude airways are used primarily by commercial air carriers, the military, and high-performance general aviation aircraft. These routes are also 8 nautical miles wide and are extended from 18,000 ft. to 45,000 ft., the top of controlled airspace. Flights above 18,000 ft. are required to be instrument flight rules flights; therefore, altitudes and routes are assigned by air traffic controllers. Because the centerline of Airway J73 is in close proximity (approximately 3 mi. west) of the BLN site, an evaluation of hazards from air traffic along high-altitude airways is presented in Section 3.5.1.6.

2.2.2.8 Projections of Industrial Growth

Four industrial parks are located in Jackson County, Alabama. As of October 2006, no additional industrial parks were proposed for the county. As of August 2006, 2533.9 ac. in Jackson County, Alabama, are available for industrial and agricultural uses. The table below indicates in which Jackson County cities the available acreage is located.

Location	Land Available for Industrial and Agricultural Uses
Bridgeport, AL	1213.9 ac.

Location	Land Available for Industrial and Agricultural Uses
Stevenson, AL	794 ac.
Scottsboro, AL	526 ac.

In addition, 44.4 ac. of existing structures are available for industrial uses in Jackson County, Alabama.

The Jackson County, Alabama, area is experiencing growth and more economic activity now than in recent history. For example, a 240-ac. industrial park that opened in 1989 recently completed its initial build-out, and the adjacent 60 ac. was purchased in 2000 for continued expansion. Companies wishing to locate to the area can be accommodated by the existing infrastructure and zoned industrial land.

No mining and quarrying activities are located within 5 mi. of the BLN site center point. Six permitted mines and one permitted non-fuel mine are located within Jackson County, but there are no drilling operations in the county (Reference 216).

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

The consideration of a variety of potential accidents, and their effects on the plant or plant operation, is included in this section. General Design Criterion 4, “Environmental and Missile Design Basis,“ of Appendix A, “General Design Criteria for Nuclear Power Plants,” to 10 CFR Part 50, “Licensing of Production and Utilization Facilities,” requires that nuclear power plant structures, systems, and components important to safety be appropriately protected against dynamic effects resulting from equipment failures that may occur within the nuclear power plant as well as events and conditions that may occur outside the nuclear power plant.

2.2.3.1 Determination of Design Basis Events

Design basis events internal and external to the nuclear power plant are defined as those accidents that have a probability of occurrence on the order of about 10^{-7} per year or greater and potential consequences serious enough to affect the safety of the plant to the extent that the guidelines in 10 CFR Part 100 could be exceeded. The following categories are considered for the determination of design basis events: explosions, flammable vapor clouds with a delayed ignition, toxic chemicals, fires, collisions with the intake structure, and liquid spills.

2.2.3.1.1 Explosions

2.2.3.1.1.1 Transportation Routes

Accidents were postulated for the nearby highways, railroads and waterways. The nearest highway with heavy commercial traffic is U.S. 72, which passes approximately 1.13 miles northwest of the BLN at its closest point to the site boundary. This distance to the site boundary is used for the explosion evaluations, instead of the distance to the nearest safety related structure, to provide some additional conservatism to the evaluation. The accident of concern along U.S. 72 is one that results in the detonation of a highly explosive cargo carried by a truck. It is necessary to demonstrate that such an explosion on the highway does not result in a peak positive incident overpressure that exceeds 1 lb/in^2 at the critical structures on the BLN site. The maximum probable hazardous cargo for a single highway truck is presented in terms of equivalent trinitrotoluene (TNT). The TNT equivalency is based on Reference 235: $W_E = \frac{H^d_{EXP}}{H^d_{TNT}} W_{EXP}$, where W_E is the effective charge weight, H^d_{EXP} is the heat of detonation of the explosive in question, H^d_{TNT} is the heat of detonation of TNT, and W_{EXP} is the weight of the explosive in question.

The methodology presented in Regulatory Guide 1.91 establishes the safe distance beyond which no damage would be expected (i.e. a peak positive incident overpressure of less than 1 lb/in^2 at the critical structures on the BLN site.) As noted in Section 2.2, any material registered with the federal government may be transported along the transportation routes within the vicinity. Therefore, a material with a TNT equivalency of 2.24 is chosen to bound the explosion hazards along transportation routes. This value is based on the military grade explosive HBX-3, which is used in missile warheads and underwater ordnance (Reference 236). This conservative approach bounds the explosive energy of commonly transported materials such as gasoline and propane.

The maximum probable hazardous solid cargo for a single highway truck, in pounds, is based on Regulatory Guide 1.91. To be conservative, a head on collision between two highway trucks carrying intended explosives is considered. An evaluation performed for materials with a TNT equivalency of 2.24 and using the maximum cargo for two trucks determined the safe distance to be 0.52 miles, hence, there is considerable margin between the required safe distance and the actual distance. Therefore, the proximity to U.S. 72 does not present an explosion hazard. The effects of blast-generated missiles are less than those associated with the blast overpressure levels considered in Regulatory Guide 1.91. Because the overpressure criteria of the guide are not exceeded, the effects of blast-generated missiles are not considered.

The Norfolk Southern Railroad passes approximately 2.13 miles northwest of the site at its closest point. The maximum probable quantity of explosive material shipped by a single railroad boxcar in terms of equivalent pounds of TNT is based on Regulatory Guide 1.91. It is recognized that cargo shipments by railroad typically constitute the usage of more than one boxcar. For the purpose of qualifying the explosion hazard involved in this railroad analysis, thirty combined boxcar values for intended explosives are incorporated into the calculation. These values may be considered conservatively bounding because it is reasonable to assume the initial explosion would involve only one boxcar associated with initiating the explosion. Should additional boxcars become involved, related explosions would be subsequent in time and neither coincident with, nor additive to, the effects associated with those from the first boxcar explosion. Second, the aggregated length of thirty boxcars extends several hundred feet. Therefore, the boxcar explosions at the far ends are at a greater distance than the referenced explosion point. It is clearly conservative to aggregate thirty boxcars at the explosion reference point. The evaluation determined the required safe distance to be 1.76 miles, which is less than the distance from the railroad to the site at its closest point. Note that this bounds the explosive energy of commonly transported materials. This conservative approach was taken because there are no restrictions on the type or quantity of materials that can be transported on the railroad. Therefore the proximity to the railroad does not present an explosion hazard.

The nearest transportation route to the BLN is the Guntersville Reservoir. Its nearest bank is located 0.65 miles from the site. An assessment was performed to evaluate potential hazards represented by flammable and explosive cargo transported via barge past the BLN on the Guntersville Reservoir. An initial screening of commodities included in cargo shipped via the Guntersville Reservoir past the BLN site was conducted to identify those materials that warranted more detailed evaluation, that is, “commodities of interest.” This initial screening of the hazardous commodities eliminated all but two requiring further analysis for potential adverse impact to the BLN site from waterway transportation (barge) accidents. These two commodities are styrene and ethanol. Commodities are screened out based on their physical properties. The primary physical parameter is the commodities' flash point. The National Fire Protection Association Hazard Identification System (NFPA 704) (Reference 237) is used. Only commodities with flammability hazards classified as three or four (serious hazard and severe hazard, respectively) are considered.

For these two commodities of interest, additional detailed shipment information was obtained from the U.S. Army Corps of Engineers Waterborne Commerce Statistics Center (WCSC) and used to develop reasonably bounding assumptions regarding the amount of each commodity included in a single barge shipment past the BLN site. This WCSC data also provided shipping frequency (pass-the-point data) for each commodity.

Analyses were then performed for each commodity, taking into account chemical and physical properties, state of the material when shipped, assumed progression of events following the incident that releases the material, reaction kinetics, and release rates. These analyses included the following:

a. Analysis of a confined space detonation,

b. Local free vapor cloud explosion, and

c. Evaluation of a vapor cloud formation and dispersion downwind toward the BLN site with a delayed ignition. (The vapor cloud with delayed ignition is discussed in Subsection 2.2.3.1.2.)

The two commodities were further investigated for the extent of overpressure based on a confined space vapor explosion. The confined vapor cloud explosion scenario assumed that the transport vessel had been breached and sufficient material lost to leave a vapor space filled with an explosive gas mixture. The mass of explosive gas mixture that can be confined in the hold of the barge is limited by the vapor space volume available. The analysis assumes the entire hold was void of any liquid thus maximizing the mass of the explosive gas mixture. An ignition source is introduced and combustion occurs. Due to the confined space, the internal pressure rises rapidly and eventually ruptures the vessel. The safe standoff distances for confined vapor explosions for styrene and ethanol were determined to be 0.85 miles and 0.53 miles, respectively. For the confined vapor explosion analysis, only styrene was shown to pose a hazard of an overpressure greater than 1 lb/in^2 at the BLN site.

Based on an evaluation for free vapor cloud explosion, styrene was determined to pose some level of risk that would have to be further evaluated. Due to its solubility in water, ethanol was determined to be unable to present a legitimate opportunity for a free vapor cloud explosion. The standoff distance for styrene was determined to be 1.6 miles. Based on this standoff distance, an “at-risk” length along the Guntersville Reservoir on which the accident could occur, and in which an overpressure of one lb/in^2 or greater at the site could potentially be created from the explosion, was determined. The “at-risk” length along the Guntersville Reservoir was determined to be less than three miles. The values for safe standoff distances and "at-risk" length conservatively take no credit for shielding provided by intervening terrain.

For those commodities from the above analyses that produced an overpressure value in excess of 1 lb/in^2 at the BLN site, a risk assessment was performed to determine the associated probability of occurrence of the event consistent with Regulatory Guide 1.91. The evaluation was performed by:

1. Reviewing the applicable historic data on spills from the United States Army Corps of Engineers and the United States Coast Guard.

2. Determining the spill frequency on the Tennessee and its feeder rivers from this data.

3. Determining an explosion frequency of similar events from the hazardous cargo traffic data obtained.

Historic data provides an acceptable predictor of future event frequencies. This is reasonable because of continuing improvements in marine transport safety and spill prevention design measures. The final risk is calculated by multiplying the spill frequency, explosion frequency, and the "at-risk" length in a manner consistent with Regulatory Guide 1.91.

From this relationship and data on commodity shipments past the BLN site and “at-risk” river lengths, the accidental detonation risk (to the site) was estimated to be less than $1.9x10^{-8}$ per year.

2.2.3.1.1.2 Pipelines

Per Subsection 2.2.2.3, there are no major pipelines in the vicinity of the BLN.

2.2.3.1.1.3 Nearby Industrial Facilities

The Fuel Center is located 2.49 miles west of the BLN site. boundary. The Fuel Center has a combined registered storage tank capacity of 184,500 gallons. For evaluation purposes, it is assumed that these tanks are filled with gasoline and they rupture simultaneously. The Fuel Center represents the largest quantity of registered storage tank capacity of the facilities near the BLN site and is the closest above-ground storage facility. The safe standoff distance for the confined vapor explosion was determined to be 0.51 miles and the safe standoff distance for the unconfined vapor explosion was determined to be 0.91 miles. Therefore, the distance from the Fuel Center to the BLN site meets the safe distance requirements as defined in Equation 1 of Regulatory Guide 1.91.

Maples Industries is located 3.79 miles from the site southwestern boundary. An assessment was performed to evaluate potential hazards represented by flammable and explosive chemicals stored at the Maples Industries facility. An initial screening of these chemicals was performed to identify those that warranted more detailed evaluation. This screening of the hazardous chemicals eliminated all but three requiring further analysis for potential adverse impact to the BLN site from an accident at this facility. These three chemicals are isopropyl alcohol, gasoline, and cyclohexylamine. For each of these three chemicals, the safe standoff distance for a confined vapor explosion was determined to be 0.13 miles or less and the safe standoff distance for an unconfined vapor explosion was determined to be 0.23 miles or less. Therefore, the distance from Maples Industries to the BLN site meets the safe distance requirements as defined in Equation 1 of Regulatory Guide 1.91.

Great Western Products is located 1.49 miles from the site western boundary. An assessment was performed to evaluate potential hazards represented by flammable and explosive chemicals stored at the Great Western Products facility. An initial screening of these chemicals was performed to identify those that warranted more detailed evaluation. This screening of the hazardous chemicals eliminated all but three requiring further analysis for potential adverse impact to the BLN site from an accident at this facility. These three chemicals are isopropyl alcohol, Calfoam®, and Glycol Ether PM. For each of these three chemicals, the safe standoff distance for a confined vapor explosion was determined to be 0.09 miles or less and the safe standoff distance for an unconfined vapor explosion was determined to be 0.12 miles or less. Therefore, the distance from Great Western Products to the BLN site meets the safe distance requirements as defined in Equation 1 of Regulatory Guide 1.91.

2.2.3.1.1.4 Onsite Chemicals

As discussed in DCD Section 1.9, the AP1000 uses small amounts of combustible gases for normal plant operation. Most of these gases are used in limited quantities and are associated with plant functions or activities that do not jeopardize any safety-related equipment. These gases are found in areas of the plant that are removed from the nuclear island. The exception to this is the hydrogen supply line to the chemical and volume control system (CVS).

The CVS is the only system on the nuclear island that uses hydrogen gas. Hydrogen is supplied to the AP1000 CVS inside containment from a single hydrogen bottle. The release of the contents of an entire bottle of hydrogen in the most limiting building volumes, both inside containment and in the auxiliary building would not result a volume percent of hydrogen large enough to reach a detonable level.

DCD Subsection 3.5.1.1.2.2 states that the battery compartments are ventilated by a system that is designed to preclude the possibility of hydrogen accumulation. The DCD states further that the storage tank area for plant gases is located sufficiently far from the nuclear island that an explosion would not result in missiles more energetic than the tornado missiles for which the nuclear island is designed.

The plant gas system provides hydrogen, carbon dioxide, and nitrogen gases to the plant systems as required. The effects of the plant gas system on main control room habitability are addressed in DCD Section 6.4 including explosive gases and burn conditions for those gases. For explosions, the plant gas system is designed for conformance with Regulatory Guide 1.91 (DCD Subsection 9.3.2.3).

2.2.3.1.2 Flammable Vapor Clouds (Delayed Ignition)

The potential for detonation and deflagrations in a plume resulting from release of the commodities from a transportation accident was evaluated, as well as a potential release from nearby facilities. This evaluation assumed dispersion downwind toward the BLN, with a delayed ignition. For each commodity of interest, the vapor dispersion was determined based on a wind speed of 1.8 miles per hour, a Stability Class of D, and a 90°F ambient air temperature. These meteorological conditions were chosen to maximize the vaporization rate of the commodity of interest while limiting the downwind dispersion. The calculation performed a sensitivity of meteorological conditions to demonstrate that this combination is bounding. The ALOHA code (Reference 234) was used to evaluate the dispersion and detonation of the vapor clouds.

The basic input into the analysis is:

1. Release location
2. Chemical of interest
3. Weather conditions
4. Chemical release information

ALOHA models the release of the hazardous chemical in two ways:

1. The chemical is a liquid and pours out of the rupture, where it forms a puddle. The exposed chemical then evaporates and forms a vapor cloud.
2. The chemical in the tank exits as a two phase mixture (gas and liquid), where it will immediately begin to move towards the plant. ALOHA determines which of these scenarios is applied based on the physical properties of the chemicals.

After release the vapor cloud travels towards the location, and the concentration of the chemical compared to the air (in parts per million [ppm]) is calculated. The vapor cloud explodes at the closest point to the location where the explosive limits of the chemical of interest permits and an overpressure value is determine at that point. ALOHA refers to a negligible overpressure as zero lb/in^2.

For the evaluation of the potential effects of accidents on U.S. 72, conservatively large tanker truck volumes, based on Alabama Department of Transportation values, were assumed along with assumed rupture sizes of 48.4 sq.ft. and 10.7 sq. ft. Because almost any commodity can be transported along the highways, various commodities were assumed. Gasoline and propane were analyzed due to the fact that these are commonly transported commodities. Other less popular commodities were analyzed that have a relatively high enough reactivity to result in a vapor cloud explosion when the cloud is ignited by a spark or a flame. The evaluation determined that there is a negligible overpressure at the site resulting from a delayed ignition of a vapor cloud and the concentrations remain below the lower explosive limit at the BLN site.

Similarly, for the Norfolk Southern Railroad, various commodities were analyzed. with the ALOHA code, assuming conservatively large tanker sizes, based on Alabama Department of Transportation values, and rupture sizes of 48.4 sq. ft. and 10.7 sq. ft. The evaluation determined that there is a negligible overpressure at the site resulting from a delayed ignition of a vapor cloud and the concentrations remain below the lower explosive limit at the BLN site.

The gasoline stored at the Fuel Center was analyzed assuming tank rupture sizes of 53.8 sq.ft. and 10.7 sq.ft. The evaluation determined that there is a negligible overpressure at the BLN site resulting from a delayed ignition of a vapor cloud and the concentrations at the BLN site are negligible.

The release rate from a postulated barge accident is based on two assumed. rupture sizes of 53.8 sq. ft. and 10.7 sq. ft. Based on the screening of the commodities transported via barge past the BLN site, only styrene was identified as having the potential to form an unconfined vapor cloud. The analysis determined that the peak overpressure resulting from a delayed ignition of styrene is 0.309 lb/in^2. The maximum concentration of styrene at the BLN site is 5670 ppm, which is less than 52 percent of the lower explosive limit concentration of 11,000 ppm, hence no deflagrations would be expected at the BLN site.

For the postulated accidents on U.S. 72, the Norfolk Southern Railroad, and the Fuel Center, the overpressure at the BLN resulting from the delayed ignition of a vapor cloud was negligible. The concentrations of the flammable and explosive vapors remain below the lower explosive limits at the BLN site. The only postulated accident with a delayed ignition of a vapor cloud that resulted in a slight overpressure at the BLN Site was the postulated rupture of a barge containing styrene. Even for this case, the overpressure was less than one lb/in^2 at the BLN site, and the concentrations of styrene vapor remain below the lower explosive limit on the site.

Therefore, it is concluded that the delayed ignition of vapor clouds from nearby transportation routes and pipelines does not pose a hazard to the BLN.

2.2.3.1.3 Toxic Chemicals

Events involving the release of toxic chemicals from onsite storage facilities and nearby mobile and stationary sources are considered for this section. For each identified source and postulated event, the Regulatory Guide 1.78 screening criteria of distance, quantity, and frequency are applied. For releases of hazardous chemicals from stationary sources or from frequently shipped mobile sources in quantities that do not meet the screening criteria, detailed analysis are performed for control room habitability. These detailed analysis are presented in Section 6.4.

2.2.3.1.3.1 Background

Figure 2.2-201 shows the potential stationary industrial sources and mobile sources (barge and river traffic, local highways, and local rail lines) within the proximity of the BLN site. Each of these is discussed and compared to the screening criteria of Regulatory Guide 1.78 in the following sections.

Regulatory Guide 1.78 establishes the Occupational Safety and Health Association (OSHA) National Institute for Safety and Health (NIOSH) Immediately Dangerous to Life and Health (IDLH) guidelines for 30 minute exposure as the required screening criteria for airborne hazardous chemicals. Per Regulatory Guide 1.78, the NIOSH IDLH values were utilized to screen chemicals and to evaluate concentrations of hazardous chemicals to determine their effect on control room habitability.

2.2.3.1.3.2 Source Evaluation

2.2.3.1.3.2.1 Stationary Sources

There are no site-specific sources of airborne hazardous materials stored on the BLN site in sufficient quantity to affect control room habitability.

Section 2.2.2.1 lists four major industrial facilities within a five mile radius of the site. One industry, Maples Industries, could be expected to have toxic materials on site. The quantities of toxic materials used at Maples Industries is well below Regulatory Guide 1.78 Appendix A thresholds given the distance to the BLN site (4.9 miles). Due to the nature of their respective industries, the other facilities would not contain inventories of toxic chemicals in large enough quantities to affect control room habitability.

2.2.3.1.3.2.2 Mobile Sources

Preliminary statistical analysis evaluated the general risk from mobile sources of hazardous materials. This preliminary risk analysis indicates that although the accident risk is quite low, it is not less than the evaluation limit of $1x10^{-6}$ per year for mobile sources set in Regulatory Guide 1.78. Therefore, a wholly risk-based approach was not considered.

2.2.3.1.3.2.2.1 Barges and River Traffic

Barge shipment frequency statistics on barge traffic for 2003 and 2004 were provided by the Waterborne Commerce Statistics Center. These statistics show a frequency of less than 50 hazardous shipments per year on barge traffic passing the site. In accordance with Regulatory Guide 1.78, further analysis on barge traffic is not required due to this low frequency.

2.2.3.1.3.2.2.2 Local Highways

State Highway 72 is located approximately 1.5 miles west of the BLN site. Highway 72 commodity flow information and rural highway risk analysis information was used to perform a bounding analysis of traffic on Highway 72. The traffic was analyzed in accordance with the methodology in Regulatory Guide 1.78.

The available commodity flow information did not identify chlorine or other hazardous chemicals traveling down Route 72 that could pose a danger to control room habitability from a distance of 1.5 miles. A risk analysis conducted with publicly available transportation statistics indicates a very low probability of a toxic chemical release along Route 72 in general. Accordingly, Highway 72 and other local roads do not pose a danger to control room habitability at the present time.

2.2.3.1.3.2.2.3 Local Rail Lines

A Norfolk Southern rail line is located approximately 2.5 miles west of the BLN site, running northeast to southwest in a line parallel to the Tennessee River. The screening method described in Regulatory Guide 1.78 is applied to the rail traffic listed in Section 2.2. The release mass of a toxic chemical was calculated based on the size of a commercially available chlorine rail tanker. The mass utilized for the hazardous release calculation was 180,000 lbs. This screening factors of distance from source to control room, release rate mass, and toxicity values of the hazardous material could not eliminate this toxic chemical source event from the potential to exceed the NIOSH IDLH threshold value.

2.2.3.1.3.3 Analysis of Hazardous Materials

As indicated above, the identified stationary industrial sources and mobile sources within the proximity of the BLN site were evaluated and eliminated as potential hazards to the control room personnel with the exception of a rail tanker of chlorine. Thus the chlorine tanker release event is further evaluated in Section 6.4 to determine control room habitability.

2.2.3.1.4 Fires

Fires originating from accidents at any of the facilities or transportation routes discussed previously will not endanger the safe operation of the station because of the distances between potential accident locations and the location of the BLN are at least 0.65 miles away.

The nuclear island is situated sufficiently clear of trees and brush. The distance exceeds the minimum fuel modification area requirements of thirty feet per NFPA-1144 (Reference 238). Therefore, there is no threat from brush or forest fires.

Fire and smoke from accidents at nearby homes, industrial facilities, transportation routes, or from area forest or brush fires, does not jeopardize the safe operation of the plant due to the separation distance of potential fires from the plant. The main control room HVAC system continuously monitors the outside air using smoke monitors located at the outside air intake plenum and monitors the return air for smoke upstream of the supply air handling units (DCD Subsection 9.4.1.2.3.1). If a high concentration of smoke is detected in the outside air intake, an alarm is initiated in the main control room and the main control room/ technical support center HVAC subsystem is manually realigned to the recirculation mode by closing the outside air and toilet exhaust duct isolation valves. Therefore, any potential heavy smoke problems at the main control room air intakes would not affect the plant operators.

Onsite fuel storage facilities are designed in accordance with applicable fire codes, and plant safety is not jeopardized by fires or smoke in these areas. A detailed description of the plant fire protection system is presented in DCD Subsection 9.5.1.

2.2.3.1.5 Collision with Intake Structure

There is no safety related equipment located at the intake structure. Therefore, collisions with the intake structure do not pose a nuclear safety hazard.

2.2.3.1.6 Liquid Spills

There is no safety related equipment located at the intake structure. Therefore, spills drawn into the intake structure do not pose a nuclear safety hazard.

2.2.3.2 Effects of Design Basis Events

Potential design basis events associated with accidents at nearby facilities and transportation routes have been analyzed in Subsection 2.2.3.1. With the exception of potential barge accidents, the effects of these events on the safety-related components of the plant are insignificant as discussed in Subsection 2.2.3.1. Postulated accidents of barges containing styrene can potentially result in overpressures at the BLN that exceed one lb/in^2, however, the probability of such a postulated accident was determined to be less than 10^{-7} events per year. Based on Regulatory Guide 1.91, this does not represent a design basis event. This also meets the criteria of 10^{-6} occurrences per year in the DCD Section 2.2 for not requiring changes to the AP1000 design for an external accident leading to severe consequences. It is also concluded that external fires do not represent a hazard to the BLN.

STD DEP 1.1-1 2.2.4 COMBINED LICENSE INFORMATION

BLN COL 2.2-1 This COL item is addressed in Subsection 2.2.3.

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BLN COL 2.2-1

TABLE 2.2-201 (Sheet 1 of 2)
REGISTERED STORAGE TANKS WITHIN AN 8-KM (5-MI.) RADIUS OF BLN

Site	Address	UST / AST	Distance from BLN	Number of Tanks	Combined Capacity(a)	Tank 1 Capacity(a)	Tank 1 Contents	Tank 2 Capacity(a)	Tank 2 Contents	Tank 3 Capacity(a)	Tank 3 Contents	Tank 4 Capacity(a)	Tank 4 Contents
Kirks Pro-AM Inc	32800 Hwy. 72	UST	2.7 km (1.7 mi.)	2	16,000	8,000	Unleaded Gasoline	8,000	Mid-grade / Premium Gasoline				
Machens Grocery	169 Railroad St.	UST	4.8 km (3.0 mi.)	3	6,000	2,000	Unleaded Gasoline	2,000	Mid-grade Gasoline	2,000	Premium Gasoline		
Hollywood Truck Stop	28646 Hwy. 72	UST	4.3 km (2.7 mi.)	1	12,000	12,000	Diesel						
Stephens Fuel Stop	28741 Hwy. 72	UST	5.3 km (3.3 mi.)	4	13,000	3,000	Diesel	3,000	Diesel	3,000	Unleaded Gasoline	4,000	Premium Gasoline
Trotter Place Chevron Food Mart	24860 John T. Reed Pkwy.	UST	5.3 km (3.3 mi.)	3	28,000	12,000	Unleaded Gasoline	8,000	Premium Gasoline	8,000	Diesel / Kerosine		
Rush Stop	23574 John T. Reed Pkwy.	UST	5.6 km (3.5 mi.)	2	16,000	10,000	Unleaded Gasoline	6,000	Premium Gasoline				
County Park Chevron Food Mart	22885 John T. Reed Pkwy.	UST	6.1 km (3.8 mi.)	3	30,000	10,000	Unleaded Gasoline	10,000	Mid-grade Gasoline	10,000	Premium Gasoline		
Pantry 3686 DBA Cowboys	21700 John T. Reed Pkwy.	UST	6.6 km (4.1 mi.)	3	42,000	10,000	Unleaded Gasoline	12,000	Premium Gasoline / Diesel	20,000	Diesel		
Southern Belle Quick Stop	3335 Hwy. 40	UST	7.9 km (4.9 mi.)	1	12,000	12,000	Unleaded / Premium Gasoline						

BLN COL 2.2-1

TABLE 2.2-201 (Sheet 2 of 2)
REGISTERED STORAGE TANKS WITHIN AN 8-KM (5-MI.) RADIUS OF BLN

Site	Address	UST / AST	Distance from BLN	Number of Tanks	Combined Capacity[a]	Tank 1 Capacity[a]	Tank 1 Contents	Tank 2 Capacity[a]	Tank 2 Contents	Tank 3 Capacity[a]	Tank 3 Contents	Tank 4 Capacity[a]	Tank 4 Contents
The Fuel Center, Inc	42 Ridge Rd.	AST	4.8 km (3.0 mi.)	14[b]	183,000	20,000	Unleaded Gasoline	20,000	Supreme Gasoline	30,000	High-sulfur Diesel	30,000	Low-sulfur Diesel
Southern Belle Quick Stop	3335 Hwy. 40	AST	7.9 km (4.9 mi.)	2	2,000	1,000	Kerosine	1,000	Diesel				
Sequatchie Concrete Service	21266 John T. Reed Pkwy.	AST	5.8 km (3.6 mi.)	1	500	500	Diesel						
Pisgah Central Office	133 Church St.	AST	7.9 km (4.9 mi.)	1	300	300	Diesel						

a) Gallons

b) Three 20,000 gallon above-ground gasoline tanks are currently empty. Also located on-site are four, 5,000- gallon and three, 1,500-gallon above-ground oil (motor and hydraulic) storage tanks.

(Reference 201)

BLN COL 2.2-1

TABLE 2.2-202
INDUSTRIAL FACILITIES NEAR BLN

Name of Facility	Primary Function / Major Products	Number Employed
Scottsboro Landfill	Solid-waste landfill (120 ac.)	25
Maples Industries	Manufacturer of carpet and rug products	2,000
Scottsboro Coca-Cola Enterprises Inc.	Distribution center for Coca-Cola products	91
Great Western Products	Manufacturer of snack food processing equipment, supplies, and accessories	40
Widows Creek Fossil Plant	Coal-fired electrical generation plant	397

(References 203, 205, 206, 207, 208, and 217)

BLN COL 2.2-1

TABLE 2.2-203
HAZARDOUS MATERIALS AT MAPLES INDUSTRIES

Chemical Inventory
#2 Diesel Fuel
Ammonia
Caustic Soda (Solution)
Fatty Amine Ethoxylate Mixture
Glycol Component Mixture
Hydrogen Peroxide (Aqueous Solution)
Isopropyl Mixture
Phosphoric Acid
Sodium Hydroxide
Sodium Hydroxide (Bleach Mixture)
Sodium Hypochlorite Solution
Sodium Hypoclorate
Sulfuric Acid
Tanaprint (Mixture)

TABLE 2.2-204

BLN COL 2.2-1

HAZARDOUS MATERIALS AT WIDOWS CREEK FOSSIL PLANT

Chemical	Amount On-Site (lbs.)
Arsenic Compounds	10,000 – 99,999
Barium Compounds	1,000,000 – 9,999,999
Beryllium Compounds	10,000 – 99,999
Chromium Compounds	100,000 – 999,999
Cobalt Compounds	10,000 – 99,999
Copper Compounds	10,000 – 99,999
Lead Compounds	10,000 – 99,999
Manganese Compounds	100,000 – 999,999
Mercury Compounds	100 – 999
Nickel Compounds	100,000 – 999,999
Thallium Compounds	10,000 – 99,999
Vanadium Compounds	100,000 – 999,999
Zinc Compounds	100,000 – 999,999
Hydrochloric Acid (Aerosol)	0 – 99
Hydrogen Fluoride	0 – 99
Benzo(g,h,i)perylene	100 – 999
Dioxin	0 – 99
Napthalene	10,000 – 99,999
Ammonia	1,000 – 9,999
Nitrate Compounds	100 – 999

(References 215, 219, and 221)

BLN COL 2.2-1

TABLE 2.2-205 (Sheet 1 of 3)
SITE SPECIFIC OSHA PERMISSIBLE EXPOSURE LIMITS
Z-1 TABLE

	Limit	
Substance	(ppm)	(mg/m^3)
Acetic acid	200	360
Acetic anhydride	10	25
Arsenic, inorganic compounds		0.3
Arsenic, organic compounds		0.5
Barium, soluble compounds		0.5
Barium sulfate		
Total Dust		15
Respirable Fraction		5
Butanols		
n-Butyl alcohol	100	300
sec-Butyl alcohol	150	450
tert-Butyl alcohol	100	300
Carbon dioxide	5000	9000
Chlorine	1	3
Chromium		
Chomium II compounds		0.5
Chromium III compounds		0.5
Chromium metals		1
Cobalt metal		0.2

BLN COL 2.2-1

TABLE 2.2-205 (Sheet 2 of 3)
SITE SPECIFIC OSHA PERMISSIBLE EXPOSURE LIMITS
Z-1 TABLE

	Limit	
Substance	(ppm)	(mg/m^3)
Copper		
Fume		0.1
Dusts and mists		1
Glycol monoethyl ether		
2-Ethoxyethyl	200	740
Hydrogen peroxide	1	1.4
Isopropyl Mixture		
Isopropyl acetate	250	950
Isopropyl alcohol	400	980
Isopropylamine	5	12
Isopropyl ether	500	2100
Isopropyl glycidyl ether	50	240
Lead inorganic		400
Manganese compounds		5
Methyl Methacrylate	100	410
Napthalene	10	50
Nickel Compounds		1
Oats (grain dust)		10
Phosphoric Acid		1
Sodium Hydroxide		2
Sulfuric Acid		1
Thallium Compounds		0.1

BLN COL 2.2-1

TABLE 2.2-205 (Sheet 3 of 3)
SITE SPECIFIC OSHA PERMISSIBLE EXPOSURE LIMITS
Z-1 TABLE

	Limit	
Substance	(ppm)	(mg/m^3)
Vanadium Compounds		0.5
Wheat (grain dust)		10
Xylenes	100	435
Zinc Compounds		
Zinc chloride fume		1
Zinc oxide fume		5
Zinc oxide		
Total dust		15
Respirable fraction		5
Zinc stearate		
Total dust		15
Respirable fraction		5

(Reference 219)

BLN COL 2.2-1

TABLE 2.2-206
SITE SPECIFIC OSHA PERMISSIBLE EXPOSURE LIMITS
Z-2 TABLE

Substance	Time Weighted Average (8-hr. shift)	Acceptable Ceiling Concentration	Acceptable Maximum Peak above Acceptable Ceiling Concentration (8-hr. shift)	
			Concentration	Maximum Duration
Beryllium	2 ug/m³	5 ug/m³	25 ug/m³	30 min.
Hydrogen Fluoride	3 ppm			
Mercury (Mercury Compounds)		1 mg/10m³		

(Reference 219)

BLN COL 2.2-1

TABLE 2.2-207
SITE SPECIFIC OSHA PERMISSIBLE EXPOSURE LIMITS
Z-3 TABLE

Substance	Limit mg/m^3
Coal dust	2.4

(Reference 219)

BLN COL 2.2-1

TABLE 2.2-208 (Sheet 1 of 2)
TOP 25 COMMODITIES SHIPPED VIA NSRC RAILROAD PAST HOLLYWOOD, AL, SEPTEMBER 2005 – SEPTEMBER 2006

HMRC	Proper Shipping Name	Class	PKG. Group	HAZ. Zone	UN / NA
4950150	FAK-Hazardous Materials	FAK	N/A	N/A	N/A
4909351	Xylenes	3	II, III	N/A	UN1307
4935230	Potassium Hydroxide	8	II, III	N/A	UN1814
4931405	Acrylic Acid, Stabilized	8	II	N/A	UN2218
4931304	Acetic Anyhdride	8	II	N/A	UN1715
4913250	Combustible Liquid, N.O.S.	CL	III	N/A	NA1993
4962137	Other Regulated	9	III	N/A	NA3082
4904509	Carbon Dioxide	2.2	N/A	N/A	UN2187
4909130	Butanols	3	II, III	N/A	UN1120
4909198	Xylenes	3	II, III	N/A	UN1307
4908119	Butyraldehyde	3	II	N/A	UN1129
4907250	Methyl Methacrylate	3	II	N/A	UN1247
4921401	Acetone Cyanohydrin	6.1	I	B	UN1541
4914223	Combustible Liquid, N.O.S.	CL	III	N/A	NA1993
4920523	Chlorine	2.3	N/A	B	UN1017
4908270	Propionaldehyde	3	II	N/A	UN1275
4966109	Other Regulated	9	III	N/A	NA3082
4950130	FAK-Hazardous Materials	FAK	N/A	N/A	N/A
4904210	Ammonia, Anhydrous	2.2	N/A	N/A	UN1005
4909267	N-Propanol	3	II, III	N/A	UN1274
4931303	Acetic Acid, Glacial	8	II	N/A	UN2789
4930040	Sulfuric Acid	8	II	N/A	UN1830

TABLE 2.2-208 (Sheet 2 of 2)
BLN COL 2.2-1 TOP 25 COMMODITIES SHIPPED VIA NSRC RAILROAD PAST HOLLYWOOD, AL, SEPTEMBER 2005 – SEPTEMBER 2006

HMRC	Proper Shipping Name	Class	PKG. Group	HAZ. Zone	UN / NA
4935240	Sodium Hydroxide Solution	8	II, III	N/A	UN1824
4918311	Ammonium Nitrate	5.1	III	N/A	UN1942
4935645	Hexamethylened Lenediamine	8	II, III	N/A	UN1783

(References 219 and 221)

TABLE 2.2-209

BLN COL 2.2-1

TENNESSEE RIVER TONNAGES BY COMMODITY GROUP

Commodity	1991 (T.)	Percent of Total	2000 (T.)	Percent of Total	2030 Forecast (T.)	Percent of Total
Coal and Coke	20,773,434	49.3	18,881,050	38.0	14,451,698	25.6
Aggregates	8,520,175	20.2	11,196,098	22.5	17,025,592	30.1
All Other	2,962,966	7.0	4,502,692	9.1	3,127,475	5.5
Iron and Steel	1,163,249	2.8	3,630,829	7.3	6,038,859	10.7
Grains	3,558,992	8.4	3,588,008	7.2	5,267,935	9.3
Chemicals	2,458,868	5.8	2,935,479	5.9	5,076,332	9.0
Ores and Minerals	1,182,924	2.8	2,915,782	5.9	3,474,664	6.1
Petroleum Fuels	1,543,064	3.7	2,013,547	4.0	2,073,810	3.7

(Reference 226)

BLN COL 2.2-1

TABLE 2.2-210 (Sheet 1 of 2)
COMMODITIES SHIPPED PAST RIVER MILE 392 ON THE TENNESSEE RIVER, 2004

Commodity	Amount (1000 Short Ton)
Coal & Lignite[(a)]	949
Coal Coke[(a)]	11
Residual Fuel Oil	77
Lube Oil & Greases[(a)]	3
Asphalt, Tar & Pitch	424
Petroleum Coke[(a)]	136
Nitrogenous Fertilizer	30
Phosphatic Fertilizer	2
Potassic Fertilizer	16
Fertilizer & Mixes NEC	61
Other Hydrocarbons	26
Alcohols[(a)]	137
Sodium Hydroxide[(a)]	108
Inorganic Elements, Oxides, & Halogen	38
Wood Chips	1
Limestone	4
Gypsum	601
Phosphate Rock	2
Sand & Gravel	308
Iron Ore	5
Iron & Steel Scrap	350
Aluminum Ore	6
Non-Ferrous Ores NEC	2
Clay & Refracted Material	9
Slag	13

BLN COL 2.2-1

TABLE 2.2-210 (Sheet 2 of 2)
COMMODITIES SHIPPED PAST RIVER MILE 392 ON THE TENNESSEE RIVER, 2004

Commodity	Amount (1000 Short Ton)
Non-Metallic Minerals NEC	598
Cement & Concrete	61
Pig Iron	59
Ferro Alloys	23
I&S Plates & Sheets	99
I&S Bars & Shapes	3
Primary I&S NEC	62
Aluminum	2
Smelted Products NEC	4
Fabricated Metal Products	67
Wheat	155
Corn	191
Animal Feed, Prep.	75
Machinery (Not Elec)	2
Manufactured Products NEC	0

a) Classified as hazardous

NEC - Not elsewhere classified

BLN COL 2.2-1

TABLE 2.2-211 (Sheet 1 of 2)
COMMODITIES SHIPPED BETWEEN RIVER MILES 358 AND 363 ON THE TENNESSEE RIVER, 2004

Commodity	Amount (1000s T.)
Coal and Lignite[(a)]	21
Coal Coke[(a)]	45
Lube Oil & Greases[(a)]	4
Petroleum Coke[(a)]	102
Alchols[(a)]	3
Sodium Hydroxide[(a)]	48
Wood Chips	27
Gypsum	26
Iron Ore	101
Iron & Steel Scrap	1
Manganese Ore	5
Clay & Refracted Materials	1
Slag	3
Non-metallic Minerals NEC	20
Cement & Concrete	73
Pig Iron	44
I&S Bars & Shapes	18
Primary I&S NEC	10
Smelted Products NEC	6
Fabricated Metal Products	5
Wheat	10
Corn	829

BLN COL 2.2-1

TABLE 2.2-211 (Sheet 2 of 2)
COMMODITIES SHIPPED BETWEEN RIVER MILES 358 AND 363 ON THE TENNESSEE RIVER, 2004

Commodity	Amount (1000s T.)
Oats	15
Soybeans	232
Oilseeds NEC	143
Vegetable Oils	34
Animal Feed, Prep.	53

(Reference 225)

---

a) Classified as hazardous.

NEC - Not elsewhere classified

BLN COL 2.2-1

TABLE 2.2-212
HISTORICAL AIR TRAFFIC AT HUNTSVILLE INTERNATIONAL AIRPORT

Year	Total Passengers	Percent Change
1996	899,834	N/A
1997	1,016,802	13.0
1998	1,022,444	0.6
1999	1,050,377	2.7
2000	1,082,349	3.0
2001	968,954	-10.5
2002	989,093	2.1
2003	1,051,644	6.3
2004	1,193,370	13.5
2005	1,265,153	6.0
	AVERAGE	4.1

(Reference 232)

BLN COL 2.2-1

TABLE 2.2-213
PROJECTIONS FOR AIR TRAFFIC AT HUNTSVILLE INTERNATIONAL AIRPORT TO FISCAL YEAR 2025

Year	Total Passengers
2006	1,316,771
2007	1,370,496
2008	1,426,412
2009	1,484,609
2010	1,545,181
2011	1,608,225
2012	1,673,840
2013	1,742,133
2014	1,813,212
2015	1,887,191
2016	1,964,189
2017	2,044,327
2018	2,127,736
2019	2,214,548
2020	2,304,901
2021	2,398,941
2022	2,496,818
2023	2,598,688
2024	2,704,715
2025	2,815,067

(Reference 232)

2.3 METEOROLOGY

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add the following information at the end of DCD Section 2.3 introductory text.

BLN SUP 2.3-1 This section discusses the regional and local meteorological conditions, the onsite meteorological measurement program, and short-term and long-term diffusion estimates. Recent improvements in the National Oceanographic and Atmosphere Administration (NOAA) National Climatic Data Center (NCDC) data systems provide easy access to local meteorological data records. Current BLN site data is available for the period from 2006 – 2007. Most of the tabular data in this section are from these recent data sources, but there was also an extensive amount of meteorological data gathered and evaluations performed for the original licensing of Bellefonte Units 1 and 2 (BLNP) for the period from 1979 – 1982. In several cases, such as the reoccurrence rate of rare events based on many decades of observation, the offsite data is preferable.

2.3.1 REGIONAL CLIMATOLOGY

Add the following text at the end of DCD Subsection 2.3.1.

BLN COL 2.3-1 The description of the general climate of the region is based primarily on climatological records for Scottsboro and Huntsville, Alabama. This description utilizes that data as appropriate and is augmented by data from the licensing of the BLNP during the time period of 1979-1982 and more recent data from the permanent BLN site meteorological tower. The BLN site is located within Alabama state climatic division 2.

Topographical considerations and examination of the records indicate that meteorological conditions at Scottsboro are representative of the general climate of the region which encompasses the site. Since Scottsboro is the closest weather station, the tables and figures included are based primarily on Scottsboro data, when the period of record and observational procedures are considered adequate. Otherwise, data from the NOAA first order weather station in Huntsville are presented.

General discussions of the regional climate dating from the BLNP licensing period are still valid so the previous meteorological discussion and references from the BLNP Final Safety Analysis Report (FSAR) (Reference 201) are still applicable.

2.3.1.1 General Climate

The BLN site is located in a temperate latitude in northeastern Alabama about 250 miles north of the Gulf of Mexico, and in a region which is strongly influenced much of the year by the Azores-Bermuda anticyclonic circulation (see Figure 2.3-201, Reference 202). In late summer and fall, the position of the subtropical high is such that the region experiences extended periods of fair weather and light wind conditions. In winter and early spring the frequency of eastward moving migratory highs or low-pressure systems increases, alternately bringing cold and warm air masses into the BLN site area. Frequent and prolonged incursions of warm moist air from the Atlantic Ocean and the Gulf of Mexico are experienced from late spring through summer. Because of the prominent valley-ridge topographical features that dominate the site area, the low-level wind pattern is characteristic of a valley-flow regime with dominant frequencies of downvalley (north through northeast) and upvalley (south through southwest) wind directions. Above the level of valley-ridge influence the airflow pattern becomes regional in character with more nearly uniform directional distribution with slightly predominant southeasterly, southwesterly, and northerly winds. It is expected that the surface area of the Tennessee River in the site area is not large enough to produce a detectable lake to land breeze resulting from differential surface heating between land and water.

Temperatures in the region indicate warm summers and mild winters. Normally, in the BLN site area, January maximum temperatures are between 50°F and 55°F with minima between 30°F and 35°F. Maximum and minimum temperatures based on data from Scottsboro spanning the years 1882-2002 are shown on Figure 2.3-202 and Figure 2.3-203, respectively. In July, average minimum temperatures are in the vicinity of 65°F and 70°F, while the average afternoon maximum exceeds 90°F. Relative humidity for the year averages near 70 percent (Figure 2.3-204).

Precipitation in this area averages 57 inches annually and is normally well distributed throughout the year (Table 2.3-201). Figure 2.3-205 shows a gradually increasing trend in the annual precipitation. Winter is usually the wettest season, with more than 15 inches, while fall is the driest season, with about 12 inches. March is the only month to average more than six inches of rainfall, while about three to four inches are recorded at most locations in the site area in both September and October (see Figure 2.3-208). Average winter snowfall in the northeast corner of Alabama is 1.8 inches (Table 2.3-201).

Air Quality

Relative potential for air pollution can be demonstrated by the seasonal distribution of atmospheric stagnation cases that persist for at least four days. Data for the 50-year period (1948 to 1998), analyzed in Reference 203, show that, in the central Gulf Coast states, air stagnation conditions exist 5-10 percent of the time. Most air stagnation events happen in an extended summer season from May to October. This is the result of the weaker pressure and temperature gradients, and therefore weaker wind circulations during this period. In the eastern U.S., there is a relative minimum of stagnation in July accompanied by relative maxima in May-June and August-October. This mid-summer decrease of air stagnation is due to the impact of the Bermuda high-pressure system on the eastern United States. The Bermuda high is strongest in July, and hence the meridional wind in the Gulf States is a maximum then due to the increased pressure gradient, resulting in a relative minimum of air stagnation. Therefore, the Bermuda high is an additional and unique controlling factor for air stagnation conditions over the eastern United States, besides the seasonal cycle of minimum wind in summer and maximum wind in winter.

Another unique feature of air stagnation in the eastern United States is its early onset in May, compared to the onset in June in the west and central U.S. This results in a prolonged, but weaker air stagnation season in the eastern United States (Reference 203). For the eastern United States, the results show a regionally averaged mean annual cycle of, six cases in the spring, 14 cases in the summer, and 11 cases in the fall, for the region. For the region around the BLN site, the mean number of stagnation cases was 0.50 in May and June, 0.25 in July, and 0.75 in August, September, and October. Based on the National Climatic Data Center (NCDC) SCRAM Mixing Height Data for Nashville, Tennessee (Reference 204), the mean midmorning mixing height for the area is about 572 meters in the winter, 542 meters in the spring, 405 meters in the summer, 415 meters in the fall, and 484 meters annually. The mean afternoon mixing height for the area is about 804 meters in the winter, 1527 meters in the spring, 1741 meters in the summer, 1231 meters in the fall, and 1325 meters annually (see Table 2.3-303). These results are in good general agreement with the data provided by Holzworth (Reference 231).

Climate

The climate of Alabama is humid and subtropical with a short cold season and a relatively long warm season. The predominant air mass over the region during most of the year is maritime tropical with origins over the Gulf of Mexico. In the winter, occasional southward movements of continental polar air from Canada bring colder and drier air into Alabama and the northern parts of the state receive occasional short-lived snowfalls. However, cold spells seldom last more than three or four days.

The summer climate is almost wholly dominated by the westward extension of the Bermuda High, a subtropical, semi-permanent anticyclone. The prevailing southerly winds provide a generous supply of moisture and this, combined with thermal instability, produces frequent afternoon and evening showers and thundershowers. The convective thundershowers of the summer season are more numerous than frontal type thunderstorms. However, the thunderstorms associated with the occasional polar front activity in late winter and early spring are more severe and sometimes produce tornadoes. Alabama is south of the average track of winter cyclones, but occasionally one moves across the state. Alabama is also occasionally in the path of tropical storms or hurricanes.

Snowfall is not a rare event in northeast Alabama. During the 79 years from 1927 through 2005, measurable snow fell on Scottsboro in 33 years. Table 2.3-202 shows that during these 79 years, snow or sleet fell in January in 16 years and, in February, in 15 years (Reference 205).

An ice storm (also called glaze ice) is the accretion of generally clear and smooth ice, formed on exposed objects by the freezing of a film of supercooled water deposited by rain, drizzle, or possibly condensed from supercooled water vapor. The weight of this ice is often sufficient to greatly damage telephone and electric power lines and poles. Most glaze is the result of freezing rain or drizzle falling on surfaces with temperatures between 25°F and 32°F (Reference 206). The glaze ice belt of the United States includes all of the area east of the Rocky Mountains. However, in the Southeast and Gulf Coast sections of the country, below freezing temperatures seldom last more than a few hours after glaze storms.

The general direction of airflow across the region is from the southerly sectors during much of the year, although the prevailing direction may be from one of the northerly sectors during some months.

The temperature regime of the region can be described by the data shown in Table 2.3-214. From 2001 to 2005, the dry bulb temperature, corresponding to the maximum wet bulb temperature, during the summer months in Huntsville was 90°F. The peak average maximum monthly temperature in Huntsville from 1959 through 2005 was 89.4°F and the lowest average minimum monthly temperature was 30.2°F (see Table 2.3-203). The maximum temperature recorded at the BLN site during 1979 - 1982 was 99.7°F while the winter extreme minimum was -3.9°F (see Table 2.3-264). From 2006 to 2007, the maximum dry bulb temperature during the summer at the BLN site was 96.4°F, while the winter extreme minimum was 16.3°F. Site data from 1979-1982 agrees with these data. The BLN design basis ambient temperature and humidity statistics for use in establishing heat loads are provided in Table 2.3-203.

Table 2.3-263 presents temperature means and extremes for Scottsboro collected over a twenty-nine year period. Table 2.3-264 gives the temperature means and extremes for the BLN site. The values from the BLNP FSAR (Reference 201) date from 1979-1982, and represent site specific data taken at that time. Current data taken at BLN over a one-year period are given in Table 2.3-265, and are consistent with the BLNP FSAR data.

Climatic records of humidity in Huntsville are shown in Table 2.3-205. These data show that relative humidity in the region is high throughout the year. Nighttime relative humidities are highest in summer and fall and lowest in the spring. Daytime humidities are highest in the summer and winter. Seasonal variations are in the vicinity of five to 15 percent. Highest relative humidities occur in the early morning hours (00:00 - 06:00), averaging greater than 80 percent during all months. Lowest relative humidities occur during early and mid afternoon with averages ranging from approximately the mid-50s to the mid-60s for all months.

The relative humidity at the BLN site follows this same general trend (see Table 2.3-206).

Mean annual precipitation in the state ranges from about 57 inches in northeastern Alabama (Figure 2.3-205) to 66 inches in the southwestern (Mobile) part of the state (Reference 207). The fall months are typically the driest of the year (see Figure 2.3-208). Yearly average precipitation at the BLN site for 1979-1982 is approximately 48 inches (Table 2.3-266) and at Huntsville for the period of 2001 to 2005 was about 52 inches (Table 2.3-267).

Local site meteorological conditions are expected to result almost entirely from synoptic-scale atmospheric processes. That is, the local site does not have a unique micro-climate but rather the local meteorology is consistent with the regional meteorology. There are two exceptions caused by local effects due to the Tennessee River. First, there is higher humidity directly adjacent to the Tennessee River, and so the site humidity data is more appropriate for site estimates than the Scottsboro or Huntsville data. Second, there is possibility of channeling of low-level winds along the River Valley. Table 2.3-204 gives the most common wind direction and wind speed at the BLN Site.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

The regional meteorological conditions that are relevant to the design and operating bases for the BLN site are discussed below. A comparison of BLN site characteristics with the AP1000 DCD design parameters is given in FSAR Table 2.0-201.

2.3.1.2.1 Severe Weather Phenomena

This section describes severe weather phenomena that may require consideration in design of safety-related structures, systems and components. Most recent data is taken from the NCDC Storm Event database that covers the period of 1950 through 2002 (Reference 208), but even longer data periods are used for some phenomena to try to capture the occurrence of rare events.

Severe synoptic-scale storms are relatively infrequent in the BLN site area. Hurricanes penetrating this far inland have dissipated to tropical depressions. The effects of such storms are generally restricted to local flooding from heavy rains. Damage from snow, freezing rain, or ice storms in midwinter are uncommon. The Southeast Regional Climate Center snowfall records for Scottsboro (1927-2005) and Huntsville (1959-2005) show maximum daily snowfall amounts of 12.0 and 15.7 inches, respectively (References 205 and 209). Based on the evaluations given in "Extreme Ice Thicknesses from Freezing Rain," September 2004 (Reference 211), the probability of freezing rain (glaze ice) with a thickness of 15 mm (0.59 in) at the BLN site, in any year is two percent. The probability of freezing rain with a thickness of 20 mm (0.79 in) at the BLN site, in any year, is one percent (Reference 205).

2.3.1.2.1.1 Hurricanes

During the period 1899 to 2002 there were 123 documented hurricanes that affected the Middle Gulf Coast (Texas, Louisiana, Mississippi, Alabama, and Florida) (References 212 and 213). This total is based on the number of unique storms impacting these states and not on the total number of storms that affect each state. Of these, 42 (34.1 percent) were Category 1, 31 (25.2 percent) were Category 2, 35 (28.5 percent) were Category 3, 12 (9.8 percent) were Category 4 and 3 (2.4 percent) were Category 5 hurricanes. Table 2.3-207 presents a monthly breakdown of the 123 hurricanes and provides a definition of the storm categories.

Tropical cyclones, including hurricanes, lose strength as they move inland from the coast and the greatest concern for an inland site is possible flooding due to excessive rainfall. Although no hurricanes have reached Jackson County, sixteen tropical storms have passed through Jackson County. The Scottsboro rainfall extremes given in Figure 2.3-207 include possible hurricane and tropical cyclone effects. The maximum one-day rainfall in Scottsboro for the years 1927-2005 was 6.8 inches and was not associated with a hurricane or tropical storm (Reference 205).

2.3.1.2.1.2 Tornadoes

The probability that a tornado will occur at the BLN site is low. Records show that in a 55-year period (1950-2005) there were 21 tornadoes reported in Jackson County, the location of the site. The data reported by the NOAA's National Environmental Satellite, Data, and Information Service (NEDSIS) (Reference 208) is given in Table 2.3-208. From this data, the average tornado area in Jackson County, ignoring events of no recorded path length, is approximately 0.8 square miles. Using the principle of geometric probability described in Reference 215, a mean tornado path area of 0.8 square miles, and an average tornado frequency of 0.38 per year for the area of Jackson County (1069 mi^2), the point probability of a tornado striking the plant is 2.84×10^{-4}/year. This corresponds to an estimated recurrence interval of 3516 years. The tornadoes reported during the years 1950-2005 in the vicinity of Jackson, De Kalb, Marshall, and Madison Counties in Alabama, Franklin and Marion Counties in Tennessee, and Dade County in Georgia are shown in Table 2.3-208.

During the period 1950 to 2005, a total of 151 tornadoes touched down in these counties that have a combined area of 4447 square miles (References 216 and 217). These local tornadoes have a mean path area of 1.06 square miles excluding tornadoes without a length specified. The site recurrence frequency of tornadoes can be calculated using the point probability method as follows:

Total area of tornado sightings = 4447 sq.mi.

Average annual frequency = 151 tornadoes/56 years = 2.70 tornadoes/year

Annual frequency of a tornado striking a particular point P = [(1.06 mi^2/tornado) (2.70 tornadoes/year)] / 4447 sq.mi. = 0.00064 yr^{-1}

Mean recurrence interval = 1/P = 1552 years.

This result shows that the frequency of a tornado in the immediate vicinity of the site is less than the frequency in the surrounding counties. Another methodology for determining the tornado strike probability at the BLN site is given in NUREG/CR-4461. Based on a two longitude and latitude box centered on the BLN site, the number of tornadoes is 385. The corresponding expected maximum tornado wind speed and upper limit (95 percentile) of the expected wind speed is given below with the associated probabilities.

Probability	Expected maximum tornado windspeed mph	Upper limit (95 percent) of the expected tornado windspeed mph
10^{-5}	182	190
10^{-6}	237	245
10^{-7}	285	294

The design basis tornado characteristics for the BLN site are given in Subsection 2.3.1.4.

2.3.1.2.1.3 Thunderstorms

Locations in northeast Alabama and extreme south central Tennessee experience approximately 17 thunderstorms events per year. Regionally, storms with wind speeds reaching 35 to 50 mph may occur several times a year. During the period 1950-2005, there were 132 thunderstorm or high wind events in Jackson County (see Table 2.3-209). Of these, 86 events had a wind speed of greater than or equal to 50 knots (≥57 mph). The number of high wind speed (50 knots) events is 1.5 per year in Jackson County. Approximately 51 percent of the thunderstorms in Jackson County occur during the warm months (June-August), indicating that the majority are warm air-mass thunderstorms. From 1950-2005, 933 thunderstorms are listed for this seven county region, with Jackson County receiving 14.1 percent, DeKalb County receiving 14.5 percent, Marshall County receiving 16.0 percent, Madison County receiving 28.9 percent, Franklin County, Tennessee receiving 11.9 percent, Marion County, Tennessee receiving 10.0 percent, and Dade County, Georgia receiving 4.6 percent of the thunderstorms. (Reference 208)

2.3.1.2.1.4 Lightning

Data on lightning strike density is becoming more readily available as a result of the National Lightning Detection Network (NLDN). The NLDN has measured cloud-to-ground (CG) lightning for the contiguous United States since 1989. Prior to the availability of this data, isokeraunic maps of thunderstorm days were used to predict the relative incidence of lightning in a particular region. A general rule, based on a large amount of data from around the world, estimates the earth flash mean density to be one to two cloud to ground flashes per 10 thunderstorm days per square kilometer (Reference 218). The annual mean number of thunderstorm days in the site area is conservatively estimated to be 55 based on interpolation from the isokeraunic map (Reference 219); therefore it is estimated that the annual lightning strike density in the BLN site area is 28 strikes per square mile per year. Other studies gave a ground flash density, GFD (strikes/km^2/yr), based on thunderstorm days per year (TSD) as GFD = 0.04 $(TSD)^{1.25}$ = 0.04 $(55)^{1.25}$ = 6 strikes/km^2/yr or 16 strikes/mi^2-yr. (Reference 220).

Recent studies based on data from the NLDN (Reference 221) indicates that the above strike densities are upper bounds for the BLN site. Mean annual flash density given in (Reference 221) for 1989-96 is 3 - 5 strikes/km^2/yr or 8-13 strikes/mi^2-yr in Northeast Alabama.

2.3.1.2.1.5 Hail

From 1950 through 2005, 504 hailstorms occurred in the region annually, with Jackson County receiving approximately 13 percent, DeKalb County receiving 19 percent, Marshall County receiving 16 percent, Madison County receiving 30 percent, Franklin County, Tennessee receiving eight percent, Marion County, Tennessee receiving seven percent, and Dade County, Georgia receiving seven percent of the hailstorms, as shown in Table 2.3-210. For this table, each occurrence of hail was counted as an individual event, even if two counties recorded hail simultaneously. The most probable months of occurrence of hail are April and May. Property damage occurs infrequently, with 16 recorded events in Jackson County, 24 in DeKalb County, 18 in Marshall county, 24 in Madison County, one in Franklin county, Tennessee, two in Marion County, Tennessee, and one in Dade County, Georgia in this 56-year period. The maximum size of hail reported from 1950 through 2005 in Jackson County, Alabama was 2.75 inches.

2.3.1.2.1.6 Regional Air Quality

The Clean Air Act, which was last amended in 1990, requires the U.S. Environmental Protection Agency (EPA) to set National Air Quality Standards for pollutants considered harmful to the public health and the environment. The EPA Office of Air Quality Planning and Standards has set National Ambient Air Quality Standards for six principle pollutants, which are called "criteria" pollutants. Units of measure for the standards are parts per million (ppm), milligrams per cubic meter (mg/m^3), and micrograms per cubic meter of air ($\mu gm/m^3$). Areas are either in attainment of the air quality standards or in non-attainment. Attainment means that the air quality is better than the standard.

The U.S. Environmental Protection Agency 8-hour ozone standard given in 40 CFR 50.10 is 0.08 ppm. The only areas in Alabama which are in nonattainment with the 8-hour ozone standard are Jefferson County and Shelby County (Reference 222). Currently designated (as of March 2, 2006) nonattainment areas of Alabama for the criteria pollutants (carbon monoxide, lead, nitrogen dioxide, particulate matter (PM-10), particulate matter with a diameter less that 10 micron), particulate matter (PM-2.5, particulate matter with a diameter less than 2.5 micron), ozone, and sulfur oxides) are:

Jackson Co		
	PM-2.5	Chattanooga, AL-TN-GA
Jefferson Co		
	8-Hr Ozone	Birmingham, AL
	PM-2.5	Birmingham, AL
Shelby Co		
	8-Hr Ozone	Birmingham, AL
	PM-2.5	Birmingham, AL
Walker Co		
	PM-2.5	Birmingham, AL

The above classification of Jackson County as nonattainment for PM-2.5 is a result of being included in the AL-TN-GA area, which includes Chattanooga Tennessee. Jackson County is part of the Tennessee River Valley (Alabama)-Cumberland Mountains (Tennessee) Interstate Air Quality Control Region. For Jackson County itself, the levels of all criteria pollutants are well within the EPA air quality standards for 2003 through 2005.

The ventilation rate is a significant consideration in the dispersion of pollutants. Higher ventilation rates are better for dispersing pollution than lower ventilation rates. The atmospheric ventilation rate is numerically equal to the product of the mixing height and the wind speed within the mixing layer. A tabulation of daily mixing heights and mixing layer wind speeds for both morning and afternoon was obtained from the National Climatic Data Center for 1984 through 1987 and 1990 through 1991 at the Nashville Metropolitan Airport (Reference 223). This data was used to generate the morning and afternoon ventilation rates in Table 2.3-211.

Morning ventilation is less than 4100 m^2/s throughout the year, and is less than 1500 m^2/s from June through October. Afternoon ventilation is higher than 7100 m^2/s from March through September, but lower than 5200 m^2/s from November through January. The highest daily air pollution potentials exist during the lower morning ventilation rates from May through October. Lowest air pollution potentials occur from November through March due to the relatively high morning mean ventilation rates.

Other data sources provide independent checks of this conclusion. The annual average air stagnation cases for Alabama over a fifty year period (1948-1998) was four cases per year with a mean duration of five days (Reference 203). The annual mean days of air stagnation were given as 20 for Alabama. This report also concluded that the highest number of air stagnation days occurred from July through October with the lowest air stagnation days from November through March. The number of air stagnation days in the Alabama region exhibited a decreasing trend over the 50 years evaluated (see Figure 2.3-303).

2.3.1.2.2 Severe Winter Storm Events

The occurrences and durations of recorded ice storms and heavy snowstorms in the vicinity of the BLN site for the period 1950-2005 is shown in Table 2.3-212. From these data, the frequency of winter storms in the BLN area is estimated to be 9.6 events per year in this regional area. For Jackson County, the frequency is 1.4 events per year.

The equivalent ice thickness due to freezing rain with concurrent 3-second gust speeds for a 100-year mean recurrence interval is given in "Extreme Ice Thicknesses from Freezing Rain" (Reference 211), as 0.75 inch for the Northeast Alabama area of the BLN site.

The observed maximum winter (November through March) precipitation amounts (water equivalent) during any consecutive 48-hour period at the BLN site for the indicated winter seasons are given in Table 2.3-213. These data were analyzed by the Gumbel-Lieblein method described in Reference 215 with the following results:

Return Period (Years)	Maximum 48 Hr Winter Precipitation, Water Equivalent inches
10	4.58
25	5.18
50	5.63
100	6.07
500	7.09
1000	7.53

Thus, it is estimated that a value of 7.53 inches (water equivalent) is ultra-conservative (based on a 1000-yr return period) for the 48-hour probable maximum winter precipitation (PMWP) at the BLN Site.

The Southeast Regional Climate Center data (Reference 205) identifies that the greatest snowfall in Scottsboro during its period of data, 01/01/1927 to 12/31/2005, occurred on March 13, 1993. This storm deposited 12 inches of snow in Scottsboro. Since this data review covers at least 79 years back to 1927, it is possible to conclude with 79 percent confidence that the 100-year snowfall maximum is 12 inches.

In the Scottsboro/Bellefonte area, snow melts and/or evaporates quickly, usually within 48 hours, and before additional snow is added. Since the plant site is subjected to a subtropical climate with mild winters, prolonged snowfalls or large accumulations of snow or ice on the ground and structures are not anticipated.

2.3.1.2.2.1 Estimated Weight of the 100-year Return Snowpack

Snowpack, as used in this section, is defined as a layer of snow and/or ice on the ground surface, and is usually reported daily, in inches, by the National Weather Service at all first order weather stations.

The density of the snowpack varies with age and the conditions to which it has been subjected. Thus, the depth of the snowpack is not a true indication of the pressure that the snowpack exerts on the surface that it covers. A more useful statistic for estimating the snowpack pressure is the water equivalent (in inches) of the snowpack.

To estimate the weight of the 100-year snowpack at the BLN site, the maximum reported snow and/or ice depths at Scottsboro, Alabama was determined. The current Southeast Regional Climate Center data (Reference 205) identifies that the greatest snow depth in its period of data, 1/1/1927 to 12/31/2005, occurred on February 15, 1958. The snow depth recorded on this date was 10 inches. Since this data review covers at least 79 years back to 1927, it is possible to conclude with 79 percent confidence that the 100-year snow-pack maximum is 10 inches.

Freshly fallen snow has a snow density (the ratio of the volume of melted water to the original volume of snow) of 0.07 to 0.15, and glacial ice formed from compacted snow has a maximum density of 0.91 (Reference 224). In the BLN site area, snow melts and/or evaporates quickly, usually within 48 hours, and before additional snow is added; thus, the water equivalent of the snowpack can be considered equal to the water equivalent of freshly fallen snow. A conservative estimate of the water equivalent of snowpack in the BLN site area would be 0.20 inches of water per inch of snowpack. Then, the water equivalent of the 100-year return snowpack would be 10 in snowpack x 0.2 inches water equivalent/inch snowpack = 2.0 inches of water.

Since one cubic inch of water is approximately 0.0361 pounds in weight, a one inch water equivalent snowpack would exert a pressure of 5.20 pounds per square foot (0.0361 lbm/cu in x 144 sq in). For the 100-year return snowpack, the water equivalent would exert a pressure of 10.4 pounds per square foot (5.2 lbm/sq ft-inch x 2.0 inches).

2.3.1.2.2.2 Estimated Weight of the 48-hour Maximum Winter Precipitation

The 48-hour PMWP at the BLN Site is estimated to be 24.7 inches based on HMR 53 (Reference 233).

The rain load is considered separately from the snow and ice roof load. The roofs of the nuclear island have no lips around the edges; therefore, water and snow melt buildup on the roofs of the nuclear island are negligible. The shield building roof is sloped with no lips around the edge of the roof to allow water buildup. The PCS tank is flat with no lip; however, there is the central hole that can allow water to drain down in between the shield wall and the SCV, but not to accumulate on the roof area. The auxiliary building has sloped roofs with three varying elevations (high points given); Area 1&2 155'-6", Area 3&4 163'-0", and Area 5&6 180'-9". The south side (directions are relative to called North in the DCD) of the nuclear island wall 1 is above the radwaste building roof elevation 136'-4". The east side of the nuclear island, wall 1, is below the annex building roof elevation 183'-4.25", but the auxiliary building roof is sloped so that Areas 3&4 drain on to Areas 1&2 roof, which is sloped from east to west. There are no lips on the roof of the auxiliary building that could prevent the flow of water. The north side of the nuclear island is also below the turbine building roof elevation 246'-3", but again Areas 1&2 are sloped such that the run-off will flow off the west side. As a result of the nuclear island roof design, there is no loading from the PMWP.

2.3.1.2.2.3 Weight of Snow and Ice on Safety-Related Structures

Because the plant site is subjected to a subtropical climate with mild winters, prolonged snowfalls or large accumulations of snow or ice on the ground and structures are not anticipated.

The estimated depth of the 100-year return snowpack is 10 inches, or 2.0 inches of water equivalent, as discussed above. Safety-related structures at the BLN site would be designed to withstand 10.4 pounds per square foot. No damage from snow or ice loading on structures is expected, because the DCD design loading is 75 pounds per square foot. Comparison of the BLN site characteristics with the AP1000 DCD design parameters is given in Table 2.0-201.

2.3.1.2.3 Probable Maximum Annual Frequency and Duration of Dust Storms

The occurrence of dust, blowing dust, or blowing sand is a rare phenomenon in the BLN site area. Although there are categories for dust and sand in the NCDC meteorological database, no hours are identified under this category for Jackson County in the period 01/01/1950 to 04/30/2006.

2.3.1.3 Meteorological Data Used for Evaluating Heat Removal Capacity

Meteorological data is used in accident analyses and other analyses to determine the effectiveness of safety related heat removal systems. This section discusses BLN site and local area meteorological data that may impact design of safety related heat removal systems.

2.3.1.3.1 Meteorological Parameters

The controlling meteorological parameters required for the analysis of cooling tower performance are the wet bulb temperature and the coincident dry bulb temperature. Table 2.3-214, Table 2.3-215, and Table 2.3-216 presents data on these parameters from Huntsville Alabama for the years 2001-2005. The meteorological data used in the evaluation of cooling tower plumes is given in Subsection 2.3.2.2.1. The maximum dry bulb temperature with coincident wet bulb temperature, the maximum wet bulb temperature (non-coincident), and the maximum and minimum dry bulb temperatures are given in Table 2.3-203. Comparison of the BLN site characteristics with the AP1000 DCD design parameters is given in Table 2.0-201.

2.3.1.3.2 Worst 1-Day, 5-Day, and 30-Day High Temperature Periods

The worst day wet bulb temperature is based on data from the Huntsville Weather Station. The hourly data for the worst 1-day, July 25, 2005 are shown in Table 2.3-214. The daily average wet bulb and coincident dry bulb temperatures for the worst 5-day period are shown in Table 2.3-215. The daily average wet bulb and coincident dry bulb temperatures for the worst 30-day period are shown in Table 2.3-216.

2.3.1.4 Design Basis Tornado Parameters

The design basis tornado characteristics are specific to the site and region of the country in which the site is located. However, rather than conducting site research on tornado characteristics, most sites in the past licensing proceedings have relied on NRC endorsed studies that set conservative values for key design basis tornado characteristics. These characteristics were then used in the design of the subject facility.

Regulatory Guide 1.76, based on WASH-1300, has been used since the 1970s by the industry to establish the appropriate design basis tornado characteristics, depending on the proposed site location in the country. The design basis tornado characteristics defined for this project, as listed below, are based on the guidance in Regulatory Guide 1.76 for an exceedance probability of 10^{-7} per year. The below listed characteristics are associated with a Region 1 site.

Design Basis Tornado Characteristics

	BLN Site
Maximum wind speed, mph	230
Rotational speed, mph	184
Maximum Translational speed, mph	46
Radius of maximum rotational speed, ft	150
Pressure drop, psi	1.2
Rate of pressure drop, psi/sec	0.5

The above maximum tornado wind speed is less than the AP1000 DCD value of 300 mph. In accordance with Regulatory Guide 1.76, the wind velocities and pressures are not assumed to vary with height.

2.3.1.5 100-Year Return Period Fastest Mile of Wind

The fastest wind speed recorded in 55 years (1950-2005) in Jackson County is 74.8 mph. A Gumbel-Lieblein extreme value analysis of this data gave an estimated value of 77 mph for the 100-year return period fastest mile of wind in Jackson County.

The fastest hourly averaged wind speed recorded by the Bellefonte Unit 1 and 2 meteorological tower at 33 feet in the period from 1979 through 1982 was 28.6 mph in 1981. A Gumbel-Lieblein extreme value analysis of this data gave an estimated value of 35 mph for the 100-year return period fastest mile of wind at the BLN site. This result may be low due to the limited data collection period.

The design basis wind velocity is based on the data from ASCE 7-95 (Reference 225). From Figure 6-1 of ASCE 7-95, the 50-year return 3-second gust wind speed at 33-ft above ground for the BLN site is 90 mph. This value is for Exposure Category C (open terrain) which is appropriate for the BLN Site. This gives a design basis 100-year return wind speed of 96 mph based on Table C6-5 of ASCE 7-95. A comparison of the AP1000 DCD design parameter wind speed with the BLN site characteristic is provided on Table 2.0-201.

2.3.2 LOCAL CLIMATOLOGY

Add the following text at the end of DCD Subsection 2.3.2.

BLN COL 2.3-2 This section discusses the local meteorological conditions at the BLN site. A comparison of BLN site characteristics with the AP1000 DCD design parameters is given in FSAR Table 2.0-201.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

The following sections contain information on wind, air temperature, atmospheric water vapor, precipitation, fog and smog, atmospheric stability, and mixing heights at the BLN site and surrounding area.

2.3.2.1.1 Winds

2.3.2.1.1.1 Site Wind Distribution

One year of data (2006 to 2007) from the permanent meteorological facility at the BLN site, has been evaluated and summarized. Both concurrent and long-term data from the nearest and most representative source (Huntsville and Scottsboro) were examined and compared with each other and with the onsite data. The onsite data collected for the BLNP FSAR was also evaluated.

The nearest federal weather station for which long-term data is available is Huntsville, Alabama, approximately 45 miles west of the site. The site is located in a transition between marked mountain-valley topography and the low rolling hills characteristic of the Appalachian foothills (Figure 2.3-288 and 2.3-289). Plots of the maximum elevation versus distance from the center of the plant in each of the sixteen 22 1/2-degree compass point sectors to a distance of five miles from the site are shown on Figure 2.3-287. Similar plots for a distance of fifty miles from the site are provided on Figure 2.3-286. A topographic plan of the area within five miles of the plant is provided on Figure 2.3-288. Figure 2.3-289 gives the topographic plan within 50 miles of the site. The terrain in the Huntsville area is more indicative of Appalachian foothill topography. The BLN site is located on a broad flat Tennessee River flood plain, with mountain ridges of 1400 to 1600 feet above MSL to the northeast, east, and southeast (Figure 2.3-288 and Figure 2.3-289).

Long-term temperature and precipitation records from Scottsboro were compared to records from Huntsville. This comparison indicates that, for these parameters, data from Huntsville reasonably represents meteorological conditions at the site. Presumably, this is indicative of the similarity in controlling synoptic influences throughout the region. Other meteorological parameters are assumed to be subject to the same synoptic controls. Data from the original BLNP FSAR is primarily used to determine the representativeness of the 1-year of onsite record for long-term averages.

Wind monthly and annual joint frequency distributions of wind direction and wind speed for wind instruments at 10 meters at BLN are presented in Tables 2.3-230, 2.3-231, 2.3-232, 2.3-233, 2.3-234, 2.3-235, 2.3-236, 2.3-237, 2.3-238, 2.3-239, 2.3-240, 2.3-241, and 2.3-242 using the original 1979 -1982 BLNP meteorological data. The data show a valley-flow regime, with dominant frequencies of upvalley (south through southwest) and downvalley (north through northeast) wind directions. Monthly wind data for the 10-meter level shows what appears to be a seasonal influence on the occurrences of upvalley and downvalley winds. Downvalley wind direction occurrences are more frequent during the late summer and early fall, while upvalley winds occur more often during late winter and early spring. This characteristic is also illustrated in Figures 2.3-209, 2.3-210, 2.3-211, 2.3-212, 2.3-213, 2.3-214, 2.3-215, 2.3-216, 2.3-217, 2.3-218, 2.3-219, 2.3-220, 2.3-221, 2.3-222, 2.3-223, 2.3-224, and 2.3-225.

The wind speed data show very few hours of calm conditions at either measurement level. About 47 percent of the hourly values were less than 4.0 mph and less than one percent were greater than 18.0 mph at the 10-meter level.

Wind speed and wind direction occurrences frequencies for the 5-year (2001-2005) Huntsville NWS station data, 4-year (1979-1982) site data, and the concurrent 1-year (2006 through 2007) are given in Tables 2.3-217, 2.3-218, 2.3-219, 2.3-220, 2.3-221, 2.3-222, 2.3-223, 2.3-224, 2.3-225, 2.3-226, 2.3-227, 2.3-228, and 2.3-229, Tables 2.3-230, 2.3-231, 2.3-232, 2.3-233, 2.3-234, 2.3-235, 2.3-236, 2.3-237, 2.3-238, 2.3-239, 2.3-240, 2.3-241, and 2.3-242, and Tables 2.3-243, 2.3-244, 2.3-245, 2.3-246, 2.3-247, 2.3-248, 2.3-249, 2.3-250, 2.3-251, 2.3-252, 2.3-253, 2.3-254, and 2.3-255 respectively.

Wind data is available from both the Huntsville meteorological station and the BLN meteorological tower. Both sets of data are discussed here to provide a fuller description of winds in the area.

2.3.2.1.1.1.1 Huntsville Wind Distribution

Tables 2.3-217, 2.3-218, 2.3-219, 2.3-220, 2.3-221, 2.3-222, 2.3-223, 2.3-224, 2.3-225, 2.3-226, 2.3-227, and 2.3-228 provide monthly percent joint frequency distributions for wind directions and speeds, based on a 5-year period of record from 2001 through 2005, for Huntsville. Table 2.3-229 provides an annual summary of the data. On an annual basis, Huntsville wind data collected in the five years 2001 through 2005 shows that northerly (N-NW through N-NE) is the most frequent (18.8 percent) wind direction. The wind is from the southern quadrant (S-SE through S-SW) 18.6 percent of the time. Westerly (W-SW and W-NW) and easterly (E-NE and E-SE) winds are the least frequent with frequencies of 11.1 percent and 17.8 percent, respectively. Southerly components prevail in spring, easterly components prevail in summer and fall, while northerly components prevail in the winter. At the Huntsville NWS station, winds average 9.1 mph from January through April, and 7.7 mph from May through December. Mean annual wind speed is 8.2 mph (Table 2.3-229).

The Huntsville meteorological station winds are presented graphically in Figures 2.3-226, 2.3-227, 2.3-228, 2.3-229, 2.3-230, 2.3-231, 2.3-232, 2.3-233, 2.3-234, 2.3-235, 2.3-236, 2.3-237, 2.3-238, 2.3-239, 2.3-240, 2.3-241, 2.3-242, and 2.3-243. These wind roses cover the period from 2001 through 2005 and represent the frequency of winds going to a particular direction by the length of the line in that direction. Huntsville records a usual pattern of winds coming from the north or south. During the summer and fall, winds from the east and southeast are more common. At Huntsville, winds from the west occur infrequently.

2.3.2.1.1.1.2 BLN Wind Data

The same wind data assessment was applied to BLN site data collected at the BLN meteorological tower for the period from 1979-1982 and 2006-2007. Monthly relative frequencies of wind direction and speed for the BLN site are shown in Tables 2.3-230, 2.3-231, 2.3-232, 2.3-233, 2.3-234, 2.3-235, 2.3-236, 2.3-237, 2.3-238, 2.3-239, 2.3-240, 2.3-241, and 2.3-242, for the years 1979 - 1982 and Tables 2.3-243, 2.3-244, 2.3-245, 2.3-246, 2.3-247, 2.3-248, 2.3-249, 2.3-250, 2.3-251, 2.3-252, 2.3-253, 2.3-254, and 2.3-255 for 2006-2007. The wind speeds are hourly averages and there are no zero speeds recorded between 1979-1982 or 2006-2007. Between 1979-1982 winds averaged 4.3 mph from May through December, 5.7 mph from January through April, and the mean annual wind speed was 4.8 mph. Between 2006-2007 winds averaged 3.8 mph from May through December, 4.9 mph from January through April, and the mean annual wind speed was 4.1 mph. The 1979-1982 and 2006-2007 BLN site winds are presented graphically in Figures 2.3-209, 2.3-210, 2.3-211, 2.3-212, 2.3-213, 2.3-214, 2.3-215, 2.3-216, 2.3-217, 2.3-218, 2.3-219, 2.3-220, 2.3-221, 2.3-222, 2.3-223, 2.3-224, and 2.3-225 and Figures 2.3-290, 2.3-291, 2.3-292, 2.3-293, 2.3-294, 2.3-295, 2.3-296, 2.3-297, 2.3-298, 2.3-299, 2.3-300, 2.3-301, and 2.3-302, respectively. In general, the wind roses for Huntsville show a more North to South trend than BLN, which has a more NE to SW trend.

2.3.2.1.1.1.3 Wind Direction Persistence

Hourly weather observation records from the National Weather Service at Huntsville, Alabama for the years 2001 through 2005 were examined for wind direction persistence. The longest persistence periods from a single sector (22.5 degrees), three adjoining sectors (67.5 degrees), and five adjoining sectors (112.5 degrees) were determined from each sector during each year. The results are shown in Tables 2.3-256, 2.3-257, and 2.3-258. During the period, the single sector persistence was greatest (19 hours) for the N, WNW, and ESE direction. The average maximum persistence (17 hours) was greatest for the north direction. For the persistence in three adjoining sectors, the NNE sector had the longest period of persistence (65 hours). The largest average maximum persistence (48 hours) was for the ESE sector, as shown in Table 2.3-257. The longest persistence period (108 hours) from five adjoining sectors occurred in the SSW sector (Table 2.3-258). The SE sector showed the greatest average maximum persistence (80.0 hours).

Wind persistence data similar to the above are shown in Tables 2.3-259, 2.3-260, and 2.3-261 for the BLN Site. The statistics shown in these tables cover the period from 1979-1982 and 2006-2007. Table 2.3-259 shows that the longest single sector persistence period was 22 hours from the SE and SW sectors. The SSW sector had the greatest average maximum persistence (14.2 hrs). For the persistence in three adjoining sectors, the SSW sector had the longest period of persistence (72 hours) and the largest average maximum persistence (48 hours) as shown in Table 2.3-260. The persistence data for five adjoining sectors (Table 2.3-261) shows the central NNE sector with the longest persistence period (88 hours) and the greatest average maximum persistence (69 hours).

Table 2.3-262 presents a summary of the maximum persistence period for the BLN site (in hours). The data demonstrate that it is not likely that any single wind direction would persist for a substantial period of time.

2.3.2.1.2 Air Temperature

Table 2.3-263 indicates that temperature extremes for Scottsboro, Alabama for the years 1971 through 2000 have ranged from the highest mean temperature of 81.8°F (July 1993) to the lowest mean of 26.8°F (January 1977) (Reference 226). Table 2.3-264 indicates that temperature extremes for BLN site during the years 1979 through 1982 have ranged from a record maximum temperature 99.7°F (July 1980) to a record minimum of -3.9°F (January 1982). The highest monthly mean was 78.6°F with the lowest monthly mean of 36.8°F. The data shows reasonable agreement between the two locations.

Table 2.3-265 presents the site temperature means and extremes for the year 2006-2007. A comparison of this year's data with the historic Bellefonte site data (1979-1982) is made in Figure 2.3-246. This figure shows good agreement between the current data and the historic data collected over a longer time period.

2.3.2.1.3 Atmospheric Moisture

Alabama experiences moderately high humidity during much of the year. At Huntsville, during the years 2001-2005, the annual average humidity is greater than 50 percent. Mean relative humidities for four time periods per day at Huntsville are shown in Table 2.3-205. The highest humidity is most frequent in the early morning hours with an annual average of 86 percent. In the summer, at times there develops a combination of high temperatures together with high humidities; this usually builds up progressively for several days and becomes oppressive for one or more days. Humidities of less than 50 percent occur on some days each month, usually in the early afternoon hours. The humidity drops under 50 percent on about eight percent of the October and November days; the number of days with such low humidities diminishes in the other months. In July and August low humidity is infrequent (Reference 227).

Table 2.3-206 and Table 2.3-306 show the mean relative humidities for four time periods per day at the BLN site for 1979-1982 and 2006-2007, respectively. This data agrees reasonably well with the Huntsville data.

2.3.2.1.3.1 Precipitation

2.3.2.1.3.1.1 Rain

Average monthly precipitation at the BLN Site between 1979-1982 follows a seasonal trend, reaching a maximum monthly mean in March (6.7 inches) and a minimum mean in October (2.2 inches). The maximum monthly precipitation at the BLN Site between 1979-1982 is 14.5 inches (Table 2.3-266). Average monthly precipitation at the BLN site between 2006-2007 follows a similar seasonal trend, reaching a maximum monthly mean in April (3.9 inches) and a minimum mean in September (0.1 inches). The maximum monthly precipitation at the BLN site between 2006-2007 is 3.9 inches (Table 2.3-307). Similar to the BLN Site between 1979-1982, the maximum mean monthly precipitation for Huntsville is in March (6.7 inches) and the minimum monthly mean is in October (2.1 inch). The maximum monthly mean precipitation in Huntsville is 14.5 inches (Table 2.3-267). The BLN Site rainfall data covers the time period from 1979-1982 and 2006-2007, while the Huntsville data covers the time period from 2001-2005 (Reference 227). Table 2.3-268 and Table 2.3-308 provides monthly frequency distribution of rainfall rates at the BLN Site for 1979-1982 and 2006-2007, respectively. Table 2.3-269 provides monthly frequency distribution of rainfall rates at Huntsville for 2001-2005.

In general, the Huntsville data appears to be representative of the BLN site area. The variations between the two locations from month to month, particularly during the summer months, are likely reflective of the occurrence of localized heavy shower and thunderstorm activity common in the area.

The maximum short period precipitation was determined for the BLN site based on data from Hershfiels and Miller (References 228 and 229). The maximum point precipitation values are given in Table 2.3-270. These values were interpolated from the maps of USWB Technical Papers 40 and 49. NOAA Technical Memorandum NWS HYDRO-35 (Reference 230) was consulted for updated (from Technical Paper 40, Reference 228) 5-minute, 15-minute, and 1-hour duration precipitation values (Table 2.3-271). Comparison of the AP1000 DCD precipitation design parameter to the BLN site characteristic is provided in Table 2.0-201.

2.3.2.1.3.1.2 Snow

Annual average snowfall in the BLN area is estimated to be two to four inches. This estimate is based on 36 years of record (1959-2005) at Huntsville (Reference 209) and 79 years of record (1927-2005) at Scottsboro (Reference 205). The annual snowfall in Scottsboro is shown on Figure 2.3-206. The Huntsville meteorology station reported an average snowfall of 3.8 inches in November through March as presented in Table 2.3-203. The maximum snowfall in Huntsville was 15.7 inches on December 31, 1963. The maximum snowfall depth recorded is 11.0 inches on January 1, 1964 (Reference 209). The maximum snowfall at Scottsboro was 10.0 inches on February 15, 1958 (Reference 205).

2.3.2.1.3.2 Fog

Fog is an aggregate of minute water droplets suspended in the atmosphere near the surface of the earth. According to National Weather Service definition, fog reduces visibility to less than 5/8 miles. Table 2.3-275 indicates that, over the period 2001 to 2005, Huntsville has averaged approximately 37 hours/year of fog. Table 2.3-275 also provides the maximum hours of fog per month and the average hours of haze per month.

2.3.2.1.3.3 Precipitation Wind Roses

Figures 2.3-247, 2.3-248, 2.3-249, 2.3-250, 2.3-251, 2.3-252, 2.3-253, 2.3-254, 2.3-255, 2.3-256, 2.3-257, 2.3-258, and 2.3-259 show the precipitation wind rose for the BLN site for the years 1979-1982. Table 2.3-272 provides the monthly precipitation by direction. This data shows that the highest rainfall frequency at BLN happens most often in the months of November through April, with the most common directions of E-SE through SE and N through N-NE. Winds speeds during precipitation average 5.3 mph annually.

Figures 2.3-273, 2.3-274, 2.3-275, 2.3-276, 2.3-277, 2.3-278, 2.3-279, 2.3-280, 2.3-281, 2.3-282, 2.3-283, 2.3-284, and 2.3-285 show the precipitation wind rose for Huntsville, Alabama based on data from the years 2001 through 2005. Table 2.3-273 provides the monthly precipitation by direction at Huntsville. This data shows that the highest rainfall frequency at Huntsville occurs most often in the months of November through April, with the most common directions of S-SW through SW and N-NE through NE. Winds speeds during precipitation average eight mph annually.

Figures 2.3-260, 2.3-261, 2.3-262, 2.3-263, 2.3-264, 2.3-265, 2.3-266, 2.3-267, 2.3-268, 2.3-269, 2.3-270, 2.3-271, and 2.3-272 show the precipitation wind rose for the BLN site for 2006-2007. Table 2.3-274 provides the monthly precipitation by direction at BLN for 2006 - 2007.

2.3.2.1.4 Atmospheric Stability

Atmospheric stability data for the BLN site were generated from the 2006-2007 site meteorological data. Wind direction by speed is presented for each resulting stability classes in Tables 2.3-309, 2.3-310, 2.3-311, 2.3-312, 2.3-313, 2.3-314, and 2.3-315. Hourly observation data for the BLN site from 1979-1982 and 2006-2007 were converted into annual stability class frequency distributions and summarized in Table 2.3-316. These annual stability class frequency distributions show that the BLN site data gathered over both time periods is relatively similar.

The frequency and strength of inversion layers are also investigated with five years of weather balloon data collected at the Nashville radiosonde station (Reference 223). Weather balloons are released twice daily at 0:00 a.m. and 12:00 p.m. to collect temperatures at increasing elevations. The monthly data are provided in Tables 2.3-276, 2.3-277, 2.3-278, 2.3-279, 2.3-280, 2.3-281, 2.3-282, 2.3-283, 2.3-284, 2.3-285, 2.3-286, and 2.3-287 in terms of percentages of mornings and afternoons containing inversions, average inversion layer elevation, and the average strength of the inversions. Table 2.3-288 provides annual average data for the period. An inversion is defined as three or more consecutive elevation readings showing temperatures increasing with elevation. The inversion layer height is the point at which temperature starts to decrease with elevation. The maximum inversion strength is the maximum temperature rise divided by elevation difference within the inversion layer.

The weather balloon data does not address how long inversion layers may persist. For this purpose, the BLNP FSAR data, based on the periods 1979 through 1982 and 2006 through 2007, is used. Tables 2.3-289, 2.3-290, 2.3-291, 2.3-292, 2.3-293, 2.3-294, 2.3-295, 2.3-296, 2.3-297, 2.3-298, 2.3-299, and 2.3-300 show similar inversion data for the BLN Site. These inversion occurrences were determined from E, F or G stability classifications resulting from onsite temperature measurements. These tables show the number of discrete periods when inversion conditions exist for one or more consecutive hours. Short periods contained within a longer period are not considered as discrete occurrences. These tables show the data for each of the years in order to show the variations from year to year. They also show the monthly mean distribution calculated from the yearly data. The monthly means are summarized in Table 2.3-301 and the monthly percentage of hours with inversions are given in Table 2.3-302.

2.3.2.1.5 Mixing Heights

Monthly mixing heights for Nashville, Tennessee are shown in Table 2.3-303. These were obtained from the EPA SCRAM Mixing Height Data collection for the period of 1984 through 1987 and 1990 through 1991 (Reference 204). The mixing heights in the mornings are lowest during the summer, and the mixing heights in the afternoon are lowest in the winter.

The ventilation rate is a measure of the dispersion of pollutants. Higher ventilation rates are better for dispersing pollution than lower ventilation rates. Mean ventilation rates by month for Nashville, Tennessee are given in Table 2.3-211. This data was obtained from National Climatic Data Center (Reference 204) for the years 1984 through 1987 and 1909 through 1991.

Morning ventilation is less than 4100 m^2/s throughout the year, and is less than 1500 m^2/s from June through October. Afternoon ventilation is higher than 7100 m^2/s from March through September, but lower than 5200 m^2/s from November through January. Based on this and the tendency of pollutants to collect during the course of a day, the highest daily air pollution potentials exist during the lower afternoon ventilation rates from November through January. Lowest air pollution potentials occur in the spring due to the relatively high mean ventilation rates.

2.3.2.2 Potential Influence of the Plant and Its Facilities on Local Meteorology

Operation of the new facility at the BLN site influences the local climatology. A discussion of the expected extent of this influence is presented in this section.

The only aspects of the BLN site that could be categorized as a unique microclimate relate to the Guntersville Lake/Tennessee River. The proximity of the river increases the local humidity by a small but measurable amount as seen when comparing the Huntsville relative humidity (Table 2.3-205) with the BLN relative humidity (Table 2.3-206). There is also a slight tendency for lower level winds to be channeled along the river valley.

New construction at the site is not expected to impact this climatic situation significantly. Figure 2.1-201 shows the intended construction areas. Although there is some ground leveling, there are no significant climate-shaping topographic features to be changed. The site is already a relatively flat area with more significant hills to the east and west that are not impacted by construction (refer to Figure 2.3-288 for a depiction of topography within 5 mi. of the site). There may be some tree removal, but the trees within the construction area are small in number compared to the surrounding forested land. There are no significant changes anticipated or proposed in terms of local hydrologic features. There are no significant changes to local roadways anticipated in support of the proposed new facility. The impacts of more structures, facilities, or activities in this relatively remote, rural area are not expected to be noticeable in terms of local meteorology.

Operation of power generation units can affect local climate in three ways, additional generation of particulates (increased fog or haze), temperature effects on local water sources, and cooling tower plume effects. Since the proposed unit is nuclear, any increase in particulate emissions during operation would be due to a modest increase in automobile traffic and the rare operation of diesel generators. Nuclear power is often described as the most environmentally benign source of energy primarily because of the lack of emitted pollutants; therefore, it can be concluded that the net increase in particulates is negligible and will not cause any noticeable climatic effects.

The impact on Tennessee River water temperature is discussed in Subsection 2.4.1. In brief, the proposed new facility would utilize cooling towers, so that the vast majority of rejected heat would go to the atmosphere. The amount of heat rejected to the flow of the Tennessee River would be relatively small, causing a concomitantly small impact on local meteorology.

The remainder of this subsection discusses the cooling tower plume effects. The center of the proposed cooling tower(s) location is approximately one mile west of the Tennessee River. From the wind rose of Figure 2.3-302, it can be seen that the prevailing winds are from the northeast. This means that the cooling tower plumes usually extend out over the BLN site itself. Therefore, it can be concluded that most of the local climatological effects such as increased moisture and shading is limited to the BLN site.

The major thrust of the following discussion is aimed at an evaluation of cooling tower plume effects. An assessment of the contribution of moisture to the ambient environment from cooling tower blowdown waste heat discharge is included. Finally, a qualitative evaluation of the effects of the cooling system on daily variations of several meteorological parameters is presented.

2.3.2.2.1 Cooling Tower Plumes

Cooling systems, which depend on evaporation of water for a major portion of the heat dissipation, may create visible vapor plumes. These vapor plumes cause shadowing of nearby lands, salt deposition, and can cause fogging or icing. An assessment of potential plumes from cooling towers at the BLN site and the cooling tower plume impacts was performed. This assessment was done using the SACTI plume modeling code (Reference 232). BLN site data from 1979-1982 and Nashville meteorology from 1984-1987 and 1990-1991 was used in the model.

The two existing natural draft cooling towers (NDCTs) providing normal heat sink cooling capability were analyzed. The heat load used is a bounding value and is the primary conservatism in the assessment; however, it is significant to note that the low air flow rate assumed provides additional conservatism by increasing the plume length to longer than what is actually expected. Each existing NDCT was analyzed simultaneously so that the two NDCT plumes produced included the assessment of plume interaction. Cooling tower dimensions, layout, and airflow rates were either defined by the existing NDCTS or reasonable estimates were made. Maximum drift rate for cooling towers of this type, and average Tennessee River water salt concentration were used to support deposition calculations.

Table 2.3-304 describes the expected plume lengths by season and direction for two NDCTs. Table 2.3-305 compares the plume lengths by frequency for two NDCTs. Additionally, the assessment shows that fogging and icing are not expected from the two NDCTs. The author of the SACTI plume modeling code cautions that the fogging predictions have not been field-tested like the plume lengths and deposition rates; however, the code predictions indicate that fogging is not a significant problem.

2.3.2.3 Topographical Description of the Surrounding Area

The terrain surrounding the BLN site is dominated by Sand Mountain across the Tennessee River to the east. This ridge runs in a northeast to south-west direction and is 1400 feet above mean sea level (MSL) through this area. To the north and west, the terrain is flatter and wooded. The only significant feature in this direction is Backbone Ridge, which are hills with an elevation less than 800 feet above mean sea level. Figures 2.3-286, 2.3-287, and 2.3-288 present topographic cross sections and a site area map. (Reference 210)

2.3.2.4 Local Meteorological Conditions for Design and Operating Bases

Site specific data was used for determination of atmospheric dispersion and diffusion estimates as discussed in Subsections 2.3.4 and 2.3.5 of this report. In general, however, given the size of the database from which to draw, regional rather than local meteorological and air quality conditions would be used for other design and operating bases of the BLN facility.

2.3.3 ONSITE METEOROLOGICAL MEASUREMENT PROGRAMS

Add the following text at the end of DCD Subsection 2.3.3.

BLN COL 2.3-3 The meteorological monitoring program is the same throughout the pre-construction, construction, and operational phases of the project. The monitoring program is a continuation of the ongoing meteorological monitoring program for the BLN facility.

The onsite meteorological measurement program has evolved over the years from the temporary meteorological towers installed in 1972 to the current system installed in 2006.

2.3.3.1 Onsite Meteorological Measurements Program - 1975-1983

The original tower at the permanent monitoring site was installed approximately 1525 meters (5000 feet) northeast of the original Unit 1 Reactor Building at 615 feet above mean sea level (MSL). The tower was 113 meters above ground level (AGL) and supported instrumentation for wind speed and direction and temperature at 10 meters, 60 meters, and 110 meters. Meteorological monitoring began on October 29, 1975 and was terminated on November 1, 1983. The system was designed to meet or exceed the requirements of Regulatory Guide 1.23, Revision 0. The following data were collected:

Meteorological Variable(s)	Elevation meters - AGL	Start Date	End Date
Wind Speed & Direction	110	10-29-75	11-01-83
	60	11-01-78	11-01-83
	46	08-19-76	11-01-78
	10	10-29-75	11-01-83

Meteorological Variable(s)	Elevation meters - AGL	Start Date	End Date
Dry-bulb Temperature	110	10-29-75	11-01-83
	60	11-01-78	11-01-83
	46	10-29-75	11-01-78
	10	10-29-75	11-01-83
	1	10-29-75	12-19-78
Dewpoint Temperature	10	08-19-76	11-01-83
	1	10-29-75	12-19-78
Rainfall	0	10-29-75	11-01-83

Table 2.3-317 gives the specifications of the meteorological equipment originally installed at BLN.

Only the historical data for 1979-1982 is used because of a sensor change in late 1978. The change in sensor levels (from 46 m to 60 m) in late 1978 means the data before 1979 are not comparable with the 2006-2007 data and are not applicable for the expected release points. Also, after the change in sensor levels, only 1979-1982 provide data for complete calendar years.

A meteorological tower at the permanent monitoring site serves as a representative observation station (i.e., meteorological conditions at that location are considered to be representative of the site). The information recorded by the meteorological instruments was stored in digital form. Operational checks of the system were made twice weekly or more frequently as necessary to achieve the required 90 percent annual data recovery.

2.3.3.1.1 Data Collection

The onsite meteorological data were recorded in both analog and digital form. Hourly values of measured meteorological variables were recorded and displayed on teletype. Wind data from the three tower levels (10 meters, 60 meters, and 110 meters), along with the 10-meter dewpoint data, were continuously recorded and displayed on analog strip chart recorders. Hourly values of measured meteorological variables were recorded on punched paper tape. Periodically, these data were removed and sent offsite for data validation, conversion to full digital format, and transfer to electronic form for permanent storage. Teletype displays, analog strip charts, and punched paper tapes were retained for five years after data were collected.

2.3.3.1.2 Meteorological Instrumentation Inspection and Maintenance

Instrument servicing, maintenance, and calibration were performed in accordance with established procedures. Routine inspections were made to verify proper operation of equipment and that no damage to the tower, environmental data station, or any other structure or equipment had occurred. The recording medium was also checked for proper operation.

Semi-annual checks for proper instrumentation readings were made at various points. Each component of the meteorological facility was checked and/or field calibrated and/or removed and replaced with a laboratory calibrated component at least semi-annually.

2.3.3.2 Onsite Meteorological Measurements Program - 2006-2007

A new meteorological tower began operation at the permanent monitoring site on April 1, 2006. The permanent meteorological facility consists of a 55 meter instrumented tower for wind and temperature measurements, a separate 10 meter tower for dewpoint measurements, a ground based instrument for rainfall measurements, and a data collection system in an instrument building (Environmental Data Station or EDS). The EDS is located west of the tower base and has been evaluated as having no adverse influence on the measurements taken at the tower. The data collected include: wind speeds, wind directions, and temperatures at the 10 meter and 55 meter levels; and dewpoint temperatures at the 10 meter level. Data collection began on April 1, 2006.

Rainfall is monitored from a rain gauge located approximately 45 feet from the tower. The meteorological sensors are connected to the data collection and recording equipment in the EDS. A system of lightning and surge protection circuitry with proper grounding is included in the facility design.

The instrumentation and measurements associated with the updated meteorological facility meet ANSI/ANS-3.11 (Reference 214) requirements and guidance provided in Regulatory Guide 1.23, Revision 1. The new meteorological facility location relative to other plant structures is shown on Figure 2.1-201. The local topography for the BLN site is shown on Figure 2.3-288. These figures illustrate that the location of the meteorological tower is sufficiently removed from any plant structures or significant topographic features. This system provides adequate data to represent onsite meteorological conditions and to describe the local and regional atmospheric transport and diffusion characteristics.

2.3.3.2.1 Instrument Description

A description of the meteorological sensors is provided in Table 2.3-317.

The main tower serves as a representative observation station (i.e., meteorological conditions at it's location are representative of the site). There are no terrain features or structures that would prevent the conditions at the main tower from being representative.

2.3.3.2.2 Meteorological Data Processing

The data processing procedure for BLN meteorological data involves three basic steps.

a. Data acquisition (Subsection 2.3.3.2.2.1).

b. Data processing (Subsection 2.3.3.2.2.2).

c. Data analysis (Subsection 2.3.3.2.2.3).

The data acquisition system is located at the EDS and consists of meteorological sensors, a personal computer (with peripherals), and various interface devices. These devices send meteorological data to an offsite computer to enable callup for data validation and archiving offsite. The onsite meteorological data are recorded in digital form.

The current meteorological data collection system is designed and replacement components are chosen to meet or exceed specifications for accuracy identified in ANSI/ANS-3.11-2005. The meteorological data collection system satisfies the ANSI/ANS-3.11-2005 accuracy requirements.

2.3.3.2.2.1 Data Acquisition

Data acquisition for the current system is under control of the EDS computer program. The output of each meteorological sensor is scanned periodically, scaled, and the data values are stored.

Meteorological sensor outputs are sampled at the following rates: horizontal wind direction and wind speed, every five seconds (720 per hour); temperature and dewpoint, every minute (60 per hour); rainfall, every 15 minutes (4 per hour). Each piece of data is checked to verify that it is between the minimum and maximum instrument limits. Data outside of specified limits is considered invalid and treated as missing.

Wind speeds are recorded in miles per hour. Wind directions are recorded on a 0-360° scale. Temperatures are recorded in degrees Fahrenheit. Precipitation is recorded in inches.

2.3.3.2.2.2 Data Processing

Software data processing routines within the EDS computer accumulate output and perform data calculations to generate the following data:

15-minutes	Hourly
average wind speed	average wind speed
vector wind speed	vector wind speed
vector wind direction	vector wind direction
horizontal wind direction sigma	horizontal wind direction sigma (15-min)
dry-bulb temperature	horizontal wind direction sigma (hourly)
15-minute precipitation	dry-bulb temperature
	dewpoint temperature
	hourly precipitation

An average is calculated every fifteen minutes and each hour from the individual readings. If there are insufficient individual samples to calculate an average (generally 25 percent for most variables, 50 percent for temperatures, and 75 percent for wind direction sigmas), an average is not calculated and the value for the hour (or 15-minutes) is classified as missing.

2.3.3.2.2.3 Data Analysis

The EDS computer sends the data to an offsite computer for validation, reporting, and archiving. These data are stored for remote access.

Meteorological data are generally reviewed every workday to identify possible data problems and notify appropriate personnel. Meteorological data are validated before they are placed into permanent archival storage to verify that the amount of valid data in the master record meets regulatory requirements for minimum data collection. Validation includes running data validation software as an aid to reviewing raw data, identifying and editing questionable or invalid data, recovering data from backup sources, and adjusting data to reflect calibration results. After validation is completed, data are permanently stored in electronic form.

Meteorological data are provided to specific users either routinely or on request. Data summaries are provided for both routine and non-routine applications.

2.3.3.2.3 Meteorological Instrumentation Inspection and Maintenance

The meteorological equipment at the EDS is kept in proper operating condition by staff that are trained and qualified for the necessary tasks.

Most equipment is calibrated or replaced at least every six months of service. The methods for maintaining a calibrated status for the components of the meteorological data collection system (sensors, recorders, electronics, data logger, etc.) include field checks, field calibration, and/or replacement by a laboratory calibrated component. More frequent calibration and/or replacement intervals for individual components may be conducted, on the basis of the operational history of the component type. Procedures and processes such as appropriate maintenance processes (procedures, work order/work request documents, etc.) are used to calibrate and maintain meteorological and station equipment. Records documenting results of calibrations, major causes of instrument outages or drift from calibration, and corrective action taken are maintained.

The operational phase of the meteorological program includes those procedures and responsibilities related to activities beginning with the initial fuel loading and continuing through the life of the plant. The meteorological data collection program is continuous without major interruptions. The meteorological program has been developed to be consistent with the guidance given in ANSI/ANS-3.11-2005 and the reporting procedures in Regulatory Guide 1.21, Revision 1. The basic objective is to maintain data collection performance to provide at least 90 percent annual joint recoverability and availability of data needed for assessing the relative concentrations and doses resulting from accidental or routine releases.

The restoration of the data collection capability of the meteorological facility in the event of equipment failure or malfunction is accomplished by replacement or repair of affected equipment. A stock of spare parts and equipment is maintained to minimize and shorten the periods of outages. Equipment malfunctions or outages are detected by personnel during routine or special checks. When an outage of one or more of the critical data items occurs, the appropriate maintenance personnel are notified.

2.3.3.2.4 Meteorological Data Comparison

The current meteorological data is good agreement with the historic site data from 1979-1982. Figure 2.3-244 compares the windspeed frequency from the current data with the historic data. As seen, there is a slight shift toward lower wind speeds for the current data although the overall windspeed distribution is similar. Figure 2.3-245 compares the frequency (percentage of occurrence) for the stability classes. This figure shows that there is a trend toward more stable conditions reflected in the current data even though the overall stability class distribution is similar to the historic data.

2.3.4 SHORT-TERM DIFFUSION ESTIMATES

Add the following text at the end of DCD Subsection 2.3.4.

BLN COL 2.3-4 The consequence of a design basis accident in terms of personnel exposure is a function of the atmospheric dispersion conditions at the site of the potential release. Atmospheric dispersion consists of two components atmospheric transport due to organized or mean airflow within the atmosphere and atmospheric diffusion due to disorganized or random air motions. Atmospheric diffusion conditions are represented by relative air concentration (χ/Q) values. This section describes the development of the short-term diffusion estimates for the site boundary and the control room.

2.3.4.1 Calculation Methodology

The efficiency of diffusion is primarily dependent on winds (speed and direction) and atmospheric stability characteristics. Dispersion is rapid within Stability Classes A through D and much slower for Classes E through G. That is, atmospheric dispersion capabilities decrease with progression from Class A to G, with an abrupt reduction from Classes D to E.

Relative concentrations of released gases, χ/Q values, as a function of direction for various time periods at the exclusion area boundary (EAB) and the outer boundary of the low population (LPZ), were determined by the use of the computer code PAVAN, NUREG/CR-2858. This code implements the guidance provided in Regulatory Guide 1.145. The χ/Q calculations are based on the theory that material released to the atmosphere are normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the point of release and the distances for which χ/Q values are calculated in accordance with NUREG/CR-2858 and Regulatory Guide 1.145.

Using joint frequency distributions of wind direction and wind speed by atmospheric stability, PAVAN provides the χ/Q values as functions of direction for various time periods at the exclusion area boundary (EAB) and the low population zone (LPZ). The meteorological data needed for this calculation included wind speed, wind direction, and atmospheric stability. The meteorological data used for this analysis was collected from the onsite monitoring equipment from April 1, 2006 through March 31, 2007. This data was averaged and are reported in Tables 2.3-309, 2.3-310, 2.3-311, 2.3-312, 2.3-313, 2.3-314, and 2.3-315. Other plant specific data included tower height at which wind speed was measured (10.0 m) and distances to the EAB and LPZ. The EAB for BLN is shown in FSAR Figure 2.1-205. The minimum exclusion area boundary (EAB) distances are reported in Table 2.3-318. The low population zone (LPZ) boundary is defined by a circular area with a radius of two miles from the plant center.

Regulatory Guide 1.145 divides release configurations into two modes, ground release and stack release. A ground release includes release points that are effectively lower than two and one-half times the height of the adjacent solid structures. Since the release points do not meet this criterion, releases are considered to be ground level releases.

The χ/Q value for the EAB or LPZ boundary evaluations is the maximum sector χ/Q or the five percent overall site χ/Q, whichever is greater in accordance with Regulatory Guide 1.145. The direction-dependent sector values are also calculated.

2.3.4.2 Calculations and Results

PAVAN requires the meteorological data in the form of joint frequency distributions of wind direction and wind speed by atmospheric stability class. These analyses were completed using data from the BLN meteorological instrumentation during the 12-month period of April 1, 2006 through March 31, 2007.

The stability classes were based on the classification system given in Table 2 of U.S. Nuclear Regulatory Commission Regulatory Guide 1.23, as follows.

Classification of Atmospheric Stability
(From Regulatory Guide 1.23)

Stability Classification	Pasquill Categories	Temperature change with height ($^{o}C/100m$)
Extremely unstable	A	$\Delta T \leq -1.9$
Moderately unstable	B	$-1.9 < \Delta T \leq -1.7$
Slightly unstable	C	$-1.7 < \Delta T \leq -1.5$
Neutral	D	$-1.5 < \Delta T \leq -0.5$
Slightly stable	E	$-0.5 < \Delta T \leq 1.5$
Moderately stable	F	$1.5 < \Delta T \leq 4.0$
Extremely stable	G	$\Delta T > 4.0$

Joint frequency distribution tables were developed from the meteorological data with the assumption that if data required as input to the PAVAN program (i.e., lower level wind direction, lower wind speed, and temperature differential) was missing from the hourly data record, all data for that hour was discarded. Also, the data in the joint frequency distribution tables was rounded for input into the PAVAN code.

Building area is defined as the smallest vertical-plane cross-sectional area of the reactor building, in square meters. The area of the reactor building to be used in the determination of building-wake effects is conservatively estimated as the above grade, cross-sectional area of the shield building. This area was determined to be 2909 m^2. Building height is the height above plant grade of the containment structure used in the building-wake term for the annual-average calculations. The Passive Containment Cooling System (PCCS) tank roof is at Elevation 334 ft. The Design Grade Elevation for the AP1000 is 100 ft; therefore, the height above plant grade of the containment structure or building height is 234 ft.

The tower height is the height at which the wind speed was measured. Based on the lower measurement location, the tower height used was 10 meters.

As described in Regulatory Guide 1.145, a ground release includes all release points that are effectively lower than two and one-half times the height of adjacent solid structures. Therefore, as stated above, a ground release was assumed.

Table 2.3-319 provides the offsite atmospheric dispersion factors. A summary of results is provided below.

BLN 5% Maximum χ/Q VALUES (sec/m^3)
(Based on 2006-2007 Meteorological Data)

	0 - 2 Hrs	0 - 8 Hrs	8 - 24 Hrs	24 - 96 Hrs	96 - 720 Hrs
EAB (NNE 1244 m)	5.85E-04	N/A	N/A	N/A	N/A
LPZ (2 miles)		1.23E-04	8.26E-05	3.49E-05	1.01E-05

Table 2.3-319 gives the directional-dependent sector and the direction independent χ/Q values at the EAB and LPZ along with the five percent maximum χ/Q values. Comparison of the BLN site characteristic X/Q values with the AP1000 DCD values is given in Table 2.0-201.

2.3.4.3 Relative Concentration Estimates at the Control Room Emergency Intake

The atmospheric dispersion estimates for the BLN Control Room were calculated based on the guidance provided in Regulatory Guide 1.194. The control room χ/Qs were calculated for the release points to the control room emergency air intake using the ARCON96 computer code (NUREG/CR-6331) based on the hourly meteorological data. The distances and directions from the assumed release points to the Control Room HVAC Intake are shown on Table 2.3-320. In each case, the intervening structures between the release point and the control room intake were ignored for calculational simplicity, thereby underestimating the true distance to the control room intakes. Atmospheric stability was determined by the vertical temperature difference (ΔT) measured over the difference in measurement height and the stability classes given in Regulatory Guide 1.23. The releases were assumed to be point ground level releases. For each of the source-to-receptor combinations, the χ/Q value that is not exceeded more than 5.0 percent of the total hours in the meteorological data set (e.g., 95-percentile χ/Q) was determined. The χ/Q values for source-receptor pairs are shown in Table 2.3-321.

2.3.5 LONG-TERM DIFFUSION ESTIMATES

Add the following to the end of DCD Subsection 2.3.5.

BLN COL 2.3-5 For a routine release, the concentration of radioactive material in the surrounding region depends on the amount of effluent released, the height of the release, the momentum and buoyancy of the emitted plume, the wind speed, atmospheric stability, airflow patterns of the site, and various effluent removal mechanisms. Annual average relative concentration, χ/Q, and annual average relative deposition, D/Q, for gaseous effluent routine releases were, therefore, calculated.

2.3.5.1 Calculation Methodology and Assumptions

The XOQDOQ Computer Program, NUREG/CR-2919, which implements the assumptions outlined in Regulatory Guide 1.111, was used to generate the annual average relative concentration, χ/Q, and annual average relative deposition, D/Q. Values of χ/Q and D/Q were determined at points of maximum potential concentration outside the site boundary, at points of maximum individual exposure and at points within a radial grid of sixteen 22-1/2° sectors and extending to a distance of 50 miles. Radioactive decay and dry deposition were considered.

Meteorological data for the period from April 1, 2006 through March 31, 2007 was used in the analysis. Receptor locations were determined from the locations obtained from the Land Use Census. Hourly meteorological data was used in the development of joint frequency distributions, in hours, of wind direction and wind speed by atmospheric stability class. The wind speed categories used were consistent with the BLN short-term (accident) diffusion χ/Q calculation discussed above. In accordance with NUREG/CR-2858 and NUREG/CR-2919, the calm array is distributed into the first wind speed class.

Joint frequency distribution tables were developed from the hourly meteorological data with the assumption that if data required as input to the XOQDOQ program (i.e., lower level wind direction and wind speed, and temperature differential as opposed to upper level wind direction and wind speed) was missing from the hourly data record, all data for that hour would be discarded. This assumption maximizes the data being included in the calculation of the χ/Q and D/Q values since hourly data is not discarded if only upper data is missing.

The analysis assumed a combined vent located at the center of the facility. At ground level locations beyond several miles from the plant, the annual average concentration of effluents are essentially independent of release mode; however, for ground level concentrations within a few miles, the release mode is important. Gaseous effluents released from tall stacks generally produce peak ground-level air concentrations near or beyond the site boundary. Near ground level releases usually produce concentrations that decrease from the release point to locations downwind. Guidance for selection of the release mode is provided in Regulatory Guide 1.111. In general, in order for an elevated release to be assumed, either the release height must be at least twice the height of adjacent buildings or detailed information must be known about the wind speed at the height of the release. For this analysis, the routine releases were conservatively modeled as ground level releases.

The building cross-sectional area and building height are used in calculation of building wake effects. Regulatory Guide 1.111 identifies the tallest adjacent building, in many cases, the reactor building, as appropriate for use. The AP1000 plant arrangement is comprised of five principal building structures; the nuclear island, the turbine building, the annex building, the diesel generator building, and the radwaste building. The nuclear island consists of a freestanding steel containment building, a concrete shield building, and an auxiliary building. As the shield building is the tallest building in the AP1000 arrangement, the shield building cross-sectional area and building height is used in calculation of building wake effects. The use of the shield building area, as opposed to the area of the nuclear island, is a conservative assumption since use of a smaller area minimizes wake effects resulting in higher relative concentrations.

Consistent with Regulatory Guide 1.111 guidance regarding radiological impact evaluations, radioactive decay and deposition were considered. For conservative estimates of radioactive decay, an overall half-life of 2.26 days is acceptable for short-lived noble gases and a half-life of eight days for iodines released to the atmosphere. At sites where there is not a well-defined rainy season associated with a local grazing season, wet deposition does not have a significant impact. In addition, the dry deposition rate of noble gases is so slow that the depletion is negligible within 50 miles. Therefore, in this analysis only the effects of dry deposition of iodines were considered. The calculation results with and without consideration of dry deposition are identified in the output as "depleted" and "undepleted." Terrain recirculation was considered consistent with Regulatory Guide 1.111.

2.3.5.2 Results

Receptor locations for the BLN were also evaluated. χ/Q and/or D/Q at points of potential maximum concentration outside the site boundary, at points of maximum individual exposure, and at points within a radial grid of sixteen 22½ degree sectors (centered on true north, north-northeast, northeast, etc.) and extending to a distance of 50 mi. from the station were determined. Receptor locations included in the evaluation are given in Table 2.3-322. A set of data points were located within each sector at increments of 0.25 mi. to a distance of 1 mi. from the plant, at increments of 0.5 mi. from a distance of 1 mi. to 5 mi, at increments of 2.5 mi. from a distance of 5 mi. to 10 mi, and at increments of 5 mi. thereafter to a distance of 50 mi. Estimates of χ/Q (undecayed and undepleted; depleted for radioiodines) and D/Q radioiodines and particulates is provided at each of these grid points. The results of the analysis, based on one year of data collected on site, are presented in Tables 2.3-323, 2.3-324, 2.3-325, 2.3-326, 2.3-327, 2.3-328, 2.3-329, 2.3-330, and 2.3-231.

2.3.6 COMBINED LICENSE INFORMATION

2.3.6.1 Regional Climatology

BLN COL 2.3-1 This COL item is addressed in Subsection 2.3.1

2.3.6.2 Local Meteorology

BLN COL 2.3-2 This COL item is addressed in Subsection 2.3.2

2.3.6.3 Onsite Meteorological Measurements Program

BLN COL 2.3-3 This COL item is addressed in Subsection 2.3.3

2.3.6.4 Short-Term Diffusion Estimates

BLN COL 2.3-4 This COL item is addressed in Subsection 2.3.4.

2.3.6.5 Long-Term Diffusion Estimates

BLN COL 2.3-5 This COL item is addressed in Subsection 2.3.5.

Add the following information after DCD Subsection 2.3.6.5.

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BLN COL 2.3-1

TABLE 2.3-201
MONTHLY CLIMATE SUMMARY – SCOTTSBORO, ALABAMA
1/1/1927 TO 9/30/2005

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	51.4	55.6	63.6	73.0	81.2	87.7	90.8	90.2	85.1	75.0	63.4	54.1	72.6
Average Min. Temperature (F)	30.0	32.5	38.7	46.7	55.2	63.2	67.0	65.6	59.4	46.9	37.3	31.5	47.8
Average Total Precipitation (in.)	5.61	5.50	6.50	4.76	4.38	4.28	4.87	3.47	4.11	3.16	4.51	5.63	56.8
Average Total Snowfall (in.)	0.8	0.5	0.3	0	0	0	0	0	0	0	0.1	0.1	1.8
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:

1. Percent of possible observations for period of record.
Max. Temp.: 92.8% Min. Temp.: 93% Precipitation: 93.6% Snowfall: 92.8% Snow Depth: 92%

(Reference 205)

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 1 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1926-27	0.00z	0.00z	0.00z	0.00z	0.00z	0.00z	0.2	0	0	0	0	0	0.2
1927-28	0	0	0	0	0	0	0.2	0	0	0	0	0	0.2
1928-29	0	0	0	0	0	0	0	0	0	0	0	0	0
1929-30	0	0	0	0.00z	0.00z	0.00z	0.00z	0.00z	0	0	0	0.00z	0
1930-31	0	0.00z	0.00z	0	0.00z	0.00z	0	0.00z	0.00z	0.00z	0	0	0
1931-32	0	0	0	0	0	0	0	0	0.2	0	0	0	0.2
1932-33	0	0	0	0	0	1.7	0	1.8	0	0	0	0	3.5
1933-34	0	0	0	0	0	0	0	1.3	4.3	0	0	0	5.6
1934-35	0	0	0	0	0	0	1	0.3	0	0	0	0	1.3
1935-36	0	0	0	0	m	0.00z	0.00z	0.00z	0.00z	0	0	0	0
1936-37	0	0	0	0	0	0	0	1	0	0	0	0	1
1937-38	0	0	0	0	0	0.8	0	0	0	0	0	0	0.8
1938-39	0	0	0	0	1.5	0	0	0	0	0	0	0	1.5
1939-40	0	0	0	0	0	0	14.7	0	0	0	0	0	14.7

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 2 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1940-41	0	0	0	0	0	0	0	2.2	0	0	0	0	2.2
1941-42	0	0	0	0	0	0	0	0	0.00z	0	0	0	0
1942-43	0	0	0	0	0	0	0.00z	0	0	0	0	0	0
1943-44	0	0	0	0	0	0	2.8	0	0	0	0	0	2.8
1944-45	0	0	0	0	0	1.8	0	0	0	0.00z	0	0	1.8
1945-46	0	0	0	0	0	0	2	0	0	0	0	0	2
1946-47	0	0	0	0	0	0	0	1	0	0	0	0	1
1947-48	0	0	0	0	0	0	5	0	0	0	0	0	5
1948-49	0	0	0	0	0	0	2.5	0	0	0	0	0	2.5
1949-50	0	0	0	0	0	0	0	0	0	0	0	0	0
1950-51	0	0	0	0	2.5	0	0	0	3	0	0	0	5.5
1951-52	0	0	0	0	0	0	0	0	0	0	0	0	0
1952-53	0	0	0	0	0	0	0	0	0	0	0	0	0
1953-54	0	0	0	0	0	0	0	0	0	0	0	0	0

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 3 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1954-55	0	0	0	0	0	0	0	0	0	0	0	0	0
1955-56	0	0	0	0	0	0	0	0	0	0	0	0	0
1956-57	0	0	0	0	0	0	0	0	0	0	0	0	0
1957-58	0	0	0	0	0	0	0	10	0	0	0	0	10
1958-59	0	0	0	0	0	0.00a	0	0	0	0	0	0	0
1959-60	0	0	0	0	0	0	0	9.3	0.00z	0	0	0	9.3
1960-61	0	0	0	0	0.00b	0	0	0	0	0	0	0	0
1961-62	0	0	0	0	0	0	5	0	0	0	0	0	5
1962-63	0	0	0	0	0	0	2	0.3	0	0	0	0	2.3
1963-64	0	0	0	0.00z	0	0.00z	0.00z	0	0	0	0	0	0
1964-65	0.00z	0	0	0	0	0	0.00z	0	0.00c	0	0	0	0
1965-66	0	0	0	0	0	0	8	0	0	0	0	0	8
1966-67	0	0	0	0	0.00a	0.00a	0	0.00z	0.00z	0.00z	0.00z	0.00z	0
1967-68	0.00z	0.00z	0.00z	0.00z	0	0	1.50c	1.5	0.00a	0	0	0	3

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 4 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1968-69	0	0	0	0	0	0	0	0	0	0	0	0	0
1969-70	0	0	0	0	0	0	0.00z	0	0	0	0	0	0
1970-71	0	0	0	0	0	0	0	0	0.00b	0	0	0	0
1971-72	0	0	0	0	0	0	0	0	0	0	0	0	0
1972-73	0	0	0	0	0	0	0	0	0	0	0	0	0
1973-74	0	0	0	0	0	0	0	0	0	0	0	0	0
1974-75	0	0	0	0	0	0	0	0	0	0	0	0	0
1975-76	0	0	0	0	0	0	0.00a	0	0	0	0	0	0
1976-77	0	0	0	0	0	0	0	0	0	0	0	0	0
1977-78	0	0	0	0	0	0	0.00z	0	0	0	0	0	0
1978-79	0.00z	0.00z	0	0.00z	0	0	0	0	0	0	0	0	0
1979-80	0	0.00z	0	0	0	0	0	0	0.00a	0	0	0	0
1980-81	0	0	0	0	0	0	0	0	0	0	0	0	0
1981-82	0	0	0	0	0	0	0.00z	0	0	0	0	0	0

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 5 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1982-83	0	0	0	0	0.00z	0	0	0	0	0	0.00z	0	0
1983-84	0	0	0	0	0	0	0	0	0.00z	0	0.00z	0.00z	0
1984-85	0.00z	0.00z	0.00z	0.00z	0.00z	0	1.5	2.5	0	0	0.00z	0.00z	4
1985-86	0.00z	0	0	0	0	0.1	0	0	0.00z	0	0.00z	0.00z	0.1
1986-87	0.00z	0.00z	0	0	0	0	0	0	0	0	0	0	0
1987-88	0	0	0.00z	0	0	0	9	0	0	0	0	0	9
1988-89	0	0	0	0	0	0.00z	0	0.2	0	0	0	0	0.2
1989-90	0	0	0	0	0	0	0	0	0	0	0	0	0
1990-91	0	0	0	0	0	0	0	0	0	0	0	0	0
1991-92	0	0	0	0	0	0	0	0	0	0	0	0	0
1992-93	0	0	0	0	0	0	0	0	12	0	0	0	12
1993-94	0	0	0	0	0	0	0	0	0	0	0	0	0
1994-95	0	0	0	0	0	0	0	2	0	0	0	0	2
1995-96	0	0	0	0	0	0	1.3	1.1	0.1	0	0	0	2.5

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 6 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
1996-97	0	0	0	0	0	0	1	1	0	0	0	0	2
1997-98	0	0	0	0	0	2	0	0	0.00a	0	0	0	2
1998-99	0	0	0	0	0	0	0.00a	0	0	0	0	0	0
1999-00	0	0	0	0	0	0	0	0	0	0	0	0	0
2000-01	0	0.00a	0	0	0.00a	0	0	0	0	0	0	0	0
2001-02	0.00b	0	0.00a	0	0	0	0	0	0	0	0	0	0
2002-03	0	0	0	0	0	0	0	0	0	0	0	0	0
2003-04	0	0.00a	0	0	0	0	0	0	0	0	0	0	0
2004-05	0	0	0	0	0	0	0	0	0	0	0	0	0
2005-06	0	0	0	0	0	0	0	0	0	0	0	0	0
MEAN	0	0	0	0	0.05	0.09	0.8	0.47	0.26	0	0	0	1.71
S.D.	0	0	0	0	0.34	0.37	2.36	1.62	1.51	0	0	0	3.13
SKEW	0	0	0	0	6.37	4.36	4.04	4.97	6.82	0	0	0	2.48

BLN COL 2.3-1

TABLE 2.3-202 (Sheet 7 of 7)
MONTHLY TOTAL SNOWFALL SCOTTSBORO, ALABAMA

YEAR(S)	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	Annual
MAX	0	0	0	0	2.5	2	14.7	10	12	0	0	0	14.7
MIN	0	0	0	0	0	0	0	0	0	0	0	0	0
NO YRS	73	73	75	73	74	74	72	76	74	77	75	75	56

*** Note *** Provisional Data *** After Year/Month 2004/12

a = 1 day missing, b = 2 days missing, c = 3 days, etc.,

z = 26 or more days missing, A = Accumulations present

NOTES:

1. Snowfall values are provided in inches of snowfall.

2. Long-term means based on columns; thus, the monthly row may not sum (or average) to the long-term annual value.

3. Maximum allowable number of missing days: 5

4. Individual Months not used for annual or monthly statistics if more than 5 days are missing. Individual Years not used for annual statistics if any month in that year has more than 5 days missing.

(Reference 205)

BLN COL 2.3-1

TABLE 2.3-203 (Sheet 1 of 2)
MONTHLY CLIMATE SUMMARY – HUNTSVILLE, ALABAMA
1/1/1959 TO 9/30/2005

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	49.2	54.2	62.9	72.9	80.1	86.6	89.4	89.1	83.5	73.6	62.2	52.4	71.3
Average Min. Temperature (F)	30.2	33.7	40.8	49.4	57.9	65.7	69.4	68.1	62.0	49.9	40.5	33.2	50.1
Average Total Precipitation (in.)	5.05	4.89	6.38	4.66	5.06	4.34	4.60	3.37	4.04	3.25	4.68	5.64	55.95
Average Total Snowfall (in.)	1.5	0.9	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.8	3.8
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:

1. Percent of possible observations for period of record.
   Max. Temp.: 100% Min. Temp.: 100% Precipitation: 100% Snowfall: 89.1% Snow Depth: 88.9%

(Reference 209)

BLN COL 2.3-1

TABLE 2.3-203 (Sheet 2 of 2)
MONTHLY CLIMATE SUMMARY – HUNTSVILLE, ALABAMA
1/1/1959 TO 9/30/2005

BLN Site Characteristics[a]

	Frequency of Occurrence		
	0.4 %	1 %	2 %
Cooling dry-bulb temperature, °F	94	92	90
Coincident wet-bulb temperature, °F	75	74	74
Evaporation wet-bulb, °F	78	77	76
Coincident dry-bulb, °F	89	88	86

	DB Temperature °F	
	Maximum	Minimum
1 percent exceedance	92	20
0.4 percent exceedance	94	15
100-year return	106	-17

a) Data from ASHRAE Fundamentals Handbook 2001, for Huntsville, Alabama.

BLN COL 2.3-2

TABLE 2.3-204
RESULTANT WIND DIRECTION AND SPEED – BLN SITE

Year	Most Common Wind Angle at 10 m (Degrees Clockwise from North)	Average Wind Speed at 10 m (mph)
1979-1982[a]	45 (NE)	4.9
2006-2007[b]	45 (NE)	4.1

a) Data from original BLN Site meteorological tower 1979-1982.

b) Data from permanent BLN Site meteorological tower 2006-2007.

BLN COL 2.3-1

TABLE 2.3-205
RELATIVE HUMIDITY FOR 4 TIME PERIODS PER DAY
HUNTSVILLE, ALABAMA 2001 – 2005

	00:00-06:00	06:00-12:00	12:00-18:00	18:00-24:00
Jan	81%	74%	60%	74%
Feb	82%	74%	60%	74%
Mar	80%	67%	52%	70%
Apr	82%	64%	48%	69%
May	88%	67%	53%	75%
Jun	92%	72%	59%	82%
Jul	94%	77%	65%	88%
Aug	93%	73%	60%	86%
Sep	91%	69%	53%	81%
Oct	89%	71%	55%	82%
Nov	82%	69%	55%	75%
Dec	82%	74%	60%	76%
Annual	86%	71%	57%	78%

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-206
RELATIVE HUMIDITY FOR 4 TIME PERIODS PER DAY BLN SITE 1979 – 1982

	00:00-06:00	06:00-12:00	12:00-18:00	18:00-24:00
Jan	76%	70%	59%	69%
Feb	76%	71%	54%	69%
Mar	76%	67%	48%	64%
Apr	81%	67%	48%	66%
May	85%	76%	55%	72%
Jun	87%	78%	55%	72%
Jul	89%	81%	61%	76%
Aug	91%	82%	58%	78%
Sep	90%	83%	59%	79%
Oct	86%	77%	52%	73%
Nov	82%	74%	55%	72%
Dec	77%	72%	59%	69%
Annual	83%	75%	55%	72%

NOTES:

1. Bellefonte (BLN) Site data is from meteorological tower measurements in 1979-1982.

2. Hourly readings are averaged over the six hour period over all the days in the given months for these four years.

BLN COL 2.3-1

TABLE 2.3-207
FREQUENCY OF TROPICAL CYCLONES (BY MONTH) FOR THE STATES OF TEXAS, LOUISIANA, MISSISSIPPI, ALABAMA, AND FLORIDA – 1899 – 2002

	Category of Storm (Saffir-Simpson Scale)							
	1 (No.)	2 (No.)	3 (No.)	4 (No.)	5 (No.)	Monthly Total (No.)	Annual Frequency (yr-1)	% of Total
Jun	7	2	1	1		11	0.11	9%
Jul	3	4	3			10	0.10	8%
Aug	8	7	9	2	2	28	0.27	23%
Sep	12	8	15	9	1	45	0.44	37%
Oct	10	8	7			25	0.24	20%
Nov	2	2				4	0.04	3%
Total	42	31	35	12	3	123	1.19	100%

Where the definition of Storm Category is as follows (Saffir-Simpson Scale):

Storm Category	Wind Speed (mph)	Storm Surge (ft. Above Normal)
1	74 to 95	4 to 5
2	96 to 110	6 to 8
3	111 to 130	9 to 12
4	131 to 155	13 to 18
5	Greater than 155	Greater than 18

(Reference 213)

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 1 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
Jackson County, AL						
1 JACKSON	4/6/1958	0003	F3	10	100	0.568
2 JACKSON	5/26/1960	1300	F1	0	33	
3 JACKSON	4/15/1965	1715	F3	3	50	0.085
4 JACKSON	5/19/1973	1615	F2	15	900	7.670
5 JACKSON	5/27/1973	1415	F2	4	500	1.136
6 JACKSON	4/3/1974	2215	F3	8	700	3.182
7 JACKSON	4/4/1977	1220	F2	7	100	0.398
8 JACKSON	7/22/1982	1400	F0	0	17	
9 JACKSON	3/24/1984	1938	F3	4	60	0.136
10 JACKSON	8/16/1985	1330	F0	0	20	
11 JACKSON	5/9/1988	1825	F2	14	50	0.398
12 JACKSON	11/15/1989	1755	F1	1	20	0.011

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 2 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
13 Pisgah	3/16/1996	1:15 PM	F1	2	80	0.091
14 Stevenson	1/5/1997	12:30 AM	F0	3	50	0.085
15 Aspel	5/24/2001	4:48 PM	F1	1	80	0.045
16 Flat Rock	3/19/2003	1:50 PM	F1	10	50	0.284
17 Section	3/19/2003	12:49 PM	F1	1	30	0.017
18 Dutton	3/19/2003	12:52 PM	F1	1	40	0.023
19 Hollywood	5/6/2003	8:45 AM	F0	3	20	0.034
20 Hollywood	5/6/2003	8:58 AM	F0	1	20	0.011
21 Skyline	8/20/2004	2:23 PM	F0	1	30	0.017
Dakalb County, AL						
1 DEKALB	2/29/1952	1700	F3	3	400	0.682
2 DEKALB	11/18/1957	1615	F1	0	0	
3 DEKALB	1/24/1964	2100	F2	0	0	

TABLE 2.3-208 (Sheet 3 of 13)
BLN COL 2.3-1
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
4 DEKALB	4/7/1964	910	F2	3	33	0.056
5 DEKALB	4/7/1964	1000	F1	0	0	
6 DEKALB	4/15/1965	1715	F3	7	50	0.199
7 DEKALB	5/8/1973	0410	F2	20	900	10.227
8 DEKALB	5/19/1973	1615	F2	4	900	2.045
9 DEKALB	5/19/1973	1845	F4	5	400	1.136
10 DEKALB	12/29/1973	1715	F2	0	100	
11 DEKALB	3/30/1977	0815	F3	9	50	0.256
12 DEKALB	3/30/1977	0835	F2	3	50	0.085
13 DEKALB	5/19/1983	1615	F3	1	473	0.269
14 DEKALB	5/9/1988	1833	F2	1	50	0.028
15 DEKALB	11/22/1992	0800	F1	6	50	0.170
16 DEKALB	11/22/1992	0815	F2	7	73	0.290

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 4 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
17 DEKALB	11/22/1992	0820	F0	3	23	0.039
18 DEKALB	11/22/1992	0820	F2	7	73	0.290
19 DEKALB	11/22/1992	0840	F2	7	73	0.290
20 Grove Oak To Rainsville	3/27/1994	1132	F4	23	700	9.148
21 Rainsville	4/22/1997	2:53 PM	F2	5	220	0.625
22 Geraldine	4/8/1998	7:23 PM	F1	2	100	0.114
23 Rainsville	4/27/1999	1:05 PM	F0	0	25	
24 Fyffe	4/27/1999	12:40 PM	F0	1	25	0.014
25 Fyffe	11/24/2001	2:25 PM	F2	7	100	0.398
26 Hammondville	5/6/2003	9:13 AM	F1	3	50	0.085
27 Ft Payne	4/22/2005	5:59 PM	F0	0	60	
Marshall County, AL						
1 MARSHALL	4/8/1957	1015	F3	5	200	0.568

TABLE 2.3-208 (Sheet 5 of 13)

BLN COL 2.3-1

TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
2 MARSHALL	11/18/1957	1730	F4	10	100	0.568
3 MARSHALL	4/6/1958	0003	F3	16	100	0.909
4 MARSHALL	3/7/1961	2340	F3	9	200	1.023
5 MARSHALL	3/25/1962	1715	F1	1	100	0.057
6 MARSHALL	4/7/1964	1000	F1	0		
7 MARSHALL	4/4/1968	1300	F2	4	33	0.075
8 MARSHALL	6/27/1972	0845	F2	0	40	
9 MARSHALL	1/26/1973	1545	F2	0		
10 MARSHALL	5/8/1973	0410	F2	9	900	4.602
11 MARSHALL	5/27/1973	1330	F2	32	500	9.091
12 MARSHALL	5/2/1974	1330	F2	2	400	0.455
13 MARSHALL	10/15/1974	1605	F1	11	33	0.206
14 MARSHALL	5/8/1975	2148	F1	0		

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 6 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
15 MARSHALL	5/6/1976	1750	F1	2	33	0.038
16 MARSHALL	7/31/1976	1200	F1	0	50	
17 MARSHALL	5/12/1978	2335	F1	8	200	0.909
18 MARSHALL	5/18/1981	1810	F1	0	17	
19 MARSHALL	1/3/1982	2245	F2	3	100	0.170
20 MARSHALL	2/22/1983	1528	F2	2	440	0.500
21 MARSHALL	5/19/1983	1435	F1	2	80	0.091
22 MARSHALL	7/5/1984	0130	F1	3	40	0.068
23 MARSHALL	4/5/1985	1645	F3	8	277	1.259
24 MARSHALL	3/12/1986	2022	F2	6	200	0.682
25 MARSHALL	2/23/1994	0340	F0	0	20	
26 Guntersville	3/27/1994	1102	F2	6	400	1.364
27 Martling	2/16/1995	0528	F2	12	700	4.773

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 7 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
28 Grant	9/28/1996	12:50 AM	F2	3	80	0.136
29 Union Grove	11/24/2001	1:41 PM	F2	2	300	0.341
30 Red Hill	3/29/2002	11:20 PM	F1	9	500	2.557
Madison County, AL						
1 MADISON	6/8/1951	0900	F2	0	0	
2 MADISON	4/5/1958	2230	F1	0	0	
3 MADISON	6/6/1961	1500	F1	0	0	
4 MADISON	3/11/1963	1740	F2	25	33	0.469
5 MADISON	11/24/1967	1305	F2	7	83	0.330
6 MADISON	12/18/1967	0325	F2	20	300	3.409
7 MADISON	12/21/1967	1930	F1	13	33	0.244
8 MADISON	4/24/1970	0630	F2	1	33	0.019
9 MADISON	4/26/1970	0800	F1	9	50	0.256

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 8 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
10 MADISON	5/19/1973	1440	F2	2	500	0.568
11 MADISON	11/27/1973	1833	F3	14	200	1.591
12 MADISON	4/1/1974	2140	F3	8	800	3.636
13 MADISON	4/3/1974	1815	F5	5	500	1.420
14 MADISON	4/3/1974	1900	F5	23	33	0.431
15 MADISON	4/3/1974	2135	F3	30	700	11.932
16 MADISON	3/20/1976	2208	F1	5	100	0.284
17 MADISON	3/20/1976	2222	F0	1	20	0.011
18 MADISON	3/20/1976	2222	F2	1	20	0.011
19 MADISON	3/20/1976	2225	F1	5	40	0.114
20 MADISON	7/17/1977	1345	F2	0	77	
21 MADISON	4/17/1982	0425	F1	2	100	0.114
22 MADISON	4/14/1985	1920	F1	0	30	

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 9 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
23 MADISON	8/16/1985	1408	F2	13	100	0.739
24 MADISON	8/16/1985	1530	F1	9	30	0.153
25 MADISON	7/28/1986	2000	F0	4	150	0.341
26 MADISON	11/15/1989	1630	F4	13	880	6.500
27 MADISON	11/15/1989	1642	F4	6	880	3.000
28 MADISON	11/22/1992	0655	F2	6	100	0.341
29 MADISON	5/3/1993	1735	F0	0	20	
30 MADISON	6/26/1994	2211	F2	7	200	0.795
31 Meridianville	5/3/1997	04:26 P M	F2	1	70	0.040
32 Owens Xrds	5/3/1997	04:34 P M	F0	1	40	0.023
33 Owens Xrds	5/3/1997	04:40 P M	F0	2	50	0.057
34 Huntsville	5/25/1997	06:23 P M	F0	0	30	
35 Toney	5/7/1998	05:03 A M	F1	2	50	0.057

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 10 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
36 New Market	5/7/1998	05:27 A M	F1	2	75	0.085
37 Huntsville	2/16/2001	01:39 P M	F0	4	30	0.068
38 New Hope	11/24/2001	01:50 P M	F2	3	300	0.511
39 Meridianville	9/18/2002	01:40 P M	F0	0	20	
40 Meridianville	10/12/2002	12:30 P M	F0	0	20	
41 Toney	3/19/2003	09:20 A M	F0	0	50	
42 Madison	5/6/2003	06:58 A M	F0	0	20	
43 Meridianville	5/6/2003	07:16 A M	F1	1	200	0.114
44 New Sharon	5/30/2004	11:55 P M	F1	9	150	0.767
45 Owens Xrds	7/6/2004	05:28 P M	F0	0	2	
46 Huntsville	7/14/2004	03:20 P M	F0	1	50	0.028
Franklin County, TN						
1 FRANKLIN	2/13/1952	2240	F4	11	100	0.625

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 11 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
2 FRANKLIN	4/3/1974	1900	F4	14	800	6.364
3 FRANKLIN	4/3/1974	1945	F4	11	33	0.206
4 FRANKLIN	4/3/1974	2000	F3	4	100	0.227
5 FRANKLIN	2/9/1990	2213	F1	3	43	0.073
6 FRANKLIN	2/9/1990	2225	F1	2	50	0.057
7 Keith Springs Mountain	6/26/1994	1930	F1	8	200	0.909
8 Belvedere	4/20/1995	2255	F1	3	30	0.051
9 Huntland	11/7/1996	4:00 PM	F2	8	175	0.795
10 Decherd	11/7/1996	4:17 PM	F1	0	18	
11 Oak Grove	11/7/1996	4:22 PM	F1	0	18	
12 Alto	11/7/1996	4:24 PM	F1	0	18	
13 Huntland	5/2/1997	5:00 PM	F2	1	150	0.085
14 Sewanee	3/5/2004	11:35 PM	F0	0	100	

BLN COL 2.3-1

TABLE 2.3-208 (Sheet 12 of 13)
TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
15 Huntland	5/31/2004	12:14 AM	F1	5	150	0.426
16 Center Grove	5/31/2004	12:20 AM	F1	2	150	0.170
Marion County, TN						
1 MARION	3/11/1963	1900	F2	15	200	1.705
2 MARION	7/6/1980	1400	F1	1	200	0.114
3 MARION	6/3/1982	1315	F1	2	77	0.088
4 MARION	10/23/1984	1315	F0	0	27	
5 MARION	4/20/1986	1825	F1	0	27	
6 South Pittsburg	3/7/1995	1930	F0	0	10	
7 Haletown	4/21/1995	0018	F1	5	25	0.071
8 Whitwell	5/10/1995	1600	F0	1	20	0.011
Dade County, GA						
1 DADE	10/23/1984	1705	F1	1	37	0.021

TABLE 2.3-208 (Sheet 13 of 13)

BLN COL 2.3-1

TORNADOES IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN, TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Location or County	Date	Time	Magnitude - Fujita Scale	Length (mi)	Width (yards)	Area (mi^2)
2 DADE	11/22/1992	0850	F2	4	500	1.136
3 Head River	11/24/2001	4:06 PM	F1	2	528	0.600

(Reference 208)

BLN COL 2.3-1

TABLE 2.3-209 (Sheet 1 of 2)
THUNDERSTORMS AND HIGH WIND EVENTS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

	Jackson County	Dekalb County	Marshall County	Madison County	Franklin County, Tenn	Marion County, Tenn	Dade County, Ga	All Seven Areas	Average per Year
Month	(#)	(#)	(#)	(#)	(#)	(#)	(#)	(#)	(#/yr)
Jan	4	8	6	13	1	1	0	33	0.59
Feb	6	10	8	13	8	4	2	51	0.91
Mar	10	5	13	22	8	5	3	66	1.18
Apr	9	16	17	24	10	6	4	86	1.54
May	18	10	15	32	15	16	8	114	2.04
Jun	24	18	25	40	24	12	6	149	2.66
Jul	33	25	36	58	24	21	11	208	3.71
Aug	10	20	13	42	9	11	0	105	1.88
Sep	5	7	4	13	6	2	3	40	0.71
Oct	2	2	1	3	1	5	1	15	0.27
Nov	6	10	7	6	5	7	3	44	0.79
Dec	5	4	4	4	0	3	2	22	0.39

TABLE 2.3-209 (Sheet 2 of 2)

BLN COL 2.3-1

THUNDERSTORMS AND HIGH WIND EVENTS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

	Jackson County	Dekalb County	Marshall County	Madison County	Franklin County, Tenn	Marion County, Tenn	Dade County, Ga	All Seven Areas	Average per Year
Annual	132	135	149	270	111	93	43	933	16.66
	14.1%	14.5%	16.0%	28.9%	11.9%	10.0%	4.6%		
Length of Record	56 yrs								

NOTES:

1. Storms listed at different sites in the same county on the same day were counted as separate events.
2. Average/yr were based on the period 1950 through 2005 (last storm in database). Prior to 1981, the yearly storm averages were markedly less frequent, suggesting less thorough storm data collection.
3. The BLN is in Jackson County. The other counties listed are adjacent to Jackson County.

(Reference 208)

BLN COL 2.3-1

TABLE 2.3-210
HAIL STORM EVENTS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

	Number of Events	Percentage	Events with Property Damage
Jackson County, AL	66	13%	16
Dakalb County, AL	95	19%	24
Marshall County, AL	82	16%	18
Madison County, AL	151	30%	24
Franklin County, TN	41	8%	1
Marion County, TN	33	7%	2
Dade County, GA	36	7%	1
TOTAL =	504	100%	86

(Reference 208)

BLN COL 2.3-1

TABLE 2.3-211
MEAN VENTILATION RATE BY MONTH – NASHVILLE, TENNESSEE – 1984 – 1987 & 1990 – 1991

	Morning Ventilation Rate (m^2/s)	Afternoon Ventilation Rate (m^2/s)	Mean Ventilation Rate (m^2/s)
Jan	3076	4645	3860
Feb	4090	6643	5367
Mar	3605	9850	6728
Apr	2909	11472	7191
May	2355	8902	5629
Jun	1351	7164	4258
July	1264	7410	4337
Aug	1433	7292	4362
Sep	1492	7334	4413
Oct	1478	6873	4175
Nov	3041	5179	4110
Dec	3383	5017	4200

NOTES:

1. Atmospheric ventilation rate is numerically equal to the product of the mixing height and the wind speed within the mixing layer.

(Reference 204)

BLN COL 2.3-1

TABLE 2.3-212 (Sheet 1 of 9)
ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
Jackson County, AL						
3/12/1993	2200	Winter Storm	4	0	5.0B	0
2/6/1995	2100	Snow/ice	0	0	0	0
2/11/1995	1300	Snow/ice	0	0	0	0
1/6/1996	8:00 PM	Winter Storm	0	0	380K	38K
2/1/1996	3:00 PM	Winter Storm	0	0	595K	0
2/16/1996	2:00 AM	Winter Storm	0	0	195K	0
1/10/1997	10:00 AM	Winter Storm	0	0	64K	0
12/29/1997	1:00 AM	Winter Storm	0	0	0	0
2/4/1998	1:30 AM	Winter Storm	0	0	27K	0
12/23/1998	6:00 AM	Ice Storm	0	0	126K	0
1/6/1999	12:00 PM	Winter Storm	0	0	0	0
12/21/1999	4:00 AM	Ice Storm	0	0	0	0
1/22/2000	9:00 AM	Ice Storm	0	0	2.7M	0
1/28/2000	6:00 AM	Ice Storm	0	0	1.1M	0
3/20/2001	12:00 AM	Heavy Snow	0	0	0	0

TABLE 2.3-212 (Sheet 2 of 9)

BLN COL 2.3-1

ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
2/5/2002	11:30 PM	Winter Storm	0	0	30K	0
1/23/2005	7:15 AM	Winter Storm	0	0	0	0
3/1/2005	6:00 AM	Winter Weather/mix	0	0	0	0
Dakalb County, AL						
3/12/1993	2200	Winter Storm	4	0	5.0B	0
2/6/1995	2100	Snow/ice	0	0	0	0
2/11/1995	1300	Snow/ice	0	0	0	0
1/6/1996	8:00 PM	Winter Storm	0	0	380K	38K
2/1/1996	3:00 PM	Winter Storm	0	0	595K	0
2/16/1996	2:00 AM	Winter Storm	0	0	195K	0
1/10/1997	10:00 AM	Winter Storm	0	0	64K	0
12/29/1997	1:00 AM	Winter Storm	0	0	0	0
2/4/1998	1:30 AM	Winter Storm	0	0	27K	0
12/23/1998	6:00 AM	Ice Storm	0	0	126K	0
1/6/1999	12:00 PM	Winter Storm	0	0	0	0
12/21/1999	4:00 AM	Ice Storm	0	0	0	0

TABLE 2.3-212 (Sheet 3 of 9)

BLN COL 2.3-1

ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
1/22/2000	9:00 AM	Ice Storm	0	0	2.7M	0
1/28/2000	6:00 AM	Ice Storm	0	0	1.1M	0
3/20/2001	12:00 AM	Heavy Snow	0	0	0	0
2/5/2002	11:30 PM	Winter Storm	0	0	30K	0
2/26/2004	2:05 AM	Winter Storm	0	0	0	0
1/28/2005	9:02 PM	Ice Storm	0	0	0	0
Marshall County, AL						
3/12/1993	2200	Winter Storm	4	0	5.0B	0
2/6/1995	2100	Snow/ice	0	0	0	0
2/11/1995	1300	Snow/ice	0	0	0	0
1/6/1996	8:00 PM	Winter Storm	0	0	380K	38K
2/1/1996	3:00 PM	Winter Storm	0	0	595K	0
2/16/1996	2:00 AM	Winter Storm	0	0	195K	0
1/10/1997	10:00 AM	Winter Storm	0	0	64K	0
12/29/1997	1:00 AM	Winter Storm	0	0	0	0
2/4/1998	1:30 AM	Winter Storm	0	0	27K	0

BLN COL 2.3-1

TABLE 2.3-212 (Sheet 4 of 9)
ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
12/23/1998	2:00 AM	Ice Storm	1	0	14.4M	0
1/6/1999	12:00 PM	Winter Storm	0	0	0	0
1/28/2000	4:00 AM	Winter Storm	0	0	75K	0
3/20/2001	12:00 AM	Heavy Snow	0	0	0	0
2/26/2004	2:05 AM	Winter Storm	0	0	0	0
Madison County, AL						
3/12/1993	2200	Winter Storm	4	0	5.0B	0
2/9/1994	2200	Ice Storm/flash Flood	0	2	0	0
2/6/1995	12:00 AM	Snow/ice	0	0	0	0
2/11/1995	12:00 AM	Snow/ice	0	0	0	0
1/6/1996	8:00 PM	Winter Storm	0	0	380K	38K
2/1/1996	3:00 PM	Winter Storm	0	0	595K	0
2/16/1996	2:00 AM	Winter Storm	0	0	195K	0
1/10/1997	10:00 AM	Winter Storm	0	0	64K	0
12/29/1997	1:00 AM	Winter Storm	0	0	0	0
2/4/1998	1:30 AM	Winter Storm	0	0	27K	0

BLN COL 2.3-1

TABLE 2.3-212 (Sheet 5 of 9)
ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
12/23/1998	2:00 AM	Ice Storm	1	0	14.4M	0
1/6/1999	12:00 PM	Winter Storm	0	0	0	0
12/21/1999	4:00 AM	Ice Storm	0	0	0	0
1/28/2000	4:00 AM	Winter Storm	0	0	75K	0
3/20/2001	12:00 AM	Heavy Snow	0	0	0	0
2/5/2002	11:30 PM	Winter Storm	0	0	30K	0
1/28/2005	9:02 PM	Ice Storm	0	0	0	0
3/15/2005	4:30 AM	Winter Weather/mix	0	0	0	0
Franklin County, TN						
2/9/1994	2000	Ice Storm	0	0	500K	0
1/17/1995	0400	Heavy Snow	0	0	0	0
1/17/1995	1700	Ice	0	0	500K	0
1/6/1996	5:00 PM	Winter Storm	0	0	10K	0
2/1/1996	5:00 PM	Winter Storm	0	1	5K	0
2/16/1996	2:00 AM	Heavy Snow	0	0	0	0
2/3/1998	5:00 PM	Heavy Snow	0	0	5.0M	0

TABLE 2.3-212 (Sheet 6 of 9)

BLN COL 2.3-1

ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
12/23/1998	7:30 AM	Winter Storm	0	11	1.5M	0
1/6/2002	3:30 AM	Heavy Snow	0	0	0	0
2/26/2004	6:00 AM	Winter Storm	0	0	0	0
1/23/2005	7:00 AM	Winter Weather/mix	0	0	0	0
3/1/2005	6:00 AM	Winter Weather/mix	0	0	0	0
3/17/2005	12:00 AM	Winter Weather/mix	1	0	0	0
Marion County, TN						
12/20/1993	2200	Snow	0	0	1K	0
1/17/1995	0400	Heavy Snow	0	0	0	0
1/17/1995	1700	Ice	0	0	500K	0
1/6/1996	5:00 PM	Winter Storm	0	0	10K	0
2/1/1996	5:00 PM	Winter Storm	0	1	5K	0
2/3/1998	5:00 PM	Heavy Snow	0	0	5.0M	0
12/22/1998	1:00 AM	Ice Storm	0	0	0	0
1/6/1999	7:00 AM	Winter Storm	0	0	0	0
3/13/1999	4:00 AM	Winter Storm	0	0	0	0

TABLE 2.3-212 (Sheet 7 of 9)

BLN COL 2.3-1

ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
1/22/2000	10:00 AM	Winter Storm	0	0	0	0
12/2/2000	6:00 PM	Winter Storm	0	0	0	0
12/18/2000	6:00 PM	Winter Storm	0	0	0	0
1/1/2001	2:00 AM	Winter Storm	0	0	0	0
1/20/2001	3:00 AM	Winter Storm	0	0	0	0
3/20/2001	3:30 AM	Winter Storm	0	0	0	0
1/5/2002	10:00 PM	Winter Storm	0	0	0	0
1/16/2003	1:00 PM	Winter Storm	0	0	0	0
1/9/2004	12:00 AM	Winter Storm	0	0	0	0
2/15/2004	8:00 PM	Heavy Snow	0	0	0	0
2/26/2004	12:00 PM	Heavy Snow	0	0	0	0
1/29/2005	12:00 AM	Ice Storm	0	0	0	0
Dade County, GA						
2/25/1993	1800	Ice Storm	0	0	50K	0
3/12/1993	2000	Heavy Snow	0	0	5.0M	500K
3/13/1993	500	Blizzard	8	15	500K	50.0M

BLN COL 2.3-1

TABLE 2.3-212 (Sheet 8 of 9)
ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
1/6/1996	3:00 PM	Winter Storm	0	0	10K	0
1/11/1996	4:00 PM	Heavy Snow	0	0	0	0
2/2/1996	10:00 AM	Winter Storm	0	0	200K	0
3/20/1996	4:00 PM	Heavy Snow	0	0	0	0
2/4/1998	1:00 AM	Snow	0	0	0	0
12/23/1998	5:00 AM	Ice Storm	0	0	0	0
12/23/1998	8:00 PM	Ice Storm	0	0	10K	0
2/23/1999	11:00 AM	Snow	0	0	0	0
1/22/2000	1:00 PM	Ice Storm	0	1	48.0M	0
1/28/2000	7:00 PM	Ice Storm	0	0	2.0M	0
12/3/2000	5:00 AM	Heavy Snow	0	0	0	0
12/17/2000	7:30 AM	Winter Storm	0	0	0	0
12/29/2000	6:30 PM	Light Snow	0	0	0	0
1/1/2001	7:58 AM	Light Snow	0	0	0	0
1/9/2001	7:30 AM	Light Snow	0	0	0	0
1/6/2002	5:00 AM	Heavy Snow	0	0	0	0

TABLE 2.3-212 (Sheet 9 of 9)

BLN COL 2.3-1

ICE STORMS IN JACKSON, DEKALB, MARSHALL, MADISON ALABAMA, FRANKLIN TENNESSEE, MARION TENNESSEE, AND DADE GEORGIA – 1950 – 2005

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
1/16/2003	12:00 PM	Snow	0	0	0	0
1/23/2003	12:00 AM	Snow	0	0	0	0
2/6/2003	3:00 PM	Winter Weather/mix	0	0	0	0
2/26/2004	12:00 AM	Winter Storm	0	0	0	0
1/28/2005	8:00 PM	Winter Storm	0	0	9.8M	0

NOTES:

1. The BLN is in Jackson County. The other counties are adjacent to Jackson County.
2. The annual frequency based on the 18 Jackson County events is 18/13 = 1.4 events per year. This assumes that the storm database covers the years of 1993 – 2005 (no events earlier than 1993 were reported).

(Reference 208)

BLN COL 2.3-1

TABLE 2.3-213
TOTAL MAXIMUM WINTER PRECIPITATION – BLN SITE –
1979 – 1982 & 2006 – 2007

Season	Maximum 48 Hour Precipitation (Inches)
1979	3.39
1980	3.87
1981	2.45
1982	4.18
2006-2007	1.38

NOTES:

1. The BLN site data is from 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-1

TABLE 2.3-214 (Sheet 1 of 2)
HOURLY METEOROLOGICAL DATA – HUNTSVILLE, ALABAMA – WORST 1-DAY – 2001 – 2005

Hour	Dry Bulb Temperature (F)	Wet Bulb Temperature (F)
1	77	76
2	76	75
3	76	75
4	75	74
5	74	73
6	76	75
7	79	77
8	83	78
9	86	79
10	90	81
11	90	80
12	91	80
13	92	80
14	94	78
15	94	78
16	94	78
17	92	80
18	90	80
19	87	80
20	83	80
21	81	79
22	80	78

BLN COL 2.3-1

TABLE 2.3-214 (Sheet 2 of 2)
HOURLY METEOROLOGICAL DATA – HUNTSVILLE, ALABAMA – WORST 1-DAY – 2001 – 2005

Hour	Dry Bulb Temperature (F)	Wet Bulb Temperature (F)
23	79	78
24	79	78
AVERAGE	84.1	77.9

NOTES:

1. Period of Record – 5 years (2001 – 2005)

2. Worst 1-Day defined as the calender day with the highest average wet bulb temperature.

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-215
DAILY AVERAGE METEOROLOGICAL DATA – HUNTSVILLE, ALABAMA – DAILY AVERAGE – WORST 5 CONSECUTIVE DAY PERIOD – 2001 – 2005

Date	Dry Bulb Temperature (F)	Wet Bulb Temperature (F)
7-23-2005	82.3	75.5
7-24-2005	82.0	76.0
7-25-2005	84.1	77.9
7-26-2005	84.1	77.8
7-27-2005	82.3	76.4
AVERAGE	83.0	76.7

NOTES:

1. Period of Record – 5 years (2001 – 2005)

2. Worst 5 Consecutive Day Period defined as the 5 consecutive calender days with the highest average wet bulb temperature.

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-216 (Sheet 1 of 2)
DAILY AVERAGE METEOROLOGICAL DATA – HUNTSVILLE, ALABAMA – WORST 30 CONSECUTIVE DAY PERIOD – 2001-2005

			Daily Average	
Year	Month	Day	Dry Bulb (°F)	Wet Bulb (°F)
2005	7	24	82.0	76.0
2005	7	25	84.1	77.9
2005	7	26	84.1	77.8
2005	7	27	82.3	76.4
2005	7	28	79.2	74.4
2005	7	29	79.5	73.1
2005	7	30	77.1	72.9
2005	7	31	78.8	73.3
2005	8	1	79.4	73.6
2005	8	2	79.8	72.2
2005	8	3	80.4	71.2
2005	8	4	81.0	72.4
2005	8	5	80.0	74.2
2005	8	6	77.7	72.7
2005	8	7	75.8	71.2
2005	8	8	77.2	72.8
2005	8	9	79.3	74.0
2005	8	10	80.5	74.1
2005	8	11	79.6	73.6
2005	8	12	80.0	75.2
2005	8	13	76.6	72.8

BLN COL 2.3-1

TABLE 2.3-216 (Sheet 2 of 2)
DAILY AVERAGE METEOROLOGICAL DATA – HUNTSVILLE, ALABAMA – WORST 30 CONSECUTIVE DAY PERIOD – 2001-2005

			Daily Average	
Year	Month	Day	Dry Bulb (°F)	Wet Bulb (°F)
2005	8	14	80.8	73.9
2005	8	15	81.8	75.2
2005	8	16	82.4	75.0
2005	8	17	80.8	74.5
2005	8	18	80.4	75.2
2005	8	19	84.6	76.5
2005	8	20	85.8	77.3
2005	8	21	84.1	76.9
2005	8	22	81.6	75.5
Average			80.6	74.4

NOTES:

1. Period of Record - 5 years (2001 - 2005)

2. Worst 30 Consecutive Day Period defined as the 30 consecutive calender days with the highest average wet bulb temperature.

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-217
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – JANUARY

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.8	3.5	5.9	2.1	0.4	0.1	0.0	12.7	9.8
N-NE	0.4	2.0	3.1	1.1	0.2	0.0	0.0	6.8	9.5
NE	0.3	1.2	1.0	0.1	0.2	0.0	0.0	2.8	8.0
E-NE	0.2	0.7	0.2	0.0	0.0	0.0	0.0	1.1	6.1
E	0.5	2.2	0.9	0.0	0.0	0.0	0.0	3.6	6.3
E-SE	0.5	2.4	1.7	0.3	0.1	0.1	0.0	5.2	8.0
SE	0.6	2.6	1.5	0.2	0.2	0.1	0.0	5.2	7.9
S-SE	0.7	1.3	1.6	0.8	0.2	0.1	0.0	4.8	9.6
S	0.7	2.9	3.8	0.7	0.2	0.1	0.0	8.4	8.6
S-SW	0.4	1.8	2.9	0.7	0.4	0.1	0.0	6.1	9.7
SW	0.5	1.3	2.4	0.8	0.0	0.0	0.0	5.1	9.2
W-SW	0.4	1.0	1.2	0.3	0.0	0.0	0.0	2.9	8.0
W	0.5	1.5	2.4	0.3	0.0	0.0	0.0	4.7	8.3
W-NW	0.6	1.5	2.4	1.1	0.2	0.0	0.0	5.9	9.7
NW	0.6	1.9	2.5	0.9	0.2	0.1	0.0	6.1	9.3
N-NW	0.5	2.1	3.0	1.0	0.2	0.1	0.0	6.9	9.6
CALM	11.7							11.7	
Total	19.9	29.9	36.6	10.5	2.5	0.6	0.0	100.0	8.6

NOTES:

1. Calm is classified as a wind speed less than 0.3 mph.

2. Period of Record – 5 years (2001 – 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-218
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – FEBRUARY

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.9	3.0	5.7	1.3	0.3	0.1	0.0	11.2	9.3
N-NE	0.3	1.7	3.0	0.9	0.0	0.0	0.0	5.9	9.3
NE	0.4	1.2	1.4	0.5	0.0	0.0	0.0	3.5	8.4
E-NE	0.6	1.2	0.6	0.2	0.0	0.0	0.0	2.6	6.8
E	1.1	3.8	1.8	0.2	0.0	0.0	0.0	6.8	6.6
E-SE	0.9	4.1	3.4	0.9	0.2	0.1	0.0	9.6	8.4
SE	1.0	2.5	2.8	0.8	0.3	0.2	0.0	7.5	9.0
S-SE	0.3	1.7	1.8	1.0	0.5	0.1	0.0	5.5	10.5
S	0.5	1.1	2.8	0.7	0.3	0.2	0.0	5.5	10.2
S-SW	0.1	0.6	1.8	0.5	0.2	0.0	0.0	3.3	10.8
SW	0.3	0.5	1.1	0.5	0.2	0.1	0.0	2.8	10.5
W-SW	0.1	0.5	0.7	0.2	0.1	0.0	0.0	1.7	9.5
W	0.5	0.7	1.6	0.8	0.2	0.0	0.0	3.9	9.9
W-NW	0.5	1.5	3.1	1.9	0.4	0.0	0.0	7.5	10.6
NW	0.6	1.6	2.9	1.3	0.3	0.0	0.0	6.7	9.8
N-NW	0.5	2.6	2.3	0.5	0.1	0.0	0.0	5.9	8.3
CALM	10.0							10.0	
Total	18.6	28.3	37.0	12.2	3.0	0.8	0.1	100.0	9.2

NOTES:

1. Period of Record - 5 years (2001 - 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-219
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – MARCH

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.6	2.7	4.4	1.3	0.4	0.0	0.0	9.3	9.8
N-NE	0.2	1.6	2.8	1.9	0.2	0.1	0.0	6.8	10.7
NE	0.4	1.4	1.2	0.8	0.1	0.0	0.0	3.8	9.1
E-NE	0.6	0.7	0.5	0.0	0.0	0.0	0.0	1.8	6.0
E	1.0	2.9	1.1	0.1	0.0	0.0	0.0	5.2	6.4
E-SE	0.6	3.1	2.1	0.6	0.1	0.0	0.0	6.5	8.0
SE	0.8	3.3	2.4	1.0	0.3	0.1	0.0	7.9	8.8
S-SE	0.5	2.2	2.1	1.1	0.1	0.1	0.0	6.1	9.3
S	0.5	2.0	3.8	2.0	0.3	0.0	0.0	8.6	10.3
S-SW	0.3	1.2	2.3	1.4	0.2	0.0	0.0	5.4	10.6
SW	0.2	0.7	1.7	1.0	0.2	0.0	0.0	3.7	11.0
W-SW	0.2	0.9	1.4	0.8	0.3	0.0	0.0	3.6	10.6
W	0.5	1.3	2.0	1.0	0.2	0.1	0.1	5.2	10.2
W-NW	0.3	1.4	2.0	1.3	0.3	0.1	0.1	5.5	11.0
NW	0.3	1.7	2.9	0.8	0.2	0.2	0.0	6.1	10.0
N-NW	0.3	1.5	2.9	0.9	0.2	0.0	0.0	5.9	9.9
CALM	8.6							8.6	
Total	15.7	28.7	35.6	15.9	3.2	0.7	0.2	100.0	9.5

NOTES:

1. Period of Record - 5 years (2001 – 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-220
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – APRIL

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.4	2.2	3.7	1.5	0.1	0.0	0.0	7.9	9.9
N-NE	0.1	1.1	2.2	0.7	0.1	0.0	0.0	4.2	9.8
NE	0.3	0.9	0.9	0.1	0.0	0.0	0.0	2.2	7.4
E-NE	0.3	0.5	0.4	0.1	0.0	0.0	0.0	1.4	7.1
E	0.9	3.6	0.8	0.0	0.0	0.0	0.0	5.3	5.9
E-SE	1.2	3.6	2.7	0.3	0.1	0.0	0.0	7.9	7.2
SE	0.7	2.5	3.1	0.7	0.1	0.0	0.0	7.1	8.3
S-SE	0.5	2.6	2.9	0.9	0.3	0.1	0.0	7.3	9.4
S	0.5	3.4	5.1	2.1	0.7	0.1	0.0	12.0	10.2
S-SW	0.3	1.8	4.6	1.5	0.4	0.0	0.0	8.7	10.3
SW	0.1	1.0	1.9	1.2	0.2	0.0	0.0	4.4	11.0
W-SW	0.2	0.9	0.9	0.7	0.1	0.0	0.0	2.9	10.3
W	0.4	1.1	1.6	0.4	0.1	0.0	0.0	3.6	8.8
W-NW	0.2	1.6	1.9	0.9	0.1	0.1	0.0	4.8	9.5
NW	0.2	1.6	2.0	1.1	0.6	0.3	0.1	5.8	11.6
N-NW	0.3	1.6	3.2	0.8	0.1	0.1	0.0	6.1	9.7
CALM	8.6							8.6	
Total	15.3	29.9	37.8	13.0	3.2	0.7	0.1	100.0	9.1

NOTES:

1. Period of Record - 5 years (2001 – 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-221
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – MAY

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.8	2.2	3.2	0.9	0.1	0.0	0.0	7.2	8.8
N-NE	0.3	1.3	1.9	0.6	0.1	0.0	0.0	4.2	9.2
NE	0.5	1.0	1.2	0.1	0.1	0.0	0.0	2.9	7.6
E-NE	0.1	0.8	0.4	0.1	0.0	0.0	0.0	1.4	7.1
E	2.1	4.1	1.1	0.3	0.0	0.0	0.0	7.6	5.8
E-SE	1.6	4.8	2.6	0.7	0.1	0.0	0.0	9.7	7.1
SE	1.4	3.0	2.5	0.5	0.1	0.0	0.0	7.5	7.3
S-SE	1.0	2.4	2.0	0.6	0.1	0.0	0.0	6.0	7.9
S	0.9	3.6	4.2	1.1	0.2	0.0	0.0	10.0	8.8
S-SW	0.5	1.5	3.1	1.5	0.2	0.0	0.0	6.7	10.1
SW	0.2	1.7	3.8	1.6	0.3	0.0	0.0	7.5	10.7
W-SW	0.2	1.3	2.7	0.8	0.1	0.0	0.0	5.0	9.9
W	0.4	1.0	1.9	0.3	0.1	0.0	0.0	3.7	8.7
W-NW	0.2	0.8	1.0	0.3	0.0	0.0	0.0	2.2	9.1
NW	0.3	0.6	1.1	0.4	0.1	0.1	0.0	2.5	9.9
N-NW	0.6	1.1	1.6	0.6	0.1	0.0	0.0	4.1	8.8
CALM	11.8							11.8	
Total	22.6	30.9	34.4	10.2	1.6	0.2	0.1	100.0	8.6

NOTES:

1. Period of Record - 5 years (2001 – 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-222
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – JUNE

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.9	2.4	2.4	0.2	0.0	0.0	0.0	5.9	7.4
N-NE	0.5	1.6	1.2	0.1	0.0	0.0	0.0	3.3	7.1
NE	0.3	1.0	0.7	0.1	0.0	0.0	0.0	2.2	7.0
E-NE	0.4	0.9	0.7	0.1	0.0	0.0	0.0	2.1	7.1
E	2.1	5.0	1.9	0.3	0.2	0.1	0.0	9.6	6.7
E-SE	2.2	5.6	2.8	0.4	0.0	0.0	0.0	11.1	6.6
SE	1.6	4.0	2.4	0.5	0.0	0.0	0.0	8.5	6.9
S-SE	0.7	2.6	2.0	0.3	0.0	0.0	0.0	5.5	7.4
S	1.3	4.4	2.6	0.2	0.1	0.1	0.0	8.7	7.1
S-SW	0.6	2.1	2.0	0.1	0.1	0.0	0.0	4.9	7.5
SW	0.4	1.5	2.1	0.2	0.0	0.0	0.0	4.2	8.0
W-SW	0.3	1.6	2.1	0.3	0.0	0.0	0.0	4.4	8.2
W	0.3	1.1	1.2	0.1	0.0	0.0	0.0	2.7	7.8
W-NW	0.4	0.7	0.8	0.1	0.1	0.0	0.0	2.1	7.5
NW	0.4	1.3	1.5	0.1	0.0	0.0	0.0	3.3	7.5
N-NW	0.6	1.4	1.1	0.1	0.0	0.0	0.0	3.3	6.9
CALM	18.2							18.2	
Total	31.2	37.2	27.4	3.4	0.6	0.3	0.0	100.0	7.3

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-223
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – JULY

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.5	1.8	1.5	0.2	0.0	0.0	0.0	4.0	7.3
N-NE	0.4	0.9	0.6	0.0	0.0	0.1	0.0	2.0	7.5
NE	0.3	0.9	0.5	0.1	0.0	0.0	0.0	1.9	7.0
E-NE	0.4	0.5	0.3	0.2	0.1	0.0	0.0	1.6	8.2
E	1.5	2.6	1.0	0.2	0.1	0.1	0.0	5.5	6.8
E-SE	2.3	3.0	0.9	0.2	0.1	0.0	0.0	6.3	5.7
SE	2.1	2.2	1.0	0.1	0.0	0.0	0.0	5.4	5.6
S-SE	1.6	2.7	0.9	0.0	0.1	0.0	0.0	5.3	5.8
S	2.5	4.2	1.8	0.1	0.1	0.0	0.0	8.7	6.1
S-SW	1.3	2.8	1.6	0.1	0.0	0.0	0.0	5.8	6.3
SW	1.1	3.2	2.2	0.2	0.0	0.0	0.0	6.7	7.0
W-SW	0.8	2.7	1.5	0.2	0.0	0.0	0.0	5.2	6.7
W	1.3	2.7	1.7	0.0	0.0	0.0	0.0	5.7	6.3
W-NW	0.9	1.7	0.8	0.0	0.1	0.0	0.0	3.6	6.9
NW	0.9	1.7	0.9	0.0	0.0	0.0	0.0	3.5	6.2
N-NW	0.5	1.5	0.5	0.0	0.0	0.0	0.0	2.5	6.4
CALM	26.1							26.1	
Total	44.5	35.1	17.8	1.6	0.6	0.3	0.1	100.0	6.6

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-224
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – AUGUST

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.9	3.3	1.5	0.2	0.0	0.0	0.0	6.0	6.7
N-NE	0.4	1.1	1.1	0.3	0.0	0.0	0.0	2.9	7.9
NE	0.3	0.9	0.5	0.2	0.0	0.0	0.0	2.0	7.2
E-NE	0.5	0.7	0.4	0.2	0.0	0.0	0.0	1.8	7.3
E	1.6	4.4	1.4	0.5	0.0	0.0	0.1	8.1	6.8
E-SE	2.5	4.7	2.3	0.3	0.0	0.0	0.1	9.8	6.4
SE	2.0	3.0	1.6	0.1	0.0	0.0	0.0	6.7	6.0
S-SE	1.2	2.5	0.9	0.0	0.0	0.0	0.1	4.7	6.2
S	1.8	3.8	1.5	0.1	0.0	0.1	0.0	7.3	6.2
S-SW	0.8	1.5	1.4	0.0	0.0	0.1	0.0	3.8	7.2
SW	0.6	1.9	1.4	0.0	0.0	0.0	0.0	3.9	6.8
W-SW	0.8	1.6	1.1	0.1	0.0	0.0	0.0	3.7	6.7
W	0.7	1.7	0.8	0.1	0.1	0.0	0.0	3.2	6.4
W-NW	0.5	1.7	0.5	0.1	0.0	0.0	0.0	2.7	6.1
NW	0.8	1.7	0.5	0.0	0.0	0.0	0.0	3.0	5.7
N-NW	1.2	2.1	1.0	0.1	0.0	0.0	0.0	4.4	6.2
CALM	26.2							26.2	
Total	42.7	36.4	18.0	2.2	0.2	0.3	0.3	100.0	6.6

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-225
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – SEPTEMBER

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.9	4.6	4.5	0.6	0.2	0.1	0.0	10.9	8.1
N-NE	0.4	2.6	3.4	0.7	0.0	0.0	0.1	7.2	8.9
NE	0.6	1.8	2.1	0.1	0.1	0.1	0.1	4.7	8.4
E-NE	0.7	1.4	0.9	0.1	0.0	0.0	0.0	3.1	6.6
E	1.6	5.9	2.3	0.3	0.1	0.0	0.0	10.1	6.5
E-SE	1.9	5.7	4.5	0.4	0.1	0.1	0.0	12.6	7.3
SE	0.7	3.2	3.2	0.7	0.1	0.1	0.0	7.9	8.3
S-SE	0.6	1.7	2.2	0.5	0.1	0.2	0.1	5.5	9.6
S	0.4	1.6	1.5	0.3	0.0	0.0	0.0	3.8	8.1
S-SW	0.3	0.7	0.7	0.1	0.0	0.0	0.0	1.8	7.2
SW	0.1	0.4	0.4	0.1	0.0	0.0	0.0	1.0	7.7
W-SW	0.2	0.4	0.5	0.0	0.0	0.0	0.0	1.1	7.0
W	0.4	0.6	0.4	0.0	0.0	0.0	0.0	1.5	6.0
W-NW	0.6	1.2	0.4	0.1	0.0	0.0	0.0	2.3	6.4
NW	0.6	1.2	0.9	0.2	0.1	0.0	0.0	3.1	7.7
N-NW	0.6	1.8	1.7	0.3	0.0	0.0	0.0	4.3	7.5
CALM	19.1							19.1	
Total	29.7	34.8	29.6	4.4	0.8	0.5	0.2	100.0	7.6

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-226
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – OCTOBER

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.1	3.8	3.6	0.7	0.0	0.0	0.0	9.2	7.8
N-NE	0.5	2.0	2.4	0.5	0.0	0.0	0.0	5.5	8.1
NE	0.4	1.2	1.1	0.1	0.0	0.0	0.0	2.9	7.3
E-NE	0.5	1.0	0.3	0.0	0.0	0.0	0.0	1.8	5.8
E	1.9	5.5	1.6	0.1	0.0	0.0	0.0	9.1	5.8
E-SE	1.2	5.4	4.9	0.7	0.1	0.0	0.0	12.2	7.6
SE	1.0	3.0	2.6	0.6	0.1	0.0	0.0	7.3	7.9
S-SE	0.5	2.3	2.3	0.4	0.1	0.0	0.0	5.7	8.0
S	0.7	2.3	3.0	0.6	0.1	0.0	0.0	6.7	8.3
S-SW	0.4	0.9	1.4	0.2	0.0	0.0	0.0	3.0	8.2
SW	0.3	0.8	0.8	0.3	0.0	0.0	0.0	2.2	8.4
W-SW	0.2	0.7	0.9	0.3	0.0	0.0	0.0	2.1	8.3
W	0.3	1.3	1.4	0.3	0.0	0.0	0.0	3.3	8.4
W-NW	0.3	1.1	1.0	0.6	0.1	0.1	0.0	3.1	9.5
NW	0.7	1.5	1.5	0.6	0.2	0.0	0.0	4.5	8.7
N-NW	1.0	1.9	1.8	0.3	0.1	0.0	0.0	5.0	7.5
CALM	16.6							16.6	
Total	27.6	34.6	30.7	6.2	0.9	0.1	0.0	100.0	7.8

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-227
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – NOVEMBER

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.5	2.8	4.6	1.2	0.1	0.0	0.0	9.2	9.1
N-NE	0.3	1.5	2.8	0.5	0.0	0.0	0.0	5.2	9.1
NE	0.3	0.9	0.9	0.0	0.0	0.0	0.0	2.2	6.8
E-NE	0.4	0.8	0.2	0.0	0.0	0.0	0.0	1.4	5.4
E	2.3	5.0	1.0	0.0	0.0	0.0	0.0	8.3	5.4
E-SE	1.5	4.2	2.4	0.6	0.1	0.0	0.0	8.8	7.2
SE	0.8	2.5	2.9	0.8	0.3	0.1	0.0	7.6	9.1
S-SE	0.5	2.5	3.3	1.3	0.3	0.3	0.1	8.2	10.4
S	0.6	2.5	3.8	1.2	0.5	0.1	0.0	8.7	9.8
S-SW	0.3	1.1	1.7	0.6	0.1	0.0	0.0	3.7	9.3
SW	0.1	0.9	1.4	0.8	0.2	0.0	0.0	3.5	10.4
W-SW	0.3	0.7	1.4	0.4	0.0	0.0	0.0	2.8	9.1
W	0.5	0.8	1.1	0.6	0.2	0.0	0.0	3.2	9.4
W-NW	0.3	1.2	1.3	0.6	0.0	0.0	0.0	3.6	8.9
NW	0.5	1.7	1.8	1.1	0.2	0.0	0.0	5.3	9.8
N-NW	0.4	1.5	2.0	0.4	0.1	0.0	0.0	4.4	8.6
CALM	14.1							14.1	
Total	23.8	30.7	32.7	10.1	2.2	0.6	0.1	100.0	8.6

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-228
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – DECEMBER

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.0	3.9	3.3	1.0	0.0	0.0	0.0	9.2	7.9
N-NE	0.2	1.2	1.8	0.4	0.1	0.0	0.0	3.6	8.7
NE	0.3	1.0	1.0	0.1	0.0	0.0	0.0	2.5	7.4
E-NE	0.4	0.7	0.6	0.0	0.0	0.0	0.0	1.7	6.5
E	1.5	3.6	0.7	0.1	0.0	0.0	0.0	5.9	5.7
E-SE	1.2	3.6	2.2	0.5	0.1	0.1	0.0	7.7	7.5
SE	0.7	2.6	2.2	1.3	0.8	0.1	0.0	7.7	10.0
S-SE	0.3	2.1	3.0	1.6	0.6	0.2	0.0	7.9	11.1
S	0.4	1.8	2.8	1.2	0.3	0.0	0.0	6.6	9.9
S-SW	0.4	1.0	1.1	0.4	0.3	0.1	0.0	3.3	9.6
SW	0.3	1.1	1.2	0.1	0.1	0.1	0.0	2.8	8.6
W-SW	0.3	0.8	1.3	0.4	0.0	0.0	0.0	2.8	9.0
W	0.5	1.1	2.0	1.3	0.3	0.1	0.0	5.2	10.6
W-NW	0.3	1.8	3.2	1.7	0.3	0.1	0.0	7.5	10.5
NW	0.8	2.0	2.6	1.0	0.2	0.0	0.0	6.6	9.0
N-NW	0.6	2.0	1.6	0.6	0.1	0.0	0.0	4.9	8.2
CALM	14.1							14.1	
Total	23.5	30.4	30.6	11.6	3.1	0.8	0.0	100.0	8.8

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-1

TABLE 2.3-229
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – HUNTSVILLE, ALABAMA – 2001 – 2005 – ALL MONTHS

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.8	3.0	3.7	0.9	0.1	0.0	0.0	8.5	8.5
N-NE	0.3	1.5	2.2	0.6	0.1	0.0	0.0	4.8	8.8
NE	0.4	1.1	1.1	0.2	0.0	0.0	0.0	2.8	7.6
E-NE	0.4	0.8	0.5	0.1	0.0	0.0	0.0	1.8	6.7
E	1.5	4.0	1.3	0.2	0.0	0.0	0.0	7.1	6.2
E-SE	1.5	4.2	2.7	0.5	0.1	0.0	0.0	9.0	7.3
SE	1.1	2.9	2.4	0.6	0.2	0.1	0.0	7.2	7.9
S-SE	0.7	2.2	2.1	0.7	0.2	0.1	0.0	6.0	8.8
S	0.9	2.8	3.1	0.9	0.2	0.1	0.0	7.9	8.6
S-SW	0.5	1.4	2.1	0.6	0.2	0.0	0.0	4.7	8.9
SW	0.3	1.2	1.7	0.6	0.1	0.0	0.0	4.0	9.1
W-SW	0.3	1.1	1.3	0.4	0.1	0.0	0.0	3.2	8.6
W	0.5	1.2	1.5	0.4	0.1	0.0	0.0	3.8	8.4
W-NW	0.4	1.3	1.5	0.7	0.1	0.0	0.0	4.2	8.8
NW	0.6	1.5	1.7	0.6	0.2	0.0	0.0	4.7	8.8
N-NW	0.6	1.8	1.9	0.5	0.1	0.0	0.0	4.8	8.1
CALM	15.4							15.4	
Total	26.3	32.2	30.7	8.5	1.8	0.5	0.1	100.0	8.2

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-230
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – JANUARY

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.7	3.9	3.1	0.8	0.0	0.0	0.0	10.5	6.8
N-NE	4.9	5.8	2.2	0.2	0.0	0.0	0.0	13.1	5.4
NE	5.0	5.4	2.0	0.2	0.0	0.0	0.0	12.6	5.3
E-NE	2.7	0.5	0.1	0.0	0.0	0.0	0.0	3.4	3.0
E	0.9	0.2	0.0	0.0	0.0	0.0	0.0	1.2	3.0
E-SE	0.4	0.2	0.0	0.0	0.0	0.0	0.0	0.7	4.0
SE	0.8	0.8	0.7	0.1	0.0	0.1	0.0	2.6	7.5
S-SE	0.9	0.8	0.3	0.1	0.0	0.0	0.0	2.1	4.9
S	1.9	2.2	0.8	0.2	0.0	0.0	0.0	5.2	5.6
S-SW	3.0	2.7	3.0	0.8	0.0	0.0	0.0	9.6	6.9
SW	3.8	2.4	1.4	0.5	0.0	0.0	0.0	8.1	5.4
W-SW	2.1	1.4	1.1	0.8	0.3	0.0	0.0	5.7	7.6
W	1.6	1.0	1.3	0.5	0.0	0.0	0.0	4.4	6.9
W-NW	1.1	1.5	1.8	1.0	0.0	0.0	0.0	5.3	8.2
NW	2.0	1.4	2.4	0.7	0.0	0.0	0.0	6.5	7.3
N-NW	2.9	2.1	3.0	0.9	0.1	0.0	0.0	9.0	7.2
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	36.8	32.3	23.3	7.0	0.5	0.1	0.0	100.0	5.6

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-231
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – FEBRUARY

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	3.9	3.9	3.2	1.1	0.2	0.1	0.0	12.5	7.2
N-NE	5.3	6.7	2.5	0.6	0.0	0.0	0.0	15.1	5.9
NE	5.9	6.2	2.2	0.6	0.1	0.0	0.0	15.0	5.5
E-NE	2.4	1.3	0.4	0.2	0.0	0.0	0.0	4.3	4.7
E	0.9	0.3	0.0	0.1	0.0	0.0	0.0	1.3	4.0
E-SE	0.3	0.3	0.0	0.0	0.0	0.0	0.0	0.6	5.0
SE	0.6	0.9	0.4	0.1	0.1	0.1	0.1	2.3	9.1
S-SE	0.7	0.8	0.3	0.1	0.1	0.0	0.0	2.0	5.8
S	1.3	2.4	1.4	0.5	0.1	0.0	0.0	5.7	7.0
S-SW	2.5	3.0	1.9	1.1	0.1	0.0	0.0	8.7	7.2
SW	2.5	2.1	1.6	0.7	0.3	0.0	0.0	7.1	7.3
W-SW	1.5	1.4	1.1	0.7	0.2	0.0	0.0	4.9	7.7
W	1.2	0.6	0.7	0.7	0.0	0.0	0.0	3.2	7.4
W-NW	0.7	1.0	0.9	0.6	0.1	0.0	0.0	3.2	8.3
NW	1.4	1.4	1.2	0.6	0.1	0.0	0.0	4.8	7.5
N-NW	2.9	2.4	1.9	1.7	0.1	0.0	0.0	9.0	7.8
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	34.3	34.6	19.6	9.3	1.6	0.3	0.2	100.0	6.3

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-232
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – MARCH

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.8	2.9	2.9	1.1	0.0	0.0	0.0	9.7	7.1
N-NE	3.4	3.6	1.9	0.0	0.0	0.0	0.0	8.9	5.4
NE	4.4	4.2	2.3	0.2	0.1	0.0	0.0	11.3	5.6
E-NE	1.8	0.9	0.7	0.4	0.1	0.0	0.0	3.9	6.2
E	0.7	0.3	0.2	0.2	0.0	0.0	0.0	1.4	6.2
E-SE	0.5	0.3	0.2	0.2	0.0	0.0	0.0	1.2	6.6
SE	0.6	0.9	1.2	0.8	0.5	0.2	0.0	4.1	10.8
S-SE	0.8	0.6	0.7	0.1	0.1	0.0	0.0	2.4	7.0
S	1.9	2.3	3.0	1.6	0.4	0.0	0.0	9.1	8.7
S-SW	3.7	3.4	4.3	2.0	0.2	0.1	0.0	13.8	8.1
SW	3.2	3.0	2.6	1.3	0.2	0.0	0.0	10.3	7.3
W-SW	1.7	1.8	1.7	0.5	0.1	0.0	0.0	5.7	7.2
W	0.9	0.7	1.2	0.4	0.0	0.0	0.0	3.2	7.4
W-NW	0.6	0.5	0.9	0.5	0.0	0.0	0.0	2.4	8.7
NW	1.5	1.4	1.4	0.7	0.1	0.1	0.0	5.1	8.0
N-NW	2.2	2.1	2.5	0.7	0.1	0.0	0.0	7.6	7.5
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2
Total	30.7	28.9	27.5	10.6	1.9	0.4	0.0	100.0	6.9

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-233
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – APRIL

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.7	2.3	2.0	0.4	0.0	0.0	0.0	6.4	6.7
N-NE	3.4	4.4	2.4	0.2	0.0	0.0	0.0	10.4	5.8
NE	4.4	5.1	2.8	0.6	0.0	0.0	0.0	12.9	5.9
E-NE	2.0	1.5	0.6	0.2	0.0	0.0	0.0	4.3	5.2
E	0.8	0.3	0.1	0.1	0.0	0.0	0.0	1.4	5.0
E-SE	0.8	1.0	0.3	0.0	0.1	0.0	0.0	2.3	5.5
SE	1.8	2.3	1.1	0.6	0.3	0.0	0.0	6.2	7.3
S-SE	1.3	1.2	0.8	0.2	0.1	0.0	0.0	3.7	6.4
S	3.1	4.0	4.1	1.3	0.3	0.0	0.0	12.9	7.7
S-SW	3.4	3.8	2.6	1.4	0.2	0.0	0.0	11.4	7.2
SW	2.8	3.3	2.2	0.4	0.0	0.0	0.0	8.8	6.3
W-SW	1.6	1.3	1.3	0.6	0.0	0.0	0.0	4.8	7.1
W	0.7	0.3	0.6	0.6	0.1	0.0	0.0	2.3	8.7
W-NW	0.4	0.5	0.7	0.6	0.0	0.0	0.0	2.2	9.3
NW	1.0	1.5	0.9	0.3	0.0	0.0	0.0	3.7	6.7
N-NW	1.5	2.2	2.2	0.3	0.0	0.0	0.0	6.3	7.1
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
Total	30.8	35.2	24.8	7.8	1.4	0.0	0.0	100.0	6.4

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-234
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – MAY

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.9	2.8	1.1	0.1	0.0	0.0	0.0	5.9	5.6
N-NE	4.2	5.3	0.8	0.0	0.0	0.0	0.0	10.4	4.8
NE	7.6	6.1	1.8	0.1	0.0	0.0	0.0	15.6	4.7
E-NE	4.0	1.6	0.5	0.0	0.0	0.0	0.0	6.2	3.8
E	1.7	1.0	0.1	0.0	0.0	0.0	0.0	2.8	3.5
E-SE	1.3	0.8	0.4	0.1	0.0	0.0	0.0	2.6	5.1
SE	2.4	1.8	0.9	0.2	0.0	0.0	0.0	5.4	5.5
S-SE	1.9	1.8	0.5	0.1	0.0	0.0	0.0	4.2	4.8
S	3.4	5.4	2.5	0.7	0.0	0.0	0.0	12.1	6.3
S-SW	3.5	3.3	2.7	0.6	0.0	0.0	0.0	10.1	6.2
SW	2.6	3.6	2.1	0.2	0.0	0.0	0.0	8.5	6.2
W-SW	1.3	1.3	1.4	0.1	0.0	0.0	0.0	4.1	6.5
W	0.4	0.7	0.4	0.0	0.0	0.0	0.0	1.5	6.0
W-NW	0.4	0.7	0.4	0.0	0.0	0.0	0.0	1.5	6.0
NW	1.5	0.9	0.5	0.2	0.0	0.0	0.0	3.1	5.5
N-NW	1.6	2.7	1.4	0.3	0.0	0.0	0.0	6.0	6.3
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	39.6	39.9	17.5	2.8	0.1	0.0	0.0	100.0	5.1

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-235
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – JUNE

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.2	1.9	0.8	0.4	0.0	0.0	0.0	5.3	5.8
N-NE	5.8	4.9	1.0	0.2	0.0	0.0	0.0	11.8	4.5
NE	6.7	5.9	1.3	0.2	0.0	0.0	0.0	14.1	4.6
E-NE	3.4	1.6	0.3	0.0	0.0	0.0	0.0	5.2	3.6
E	1.9	0.6	0.0	0.0	0.0	0.0	0.0	2.4	2.8
E-SE	1.5	0.5	0.1	0.0	0.0	0.0	0.0	2.0	3.0
SE	2.7	1.6	0.7	0.0	0.0	0.0	0.0	5.0	4.5
S-SE	2.1	1.3	0.4	0.0	0.0	0.0	0.0	3.8	4.1
S	3.9	5.6	2.6	0.0	0.0	0.0	0.0	12.1	5.5
S-SW	4.5	4.5	3.2	0.3	0.0	0.0	0.0	12.6	5.9
SW	3.2	3.0	2.2	0.4	0.0	0.0	0.0	8.8	6.1
W-SW	1.9	2.2	0.9	0.0	0.0	0.0	0.0	5.0	5.6
W	1.0	0.9	0.4	0.0	0.0	0.0	0.0	2.3	5.0
W-NW	0.9	0.3	0.3	0.1	0.0	0.0	0.0	1.6	4.9
NW	1.3	1.0	0.5	0.0	0.0	0.0	0.0	2.8	5.1
N-NW	1.9	1.5	1.2	0.4	0.0	0.0	0.0	5.0	6.2
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	44.9	37.1	15.8	2.1	0.1	0.0	0.0	100.0	4.6

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-236
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – JULY

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.1	2.3	0.6	0.1	0.0	0.0	0.0	5.1	5.0
N-NE	5.8	3.9	0.5	0.0	0.0	0.0	0.0	10.2	4.0
NE	6.9	5.7	1.4	0.0	0.0	0.0	0.0	14.0	4.4
E-NE	4.6	1.7	0.3	0.0	0.0	0.0	0.0	6.7	3.4
E	1.7	0.6	0.3	0.0	0.0	0.0	0.0	2.7	3.8
E-SE	1.0	0.6	0.3	0.0	0.0	0.0	0.0	2.0	4.4
SE	2.1	2.2	0.8	0.1	0.0	0.0	0.0	5.1	5.0
S-SE	1.7	2.0	0.4	0.0	0.0	0.0	0.0	4.1	4.6
S	3.7	4.2	0.9	0.2	0.0	0.0	0.0	9.0	5.1
S-SW	5.1	4.0	2.1	0.2	0.0	0.0	0.0	11.4	5.2
SW	4.3	3.6	1.7	0.1	0.0	0.0	0.0	9.7	5.1
W-SW	2.9	2.8	1.5	0.0	0.0	0.0	0.0	7.2	5.4
W	1.2	0.9	0.7	0.0	0.0	0.0	0.0	2.8	5.3
W-NW	0.7	0.9	0.9	0.0	0.0	0.0	0.0	2.5	6.0
NW	1.3	1.7	0.5	0.0	0.0	0.0	0.0	3.5	5.1
N-NW	1.4	1.9	0.7	0.1	0.0	0.0	0.0	4.0	5.5
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	46.4	38.9	13.8	0.9	0.0	0.0	0.0	100.0	4.6

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-237
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – AUGUST

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.9	3.0	0.7	0.0	0.0	0.0	0.0	6.6	4.6
N-NE	9.6	6.4	0.4	0.0	0.0	0.0	0.0	16.4	3.9
NE	9.3	7.9	0.6	0.0	0.0	0.0	0.0	17.8	4.0
E-NE	5.2	1.9	0.2	0.0	0.0	0.0	0.0	7.4	3.4
E	2.0	0.8	0.0	0.0	0.0	0.0	0.0	2.8	3.1
E-SE	1.0	0.6	0.0	0.0	0.0	0.0	0.0	1.5	3.2
SE	3.1	1.4	0.1	0.0	0.0	0.0	0.0	4.6	3.6
S-SE	1.6	1.5	0.4	0.0	0.0	0.0	0.0	3.5	4.5
S	4.5	3.6	1.2	0.1	0.0	0.0	0.0	9.4	4.7
S-SW	3.2	3.1	0.4	0.0	0.0	0.0	0.0	6.7	4.2
SW	3.4	2.3	0.5	0.0	0.0	0.0	0.0	6.3	4.2
W-SW	2.0	2.6	0.4	0.0	0.0	0.0	0.0	4.9	4.8
W	1.6	1.5	0.1	0.0	0.0	0.0	0.0	3.2	4.0
W-NW	0.7	0.6	0.1	0.0	0.0	0.0	0.0	1.4	4.1
NW	1.8	1.2	0.1	0.0	0.0	0.0	0.0	3.1	3.8
N-NW	2.4	1.6	0.2	0.0	0.0	0.0	0.0	4.3	4.1
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	54.3	40.0	5.4	0.2	0.0	0.0	0.0	100.0	3.8

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-238
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – SEPTEMBER

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	3.8	3.6	1.3	0.2	0.0	0.0	0.0	8.9	5.0
N-NE	9.5	7.8	1.6	0.0	0.0	0.0	0.0	18.8	4.3
NE	9.6	9.0	2.3	0.1	0.0	0.0	0.0	21.0	4.6
E-NE	5.2	3.0	0.9	0.0	0.0	0.0	0.0	9.2	4.2
E	2.3	1.1	0.3	0.3	0.0	0.0	0.0	4.0	4.7
E-SE	0.9	0.5	0.1	0.0	0.0	0.0	0.0	1.5	3.8
SE	1.5	1.0	0.3	0.0	0.0	0.0	0.0	2.9	4.8
S-SE	1.1	0.9	0.3	0.1	0.0	0.0	0.0	2.5	5.1
S	2.0	2.3	1.3	0.2	0.2	0.0	0.0	6.0	5.9
S-SW	2.4	1.8	1.0	0.0	0.0	0.0	0.0	5.2	5.0
SW	2.7	1.8	0.5	0.0	0.0	0.0	0.0	5.0	4.0
W-SW	2.0	0.7	0.2	0.0	0.0	0.0	0.0	2.8	3.4
W	1.3	0.3	0.1	0.1	0.0	0.0	0.0	1.7	3.7
W-NW	0.8	0.6	0.1	0.0	0.0	0.0	0.0	1.5	4.4
NW	1.7	1.1	0.3	0.0	0.0	0.0	0.0	3.0	4.4
N-NW	2.4	2.2	1.0	0.0	0.0	0.0	0.0	5.7	5.1
CALM	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.2
Total	49.5	37.7	11.5	1.0	0.3	0.0	0.0	100.0	4.3

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-239
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – OCTOBER

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	3.2	2.8	1.7	0.3	0.0	0.0	0.0	8.1	5.8
N-NE	7.7	5.7	2.0	0.1	0.0	0.0	0.0	15.5	4.7
NE	7.6	6.0	1.4	0.1	0.0	0.0	0.0	15.2	4.5
E-NE	3.6	1.5	0.2	0.0	0.0	0.0	0.0	5.3	3.5
E	1.6	0.5	0.0	0.0	0.0	0.0	0.0	2.0	3.0
E-SE	0.7	0.6	0.2	0.0	0.0	0.0	0.0	1.5	4.7
SE	1.7	2.1	1.1	0.1	0.0	0.0	0.0	5.1	5.9
S-SE	1.6	1.6	0.7	0.1	0.0	0.0	0.0	4.1	5.1
S	2.3	3.0	2.0	0.5	0.0	0.0	0.0	7.8	6.5
S-SW	2.9	3.4	1.5	0.6	0.0	0.0	0.0	8.4	5.9
SW	3.8	2.4	0.7	0.2	0.0	0.0	0.0	7.1	4.5
W-SW	2.6	1.5	0.5	0.0	0.0	0.0	0.0	4.6	4.2
W	0.9	0.5	0.9	0.1	0.0	0.0	0.0	2.3	6.5
W-NW	1.0	1.0	0.6	0.1	0.0	0.0	0.0	2.7	5.6
NW	1.8	0.9	0.6	0.0	0.0	0.0	0.0	3.3	4.8
N-NW	3.0	2.0	1.3	0.2	0.0	0.0	0.0	6.5	5.4
CALM	0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.2
Total	46.7	35.5	15.4	2.4	0.0	0.0	0.0	100.0	4.7

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-240
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – NOVEMBER

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	4.3	3.5	2.6	0.1	0.0	0.0	0.0	10.4	5.5
N-NE	5.8	6.7	1.8	0.2	0.0	0.0	0.0	14.6	5.1
NE	5.3	4.6	2.2	0.3	0.0	0.0	0.0	12.4	5.3
E-NE	2.3	1.4	0.3	0.0	0.0	0.0	0.0	4.0	3.8
E	0.8	0.3	0.0	0.0	0.0	0.0	0.0	1.2	3.2
E-SE	0.5	0.2	0.1	0.0	0.0	0.0	0.0	0.8	4.2
SE	1.2	1.5	0.9	0.3	0.0	0.0	0.0	3.8	6.3
S-SE	0.9	1.3	0.7	0.3	0.0	0.0	0.0	3.2	6.4
S	2.7	3.0	1.5	0.5	0.2	0.0	0.0	8.0	6.5
S-SW	3.3	2.7	2.7	0.7	0.0	0.0	0.0	9.5	6.5
SW	2.7	2.7	1.9	0.2	0.0	0.0	0.0	7.5	5.8
W-SW	2.2	2.0	0.5	0.2	0.0	0.0	0.0	4.9	4.8
W	1.2	0.5	0.9	0.1	0.0	0.0	0.0	2.7	5.9
W-NW	1.2	1.2	0.9	0.2	0.0	0.0	0.0	3.4	6.5
NW	1.9	1.5	1.5	0.2	0.0	0.0	0.0	5.0	6.1
N-NW	4.1	2.5	1.5	0.4	0.0	0.0	0.0	8.5	5.2
CALM	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.2
Total	40.4	35.7	20.0	3.8	0.2	0.0	0.0	100.0	5.1

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-241
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – DECEMBER

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.9	4.0	4.7	1.3	0.1	0.0	0.0	13.1	7.9
N-NE	3.7	6.3	2.9	0.2	0.0	0.0	0.0	13.1	6.0
NE	3.4	4.6	2.1	0.4	0.0	0.0	0.0	10.4	5.9
E-NE	1.7	0.9	0.2	0.0	0.0	0.0	0.0	2.8	3.9
E	0.9	0.2	0.1	0.0	0.0	0.0	0.0	1.2	3.4
E-SE	0.4	0.2	0.1	0.0	0.0	0.0	0.0	0.7	4.4
SE	1.1	2.2	1.1	0.1	0.0	0.0	0.0	4.5	6.4
S-SE	1.4	1.6	0.6	0.2	0.0	0.0	0.0	3.8	6.0
S	2.1	3.8	1.5	0.3	0.0	0.0	0.0	7.7	6.2
S-SW	2.2	3.9	2.9	1.5	0.0	0.0	0.0	10.6	7.6
SW	2.9	2.9	2.4	0.5	0.0	0.0	0.0	8.7	6.5
W-SW	2.0	1.7	0.6	0.3	0.0	0.0	0.0	4.7	5.6
W	1.6	1.2	0.3	0.1	0.0	0.0	0.0	3.3	4.7
W-NW	0.6	0.7	0.5	0.1	0.0	0.0	0.0	1.8	6.1
NW	1.8	0.7	0.7	0.1	0.0	0.0	0.0	3.4	5.0
N-NW	3.6	2.5	2.8	0.9	0.1	0.0	0.0	10.1	6.8
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	32.5	37.6	23.6	5.9	0.4	0.0	0.0	100.0	5.4

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.
2. Data measured at 10 meter elevation.
3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.
4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-242
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 1979 – 1982 – ALL MONTHS

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.9	3.1	2.1	0.5	0.0	0.0	0.0	8.5	6.3
N-NE	5.8	5.6	1.7	0.2	0.0	0.0	0.0	13.2	4.9
NE	6.4	5.9	1.8	0.2	0.0	0.0	0.0	14.4	4.9
E-NE	3.3	1.5	0.4	0.1	0.0	0.0	0.0	5.2	4.0
E	1.4	0.5	0.1	0.0	0.0	0.0	0.0	2.0	3.8
E-SE	0.8	0.5	0.2	0.0	0.0	0.0	0.0	1.5	4.5
SE	1.6	1.6	0.8	0.2	0.1	0.0	0.0	4.3	6.2
S-SE	1.3	1.3	0.5	0.1	0.0	0.0	0.0	3.3	5.3
S	2.8	3.5	1.9	0.5	0.1	0.0	0.0	8.7	6.3
S-SW	3.3	3.3	2.3	0.8	0.0	0.0	0.0	9.8	6.5
SW	3.2	2.8	1.6	0.4	0.0	0.0	0.0	8.0	5.8
W-SW	2.0	1.7	0.9	0.3	0.1	0.0	0.0	5.0	5.9
W	1.1	0.8	0.6	0.2	0.0	0.0	0.0	2.7	6.0
W-NW	0.8	0.8	0.7	0.2	0.0	0.0	0.0	2.5	6.9
NW	1.6	1.2	0.9	0.2	0.0	0.0	0.0	3.9	6.0
N-NW	2.5	2.1	1.6	0.5	0.0	0.0	0.0	6.8	6.4
CALM	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
Total	40.8	36.2	18.0	4.4	0.5	0.1	0.0	100.0	5.3

NOTES:

1. Calm wind speed is defined as a wind speed less than 0.3 mph.

2. Data measured at 10 meter elevation.

3. Totals may not exactly equal the sum of the directional percentages since results rounded to one decimal place.

4. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-2

TABLE 2.3-243
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – JANUARY

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	3.7	3.7	2.5	0.0	0.0	0.0	0.0	9.8	5.4
N-NE	4.2	2.9	1.1	0.0	0.0	0.0	0.0	8.2	4.3
NE	6.9	1.9	0.5	0.0	0.0	0.0	0.0	9.4	3.4
E-NE	2.3	0.4	0.0	0.0	0.0	0.0	0.0	2.7	2.4
E	0.3	0.1	0.0	0.0	0.0	0.0	0.0	0.4	2.4
E-SE	0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.5	1.8
SE	1.2	0.0	0.0	0.0	0.0	0.0	0.0	1.2	2.4
S-SE	1.6	0.8	0.0	0.0	0.0	0.0	0.0	2.5	3.4
S	2.2	3.3	0.8	0.0	0.0	0.0	0.0	6.3	5.1
S-SW	5.9	9.3	2.0	0.1	0.0	0.0	0.0	17.3	5.3
SW	8.9	4.8	1.1	0.0	0.0	0.0	0.0	14.7	4.0
W-SW	3.1	1.4	0.3	0.0	0.0	0.0	0.0	4.8	3.6
W	1.6	1.5	0.8	0.0	0.0	0.0	0.0	4.0	5.7
W-NW	1.8	1.6	1.1	0.0	0.0	0.0	0.0	4.5	5.8
NW	1.5	2.0	2.0	0.1	0.0	0.0	0.0	5.7	6.7
N-NW	2.3	4.1	1.6	0.0	0.0	0.0	0.0	8.0	5.8
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	48.1	37.7	13.9	0.3	0.0	0.0	0.0	100.0	4.7

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-244
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – FEBRUARY

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	4.5	6.8	2.4	0.0	0.0	0.0	0.0	13.7	5.6
N-NE	6.5	6.9	0.9	0.0	0.0	0.0	0.0	14.3	4.5
NE	5.1	3.8	0.6	0.0	0.0	0.0	0.0	9.5	4.4
E-NE	1.8	0.9	0.2	0.0	0.0	0.0	0.0	2.9	3.7
E	1.5	0.0	0.0	0.0	0.0	0.0	0.0	1.5	1.3
E-SE	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.3	1.1
SE	0.8	0.2	0.6	0.2	0.0	0.0	0.0	1.7	6.3
S-SE	0.8	0.6	0.8	0.0	0.0	0.0	0.0	2.1	6.1
S	2.3	3.0	0.9	0.3	0.0	0.0	0.0	6.5	5.5
S-SW	3.9	5.4	5.0	0.3	0.0	0.0	0.0	14.6	6.6
SW	4.2	2.9	1.2	0.5	0.0	0.0	0.0	8.7	5.3
W-SW	2.4	1.7	0.9	0.0	0.0	0.0	0.0	5.0	5.0
W	0.9	1.4	0.8	0.0	0.0	0.0	0.0	3.0	5.7
W-NW	1.2	2.3	0.6	0.0	0.0	0.0	0.0	4.1	5.8
NW	2.0	1.4	1.5	0.0	0.0	0.0	0.0	4.8	5.8
N-NW	1.5	2.6	3.3	0.0	0.0	0.0	0.0	7.4	7.2
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	39.6	39.6	19.6	1.2	0.0	0.0	0.0	100.0	5.4

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-245
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – MARCH

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.8	1.4	1.6	0.0	0.0	0.0	0.0	5.8	5.1
N-NE	5.3	1.6	0.1	0.0	0.0	0.0	0.0	7.0	2.9
NE	7.6	3.2	0.3	0.0	0.0	0.0	0.0	11.1	3.0
E-NE	5.0	0.0	0.0	0.0	0.0	0.0	0.0	5.0	1.7
E	1.9	0.1	0.0	0.0	0.0	0.0	0.0	2.0	1.5
E-SE	1.5	0.3	0.0	0.0	0.0	0.0	0.0	1.8	2.4
SE	2.0	0.4	0.1	0.0	0.0	0.0	0.0	2.6	2.9
S-SE	1.8	3.9	0.1	0.0	0.0	0.0	0.0	5.8	4.6
S	3.2	5.5	1.6	0.0	0.0	0.0	0.0	10.4	5.3
S-SW	5.9	8.0	1.8	0.1	0.0	0.0	0.0	15.8	5.0
SW	4.2	4.6	3.1	0.4	0.0	0.0	0.0	12.3	5.9
W-SW	3.0	1.1	0.8	0.5	0.1	0.0	0.0	5.5	5.9
W	1.5	1.1	0.3	0.0	0.0	0.0	0.0	2.8	4.1
W-NW	0.8	1.2	0.3	0.0	0.0	0.0	0.0	2.3	5.2
NW	1.9	1.8	0.3	0.0	0.0	0.0	0.0	3.9	4.6
N-NW	2.7	0.8	2.3	0.0	0.0	0.0	0.0	5.8	5.7
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	51.1	35.0	12.7	1.1	0.1	0.0	0.0	100.0	4.5

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-246
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – APRIL

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	2.1	3.4	1.8	0.0	0.0	0.0	0.0	7.3	6.0
N-NE	3.1	2.5	0.6	0.0	0.0	0.0	0.0	6.2	4.2
NE	4.3	2.8	0.1	0.0	0.0	0.0	0.0	7.3	3.7
E-NE	2.7	0.3	0.0	0.0	0.0	0.0	0.0	2.9	2.2
E	1.4	0.3	0.0	0.0	0.0	0.0	0.0	1.7	2.4
E-SE	1.4	0.3	0.0	0.0	0.0	0.0	0.0	1.7	2.0
SE	1.7	1.7	0.8	0.0	0.0	0.0	0.0	4.2	4.8
S-SE	2.2	1.7	0.8	0.0	0.0	0.0	0.0	4.8	4.5
S	4.8	3.8	1.3	0.0	0.0	0.0	0.0	9.8	4.4
S-SW	7.7	11.8	6.0	0.1	0.0	0.0	0.0	25.6	5.7
SW	4.9	4.5	3.8	0.1	0.0	0.0	0.0	13.3	5.7
W-SW	2.0	1.5	1.5	0.1	0.1	0.0	0.0	5.3	6.1
W	0.4	0.7	0.7	0.1	0.0	0.0	0.0	2.0	7.1
W-NW	1.1	0.6	0.4	0.1	0.0	0.0	0.0	2.2	5.5
NW	0.7	0.6	0.1	0.0	0.0	0.0	0.0	1.4	4.5
N-NW	1.3	2.1	1.0	0.0	0.0	0.0	0.0	4.3	6.1
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	41.7	38.4	19.0	0.7	0.1	0.0	0.0	100.0	5.1

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-247
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – MAY

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	3.8	0.7	0.0	0.0	0.0	0.0	0.0	4.4	3.1
N-NE	6.8	2.8	0.0	0.0	0.0	0.0	0.0	9.7	3.5
NE	5.8	3.8	0.0	0.0	0.0	0.0	0.0	9.5	3.7
E-NE	3.5	1.5	0.1	0.0	0.0	0.0	0.0	5.1	3.1
E	2.0	0.4	0.0	0.0	0.0	0.0	0.0	2.4	2.5
E-SE	1.9	0.0	0.0	0.0	0.0	0.0	0.0	1.9	1.5
SE	1.2	0.4	0.0	0.0	0.0	0.0	0.0	1.6	2.9
S-SE	3.1	0.5	0.1	0.0	0.0	0.0	0.0	3.8	2.7
S	4.8	2.6	0.3	0.0	0.0	0.0	0.0	7.7	3.4
S-SW	9.4	6.8	1.6	0.0	0.0	0.0	0.0	17.9	4.2
SW	6.0	6.7	1.6	0.0	0.0	0.0	0.0	14.4	4.7
W-SW	3.1	3.1	3.6	0.0	0.0	0.0	0.0	9.8	6.7
W	1.7	3.0	1.5	0.0	0.0	0.0	0.0	6.2	5.9
W-NW	0.9	0.8	0.7	0.0	0.0	0.0	0.0	2.4	5.8
NW	1.5	0.3	0.0	0.0	0.0	0.0	0.0	1.7	3.2
N-NW	1.2	0.4	0.0	0.0	0.0	0.0	0.0	1.6	3.5
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	56.8	33.7	9.5	0.0	0.0	0.0	0.0	100.0	4.2

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-248
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – JUNE

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	5.2	2.5	0.7	0.0	0.0	0.0	0.0	8.4	3.9
N-NE	9.7	4.6	0.3	0.0	0.0	0.0	0.0	14.6	3.7
NE	11.3	5.8	1.3	0.0	0.0	0.0	0.0	18.4	4.0
E-NE	3.5	0.8	0.1	0.0	0.0	0.0	0.0	4.5	3.2
E	3.3	0.7	0.0	0.0	0.0	0.0	0.0	4.0	2.2
E-SE	2.6	0.1	0.0	0.0	0.0	0.0	0.0	2.8	1.6
SE	1.8	0.4	0.0	0.0	0.0	0.0	0.0	2.2	2.0
S-SE	3.1	0.8	0.1	0.0	0.0	0.0	0.0	4.0	2.2
S	5.0	1.7	0.0	0.0	0.0	0.0	0.0	6.7	2.6
S-SW	7.4	2.2	0.3	0.0	0.0	0.0	0.0	9.9	2.8
SW	3.6	1.3	0.0	0.0	0.0	0.0	0.0	4.9	2.9
W-SW	1.7	1.1	0.0	0.0	0.0	0.0	0.0	2.8	3.2
W	1.5	1.8	0.3	0.0	0.0	0.0	0.0	3.6	4.5
W-NW	0.8	2.5	0.0	0.0	0.0	0.0	0.0	3.3	5.0
NW	2.6	1.3	0.0	0.0	0.0	0.0	0.0	3.9	3.4
N-NW	2.6	2.5	0.8	0.0	0.0	0.0	0.0	6.0	4.9
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	65.9	30.2	3.9	0.0	0.0	0.0	0.0	100.0	3.5

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-249
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – JULY

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	5.4	2.0	0.0	0.0	0.0	0.0	0.0	7.4	3.1
N-NE	10.4	4.2	0.1	0.0	0.0	0.0	0.0	14.7	3.4
NE	9.8	6.1	1.3	0.0	0.0	0.0	0.0	17.3	4.0
E-NE	4.0	1.5	0.0	0.0	0.0	0.0	0.0	5.5	3.1
E	1.3	0.3	0.0	0.0	0.0	0.0	0.0	1.6	2.1
E-SE	1.6	0.7	0.1	0.0	0.0	0.0	0.0	2.4	3.5
SE	1.1	0.1	0.0	0.0	0.0	0.0	0.0	1.2	2.0
S-SE	1.9	0.5	0.0	0.0	0.0	0.0	0.0	2.4	2.7
S	3.4	1.3	0.0	0.0	0.0	0.0	0.0	4.7	2.9
S-SW	5.5	3.2	0.8	0.0	0.0	0.0	0.0	9.6	3.8
SW	6.9	4.3	1.8	0.0	0.0	0.0	0.0	12.9	4.5
W-SW	3.6	3.2	0.4	0.0	0.0	0.0	0.0	7.3	4.4
W	2.6	1.1	0.0	0.0	0.0	0.0	0.0	3.6	3.5
W-NW	1.3	1.8	0.0	0.0	0.0	0.0	0.0	3.1	4.0
NW	3.0	0.5	0.0	0.0	0.0	0.0	0.0	3.5	2.8
N-NW	1.6	1.1	0.0	0.0	0.0	0.0	0.0	2.7	3.4
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	63.5	31.9	4.6	0.0	0.0	0.0	0.0	100.0	3.6

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-250
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – AUGUST

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	4.4	1.9	0.0	0.0	0.0	0.0	0.0	6.3	3.3
N-NE	9.4	2.8	0.0	0.0	0.0	0.0	0.0	12.2	3.1
NE	8.6	5.0	0.1	0.1	0.0	0.0	0.0	13.8	3.7
E-NE	3.4	1.1	0.0	0.0	0.0	0.0	0.0	4.4	2.7
E	1.7	0.5	0.0	0.0	0.0	0.0	0.0	2.3	2.2
E-SE	3.4	0.5	0.0	0.0	0.0	0.0	0.0	3.9	2.4
SE	2.4	1.1	0.0	0.0	0.0	0.0	0.0	3.5	3.1
S-SE	4.6	1.3	0.0	0.0	0.0	0.0	0.0	5.9	2.9
S	3.4	1.6	0.0	0.0	0.0	0.0	0.0	5.0	2.8
S-SW	7.8	2.8	0.0	0.0	0.0	0.0	0.0	10.6	2.8
SW	6.3	2.6	0.3	0.0	0.0	0.0	0.0	9.1	3.3
W-SW	4.4	2.3	0.0	0.0	0.0	0.0	0.0	6.7	3.2
W	3.9	3.8	0.0	0.0	0.0	0.0	0.0	7.7	4.0
W-NW	1.7	1.6	0.0	0.0	0.0	0.0	0.0	3.4	3.9
NW	1.7	0.4	0.1	0.0	0.0	0.0	0.0	2.3	3.2
N-NW	1.7	1.1	0.0	0.0	0.0	0.0	0.0	2.8	3.6
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	69.0	30.4	0.5	0.1	0.0	0.0	0.0	100.0	3.2

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-251
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – SEPTEMBER

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	6.8	3.5	0.0	0.0	0.0	0.0	0.0	10.3	3.4
N-NE	12.4	7.4	0.0	0.0	0.0	0.0	0.0	19.8	3.6
NE	9.4	6.1	0.0	0.0	0.0	0.0	0.0	15.5	3.5
E-NE	3.1	1.1	0.0	0.0	0.0	0.0	0.0	4.2	2.9
E	1.4	0.3	0.0	0.0	0.0	0.0	0.0	1.7	1.9
E-SE	2.1	1.0	0.0	0.0	0.0	0.0	0.0	3.1	3.4
SE	1.8	3.4	0.0	0.0	0.0	0.0	0.0	5.2	4.0
S-SE	2.7	2.1	0.0	0.0	0.0	0.0	0.0	4.7	3.8
S	4.3	1.5	0.3	0.0	0.0	0.0	0.0	6.1	3.3
S-SW	4.9	4.1	2.1	0.0	0.0	0.0	0.0	11.0	4.9
SW	2.5	1.7	0.7	0.0	0.0	0.0	0.0	4.9	4.4
W-SW	2.1	0.8	0.4	0.0	0.0	0.0	0.0	3.4	4.1
W	1.8	1.0	0.3	0.0	0.0	0.0	0.0	3.1	4.3
W-NW	1.1	0.0	0.0	0.0	0.0	0.0	0.0	1.1	2.9
NW	1.1	1.0	0.1	0.0	0.0	0.0	0.0	2.2	4.3
N-NW	1.8	1.7	0.1	0.0	0.0	0.0	0.0	3.6	4.2
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	59.4	36.6	4.1	0.0	0.0	0.0	0.0	100.0	3.8

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-252
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – OCTOBER

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	5.1	2.7	0.1	0.0	0.0	0.0	0.0	8.0	3.7
N-NE	11.5	6.8	0.3	0.0	0.0	0.0	0.0	18.5	3.6
NE	9.7	3.8	0.3	0.0	0.0	0.0	0.0	13.8	3.2
E-NE	4.3	0.5	0.0	0.0	0.0	0.0	0.0	4.9	2.1
E	2.8	0.1	0.0	0.0	0.0	0.0	0.0	3.0	1.3
E-SE	2.8	0.3	0.0	0.0	0.0	0.0	0.0	3.1	1.6
SE	1.4	0.4	0.1	0.5	0.0	0.0	0.0	2.4	6.1
S-SE	3.1	0.5	0.4	0.0	0.0	0.0	0.0	4.1	3.0
S	4.1	1.9	0.3	0.0	0.0	0.0	0.0	6.2	3.1
S-SW	4.7	2.7	0.7	0.0	0.0	0.0	0.0	8.1	3.8
SW	2.6	2.6	0.4	0.0	0.0	0.0	0.0	5.5	4.4
W-SW	2.6	1.6	0.3	0.0	0.0	0.0	0.0	4.5	3.8
W	1.9	1.5	0.5	0.0	0.0	0.0	0.0	3.9	4.7
W-NW	1.4	2.6	0.7	0.0	0.0	0.0	0.0	4.6	5.6
NW	2.0	2.0	0.5	0.0	0.0	0.0	0.0	4.6	4.6
N-NW	2.8	1.4	0.5	0.0	0.0	0.0	0.0	4.7	4.0
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	62.9	31.4	5.1	0.5	0.0	0.0	0.0	100.0	3.6

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-253
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – NOVEMBER

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	5.6	5.6	3.2	0.0	0.0	0.0	0.0	14.3	5.4
N-NE	9.9	3.3	0.3	0.0	0.0	0.0	0.0	13.5	3.1
NE	11.8	3.6	0.6	0.0	0.0	0.0	0.0	16.0	3.3
E-NE	4.9	0.7	0.0	0.0	0.0	0.0	0.0	5.6	2.4
E	1.3	0.3	0.0	0.0	0.0	0.0	0.0	1.5	2.7
E-SE	0.4	0.6	0.6	0.0	0.0	0.0	0.0	1.5	5.9
SE	1.4	1.7	0.6	0.4	0.0	0.0	0.0	4.0	6.6
S-SE	1.9	2.4	1.1	0.0	0.0	0.0	0.0	5.4	5.5
S	1.9	1.4	1.0	0.4	0.0	0.0	0.0	4.7	6.1
S-SW	4.3	2.2	0.7	0.1	0.0	0.0	0.0	7.4	4.2
SW	3.3	1.9	1.1	0.1	0.0	0.0	0.0	6.5	5.2
W-SW	1.7	0.7	0.3	0.0	0.0	0.0	0.0	2.6	4.5
W	2.8	0.8	0.0	0.0	0.0	0.0	0.0	3.6	2.9
W-NW	2.6	0.3	0.0	0.0	0.0	0.0	0.0	2.9	2.4
NW	2.4	0.3	0.1	0.0	0.0	0.0	0.0	2.8	2.9
N-NW	2.6	2.8	1.9	0.0	0.0	0.0	0.0	7.4	5.5
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	58.9	28.6	11.4	1.1	0.0	0.0	0.0	100.0	4.3

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-254
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – DECEMBER

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	5.1	3.4	1.3	0.0	0.0	0.0	0.0	9.8	4.5
N-NE	9.3	6.3	0.3	0.0	0.0	0.0	0.0	15.9	3.5
NE	9.6	5.7	0.1	0.0	0.0	0.0	0.0	15.3	3.4
E-NE	3.9	0.8	0.0	0.0	0.0	0.0	0.0	4.7	2.6
E	0.9	0.8	0.1	0.0	0.0	0.0	0.0	1.9	4.0
E-SE	1.2	0.4	0.0	0.0	0.0	0.0	0.0	1.6	2.9
SE	1.1	1.1	0.8	0.0	0.0	0.0	0.0	3.0	5.3
S-SE	1.3	2.2	0.3	0.0	0.0	0.0	0.0	3.8	4.8
S	3.8	2.0	0.9	0.1	0.0	0.0	0.0	6.9	4.4
S-SW	4.3	2.3	0.9	0.4	0.0	0.0	0.0	7.9	4.7
SW	6.9	2.2	0.8	0.4	0.0	0.0	0.0	10.2	3.7
W-SW	2.0	0.8	0.5	0.5	0.0	0.0	0.0	3.9	5.9
W	2.3	0.9	0.1	0.0	0.0	0.0	0.0	3.4	3.0
W-NW	0.7	1.2	0.0	0.0	0.0	0.0	0.0	1.9	4.9
NW	3.2	0.5	0.1	0.0	0.0	0.0	0.0	3.9	3.1
N-NW	4.2	0.7	0.9	0.1	0.0	0.0	0.0	5.9	4.0
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	59.8	31.2	7.4	1.6	0.0	0.0	0.0	100.0	3.9

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-255
PERCENTAGE FREQUENCY OF WIND DIRECTION AND SPEED – BLN SITE – 2006 – 2007 – ALL MONTHS

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	4.5	3.1	1.1	0.0	0.0	0.0	0.0	8.7	4.5
N-NE	8.2	4.3	0.3	0.0	0.0	0.0	0.0	12.9	3.6
NE	8.4	4.3	0.4	0.0	0.0	0.0	0.0	13.1	3.6
E-NE	3.5	0.8	0.0	0.0	0.0	0.0	0.0	4.4	2.7
E	1.7	0.3	0.0	0.0	0.0	0.0	0.0	2.0	2.2
E-SE	1.7	0.3	0.1	0.0	0.0	0.0	0.0	2.1	2.6
SE	1.5	0.9	0.3	0.1	0.0	0.0	0.0	2.7	4.3
S-SE	2.4	1.5	0.3	0.0	0.0	0.0	0.0	4.1	3.8
S	3.6	2.5	0.6	0.1	0.0	0.0	0.0	6.7	4.1
S-SW	6.0	5.1	1.8	0.1	0.0	0.0	0.0	13.0	4.7
SW	5.0	3.3	1.3	0.1	0.0	0.0	0.0	9.8	4.6
W-SW	2.6	1.6	0.8	0.1	0.0	0.0	0.0	5.2	4.9
W	1.9	1.5	0.4	0.0	0.0	0.0	0.0	3.9	4.6
W-NW	1.3	1.4	0.3	0.0	0.0	0.0	0.0	3.0	4.9
NW	2.0	1.0	0.4	0.0	0.0	0.0	0.0	3.4	4.3
N-NW	2.2	1.7	1.0	0.0	0.0	0.0	0.0	5.0	5.1
CALM	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	56.5	33.7	9.2	0.6	0.0	0.0	0.0	100.0	4.1

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-256
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM A SINGLE SECTOR – HUNTSVILLE, ALABAMA – 2001 – 2005

Sector	2001	2002	2003	2004	2005	Maximum	Avg.
N	18	15	16	17	19	19	17.0
NNE	7	10	8	9	7	10	8.2
NE	4	6	5	9	9	9	6.6
ENE	3	4	3	4	5	5	3.8
E	8	8	8	11	10	11	9.0
ESE	19	12	13	11	7	19	12.4
SE	6	10	10	9	11	11	9.2
SSE	11	13	9	9	13	13	11.0
S	9	10	11	9	16	16	11.0
SSW	5	10	11	11	6	11	8.6
SW	5	9	11	6	8	11	7.8
WSW	6	9	5	5	7	9	6.4
W	11	8	7	6	11	11	8.6
WNW	7	11	9	19	11	19	11.4
NW	6	8	7	9	9	9	7.8
NNW	8	10	7	8	9	10	8.4

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-257
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM 3 ADJACENT SECTORS – HUNTSVILLE, ALABAMA – 2001 – 2005

Sector	2001	2002	2003	2004	2005	Maximum	Avg.
N	43	36	41	48	53	53	44.2
NNE	40	50	65	31	42	65	45.6
NE	10	24	12	27	52	52	25.0
ENE	10	31	11	16	17	31	17.0
E	30	36	35	30	41	41	34.4
ESE	53	50	50	50	37	53	48.0
SE	32	48	51	47	45	51	44.6
SSE	28	40	29	31	36	40	32.8
S	30	44	43	39	23	44	35.8
SSW	28	33	36	39	29	39	33.0
SW	15	22	30	31	23	31	24.2
WSW	17	25	15	17	31	31	21.0
W	21	18	20	30	20	30	21.8
WNW	32	42	32	39	33	42	35.6
NW	38	26	28	36	33	38	32.2
NNW	37	38	52	37	30	52	38.8

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-258
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM 5 ADJACENT SECTORS – HUNTSVILLE, ALABAMA – 2001 – 2005

Sector	2001	2002	2003	2004	2005	Maximum	Avg.
N	53	67	83	103	82	103	77.6
NNE	52	62	71	63	105	105	70.6
NE	40	92	65	51	78	92	65.2
ENE	30	36	35	54	52	54	41.4
E	53	53	52	100	52	100	62.0
ESE	87	52	52	76	88	88	71.0
SE	102	71	68	70	89	102	80.0
SSE	53	58	66	72	61	72	62.0
S	64	64	69	66	55	69	63.6
SSW	30	61	108	68	51	108	63.6
SW	39	41	68	55	43	68	49.2
WSW	28	29	41	36	47	47	36.2
W	32	42	35	43	35	43	37.4
WNW	40	42	36	47	38	47	40.6
NW	82	55	78	91	61	91	73.4
NNW	75	57	83	84	65	84	72.8

NOTES:

1. Period of Record - 5 years (2001 –- 2005)

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-259
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM A SINGLE SECTOR - BLN SITE

Sector	1979	1980	1981	1982	2006-2007	Maximum	Avg.
N	10	17	9	17	12	17	13.0
NNE	14	13	14	14	8	14	12.6
NE	14	11	11	8	9	14	10.6
ENE	6	5	5	6	4	6	5.2
E	8	6	3	3	3	8	4.6
ESE	2	4	4	3	4	4	3.4
SE	22	8	8	9	10	22	11.4
SSE	7	5	5	7	7	7	6.2
S	9	8	17	12	6	17	10.4
SSW	12	19	19	11	10	19	14.2
SW	11	22	8	8	8	22	11.4
WSW	16	7	7	7	8	16	9.0
W	6	5	5	7	5	7	5.6
WNW	6	5	7	3	7	7	5.6
NW	11	8	8	9	10	11	9.2
NNW	7	14	9	12	9	14	10.2

NOTES:

1. Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-260
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM 3 ADJACENT SECTORS - BLN SITE

Sector	1979	1980	1981	1982	2006-2007	Maximum	Avg.
N	26	44	29	39	41	44	35.8
NNE	35	50	30	48	36	50	39.8
NE	36	35	34	33	40	40	35.6
ENE	32	12	14	13	12	32	16.6
E	19	8	7	12	6	19	10.4
ESE	32	17	14	15	16	32	18.8
SE	31	18	14	30	20	31	22.6
SSE	29	25	36	47	20	47	31.4
S	24	23	22	30	33	33	26.4
SSW	30	45	55	38	72	72	48.0
SW	34	39	38	29	44	44	36.8
WSW	33	32	25	20	17	33	25.4
W	30	13	24	14	15	30	19.2
WNW	35	25	15	14	14	35	20.6
NW	23	30	17	12	18	30	20.0
NNW	24	44	41	37	30	44	35.2

NOTES:

1. Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-261
MAXIMUM NUMBER OF CONSECUTIVE HOURS WITH WIND FROM 5 ADJACENT SECTORS - BLN SITE

Sector	1979	1980	1981	1982	2006-2007	Maximum	Avg.
N	57	67	68	70	60	70	64.4
NNE	55	68	64	88	70	88	69.0
NE	68	67	45	51	79	79	62.0
ENE	73	37	34	33	40	73	43.4
E	43	27	14	19	19	43	24.4
ESE	44	18	19	30	31	44	28.4
SE	38	25	36	64	20	64	36.6
SSE	37	32	41	64	42	64	43.2
S	37	53	55	57	72	72	54.8
SSW	70	67	59	47	78	78	64.2
SW	53	67	80	47	80	80	65.4
WSW	35	60	60	34	53	60	48.4
W	61	32	48	34	27	61	40.4
WNW	46	37	37	24	31	46	35.0
NW	49	44	43	37	30	49	40.6
NNW	44	45	46	48	51	51	46.8

NOTES:

1. Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-262
MAXIMUM WIND PERSISTENCE AT BLN SITE

	Wind Persistence (hrs)		
Sector	Single Sector	Three Adjacent Sectors	Five Adjacent Sectors
N	17	44	70
N-NE	14	50	88
NE	14	40	79
E-NE	6	32	73
E	8	19	43
E-SE	4	32	44
SE	22	31	64
S-SE	7	47	64
S	17	33	72
S-SW	19	72	78
SW	22	44	80
W-SW	16	33	60
W	7	30	61
W-NW	7	35	46
NW	11	30	49
N-NW	14	44	51

NOTES:

1. Wind persistence values above are the maximum persistence durations for the period of record.

2. Period of record at BLN Site, 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-263
TEMPERATURE MEANS AND EXTREMES AT SCOTTSBORO, ALABAMA – 1971 – 2000

	MAX	MEAN	MIN	HIGHEST MEAN	MEDIAN	LOWEST MEAN	HIGHEST MEAN YEAR	LOWEST MEAN YEAR
JAN	49.8	39.1	28.3	48.5	39.5	26.8	1974	1977
FEB	54.9	43.0	31.0	49.6	43.3	35.0	1990	1980
MAR	63.8	51.1	38.3	59.3	50.6	44.8	1989	1971
APR	72.3	58.7	45.1	64.1	58.3	54.7	1981	1983
MAY	80.0	67.3	54.5	73.7	67.2	62.1	1987	1997
JUN	86.9	74.9	62.9	77.6	75.5	71.1	1998	1974
JUL	90.3	78.6	66.8	81.8	78.6	76.0	1993	1984
AUG	89.9	77.7	65.4	81.4	77.2	74.4	1983	1992
SEP	84.3	71.5	58.6	76.1	71.2	67.0	1978	1975
OCT	74.3	60.0	45.6	66.4	59.8	52.9	1984	1988
NOV	63.2	50.2	37.1	58.8	49.8	41.5	1985	1976
DEC	53.6	42.1	30.5	50.5	41.1	32.7	1971	1989
ANNUAL	71.9	59.5	47.0	81.8	59.5	26.8	1993	1977

NOTES:

1. Temperatures provided in the table above are listed in degrees Fahrenheit (°F).

(Reference 226)

BLN COL 2.3-2

TABLE 2.3-264
TEMPERATURE MEANS AND EXTREMES AT BLN SITE – 1979 – 1982

	Mean Daily Max	Mean Daily Min	Monthly Mean	Record Max	Year	Record Min	Year
Jan	44.8	29.2	36.8	64.3	1982	-3.9	1982
Feb	50.5	33.1	41.2	77.0	1980	7.9	1981
Mar	61.3	42.3	51.7	84.5	1982	12.5	1980
Apr	70.2	51.3	60.6	86.3	1980	30.4	1982
May	77.2	59.1	67.6	90.1	1982	44.9	1980
Jun	83.7	66.4	74.6	92.6	1981	53.4	1980
Jul	87.4	71.4	78.6	99.7	1980	60.8	1979
Aug	86.1	69.4	76.8	97.2	1980	59.9	1982
Sep	80.0	63.5	70.8	93.7	1980	45.9	1981
Oct	69.7	50.3	59.3	85.4	1981	33.9	1981
Nov	60.6	42.7	51.0	78.7	1982	21.6	1979
Dec	51.8	35.7	43.6	74.6	1982	12.1	1981
Annual	68.6	51.2	59.4	99.7	1980	-3.9	1982

NOTES:

1. Temperatures provided in the table above are listed in degrees Fahrenheit (°F).
2. Data from BLN Meteorological Tower, 1979 – 1982.
3. Temperature at 10 meters.

BLN COL 2.3-2

TABLE 2.3-265
TEMPERATURE MEANS AND EXTREMES AT BLN SITE – 2006 – 2007

	Mean Daily Max	Mean Daily Min	Monthly Mean
Jan	69.5	16.3	43.9
Feb	73.9	17.8	41.6
Mar	84.5	28.0	59.3
Apr	87.0	40.5	65.9
May	89.6	45.5	67.4
Jun	94.0	56.0	74.6
Jul	96.1	63.7	79.5
Aug	96.4	65.7	80.5
Sep	87.3	45.5	69.9
Oct	84.8	35.3	58.1
Nov	74.9	29.0	50.5
Dec	68.9	16.3	45.1
Annual	96.4	16.3	61.5

NOTES:

1. Temperatures provided in the table above are listed in degrees Fahrenheit (°F).
2. Bellefonte site data measured from 4/1/2006 through 3/31/2007.
3. Temperature at 10 meters.

BLN COL 2.3-2

TABLE 2.3-266
PRECIPITATION DATA AT BLN SITE – 1979 – 1982

Month	Monthly Mean	Max Monthly Precipitation	Min Monthly Precipitation	Max 24 hour	Mean No. days >0.01 in
Jan	4.7	7.8	0.9	3.6	11.0
Feb	3.9	6.3	1.9	1.9	12.0
Mar	6.7	14.5	2.6	3.8	15.5
Apr	5.5	6.4	4.7	2.2	14.3
May	4.1	7.2	1.5	2.6	12.0
Jun	2.8	5.7	1.1	1.6	8.0
Jul	2.6	7.1	0.0	2.0	13.3
Aug	3.0	5.8	0.3	3.1	10.5
Sep	3.5	6.5	1.7	3.1	9.5
Oct	2.2	2.9	1.4	1.8	9.5
Nov	5.3	7.0	3.2	4.1	11.0
Dec	3.9	7.4	1.0	2.3	9.3

Annual Average Rainfall (in) = 48.1

NOTES:

1. Precipitation data measured in inches of rain.
2. Data from Site Meteorological Tower, 1979 – 1982.

BLN COL 2.3-1

TABLE 2.3-267
PRECIPITATION DATA AT HUNTSVILLE, ALABAMA – 2001 – 2005

Month	Monthly Mean	Max Monthly Precipitation	Min Monthly Precipitation	Max 24 hour	Mean No. days >0.01 in
Jan	3.5	5.6	1.5	2.9	9.2
Feb	5.2	7.8	1.9	4.5	11.8
Mar	6.7	14.5	1.8	3.8	13.4
Apr	4.1	6.4	3.0	2.2	11.4
May	6.0	10.4	2.9	4.6	10.4
Jun	4.8	7.4	1.1	2.0	10.2
Jul	3.8	7.6	0.0	5.2	10.0
Aug	3.0	5.0	0.3	2.7	10.2
Sep	4.0	6.0	2.9	3.9	6.2
Oct	2.1	4.2	0.2	2.9	9.8
Nov	4.6	7.6	2.9	2.1	8.8
Dec	4.8	7.7	1.0	3.6	7.8
Annual Average Rainfall (in) = 52.4					

NOTES:

1. Precipitation data measured in inches of rain.

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-268
RAINFALL FREQUENCY DISTRIBUTION AT BELLEFONTE – 1979 – 1982
NUMBER OF HOURS PER MONTH, AVERAGE YEAR

Rainfall (inch/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	23.8	26.3	28.3	33.3	27.5	12.3	37.8	21.5	20.3	16.3	25.5	21.8
0.02-.099	31.3	32.3	31.3	29.5	32.8	13.8	20.8	8.3	21.5	15.5	33.5	24.0
0.10-0.249	12.8	9.3	11.3	13.3	5.3	6.5	2.0	2.3	8.8	5.0	11.5	8.8
0.25-0.499	3.0	2.3	8.3	3.5	2.3	0.8	1.0	2.5	2.5	1.8	4.0	2.0
0.50-0.99	0.3	0.0	0.8	1.0	1.0	0.8	1.5	1.0	0.3	0.3	1.5	0.3
1.00-1.99	0.0	0.0	0.0	0.0	0.3	0.3	0.0	0.5	0.0	0.0	0.0	0.0
2.0 & over	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	71.0	70.0	79.8	80.5	69.0	34.3	63.0	36.0	53.3	38.8	76.0	56.8

NOTES:

1. Data from Site Meteorological Tower, 1979 – 1982.

TABLE 2.3-269

BLN COL 2.3-2

RAINFALL FREQUENCY DISTRIBUTION AT HUNTSVILLE, AL - 2001 – 2005 - NUMBER OF HOURS PER MONTH - AVERAGE YEAR

Rainfall (inch/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	18.6	22.4	19.8	19.4	13.0	10.8	7.6	9.4	7.8	11.2	13.6	13.4
0.02-.099	27.2	42.2	33.2	21.8	26.6	19.6	13.8	12.8	17.4	9.6	28.6	25.4
0.10-0.249	11.8	11.4	13.6	9.6	12.6	9.2	4.0	6.0	8.2	4.2	9.4	12.6
0.25-0.499	0.6	4.4	6.6	3.2	4.2	3.4	2.8	1.8	2.2	1.4	4.0	3.6
0.50-0.99	0.2	0.2	0.6	0.6	1.8	1.4	1.2	0.8	1.6	0.4	0.6	0.4
1.00-1.99	0.0	0.0	0.2	0.0	0.2	0.4	0.4	0.2	0.2	0.2	0.0	0.0
2.0 & over	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0
Total	58.4	80.6	74.0	54.6	58.4	44.8	30.0	31.0	37.4	27.0	56.2	55.4

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-270
ESTIMATED MAXIMUM POINT PRECIPITATION AMOUNTS FOR SELECTED DURATIONS AND RECURRENCE INTERVALS – BLN SITE

	Recurrence Intervals (Yr.)						
Duration	1	2	5	10	25	50	100
30 minutes	1.2	1.4	1.8	2.0	2.2	2.4	3.0
1 hour	1.4	1.8	2.2	2.4	2.8	3.0	3.5
2 hours	1.8	2.2	3.0	3.0	3.5	4.0	4.5
3 hours	2.0	2.5	3.0	3.5	4.0	4.5	4.5
6 hours	2.5	3.0	3.5	4.0	5.0	5.0	6.0
12 hours	3.0	3.5	4.5	5.0	6.0	6.0	7.0
24 hours	3.5	4.0	5.0	6.0	7.0	7.0	8.0
2 days	-	4.5	6.0	7.0	8.0	8.0	9.0
4 days	-	6.0	7.0	8.0	9.0	12.0	12.0
7 days	-	6.0	8.0	9.0	12.0	12.0	14.0
10 days	-	6.5	9.0	10.0	12.0	14.0	16.0

NOTES:

1. Precipitation values provided in inches of rainfall.

(References 228 and 229)

BLN COL 2.3-2

TABLE 2.3-271
ESTIMATED MAXIMUM POINT PRECIPITATION AMOUNTS FOR SELECTED DURATIONS AND RECURRENCE INTERVALS – BLN SITE

	Recurrence Intervals (Yr.)	
Duration	2	100
5 minutes	0.475	0.85
15 minutes	1.0	1.8
60 minutes	1.7	3.5

NOTES:

1. Precipitation values provided in inches of rainfall.

(Reference 230)

BLN COL 2.3-2

TABLE 2.3-272 (Sheet 1 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – BLN SITE – 1979 – 1982

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
N	0.69	0.80	1.04	0.52	0.42	0.35	0.45	0.42	0.73	0.62	1.15	0.90	8.09
N-NE	0.94	1.87	0.83	1.28	0.97	0.21	0.87	0.42	1.42	1.39	1.63	1.01	12.84
NE	1.04	1.56	0.97	1.53	0.97	0.52	1.18	0.80	0.97	0.56	0.94	0.45	11.49
E-NE	0.31	0.21	0.59	0.31	0.35	0.14	0.45	0.31	0.52	0.10	0.38	0.07	3.75
E	0.14	0.07	0.35	0.14	0.14	0.07	0.28	0.03	0.24	0.00	0.10	0.07	1.63
E-SE	0.14	0.07	0.21	0.31	0.21	0.03	0.10	0.07	0.21	0.03	0.03	0.14	1.56
SE	0.62	0.56	1.01	0.66	0.83	0.21	0.56	0.17	0.38	0.10	0.69	0.52	6.32
S-SE	0.35	0.14	0.49	0.24	0.42	0.10	0.24	0.17	0.31	0.03	0.38	0.45	3.33
S	0.69	0.76	1.08	1.35	1.15	0.69	0.83	0.35	0.35	0.38	1.08	0.83	9.55
S-SW	1.46	0.76	0.94	1.42	1.49	0.90	1.28	0.69	0.52	0.59	0.59	1.28	11.94
SW	1.21	0.83	1.39	1.35	0.62	0.76	0.94	0.31	0.59	0.38	0.97	0.87	10.24
W-SW	0.69	0.35	0.73	0.73	0.52	0.35	0.42	0.28	0.35	0.42	0.59	0.45	5.87
W	0.21	0.49	0.24	0.17	0.17	0.24	0.17	0.17	0.24	0.21	0.31	0.42	3.05

BLN COL 2.3-2

TABLE 2.3-272 (Sheet 2 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – BLN SITE – 1979 – 1982

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
W-NW	0.24	0.28	0.31	0.24	0.07	0.14	0.28	0.07	0.14	0.14	0.35	0.31	2.57
NW	0.42	0.31	0.35	0.28	0.17	0.14	0.10	0.21	0.21	0.10	0.24	0.21	2.74
N-NW	0.69	0.49	0.69	0.59	0.35	0.03	0.31	0.28	0.42	0.28	0.59	0.35	5.07
Total	9.86	9.55	11.21	11.14	8.85	4.89	8.47	4.76	7.60	5.35	10.03	8.33	100

NOTES:

1. BLN Site data 1979 – 1982

BLN COL 2.3-2

TABLE 2.3-273 (Sheet 1 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – HUNTSVILLE, ALABAMA - 2001 – 2005

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
N	1.32	1.45	1.52	1.81	0.94	1.19	1.19	1.19	0.94	0.74	1.13	0.55	13.98
N-NE	0.48	0.52	0.90	0.45	0.32	0.26	0.13	0.06	0.23	0.13	0.13	0.32	3.94
NE	0.23	1.16	0.58	0.29	0.36	0.26	0.13	0.10	0.68	0.16	0.26	0.36	4.55
E-NE	0.26	1.00	0.29	0.19	0.16	0.16	0.26	0.16	0.81	0.29	0.39	0.45	4.42
E	0.36	1.78	0.84	0.32	0.81	0.68	0.48	0.26	0.87	0.23	0.68	0.81	8.10
E-SE	1.29	2.29	0.65	0.42	1.00	0.74	0.23	0.39	0.58	0.71	1.36	0.87	10.53
SE	0.90	1.13	0.52	0.52	1.23	0.32	0.42	0.36	0.36	0.71	1.26	1.61	9.33
S-SE	0.61	0.42	0.36	0.39	0.48	0.61	0.52	0.61	0.61	0.81	1.32	1.97	8.72
S	0.52	0.32	0.84	0.58	0.84	0.94	0.74	0.29	0.65	0.74	1.49	1.10	9.04
S-SW	0.39	0.23	0.90	0.16	0.42	0.68	0.32	0.42	0.19	0.06	0.58	0.36	4.71
SW	0.29	0.19	0.26	0.19	0.58	0.48	0.29	0.23	0.06	0.23	0.13	0.48	3.42
W-SW	0.19	0.16	0.23	0.23	0.16	0.55	0.16	0.29	0.06	0.26	0.32	0.32	2.94
W	0.55	0.45	0.29	0.45	0.32	0.48	0.32	0.13	0.03	0.29	0.52	0.36	4.20

BLN COL 2.3-2

TABLE 2.3-273 (Sheet 2 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – HUNTSVILLE, ALABAMA - 2001 – 2005

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
W-NW	0.29	0.61	0.61	0.55	0.19	0.13	0.39	0.29	0.06	0.29	0.39	0.52	4.33
NW	0.39	0.42	0.42	0.45	0.58	0.45	0.29	0.19	0.39	0.06	0.29	0.42	4.36
N-NW	0.65	0.19	0.58	0.29	0.39	0.36	0.32	0.10	0.16	0.16	0.16	0.06	3.42
Total	8.72	12.33	9.78	7.30	8.78	8.30	6.20	5.07	6.68	5.88	10.40	10.56	100

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-274 (Sheet 1 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – BLN SITE - 2006 – 2007

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
N	0.00	1.27	0.51	0.25	1.02	0.51	0.51	0.51	0.00	0.51	0.51	0.51	6.11
N-NE	0.00	0.76	0.51	1.02	1.02	0.25	0.51	0.51	0.00	2.54	2.04	1.27	10.43
NE	0.00	0.00	0.51	0.00	0.51	0.51	0.76	0.76	0.25	1.27	1.27	0.51	6.36
E-NE	0.00	0.00	0.76	0.25	1.27	0.25	0.51	0.51	0.00	0.51	0.76	0.25	5.09
E	0.00	0.00	0.00	0.25	0.51	0.76	0.00	0.25	0.00	0.25	0.25	0.00	2.29
E-SE	0.00	0.00	0.00	0.76	0.51	0.00	0.00	0.00	0.00	0.76	0.25	0.25	2.54
SE	0.51	0.00	0.00	0.51	0.00	0.00	0.25	0.25	0.00	1.53	1.27	1.53	5.85
S-SE	0.76	0.00	0.00	0.51	0.76	0.25	0.25	0.00	0.00	1.27	0.00	1.53	5.34
S	1.27	0.51	0.76	1.27	0.51	0.51	0.25	0.25	0.00	0.76	0.76	1.02	7.89
S-SW	3.05	1.78	1.02	2.54	1.53	1.27	0.76	1.27	0.00	1.02	1.27	0.51	16.03
SW	1.78	0.51	0.76	0.25	0.76	0.00	1.02	1.27	0.00	1.27	0.76	2.04	10.43
W-SW	0.25	0.00	0.51	1.02	0.76	0.25	0.25	0.25	0.00	1.27	0.76	0.51	5.85
W	0.51	0.25	0.51	0.00	0.76	0.25	0.51	0.25	0.00	1.53	0.25	0.51	5.34
W-NW	0.25	0.25	0.00	0.25	0.25	0.51	0.76	0.00	0.00	0.51	0.76	0.00	3.56

BLN COL 2.3-2

TABLE 2.3-274 (Sheet 2 of 2)
PERCENT OF TOTAL OBSERVATIONS (BY MONTH) OF INDICATED WIND DIRECTIONS AND PRECIPITATION – BLN SITE - 2006 – 2007

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Sum
NW	0.00	0.25	0.00	0.00	0.51	0.25	0.51	0.51	0.25	0.76	0.25	0.25	3.56
N-NW	0.25	0.25	0.51	0.25	0.76	0.00	0.51	0.00	0.00	0.25	0.25	0.25	3.31
Total	8.65	5.85	6.36	9.16	11.45	5.60	7.38	6.62	0.51	16.03	11.45	10.94	100

NOTES:

1. BLN Site data 4/1/2006 – 3/31/2007

BLN COL 2.3-2

TABLE 2.3-275
AVERAGE HOURS OF FOG AND HAZE AT HUNTSVILLE, ALABAMA - 2001 – 2005

	Fog (hours/month)			Haze (hours/month)		
Month	Average	Maximum	Minimum	Average	Maximum	Minimum
Jan	2.6	6.0	0.6	0.6	1.8	0.0
Feb	3.9	8.0	0.1	1.2	2.0	0.5
Mar	2.1	3.8	1.0	1.9	4.6	0.0
Apr	0.5	1.7	0.0	1.4	4.0	0.0
May	2.6	5.4	0.1	4.0	9.1	0.3
Jun	2.3	4.2	0.5	5.0	9.5	0.0
Jul	4.2	8.5	1.4	5.5	8.3	2.5
Aug	5.2	11.1	0.3	9.3	15.2	1.1
Sep	1.7	5.1	0.0	4.5	6.5	0.6
Oct	4.7	10.3	0.0	4.2	8.3	0.0
Nov	3.9	8.0	1.4	0.4	0.9	0.0
Dec	3.3	8.6	0.0	0.3	0.8	0.0
Annual (hours/year)	36.8	58.2	25.5	38.2	48.1	26.2

1. Period of Record – 5 years (2001 – 2005)

(Reference 227)

BLN COL 2.3-2

TABLE 2.3-276
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – JANUARY

January	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	13	562	0.392	8	994	0.258
2001	16	655	0.353	11	987	0.272
2002	15	974	0.257	4	1046	0.185
2003	16	840	0.222	12	1184	0.247
2004	16	739	0.291	6	1132	0.348
2005	11	892	0.248	10	1350	0.496
Total	87	775	0.293	51	1127	0.310

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-277
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – FEBRUARY

February	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	12	545	0.490	9	1397	0.313
2001	16	863	0.374	5	1640	0.268
2002	13	817	0.329	7	1338	0.239
2003	10	746	0.317	8	1103	0.395
2004	10	807	0.396	6	1137	0.501
2005	10	859	0.393	5	943	0.245
Total	71	776	0.383	40	1263	0.331

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-278
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – MARCH

March	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	9	474	0.610	1	1341	0.240
2001	7	1026	0.281	3	1421	0.229
2002	15	717	0.283	8	1516	0.359
2003	9	1096	0.400	4	2004	0.376
2004	11	623	0.327	2	1530	0.396
2005	3	1320	0.243	2	1154	0.412
Total	54	794	0.363	20	1556	0.346

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-279
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – APRIL

April	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	9	359	0.865	3	1513	0.228
2001	5	478	0.449	3	2283	0.289
2002	12	716	0.456	2	1729	0.225
2003	8	1039	0.439	1	449	0.312
2004	7	283	0.417	0	N/A	N/A
2005	5	185	0.586	0	N/A	N/A
Total	46	553	0.541	9	1699	0.257

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-280
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – MAY

May	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	6	920	0.371	2	1476	0.397
2001	4	580	0.364	0	N/A	N/A
2002	8	1105	0.315	3	1355	0.271
2003	3	1723	0.335	0	N/A	N/A
2004	4	815	0.348	2	2178	0.121
2005	10	482	0.430	1	1853	0.212
Total	35	855	0.369	8	1653	0.258

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-281
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – JUNE

June	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	4	859	0.345	4	1877	0.259
2001	6	593	0.340	0	N/A	N/A
2002	2	180	0.430	0	N/A	N/A
2003	6	1376	0.209	3	1680	0.241
2004	4	1243	0.147	0	N/A	N/A
2005	2	180	0.509	0	N/A	N/A
Total	24	873	0.298	7	1793	0.251

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-282
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – JULY

July	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	4	1239	0.295	1	180	0.400
2001	2	849	0.261	1	2420	0.476
2002	0	N/A	N/A	0	N/A	N/A
2003	2	180	0.356	2	1371	0.243
2004	1	180	0.233	3	1207	0.352
2005	1	1177	0.333	1	2052	0.115
Total	10	837	0.298	8	1377	0.317

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-283
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – AUGUST

August	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	1	180	0.353	1	1658	0.520
2001	2	180	0.368	1	2204	0.476
2002	3	180	0.326	0	N/A	N/A
2003	1	180	0.226	0	N/A	N/A
2004	4	1082	0.363	2	2088	0.278
2005	2	180	0.390	0	N/A	N/A
Total	13	458	0.348	4	2009	0.388

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-284
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – SEPTEMBER

September	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	5	933	0.417	2	1354	0.112
2001	11	937	0.433	2	2198	0.442
2002	3	632	0.299	0	N/A	N/A
2003	11	918	0.384	2	2088	0.418
2004	6	767	0.242	2	1560	0.285
2005	10	1401	0.224	2	1834	0.692
Total	46	991	0.340	10	1806	0.390

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-285
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – OCTOBER

October	Mornings with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)	Afternoons with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)
2000	14	533	0.501	3	1815	0.311
2001	15	780	0.369	3	1940	0.245
2002	8	971	0.464	3	1262	0.272
2003	8	524	0.369	2	2430	0.343
2004	8	588	0.368	0	N/A	N/A
2005	13	837	0.359	2	1316	1.047
Total	66	708	0.406	13	1734	0.405

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-286
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – NOVEMBER

November	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	5	820	0.640	2	969	0.088
2001	16	243	0.504	5	1530	0.348
2002	13	888	0.326	8	1608	0.219
2003	12	864	0.313	5	1174	0.168
2004	14	479	0.313	0	N/A	N/A
2005	12	785	0.391	6	1394	0.287
Total	72	639	0.393	26	1411	0.240

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-287
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – DECEMBER

December	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	15	730	0.310	13	973	0.273
2001	13	705	0.370	4	1255	0.178
2002	15	739	0.313	8	1052	0.341
2003	15	825	0.294	7	1338	0.205
2004	14	797	0.317	14	1240	0.264
2005	17	718	0.339	10	1151	0.322
Total	89	752	0.323	56	1149	0.274

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-288
INVERSION HEIGHTS AND STRENGTHS – NASHVILLE, TENNESSEE – 2000 – 2005 – ANNUAL

Annual	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	97	645	0.478	49	1254	0.274
2001	113	680	0.387	38	1518	0.287
2002	107	809	0.334	43	1355	0.275
2003	101	892	0.320	46	1382	0.281
2004	99	693	0.323	37	1332	0.325
2005	96	797	0.355	39	1297	0.404
Total	613	752	0.366	252	1352	0.305

a) Inversion is defined as three NOAA weather balloon elevation readings showing consecutive increases in temperature with height (below 3000 m).

b) Balloons were released each day at 0:00 GMT (7:00 a.m. EST) and 12:00 GMT (7:00 p.m. EST). Height is defined as elevation in meters where temperature first decreases and is averaged only over those days with inversions.

c) Strength is the maximum temperature gradient in tenths of a degrees Centigrade per meter within the inversion layer.

(Reference 223)

BLN COL 2.3-2

TABLE 2.3-289
NUMBER OF INVERSION[a] OCCURRENCES DURING JANUARY AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	6	8	4	7	0	5.0
2	5	9	1	3	4	4.4
3	4	3	2	2	1	2.4
4	1	1	0	2	0	0.8
5	2	1	2	1	1	1.4
6	1	0	2	0	1	0.8
7	1	0	0	0	1	0.4
8	0	2	1	2	1	1.2
9	1	2	0	1	3	1.4
10	0	2	1	3	0	1.2
11	1	0	1	1	3	1.2
12	2	1	1	2	0	1.2
13	2	2	2	0	2	1.6
14	3	1	1	0	2	1.4
15	1	2	2	3	0	1.6
16	1	3	6	2	0	2.4
17	3	0	5	1	0	1.8
≥18	3	1	2	1	12	3.8
Total	37	38	33	31	31	34.0

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-290
NUMBER OF INVERSION[a] OCCURRENCES DURING FEBRUARY AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	10	7	4	10	4	7.0
2	6	2	5	7	0	4.0
3	5	4	3	1	0	2.6
4	3	3	3	2	0	2.2
5	4	3	3	3	0	2.6
6	2	5	2	2	0	2.2
7	0	1	1	2	0	0.8
8	1	1	1	0	0	0.6
9	1	1	0	0	2	0.8
10	2	1	1	2	2	1.6
11	0	0	0	1	1	0.4
12	1	3	2	0	3	1.8
13	0	0	3	1	1	1.0
14	1	2	3	2	1	1.8
15	1	2	5	3	0	2.2
16	0	1	4	0	0	1.0
17	1	0	1	1	2	1.0
≥18	1	0	0	1	9	2.2
Total	39	36	41	38	25	35.8

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-291
NUMBER OF INVERSION[a] OCCURRENCES DURING MARCH
AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	6	0	5	5	2	3.6
2	7	2	2	7	3	4.2
3	3	1	1	3	2	2.0
4	2	1	2	4	2	2.2
5	3	2	0	0	1	1.2
6	3	0	1	0	1	1.0
7	1	2	3	0	0	1.2
8	2	0	1	1	3	1.4
9	1	0	1	0	0	0.4
10	0	0	2	1	6	1.8
11	4	1	0	0	5	2.0
12	1	0	1	2	2	1.2
13	4	0	5	1	1	2.2
14	1	2	9	5	0	3.4
15	2	0	4	4	1	2.2
16	0	0	0	3	2	1.0
17	0	0	0	0	0	0.0
≥18	0	1	0	1	8	2.0
Total	40	12	37	37	39	33.0

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-292
NUMBER OF INVERSION[(a)] OCCURRENCES DURING APRIL
AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[(b)]	1979	1980	1981	1982	2006-2007	Mean
1	7	2	10	11	1	6.2
2	3	1	3	5	1	2.6
3	3	0	0	1	1	1.0
4	3	1	2	1	0	1.4
5	2	0	0	5	0	1.4
6	2	0	0	0	0	0.4
7	2	1	1	0	0	0.8
8	1	1	0	0	0	0.4
9	0	0	1	1	0	0.4
10	0	0	1	1	2	0.8
11	0	1	0	1	3	1.0
12	6	3	6	4	2	4.2
13	5	0	5	6	1	3.4
14	0	2	8	1	0	2.2
15	1	0	3	2	0	1.2
16	0	0	0	0	0	0.0
17	0	0	0	0	2	0.4
≥18	0	0	0	1	6	1.4
Total	35	12	40	40	19	29.2

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-293
NUMBER OF INVERSION[a] OCCURRENCES DURING MAY AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	3	4	5	3	5	4.0
2	7	4	4	0	6	4.2
3	5	1	3	1	4	2.8
4	1	2	3	1	0	1.4
5	2	1	1	0	0	0.8
6	2	4	3	0	0	1.8
7	1	1	1	0	1	0.8
8	2	0	0	0	0	0.4
9	2	2	2	1	0	1.4
10	0	0	2	4	1	1.4
11	8	4	4	0	0	3.2
12	1	8	9	8	5	6.2
13	5	7	2	15	3	6.4
14	0	3	1	1	1	1.2
15	1	1	1	2	3	1.6
16	0	0	0	0	4	0.8
17	0	0	0	0	4	0.8
≥18	0	0	0	0	8	1.6
Total	40	42	41	36	45	40.8

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-294
NUMBER OF INVERSION[a] OCCURRENCES DURING JUNE AT
BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	3	4	5	5	5	4.4
2	1	7	2	4	5	3.8
3	4	1	4	3	1	2.6
4	0	0	0	4	0	0.8
5	5	2	0	1	1	1.8
6	1	1	0	1	1	0.8
7	1	0	2	1	0	0.8
8	2	2	1	3	1	1.8
9	3	1	1	1	1	1.4
10	3	1	1	3	2	2.0
11	4	3	4	6	0	3.4
12	4	6	12	5	3	6.0
13	2	7	4	5	7	5.0
14	0	0	2	1	2	1.0
15	0	0	0	0	4	0.8
16	0	0	0	0	0	0.0
17	0	0	1	0	0	0.2
≥18	0	0	0	0	8	1.6
Total	33	35	39	43	41	38.2

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-295
NUMBER OF INVERSION[a] OCCURRENCES DURING JULY AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	9	3	3	11	10	7.2
2	2	1	5	7	2	3.4
3	4	2	1	5	2	2.8
4	0	0	2	0	1	0.6
5	1	0	3	2	1	1.4
6	1	0	0	3	2	1.2
7	1	2	0	1	0	0.8
8	4	1	0	1	1	1.4
9	1	2	2	5	0	2.0
10	3	3	0	4	0	2.0
11	3	7	3	5	1	3.8
12	0	10	9	1	2	4.4
13	6	4	5	5	1	4.2
14	2	0	3	2	3	2.0
15	0	0	0	0	6	1.2
16	0	0	0	0	4	0.8
17	0	0	0	0	2	0.4
≥18	0	0	0	0	9	1.8
Total	37	35	36	52	47	41.4

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-296

NUMBER OF INVERSION[(a)] OCCURRENCES DURING AUGUST AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[(b)]	1979	1980	1981	1982	2006-2007	Mean
1	8	2	10	6	10	7.2
2	2	3	9	2	7	4.6
3	0	2	3	2	4	2.2
4	4	2	1	2	1	2.0
5	1	1	1	1	1	1.0
6	0	2	1	1	2	1.2
7	6	0	0	1	0	1.4
8	2	0	1	2	0	1.0
9	2	2	3	4	1	2.4
10	4	2	3	1	0	2.0
11	3	5	3	1	3	3.0
12	4	6	5	6	4	5.0
13	0	3	5	4	5	3.4
14	1	4	4	6	2	3.4
15	2	1	0	1	1	1.0
16	0	0	1	0	5	1.2
17	0	0	0	0	3	0.6
≥18	0	0	0	0	3	0.6
Total	39	35	50	40	52	43.2

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-297
NUMBER OF INVERSION[a] OCCURRENCES DURING SEPTEMBER AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	11	6	0	6	5	5.6
2	3	10	1	4	2	4.0
3	3	2	1	0	2	1.6
4	4	0	1	0	0	1.0
5	1	0	2	1	2	1.2
6	2	3	0	0	0	1.0
7	1	0	1	1	0	0.6
8	7	0	0	1	0	1.6
9	3	1	0	2	1	1.4
10	2	0	1	3	2	1.6
11	2	4	2	3	0	2.2
12	2	4	3	3	1	2.6
13	2	9	3	4	2	4.0
14	1	3	5	6	1	3.2
15	0	1	4	1	3	1.8
16	0	0	3	0	4	1.4
17	0	0	0	0	3	0.6
≥18	0	0	0	0	5	1.0
Total	44	43	27	35	33	36.4

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-298
NUMBER OF INVERSION[a] OCCURRENCES DURING OCTOBER AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	3	7	5	3	2	4.0
2	5	2	7	3	2	3.8
3	1	2	5	3	2	2.6
4	1	1	4	1	1	1.6
5	1	2	1	0	2	1.2
6	0	2	1	1	3	1.4
7	1	2	3	0	2	1.6
8	0	1	2	2	0	1.0
9	1	0	0	1	2	0.8
10	5	0	1	0	4	2.0
11	2	0	2	1	4	1.8
12	2	1	1	2	0	1.2
13	2	4	2	3	2	2.6
14	5	6	4	5	2	4.4
15	6	8	5	9	3	6.2
16	1	2	1	2	2	1.6
17	0	0	1	0	1	0.4
≥18	0	0	0	0	6	1.2
Total	36	40	45	36	40	39.4

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-299
NUMBER OF INVERSION[a] OCCURRENCES DURING NOVEMBER AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	9	5	9	9	9	8.2
2	4	2	3	3	1	2.6
3	2	1	2	4	3	2.4
4	2	1	2	2	1	1.6
5	4	1	4	0	1	2.0
6	2	0	0	2	0	0.8
7	2	0	2	0	2	1.2
8	0	1	0	0	3	0.8
9	0	1	3	2	3	1.8
10	0	1	2	0	5	1.6
11	1	1	0	1	1	0.8
12	3	1	0	0	0	0.8
13	0	0	0	0	1	0.2
14	4	0	1	4	0	1.8
15	2	8	5	4	0	3.8
16	4	6	8	4	2	4.8
17	1	1	2	2	1	1.4
≥18	1	1	1	1	6	2.0
Total	41	31	44	38	39	38.6

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-300
NUMBER OF INVERSION[a] OCCURRENCES DURING
DECEMBER AT BLN SITE – 1979 – 1982 & 2006 – 2007

Duration[b]	1979	1980	1981	1982	2006-2007	Mean
1	3	3	8	16	3	6.6
2	3	0	7	3	3	3.2
3	2	2	4	3	1	2.4
4	2	1	3	4	1	2.2
5	2	0	4	2	1	1.8
6	3	0	2	1	3	1.8
7	1	0	3	1	1	1.2
8	2	2	0	0	6	2.0
9	4	1	2	1	4	2.4
10	0	1	2	4	2	1.8
11	2	0	1	0	0	0.6
12	1	0	2	1	1	1.0
13	0	1	1	2	1	1.0
14	1	0	1	0	0	0.4
15	1	2	0	0	0	0.6
16	5	6	1	2	0	2.8
17	1	7	1	0	0	1.8
18	1	4	2	0	8	3.0
Total	34	30	44	40	35	36.6

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G stability for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-301 (Sheet 1 of 2)
ANNUAL NUMBER OF INVERSION[a] OCCURRENCES AT BLN SITE – 1979 – 1982 & 2006 – 2007

	Months												
Duration[b]	J	F	M	A	M	J	J	A	S	O	N	D	Annual
1	5.0	7.0	3.6	6.2	4.0	4.4	7.2	7.2	5.6	4.0	8.2	6.6	69.0
2	4.4	4.0	4.2	2.6	4.2	3.8	3.4	4.6	4.0	3.8	2.6	3.2	44.8
3	2.4	2.6	2.0	1.0	2.8	2.6	2.8	2.2	1.6	2.6	2.4	2.4	27.4
4	0.8	2.2	2.2	1.4	1.4	0.8	0.6	2.0	1.0	1.6	1.6	2.2	17.8
5	1.4	2.6	1.2	1.4	0.8	1.8	1.4	1.0	1.2	1.2	2.0	1.8	17.8
6	0.8	2.2	1.0	0.4	1.8	0.8	1.2	1.2	1.0	1.4	0.8	1.8	14.4
7	0.4	0.8	1.2	0.8	0.8	0.8	0.8	1.4	0.6	1.6	1.2	1.2	11.6
8	1.2	0.6	1.4	0.4	0.4	1.8	1.4	1.0	1.6	1.0	0.8	2.0	13.6
9	1.4	0.8	0.4	0.4	1.4	1.4	2.0	2.4	1.4	0.8	1.8	2.4	16.6
10	1.2	1.6	1.8	0.8	1.4	2.0	2.0	2.0	1.6	2.0	1.6	1.8	19.8
11	1.2	0.4	2.0	1.0	3.2	3.4	3.8	3.0	2.2	1.8	0.8	0.6	23.4
12	1.2	1.8	1.2	4.2	6.2	6.0	4.4	5.0	2.6	1.2	0.8	1.0	35.6
13	1.6	1.0	2.2	3.4	6.4	5.0	4.2	3.4	4.0	2.6	0.2	1.0	35.0
14	1.4	1.8	3.4	2.2	1.2	1.0	2.0	3.4	3.2	4.4	1.8	0.4	26.2
15	1.6	2.2	2.2	1.2	1.6	0.8	1.2	1.0	1.8	6.2	3.8	0.6	24.2
16	2.4	1.0	1.0	0.0	0.8	0.0	0.8	1.2	1.4	1.6	4.8	2.8	17.8
17	1.8	1.0	0.0	0.4	0.8	0.2	0.4	0.6	0.6	0.4	1.4	1.8	9.4

TABLE 2.3-301 (Sheet 2 of 2)
ANNUAL NUMBER OF INVERSION[a] OCCURRENCES AT BLN SITE – 1979 – 1982 & 2006 – 2007

	Months													
	Duration[b]	J	F	M	A	M	J	J	A	S	O	N	D	Annual
BLN COL 2.3-2	18	3.8	2.2	2.0	1.4	1.6	1.6	1.8	0.6	1.0	1.2	2.0	3.0	22.2
	Total	34.0	35.8	33.0	29.2	40.8	38.2	41.4	43.2	36.4	39.4	38.6	36.6	446.6

a) Based on Pasquill-Turner calculation of E, F or G from hourly surface observations.

b) Consecutive hours of E, F or G stability for each discreet occurrence.

c) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

d) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

TABLE 2.3-302

BLN COL 2.3-2

PERCENT OF HOURS WITH INVERSION[a] AT BLN SITE – 1979 – 1982

YEAR	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
1979	41.9	28.0	34.1	32.1	37.7	36.2	34.2	38.3	35.6	45.7	43.1	40.2	37.0
1980	33.2	32.4	33.3	43.2	49.6	43.5	46.4	48.9	45.5	49.5	49.5	54.8	43.2
1981	51.1	50.0	46.7	48.4	41.6	48.4	41.3	47.1	57.1	45.2	52.9	38.2	48.2
1982	31.8	32.8	39.7	37.7	53.4	45.5	45.4	45.7	42.4	52.8	43.8	27.0	42.8
2006-7	77.0	77.5	66.8	74.3	69.9	68.2	74.7	68.8	75.6	68.2	68.5	63.4	70.9
MEAN	47.0	44.1	44.1	47.1	50.4	48.4	48.4	49.8	51.2	52.3	51.6	44.7	48.4

a) Based on Pasquill-Turner calculations of E, F or G stability from hourly surface observations.

b) Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982.

c) Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-1

TABLE 2.3-303
MIXING HEIGHTS AT NASHVILLE, TENNESSEE – 1984 – 1987 & 1990 – 1991

	Morning (m)	Afternoon (m)
January	566	747
February	595	949
March	580	1310
April	540	1718
May	505	1559
June	412	1706
July	382	1806
August	420	1709
September	376	1590
October	354	1244
November	518	857
December	559	726
Average	484	1327

(Reference 204)

BLN COL 2.3-2

TABLE 2.3-304
AVERAGE PLUME LENGTHS IN MILES

Direction[a]	Winter	Spring	Summer	Fall
S	3.08	1.87	1.07	2.15
S-SW	3.11	2.07	1.63	2.39
SW	2.66	1.71	1.62	2.19
W-SW	2.05	1.49	1.49	1.50
W	1.88	1.73	1.12	1.34
W-NW	2.03	1.13	0.80	1.33
NW	2.06	1.57	0.98	2.08
N-NW	2.28	1.24	0.86	1.60
N	2.47	1.21	0.92	1.30
N-NE	3.06	1.85	1.43	2.07
NE	2.57	1.83	0.96	2.19
E-NE	2.91	1.46	0.83	1.94
E	3.00	1.72	0.74	1.79
E-SE	3.06	1.73	0.78	1.55
SE	2.85	1.68	0.63	1.44
S-SE	2.93	1.73	0.64	1.65
All	2.80	1.65	1.17	1.93

a) Plume from 2 NDCTs moving in the indicated direction.

BLN COL 2.3-2

TABLE 2.3-305
VISIBLE PLUME LENGTH SUMMARY - NDCT

	Winter	Spring	Summer	Fall
Most Frequent Plume Heading Directions	S, S-SW, SW	N, SW	N, S-SW, SW	S, S-SW, SW
Percent of Plumes < 1/3 miles	13.0	37.3	53.6	32.8
Percent of Plumes > 1/3 to 2/3 miles	12.3	16.7	14.8	15.6
Percent of Plumes > 2/3 to 5 miles	49.2	32.2	22.3	34.2
Percent of Plumes > 5 Miles	25.5	13.8	9.3	17.4

BLN COL 2.3-2

TABLE 2.3-306
RELATIVE HUMIDITY FOR 4 TIME PERIODS PER DAY –
BLN SITE – 2006 – 2007

	00:00-06:00	06:00-12:00	12:00-18:00	18:00-24:00
Jan	73%	70%	53%	64%
Feb	67%	57%	38%	52%
Mar	72%	67%	41%	54%
Apr	78%	71%	43%	59%
May	90%	77%	57%	75%
Jun	88%	74%	48%	69%
Jul	87%	76%	48%	66%
Aug	86%	76%	52%	71%
Sep	86%	79%	52%	70%
Oct	85%	80%	52%	76%
Nov	81%	72%	54%	71%
Dec	80%	74%	53%	73%
Annual	81%	73%	49%	67%

NOTES:

1. Bellefonte (BLN) Site data is from meteorological tower measurements from 4/1/2006 through 3/31/2007.

2. Hourly readings are averaged over the six hour period over all the days in the given months for these four years.

BLN COL 2.3-2

TABLE 2.3-307
PRECIPITATION DATA AT BLN SITE – 2006 – 2007

Month	Monthly Mean	Max 24 hour	No. days > 0.01 in
Jan	2.1	1.2	7
Feb	2.0	1.0	5
Mar	1.0	0.5	3
Apr	3.9	1.3	10
May	3.4	1.2	13
Jun	1.7	0.7	10
Jul	2.0	0.7	11
Aug	3.6	0.8	14
Sep	0.1	0.4	1
Oct	5.1	2.2	9
Nov	3.3	2.1	10
Dec	2.9	1.4	8

Annual Average Rainfall (in) = 30.9

NOTES:

1. Precipitation data measured in inches of rain.
2. The month in which the Max 24 hour period is reported is the month when the 24 hour period began.
3. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-2

TABLE 2.3-308
RAINFALL FREQUENCY DISTRIBUTION AT BLN SITE – 2006 – 2007

Rainfall (inch/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	11	3	12	12	15	8	13	6	0	19	16	12
0.02-.099	14	11	9	14	21	6	12	8	2	27	21	21
0.10-0.249	7	8	4	5	5	6	2	7	0	12	4	7
0.25-0.499	2	1	0	4	4	2	1	4	0	4	3	3
0.50-0.99	0	0	0	2	0	0	1	1	0	1	1	0
1.00-1.99	0	0	0	0	0	0	0	0	0	0	0	0
2.0 & over	0	0	0	0	0	0	0	0	0	0	0	0
Total	34	23	25	37	45	22	29	26	2	63	45	43

NOTES:

1. Data from Site Meteorological Tower, 4/1/2006 – 3/31/2007.

BLN COL 2.3-4

TABLE 2.3-309 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS A
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS A

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
NNE	0	0	0	0	0	0	2	1	0	0	0	0	3	3.03
NE	0	0	0	0	0	0	1	1	0	0	0	0	2	3.29
ENE	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
E	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
ESE	0	0	0	0	0	1	0	0	0	0	0	0	1	1.97
SE	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
SSE	0	0	0	0	0	0	0	3	0	0	0	0	3	3.29
S	0	0	0	0	0	0	1	2	0	0	0	0	3	3.10
SSW	0	0	0	0	0	0	0	3	1	0	0	0	4	3.86

BLN COL 2.3-4

TABLE 2.3-309 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS A
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS A

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
SW	0	0	0	0	0	0	2	1	4	0	0	0	7	3.59
WSW	0	0	0	0	0	0	0	3	0	3	0	0	6	4.46
W	0	0	0	0	0	0	3	3	0	0	0	0	6	3.10
WNW	0	0	1	1	0	0	2	5	0	0	0	0	9	2.70
NW	0	0	0	0	0	0	0	1	0	0	0	0	1	3.04
NNW	0	0	1	0	0	0	1	3	5	1	0	0	11	3.68
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
TOTAL	0	0	2	1	0	1	13	27	10	4	0	0	58	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-310 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS B
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS B

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	0	0	0	0	0	0	7	8	3	0	0	0	19	3.24
NNE	0	0	0	0	1	0	2	2	0	0	0	0	5	2.78
NE	0	0	0	0	0	0	0	1	2	0	0	0	3	4.07
ENE	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
E	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
ESE	0	0	0	0	0	0	0	1	0	0	0	0	1	3.09
SE	0	0	0	0	0	0	1	0	0	0	0	0	1	2.68
SSE	0	0	0	0	0	0	1	0	0	0	0	0	1	2.77
S	0	1	0	0	0	1	5	6	3	0	0	0	17	3.07
SSW	0	0	0	0	0	0	3	11	2	0	0	0	17	3.46
SW	0	0	0	0	0	0	4	3	3	2	1	0	14	4.14
WSW	0	0	0	0	0	0	2	8	1	6	0	0	18	4.15

BLN COL 2.3-4

TABLE 2.3-310 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS B
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS B

	Wind Speed (m/sec)												Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	0	0	0	0	0	0	7	8	4	1	1	0	22	3.53
WNW	0	0	0	0	0	0	4	7	0	1	1	0	14	3.47
NW	0	0	0	0	0	1	1	1	1	0	0	0	4	3.26
NNW	0	0	0	1	0	2	1	7	5	1	0	0	18	3.51
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
TOTAL	0	1	0	1	1	4	40	66	25	11	3	0	152	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-311 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS C
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS C

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	0	0	0	0	1	0	18	11	6	0	0	0	36	3.19
NNE	0	0	0	0	0	5	29	7	1	0	0	0	43	2.65
NE	0	0	0	0	0	3	11	11	8	1	0	0	35	3.25
ENE	0	0	0	0	0	2	6	1	0	0	0	0	9	2.38
E	0	0	0	0	0	0	2	0	0	0	0	0	2	2.73
ESE	0	0	0	0	0	0	3	0	1	0	0	0	4	2.98
SE	0	0	0	0	0	1	4	0	0	0	0	0	5	2.29
SSE	0	0	0	0	0	0	18	3	0	0	0	0	21	2.53
S	0	0	0	1	0	2	11	9	2	0	0	0	26	2.93
SSW	0	0	0	0	0	3	14	24	10	1	0	0	52	3.31
SW	0	0	0	0	0	2	15	17	7	6	3	0	50	3.85
WSW	0	0	0	0	1	4	13	8	9	1	2	2	41	3.66

BLN COL 2.3-4

TABLE 2.3-311 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS C
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS C

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	0	0	0	0	1	2	17	13	3	0	0	0	35	2.89
WNW	0	0	0	0	1	2	20	6	4	1	0	0	34	2.93
NW	0	1	0	0	0	2	7	0	2	1	0	0	14	2.86
NNW	0	0	0	0	0	4	5	7	7	0	0	0	24	3.28
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
TOTAL	0	1	0	1	4	33	193	119	63	11	5	2	433	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-312 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS D
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS D

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	4	7	41	35	45	72	129	86	44	6	0	0	469	2.38
NNE	3	9	31	42	68	162	171	43	8	0	0	0	537	1.96
NE	4	2	22	44	71	128	175	49	8	2	0	0	506	2.04
ENE	2	4	15	10	19	35	40	4	1	0	0	0	130	1.75
E	0	3	2	4	4	11	19	0	1	0	0	0	45	1.84
ESE	1	1	4	7	7	17	19	5	2	0	0	0	64	1.93
SE	0	1	2	4	10	24	31	15	10	4	8	0	111	2.86
SSE	0	0	4	8	11	38	53	23	5	6	0	0	149	2.43
S	2	3	7	10	13	36	86	49	15	5	3	1	230	2.62
SSW	0	2	5	13	22	60	100	90	41	21	2	0	356	2.87
SW	1	2	10	18	18	47	104	50	26	18	1	0	295	2.68
WSW	1	1	7	8	22	43	60	34	14	9	7	0	208	2.63

BLN COL 2.3-4

TABLE 2.3-312 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS D
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS D

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	0	5	13	15	21	29	51	23	9	3	0	0	169	2.23
WNW	1	0	15	13	9	27	36	33	5	4	0	0	144	2.36
NW	5	8	11	8	13	40	42	33	14	8	0	0	182	2.41
NNW	4	5	22	21	27	36	72	64	22	14	1	0	288	2.52
CALM	10	0	0	0	0	0	0	0	0	0	0	0	10	0.43
TOTAL	40	55	212	261	380	806	1189	601	225	101	23	1	3893	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-313 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS E
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS E

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	13	28	42	25	14	34	17	3	0	0	0	0	175	1.23
NNE	26	26	92	55	65	88	57	6	0	0	0	0	415	1.37
NE	20	38	75	67	73	78	59	5	0	0	1	0	416	1.37
ENE	13	14	23	9	9	20	4	3	1	0	0	0	96	1.22
E	3	3	9	0	2	1	2	2	0	0	0	0	23	1.22
ESE	1	4	9	7	4	2	0	0	0	0	0	0	28	1.01
SE	2	0	4	5	6	3	13	5	1	0	0	0	40	1.96
SSE	4	3	15	9	6	9	8	11	0	0	0	0	67	1.67
S	7	9	14	8	23	26	18	10	0	0	2	0	118	1.72
SSW	18	15	31	23	40	62	105	41	15	3	3	0	355	2.07
SW	17	17	27	26	32	42	66	20	4	1	2	0	253	1.80
WSW	7	9	17	17	15	19	15	1	2	0	0	0	101	1.44

BLN COL 2.3-4

TABLE 2.3-313 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS E
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS E

	Wind Speed (m/sec)												Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	7	9	15	11	7	6	8	5	0	0	0	0	70	1.35
WNW	1	4	9	5	3	7	4	1	0	0	0	0	35	1.37
NW	4	13	8	8	10	9	0	3	0	0	0	0	56	1.19
NNW	9	4	9	10	11	7	5	2	1	0	0	0	60	1.31
CALM	113	0	0	0	0	0	0	0	0	0	0	0	113	0.39
TOTAL	265	196	399	288	321	414	382	120	24	4	8	0	2421	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-314 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS F
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS F

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	2	15	8	3	1	0	0	0	0	0	0	0	29	0.78
NNE	26	16	18	7	5	2	1	0	0	0	0	0	75	0.82
NE	28	16	31	9	8	4	3	0	0	0	0	0	100	0.87
ENE	11	17	15	6	3	3	1	0	0	0	0	0	56	0.87
E	15	6	6	2	1	1	1	0	0	0	0	0	32	0.78
ESE	8	6	4	0	2	0	0	0	0	0	0	0	21	0.70
SE	9	10	3	1	0	2	1	0	0	0	0	0	27	0.81
SSE	18	7	5	2	0	0	0	0	0	0	0	0	32	0.65
S	30	15	14	3	2	1	1	0	0	0	0	0	66	0.72
SSW	44	32	21	14	14	6	9	0	0	0	0	0	140	0.92
SW	22	29	27	13	7	6	3	1	0	0	0	0	108	0.93
WSW	6	4	10	3	3	1	0	0	0	0	0	0	28	0.85

BLN COL 2.3-4

TABLE 2.3-314 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS F
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS F

	Wind Speed (m/sec)												Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	4	5	3	7	0	0	0	0	0	0	0	0	20	0.83
WNW	2	5	3	0	0	0	0	0	0	0	0	0	10	0.70
NW	1	6	6	2	2	0	1	0	0	0	0	0	19	0.91
NNW	0	4	2	2	0	1	0	0	0	0	0	0	9	0.95
CALM	274	0	0	0	0	0	0	0	0	0	0	0	274	0.37
TOTAL	502	194	177	75	49	28	22	1	0	0	0	0	1048	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.

2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-4

TABLE 2.3-315 (Sheet 1 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS G
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS G

	Wind Speed (m/sec)													Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
N	9	3	2	1	0	0	0	0	0	0	0	0	16	0.64
NNE	7	9	4	1	0	0	0	0	0	0	0	0	22	0.68
NE	17	6	9	4	2	0	0	0	0	0	0	0	39	0.74
ENE	15	15	6	3	0	0	0	0	0	0	0	0	39	0.68
E	10	2	2	1	0	0	0	0	0	0	0	0	16	0.64
ESE	8	4	1	0	0	0	0	0	0	0	0	0	14	0.60
SE	11	2	2	0	0	0	0	0	0	0	0	0	16	0.58
SSE	11	4	3	2	0	0	0	0	0	0	0	0	21	0.66
S	14	15	6	0	0	0	0	0	0	0	0	0	34	0.64
SSW	40	20	17	5	3	1	1	0	0	0	0	0	87	0.73
SW	18	13	21	3	3	0	0	0	0	0	0	0	57	0.75
WSW	8	4	3	3	2	0	1	0	0	0	0	0	22	0.84

BLN COL 2.3-4

TABLE 2.3-315 (Sheet 2 of 2)
JOINT FREQUENCY DISTRIBUTION OF WIND SPEED AND DIRECTION BY ATMOSPHERIC STABILITY CLASS – STABILITY CLASS G
HOURS AT EACH WIND SPEED AND DIRECTION

STABILITY CLASS G

	Wind Speed (m/sec)												Average Wind Speed (m/sec)
DIR	≤0.6	≤0.75	≤1.0	≤1.3	≤1.5	≤2.0	≤3.0	≤4.0	≤5.0	≤6.0	≤8.0	≤10	Total	
W	4	2	2	0	0	0	0	0	0	0	0	0	8	0.67
WNW	4	1	3	0	0	0	0	0	0	0	0	0	8	0.68
NW	2	5	2	2	0	0	0	0	0	0	0	0	11	0.74
NNW	8	6	3	2	0	0	0	0	0	0	0	0	20	0.69
CALM	326	0	0	0	0	0	0	0	0	0	0	0	326	0.36
TOTAL	514	112	88	28	10	1	2	0	0	0	0	0	755	

NOTES:

1. Data from BLN Site Meteorological Tower, 4\1\2006 - 3\31\2007.
2. Calms are windspeeds less than or equal to 0.45 m/sec.

BLN COL 2.3-2

TABLE 2.3-316
ANNUAL STABILITY CLASS PERCENTAGE FREQUENCY, BLN SITE

Stability Class	1979	1980	1981	1982	2006-2007	Avg.
A	1.0%	2.3%	0.2%	0.6%	0.7%	0.9%
B	2.9%	3.2%	1.2%	1.8%	1.7%	2.1%
C	7.0%	6.5%	4.2%	5.2%	4.9%	5.5%
D	51.9%	43.2%	48.2%	51.1%	44.4%	47.8%
E	26.9%	31.1%	33.1%	29.8%	27.6%	29.7%
F	7.5%	9.5%	8.8%	8.4%	12.0%	9.2%
G	2.8%	4.9%	4.3%	3.1%	8.6%	4.7%

NOTES:

1. Data from Site Meteorological Tower, 1/1/1979 – 12/31/1982 and 4/1/2006 – 3/31/2007.

BLN COL 2.3-3

TABLE 2.3-317 (Sheet 1 of 2)
BELLEFONTE NUCLEAR PLANT METEOROLOGICAL INSTRUMENTATION

BLN Meteorological Instrumentation
October 29, 1975 - November 1, 1983

Sensor	Height meters	Description
Wind Direction	10, 60, and 110	Climet Instruments, Inc., Model 012-10; threshold, 0.75 mph; accuracy ±3°.
Wind Speed	10, 60, and 110	Climet Instruments, Inc., Model 011-1; threshold, 0.6 mph; accuracy ±1% or 0.15 mph, whichever is greater.
Temperature	10, 60, and 110	Weed Instrument Co., Model 101; accuracy ±0.06°F; Climet Instruments, Inc., Model 016-1 aspirated radiation shield; error, 0°F to 0.2°F.
Dewpoint	10	EG&G, Inc. Model 440; accuracy ±0.7°F.
Rainfall	1	Belfort Instrument Co., Model 5915-12; accuracy ±0.06 inch.

Meteorological Instrumentation
2006-2007

Sensor	Level, meters (feet)	Sensor Specifications
Wind Direction (WD) and Wind Speed (WS)	10, 54	Ultrasonic wind sensor; starting threshold, 0 mph WD: resolution, 1°; range, 0 to 360°; accuracy ±2°. WS: resolution, 0.1 mph; range, 0 to 144 mph; accuracy ±0.3 mph or 3% of reading, whichever is greater.

BLN COL 2.3-3

TABLE 2.3-317 (Sheet 2 of 2)
BELLEFONTE NUCLEAR PLANT METEOROLOGICAL INSTRUMENTATION

Ambient Air Temperature	10, 54	RTD Temperature (platinum wire resistance temperature detector) mounted in motor-fan aspirated solar radiation shield, R. M. Young, Co. model 43408.
		Sensor: Data recording range -30.0 to 120.0°F
		RTD stability, ±0.25°F/year
		RTD repeatability, ±0.25°F
		time response, 5 seconds.
		Aspirated Shield: Maximum radiation error, 0 to +0.4°F
		Delta-T error, 0.1°F with like shields
		Aspiration flow rate, 3.5 to 7.6 m/s.
Dewpoint Temperature	10	Humidity and Temperature Transmitter for High Humidity Applications; capacitive humidity sensor with warmed probe head.
		Temperature range, -70 to +180°C
		Measurement range, 0 to 100% RH
		Factory calibration uncertainty, ±0.6 % RH for 0.to
		40% RH and ± 1.0 % for 40.to 97% RH.
Rainfall	1 (3.3)	Heated tipping bucket rain gauge.
		Accuracy ±0.5% at 0.5 inch/hour
		and ±2.0% at 2 inches/hour
		Sensitivity, ±0.01 inches
		Resolution 0.01 inch.

BLN COL 2.3-4

TABLE 2.3-318
MINIMUM EXCLUSION AREA BOUNDARY (EAB) DISTANCES
[FROM INNER 160 M (525 FT) RADIUS CIRCLE
ENCOMPASSING ALL SITE RELEASE POINTS]

	Minimum Distance from Effluent Release Boundary to EAB[1]	
Direction	Distance (ft)	Distance (m)
S	3755	1145
SSW	5445	1660
SW	4098	1249
WSW	3861	1177
W	3114	949
WNW	2805	855
NW	2805	855
NNW	2840	866
N	3069	935
NNE	4081	1244
NE	5805	1769
ENE	4100	1250
E	3108	947
ESE	3041	927
SE	3041	927
SSE	3059	932

Notes:

1. Exclusion Area Boundary (EAB) for the BLN is shown in FSAR Figure 2.1-205.

2. The minimum distance is based on the shortest distance from the 525 ft. effluent release boundary to the EAB within a 45° sector centered on each compass direction.

3. The above distances are used in the short term atmospheric dispersion estimates.

BLN COL 2.3-4

TABLE 2.3-319
BLN OFFSITE ATMOSPHERIC DISPERSION
EXCLUSION AREA BOUNDARY χ/Q VALUES (sec/m^3)

	Exclusion Area Boundary χ/Q (sec/m^3)		
	Direction Dependent χ/Q		Direction Independent χ/Q
Time Period	0.5% Max Sector χ/Q[a]	Sector/Distance	5% Overall Site Limit
0-2 Hrs	5.85E-04	NNE/1244 m	4.20E-04

	Low Population Zone χ/Q Values (sec/m^3)		
	Low Population Zone χ/Q (sec/m^3)		
	Direction Dependent χ/Q		Direction Independent χ/Q
Time Period	0.5% Max Sector χ/Q[a]	Sector/Distance	5% Overall Site Limit
0-8 Hrs	1.23E-04	NNE	9.06E-05
8-24 Hrs	8.26E-05	NNE	6.28E-05
1-4 Days	3.49E-05	NNE	2.83E-05
4-30 Days	1.01E-05	NNE	9.03E-06

a) 0.5% χ/Q values represent the maximum for all sector-dependent values

BLN 5% Maximum χ/Q Values (sec/m^3)

	0 – 2 Hrs	0 –8 Hrs	8 – 24 Hrs	24 -96 Hrs	96 – 720 Hrs
EAB (NNE, 1244 m)	5.85E-04	N/A	N/A	N/A	N/A
LPZ (2 miles)	N/A	1.23E-04	8.26E-05	3.49E-05	1.01E-05

BLN COL 2.3-4

TABLE 2.3-320
CONTROL ROOM HVAC INTAKE DISTANCES AND DIRECTIONS

Release Point	Distance (m)	Direction to Source (degrees)
Plant Vent	39.6	53
PCS Air Diffuser	32.3	84
Fuel Building Blowout Panel	50.0	36
Fuel Building Rail Bay Door	52.4	36
Steam Vent	18.3	126
PORV/Safety Valves	19.8	136
Condenser Air Removal Stack	63.0	166.5
Containment Shell	11	75

Annex Building Access (El. 1.5 m) Distances and Directions

Release Point	Distance (m)	Direction to Source (degrees)
Plant Vent	76.8	62
PCS Air Diffuser	68.9	78.5
Fuel Building Blowout Panel	89.7	50
Fuel Building Rail Bay Door	92.1	49.5
Steam Vent	48.8	90.5
PORV/Safety Valves	44.1	94.5
Condenser Air Removal Stack	59.9	131
Containment Shell	47.2	74

BLN COL 2.3-4

TABLE 2.3-321 (Sheet 1 of 2)
CONTROL ROOM ATMOSPHERIC DISPERSION FACTORS
(χ/Q) FOR ACCIDENT DOSE ANALYSIS
χ/Q (S/M^3) AT HVAC INTAKE, BLN UNITS 3 AND 4

Time Interval	Plant Vent	PCS Air Diffuser	Fuel Bldg. Blowout Panel	Fuel Bldg. Rail Bay Door
0 -2 hours	2.2E-03	1.6E-03	2.2E-03	1.7E-03
2 – 8 hours	1.9E-03	7.8E-04	1.8E-03	1.4E-03
8 – 24 hours	8.6E-04	3.6E-04	8.8E-04	6.8E-04
1 – 4 days	6.3E-04	2.7E-04	6.8E-04	5.2E-04
4 – 30 days	4.8E-04	2.2E-04	4.8E-04	3.6E-04
	Steam Vent	PORV & Safety Valves	Condenser Air Removal Stack	Containment Shell
0 -2 hours	1.1E-02	1.0E-02	1.3E-03	2.4E-03
2 – 8 hours	3.4E-03	3.8E-03	8.4E-04	1.8E-03
8 – 24 hours	2.2E-03	2.2E-03	3.3E-04	7.1E-04
1 – 4 days	1.6E-03	1.5E-03	2.5E-04	6.4E-04
4 – 30 days	9.8E-04	9.3E-04	1.9E-04	5.4E-04

Control Room Atmospheric Dispersion Factors (χ/Q) for Accident Dose Analysis
χ/Q (s/m^3) at Annex Building Access

Time Interval	Plant Vent	PCS Air Diffuser	Fuel Bldg. Blowout Panel	Fuel Bldg. Rail Bay Door
0 -2 hours	7.3E-04	6.8E-04	6.8E-04	6.4E-04
2 – 8 hours	6.3E-04	4.4E-04	5.7E-04	5.2E-04
8 – 24 hours	2.8E-04	2.0E-04	2.7E-04	2.5E-04
1 – 4 days	2.1E-04	1.5E-04	2.0E-04	1.8E-04
4 – 30 days	1.6E-04	1.2E-04	1.6E-04	1.4E-04

BLN COL 2.3-4

TABLE 2.3-321 (Sheet 2 of 2)
CONTROL ROOM ATMOSPHERIC DISPERSION FACTORS
(χ/Q) FOR ACCIDENT DOSE ANALYSIS
χ/Q (S/M^3) AT HVAC INTAKE, BLN UNITS 3 AND 4

Time Interval	Plant Vent	PCS Air Diffuser	Fuel Bldg. Blowout Panel	Fuel Bldg. Rail Bay Door
	Steam Vent	PORV & Safety Valves	Condenser Air Removal Stack	Containment Shell
0 -2 hours	1.7E-03	1.8E-03	1.1E-03	7.4E-04
2 – 8 hours	5.6E-04	6.0E-04	4.2E-04	5.8E-04
8 – 24 hours	3.1E-04	2.9E-04	2.5E-04	2.5E-04
1 – 4 days	2.5E-04	2.7E-04	1.7E-04	2.0E-04
4 – 30 days	1.9E-04	1.9E-04	1.1E-04	1.6E-04

BLN COL 2.3-5

TABLE 2.3-322
BLN OFFSITE RECEPTOR LOCATIONS

Sector	Garden	Milk Cow/Goat	House	Animal for Meat	School
S		7681		7681	
SSW	7338				
SW	2807				
WSW	6780	7406		7406	
W	4244	2348	2348	2348	
WNW	1143	1214	1169	1214	4243
NW	1289	1586	1103	1586	
NNW	1821	3520		3520	
N	3310	3417		3417	
NNE	2006	3571		3571	
NE	6648	6648		6648	
ENE	5588	6135		6135	
E	3861	4036		3861	
ESE	4388	4362		4362	
SE	7204				
SSE					

Notes:

1. Distances, in meters, from the site center to the nearest receptor of each type for a given sector.

BLN COL 2.3-5

TABLE 2.3-323 (Sheet 1 of 2)
ANNUAL AVERAGE χ/Q (sec/m^3) FOR NO DECAY, UNDEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.54E-06	1.39E-06	1.12E-06	8.46E-07	6.88E-07	2.06E-06	1.29E-06	8.88E-07	6.56E-07	5.09E-07	4.09E-07
SSW	4.82E-06	1.81E-06	1.51E-06	1.22E-06	9.04E-07	6.85E-07	5.35E-07	4.30E-07	3.56E-07	8.33E-07	7.33E-07
SW	4.86E-06	1.76E-06	1.45E-06	1.17E-06	8.63E-07	6.56E-07	5.14E-07	4.16E-07	3.45E-07	2.93E-07	2.53E-07
WSW	1.06E-06	3.93E-07	3.38E-07	2.91E-07	2.40E-07	1.95E-07	1.60E-07	1.33E-07	1.14E-07	9.87E-08	8.69E-08
W	3.26E-07	1.22E-07	1.05E-07	8.70E-08	7.18E-08	6.21E-08	5.46E-08	4.86E-08	4.37E-08	3.96E-08	3.62E-08
WNW	4.26E-07	1.64E-07	1.42E-07	1.15E-07	8.66E-08	6.80E-08	5.54E-08	4.65E-08	4.00E-08	2.86E-07	2.32E-07
NW	1.07E-06	3.94E-07	2.95E-07	2.05E-07	1.28E-07	9.21E-08	7.16E-08	5.85E-08	2.05E-07	3.58E-07	2.90E-07
NNW	1.31E-06	5.21E-07	4.16E-07	3.03E-07	2.00E-07	1.48E-07	1.17E-07	9.63E-08	3.05E-07	4.77E-07	3.86E-07
N	2.33E-06	9.03E-07	6.88E-07	4.89E-07	3.21E-07	2.40E-07	1.91E-07	1.59E-07	1.36E-07	1.19E-07	1.05E-07
NNE	5.80E-06	2.06E-06	1.45E-06	1.00E-06	6.50E-07	4.83E-07	3.83E-07	3.15E-07	2.68E-07	2.32E-07	2.05E-07
NE	4.12E-06	1.49E-06	1.08E-06	7.70E-07	5.19E-07	3.89E-07	3.07E-07	1.10E-06	1.23E-06	9.60E-07	7.75E-07
ENE	2.31E-06	8.81E-07	6.52E-07	4.59E-07	2.93E-07	1.50E-06	9.41E-07	6.53E-07	4.84E-07	3.77E-07	3.04E-07
E	1.50E-06	6.26E-07	4.85E-07	3.54E-07	6.15E-07	9.94E-07	6.20E-07	4.28E-07	3.17E-07	2.46E-07	1.98E-07
ESE	1.21E-06	5.16E-07	3.94E-07	2.74E-07	1.17E-06	6.98E-07	4.36E-07	3.02E-07	2.24E-07	1.74E-07	1.40E-07
SE	1.38E-06	5.36E-07	4.26E-07	3.16E-07	1.31E-06	7.70E-07	4.79E-07	3.31E-07	2.44E-07	1.89E-07	1.52E-07
SSE	2.25E-06	8.75E-07	6.72E-07	4.80E-07	2.11E-06	1.28E-06	8.02E-07	5.55E-07	4.11E-07	3.20E-07	2.58E-07

BLN COL 2.3-5

TABLE 2.3-323 (Sheet 2 of 2)
ANNUAL AVERAGE χ/Q (sec/m^3) FOR NO DECAY, UNDEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.38E-07	1.73E-07	1.12E-07	6.46E-08	4.39E-08	3.26E-08	2.56E-08	2.09E-08	1.76E-08	1.51E-08	1.31E-08
SSW	6.06E-07	3.10E-07	2.01E-07	1.15E-07	7.82E-08	5.80E-08	4.55E-08	3.71E-08	3.11E-08	2.66E-08	2.32E-08
SW	2.63E-07	3.86E-07	2.52E-07	1.46E-07	9.97E-08	7.44E-08	5.87E-08	4.80E-08	4.04E-08	3.48E-08	3.04E-08
WSW	7.75E-08	2.22E-07	1.46E-07	8.61E-08	5.94E-08	4.47E-08	3.55E-08	2.92E-08	2.47E-08	2.13E-08	1.87E-08
W	3.33E-08	1.35E-07	8.92E-08	5.27E-08	3.65E-08	2.75E-08	2.18E-08	1.80E-08	1.52E-08	1.32E-08	1.16E-08
WNW	1.93E-07	1.01E-07	6.68E-08	3.94E-08	2.72E-08	2.05E-08	1.63E-08	1.34E-08	1.14E-08	9.83E-09	8.63E-09
NW	2.41E-07	1.27E-07	8.37E-08	4.95E-08	3.42E-08	2.58E-08	2.05E-08	1.69E-08	1.43E-08	1.24E-08	1.09E-08
NNW	3.20E-07	1.68E-07	1.10E-07	6.48E-08	4.47E-08	3.36E-08	2.67E-08	2.19E-08	1.86E-08	1.60E-08	1.40E-08
N	9.42E-08	6.38E-08	1.73E-07	1.01E-07	6.98E-08	5.24E-08	4.16E-08	3.42E-08	2.89E-08	2.49E-08	2.18E-08
NNE	1.83E-07	1.23E-07	3.75E-07	2.21E-07	1.53E-07	1.15E-07	9.13E-08	7.52E-08	6.37E-08	5.50E-08	4.82E-08
NE	6.44E-07	3.36E-07	2.21E-07	1.29E-07	8.89E-08	6.67E-08	5.29E-08	4.35E-08	3.67E-08	3.16E-08	2.77E-08
ENE	2.52E-07	1.31E-07	8.57E-08	5.01E-08	3.44E-08	2.57E-08	2.04E-08	1.67E-08	1.41E-08	1.22E-08	1.06E-08
E	1.64E-07	8.42E-08	5.48E-08	3.17E-08	2.16E-08	1.61E-08	1.27E-08	1.04E-08	8.72E-09	7.49E-09	6.54E-09
ESE	1.16E-07	5.99E-08	3.91E-08	2.28E-08	1.56E-08	1.17E-08	9.22E-09	7.57E-09	6.38E-09	5.50E-09	4.81E-09
SE	1.25E-07	6.40E-08	4.14E-08	2.38E-08	1.61E-08	1.20E-08	9.38E-09	7.65E-09	6.42E-09	5.50E-09	4.80E-09
SSE	2.13E-07	1.10E-07	7.20E-08	4.19E-08	2.87E-08	2.14E-08	1.69E-08	1.39E-08	1.17E-08	1.01E-08	8.83E-09

TABLE 2.3-324

BLN COL 2.3-5

ANNUAL AVERAGE χ/Q (sec/m^3) FOR NO DECAY, UNDEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	1.06E-06	1.33E-06	1.33E-06	6.66E-07	4.12E-07	1.83E-07	6.59E-08	3.28E-08	2.10E-08	1.51E-08
SSW	1.45E-06	8.77E-07	5.33E-07	5.59E-07	7.15E-07	3.27E-07	1.18E-07	5.84E-08	3.72E-08	2.67E-08
SW	1.40E-06	8.39E-07	5.12E-07	3.46E-07	2.69E-07	2.99E-07	1.49E-07	7.49E-08	4.82E-08	3.48E-08
WSW	3.29E-07	2.31E-07	1.58E-07	1.14E-07	8.69E-08	1.56E-07	8.76E-08	4.49E-08	2.93E-08	2.13E-08
W	1.01E-07	7.08E-08	5.42E-08	4.35E-08	3.61E-08	9.21E-08	5.36E-08	2.76E-08	1.80E-08	1.32E-08
WNW	1.35E-07	8.47E-08	5.52E-08	1.36E-07	2.33E-07	1.06E-07	4.01E-08	2.06E-08	1.35E-08	9.84E-09
NW	2.77E-07	1.29E-07	7.18E-08	2.21E-07	2.92E-07	1.33E-07	5.03E-08	2.60E-08	1.70E-08	1.24E-08
NNW	3.89E-07	2.00E-07	1.17E-07	3.11E-07	3.89E-07	1.76E-07	6.60E-08	3.38E-08	2.20E-08	1.60E-08
N	6.47E-07	3.22E-07	1.91E-07	1.36E-07	1.05E-07	1.19E-07	1.03E-07	5.27E-08	3.43E-08	2.49E-08
NNE	1.39E-06	6.54E-07	3.83E-07	2.68E-07	2.05E-07	2.48E-07	2.25E-07	1.16E-07	7.54E-08	5.50E-08
NE	1.03E-06	5.17E-07	6.46E-07	1.09E-06	7.81E-07	3.53E-07	1.32E-07	6.71E-08	4.36E-08	3.17E-08
ENE	6.17E-07	8.66E-07	9.75E-07	4.92E-07	3.07E-07	1.38E-07	5.10E-08	2.59E-08	1.68E-08	1.22E-08
E	4.58E-07	7.25E-07	6.43E-07	3.22E-07	1.99E-07	8.88E-08	3.23E-08	1.62E-08	1.04E-08	7.51E-09
ESE	3.68E-07	7.60E-07	4.52E-07	2.27E-07	1.41E-07	6.31E-08	2.32E-08	1.17E-08	7.59E-09	5.50E-09
SE	4.02E-07	8.50E-07	4.97E-07	2.48E-07	1.53E-07	6.76E-08	2.43E-08	1.20E-08	7.68E-09	5.51E-09
SSE	6.32E-07	1.38E-06	8.31E-07	4.18E-07	2.60E-07	1.16E-07	4.27E-08	2.16E-08	1.39E-08	1.01E-08

BLN COL 2.3-5

TABLE 2.3-325 (Sheet 1 of 6)
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.30E-06	1.28E-06	1.04E-06	8.00E-07	6.61E-07	2.01E-06	1.23E-06	8.38E-07	6.10E-07	4.66E-07	3.70E-07
SSW	4.49E-06	1.66E-06	1.41E-06	1.16E-06	8.69E-07	6.60E-07	5.16E-07	4.15E-07	3.43E-07	8.19E-07	7.13E-07
SW	4.53E-06	1.62E-06	1.36E-06	1.11E-06	8.30E-07	6.32E-07	4.96E-07	4.01E-07	3.33E-07	2.82E-07	2.44E-07
WSW	9.82E-07	3.62E-07	3.17E-07	2.78E-07	2.32E-07	1.89E-07	1.55E-07	1.30E-07	1.11E-07	9.60E-08	8.45E-08
W	3.04E-07	1.13E-07	9.78E-08	8.29E-08	6.94E-08	6.03E-08	5.32E-08	4.74E-08	4.27E-08	3.87E-08	3.54E-08
WNW	3.97E-07	1.52E-07	1.33E-07	1.10E-07	8.34E-08	6.57E-08	5.36E-08	4.50E-08	3.87E-08	2.83E-07	2.26E-07
NW	9.96E-07	3.62E-07	2.73E-07	1.92E-07	1.21E-07	8.71E-08	6.78E-08	5.54E-08	2.02E-07	3.53E-07	2.82E-07
NNW	1.22E-06	4.81E-07	3.88E-07	2.86E-07	1.90E-07	1.41E-07	1.12E-07	9.23E-08	3.01E-07	4.70E-07	3.75E-07
N	2.17E-06	8.34E-07	6.40E-07	4.59E-07	3.04E-07	2.29E-07	1.83E-07	1.52E-07	1.30E-07	1.14E-07	1.01E-07
NNE	5.40E-06	1.89E-06	1.34E-06	9.32E-07	6.14E-07	4.59E-07	3.65E-07	3.02E-07	2.56E-07	2.23E-07	1.96E-07
NE	3.84E-06	1.37E-06	9.96E-07	7.20E-07	4.92E-07	3.71E-07	2.93E-07	1.09E-06	1.21E-06	9.30E-07	7.42E-07
ENE	2.15E-06	8.13E-07	6.05E-07	4.31E-07	2.78E-07	1.48E-06	9.09E-07	6.20E-07	4.54E-07	3.48E-07	2.78E-07
E	1.40E-06	5.82E-07	4.54E-07	3.35E-07	6.03E-07	9.69E-07	5.93E-07	4.03E-07	2.94E-07	2.25E-07	1.79E-07
ESE	1.13E-06	4.81E-07	3.69E-07	2.59E-07	1.16E-06	6.77E-07	4.15E-07	2.83E-07	2.06E-07	1.58E-07	1.26E-07
SE	1.29E-06	4.94E-07	3.97E-07	2.98E-07	1.30E-06	7.46E-07	4.56E-07	3.09E-07	2.25E-07	1.72E-07	1.36E-07
SSE	2.10E-06	8.08E-07	6.26E-07	4.52E-07	2.09E-06	1.24E-06	7.63E-07	5.20E-07	3.79E-07	2.91E-07	2.31E-07

BLN COL 2.3-5

TABLE 2.3-325 (Sheet 2 of 6)
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.02E-07	1.48E-07	9.22E-08	4.99E-08	3.22E-08	2.30E-08	1.74E-08	1.37E-08	1.12E-08	9.33E-09	7.92E-09
SSW	5.83E-07	2.86E-07	1.79E-07	9.69E-08	6.28E-08	4.48E-08	3.40E-08	2.69E-08	2.19E-08	1.83E-08	1.56E-08
SW	2.54E-07	3.67E-07	2.31E-07	1.27E-07	8.30E-08	5.97E-08	4.56E-08	3.63E-08	2.97E-08	2.49E-08	2.13E-08
WSW	7.53E-08	2.15E-07	1.37E-07	7.63E-08	5.05E-08	3.66E-08	2.81E-08	2.25E-08	1.85E-08	1.56E-08	1.34E-08
W	3.26E-08	1.31E-07	8.38E-08	4.69E-08	3.11E-08	2.26E-08	1.74E-08	1.39E-08	1.15E-08	9.68E-09	8.31E-09
WNW	1.86E-07	9.36E-08	5.96E-08	3.32E-08	2.20E-08	1.59E-08	1.22E-08	9.77E-09	8.05E-09	6.78E-09	5.81E-09
NW	2.32E-07	1.17E-07	7.45E-08	4.16E-08	2.75E-08	1.99E-08	1.53E-08	1.23E-08	1.01E-08	8.51E-09	7.30E-09
NNW	3.08E-07	1.55E-07	9.81E-08	5.44E-08	3.59E-08	2.60E-08	1.99E-08	1.59E-08	1.31E-08	1.10E-08	9.40E-09
N	9.06E-08	6.15E-08	1.66E-07	9.25E-08	6.11E-08	4.43E-08	3.41E-08	2.72E-08	2.24E-08	1.89E-08	1.62E-08
NNE	1.76E-07	1.18E-07	3.62E-07	2.02E-07	1.34E-07	9.74E-08	7.50E-08	6.01E-08	4.96E-08	4.18E-08	3.59E-08
NE	6.09E-07	3.04E-07	1.93E-07	1.07E-07	6.99E-08	5.05E-08	3.86E-08	3.08E-08	2.53E-08	2.12E-08	1.81E-08
ENE	2.28E-07	1.13E-07	7.11E-08	3.90E-08	2.55E-08	1.83E-08	1.40E-08	1.11E-08	9.08E-09	7.61E-09	6.49E-09
E	1.46E-07	7.18E-08	4.49E-08	2.44E-08	1.58E-08	1.13E-08	8.58E-09	6.78E-09	5.53E-09	4.62E-09	3.93E-09
ESE	1.03E-07	5.08E-08	3.19E-08	1.74E-08	1.14E-08	8.14E-09	6.20E-09	4.92E-09	4.02E-09	3.36E-09	2.86E-09
SE	1.11E-07	5.42E-08	3.37E-08	1.82E-08	1.17E-08	8.32E-09	6.29E-09	4.96E-09	4.04E-09	3.36E-09	2.85E-09
SSE	1.89E-07	9.35E-08	5.87E-08	3.21E-08	2.09E-08	1.50E-08	1.14E-08	9.03E-09	7.38E-09	6.17E-09	5.26E-09

BLN COL 2.3-5

TABLE 2.3-325 (Sheet 3 of 6)
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

Release ID	Type of Location	Sector	Distance (miles)	Distance (meters)	X/Q (sec/m^3) No Decay Undepleted	X/Q (sec/m^3) No Decay Depleted	D/Q (m^{-2})
P	EAB	S	0.71	1145	1.10E-06	1.10E-06	1.10E-08
P	EAB	SSW	1.03	1660	1.20E-06	1.10E-06	5.90E-09
P	EAB	SW	0.78	1249	1.40E-06	1.30E-06	9.80E-09
P	EAB	WSW	0.73	1177	3.40E-07	3.10E-07	2.30E-09
P	EAB	W	0.59	949	1.10E-07	1.00E-07	9.60E-10
P	EAB	WNW	0.53	855	1.60E-07	1.40E-07	1.60E-09
P	EAB	NW	0.53	855	3.70E-07	3.40E-07	4.20E-09
P	EAB	NNW	0.54	866	4.90E-07	4.50E-07	5.60E-09
P	EAB	N	0.58	935	7.90E-07	7.30E-07	9.10E-09
P	EAB	NNE	0.77	1244	1.40E-06	1.30E-06	1.20E-08
P	EAB	NE	1.1	1769	7.00E-07	6.60E-07	4.50E-09
P	EAB	ENE	0.78	1250	6.20E-07	5.80E-07	6.40E-09
P	EAB	E	0.59	947	5.50E-07	5.10E-07	7.10E-09
P	EAB	ESE	0.58	927	4.60E-07	4.30E-07	6.30E-09
P	EAB	SE	0.58	927	4.80E-07	4.40E-07	5.40E-09
P	EAB	SSE	0.58	932	7.70E-07	7.10E-07	1.00E-08
P	GARDEN	SSW	4.56	7338	6.70E-07	6.80E-07	8.40E-10
P	GARDEN	SW	1.74	2807	7.40E-07	7.20E-07	2.00E-09

BLN COL 2.3-5

TABLE 2.3-325 (Sheet 4 of 6)
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

Release ID	Type of Location	Sector	Distance (miles)	Distance (meters)	X/Q (sec/m^3) No Decay Undepleted	X/Q (sec/m^3) No Decay Depleted	D/Q (m^{-2})
P	GARDEN	WSW	4.21	6780	8.90E-08	9.00E-08	8.20E-11
P	GARDEN	W	2.64	4244	5.10E-08	5.10E-08	7.00E-11
P	GARDEN	WNW	0.71	1143	1.40E-07	1.30E-07	1.20E-09
P	GARDEN	NW	0.8	1289	2.70E-07	2.50E-07	2.40E-09
P	GARDEN	NNW	1.13	1821	2.70E-07	2.50E-07	1.70E-09
P	GARDEN	N	2.06	3310	2.30E-07	2.20E-07	7.90E-10
P	GARDEN	NNE	1.25	2006	7.80E-07	7.40E-07	4.30E-09
P	GARDEN	NE	4.13	6648	8.50E-07	8.60E-07	7.50E-10
P	GARDEN	ENE	3.47	5588	4.70E-07	4.50E-07	5.50E-10
P	GARDEN	E	2.4	3861	6.50E-07	6.40E-07	9.90E-10
P	GARDEN	ESE	2.73	4388	3.50E-07	3.40E-07	5.50E-10
P	GARDEN	SE	4.48	7204	1.50E-07	1.40E-07	2.00E-10
P	MILK COW/GOAT	S	4.77	7681	3.50E-07	3.20E-07	4.70E-10
P	MILK COW/GOAT	WSW	4.6	7406	8.10E-08	8.10E-08	6.80E-11
P	MILK COW/GOAT	W	1.46	2348	7.20E-08	7.00E-08	2.30E-10
P	MILK COW/GOAT	WNW	0.75	1214	1.40E-07	1.30E-07	1.10E-09
P	MILK COW/GOAT	NW	0.99	1586	2.10E-07	1.90E-07	1.60E-09
P	MILK COW/GOAT	NNW	2.19	3520	1.30E-07	1.30E-07	4.10E-10

BLN COL 2.3-5

TABLE 2.3-325 (Sheet 5 of 6)
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

Release ID	Type of Location	Sector	Distance (miles)	Distance (meters)	X/Q (sec/m^3) No Decay Undepleted	X/Q (sec/m^3) No Decay Depleted	D/Q (m^{-2})
P	MILK COW/GOAT	N	2.12	3417	2.20E-07	2.10E-07	7.40E-10
P	MILK COW/GOAT	NNE	2.22	3571	4.30E-07	4.10E-07	1.20E-09
P	MILK COW/GOAT	NE	4.13	6648	8.50E-07	8.60E-07	7.50E-10
P	MILK COW/GOAT	ENE	3.81	6135	3.90E-07	3.80E-07	4.40E-10
P	MILK COW/GOAT	E	2.51	4036	5.90E-07	5.80E-07	8.90E-10
P	MILK COW/GOAT	ESE	2.71	4362	3.60E-07	3.50E-07	5.60E-10
P	HOUSE	W	1.46	2348	7.20E-08	7.00E-08	2.30E-10
P	HOUSE	WNW	0.73	1169	1.40E-07	1.30E-07	1.20E-09
P	HOUSE	NW	0.69	1103	3.10E-07	2.80E-07	3.10E-09
P	SCHOOL	WNW	2.64	4243	5.20E-08	5.10E-08	1.00E-10
P	ANIMAL FOR MEAT	S	4.77	7681	3.50E-07	3.20E-07	4.70E-10
P	ANIMAL FOR MEAT	WSW	4.6	7406	8.10E-08	8.10E-08	6.80E-11
P	ANIMAL FOR MEAT	W	1.46	2348	7.20E-08	7.00E-08	2.30E-10
P	ANIMAL FOR MEAT	WNW	0.75	1214	1.40E-07	1.30E-07	1.10E-09
P	ANIMAL FOR MEAT	NW	0.99	1586	2.10E-07	1.90E-07	1.60E-09
P	ANIMAL FOR MEAT	NNW	2.19	3520	1.30E-07	1.30E-07	4.10E-10
P	ANIMAL FOR MEAT	N	2.12	3417	2.20E-07	2.10E-07	7.40E-10

TABLE 2.3-325 (Sheet 6 of 6)
BLN COL 2.3-5
ANNUAL AVERAGE χ/Q VALUES FOR NO DECAY, DEPLETED

Release ID	Type of Location	Sector	Distance (miles)	Distance (meters)	X/Q (sec/m^3) No Decay Undepleted	X/Q (sec/m^3) No Decay Depleted	D/Q (m^{-2})
P	ANIMAL FOR MEAT	NNE	2.22	3571	4.30E-07	4.10E-07	1.20E-09
P	ANIMAL FOR MEAT	NE	4.13	6648	8.50E-07	8.60E-07	7.50E-10
P	ANIMAL FOR MEAT	ENE	3.81	6135	3.90E-07	3.80E-07	4.40E-10
P	ANIMAL FOR MEAT	E	2.4	3861	6.50E-07	6.40E-07	9.90E-10
P	ANIMAL FOR MEAT	ESE	2.71	4362	3.60E-07	3.50E-07	5.60E-10

BLN COL 2.3-5

TABLE 2.3-326 (Sheet 1 of 2)
ANNUAL AVERAGE χ/Q (SEC/M^3) FOR A 2.26 DAY DECAY, UNDEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

X/Q 2.26 day decay, undepleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.53E-06	1.38E-06	1.12E-06	8.43E-07	6.83E-07	2.01E-06	1.25E-06	8.55E-07	6.27E-07	4.83E-07	3.86E-07
SSW	4.82E-06	1.80E-06	1.51E-06	1.22E-06	8.96E-07	6.76E-07	5.26E-07	4.21E-07	3.47E-07	7.92E-07	6.91E-07
SW	4.86E-06	1.76E-06	1.45E-06	1.17E-06	8.57E-07	6.48E-07	5.06E-07	4.07E-07	3.36E-07	2.84E-07	2.44E-07
WSW	1.05E-06	3.92E-07	3.37E-07	2.90E-07	2.37E-07	1.91E-07	1.56E-07	1.30E-07	1.10E-07	9.48E-08	8.29E-08
W	3.26E-07	1.22E-07	1.04E-07	8.66E-08	7.10E-08	6.10E-08	5.32E-08	4.70E-08	4.18E-08	3.76E-08	3.41E-08
WNW	4.25E-07	1.64E-07	1.41E-07	1.15E-07	8.59E-08	6.72E-08	5.44E-08	4.54E-08	3.88E-08	2.66E-07	2.13E-07
NW	1.07E-06	3.94E-07	2.95E-07	2.05E-07	1.27E-07	9.11E-08	7.05E-08	5.72E-08	1.94E-07	3.32E-07	2.66E-07
NNW	1.31E-06	5.20E-07	4.15E-07	3.02E-07	1.99E-07	1.46E-07	1.15E-07	9.40E-08	2.89E-07	4.44E-07	3.55E-07
N	2.33E-06	9.02E-07	6.86E-07	4.87E-07	3.18E-07	2.37E-07	1.88E-07	1.55E-07	1.32E-07	1.14E-07	1.00E-07
NNE	5.79E-06	2.05E-06	1.45E-06	9.99E-07	6.46E-07	4.77E-07	3.76E-07	3.08E-07	2.60E-07	2.24E-07	1.96E-07
NE	4.12E-06	1.49E-06	1.08E-06	7.67E-07	5.15E-07	3.84E-07	3.01E-07	1.06E-06	1.17E-06	9.02E-07	7.23E-07
ENE	2.30E-06	8.80E-07	6.51E-07	4.57E-07	2.91E-07	1.46E-06	9.06E-07	6.24E-07	4.59E-07	3.55E-07	2.84E-07
E	1.49E-06	6.25E-07	4.84E-07	3.53E-07	6.08E-07	9.66E-07	5.99E-07	4.11E-07	3.02E-07	2.32E-07	1.86E-07
ESE	1.21E-06	5.15E-07	3.94E-07	2.73E-07	1.14E-06	6.78E-07	4.21E-07	2.89E-07	2.13E-07	1.64E-07	1.31E-07
SE	1.38E-06	5.35E-07	4.26E-07	3.15E-07	1.29E-06	7.53E-07	4.66E-07	3.20E-07	2.35E-07	1.81E-07	1.44E-07
SSE	2.25E-06	8.74E-07	6.71E-07	4.78E-07	2.07E-06	1.25E-06	7.73E-07	5.31E-07	3.91E-07	3.01E-07	2.41E-07

TABLE 2.3-326 (Sheet 2 of 2)

BLN COL 2.3-5

ANNUAL AVERAGE χ/Q (SEC/M^3) FOR A 2.26 DAY DECAY, UNDEPLETED FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

X/Q 2.26 day decay, undepleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.17E-07	1.57E-07	9.82E-08	5.29E-08	3.36E-08	2.34E-08	1.73E-08	1.32E-08	1.05E-08	8.45E-09	6.96E-09
SSW	5.67E-07	2.81E-07	1.76E-07	9.45E-08	6.01E-08	4.18E-08	3.09E-08	2.37E-08	1.87E-08	1.52E-08	1.25E-08
SW	2.51E-07	3.46E-07	2.17E-07	1.17E-07	7.46E-08	5.19E-08	3.82E-08	2.93E-08	2.31E-08	1.86E-08	1.53E-08
WSW	7.35E-08	1.96E-07	1.24E-07	6.71E-08	4.27E-08	2.97E-08	2.18E-08	1.66E-08	1.30E-08	1.05E-08	8.54E-09
W	3.11E-08	1.17E-07	7.34E-08	3.94E-08	2.48E-08	1.70E-08	1.23E-08	9.28E-09	7.19E-09	5.69E-09	4.58E-09
WNW	1.76E-07	8.81E-08	5.55E-08	2.99E-08	1.89E-08	1.30E-08	9.48E-09	7.17E-09	5.58E-09	4.44E-09	3.59E-09
NW	2.19E-07	1.10E-07	6.93E-08	3.73E-08	2.36E-08	1.62E-08	1.18E-08	8.91E-09	6.93E-09	5.50E-09	4.45E-09
NNW	2.92E-07	1.46E-07	9.18E-08	4.93E-08	3.11E-08	2.14E-08	1.56E-08	1.18E-08	9.18E-09	7.30E-09	5.92E-09
N	8.93E-08	5.83E-08	1.45E-07	7.82E-08	4.95E-08	3.42E-08	2.50E-08	1.90E-08	1.49E-08	1.19E-08	9.68E-09
NNE	1.74E-07	1.13E-07	3.15E-07	1.70E-07	1.08E-07	7.49E-08	5.48E-08	4.17E-08	3.27E-08	2.62E-08	2.13E-08
NE	5.96E-07	2.99E-07	1.89E-07	1.03E-07	6.55E-08	4.57E-08	3.38E-08	2.59E-08	2.04E-08	1.65E-08	1.36E-08
ENE	2.34E-07	1.17E-07	7.36E-08	3.98E-08	2.54E-08	1.77E-08	1.30E-08	9.97E-09	7.86E-09	6.34E-09	5.20E-09
E	1.52E-07	7.56E-08	4.74E-08	2.55E-08	1.62E-08	1.13E-08	8.28E-09	6.34E-09	5.00E-09	4.03E-09	3.31E-09
ESE	1.08E-07	5.37E-08	3.37E-08	1.82E-08	1.16E-08	8.06E-09	5.94E-09	4.54E-09	3.58E-09	2.89E-09	2.37E-09
SE	1.18E-07	5.88E-08	3.69E-08	2.00E-08	1.28E-08	9.01E-09	6.70E-09	5.18E-09	4.13E-09	3.37E-09	2.79E-09
SSE	1.98E-07	9.85E-08	6.19E-08	3.33E-08	2.12E-08	1.47E-08	1.08E-08	8.27E-09	6.50E-09	5.23E-09	4.29E-09

BLN COL 2.3-5

TABLE 2.3-327

ANNUAL AVERAGE χ/Q (SEC/M^3) FOR A 2.26 DAY DECAY, UNDEPLETED
AT EACH 22.5° SECTOR FOR EACH SEGMENT (MILES) SHOWN AT THE TOP

Sector	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	1.05E-06	1.31E-06	1.29E-06	6.37E-07	3.89E-07	1.66E-07	5.44E-08	2.37E-08	1.33E-08	8.49E-09
SSW	1.45E-06	8.70E-07	5.24E-07	5.38E-07	6.75E-07	2.98E-07	9.73E-08	4.23E-08	2.39E-08	1.52E-08
SW	1.39E-06	8.33E-07	5.04E-07	3.37E-07	2.59E-07	2.68E-07	1.21E-07	5.25E-08	2.95E-08	1.87E-08
WSW	3.28E-07	2.29E-07	1.55E-07	1.10E-07	8.29E-08	1.37E-07	6.89E-08	3.00E-08	1.67E-08	1.05E-08
W	1.00E-07	7.00E-08	5.28E-08	4.17E-08	3.40E-08	7.84E-08	4.05E-08	1.72E-08	9.35E-09	5.72E-09
WNW	1.35E-07	8.40E-08	5.42E-08	1.27E-07	2.15E-07	9.31E-08	3.07E-08	1.32E-08	7.22E-09	4.46E-09
NW	2.77E-07	1.28E-07	7.07E-08	2.08E-07	2.68E-07	1.16E-07	3.83E-08	1.64E-08	8.98E-09	5.53E-09
NNW	3.88E-07	1.98E-07	1.15E-07	2.92E-07	3.58E-07	1.55E-07	5.07E-08	2.17E-08	1.19E-08	7.35E-09
N	6.46E-07	3.20E-07	1.88E-07	1.32E-07	1.00E-07	1.04E-07	8.04E-08	3.46E-08	1.92E-08	1.20E-08
NNE	1.38E-06	6.49E-07	3.76E-07	2.60E-07	1.96E-07	2.16E-07	1.75E-07	7.57E-08	4.20E-08	2.63E-08
NE	1.03E-06	5.13E-07	6.26E-07	1.04E-06	7.29E-07	3.16E-07	1.05E-07	4.62E-08	2.61E-08	1.66E-08
ENE	6.16E-07	8.46E-07	9.40E-07	4.67E-07	2.86E-07	1.24E-07	4.09E-08	1.79E-08	1.00E-08	6.37E-09
E	4.57E-07	7.10E-07	6.22E-07	3.06E-07	1.87E-07	8.01E-08	2.62E-08	1.14E-08	6.38E-09	4.05E-09
ESE	3.67E-07	7.44E-07	4.37E-07	2.16E-07	1.32E-07	5.68E-08	1.87E-08	8.15E-09	4.58E-09	2.90E-09
SE	4.01E-07	8.35E-07	4.84E-07	2.38E-07	1.46E-07	6.23E-08	2.06E-08	9.10E-09	5.22E-09	3.38E-09
SSE	6.30E-07	1.35E-06	8.02E-07	3.97E-07	2.43E-07	1.04E-07	3.43E-08	1.49E-08	8.32E-09	5.26E-09

TABLE 2.3-328 (Sheet 1 of 2)

BLN COL 2.3-5

ANNUAL AVERAGE χ/Q (SEC/M^3) FOR AN 8.00 DAY DECAY, DEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

X/Q 8 day decay, depleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.30E-06	1.28E-06	1.04E-06	7.99E-07	6.60E-07	2.00E-06	1.22E-06	8.28E-07	6.02E-07	4.59E-07	3.64E-07
SSW	4.49E-06	1.66E-06	1.41E-06	1.16E-06	8.67E-07	6.58E-07	5.13E-07	4.12E-07	3.40E-07	8.07E-07	7.01E-07
SW	4.53E-06	1.62E-06	1.36E-06	1.11E-06	8.28E-07	6.30E-07	4.93E-07	3.98E-07	3.30E-07	2.80E-07	2.41E-07
WSW	9.82E-07	3.61E-07	3.16E-07	2.78E-07	2.31E-07	1.88E-07	1.54E-07	1.29E-07	1.10E-07	9.48E-08	8.33E-08
W	3.04E-07	1.13E-07	9.77E-08	8.28E-08	6.92E-08	6.00E-08	5.28E-08	4.70E-08	4.22E-08	3.81E-08	3.48E-08
WNW	3.97E-07	1.52E-07	1.33E-07	1.10E-07	8.33E-08	6.55E-08	5.33E-08	4.47E-08	3.83E-08	2.77E-07	2.21E-07
NW	9.96E-07	3.62E-07	2.73E-07	1.91E-07	1.20E-07	8.68E-08	6.75E-08	5.51E-08	1.99E-07	3.45E-07	2.75E-07
NNW	1.22E-06	4.81E-07	3.88E-07	2.86E-07	1.90E-07	1.41E-07	1.11E-07	9.16E-08	2.97E-07	4.60E-07	3.67E-07
N	2.17E-06	8.34E-07	6.39E-07	4.59E-07	3.04E-07	2.28E-07	1.82E-07	1.51E-07	1.29E-07	1.12E-07	9.95E-08
NNE	5.40E-06	1.89E-06	1.34E-06	9.32E-07	6.12E-07	4.57E-07	3.63E-07	2.99E-07	2.54E-07	2.20E-07	1.94E-07
NE	3.84E-06	1.37E-06	9.96E-07	7.19E-07	4.91E-07	3.70E-07	2.92E-07	1.08E-06	1.19E-06	9.13E-07	7.27E-07
ENE	2.15E-06	8.13E-07	6.05E-07	4.30E-07	2.78E-07	1.46E-06	8.99E-07	6.12E-07	4.47E-07	3.42E-07	2.72E-07
E	1.40E-06	5.81E-07	4.53E-07	3.35E-07	6.01E-07	9.61E-07	5.87E-07	3.98E-07	2.90E-07	2.21E-07	1.76E-07
ESE	1.13E-06	4.80E-07	3.69E-07	2.59E-07	1.15E-06	6.71E-07	4.11E-07	2.79E-07	2.03E-07	1.56E-07	1.23E-07
SE	1.29E-06	4.94E-07	3.97E-07	2.98E-07	1.29E-06	7.42E-07	4.52E-07	3.06E-07	2.22E-07	1.70E-07	1.34E-07
SSE	2.10E-06	8.08E-07	6.25E-07	4.51E-07	2.08E-06	1.23E-06	7.55E-07	5.13E-07	3.74E-07	2.86E-07	2.27E-07

TABLE 2.3-328 (Sheet 2 of 2)

BLN COL 2.3-5

ANNUAL AVERAGE χ/Q (SEC/M^3) FOR AN 8.00 DAY DECAY, DEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

X/Q 8 day decay, depleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	2.97E-07	1.44E-07	8.87E-08	4.70E-08	2.98E-08	2.08E-08	1.54E-08	1.19E-08	9.52E-09	7.77E-09	6.47E-09
SSW	5.72E-07	2.78E-07	1.72E-07	9.14E-08	5.80E-08	4.06E-08	3.02E-08	2.34E-08	1.87E-08	1.53E-08	1.28E-08
SW	2.51E-07	3.56E-07	2.22E-07	1.19E-07	7.62E-08	5.36E-08	4.00E-08	3.11E-08	2.50E-08	2.05E-08	1.71E-08
WSW	7.41E-08	2.08E-07	1.31E-07	7.10E-08	4.58E-08	3.24E-08	2.43E-08	1.90E-08	1.53E-08	1.26E-08	1.05E-08
W	3.19E-08	1.26E-07	7.93E-08	4.31E-08	2.78E-08	1.96E-08	1.47E-08	1.14E-08	9.18E-09	7.53E-09	6.29E-09
WNW	1.81E-07	9.00E-08	5.65E-08	3.07E-08	1.97E-08	1.39E-08	1.04E-08	8.10E-09	6.50E-09	5.33E-09	4.45E-09
NW	2.26E-07	1.12E-07	7.05E-08	3.83E-08	2.47E-08	1.74E-08	1.30E-08	1.01E-08	8.12E-09	6.65E-09	5.55E-09
NNW	3.00E-07	1.49E-07	9.30E-08	5.03E-08	3.23E-08	2.27E-08	1.70E-08	1.32E-08	1.06E-08	8.65E-09	7.21E-09
N	8.91E-08	5.98E-08	1.58E-07	8.58E-08	5.52E-08	3.90E-08	2.93E-08	2.28E-08	1.83E-08	1.51E-08	1.26E-08
NNE	1.73E-07	1.15E-07	3.44E-07	1.87E-07	1.21E-07	8.57E-08	6.44E-08	5.03E-08	4.05E-08	3.33E-08	2.79E-08
NE	5.96E-07	2.94E-07	1.84E-07	9.95E-08	6.39E-08	4.51E-08	3.37E-08	2.63E-08	2.11E-08	1.73E-08	1.45E-08
ENE	2.23E-07	1.09E-07	6.80E-08	3.65E-08	2.33E-08	1.64E-08	1.22E-08	9.47E-09	7.58E-09	6.21E-09	5.17E-09
E	1.43E-07	6.96E-08	4.31E-08	2.29E-08	1.45E-08	1.02E-08	7.54E-09	5.83E-09	4.66E-09	3.80E-09	3.17E-09
ESE	1.01E-07	4.92E-08	3.06E-08	1.63E-08	1.04E-08	7.29E-09	5.42E-09	4.21E-09	3.36E-09	2.75E-09	2.29E-09
SE	1.09E-07	5.29E-08	3.26E-08	1.73E-08	1.09E-08	7.64E-09	5.68E-09	4.40E-09	3.52E-09	2.88E-09	2.40E-09
SSE	1.85E-07	9.05E-08	5.62E-08	3.00E-08	1.91E-08	1.34E-08	9.93E-09	7.70E-09	6.15E-09	5.03E-09	4.19E-09

BLN COL 2.3-5

TABLE 2.3-329

ANNUAL AVERAGE χ/Q (SEC/M^3) FOR AN 8.00 DAY DECAY, DEPLETED
FOR EACH 22.5° SECTOR AT THE DISTANCES (MILES) SHOWN AT THE TOP

Sector	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	9.86E-07	1.29E-06	1.27E-06	6.12E-07	3.67E-07	1.53E-07	4.86E-08	2.10E-08	1.20E-08	7.81E-09
SSW	1.36E-06	8.40E-07	5.11E-07	5.39E-07	6.84E-07	2.96E-07	9.44E-08	4.11E-08	2.36E-08	1.54E-08
SW	1.30E-06	8.02E-07	4.92E-07	3.30E-07	2.56E-07	2.73E-07	1.23E-07	5.42E-08	3.13E-08	2.06E-08
WSW	3.09E-07	2.22E-07	1.53E-07	1.09E-07	8.33E-08	1.44E-07	7.30E-08	3.27E-08	1.91E-08	1.26E-08
W	9.44E-08	6.81E-08	5.24E-08	4.20E-08	3.47E-08	8.43E-08	4.43E-08	1.98E-08	1.15E-08	7.56E-09
WNW	1.27E-07	8.13E-08	5.31E-08	1.31E-07	2.23E-07	9.54E-08	3.15E-08	1.41E-08	8.15E-09	5.35E-09
NW	2.57E-07	1.21E-07	6.77E-08	2.14E-07	2.78E-07	1.19E-07	3.94E-08	1.76E-08	1.02E-08	6.68E-09
NNW	3.63E-07	1.89E-07	1.11E-07	3.00E-07	3.70E-07	1.58E-07	5.18E-08	2.30E-08	1.33E-08	8.69E-09
N	6.02E-07	3.05E-07	1.82E-07	1.29E-07	9.95E-08	1.10E-07	8.83E-08	3.95E-08	2.30E-08	1.51E-08
NNE	1.28E-06	6.14E-07	3.63E-07	2.54E-07	1.94E-07	2.30E-07	1.93E-07	8.66E-08	5.06E-08	3.34E-08
NE	9.56E-07	4.88E-07	6.27E-07	1.05E-06	7.34E-07	3.12E-07	1.03E-07	4.56E-08	2.64E-08	1.74E-08
ENE	5.73E-07	8.38E-07	9.35E-07	4.54E-07	2.75E-07	1.16E-07	3.76E-08	1.66E-08	9.53E-09	6.23E-09
E	4.29E-07	7.02E-07	6.11E-07	2.95E-07	1.77E-07	7.42E-08	2.37E-08	1.03E-08	5.87E-09	3.82E-09
ESE	3.45E-07	7.39E-07	4.28E-07	2.07E-07	1.25E-07	5.24E-08	1.69E-08	7.37E-09	4.23E-09	2.76E-09
SE	3.74E-07	8.26E-07	4.71E-07	2.26E-07	1.36E-07	5.64E-08	1.79E-08	7.74E-09	4.43E-09	2.89E-09
SSE	5.89E-07	1.34E-06	7.86E-07	3.80E-07	2.29E-07	9.63E-08	3.10E-08	1.35E-08	7.75E-09	5.05E-09

TABLE 2.3-330 (Sheet 1 of 2)

BLN COL 2.3-5

D/Q (M^{-2}) AT EACH 22.5° SECTOR FOR EACH DISTANCE (MILES)

D/Q SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.73E-08	1.66E-08	1.07E-08	6.10E-09	2.83E-09	3.44E-09	2.03E-09	1.33E-09	9.33E-10	6.92E-10	5.33E-10
SSW	3.50E-08	1.59E-08	1.06E-08	6.27E-09	2.82E-09	1.59E-09	1.01E-09	7.03E-10	5.14E-10	9.80E-10	8.66E-10
SW	3.50E-08	1.56E-08	1.04E-08	6.09E-09	2.72E-09	1.52E-09	9.65E-10	6.66E-10	4.86E-10	3.69E-10	2.89E-10
WSW	7.02E-09	3.24E-09	2.23E-09	1.36E-09	6.29E-10	3.60E-10	2.33E-10	1.62E-10	1.19E-10	9.11E-11	7.15E-11
W	2.36E-09	1.12E-09	7.77E-10	4.70E-10	2.15E-10	1.22E-10	7.79E-11	5.40E-11	3.96E-11	3.01E-11	2.36E-11
WNW	3.58E-09	1.68E-09	1.15E-09	6.88E-10	3.13E-10	1.78E-10	1.14E-10	7.92E-11	5.80E-11	1.49E-10	1.15E-10
NW	1.10E-08	4.51E-09	2.77E-09	1.53E-09	6.43E-10	3.48E-10	2.17E-10	1.47E-10	1.33E-10	2.15E-10	1.65E-10
NNW	1.36E-08	6.09E-09	3.83E-09	2.14E-09	9.14E-10	4.98E-10	3.12E-10	2.13E-10	1.94E-10	3.18E-10	2.45E-10
N	2.46E-08	1.09E-08	6.77E-09	3.72E-09	1.56E-09	8.36E-10	5.18E-10	3.52E-10	2.54E-10	1.92E-10	1.50E-10
NNE	5.51E-08	2.24E-08	1.32E-08	6.97E-09	2.82E-09	1.49E-09	9.15E-10	6.17E-10	4.44E-10	3.34E-10	2.60E-10
NE	4.23E-08	1.73E-08	1.03E-08	5.49E-09	2.25E-09	1.19E-09	7.36E-10	6.12E-10	1.08E-09	8.03E-10	6.19E-10
ENE	2.81E-08	1.16E-08	6.90E-09	3.70E-09	1.52E-09	1.98E-09	1.17E-09	7.64E-10	5.38E-10	3.98E-10	3.07E-10
E	1.93E-08	8.64E-09	5.21E-09	2.85E-09	1.55E-09	1.53E-09	8.98E-10	5.88E-10	4.14E-10	3.07E-10	2.36E-10
ESE	1.65E-08	7.46E-09	4.52E-09	2.47E-09	2.25E-09	1.15E-09	6.77E-10	4.43E-10	3.12E-10	2.31E-10	1.78E-10
SE	1.44E-08	6.27E-09	4.04E-09	2.31E-09	2.49E-09	1.30E-09	7.65E-10	5.01E-10	3.53E-10	2.61E-10	2.01E-10
SSE	2.79E-08	1.21E-08	7.55E-09	4.20E-09	3.75E-09	1.96E-09	1.16E-09	7.57E-10	5.32E-10	3.95E-10	3.04E-10

TABLE 2.3-330 (Sheet 2 of 2)

BLN COL 2.3-5

D/Q (M^{-2}) AT EACH 22.5° SECTOR FOR EACH DISTANCE (MILES)

D/Q SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	4.24E-10	1.88E-10	1.14E-10	5.76E-11	3.49E-11	2.34E-11	1.68E-11	1.26E-11	9.78E-12	7.81E-12	6.38E-12
SSW	6.89E-10	3.06E-10	1.85E-10	9.37E-11	5.67E-11	3.80E-11	2.72E-11	2.05E-11	1.59E-11	1.27E-11	1.04E-11
SW	2.46E-10	3.19E-10	1.93E-10	9.77E-11	5.91E-11	3.96E-11	2.84E-11	2.13E-11	1.66E-11	1.32E-11	1.08E-11
WSW	5.74E-11	1.06E-10	6.43E-11	3.25E-11	1.97E-11	1.32E-11	9.44E-12	7.09E-12	5.51E-12	4.40E-12	3.60E-12
W	1.90E-11	4.31E-11	2.61E-11	1.32E-11	7.98E-12	5.35E-12	3.83E-12	2.88E-12	2.24E-12	1.79E-12	1.46E-12
WNW	9.13E-11	4.06E-11	2.46E-11	1.24E-11	7.52E-12	5.04E-12	3.61E-12	2.71E-12	2.11E-12	1.69E-12	1.38E-12
NW	1.31E-10	5.84E-11	3.54E-11	1.79E-11	1.08E-11	7.25E-12	5.20E-12	3.90E-12	3.04E-12	2.42E-12	1.98E-12
NNW	1.95E-10	8.65E-11	5.24E-11	2.65E-11	1.60E-11	1.08E-11	7.70E-12	5.78E-12	4.50E-12	3.59E-12	2.93E-12
N	1.20E-10	5.46E-11	9.38E-11	4.74E-11	2.87E-11	1.92E-11	1.38E-11	1.04E-11	8.05E-12	6.43E-12	5.25E-12
NNE	2.08E-10	9.42E-11	1.95E-10	9.84E-11	5.95E-11	3.99E-11	2.86E-11	2.15E-11	1.67E-11	1.33E-11	1.09E-11
NE	4.92E-10	2.18E-10	1.32E-10	6.69E-11	4.05E-11	2.71E-11	1.95E-11	1.46E-11	1.14E-11	9.07E-12	7.40E-12
ENE	2.44E-10	1.08E-10	6.56E-11	3.32E-11	2.01E-11	1.35E-11	9.65E-12	7.24E-12	5.63E-12	4.50E-12	3.67E-12
E	1.88E-10	8.34E-11	5.05E-11	2.55E-11	1.55E-11	1.04E-11	7.42E-12	5.57E-12	4.33E-12	3.46E-12	2.83E-12
ESE	1.42E-10	6.28E-11	3.81E-11	1.92E-11	1.16E-11	7.81E-12	5.60E-12	4.20E-12	3.27E-12	2.61E-12	2.13E-12
SE	1.60E-10	7.10E-11	4.30E-11	2.18E-11	1.32E-11	8.83E-12	6.33E-12	4.75E-12	3.69E-12	2.95E-12	2.41E-12
SSE	2.42E-10	1.07E-10	6.50E-11	3.29E-11	1.99E-11	1.33E-11	9.55E-12	7.17E-12	5.58E-12	4.46E-12	3.64E-12

BLN COL 2.3-5

TABLE 2.3-331
D/Q (M^{-2}) AT EACH 22.5° SECTOR FOR EACH DISTANCE (MILES)

Sector	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	9.97E-09	3.83E-09	2.12E-09	9.54E-10	5.40E-10	2.08E-10	6.00E-11	2.38E-11	1.27E-11	7.86E-12
SSW	9.87E-09	3.04E-09	1.04E-09	7.45E-10	8.34E-10	3.37E-10	9.76E-11	3.87E-11	2.07E-11	1.28E-11
SW	9.64E-09	2.93E-09	9.92E-10	4.93E-10	2.97E-10	2.47E-10	1.02E-10	4.03E-11	2.15E-11	1.33E-11
WSW	2.07E-09	6.72E-10	2.39E-10	1.21E-10	7.21E-11	7.67E-11	3.38E-11	1.34E-11	7.16E-12	4.43E-12
W	7.16E-10	2.30E-10	8.00E-11	4.01E-11	2.38E-11	3.02E-11	1.37E-11	5.44E-12	2.91E-12	1.80E-12
WNW	1.06E-09	3.36E-10	1.17E-10	9.88E-11	1.16E-10	4.48E-11	1.30E-11	5.13E-12	2.74E-12	1.70E-12
NW	2.61E-09	7.09E-10	2.24E-10	1.68E-10	1.67E-10	6.44E-11	1.86E-11	7.38E-12	3.94E-12	2.44E-12
NNW	3.58E-09	1.00E-09	3.22E-10	2.47E-10	2.48E-10	9.54E-11	2.76E-11	1.09E-11	5.84E-12	3.62E-12
N	6.33E-09	1.72E-09	5.36E-10	2.58E-10	1.51E-10	8.66E-11	4.94E-11	1.96E-11	1.05E-11	6.47E-12
NNE	1.25E-08	3.15E-09	9.50E-10	4.51E-10	2.63E-10	1.64E-10	1.03E-10	4.06E-11	2.17E-11	1.34E-11
NE	9.71E-09	2.50E-09	8.08E-10	8.42E-10	6.26E-10	2.41E-10	6.97E-11	2.76E-11	1.48E-11	9.13E-12
ENE	6.53E-09	2.21E-09	1.22E-09	5.49E-10	3.11E-10	1.20E-10	3.46E-11	1.37E-11	7.32E-12	4.53E-12
E	4.92E-09	1.83E-09	9.41E-10	4.23E-10	2.39E-10	9.19E-11	2.66E-11	1.05E-11	5.63E-12	3.48E-12
ESE	4.26E-09	1.81E-09	7.09E-10	3.19E-10	1.80E-10	6.93E-11	2.01E-11	7.95E-12	4.24E-12	2.63E-12
SE	3.76E-09	1.92E-09	8.02E-10	3.60E-10	2.04E-10	7.83E-11	2.27E-11	8.98E-12	4.80E-12	2.97E-12
SSE	7.06E-09	3.06E-09	1.21E-09	5.44E-10	3.08E-10	1.18E-10	3.42E-11	1.36E-11	7.25E-12	4.49E-12

2.4 HYDROLOGIC ENGINEERING

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Subsection 2.4.1 of the DCD is renumbered as Subsection 2.4.15. This is being done to accommodate the incorporation of Regulatory Guide 1.206 numbering conventions for Section 2.4.

STD DEP 1.1-1

2.4.1 HYDROLOGIC DESCRIPTION

BLN COL 2.4-1

2.4.1.1 Site and Facilities

The Bellefonte Nuclear Plant, Units 3 and 4, (BLN) is located on a peninsula formed by the Town Creek embayment on the western shore of Guntersville Reservoir at Tennessee river mile (TRM) 391.5, about 7 mi. northeast of Scottsboro in Jackson County, Alabama. The BLN is located approximately 3 mi. east of Hollywood, Alabama, and (43 river mi. upstream of Guntersville Dam (Figure 2.4.1-201).

The BLN is located northeast of Bellefonte Units 1 and 2 as shown in Figure 2.4.1-202. The BLN uses the existing natural-draft towers for circulating-water-system cooling and mechanical draft towers for service-water-system cooling, with makeup water coming from the Tennessee River/Guntersville Reservoir.

The peninsula has elevations varying from approximately 594 ft. above mean sea level (msl) along the banks of the Town Creek embayment to 830 ft. above msl along the hilltops of River Ridge, which forms the southeastern border of the peninsula. The elevation of the planned development area northwest of River Ridge is between 600 ft., at the Town Creek embayment, and 670 ft. above msl at the base of River Ridge. The standard plant-floor elevation of the safety-related facilities is established at 628.6 ft. above msl. The center of the nonsafety-related natural-draft cooling towers is located about 2400 ft. to the southwest of the reactor buildings at a grade elevation of 627 ft. above msl (Figure 2.4.1-202). Locations and topographic profiles showing the relationship between the BLN site and the Tennessee River Valley/Guntersville Reservoir are illustrated on Figures 2.4.1-201 and 2.4.1-202. Grading and drainage improvements are illustrated on Figure 2.4.2-202.

The Guntersville Reservoir, the principal source of makeup water for the cooling-tower system, is discussed in detail in Subsection 2.4.1.2. Makeup water is withdrawn through an inlet channel located at TRM 392.1 and pumped to the site via a pipeline. Blowdown water from the cooling-water system is expected to be discharged through a separate pipeline to the Guntersville Reservoir about 4600 ft. downstream from the intake structure at TRM 391.2 (Figure 2.4.1-202).

The plant arrangement is composed of five principal building structures: the nuclear island, turbine building, annex building, diesel generator building, and radwaste building as described in DCD Section 1.2. Of the five principal structures, only the nuclear island is designed to Category I seismic requirements and contains safety-related equipment for accident mitigation. The nuclear island consists of a free-standing steel containment building, a concrete shield building, and an auxiliary building. Floor elevation of the nuclear island is set at 628.6 ft. above msl. The locations of these safety-related components are shown on Figure 2.1-201. The elevation for the BLN facilities and accesses are listed in Table 2.4.1-201.

The majority of the natural surface runoff surrounding the BLN site area flows in a north or northwesterly direction into the Town Creek drainage basin with a minor amount of flow along natural gaps in River Ridge into the Tennessee River/ Guntersville Reservoir. At the location of the plant facilities, the surface drainage is directed to the yard holding pond and probable maximum precipitation ditch. Runoff collected in the yard holding pond and probable maximum precipitation ditch drains by overflow weirs or sheet flow into the Town Creek embayment. A small amount of surface runoff on the northeast side of the plant facilities flows along the natural gap and piping grade towards the inlet structure and into Guntersville Reservoir. The higher topography of River Ridge to the southeast of the plant site area directs surface-water flow from the northwestern slopes of River Ridge towards the inlet structure, or southwest towards the natural gap leading to the barge loading dock and into the Guntersville Reservoir. A description of the site grading and earthwork is presented in Subsection 2.4.2.3.

A bathymetric survey was conducted on September 25-27, 2006, in the Tennessee River, and in the vicinity of the intake and discharge structures. Figure 2.4.1-203 depicts water depth obtained from the bathymetric survey within the adjacent portions of the Tennessee River and in the intake channel. Water temperatures were taken at the surface, then at 10-ft. increments to a depth of 20 ft. where allowable, due to the total depth of the water at that location. Water-velocity measurements were taken at the surface, then at 5-ft. increments to a depth of 4.6 m (15 ft.) where allowable, due to the total depth of the water at that location. In general, temperature did not vary with depth.

Soil characteristics are discussed in Subsection 2.5.4 and land-use maps are provided in Section 2.1.

2.4.1.2 Hydrosphere

The BLN is located in the Guntersville watershed, U.S. Geological Survey (USGS) hydrologic unit code 06030001, one of 32 watersheds in Region 06 – Tennessee River watershed (Figure 2.4.1-204). The Guntersville watershed incorporates portions of Marion, Franklin, and Grundy counties in Tennessee and Jackson, Marshall, and Dekalb counties in Alabama.

The Tennessee River has been identified as the most intensively used river in the country; however, about 94 percent of the water taken from the river is returned to the system and reused downstream, making the region one of the lowest water consumers in the United States. About 12 billion gal. of water are taken from the river system each day. In 2000, 84 percent of that water was used for cooling at power plants with greater than 99 percent of the cooling water returned to the river. The other withdrawals were for industrial use (10 percent), public supply (5 percent), and irrigation (<1 percent) (Reference 228).

2.4.1.2.1 Tennessee River/Guntersville Reservoir

The Tennessee River system is the nation's fifth largest river system with a drainage area of 40,910 sq. mi. (Reference 241) and a length of approximately 652 mi. (Reference 225). At the BLN, the Tennessee River is approximately 3400 ft. wide with depths up to 30 ft. at normal pool elevation. Navigation is provided by maintaining a minimum channel depth of 11 ft. Flow is generally toward the southwest. The average flow rate of the Tennessee River at the BLN is 38,850 cfs. The drainage area of the Tennessee River at Nickajack Dam, 33 mi. upstream, is 21,870 sq. mi. Downstream from the BLN at Guntersville Dam, the drainage area is 24,450 sq. mi.

There are currently 30 major reservoirs in the TVA system upstream from the BLN, 11 of which provide nearly 5 million ac.-ft. of reserved flood-detention capacity during the main flood season (Reference 223). Reservoirs, dams, dam construction, reservoir operations, and modeling data are discussed in Subsection 2.4.4. Information for the nine primary dams along the Tennessee River upstream and downstream on the BLN site are tabulated in Table 2.4.1-203 (Reference 227).

The Guntersville Reservoir is approximately 76 mi. long and provides almost 890 mi. of shoreline. Guntersville Reservoir is the second largest reservoir on the Tennessee River with 67,900 ac. of water surface and a normal maximum pool volume of 1,018,000 ac.-ft. Because a certain water depth must be maintained for river navigation, Guntersville is one of the most stable TVA reservoirs, fluctuating only two ft. between its normal minimum pool in the winter and maximum pool in the summer. When the TVA established the stairway of dams and locks that turned the Tennessee River into a river highway 652 mi. long, the rural town of Guntersville was transformed into a major port. Several large companies now have terminals at Guntersville for processing and distributing grain, petroleum, and wood products.

Elevation-storage relationships for Guntersville Reservoir and Nickajack Reservoir are shown on Figures 2.4.1-205 and 2.4.1-206, respectively. Curves determined at selected years as part of the TVA's program of monitoring changes due to sedimentation are also shown. Actual sediment deposits in 14 flood-detention reservoirs were reported to have reduced total reservoir capacity by only 1.8 percent, or 226,000 ac.-ft. between dam closures and 1961. Projection to the year 2020 shows an additional 300,000 ac.-ft. of accumulation or an additional 2.4 percent reduction in total capacity; however, less than 2 percent of the sediment deposits are within the reserved flood-detention capacity of the reservoirs. Thus, sediment deposits are not expected to significantly reduce the flood-detention capacity of the reservoirs.

2.4.1.2.2 Town Creek

Town Creek begins about 2.5 mi. southwest of the BLN and flows northeastward into Guntersville Reservoir at TRM 393.5 via the Town Creek embayment. The drainage area of the Town Creek embayment at the plant is 5.94 sq. mi. Town Creek forms a 4.2-mi. embayment that is also fed by six small unnamed tributaries with less than 1 sq. mi. of drainage area. The depth of the Town Creek embayment varies from approximately 2 ft., in the area of the County Rd. 33 bridge, to approximately 10 ft. in the embayment area north of the Bellefonte Road bridge. In general, depth is less than 5 ft. Surface elevations are generally consistent with those of Guntersville Reservoir and fluctuate based on pool elevations and daily operations of Nickajack and Guntersville Dams.

2.4.1.2.3 Water-Control Structures

2.4.1.2.3.1 New Water-Control Structures

The Guntersville Reservoir is bounded by two existing dams; Guntersville Dam, located 43 mi.downstream of the BLN, and Nickajack Dam, located 33 mi. upstream. Both of these dams are owned and operated by the TVA and are used for flood control, navigation, and hydroelectric power generation. The dams include an integrated system of locks for barge and river transportation. No additional water-control structures are planned or required for the facility.

2.4.1.2.3.2 Raw Water Intake Pumping Station

The intake pumping station is a reinforced concrete box-type structure housing the cooling-tower makeup pumps, service water makeup pumps, strainers, valves, and associated piping. The raw water system contains no safety-related equipment, nor does loss of its normal operating capability adversely affect any safety-related components.

The intake structure is located at the end of a manmade channel on the west bank of the Tennessee River near TRM 392.1. The blowdown discharge line is located downstream of this channel to avoid recirculation of plant effluent to the intake. The channel is a mid-channel trench approximately 7.6 m (25 ft.) wide, excavated into rock for maintenance of the raw water supply.

The bottom of the intake structure is at elevation 537 ft. above msl to allow for operation under low-water conditions. The operating deck is at elevation 607 ft.

above msl to protect the pumps and motors from the Tennessee River design flood level. The structure houses five pumps per unit. There are three cooling-tower makeup pumps, each sized such that two pumps adequately supply the required makeup flow of 22,500 gpm. There are also two ancillary raw water pumps, each sized to provide 100 percent of the required makeup to the service-water system and the demineralized water treatment system under normal operating conditions and during periods of peak demand.

Traveling water screens provide coarse screening of floating and suspended debris, and prevent aquatic life from entering the structure. The screens are the single-flow-through automatic cleaning type. Two screens are provided for each of the two supply loops at the inlet to the intake structure. Each of the two screens on each loop has sufficient capacity to screen the total water required for one loop. The river intake screens are sized so that the through-screen flow velocity is less than 0.5 fps. If fouling occurs, the screens are cleaned by back-flushing.

Sediment buildup in the intake channel is monitored and removed as required.

2.4.1.2.3.3 Guntersville Dam

Guntersville Dam was completed in 1939 and is presently used for navigation, flood control, hydroelectric power, and recreation. It consists of a soils and rock foundation with a combination concrete and gravity earthfill structure. The dam measures 3979 ft. in length, with a structural height of 94 ft. and a hydraulic height of 78 ft. The dam contains two locks. The main lock measures 110 ft. wide and 600 ft. long; the auxiliary lock measures 60 ft. wide and 360 ft. long. The gated spillway measures 720 ft. long. The embankments were raised 7.5 ft. to elevation 617.5 ft. above msl in 1995.

Guntersville Dam controls a drainage area of 24,450 sq. mi. with a maximum dam discharge rate of 650,000 cfs. Guntersville Reservoir has a reported surface area of 67,900 ac. (normal minimum pool) with a maximum storage capacity of 1,049,000 ac.-ft. (normal maximum pool) (Reference 234).

Guntersville Dam is designed to withstand the probable maximum flood (PMF) event (described in further detail in Subsection 2.4.2). Seismic effects on hydrology at the site are discussed in Subsection 2.4.3.

2.4.1.2.3.4 Nickajack Dam

Nickajack Dam was completed in 1967 and is presently used for navigation, flood control, hydroelectric power, and recreation. It consists of a soils and rock foundation with a combination concrete and gravity earthfill structure. The dam measures 3767 ft. in length, with a structural height of 81 ft. and a hydraulic height of 74 ft. The dam contains two locks, measuring 110 ft. wide and 800 ft. long, and a controlled, gated spillway that is 400 ft. long.

Nickajack Dam controls a drainage area of 21,870 sq. mi. with a maximum dam discharge rate of 1500,000 cfs. Nickajack Reservoir has a reported surface area of 9930 ac., with a normal storage capacity of 220,100 ac.-ft. and a maximum storage capacity of 251,600 ac.-ft.

In 1992, the south embankment was raised 5 ft. to elevation 2657 ft. above msl. A roller-compacted concrete overflow dam 1900 ft. long with top elevation at 634 ft. above msl was added below the north embankment. The north embankment was left with top elevation at 652 ft. above msl, and is allowed to overtop and fail down to the concrete overflow dam in extreme flood events.

Nickajack Dam (north embankment) would be overtopped during the PMF event, (described in further detail in Subsection 2.4.2). Seismic effects on hydrology at the site are discussed in Subsection 2.4.4.

2.4.1.2.4 Surface-Water Use

There are approximately 18 significant water users in the Guntersville Reservoir watershed area that withdraw approximately 1600 Mgd. Fourteen of these water users are public-supply providers to local communities, and they withdraw from 0.8 Mgd to 10 Mgd. The largest water user is TVA's Widows Creek Fossil Plant, which utilizes up to 1546 Mgd for thermoelectric power generation. TVA records did not provide water return volumes; therefore, USGS cumulative net demand of 60.6 Mld (16 Mgd) is utilized as the local, net water volume. Table 2.4.1-202 lists local surface-water users as well as detailed information such as facility name, county, intake location (if known), maximum withdrawal rate (if known), and water source. Due to its sensitive nature, distance from the BLN site and water withdrawal locations have been omitted from Table 2.4.1-202 and are provided, as required, to the appropriate personnel on an as-needed basis. There may be several private, small-quantity water users (irrigation) in this area, including two golf courses and two farms that are not listed in Table 2.4.1-202, because their use is not significant with regard to the total river flow.

2.4.1.2.5 Groundwater Use

Groundwater is not used at the BLN. Groundwater is fully discussed in Subsection 2.4.12.

2.4.2 FLOODS

2.4.2.1 Flood History

BLN COL 2.4-2 Floods on the Tennessee River occur primarily as a result of precipitation runoff from its major tributaries, the Clinch, French Broad, Holston, Little Tennessee, and Hiwassee Rivers. The Tennessee Valley Authority (TVA) reservoir system was designed with flood control as one of its primary purposes. Available flood control storage in the system varies with the time of year and potential flood threat. The reservoir system in the eastern portion of the basin was primarily planned to protect Chattanooga, Tennessee from flooding. This portion of the basin is drained by five of the Tennessee River's largest tributaries, the Hiwassee, Clinch, Little Tennessee, French Broad, and Holston rivers, and by 180 mi. of the main river itself. The multipurpose tributary reservoirs in the upper system provide approximately 4 million ac.-ft. of storage, or approximately 6 in. of runoff between January 1 and March 15. Almost 90 percent of this storage is provided by five major reservoirs (Norris, Cherokee, Douglas, Fontana, and Hiwassee reservoirs), each of which is located on one of the major tributary rivers. Flood storage is maximized from January to March to accommodate the flood season.

The three main river reservoirs above Chattanooga, Tennessee (Chickamauga Watts Bar, and Ft. Loudoun-Tellico) provide only 955,300 ac.-ft. of storage, or 2.8 in. of runoff on January 1, a relatively small amount of the total upper system flood storage. These mainstream reservoirs, however, play an essential part in, reducing the flood crest at Chattanooga as they provide regulation of the otherwise uncontrolled 7400 sq. mi. area between Chattanooga and the tributary dams.

Prior to the completion of the TVA reservoir system, most valleys in the Basin were subject to periodic flooding. Reducing the flood risk at Chattanooga became a major priority in the design of the TVA reservoir system and remains a major operating priority today (Reference 211). The operation of the reservoir system upstream of Chattanooga, Tennessee effectively regulates flood flows at the BLN.

There have been dams in the drainage basin since the early 1910s. Significant regulation began with the completion of Norris Dam in 1936. By 1944, the major flood control dams had been completed by the TVA. Several smaller flood control structures were completed by 1952. Significant changes to the watershed had been completed by 1979. However, flood records after 1952 can be considered representative of the current regulated conditions of the Tennessee River. Elevations provided in this subsection are above mean sea level (msl).

The drainage area of the Tennessee River at the BLN, Tennessee River mile (TRM) 391.5, is 23,340 sq. mi. Four U.S. Geological Survey (USGS) gauges are used to determine flood history. The South Pittsburg gauging station (USGS No. 03571850) is located upstream of the site and downstream of Nickajack Dam at about Tennessee River Mile 418. The gauge has a drainage area of 22,640 sq. mi., about 97 percent of the drainage area at the BLN. The gauge has been discontinued and has a peak flow broken period of record from 1917 to 1987. Table 2.4.2-201 summarizes the peak flows for periods prior to and after regulation.

The Chattanooga gauging station (USGS No. 03568000) has a drainage area of 21,400 sq. mi. and is located upstream of Nickajack Dam at about TRM 467.6. The gauge currently operates and provides annual peak flow data. For comparison, Table 2.4.2-202 summarizes the Chattanooga gauge peak flows.

The Guntersville gauging station (USGS No. 03573500) is located downstream of the site at TRM 358 and has a drainage area of 24,340 sq. mi. The gauge has been discontinued and has a peak flow broken period of record from 1867 to 1938. Guntersville Dam, at TRM 349, was completed in 1939. The maximum recorded gauge height occurred in 1867. Table 2.4.2-203 summarizes the peak flows for the period prior to regulation.

The Whitesburg gauging station (USGS No. 03575500) has a drainage area of 25,610 sq. mi. and is located downstream at about TRM 334, below Guntersville Dam. The gauge currently operates and provides annual peak flow data. For comparison, Table 2.4.2-204 summarizes the Whitesburg gauge peak flows.

Prior to current regulated conditions, the maximum flood occurred in March 1867. A peak flow of 459,000 cfs, measured at Chattanooga, Tennessee, occurred on March 11, 1867. The peak flood elevation at South Pittsburg, Tennessee was 625.61 ft. (Reference 206) The flood peaked at elevation 594.31 ft. near the present day Guntersville Dam on March 13, 1867. Flow was not recorded at this location. In 1986, the peak flood elevation of this event at the BLN site was estimated to be 610.80 ft. Present day regulation would significantly reduce this estimate. Additional significant floods prior to current regulated conditions occurred in 1875, 1886, and 1917. Major regional historical floods are summarized in Table 2.4.2-205.

The maximum flood in this area under current regulated conditions occurred on March 18, 1973. The peak flow measured at South Pittsburg, Tennessee was 315,000 cfs with a peak elevation of 615.34 ft. The flood peaked at elevation 595.72 ft. at the Guntersville Dam (Reference 206). The March 18, 1973 flood elevation at the BLN site is estimated to be 602.2 ft. Additional significant floods under current regulated conditions occurred in 1984 and 2003. Major regional historical floods are summarized in Table 2.4.2-205.

Since the completion of Guntersville Dam, the highest recorded elevation for Guntersville Reservoir is 596.29 ft. and occurred on March 2, 1944 (Reference 215). Reservoir elevation is measured at a location near the dam at about TRM 349. Based on interpolation of the TVA flood risk profile (Reference 224), this corresponds to a water surface elevation of 601.4 ft. at the BLN. With the reservoir elevation at the top of the gates, 595.44 ft., Guntersville Dam can discharge about 511,000 cfs (Reference 215). Figure 2.4.2-201 provides the TVA flood risk profile with historical flood data included.

No historical data exists regarding flooding due to surges, seiches, tsunamis, dam failures, or flooding due to landslides. Surge and seiches are discussed in Subsection 2.4.5. Tsunamis are discussed in Subsection 2.4.6. Dam failures are discussed in Subsection 2.4.3 and Subsection 2.4.4. Landslides are discussed in Subsection 2.4.9. Historical information related to icing and ice jams is provided in Subsection 2.4.7.

2.4.2.2 Flood Design Considerations

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

The type of events evaluated to determine the worst potential flood include (1) probable maximum precipitation (PMP) on the total watershed and critical sub-watersheds including seasonal variations and potential consequent dam failures, as discussed in Subsection 2.4.3, (2) dam failures, as discussed in Subsection 2.4.4, including in a postulated safe shutdown earthquake with a coincident 25-year flood or operating basis earthquake with a coincident one-half PMF, (3) local intense precipitation, and (4) two year coincident wind waves, as discussed in Subsection 2.4.3. Local intense precipitation is discussed below. Both static and dynamic assumed hypothetical conditions to determine the design flood protection level are evaluated in Subsection 2.4.3 and Subsection 2.4.4.

Specific analysis of Tennessee River flood levels resulting from ocean front surges, seiches, and tsunamis is not required because of the inland location and elevation characteristics of the BLN. Additional details are provided in Subsections 2.4.5 and 2.4.6. Snowmelt and ice effect considerations are unnecessary because of the temperate zone location of the BLN. Additional details are provided in Subsection 2.4.7. Flood waves from landslides into upstream reservoirs required no specific analysis, in part because of the absence of major elevation relief in nearby upstream reservoirs and because the prevailing thin soils offer small slide volume potential compared to the available detention space in reservoirs. Additional details are provided in Subsection 2.4.9.

The maximum flood level at the BLN is elevation 622.5 ft. This elevation would result from a sequence of March storms producing a maximum rainfall on the 21,400 sq. mi. watershed above Chattanooga as described in Subsection 2.4.3. Coincident wind waves would create maximum waves of 5.41 ft. (trough to crest) and produce maximum flood levels of elevation 624.03 ft., including wind wave setup and runup. The BLN safety-related structures are located above the worst potential flood considerations at elevation 628.6 ft.

2.4.2.3 Effects of Local Intense Precipitation

The BLN drainage system was evaluated for a storm producing the PMP on the local area. The site is graded such that runoff will drain away from safety-related structures to drainage channels and subsequently to the Tennessee River. The PMP flood analysis assumes that all discharge structures are non-functioning. The site grading and drainage plan is shown in Figure 2.4.2-202.

Flow for drainage area A is directed away from the site by channel flow. However, under local intense precipitation conditions overflow spills into drainage area B. A typical channel cross section is represented by a 4 ft. deep, grass-lined, V-shaped channel with 3:1 (horizontal:vertical) side slopes. The upper length of the channel that overflows into drainage area B has a 0.66 percent slope. The lower length of the channel has a 2 percent slope.

Drainage area B captures runoff in a low area catch basin. The catch basin is assumed non-functional. Weir flow determines the water surface elevation for runoff exiting drainage area B. The weir is modeled using a low point of 625.5 ft. with 100:1 (horizontal:vertical) side slopes up to 626 ft., and 50:1 (horizontal:vertical) side slopes beyond that.

Drainage area C receives the overflow from drainage area B and captures runoff in a low area catch basin. The catch basin is assumed non-functional. Weir flow determines the water surface elevation for runoff exiting drainage area C. Runoff from drainage area C exits the site unobstructed.

Drainage area D captures a portion of the runoff from the two units. Flow is constricted at one point and analyzed as channel flow. A typical channel cross section is represented by a 87 ft. wide trapezoidal channel with average side slopes of 57:1 (horizontal:vertical). At the point of evaluation, the channel invert is 623 ft. elevation with a channel depth of 4 ft. and a minimum slope is 0.67 percent. Flow then exits the site unobstructed.

Drainage areas E and F capture flow from a portion of the two units where runoff is partially obstructed by a building. Small channels direct runoff along the building and away from the safety-related structures. Runoff then exits the site unobstructed. A typical cross section is represented by a 4 ft. wide trapezoidal channel with 50:1 (horizontal:vertical) and 3:1 (horizontal) side slopes. At the point of evaluation the channel invert is elevation 625. The channel depth is limited to 2 ft. and the slope is 0.5 percent. All other areas direct water away from safety-related structures unobstructed over open sloped paved areas between 0.5 and 2 percent and grass covered areas at 2 percent.

The local intense PMP is defined by Hydrometeorological Report (HMR) No. 56 (Reference 248). The 1 sq. mi. PMP values for durations from 5-minutes to 24-hours are determined using the procedures as described in HMR No. 56. As indicated in HMR 56, the 1 sq. mi. PMP rates may also be considered the point rainfall for areas less than 1 sq. mi. The derived PMP curve is detailed in Table 2.4.2-206 and Figure 2.4.2-203. The corresponding PMP intensity duration curve is shown in Figure 2.4.2-204.

The rational method is used to determine peak runoff rates from specified areas (Reference 248). The rational method is given by the equation:

$$Q = k * C * i * A$$

where: Q = runoff in cfs

k = constant = 1 for English units

C = unitless coefficient of runoff

i = intensity in in/hr

A = drainage area in ac.

Rainfall duration is assumed to be equal to or greater than the time of concentrations for each site drainage area. The corresponding intensity is determined using Figure 2.4.2-204. Runoff coefficients are assumed equal to one, to maximize runoff and account for saturated antecedent conditions.

Time of concentration is the time required for runoff to travel from the most hydraulically distant point of the drainage area to the point of interest. Time of concentration is determined using the methods of the Natural Resources Conservation Service Technical Release 55 (Reference 214). The time of concentration includes travel time components for overland flow, shallow concentrated flow and channel flow.

Water surface elevations for overflow areas are derived from the broad crested weir flow equation, given by:

$$Q = C * L * H^{3/2}$$

where: Q = volumetric flow rate in cfs

C = weir flow coefficient

L = weir length in ft.

H = weir energy head in ft.

The U.S. Army Corps of Engineers HEC-RAS version 3.1.3 standard-step, backwater analysis computer software was used to model the interaction of drainage areas B and C. Water surface elevations were determined using the HEC-RAS inline weir structure feature and a weir flow coefficient of 2.6. Cross sections were developed using the graded contours. Flow was modeled using steady state conditions.

Water surface elevations for channel flow areas are derived from the mass continuity equation, given by:

$$Q = V * A$$

where: Q = volumetric flow rate cfs

V = mean flow velocity in ft/s

A = cross sectional flow area in sq. ft.

Velocity for open channel flow is determined using the Manning's formula given by:

$$V = (k * r^{2/3} * s^{1/2}) / n$$

where:
- V = average velocity in ft/s
- k = constant = 1.49 for English units
- r = hydraulic radius in ft. and is equal to a/p_w
- a = cross sectional flow area in sq. ft.
- p_w = wetted perimeter in ft.
- s = slope of hydraulic grade line in ft/ft
- n = Manning's roughness coefficient for open channel flow

A Manning's roughness coefficient of n = 0.035 is used for the developed site area. Offsite undeveloped areas are represented by a Manning's roughness coefficient of n = 0.050.

Table 2.4.2-207 contains details and resulting water surface elevations for the drainage areas identified in Figure 2.4.2-202. Backwater analysis from drainage areas B and C results in a maximum water surface elevation of 627.53 ft. in the vicinity of the safety-related structures. A sensitivity analysis was performed by increasing and decreasing the roughness coefficient by 50 percent. The resulting water surface elevations did not exceed plant elevation. The BLN safety-related structures are located above the effects of local intense precipitation at elevation 628.6 ft.

The plant design is based on a PMP of 19.4 in/hr and 6.3 in/5 min. As shown in Figure 2.4.2-203 and Table 2.4.2-201, the site is within the plant design limits for PMP. Roofs are sloped to preclude ponding of water.

Town Creek is the largest tributary stream in the vicinity of the BLN. The Town Creek watershed is approximately 10.84 sq. mi. Because of its small size and drainage into the Guntersville Reservoir, Town Creek will not create potential flood problems for the BLN safety-related facilities. Based on USGS quadrangle contours and the normal full pool elevation of 595 ft., the Town Creek Reservoir can accommodate the total 24-hr, 10 sq. mi. PMP without discharge to the Guntersville Reservoir. The resulting water surface elevation is 610.68 ft. This accounts for total rainfall runoff conversion without any precipitation losses.

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

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

The probable maximum flood (PMF) was determined from the probable maximum precipitation (PMP) for the watershed located hydrologically above the plant with consideration given to seasonal and aerial variations in rainfall. The guidance of Appendix A of Regulatory Guide 1.59 was followed in determining the PMF by applying the guidance of ANSI/ANS-2.8-1992 (Reference 203). ANSI/ANS-2.8-1992 was issued to supersede ANSI N170-1976, which is referred to by Regulatory Guide 1.59. ANSI/ANS-2.8-1992 is the latest available standard.

2.4.3.1 Probable Maximum Precipitation

PMP was defined for the TVA by the National Weather Service and is published in Hydrometeorological Reports (HMR) 41 (Reference 220), 47, (Reference 219), and 56 (Reference 248). These reports define depth-area-duration characteristics and antecedent storm potentials and incorporate orographic effects of the Tennessee River valley. HMR 56, the most recent report covering the watershed, but only applies to watershed basins up to 3000 sq. mi.; however, HMR 56 indicates that for basins larger than 3000 sq. mi. individual basin studies, such as HMR 41, should be used.

A March storm was determined to be critical for main Tennessee River watersheds. Due to the temperate climate of the watershed and relatively light snowfall, snow melt is not a factor in generating the maximum floods for the Tennessee River in the area of the BLN.

The PMF discharge at the BLN was determined to result from the 21,400 sq. mi. storm producing the PMP on the watershed above Chattanooga with the downstream orographically fixed storm pattern, as defined in HMR 41. A standard time distribution pattern was adopted for the storms based upon major observed storms transposable to the Tennessee Valley and in conformance with the usual practice of Federal agencies. This places the heavy precipitation in the middle of the storm. The adopted distribution mass curve is shown in Figure 2.4.3-201. The adopted sequence conforms closely to that used by the U.S. Army Corps of Engineers.

There are two possible isohyetal patterns producing the 21,400 sq. mi. area depths presented in HMR 41. The critical downstream isohyetal pattern is shown on Figure 2.4.3-202. The PMP storm would occur in the month of March and would produce 15.6 in. of rainfall in three days. The storm producing the PMP would be preceded by a 3-day antecedent storm producing 6.4 in. of rainfall, which would end 3 days prior to the start of the PMP storm. This is the same PMP storm that produces the PMF at the Sequoyah Nuclear Plant. Figure 2.4.3-202 also includes the maximum 3-day PMP. Precipitation temporal distribution is determined by applying the mass curve, Figure 2.4.3-201, to the basin rainfall depths in Table 2.4.3-201.

2.4.3.2 Precipitation Losses

A multi-variable relationship, used in the day-to-day operation of the TVA system, has been applied to determine precipitation excess directly. The relationships were developed from observed data. They relate precipitation excess to the rainfall, week of the year, geographic location, and antecedent precipitation index (API). In their application, precipitation excess becomes an increasing fraction of rainfall as the storm progresses in time and becomes equal to rainfall when from 6 to 16 in. have fallen. An API determined from historical floods was used at the start of the antecedent storm.

Basin rainfall, precipitation excess, and API are provided in Table 2.4.3-201. The average precipitation losses for the watershed above Guntersville Dam are 2.24 in. for the 3-day antecedent storm and 1.76 in. for the 3-day main storm. The losses are approximately 35 percent of antecedent rainfall and 11 percent of the PMP respectively.

2.4.3.3 Runoff and Stream Course Models

The runoff model used to determine Tennessee River flood hydrographs at the BLN is divided into 50 unit areas and includes the total watershed above Guntersville Dam. The watershed unit areas are shown in Figure 2.4.3-203. The watershed rises to the east and north in the rugged southern Appalachian Highlands and the valley and ridge physiographic province to the northeast. About 20 percent of the total watershed rises above elevation 3000 ft. above mean sea level (msl) with a maximum elevation of 6684 ft. msl at Mt. Mitchell North Carolina. Topographic details in the area of the BLN are discussed in Subsection 2.4.1.

A TVA developed flood hydrology computer model was used to model the rainfall runoff using unit hydrographs. Unit area and hydrograph details are provided in Table 2.4.3-202. The unit area flows are combined with appropriate time sequencing or channel routing procedures to compute inflows into the most upstream reservoirs which in turn are routed through the reservoirs using standard hydrology techniques. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures including unsteady flow routing. A standard base flow of 2.5 cfs/sq. mi. was also included in the model.

Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream gauging stations or estimated from reservoir headwater elevation, inflow and discharge data. For ungauged unit areas synthetic unit graphs were developed from relationships relating the unit graph peak flow to the drainage area size and time to peak in terms of watershed slope and length developed from the computed unit graph parameters. Unit hydrographs are provided in

Figures 2.4.3-204, 2.4.3-205, 2.4.3-206, 2.4.3-207, 2.4.3-208, 2.4.3-209, 2.4.3-210, 2.4.3-211, 2.4.3-212, 2.4.3-213, 2.4.3-214, 2.4.3-215, and 2.4.3-216.

Tributary reservoir routings, except for Tellico, were made using the Goodrich semigraphical method and flat pool storage conditions. Main river reservoir and Tellico routings were made using unsteady flow techniques. Unsteady flow routings were computer solved with the Simulated Open Channel Hydraulics (SOCH) mathematical model, based on the equations of unsteady flow, developed by TVA. Boundary conditions prescribed were inflow hydrographs at the upstream boundary, local inflows, and headwater discharge relationships at the downstream boundary based upon normal operating rules, or based upon rated curves when the structure geometry controlled. Reservoir operating curves are provided in Figures 2.4.3-217, 2.4.2-218, 2.4.3-219, 2.4.3-220, 2.4.3-221, 2.4.3-222, 2.4.3-223, 2.4.2-224, 2.4.3-225, 2.4.3-226, 2.4.3-227, 2.4.3-228, 2.4.3-229, 2.4.3-230, 2.4.3-231, 2.4.3-232, 2.4.3-233, 2.4.3-234, 2.4.3-235, 2.4.3-236, and 2.4.3-237. Ocoee #2 is a run-of-river project and does not have an operating curve.

Stage discharge rating curves are provided in Figures 2.4.3-238, 2.4.3-239, 2.4.3-240, 2.4.3-241, 2.4.3-242, and 2.4.3-243 for the Tennessee River reservoirs. The figure for Nickajack Dam contains a composite of two headwater rating curves. One is based on no failure and the second is based on failure of the north embankment. The PMF is developed using the curve incorporating failure of the north embankment.

The figure for Chickamauga Dam contains three headwater rating curves. Proposed dam safety modifications to allow overtopping have not been performed. One curve represents existing conditions, a second curve represents conditions during modifications requiring a cofferdam, and the third curve represents completed modifications. The PMF is developed using the curve incorporating completed modifications. However, Chickamauga failure under current conditions is also considered in Subsection 2.4.3.4.

An unsteady flow mathematical model for the 75.7 mi. long Guntersville Reservoir was divided into thirty-six, 2.1 mi. reaches providing thirty-seven equally spaced grid points. A 2.5 minute time step was used and represents the largest time step which maintained a stable numerical solution and also reproduced observed flow conditions. The unsteady flow model was verified at six gauged points within Guntersville Reservoir using 1973 flood data. Comparison between observed and computed stages in Guntersville Reservoir is shown in Figure 2.4.3-244. Nickajack Reservoir (Figure 2.4.3-244) was also verified using 1973 flood data.

The unsteady flow mathematical model for the 49.9 mi. long Fort Loudoun Reservoir was divided into twenty-four, 2.08 mi. reaches. The model was verified at three gauged points in Fort Loudoun Reservoir using 1963 and 1973 flood data. The unsteady flow model was extended upstream on the French Broad and Holston Rivers to Douglas and Cherokee Dams, respectively. The French Broad and Holston River unsteady flow models were verified at one gauged point each (Mile 7.4 and 5.5, respectively), using 1963 and 1973 flood data.

The Little Tennessee River was modeled from Tellico Dam, Mile 0.3, through Tellico Reservoir to Chilhowee Dam at Mile 33.6, and upstream to Fontana Dam at Mile 61.0. The model for Tellico Reservoir to Chilhowee Dam was tested for adequacy by comparing its results with steady-state profiles at 1,000,000 cfs and 2,000,000 cfs computed by the standard-step method. Minor decreases in conveyance in the unsteady flow model yielded good agreement. The average conveyance correction found necessary in the reach below Chilhowee Dam to make the unsteady flow model agree with the standard-step method was also used in the river reach from Chilhowee to Fontana Dams.

Fort Loudoun and Tellico unsteady flow models were joined by a canal unsteady flow model. The canal was modeled with five equally-spaced cross sections at 525 ft. intervals for the 2100 ft. long canal.

The unsteady flow routing model for the 72.4 mi. long Watts Bar Reservoir was divided into thirty-four, 2.13 mi. reaches. The Watts Bar model was verified at two gauged points within the reservoir using 1963 flood data.

The unsteady flow routing model for the total 58.9 mi. long Chickamauga Reservoir was divided into twenty-eight, 2.1 mi. reaches. The Chickamauga Reservoir unsteady flow model was verified at four gauged points within the reservoir 1973 flood data.

Verifying the models with actual data approaching the magnitude of the PMF is not possible. Therefore, using extreme flows, steady-state model elevations were compared with elevations computed using the standard step method. The example rating curve shown in Figure 2.4.3-245 depicts this comparison.

The watershed runoff model was verified by using it to reproduce the March 1963 and March 1973 floods. Observed volumes of precipitation excess were used in the verification. Comparisons between observed and computed outflows from Hales Bar and Guntersville Dams for the 1963 flood are shown in Figure 2.4.3-246. The comparisons for the 1973 flood at Nickajack and Guntersville dams are shown in Figure 2.4.3-247.

Normal reservoir operating procedures were used in the antecedent storm. These used turbine and sluice discharges in the tributary reservoirs. Turbine discharges are not used in the main river reservoirs after large flood flows develop because head differentials are too small. Normal operating procedures were used in the principal storm except that turbine discharge was not used in either the tributary or main river dams. All spillway gates were determined to be operable without failures during the flood. TVA's operation and maintenance procedures, updated as an integral part of its dam safety program consistent with the Federal guidelines for dam safety, provide a basis for expecting the spillway gates to be operated when and as needed.

Median initial reservoir elevations for the appropriate season were used at the start of the storm sequence. The reservoir elevations used to define the PMF are consistent with statistical experience and avoid unreasonable combinations of extreme events.

The flood from the antecedent storm occupies 67 percent of the reserved system detention capacity by the time of the start of the flood generated by the main storm. Reservoir levels are at or above guide levels in all but one reservoir. Operating rules had no significant effect on maximum flood discharges because spillway capacities, and hence uncontrolled conditions, were reached early in the flood.

2.4.3.4 Probable Maximum Flood Flow

The PMF discharge at the BLN was determined to be 1,041,000 cfs. The PMF hydrograph is shown in Figure 2.4.3-248. This includes the effects of the following postulated dam failures. The west saddle dike at Watts Bar Dam and the north embankment at Nickajack Dam would be overtopped and breached. At Nickajack Dam, the north embankment would fail down to the roller compacted concrete overflow dam with top at elevation 634 ft. Chickamauga Dam, 79.5 mi. upstream from the BLN, would be overtopped but was assumed not to fail, reflecting the conditions of completed dam safety modifications.

Proposed dam safety modifications to allow overtopping at Chickamauga Dam have not been performed. When considering overtopping failure, flood levels would increase at the BLN, but the increase would be small. Dam safety studies showed that with Watts Bar, Nickajack, and Chickamauga overtopping failures, the flood level at the BLN would be increased by only 0.40 ft. Failure of downstream dams would potentially lower the resulting flood level at the BLN; however, any potential lowering of the flood levels at the BLN due to downstream dam failure effects was not considered in the resulting water surface elevation. There are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during dam failure-induced flooding.

Details of hydrologic dam failure analyses are provided below.

Concrete Section Analysis

For concrete dam sections, comparisons were made between the original design headwater and tailwater levels and those that would prevail in the PMF. If the overturning moments and horizontal forces were not increased by more than 20 percent, the structures were considered safe against failure. The upstream dams passed this test except Douglas, Fort Loudoun, and Watts Bar. Original designs showed the spillway sections of these dams to be most vulnerable. These spillway sections were examined further and are expected to be stable.

Spillway Gates

During the peak PMF conditions the radial spillway gates of Fort Loudoun and Watts Bar Dams are wide open with flow over the gates and under the gates. For this condition both the static and dynamic load stresses in the main structural members of the gate are less than the yield stress by a factor of three. The stress in the trunnion pin is less than the allowable design stress by a factor greater than 10.

The gates were also investigated for the condition when rising headwater level first begins to exceed the bottom of the gates in the wide-open position. This condition produces the largest forces tending to rotate the radial gates upward. In the wide-open position the gates are dogged against steel gate stops anchored to the concrete piers. The stresses in the gate stop members are less than the yield stress of the material by a factor of two.

It is concluded that the above-listed margins are sufficient to provide assurance also that gates will not fail as a result of additional stresses which may result from possible vibrations of the gates acting as orifices.

Lock Gates

The lock gates at the main river dams, Fort Loudoun, Watts Bar, Chickamauga, Nickajack, and Guntersville, were examined for possible failure with the conclusion that no potential for failure exists because the gates are designed for a differential hydrostatic head greater than that which exists during the PMF.

Embankment Breaching

The adopted relationship to compute the rate of erosion in an earth dam failure is that developed and used by the Bureau of Reclamation in connection with its safety of dams program. The expression relates the volume of eroded fill material to the volume of water flowing through the breach. The equation is:

$$\frac{Q_{soil}}{Q_{water}} = Ke^{-x}$$

where

Q_{soil} = Volume of soil eroded in each time period

Q_{water} = Volume of water discharged each time period

K = Constant of proportionality, 1 for soil and discharge relationships in this study

e = Base of natural logarithm system

$$X = \frac{b}{H}\tan\varnothing_d$$

where

b = Base length of overflow channel at any given time

H = Hydraulic head at any given time

$\varnothing_d$ = Developed angle of friction of soil material. A conservative value of 13

degrees was adopted for materials in the dams investigated.

Solving the equation, which was computerized, involves a trial and error procedure over short depths and time increments. In the program, depth changes of 0.1 ft. or less are used to keep time increments to less than one second during rapid failure and up to about 350 seconds prior to breaching.

The solution of an earth embankment breach begins by solving the erosion equation using a headwater elevation hydrograph assuming no failure. Erosion is postulated to occur across the entire earth section and to start at the downstream edge when headwater elevations reached a selected depth above the dam top elevation. Subsequently, when erosion reaches the upstream edge of the embankment, breaching and rapid lowering of the embankment begins. Thereafter, computations include headwater adjustments for increased reservoir outflow resulting from the breach.

Some verification for the breaching computational procedures was obtained by comparison with actual failures reported in literature and in informal discussion with hydrologic engineers. These reports show that overtopped earth embankments do not necessarily fail. Earth embankments have sustained overtopping of several feet for several hours before failure occurred. An extreme example is Oros earth dam in Brazil which was overtopped to a depth of approximately 2.6 ft. along a 2000 ft. length for 12 hours before breaching began. Once an earth embankment is breached, failure tends to progress rapidly, however. How rapidly depends upon the material and headwater depths during failure. Complete failures computed in this and other studies have varied from about one-half to six hours after initial breaching. This is consistent with actual failures.

2.4.3.5 Water Level Determinations

The maximum flood elevation at the BLN was determined to be 622.1 ft. msl, produced by the 21,400 sq. mi. storm and coincident overtopping failure of the west saddle dike at Watts Bar Dam and the north embankment at Nickajack Dam. Chickamauga Dam is overtopped but was assumed not to fail. The flood elevation hydrograph is shown in Figure 2.4.3-249; however, proposed dam safety modifications to allow overtopping at Chickamauga Dam have not been performed. Without the dam safety modifications at Chickamauga Dam, the maximum flood elevation was determined to be 622.5 ft. msl. Elevations were computed concurrently with discharges using the unsteady flow reservoir model previously described. The BLN safety-related structures are located at elevation 628.6 ft. msl and are unaffected by flood conditions.

2.4.3.6 Coincident Wind Wave Activity

Fetch length was estimated based on U.S. Geological Survey Quadrangles as shown in Figure 2.4.3-250. Fetch distances from the northeast and northwest were examined. A 3.4 mi. effective fetch length from the northeast was found to be critical. The BLN is protected from wind wave activity from the south by the local topography. Wave height, setup, and run-up are estimated using U.S. Army Corps of Engineers guidance (Reference 231).

A 2-year annual extreme mile wind speed of 50 mph was estimated based on ANSI/ANS-2.8-1992 as shown in Figure 2.4.3-251. The 2-year annual extreme mile wind speed was adjusted for duration, based on effective fetch length, level, over land or, over water, and stability. The northeast critical duration was found to be about 63 min. This corresponds to an adjusted wind speed of 49.66 mph. Significant wave height (average height of the maximum 33-$^{1}/_{3}$ percent of waves) is estimated to be 3.25 ft., crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 5.41 ft., crest to trough. The corresponding wave period is 2.4 sec.

Slopes of 50:1, horizontal to vertical, in the vicinity of the BLN are used to determine the wave setup and run-up. The maximum wind setup is estimated to be 0.28 ft. The maximum run-up, including wave setup, is estimated to be 1.25 ft. Therefore, total wind wave activity is estimated to be 1.53 ft. The PMF and coincident wind wave activity results in a flood elevation of 624.03 ft. msl. The BLN safety-related structures are located at elevation 628.6 ft. msl and are unaffected by flood conditions and coincident wind wave activity.

2.4.4 POTENTIAL DAM FAILURES

The procedures referred to in Appendix A of Regulatory Guide (RG) 1.59[a] were followed for evaluating potential flood levels from seismically induced dam failures. In accordance with this guidance, seismic dam failure is examined using two alternatives: 1) the Safe Shutdown Earthquake (SSE) coincident with the

a. The material previously contained within Appendix A of Regulatory Guide (RG) 1.59 was replaced by American National Standards Institute (ANSI) Standard N170-1976. This ANSI standard has since been replaced by ANSI/American Nuclear Society (ANS) standard 2.8-1992 (Reference 203). The procedures described in ANSI/ANS 2.8-1992 were followed for evaluating potential flood levels from seismically induced dam failures.

peak of the 25-yr. flood and a 2-yr. wind speed applied in the critical direction, and 2) the Operating Basis Earthquake (OBE) coincident with the peak of the one-half PMF or the 500-yr. flood, whichever is less, and a 2-yr. wind speed applied in the critical direction.

In the 1970's, an analysis for maximum flood levels was completed for Bellefonte Nuclear (BLN), Browns Ferry Nuclear (BFN), Sequoyah Nuclear (SQN) and Watts Bar Nuclear (WBN). In 1998, a reassessment was performed for the maximum flood levels in light of modifications made as part of the TVA Dam Safety Program (DSP).

The earthquake assumed in these dam failure analyses was a deterministic earthquake based on the largest historic earthquake to occur in the area consistent with the requirements of 10 CFR 100.23d(3), but differs from the probabilistic earthquake required by 10 CFR 100.23d(1) for new plant design as discussed in Section 2.5. These analyses are adequate and bounding for BLN based on the following considerations, which are discussed in greater detail later in this introductory subsection:

- As required by GDC-2, the largest historic earthquake to occur in the area was used to pseudo-statically evaluate the dams. Current information from the TVA DSP demonstrates seismic ruggedness of concrete gravity dams. Also, TVA DSP has completed dynamic stability analysis on Fontana and Hiwassee dams using probabilistic earthquake response spectra which envelope the BLN OBE spectra, and were shown by this recent analysis to withstand high seismic demand.
- One-half PMF assumed in the analysis bounds the lesser 500-yr. flood required by the guidance.
- The combined event probability of exceedance of 1 x 10^{-6} required by the guidance is bound by the combined events considered in the analyses.
- The seismically induced flood elevations required by the guidance are bounded by the PMF elevation determined in the analysis.

The original dam failure analyses determined that three separate, combined events have the potential to create maximum flood levels at BLN; these results are shown in Table 2.4.4-201 and discussed further in Subsection 2.4.4.1. The same three events produced the maximum seismically induced flood levels at SQN upstream of BLN. These events are:

1. The simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams in the OBE during one-half PMF.
2. The simultaneous failure of Norris, Cherokee and Douglas Dams in the SSE during a 25-yr. flood.

3. The simultaneous failure of Cherokee and Douglas Dams in the OBE during one-half PMF, respectively.

<u>Seismic Ruggedness of Concrete Gravity Dams</u>

The plant site and upstream reservoirs are located in the Southern Appalachian Tectonic Province and, therefore, are subject to moderate earthquake forces. The upstream dams whose failure has the potential to cause flooding at BLN were investigated to determine if failure from seismic events would endanger plant safety.

General Design Criterion 2, *Design Basis for Protection against Natural Phenomena*, of 10 CFR Part 50, Appendix A, requires that the design bases for structures, systems, and components important to safety shall reflect the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin. To satisfy this criterion, the earthquake event used for the TVA dam failure analyses was based on the largest historic earthquake to occur in the Southern Appalachian Tectonic Province - the 1897 Giles County, Virginia earthquake. This earthquake was estimated to have had a body wave magnitude (m_b) of 5.7. The SSE for these studies was conservatively established as having a maximum horizontal acceleration of 0.18g and a simultaneous maximum vertical acceleration of 0.12g. The OBE was established as 1/2 SSE, therefore having a maximum horizontal acceleration of 0.09g and a simultaneous maximum vertical acceleration of 0.06g.

The BLN OBE for this comparison is defined as 1/2 Ground Motion Response Spectra (GMRS). The GMRS is discussed in Section 2.5.2 and shown in Figure 2.5-290.

The TVA DSP, which is designed to be consistent with the Federal Guidelines for Dam Safety (Reference 207), conducts technical studies and engineering analyses to assess the hydrologic and seismic integrity of agency dams and verifies that they can be operated in accordance with Federal Emergency Management Agency (FEMA) guidelines. These guidelines were developed to enhance national dam safety such that loss of life and property damage is minimized. As part of the TVA DSP, inspection and maintenance activities are carried out on a regular schedule to confirm the dams are maintained in a safe condition. Instrumentation to monitor the dams' behavior was installed in many of the dams during original construction. Other instrumentation has been added since and is still being added as the need arises or as new techniques become available. Based on the implementation of the DSP, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.

The summary of analyzed floods from the postulated seismic failure of upstream dams is presented in Table 2.4.4-201. As shown in this table, the controlling combined event scenario is the assumed failure of Fontana, Hiwassee, Apalachia and Blue Ridge dams for an OBE with one-half PMF. The catastrophic failure of

Fontana and instant disappearance of Hiwassee and Apalachia, which are concrete gravity dams, is conservative based on past earthquake experience.

Concrete gravity dams similar to many TVA dams have performed very well during earthquakes throughout the world. Only one concrete gravity dam, the Shih-Kang Dam in Taiwan, is known to have failed because of an earthquake. This dam failure was caused by the fault rupture crossing directly beneath the dam and offsetting portions of the dam 29 ft. vertically and 6.5 ft. horizontally (Reference 246). Surface ruptures such as this are not expected to occur in the BLN area, as discussed in Section 2.5, or beneath any of the dams upstream of the BLN site. Worldwide, no other concrete gravity dams have failed due to earthquakes and only a few concrete dams have experienced any damage due to earthquakes, although dams have been subjected to earthquakes with Modified Mercalli (MM) intensities ranging from VIII to IX and ground accelerations have been measured to be as high as 0.51g perpendicular to the dam axis and 0.36g peak vertical acceleration (Reference 207).

Therefore, based on the known seismic ruggedness of these concrete dams, the analyses assuming catastrophic failure of the concrete dams in the controlling combined event scenario are conservative and the previous analyses are adequate and bounding.

Since the implementation of the TVA DSP in 1982, additional analyses and studies have been completed on several TVA dams based on a priority ranking. The TVA DSP has recently completed a dynamic stability analysis of Fontana and Hiwassee dams using probabilistic earthquake spectra. Other TVA dams have pseudo-static stability analysis performed, while others have no unique stability analysis but are compared to other analyzed dams. The results of these efforts under the TVA DSP, as further discussed below, have shown that catastrophic failure of these dams during a seismic event are less probable than previously assumed.

The controlling event for seismically induced floods is simultaneous failure of Fontana, Hiwassee, Apalachia and Blue Ridge dams during an OBE coincident with a one-half PMF. A comparison of the BLN OBE[b] to the response spectra used in the TVA DSP analyses for Fontana and Hiwassee is made in Figure 2.4.4-201. These analyses envelope the BLN OBE demand and therefore provide further evidence that the assumption of catastrophic failure of Fontana and instant disappearance of Hiwassee dam is extremely conservative. In addition, these analyses give confidence of the ability of other similar TVA dams to withstand the high frequency demand of the BLN Ground Motion Response Spectra (GMRS). Apalachia is a concrete gravity dam without unique stability analysis since it is not considered a dam of concern for detailed design studies within the TVA DSP. Cherokee, Douglas and Norris are concrete gravity dams

b. The BLN OBE for this comparison is defined as 1/2 Ground Motion Response Spectra (GMRS). The GMRS is discussed in Section 2.5.2 and shown in Figure 2.5-290.

with embankments with limited stability analyses. Because of the lack of any known structural deficiencies, and the relatively low seismic hazard for these dams, performance of a detailed seismic evaluation for theses dams was not considered necessary.

The original analyses assume catastrophic failure of four major dams. Two of these dams (Fontana and Hiwassee) have been shown by recent analyses to withstand high seismic demand.

One-half PMF versus 500-yr. flood

The procedures referred to by the guidance require seismic dam failure to be examined using the SSE coincident with the peak of the 25-yr. flood, and the OBE coincident with the peak of one-half the PMF or 500-yr. flood, whichever is less. The analyses consider a more severe one-half PMF instead of the 500-yr. flood; therefore, these analyses bound those prescribed by the Regulatory Guide.

Probability of Exceedance

The cumulative annual probability of exceedance for each of the combined events is tabulated in Table 2.4.4-202. These exceedance probabilities are calculated for the SSE/OBE which was used in the original analyses. The cumulative annual probabilities are 1.3×10^{-8} and 2.4×10^{-9} which satisfies the acceptance level of 1.0×10^{-6} set forth by Regulatory Guide 1.59.

The low probabilities of exceedance for the events used in the original analyses confirm that these analyses are bounding.

Bounding PMF Analysis and PMF Margin

The analyses result in a limiting flood at BLN due to the PMF flood elevation of 622.5 ft. as discussed in Subsection 2.4.3. The PMF is expected to remain bounding since this maximum flood depth exceeds the floods resulting from the previously analyzed seismically induced dam failures by at least 7 ft. (elevation 615.1 ft. seismically induced dam failures versus elevation 622.5 ft. for the PMF).

There also exists an additional 6 ft. of flood depth margin between the PMF of 622.5 ft. and the plant floor level of 628.6 ft.

Summary

While the 2-yr. wind speed applied in the critical direction was not included in the original or the reassessment flood analyses, it is not a significant contribution to the overall flood elevation (Subsection 2.4.3). The 1998 flood reassessment did not recalculate the elevations past Chickamauga Dam. Because the elevations at Chickamauga Dam were lower than calculated in the 1970's original analysis as summarized in Table 2.4.4-201, the elevations at BLN would also be lower than those determined from the original analysis (elevation 615.1 ft.). Therefore, additional conservatism is added into the seismically induced flood elevations discussed above that would more than account for the 2-yr. wind speed resulting elevation.

Based on the considerations above, the seismically induced dam failure analyses and associated flooding impacts were determined to be adequate and bounding.

It should be understood that these studies of postulated dam failures have been made solely to ensure the safety-related facilities of BLN are protected against floods caused by the assumed failure of dams because of seismic forces.

2.4.4.1 Dam Failure Permutations

According to guidance, seismic dam failure is to be examined using the SSE coincident with the peak of the 25-yr. flood, and operating basis earthquake OBE coincident with the peak of one-half PMF or 500-yr. flood. The guidance also specifies a 2-yr. wind speed applied in the critical direction.

The discussion in Subsection 2.4.4.1 is based on the flood analyses that were conducted in the 1970's (original analysis) and the 1998 reassessment. All references to SSE and OBE in this subsection are based on previous dam failure analyses and refer to SSE and OBE as defined per 10 CFR 100 Appendix A, consistent with guidance of Regulatory Guide 1.59, in accordance with the requirements of 10 CFR 100.23d(3).

There are several major dams above the BLN. Dam locations with respect to the BLN site are shown in Figure 2.4.4-202. Figure 2.4.4-203 presents a simplified flow diagram for the Tennessee River system. Table 2.4.4-203 provides the relative distances of structures to the BLN site. These structures were originally examined in the late 1970s and reassessed by the TVA in 1998 to address dam safety modifications since the original analyses. The results of the 1998 assessment are applicable to the current TVA system and the BLN. Details for TVA dams are provided in Table 2.4.4-204 and Table 2.4.4-205. Details for non-TVA dams are provided in Table 2.4.4-206.

The standard method of computing stability of concrete structures is used. The maximum base compressive stress, average base shear stress, the factor of safety against overturning, and the shear strength required for a shear-friction factor of safety of 1 are determined. To find the shear strength required to provide a safety factor of 1, a coefficient of friction of 0.65 is assigned at the elevation of the base under consideration.

The analyses for earthquakes are based on the pseudo-static analysis methods as given in Reference 208, with increased hydrodynamic pressures determined by the method developed by in Reference 204. These analyses include applying masonry inertia forces and increased water pressure to the structure resulting from the acceleration of the structure horizontally in the upstream direction and simultaneously in a downward direction. The masonry inertia forces are determined by a dynamic analysis of the structure which takes into account amplification of the accelerations above the foundation rock.

No reduction of hydrostatic or hydrodynamic forces because of the decrease of the unit weight of water from the downward acceleration of the reservoir bottom is included in this analysis.

Waves created at the free surface of the reservoir by an earthquake are considered of no importance. Based upon studies in Reference 205 and Reference 247, it is expected that before waves of any significant height have time to develop, the earthquake is over. The duration of earthquake used in this analysis is in the range of 20 to 30 seconds.

Although accumulated silt on the reservoir bottom would dampen vertically traveling waves, the effect of silt on structures is not considered. There is only a small amount of silt now present, and the accumulation rate is slow, as measured by TVA for many years.

Embankment analysis was made using the standard slip circle method, except for Chatuge and Nottely Dams where the Newmark method for dynamic analysis of embankment slopes was used. The effect of the earthquake is taken into account by applying the appropriate static inertia forces to the dam mass within the assumed slip circle.

In the analysis, the embankment design constants used, including shear strength of the materials in the dam and the foundation, are the same as those used in the original stability analysis.

Although detailed dynamic soil properties are not available, a value for seismic amplification through the soil has been assumed based on previous studies pertaining to TVA nuclear plants. These studies have indicated maximum amplification values slightly in excess of two for a rather wide range of shear wave velocity to soil height ratios. For these analyses, a straight-line variation is used with acceleration at the top of the embankment being two times the top of rock acceleration.

The SSE and OBE are defined as having maximum horizontal rock acceleration levels of 0.18 g and 0.09 g respectively. In order to fail three dams, Norris, Cherokee, and Douglas, in the SSE, the epicenter must be confined to a relatively small oval shaped area about 10 mi. wide and 20 mi. long. In order to fail four dams, Norris, Douglas, Fort Loudoun and Tellico, the epicenter of the SSE must be confined to a triangular area with sides approximately 1 mi. in length. However additional events were also considered. Of the events considered, the three events listed below had the potential to create maximum flood levels at the BLN. The 1998 reassessment found that all the events resulted in lower flood elevations. The results of the 1998 assessment are applicable to the current TVA system and the BLN.

1. Simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams in the OBE during one-half the PMF.

2. Simultaneous failure of Norris, Cherokee, and Douglas Dams in the SSE during a 25-year flood.

3. Simultaneous failure of Cherokee and Douglas Dams in the OBE during one-half the PMF.

Failure scenarios for Fontana Dam includes assumed simultaneous failure of non-TVA dams in the OBE or SSE on the Little Tennessee River and its tributary including Nantahala, Santeetlah, Cheoah, Calderwood, and Chilhowee Dams. The failure scenario for Norris Dam includes the subsequent overtopping failure of Melton Hill Dam.

The multiple structure failure scenarios are described in further detail below.

1. Fontana, Hiwassee, Apalachia, and Blue Ridge Dams

Original analysis found that simultaneous failure of Fontana, Hiwassee, Apalachia and Blue Ridge Dams in the OBE during one-half the PMF produced the maximum flood levels in the vicinity of the BLN. The flood elevation was 615.1 ft. msl. This failure scenario remains the worst combination of dam failures resulting in the maximum flood elevation, with respect to seismically induced failures. The result is less than the peak flow rate and maximum flood elevation resulting from the PMF, including hydrologic dam failures, as described in Subsection 2.4.3.

The OBE event produces maximum ground accelerations of 0.09 g at Fontana, 0.09 g at Hiwassee, 0.07 g at Apalachia, 0.08 g at Chatuge, 0.05 g at Nottely, 0.03 g at Ocoee No.1, 0.04 g at Blue Ridge, 0.04 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. The center 950 ft. portion of Fontana Dam is estimated to fail. Hiwassee, Apalachia, and Blue Ridge Dams are assumed to completely fail in the OBE. Chatuge is not expected to fail. See the single structure failure discussion for Chatuge below. The Fontana Dam failure is also discussed below.

Nottely Dam is a rockfill dam with large central impervious rolled fill core. The maximum attenuated ground acceleration at Nottely is only 0.054 g. A field exploration boring program and laboratory testing program of samples obtained was conducted. During the field exploration program, standard penetration tests blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. The Newmark Method of Analysis utilizing the information obtained from the testing program was used to determine the structural stability of Nottely Dam. It was concluded that Nottely Dam can resist the attenuated ground acceleration of 0.054 g with no detrimental damage.

Ocoee No.1 Dam is a concrete gravity structure. The maximum attenuated ground acceleration is 0.03 g. The 0.03 g with the proper amplification was used to analyze the structural stability of structures at Ocoee No.1. The concrete section method of analysis is previously described. The analysis results in low stresses and satisfactory factors of safety against sliding and overturning. It was concluded that Ocoee No.1 would not fail.

Fort Loudoun, Tellico, and Watts Bar spillways would remain operable. The Fontana failure wave would overtop and fail the Tellico embankment. Water would transfer into Fort Loudoun but it would not be sufficient to overtop the dam or to prevent overtopping failure of Tellico Dam. Watts Bar headwater would reach elevation 761.3 ft. msl, 5.7 ft. below the top of dam. The west saddle dike at Watts Bar with a top elevation of 757 ft. msl would be overtopped and breached. The saddle dike is assumed to fail completely to elevation 750 ft. msl.

The discharge from Watts Bar Dam and the failed saddle dike combined with the combined failure flow of Hiwassee, Apalachia, and Blue Ridge Dams would produce a maximum headwater elevation of 704.1 ft. msl at Chickamauga Dam, 1.9 ft. below the top of dam.

Routing was not carried below Chickamauga Dam to the BLN because the resulting flood elevation would be significantly lower than originally determined. The 1998 reassessment results indicate that lower flood elevations than the PMF, and the dam safety modifications at Nickajack Dam would provide additional attenuation. The dam safety modifications at Guntersville Dam would have no effect on a reassessment because the dam was not overtopped in the original analysis.

2. Norris, Cherokee, and Douglas Dams

The SSE event produces maximum ground accelerations of 0.15 g at Norris, 0.09 g at Cherokee and Douglas, 0.08 g at Fort Loudoun and Tellico, 0.05 g at Fontana, and 0.03 g at Watts Bar. Fort Loudoun, Tellico, and Watts Bar are not expected to fail. See the single structure failure discussion for each dam below. The bridge at Fort Loudoun Dam might fail, falling on any open gates and on gate-hoisting machinery. Trunnion anchor bolts of open gates would fail and the gates would be washed downstream, leaving an open spillway. Closed gates could not be opened. The most conservative assumption was used that at the time of the seismic event on the upstream tributary dams, the crest of the 25-year flood would likely have passed Fort Loudoun and flows would have been reduced to turbine capacity. Hence spillway gates would be closed. Fontana Dam was excluded on the basis of its distant location from the cluster of dams under consideration.

The center 833 ft. failure section of Norris Dam includes the spillway and intake portions of the dam. The resulting debris downstream would occupy the valley cross section with a top elevation of 970 ft. msl. The discharge rating for this controlling debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified by mathematical analysis. The SSE will produce the same postulated failures of Cherokee and Douglas Dams as in the OBE. The failure of Cherokee and Douglas Dams are described below.

The flood for the postulated failure combination would overtop and breach Fort Loudoun Dam. Although transfer of water into Tellico would occur, the maximum headwater would only reach elevation 820 ft. msl which is 10 ft. below top of dam. The headwater at Watts Bar Dam would reach elevation 764.9 ft. msl, 2.1 ft. below top of dam. The west saddle dike would be overtopped and breached. At Chickamauga Dam the headwater elevation would reach 702 ft. msl, 4 ft. below top of dam. Routing was not carried below Chickamauga Dam as this flood would not present a problem at the BLN. The elevation would be significantly lower than 612.7 ft. msl originally determined.

3. Cherokee and Douglas Dams

The results of the Cherokee Dam stability analysis in the OBE indicate the spillway is stable at the foundation base elevation 900 ft. Analyses made for other elevations indicate the resultant forces fall outside the base at elevation 1010 ft. msl. The spillway is assumed to fail at that elevation. The non-overflow dam is embedded in fill to elevation 981.5 ft. msl and is considered stable below that elevation. However, stability analysis indicates failure will occur above the fill line.

Analysis was made for the highest portion of the south embankment using the same shear strengths of material as were used in the original analysis. The resulting factor of safety was 0.85. Therefore, the south embankment is assumed to fail. Because the north embankment and saddle dams 1, 2, and 3 are generally about one-half or less as high as the south embankment, they are expected to be stable.

All debris from the failure of the concrete portion is assumed to be located downstream in the channel at elevations lower than the remaining portions of the dam, and therefore, will not obstruct flow. The powerhouse intake is massive and backed up by the powerhouse. Therefore, it is expected to be stable.

The upper part of the Douglas spillway is approximately 12 ft. higher than Cherokee, but the amplification of the rock surface acceleration is the same. Therefore, based on the Cherokee analysis, it is projected that the Douglas spillway will fail at elevation 937 ft. msl, which corresponds to the assumed failure elevation of the Cherokee spillway.

The Douglas non-overflow dam is similar to that at Cherokee and is embedded in fill to elevation 927.5 ft. msl. The spillway is considered stable below that elevation. However, based on the Cherokee analysis, it is assumed to fail above the fill line in the OBE. The powerhouse intake is massive and backed up downstream by the powerhouse. Therefore, it is considered stable. This results in a 538 ft. failure section including the spillway and portions of the non-overflow dam to the left abutment side of the powerhouse. Additionally, there is a 279 ft. failure section of the non-overflow dam to the right abutment side of the powerhouse.

All debris from the failed portions is assumed to be located downstream in the channel at elevations lower than the remaining portions of the dam, and therefore, will not obstruct flow. The result of the original analysis of the saddle dam indicates a factor of safety of one. Therefore, the saddle dam is considered to be stable.

The postulated failure combination would reach a maximum headwater elevation of 833.8 ft. msl at Fort Loudoun Dam, 0.55 ft. above the top of dam. Fort Loudoun would be overtopped for only about six hours to a maximum depth of 0.55 ft. Breaching analysis indicates that this short overtopping time and shallow overflow depth would not fail the dam. Although transfer of water into Tellico would occur, the maximum headwater would only reach elevation 826 ft. msl which is 4 ft. below top of dam. At Watts Bar Dam the headwater would reach elevation 758.2 ft. msl, 8.8 ft. below top of dam. The west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. The headwater at Chickamauga Dam would reach elevation 697.8 ft. msl, 8.2 ft. below top of dam. Routing was not carried downstream of Chickamauga Dam as this flood would not present a problem at the BLN. The elevation would be significantly lower than 614.2 ft. msl originally determined.

Three additional events were evaluated and eliminated based on the results associated with the Sequoyah Nuclear Plant evaluation in comparison with the results of the above listed events at the Sequoyah Nuclear Plant. The three additional events are listed below.

4. Failure of Fontana Dam in the OBE during one-half the PMF.

5. Simultaneous failure of Norris, Douglas, Fort Loudoun, and Tellico Dams in the SSE during a 25-year flood.

6. Failure of Norris Dam in the OBE during one-half the PMF.

4. Fontana Dam

Fontana Dam was assumed to fail in the OBE, although no stability analysis was made. Fontana Dam is a high dam constructed with three longitudinal contraction joints in the higher blocks. A structural defect was found in October 1972 and consists of a longitudinal crack in three blocks in the curved portion at the left abutment. Strengthening of these blocks by post-tensioning and grouting of the cracks was completed October 1973. Only these three blocks are cracked, and there is no evidence that any other portion of the dam is weakened.

The strengthening work has reestablished the structural integrity of the cracked blocks. Although the joints are keyed and grouted, the conservative assumption is that Fontana Dam will not resist the OBE without failure. The center 950 ft. of the structure is projected to fail, depositing debris in the downstream channel. The elevation of debris is estimated to be between 1455 ft. and 1500 ft. msl.

Although not investigated, it was assumed that Nantahala Dam, upstream from Fontana and Santeetlah on a downstream tributary, and the three ALCOA dams, downstream on the Little Tennessee River, Cheoah, Calderwood, and Chilhowee, would fail along with Fontana in the OBE. Instant disappearance is assumed. Tellico and Watts Bar Dam spillway gates would remain operable during and after the OBE. Failure of the bridge at Fort Loudoun would render the spillway gates inoperable in the wide-open position.

The Fontana failure wave would overtop and fail the Tellico embankment.Transfer of water into Fort Loudoun would occur but would not be sufficient to overtop the dam or prevent overtopping and failure of Tellico Dam. Tellico was assumed to completely fail. Watts Bar headwater would reach elevation 761.3 ft. msl. This is 5.7 ft. below top of dam. However, the west saddle dike at Watts Bar with top at elevation 757 ft. msl would be overtopped and breached. A complete washout of the dike down to ground elevation 750 ft. msl was assumed. The headwater at Chickamauga Dam would reach 699.8 ft. msl, 6.2 ft. below top of dam. Routing for this event was not carried below Chickamauga Dam because the simultaneous failure of Hiwassee, Apalachia, and Blue Ridge together with Fontana, as previously discussed, is more critical.

5. Norris, Douglas, Fort Loudoun, and Tellico Dams

The SSE event produces maximum ground accelerations with attenuation of 0.12 g at Norris, 0.08 g at Douglas, 0.12 g at Fort Loudoun and Tellico, 0.07 g at Cherokee, 0.06 g at Fontana, and 0.04 g at Watts Bar. Cherokee is not expected to fail at 0.07 g. Watts Bar is also not expected to fail at 0.04 g. Fontana Dam was excluded on the basis of its distant location from the cluster of dams under consideration.

The postulated failure of Norris Dam is the same as previously discussed. The SSE will produce the same postulated failure of Douglas Dam as previously discussed for the OBE.

The results of the Fort Loudoun Dam stability analysis indicate the spillway section will fail. Based on the analyses of Cherokee and Douglas, the entire spillway section is projected to fail above elevation 750 ft. msl, as well as the bridge supported by the spillway piers. The results of the slip circle analysis for the highest portion of the embankment indicate a factor of safety less than one. The entire embankment is assumed to fail. All debris from failure of the concrete portions is assumed to be located in the channel below the failure elevations.

No analysis was made for the powerhouse under SSE. However, an analysis was made for the OBE with no water in the units, a condition believed to be extremely remote to occur. Because stresses were low a large percentage of the base was in compression, it is considered that the addition of water in the units would be a stabilizing factor, and the powerhouse section is not expected to fail.

No structural analysis was made for Tellico Dam failure in the SSE. Because of the similarity to Fort Loudoun, the spillway and entire embankment are projected to fail in a manner similar to Fort Loudoun. All debris is assumed located in the channel below the failure elevation.

The postulated failure combination results in Watts Bar headwater elevation of 758.9 ft. msl, 8.1 ft. below top of dam. The west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. The maximum headwater would reach elevation 695.8 ft. msl at Chickamauga Dam, 10.2 ft. below top of dam. Routing was not carried below Chickamauga Dam as this flood would not present a problem at the BLN. Based on the previously described analyses, the elevation at the BLN would be significantly lower than determined by those analyses.

6. Norris Dam

The results of the Norris Dam stability analysis in the OBE for a typical spillway block and typical non-overflow section of the maximum height indicate only a small percentage of the spillway base is in compression. The high non-overflow section also resulted with a small percentage of the base in compression and with high compressive and shearing stresses.

The center 665 ft. failure section of Norris Dam includes the spillway and intake portions of the dam. Based on stability analysis the remaining non-overflow section is expected to withstand the OBE. The resulting debris downstream would occupy the valley cross section with a top elevation of 970 ft. msl. The discharge rating for this controlling debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified by mathematical analysis.

The Norris failure wave would overtop Melton Hill Dam. Melton Hill Dam was postulated to completely fail when the flood wave reached headwater elevation 804 ft. msl. The headwater at Watts Bar Dam would reach elevation 758.1 ft. msl, 8.9 ft. below the dam. The west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. Chickamauga headwater would reach elevation 694.5 ft. msl, 11.5 ft. below top of dam. Routing was not carried below Chickamauga Dam as this flood would present no problem at the BLN. Based on the previously described analyses, the elevation at the BLN would be significantly lower than determined by those analyses.

Additional structure failures in the OBE and SSE are discussed below. These scenarios are non-controlling and would result in flood levels less than those previously described.

Chickamauga and Nickajack Dams

The original seismic failure analyses also considered the potential effects of failure of Chickamauga and Nickajack Dams at the BLN. The dams were not analyzed structurally. Instead Chickamauga and Nickajack Dams were considered to fail instantly and completely, both singly and simultaneously during the one-half PMF. This corresponds to the OBE event. A reevaluation has not been made, but flood levels from simultaneous failure of both dams would not differ significantly from the original analysis result of elevation 609 ft. msl at the BLN.

Watts Bar Dam

Stability analysis of Watts Bar Dam powerhouse and spillway sections in the OBE result in the expectation that these structures will not fail. The analyses show low stresses with about 38 percent of the spillway base in compression and about 42 percent of the powerhouse base in compression. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base. The slip circle analysis of the earth embankment section results in a factor of safety of 1.52, and the embankment is not expected to fail.

Normally for the condition of peak discharge at the dam for one-half the PMF, the spillway gates would be in a wide open position. However, analysis of the bridge structure for forces resulting from the OBE, including amplification of acceleration results in the determination that the bridge could fail as a result of shearing the anchor bolts. The downstream bridge girders could strike the spillway gates. The impact of the girders striking the gates could fail the bolts which anchor the gate trunnions to the pier allowing the gates to fall. The flow over the spillway crest would be the same as that prior to bridge and gate failure. Hence, bridge failure will cause no adverse effect on the flood.

A potentially severe condition is the bridge falling when most spillway gates would be closed. The gate hoisting machinery would be inoperable after being struck by the bridge. As a result, the flood would crest with the gates closed and the bridge deck and girders lying on top of the spillway piers. Analysis of the concrete portions of the dam for the headwater for this condition shows that they will not fail.

For the condition described above with the most probable embankment breaching, the outflow would increase rapidly from about 200,000 cfs to 660,000 cfs when breaching is complete. Breach time would be about 5 hours. Flood levels would not be controlling.

For flow conditions between the 25-year flood and one-half the PMF, when the bottom of the gates are in the water, failure of the bridge during the OBE with consequent striking of the gates by the downstream bridge girders will result in failure of the gate lifting chains. The gates will rotate to the closed position. This condition is less severe than that described above.

A reevaluation was not made for Watts Bar Dam in SSE conditions. Previous evaluation determined that if the dam is arbitrarily removed instantly, the flood levels would not be controlling.

Fort Loudoun Dam

Stability analysis of Fort Loudoun Dam powerhouse and spillway sections indicate these structures will not fail in the OBE. Slip circle analysis of the earth embankment results in a factor of safety of 1.26, indicating the embankment is not expected to fail.

The spillway gates and bridge are of the same design as those at Watts Bar Dam. Conditions of failure during the OBE are the same and no problems are likely. Coincident failure at Fort Loudoun and Watts Bar does not occur.

For the potentially critical case of Fort Loudoun bridge failure at the onset of the main portion of one-half PMF flow into Fort Loudoun Reservoir, it was found that the Watts Bar inflows are much less than the condition resulting from simultaneous failure of Cherokee and Douglas.

No hydrologic routing for the single failure of Fort Loudoun in the SSE, including the bridge structure, is made because its simultaneous failure with Tellico and Fontana, as well as with Tellico, Norris, and Douglas, are controlling.

Tellico Dam

Results of the stability analyses in the OBE for a typical non-overflow block and typical spillway block indicate no part of Tellico is expected to fail. Stability analysis of the earth embankment results in a factor of safety of 1.28, indicating failure is not expected. No routing for the single failure of Tellico in the SSE is made for similar reasons provided for Fort Loudoun.

Norris Dam

The postulated single failure of Norris Dam in the SSE would result in peak headwater at Watts Bar below the top of the earth portions of the dam. Routing was not carried further. It is evident that flood levels would be considerably lower than for Norris failure in the OBE as previously discussed.

Cherokee and Douglas Dams Separately

No hydrologic results are given for the single failure of Cherokee or Douglas Dams in the OBE because the simultaneous failure of Cherokee and Douglas, as previously discussed, is more critical. The SSE will produce the same postulated failure for Cherokee and Douglas as described for the OBE. The SSE failure would produce flood elevations less than the OBE failure.

Hiwassee, Apalachia, Blue Ridge, Ocoee #1, and Nottely Dams Separately

Each dam was assumed to fail singly in the OBE. No hydrologic results are given for the single failure of each dam because the simultaneous failure with other dams is more critical. No routing for the failure of each dam singly in the SSE is made because their simultaneous failure with Fontana is more critical.

Chatuge Dam

Chatuge Dam is a homogeneous, impervious rolled-fill dam. With the epicenter of the OBE located at the dam, the maximum ground acceleration is 0.09 g. Ground accelerations of this magnitude would have no detrimental effect on a well-constructed compacted earthfill embankment. There are no known failures of compacted earth embankment slopes from earthquake motions. Failures to date have been associated with liquefaction of hydraulic fill embankments or with other loose granular foundation materials. The rolled embankment materials in Chatuge are not sensitive to liquefaction.

A field exploration boring program and laboratory testing program of samples obtained was conducted. During the field exploration program, standard penetration tests blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. The Newmark Method of Analysis utilizing the information obtained from the testing program was used to determine the structural stability of Chatuge Dam. It was concluded that Chatuge Dam can resist the ground acceleration of 0.09 g with no detrimental damage.

No routing for the failure of Chatuge Dam singly in the SSE is made because simultaneous failure with Fontana is more critical.

Fort Loudoun, Tellico, and Fontana Dams

An SSE centered between Fontana and the Fort Loudoun-Tellico complex was postulated to fail these three dams. The four ALCOA dams downstream from Fontana and Nantahala, an ALCOA dam, upstream were also postulated to fail completely in this event. Watts Bar Dam and spillway gates would remain intact, but failure of the roadway bridge was postulated which would render the spillway gates inoperable. At the time of seismic failure, discharges would be small in the coincident 25-year flood. For conservatism, Watts Bar gates were assumed inoperable in the closed position after the SSE event. The resulting flood levels would not be controlling.

Douglas and Fontana Dams

Douglas and Fontana were postulated to fail simultaneously in the SSE. The location of the SSE required to fail both dams would produce 0.14 g at Douglas, 0.09 g at Fontana, 0.07 g at Cherokee, 0.05 g at Norris, 0.06 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. The postulated failures of Douglas and

Fontana would be similar to that for each dam included with multiple failures previously described. Fort Loudoun, Tellico, and Watts Bar have previously been are not expected to fail in the OBE, as previously discussed. The bridge at Fort Loudoun Dam, however, might fail under 0.06 g forces, falling on gates and on gate hoisting machinery. Fort Loudoun gates were assumed inoperable in the closed position following the SSE event. The resulting flood levels would not be controlling.

Fontana and Hiwassee River Dams

Fontana and six Hiwassee River Dams, Hiwassee, Apalachia, Chatuge, Nottely, Blue Ridge, and Ocoee No. 1, were postulated to fail simultaneously in the SSE. The postulated failure of Fontana would be similar to that included with the multiple failures previously described. The six Hiwassee dams were assumed to fail completely. Fort Loudoun, Tellico, and Watts Bar are judged not to fail with all gates operable. The Fontana surge combined with that of the six Hiwassee River dams would not be controlling.

Raccoon Mountain Dam

Raccoon Mountain pumped storage dam was not analyzed because of its small capacity, 37,800 ac.-ft., and its considerable upstream distance, 53 mi. Its complete and coincidental failure would not add measurably to the flood level.

There are no safety-related facilities that could be affected by loss of water supply due to dam failure. This is addressed further in Subsection 2.4.11. Additionally, there are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during dam failure-induced flooding. Landslide potential is addressed in Subsection 2.4.9. There are no onsite water control or storage structures located above site grade that may induce flooding. There are no safety-related structures that could be affected by waterborne objects.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The unsteady flow models are described in Subsection 2.4.3.3. Unsteady flow routing techniques were applied to reservoirs in sufficient detail to define the manner in which the reservoir would supply and sustain outflow following postulated dam failure.

2.4.4.3 Water Level at the Plant Site

The translation of flow to elevation is discussed in Subsection 2.4.3.3. The greatest flood elevation as a result of seismically induced multiple dam failures would be less than 615.1 ft. msl. Table 2.4.4-201 provides a summary of results for the original analyses and the 1998 reassessment. The results of the 1998 assessment are applicable to the current TVA system and the BLN. Routing was not carried beyond Chickamauga Dam. The 1998 assessment resulted in lower water surface elevations at Chickamauga Dam when compared to the original analysis. Therefore, water surface elevations at the BLN would be less than determined by the original analysis and subsequent routing was not warranted. Coincident wind wave activity is described in Subsection 2.4.3.6. Superimposed wind wave activity would not result in a water surface elevation exceeding the PMF described in Subsection 2.4.3.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

Regulatory Guide 1.59 describes the probable maximum surge and seiche flooding based on a probable maximum hurricane (PMH), probable maximum windstorm (PMWS), or moving squall line. The region of occurrence for a PMH is along U.S. coastline areas (Reference 203). The PMWS region of occurrence is along coastline areas and large bodies of water such as the Great Lakes. A moving squall line is considered for the Great Lakes region.

According to U.S. Army Corps of Engineers guidelines, meteorological wind systems generated by thunderstorms and frontal squall lines can generate waves up to 16.4 ft. high for inland waters (Reference 231). Additionally, mesoscale convective complex wind systems affecting inland waters are fetch-limited and based on wind speeds of up to about 66 fps or 45 mph. Similar wind speeds are used to determine the coincident wind-generated wave activity discussed in Subsection 2.4.3.

The U.S. Army Corps of Engineers guideline procedures for geologic hazard evaluations consider seiche waves greater than 7 ft. to be rare (Reference 237). According to U.S. Army Corps of Engineers guidance, the seiche hazard can be screened out for sites located more than 7 ft. above the adjacent water body.

The BLN is located approximately 300 mi. inland from the Gulf Coast. Safety-related facilities are located at elevation 628.6 ft. The normal maximum water surface elevation of Guntersville Reservoir is 595 ft. The top of gates at Guntersville Dam, is 595.4 ft. (Reference 222). According to the Tennessee Valley Authority (TVA) 100-year flood risk profile, the Tennessee River at the BLN is about 5 ft. higher than the reported water surface elevation at Guntersville Dam (Reference 224).

The Tennessee River does not connect directly with any of the bodies of water considered for meteorological events associated with surge and seiche flooding. There are no known documented surge or seiche occurrences on the Tennessee River. Based on data provided above, and site location and elevation characteristics, the BLN safety-related facilities are not considered at risk from surge and seiche flooding. Additionally, there are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during storm surge or seiching.

2.4.6 PROBABLE MAXIMUM TSUNAMI HAZARDS

The U.S. Army Corps of Engineers has developed a general tsunami risk map (Figure 2.4.6-201) (Reference 237). Both the East Coast and the Gulf Coast are located in Zone 1, which corresponds to a wave height of 5 ft. According to the National Oceanic & Atmospheric Administration (NOAA) tsunami database (Reference 213), the maximum recorded tsunami wave height located along the Gulf Coast or East Coast is about 20 ft. at Daytona Beach, Florida on July 3, 1992. The database notes that the wave was probably meteorologically induced.

The BLN is located approximately 300 mi. inland from the Gulf Coast. Safety-related facilities are located at elevation 628.6 ft. Based on data provided above, and site location and elevation characteristics, the BLN safety-related facilities are not considered at risk from tsunami flooding.

2.4.7 ICE EFFECTS

There are nine U.S. Geological Survey (USGS) gauging stations, located upstream of the BLN on the Tennessee River from Knoxville, Tennessee, to South Pittsburg, Tennessee, that recorded water temperatures for different periods between 1967 and 2002 (Reference 244). The lowest recorded water temperatures during winter periods range from 35.6 °F to 42.8 °F. The lowest was recorded in the tailwater of Watts Bar Dam (USGS No. 03543005), located about 137 river mi. upstream of the BLN. There are also five USGS gauging stations in Alabama, located downstream of the BLN and below Guntersville dam, which recorded water temperatures for different periods between 1956 and 2000 (Reference 244). The gauge at Whitesburg, Alabama (USGS No. 03575500), located about 57 river mi. downstream of the BLN, recorded the only winter water temperatures. The lowest recorded water temperature was 37.4 °F.

The USGS gauging station at South Pittsburg, Tennessee (USGS No. 03571850), located about 26 river mi. upstream of the BLN and below Nickajack Dam, is most representative of water temperatures near the site. The South Pittsburg gauge recorded water temperatures for different periods between 1967 and 1987. The lowest recorded water temperature was 36.5 °F. A summary of the gauge is presented in Table 2.4.7-201.

According to the EPA STORET database (Reference 240), 44 stations in Jackson County located on Guntersville Reservoir recorded water temperatures between 1960 and 1997. The lowest water temperature recorded was 35.6 °F near Bridgeport, Alabama, about 16 river mi. upstream of the BLN (Station 10077). Three stations located on the Tennessee River adjacent to the BLN recorded water temperatures between 1973 and 1991. The lowest water temperature recorded was 39.2 °F (Station 17100).

TVA water temperature data recorded from 1998 to 2005 only includes the months from April to September. Water temperature was measured at points about

16.3 river mi. and 41.5 river mi. downstream. The lowest recorded water temperature was 55.6 °F.

Additional TVA water temperature data was recorded from 2000 to 2006 at the Nickajack Dam tailwater releases, about 33.2 river mi. upstream; the Widows Creek Fossil Plant intake, about 16.7 river mi. upstream; and the Guntersville Dam tailwater releases, about 42.5 river mi. downstream. Minimum monthly temperatures are provided in Table 2.4.7-202. The lowest recorded water temperature was 40.2 °F at the Widows Creek Fossil Plant intake and the Guntersville Dam tailwater releases. EPA STORET data indicates that the Guntersville tailrace temperatures reach as low as 35.6 °F (Station 16909). Water temperature data suggests that the Tennessee River generally remains above the freezing point at the BLN significantly reducing the potential for ice effects.

According to the U.S. Army Corps of Engineers (Reference 236), ice jams occur in 36 states, primarily in the northern tier of the United States (Figure 2.4.7-201). Neither Alabama nor Tennessee is included in this coverage. The U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory historical ice jam database was consulted. One recorded occurrence of an ice jam on the Tennessee River was found (Reference 235). The Tennessee River was obstructed by rough ice at Guntersville, Alabama from January 29, 1940 to February 1, 1940. The maximum stage was not recorded. Flows recorded at the South Pittsburg USGS gauge noticeably decreased, while flows recorded at the Whitesburg USGS gauge noticeably increased, during this time period. There are no known documented ice sheet or ice ridge occurrences on the Tennessee River.

During the winter of 1899, a cold wave struck most of the nation. Its effects are documented by the NOAA's National Climatic Data Center. It was reported that heavy ice was floating in the Tennessee and Cumberland Rivers from February 13, 1899 to February 18, 1899 (Reference 201).

The BLN safety-related facilities are located at elevation 628.6 ft. The normal maximum water surface elevation for the Tennessee River at Guntersville Dam is 595 ft. Due to operation of the TVA hydropower projects within the drainage basin and specifically Guntersville Dam, the winter operating water surface elevation is 593 ft. The Tennessee River at the BLN is about 5 ft. higher than the reported water surface elevation at Guntersville Dam, according to the TVA 100-year flood risk profile (Reference 224). The possibility of inundating the BLN due to an ice jam appears to be remote.

According to the U.S. Army Corps of Engineers (Reference 236), frazil ice forms in supercooled, turbulent water in rivers and lakes. Anchor ice is identified as frazil ice attached to the river bottom, irrespective of the nature of its formation. The Tennessee River at the BLN has little potential for freezing (i.e., frazil or anchor ice) and subsequent ice jams. Sustained periods of subfreezing water temperatures are not characteristic of the region. Although ice has formed along shorelines and across protected inlets, the climate and operation of Guntersville Reservoir prevent any significant icing in the vicinity of the BLN. Based on data prior to 1978, the probability of glaze ice with a thickness of 1 in. or more in any year is 0.004.

According to U.S. Army Corps of Engineers methods (Reference 233), the maximum potential ice thickness is a function of accumulated freezing degree days (AFDD). The average maximum AFDD for the site is about 100 °F days (Reference 210). The resulting maximum potential ice thickness is 6 in. There are no safety-related facilities that could be affected by ice-induced low flow of the Tennessee River or reduction in capacity of water storage facilities.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

BLN COL 2.4-1 There are no current or proposed safety-related cooling water canals or reservoirs required for the BLN. The existing intake channel and intake structure are used to supply raw water from the Tennessee River to the existing cooling towers functioning as the normal heat sink. The normal heat sink water requirements are discussed in Subsection 2.4.11.5. The atmosphere provides the ultimate heat sink (UHS) with the containment vessel and passive containment cooling system (PCS) providing the heat transfer mechanism. Makeup water to the PCS is provided by a connection to the municipal water system. UHS water requirements are discussed in Subsection 2.4.11.6.

2.4.9 CHANNEL DIVERSIONS

There is no evidence suggesting there have been significant historical diversions or realignments of the Tennessee River. The topography does not suggest any
BLN COL 2.4-3 potential diversions. The streams and rivers in the region are characterized by traditional shaped valleys with no steep, unstable side slopes that could contribute to landslide cutoffs or diversions.

Several instream dams are located on the Tennessee River and primarily used for hydroelectric power, navigation, and flood control (Reference 211). The TVA owns and operates the dams on the Tennessee River, providing the necessary flow releases from the dams in the system. Nickajack Dam, completed in 1967 and replacing the older Hales Bar Dam, is located upstream from the BLN, but is not used for flood control. Chickamauga Dam, completed in 1940, is located further upstream and provides flood control for Chattanooga, Tennessee. Guntersville Dam, completed in 1939, is located downstream.

There are several diversions in the TVA system (Reference 211). However, none of the diversions directly affect the BLN. Upstream of BLN, the Little Tennessee River was diverted by construction of Tellico Dam and Canal in 1979. This closure forces the Little Tennessee River to flow into the Tennessee River above Fort Loudoun Dam rather than below the dam, as it had prior to 1979. The Tennessee-

Tombigbee Waterway, located downstream of the BLN, was constructed using a series of locks and dams to connect the Tennessee and Tombigbee rivers for navigation. Additionally, further downstream near the junction with the Ohio River, there is a short navigation canal joining the Tennessee River at Kentucky Dam and the Cumberland River at Barkley Dam.

Channel diversion due to geothermal activity was also investigated and is not expected. The greatest potential for geothermal energy exists in areas of above average heat flow, generally the result of recent volcanic activity or active tectonics. The eastern United States has below average to average geothermal heat flow and is characterized as low temperature (Reference 216). The eastern United States is relatively tectonically stable (Reference 217). No thermal anomalies in the eastern United States are attributed to young-to-contemporary volcanic or other igneous activity (Reference 245).

The atmosphere provides the UHS with the containment vessel and PCS providing the heat transfer mechanism. The UHS does not directly rely on the Tennessee River intake. Therefore, channel diversion can not adversely affect safety-related structures or systems. Additional details are provided in Subsection 2.4.11. The potential for ice-induced diversion and flooding is discussed in Subsection 2.4.7. Geologic and seismic characteristics of the region are discussed in Section 2.5.

2.4.10 FLOODING PROTECTION REQUIREMENTS

BLN COL 2.4-2 Safety-related SSCs are not exposed to flooding from the probable maximum flood (PMF) levels identified by the evaluation described in Subsection 2.4.2. The critical flooding event is identified in Subsection 2.4.2 and discussed in detail in Subsection 2.4.3. The maximum flood levels are a result of the probable maximum precipitation and include the effects of upstream dam failures and coincident wind wave activity. Based on the design information provided above, flood protection measures and emergency procedures to address flood protection are not required.

2.4.11 LOW WATER CONSIDERATIONS

2.4.11.1 Low Flow in Rivers and Streams

BLN COL 2.4-3 The major tributaries of the Tennessee River originate in the higher elevations of the Appalachian Mountains of Virginia, North Carolina and Tennessee. The BLN is located above the western shore of Guntersville Reservoir at Tennessee River Mile 391.5 (Reference 222). Guntersville Reservoir is bounded by Guntersville Dam downstream and Nickajack Dam upstream.

The Tennessee River drainage basin above the BLN derives water from several smaller tributaries that contain numerous dams (Reference 234). Therefore, low flow conditions of the Tennessee River are a function of natural flow in the rivers and streams, available storage capacity of upstream reservoirs, and regulated discharge flow from upstream dams.

Dam failure could affect low flow conditions. There are no safety-related facilities that could be affected by low flow or drought conditions of the Tennessee River. Failure of Guntersville Dam would drain Guntersville Reservoir. Flow control could be shifted to Nickajack Dam upstream. Coincident loss of both Guntersville and Nickajack would additionally drain Nickajack Lake. However, flow control could be shifted to Chickamauga Dam. Adequate non-safety related water supply during a 100-year drought is addressed in Subsection 2.4.11.5.

2.4.11.2 Low water resulting from Surges, Seiches, or Tsunami

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

Flooding because of ice jams has not previously been recorded at the site. It is unlikely that an ice jam would occur because of the water temperatures at the site. Therefore, low flow because of, or exaggerated by, ice effects is not expected to occur at the site. See Subsection 2.4.7 for additional details.

2.4.11.3 Historical Low Water

The Tennessee River upstream of the BLN is primarily regulated by the TVA dams on the Tennessee River and its major tributaries: the Clinch, French Broad, Holston, Little Tennessee, Hiwassee, and Ocoee Rivers (Reference 211). The flood control dams in the drainage basin not owned by the TVA have small to insignificant storage capacity compared to the TVA system (Reference 234). Five major reservoirs provide almost 90 percent of the flood storage capacity of the watershed above Chattanooga Tennessee (Reference 211). Statistically based analysis of available streamflow records may be used to estimate low-flow values in the Tennessee River even though the drainage basin is regulated.

Low-flow conditions at the BLN were analyzed based on streamflow records at USGS gauging stations on the Tennessee River (Reference 244). Portions of the drainage basin upstream of the BLN have been regulated to some degree since the early 1910’s. Hales Bar Dam, completed in 1913 and later decommissioned, was located just upstream of Nickajack Dam, completed in 1967. Neither dam is used for flood control. The major flood control dams had been completed by the TVA by 1944. Several smaller flood control structures were completed by 1952 at the latest. The significant changes to the watershed had been completed by 1979 (References 211, 229 and 234).

The South Pittsburg gauging station (USGS No. 03571850) is located about 26 river mi. upstream of the BLN and below Nickajack Dam. The annual minimum daily flows for the period of record daily flow from 1930 to 1987 are presented in Table 2.4.11-201. The minimum flow observed during the period of record is 2900 cfs on November 1 and 15, 1953. The drainage area of the Tennessee River at the BLN is 23,340 sq. mi. (Reference 222). The South Pittsburg gauge has a drainage area of 22,640 sq. mi. This represents about 97 percent of the drainage area at the site.

Because the South Pittsburg gauge was discontinued in 1987, gauges at upstream Chattanooga, TN (USGS No. 03568000) and downstream Whitesburg, AL (USGS No. 03575500) were used to estimate more recent minimum flow data. For the period of record from 1988 to 2005, the BLN minimum daily flows were interpolated based on the Chattanooga and Whitesburg gauges. The annual minimum daily flows for the Chattanooga and Whitesburg gauges are presented in Table 2.4.11-202. The gauge locations are shown relative to the BLN in Figure 2.4.11-201.

Low-flow frequency analysis was performed in accordance with USGS Bulletin 17B using the Log-Pearson Type III distribution method (References 218, 232, and 243). The South Pittsburg gauge period of record from 1953 to 1987, supplemented with interpolated data from USGS gauges on the Tennessee River at Chattanooga and Whitesburg from 1988 to 2005, was used to analyze the current regulated condition of flows past the BLN. Table 2.4.11-203 provides 1-, 7-, and 30-day low flows for different recurrence intervals. Low-flow frequency plots for 1, 7, and 30 days are provided in Figures 2.4.11-202, 2.4.11-203, and 2.4.11-204.

The Guntersville gauging station (USGS No. 03573500) is located about 33 river mi. downstream of the site and has a drainage area of 24,340 sq. mi. The annual minimum daily flows for the period of record of daily flow from 1930 to 1938 are presented in Table 2.4.11-204. Guntersville Dam was completed in 1939. The minimum flow observed during the period of record is 5940 cfs on October 28, 1931.

Historical maps of the Palmer Drought Index, indicating drought conditions over various time periods were obtained from the National Drought Mitigation Center (Reference 212). The Guntersville Reservoir region experienced drought conditions 30 – 40 percent of the time during 1986, according to the drought maps. During 1988, the region experienced drought conditions 20 – 30 percent of the time. Drought conditions were experienced 10 – 20 percent of the time from 1928 to 1934, from 1930 to 1939, and from 1954 to 1956. The more severe drought conditions of 1986 and those from 1954 to 1956 are included in the data used for low-flow frequency analysis.

Because the flow past the BLN is completely regulated, water levels depend on the TVA owned and operated Nickajack Dam upstream and Guntersville Dam downstream. Hourly flows at the BLN may vary considerably from daily average flows, depending on turbine operations at the Nickajack and Guntersville Hydro Plants. Hourly flows may be zero or may be in an upstream direction for up to 6 hours per day (Reference 238).

The normal minimum pool level of Guntersville Reservoir is 593 ft. Normal full pool is at elevation 595 ft. and the top of gates is 595.4 ft. The reservoir may be drawn down to elevation 591 ft. during flood control operations (Reference 222). After the start of operation in April 1940 the minimum reservoir level was 590.7 ft. on November 12, 1968. The maximum reservoir level was 596.3 ft. on March 2, 1944. Reservoir levels are recorded at the Guntersville Dam, which has a drainage area of approximately 24,450 sq. mi. (Reference 215). According to the TVA 100-year flood risk profile, the Tennessee River at the BLN is about 5 ft. higher than the reported water surface elevation at Guntersville Dam (Reference 224).

The U.S. Army Corps of Engineers historical database of ice jams on the Tennessee River was reviewed (Reference 235). See Subsection 2.4.7 for additional discussion. Ice effects are not a concern for low water considerations, due to the climate and reservoir operations.

2.4.11.4 Future Controls

Total water use in the Tennessee River watershed from 2000 to 2030 is forecasted to increase by 15 percent, according to the TVA reservoir operations study (Reference 226). This is an increase from 12,211 million gpd to 13,990 million gpd. By 2030, the projected increase in flow to the Tennessee-Tombigbee Waterway for barge traffic ranges from 36 to 193 million gpd. The increase could be as much as 600 million gpd if traffic through the waterway reaches design capacity. The Tennessee-Tombigbee Waterway is located on Pickwick Lake downstream of Guntersville Reservoir.

Inter-basin transfers of water to areas immediately adjacent to the Tennessee watershed are currently estimated to be about 5.6 million gpd. Future water demands are expected to increase the inter-basin transfers to about 27 million gpd by 2030. It is speculated that additional future transfers to more distant areas could reach 461 million gpd by 2030.

According to the USGS study associated with the TVA reservoir operations study (Reference 241), the total water use for the area between Nickajack Dam and Guntersville Dam was estimated to be 1602.3 million gpd. By 2030, the total water use for the same area is estimated to be 1626 million gpd. This is much less than the 15 percent increase for the entire Tennessee River watershed. The Guntersville Reservoir water use increase represents about 1.3 percent of the historical minimum flow.

The Guntersville Reservoir area consumptive use is expected to increase from 16 to 28 million gpd. This increase represents less than 1 percent of the historical minimum flow. See Subsection 2.4.11.3 for historical flow details. Cumulative consumptive water use for the entire Tennessee River watershed is expected to increase from 649 to 980 million gpd. There are no safety-related facilities that could be affected by any increase in water use or drought conditions.

State regulations for Alabama currently require a declaration of beneficial use for water withdrawals exceeding 100,000 gpd (Reference 230). The TVA regulates structures, including intakes constructed at the shoreline of TVA reservoirs. New intake structures or expansion of existing intakes are examined on a case-by-case basis. If dredging or fill is required, the U.S. Army Corps of Engineers is also involved in the permitting process. The intake structure is designed in accordance with federal and state regulations.

2.4.11.5 Plant Requirements

Raw water needs, including makeup to the normal heat sink cooling towers, are supplied by the existing intake located on the northwest bank of the Tennessee River immediately upstream and adjacent to the BLN. The intake structure includes necessary intake screens, pumps, etc., to convey the river water to a system of clarifiers or other types of raw water pretreatment equipment before its use in the BLN. The intake invert elevation is 557 ft. There are no safety-related plant requirements provided by the Tennessee River.

The normal makeup flow rate for the BLN is approximately 42,000 gpm. The maximum expected makeup flow rate is approximately 60,000 gpm. Using the most conservative observed minimum flow rate of 2900 cfs, measured at the USGS South Pittsburg gauging station (Subsection 2.4.11.3), the maximum facility withdrawal would be about 4.6 percent of the minimum river flow.

The estimated 100-year drought flow rates for the Tennessee River presented in Table 2.4.11-203 are greater than the maximum expected makeup flow rate. Therefore, it is anticipated that there is sufficient non-safety related water supply available during the projected 100-year drought. Estimates are based on statistical analysis as discussed in Subsection 2.4.11.3.

The normal heat sink circulating water system for the BLN is a closed-cycle type system coupled with hyperbolic, natural draft, wet cooling towers. Circulating water system flow through the cooling towers is estimated at 500,000 gpm (Subsection 10.4.5). Maximum operating flow requirements for the BLN represents about 38.4 percent of the observed minimum flow rate measured at the USGS South Pittsburg gauging station (Subsection 2.4.11.3). Emergency cooling is discussed in Subsection 2.4.11.6.

2.4.11.6 Heat Sink Dependability Requirements

The PCS provides emergency cooling. A continuous natural circulation flow of air removes heat from the containment vessel. The steel containment vessel provides the heat transfer mechanism. A separate PCS gravity drained, water storage basin provides containment wetting. The PCS is not reliant on the source of water from the river intake. Makeup is provided by connection to the municipal water supply. Therefore, no warning of impending low flow from the river water makeup system is required. Low river water conditions would not affect the ability of emergency cooling water systems and the UHS to provide the required cooling for normal operations, anticipated operational occurrences, and emergency conditions.

The AP1000 UHS satisfies the guidance in Regulatory Guide 1.27 as follows. The atmosphere provides the UHS with the containment vessel and PCS providing the heat transfer mechanism, as described in Section 6.2. The PCS has a reserve capacity for 72 hours of heat removal without makeup. The passive containment cooling ancillary water storage tank or alternate external resources can provide water replenishment for long-term heat removal. Site-related events and natural phenomena would not affect the atmosphere functioning as the UHS. As described in Subsection 2.4.3, the BLN is capable of withstanding the PMF. Seismic design is addressed in Section 3.7.

2.4.12 GROUNDWATER

BLN COL 2.4-4

2.4.12.1 Description and On-Site Use

2.4.12.1.1 Regional Aquifers, Formations, Sources, and Sinks

The BLN is located within the Sequatchie Valley (Figure 2.4.12-201), an atypical portion of the Cumberland Plateau. This linear valley is incised into the plateau and is characterized by valley-and-ridge-style folding and faulting. The BLN sits on the gently dipping eastern limb of the Sequatchie anticline. West of the BLN, the anticline rolls over forming a steeply dipping western limb, which terminates at the Sequatchie Valley fault (Reference 242).

At the BLN site, the Sequatchie Valley is approximately 6.0 mi. wide, with the Tennessee River flowing southwest forming the upper reaches of Guntersville Reservoir. The BLN site is located on the former floodplain and gently rolling terrain of the river valley.

The principal water-bearing units of regional significance are limestone of Mississippian age and dolomite of Cambrian and Ordovician age. The general flow pattern in the folded rocks is typified by that of the Sequatchie Valley, as shown in Figure 2.4.12-202 (Reference 242). The cap of Pennsylvanian clastic rocks and coal beds has been completely eroded from the axis of the valley, exposing a wide band of Mississippian limestone that is displaced by the Sequatchie Valley fault.

According to the U.S. Environmental Protection Agency (EPA) Sole Source Aquifer Protection Program (Reference 239), there are three designated sole source aquifers in EPA Region IV (Southeastern United States, including Alabama): Biscayne Aquifer, located in Broward, Dade, Monroe and Palm Beach counties, Florida: Volusia-Floridian Aquifer, located in Flagler and Putnam counties, Florida; and Southern Hills Regional Aquifer System, located in southwestern Mississippi and eastern Louisiana. The BLN and surrounding region do not lie within a sole source aquifer protection zone (Reference 239).

2.4.12.1.2 Local Aquifers, Formations, Sources, and Sinks

Regional and local geology within the BLN region is described in detail in Section 2.5.1. In and near the plant area, the principal water-bearing formations are the Knox Dolomite of Cambrian and Ordovician age, and the Fort Payne Chert of Mississippian age. The southeastern edge of an outcrop belt of the Knox Dolomite is located approximately 3200 ft. northwest of the BLN site. The Knox dips to the southeast and is at a depth of approximately 1000 ft. below ground surface at the location of Units 3 and 4. The northwestern edge of an outcrop belt of Fort Payne Chert is located about 3000 ft. southeast of the plant site along River Ridge. The Fort Payne Chert dips away from the plant site with the majority of the outcrop area below the level of Guntersville Reservoir (Reference 221).

The site bedrock is overlain by a relatively thin (five to 40-ft.) cover of residual silts and clays. No alluvium sediments were encountered during the 2006 geotechnical drilling program, although they may exist in other portions of the site.

The BLN site is underlain by the Stones River Group Limestone, which in the area of the BLN is a poor water-bearing formation (Reference 242). Water generally occurs in bedrock in openings along fractures and bedding planes (some of which are solutionally enlarged), and in pore spaces in the overburden.

Differential weathering has produced a zone of material above bedrock that consists of chert gravel, boulders, and weathered shales in a silty clay residuum matrix. This irregular weathering front, also known as the epikarst, may also leave a pinnacled bedrock surface, especially where purer limestone units are encountered. This epikarst zone possesses void spaces where residual material has been “piped” (soil transported by water flow from the surface into the void spaces) through the deeper bedrock drainage network. No sharp interface exists between this residuum and sound rock at the BLN site.

Recharge to the groundwater system at the BLN is from local precipitation, which averages about 50 in. per year. Approximately 8 in. of this precipitation is estimated to enter groundwater storage. There is no apparent regional subsurface transport of water into the site. The majority of groundwater discharge from the BLN site is to the Town Creek embayment. Some discharge from the southeastern portions of the BLN site to the Tennessee River through natural divides in River Ridge at the intake structure channel and the barge dock is apparent.

The groundwater beneath the site can be characterized as flowing through the soil overburden and the weathered limestone or epikarst, between the soil and bedrock, and likely within the deeper limestone bedrock regions. Thin shale beds encountered within the bedrock aquifer serve as lithologic controls to the movement of groundwater in this regime.

During normal, dry weather conditions the epikarst aquifer is partially filled. Little water movement through the soil overburden occurs except in areas of deeper bedrock weathering. Groundwater slowly drains horizontally through the epikarst aquifer fissures and joints, generally toward Town Creek. The groundwater in the soil and epikarst on the northeastern portion of the site travels down slope to the intake structure channel and into Guntersville Reservoir.

Following significant rain events and during periods of frequent rainfall, rainwater percolation into the soil layer creates a flow pathway through this zone that flows generally toward Town Creek. Because of the clay soils, most of the rainfall is discharged to the Town Creek embayment and the Guntersville Reservoir as surface runoff, either by sheet flow or by drainage channels. Rainwater percolation downward through the site soils begins to fill the epikarst aquifer. However, the rate of water recharge into the epikarst aquifer is much greater than the drainage rate provided by the epikarst and bedrock fractures, joints, and solution channels, resulting in perched or semi-perched conditions.

Once the epikarst aquifer is filled, groundwater enters and flows across the epikarst surface in the lower soil zone. Groundwater flow in the soil follows the surface of the epikarst and topography.

Actual groundwater flow is subject to three-dimensional control structures (horizontal, vertical, and inclined fractures, joints, and bedding planes) and is not uniform across the site. Groundwater availability was inconsistent in all three zones and dry wells (those with little to no groundwater accumulation or with slow recharge characteristics) were encountered where wells were completed in the soil, epikarst, and deeper bedrock zones. Dry wells in the soil zone are most likely attributed to areas that do not have an overflow from an epikarst feature or are on high bedrock features within the soil. Dry wells in the epikarst and deeper bedrock zones are attributed to completion of the well in a zone without a significant fracture system, or one that is isolated (not connected) from the epikarst or deeper bedrock flow pathways.

2.4.12.1.3 On-site Use

The plant potable water supply is furnished by the City of Scottsboro, Alabama, which uses a surface water intake from the Guntersville Reservoir at North Sauty Creek. Groundwater is not used at the BLN and is not used to support any safety-related functions.

2.4.12.2 Sources

2.4.12.2.1 Present Groundwater Use

No groundwater supply wells exist on the BLN site, and none are expected to be installed during the operational lifetime of the facility. Groundwater is not used as a municipal or industrial groundwater source within a 2.0-mi. radius of the BLN. Several private groundwater sources were identified northwest of the BLN within a 2.0-mi. radius of the site, and they are listed in Table 2.4.12-201; their locations are shown in Figure 2.4.12-203. Private water wells listed in Table 2.4.12-201 were identified during construction of Bellefonte Units 1 and 2, and may have changed since these wells were surveyed. The State of Alabama does not require registration of private water wells; therefore, no records of existing or new private water wells were available. It is estimated that in 2007 approximately 367 people live within a 2.0-mi. radius from the center point of the BLN.

2.4.12.2.2 Projected Future Groundwater Use

The BLN site is hydraulically buffered by Guntersville Reservoir and the Town Creek embayment, except to the southwest, along a strike of the Stones River Group Limestone. Because of the poor water-bearing potential of the Stones River Group within the BLN area, any regional development of large groundwater supplies is expected to be in areas underlain by Mississippian limestone or the Knox Dolomite. The possibility of a cone of influence developing large enough to extend to the BLN site is very remote because of the low transmissivity of the Stones River Group Limestone and the probability that such development pumpage would be balanced by induced infiltration from the Town Creek embayment prior to the cone extending to the BLN site.

No cones of influence exist at the BLN site. The nearest municipally owned groundwater supply well (Hollywood Number 2 well) is owned by the Town of Hollywood, Alabama, and is located 2.5 mi. northwest of the BLN site. The Town of Hollywood purchases potable water from the Scottsboro, Alabama, municipal supply with no present or future plans to upgrade or use the Hollywood Number 2 well for the municipal water supply. The Town Creek embayment lies between the town of Hollywood and the plant site, forming a recharge boundary. The recharge boundary acts as a hydraulic buffer and limits the potential for a cone of influence that could be created by a substantial increase in groundwater withdrawals from the Hollywood Number 2 well to extend to the plant site.

2.4.12.2.3 Water Levels and Groundwater Movement

Multiple groundwater monitoring wells have been installed at the BLN site between 1973 and 2005, as shown in Figure 2.4.12-203.

Water level measurements in 12 core holes at the BLN site and in 21 private wells nearby, made in January 1961 (Figure 2.4.12-204), showed that the water table conforms closely to surface topography and the hydraulic gradient slopes with the land surface towards the Town Creek embayment.

Bedrock monitoring wells WT1 – WT6 were installed in 1973 for hydrology investigations related to construction of Bellefonte Units 1 and 2. Two additional bedrock monitoring wells were installed in 1978 to monitor groundwater near the trisodium phosphate ponds (metal cleaning waste ponds). These wells were initially labeled as B1 and B2; however, they are now referred to as wells B7 and B8, respectively. Overburden monitoring wells W9, W10, and W11 were installed in 1984 to monitor groundwater in the vicinity of the trisodium phosphate land application areas (areas containing material removed from the trisodium phosphate ponds). Monitoring wells W12 – W19 were installed in May 1990 to provide additional background groundwater quality and water level data. The wells are both bedrock wells (W12, W13, W16, and W19) and overburden wells (W14, W15, W17, and W18). Four bedrock monitoring wells (P-1 through P-4) were installed in conjunction with a geotechnical, geologic, and seismological evaluation conducted in the area southwest of the BLN cooling towers in 2005.

Selected historic groundwater piezometric contour maps are presented in Figures 2.4.12-205, 2.4.12-206, 2.4.12-207, 2.4.12-208, 2.4.12-209, and 2.4.12-210. Long-term groundwater level fluctuations were observed monthly in bedrock monitoring wells WT1 – WT6 between January 1973 and February 1993 (Reference 205). Historic groundwater level elevations in monitoring wells WT1 – WT6 are presented in Table 2.4.12-202 and graphically illustrated in Figure 2.4.12-211.

Previous investigations show that groundwater levels normally reach maximum elevations between the months of January through March, and are at minimum elevations between the months of September through October. Depth to the water table is generally less than 20 ft. throughout the BLN with general groundwater flow direction towards the Town Creek embayment.

A total of 45 clustered groundwater monitoring wells at 17 locations, and 12 aquifer test observation wells, were installed between May 11 and June 9, 2006. Groundwater monitoring well locations are presented in Figure 2.4.12-212. Groundwater monitoring well construction information is presented in Table 2.4.12-203.

Following installation, development, and surveying of the groundwater monitoring wells, the wells were gauged monthly from June 11, 2006, to May 8, 2007, to determine depth to groundwater. Monthly surface water measurements at six surface water gauging stations were obtained at the same time as the monthly groundwater well gauging activities for comparison to groundwater levels. Monthly groundwater and surface water elevations are presented in Table 2.4.12-204.

Surface water gauging station locations are presented in Figure 2.4.12-213. Quarterly groundwater potentiometric surface maps from July 2006 to May 2007 are presented in Figures 2.4.12-214, 2.4.12-215, 2.4.12-216, and 2.4.12-217.

Groundwater monitoring wells exhibiting slow recharge characteristics or no water availability (dry) were not used to construct the groundwater potentiometric surface maps. Table 2.4.12-205 lists the recharge response of each groundwater well installed during 2006 and indicates which wells were used to construct the groundwater potentiometric surface maps. Because of inconsistent water availability, groundwater potentiometric surface maps were not constructed for those wells completed in the soil zone.

In general, groundwater levels reached their maximum heights during the winter months (January – March 2007) and appear to show good correlation with previous groundwater investigations at the site. Groundwater levels versus time is illustrated in Figure 2.4.12-218. Groundwater levels were observed to change up to 7.16 ft. from the typical summer low levels to typical winter high levels.

June 2006 groundwater levels were determined unusable as groundwater gauging data showed evidence of non-equilibrium conditions in the majority of the groundwater monitoring wells. This circumstance was apparently because of insufficient time being provided for groundwater equilibration following well installation and development and concurrent geotechnical drilling operations.

The highest recorded groundwater level for the BLN site is 640.8 ft. above mean seal level (msl) in MW-1206b on September 26, 2006. MW-1206 is located along the base of River Ridge in the vicinity of the Materials Receiving Building and associated parking lot.

The highest recorded groundwater level in the area of the BLN is 614.5 ft. msl in MW-1204a on March 5, 2007. This is a soil overburden well with limited water availability.

The highest recorded epikarst or bedrock groundwater level in the area of the BLN is 613.2 ft. msl in MW-1205b on January 11, 2007.

During dry periods (July and August, 2006) a groundwater depression was observed adjacent to Town Creek to the northwest of Unit 3. This appears to represent a depletion of the epikarst aquifer and slow drainage into the lower bedrock zone. As precipitation events occur with greater frequency in September and the following fall and winter months, the epikarst aquifer refills and groundwater reestablishes its normal drainage pattern to Town Creek.

2.4.12.2.4 Hydrogeologic Properties of Subsurface Materials

Subsurface conditions related to the underlying geology and geohydrology have been described in multiple reports and investigations. The hydrogeologic properties of the materials beneath the site were determined from previous subsurface investigations and in-situ testing during the 2006 investigations.

Subsurface investigations performed in preparation of the FSAR for Bellefonte Units 1 and 2 (Reference 221) describe the subsurface conditions at the location of Bellefonte Units 1 and 2, located south of the locations for the proposed Units 3 and 4. Based on the geotechnical and geologic information obtained thus far through present site investigation activities, subsurface lithology is very similar between the two locations.

2.4.12.2.4.1 Soil Zone

The average soil dry bulk density is 90 pcf based on previous environmental testing performed at the BLN. Within the soil zone, groundwater movement and availability is inconsistent. Hydraulic conductivities, obtained from slug testing in 1988, ranges from $1x10^{-6}$ to $1x10^{-8}$ cm/s with a total soil porosity of 0.45. Due to the low hydraulic conductivity, the majority of groundwater flow in the soil zone appears to be vertical into the epikarst portion of the bedrock zone. Deeper, soil-filled portions of the weathered bedrock (epikarst) exist with groundwater movement apparent within the channels and fractures.

2.4.12.2.4.2 Bedrock Zone

Water occurs in the Stones River Group Limestone in openings along fractures and bedding planes in bedrock (some of which are solutionally enlarged), and in pore spaces in the overburden. Porosity of the Stones River Group Formation limestone above and below a 20-ft. depth is 0.04 and 0.01, respectively. Total porosity is estimated at 0.05 based on total number of voids encountered during the 2006 pre-COL application site investigation; however, this is probably an underestimation of total porosity as many small cavities and fractures were not captured in the estimate.

The bedrock zone can be further subdivided into the upper, “weathered” epikarst zone and the deeper, unweathered bedrock zone. The division between these two zones is indistinct with highly variable depth across the site. The epikarst tends to have greater connectivity of weathered fractures and conduits than is present in the lower bedrock and tends to be most prevalent in the upper 20-ft. of the bedrock. Below this depth, the occurrence and size of the fractures and solutional openings are smaller with a higher percentage of fresh, unweathered limestone. Occurrence and variations of the epikarst development at the BLN is discussed in detail in Subsection 2.5.4.1.

In support of the FSAR for Bellefonte Units 1 and 2, borehole packer tests were conducted in selected exploration holes for the purpose of measuring the hydraulic conductivity of the limestone-shale bedrock. Hydraulic conductivities ranging from less than $4.0x10^{-6}$ cm/s to $3.8x10^{-3}$ cm/s, with 92 percent of the values less than $3.5x10^{-4}$ cm/s.

Borehole packer tests were conducted in 2006 in selected geotechnical exploration borings for the purpose of measuring the hydraulic conductivity of the limestone-shale bedrock. Hydraulic conductivities ranging from $2.5x10^{-5}$ cm/s to $4.2x10^{-3}$ cm/s, with 86 percent of the values less than $6.3x10^{-4}$ cm/s.

Horizontal hydraulic conductivities, measured from monitoring well pump tests, ranged from a high value of 3.95×10^{-3} cm/s for observation well OW-12 to 6.11×10^{-7} cm/s for monitoring well MW-1203b.

Horizontal groundwater flow velocities were determined using a conservative straight-line-flow bounding method from the groundwater well nearest the liquid radioactive waste tank in the unit closest to the discharge point using the highest measured hydraulic conductivity on-site. A straight line flow path is considered the most conservative because the actual groundwater pathways are expected to be tortuous, resulting in longer transport times, and hydraulic conductivities of the fractures/joints lower than the highest measured on-site. Because of the lower hydraulic conductivities in the soil and deeper bedrock, the majority of groundwater flow is conservatively assumed to be within the epikarst zone. Groundwater conditions are further discussed in Subsection 2.5.4.6. Groundwater characteristics associated with the karst development at the BLN is discussed in detail in Subsection 2.5.4.1.

2.4.12.2.5 Potential Reversibility of Groundwater Flow

During times of low groundwater levels in July and August 2006, some reversal of groundwater flow was apparent from Town Creek inland towards the previously discussed groundwater depression, and some surface water recharge into the bedrock aquifer may have resulted. However, the affect appears to be of a short duration and localized to the areas along Town Creek. No influence on the groundwater beneath the BLN was observed.

2.4.12.3 Subsurface Pathways

BLN COL 2.4-5 Although the discussions of groundwater movement in Subsection 2.4.12.2 is a reasonable scenario for groundwater flow, it is assumed that actual groundwater flow is subject to three-dimensional control structures (horizontal, vertical, and inclined fractures, joints, and bedding planes) and will not be uniform across the site.

Two postulated groundwater pathway scenarios, Unit 3 to the Town Creek embayment and Unit 4 to the intake structure channel, represent the most conservative pathways from a two reactor site where groundwater flow is possibly in different directions from each unit. Both flow paths utilize a conservative, straight-line flow path approach using the shortest distance and highest measured hydraulic conductivity. A straight line flow path would be considered the most conservative as the actual groundwater pathways are expected to be tortuous, resulting in longer transport times, and hydraulic conductivities (K_h) of the fractures/joints would be (or are) expected to be lower than the highest measured on-site.

Monthly groundwater gradients were calculated to be 1.8×10^{-3} - 5.0×10^{-3} from Unit 3 to the Town Creek embayment, and 5.0×10^{-3} - 6.7×10^{-3} between Unit 4 and the intake structure channel. Monthly groundwater flow velocities were calculated to be 0.5 – 1.4 ft/day from Unit 3 to the Town Creek embayment, and 1.4 – 1.9 ft/day between Unit 4 and the intake structure channel. A summary of the monthly groundwater hydraulic gradients and flow velocities is presented in Table 2.4.12-206. Groundwater flow from Unit 4 towards the intake structure channel only occurs for short periods of time during wet months and normally flows towards Town Creek during the majority of the year.

Evaluation of the accident effects of a contaminant release to groundwater from the BLN is discussed in detail in Subsection 2.4.13.

2.4.12.4 Monitoring and Safeguard Requirements

BLN COL 2.4-4 The Radiation Protection Program is described in Section 12.5 and Appendix 12AA.5.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

The installation and operation of a permanent dewatering system is not required at the BLN. No wells are installed for safety-related purposes.

The maximum static groundwater level observed in the vicinity of the BLN power blocks during the 2006 investigation was 614.6 ft. msl, significantly lower than the AP1000 design maximum groundwater elevation of 626.6 ft. msl (or two feet below grade as identified in DCD Table 2-1) based on a site finished floor level elevation of 628.6 ft. msl.

Evaluation of subsurface hydrostatic loading is discussed in Section 2.5.

2.4.13 ACCIDENTAL RELEASE OF RADIOACTIVE LIQUID EFFLUENTS IN GROUND AND SURFACE WATERS

BLN COL 2.4-5 An evaluation of the effects of an accidental release of radiological contaminants to groundwater from a postulated failure of one of the Unit 3 effluent holdup tanks, located in the Unit 3 auxiliary building, was analyzed to estimate the concentration of radioactive contaminants entering Town Creek embayment.

Radionuclide concentrations in Town Creek were modeled using RESRAD-Offsite Version 2, developed by Argonne National Laboratories. The contents of the effluent holdup tank were conservatively assumed to enter the groundwater instantaneously through surface infiltration, and the nuclides were assumed to travel with the water directly in a straight line to the nearest point on the shore of

Town Creek embayment. A straight line flow path is considered the most conservative as the actual groundwater pathways are expected to be more tortuous, transport times much longer, and hydraulic conductivities of the fractures/joints lower. Because of the higher hydraulic conductivities in the soil and deeper bedrock, the majority of groundwater flow is conservatively assumed to be within the epikarst zone. Site specific radiological distribution coefficients (Kd) were measured in three soil borings on the BLN during the 2006 pre-COL application investigation. Results of the isotopic K_d analysis are presented in Table 2.4.13-201. Site specific groundwater flow velocities and travel times are presented in Table 2.4.12-206. Hydraulic conductivities, porosity, and bulk density of the subsurface soils and bedrock are described in Subsection 2.4.12.2.4. Groundwater pathways are discussed in Subsection 2.4.12.3.

BLN COL 15.7-1 This event is defined as an unexpected and uncontrolled release of radioactive water produced by plant operation from a tank rupture. The AP1000 tanks which normally contain radioactive liquid are listed in Table 2.4.13-201.

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

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

The Liquid Radwaste System (WLS) monitor tanks located in the radwaste building extension are considered because of their location in a non-seismic building. These tanks have a maximum capacity of 15,000 gallons each, and contain processed fuild ready for discharge. The radwaste building has a well sealed, contiguous basement with integral curbing that can hold the maximum liquid inventory of any tank. Floor drains in the area lead to the liquid radwaste system. The foundation for the entire building is a reinforced concrete mat on grade. Failure of any one of these tanks would be contained within the building, and would involve low activity processed liquids being held for pending discharge. Any release to the environment would be leakage through cracks in the concrete. The radiological consequences of such leakage are bounded by the effluent holdup tanks. Therefore, these tanks are excluded as a limiting fault.

The remaining four tank applications were considered - the effluent holdup tanks, waste holdup tanks, monitor tanks (located in the auxiliary building), and chemical waste tanks. Of these tanks, the effluent holdup tanks have both the highest potential radioactive isotope inventory and the largest volume. The other tanks need not be considered further because they have lower isotopic inventory and because the rooms in which they are located are not on the lowest level of the auxiliary building (and thus intervening interior floors would mitigate the uncontrolled release of a ruptured tank). Therefore, the AP1000 effluent holdup tank is limiting for the purpose of calculating the effects of the failure of a radioactive liquid-containing tank. This failure is classified as a limiting fault.

The effluent holdup tanks are located in an unlined room on the lowest level of the auxiliary building. This level is 33 feet 6 inches below the existing surface grade elevation of the plant. Each unit has two effluent holdup tanks, one of which is postulated to fail.

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

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

The radioactive source term is:

- Tritium source term concentration is 1.0 microcuries per gram taken from DCD Table 11.1-8
- Corrosion product source terms Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58, and Co-60 taken from DCD Table 11.1-2
- Other isotope source terms taken from DCD Table 11.1-2 are multiplied by 0.12/0.25 to adjust the radionuclide concentrations to the 0.12 percent failed fuel fraction outlined in Branch Technical Position 11-6, March 2007.

Analysis of failure of the effluent holdup tank of Unit 3 rather than Unit 4 is conservative. As discussed in Subsection 2.4.12.3, groundware transport is in a west-northwest direction to the nearest point of the nearest surface water body (Town Creek). The distance from Unit 3 to the Town Creek embayment is 1,188 feet. The location of the auxiliary building for Unit 4 and the corresponding groundwater transport of radionuclides for a tank failure in the auxiliary building of Unit 4 requires a longer transport distance of 3,106 feet generally northest through similiar soils to the intake structure channel and Guntersville Reservoir (Subsection 2.4.12.3). As discussed in Subsection 2.4.12.3, groundwater normally flows from Unit 4 toward Town Creek, but during wet months of the year the flow is toward the intake structure. Although each scenario has a similar transport time, the shorter transport distance because of a failure in the auxiliary building of Unit 3 decreases the time for isotope retardation during transport and thus provides more conservative values. The groundwater flow is assumed to be a straight transport line from the Unit 3 auxiliary building to the nearest point of Town Creek, minimizing the transport distance and time.

As discussed in Subsection 2.4.12.2.1, no local wells are present. No wells are assumed to be installed because groundwater use is not planned during site operations or construction.

The conceptual model of radionuclide transport through groundwater is shown in Figure 2.4.13-201. With the failure of the effluent holdup tank and subsequent liquid release to the environment, radionuclides enter the subgrade soild at an elevation of 33 feet 6 inches below the surrounding grade. The effluent liquids is assumed to completely fill the soil pore space in an area large enough to contain 22,400 gallons. Radionuclides are then released in the groundwater and transported through the epikarst zone to the nearest point of Town Creek. The clayey overburden soils continually receive the average annual onsite precipitation. The precipitation that does not runoff or is lost to evapotranspiration infiltrates through the unsaturated zone to the epikarst and contributes to groundwater transport to Town Creek.

This conceptual model is conservative. It provides for the shortest travel distance to Town Creek, includes the limiting fault tank, does not take credit for dilution in Town Creek, and uses conservative estimates for parameters that are not developed from site data. A straight line flow path is considered the most conservative as the actual groundwater pathways are expected to be more tortuous, transport times much longer, and hydraulic conductivities of the fractures/joint lower. Due to the lower hydraulic conductivities in the soil and deeper bedrock, the majority of groundwater flow is conservatively assumed to be within the epikarst zone.

Radionuclide decay during transport by groundwater occurs and is considered in the analysis. Radionuclide transport by groundwater is assumed to be affected by absorption by the surrounding soils. As discussed in Subsection 2.4.12.1.2, the soils surrounding the auxiliary building at the elevation fo the liquid release are epikarst bedrock, and moderate to highly fractured and corroded limestone. Site specific radiological distribution coefficients (K_d) were measured in three soils borings on the BLN during the 2006 pre-COL application investigation. Results of the isotopic K_d analysis are presented in Table 2.4.13-202. Site-specific groundwater flow velocities and travel times are presented in Table 2.4.12-206. Hydraulic conductivities, porosity, and bulk density of the subsurface soils and bedrock are described in Subsection 2.4.12.2.4. Groundwater pathways are discussed in Subsection 2.4.12.3.

The highest measured bedrock hydraulic conductivity measured at the site (Subsection 2.4.12) is used. Site-specific parameters such as unsaturated zone density, unsaturated zone porosity, saturated zone porosity, hydraulic conductivity, dispersion coefficients, flow velocities, and travel times used in this model are provided in Table 2.4.13-203.

Radionuclide concentrations in Town Creek were modeled using RESRAD-Offsite (Reference 209). The groundwater pathway mechanism is a first-order release model that considers the effects of different transport rates for radionuclides and progeny nuclides, while allowing decay during the transport process. The concentration of each radionuclide transmitted to the environment is determined by the transport through the groundwater system, dilution by groundwater and infiltrating surface water from the overburden soils, absorption, and decay.

No credit is taken for dilution of radionuclides in Town Creek by water flow. Radionuclides are assumed to remain in Town Creek near the groundwater discharge point for a period of one year. Individual radionuclide concentrations in Town Creek were modeled using RESRAD-Offsite and concentrations were calculated on a periodic interval of approximately 70 days for an evaluation period of 50 years.

The radiological consequences of a postulated failure of the effluent holdup tank as the limiting fault do not exceed 10 CFR Part 20, Appendix B, Table 2, Column 2, at the nearest surface water body (Town Creek) that contributes to a potable surface water supply (Guntersville Reservoir) located in an unrestricted area. The Guntersville Reservoir is located approximately 2.3 miles downstream; the nearest withdrawal point is an additional 6.5 miles from the point where Town Creek enters the reservoir.

The maximum radionuclide concentration for each isotope calculated to be in Town Creek during a 50-year evaluation period was compared to 10 CFR Part 20, Appendix B, Table 2, Column 2. The maximum concentration for each radionuclide is less than 10 CFR Part 20, Appendix B, Table 2, Column 2 limits. Table 2.4.13-204 provides the concentration of the source term radionuclides calculated to be in Town Creek.

The maximum radionuclide concentration for each isotope calculated to be in Town Creek during the 50-year period was used to calculate a fraction of effluent concentration. The fraction of effluent concentration using all maximum isotope concentrations is well below a value of 1.0. Table 2.4.13-204 provides the fraction of effluent concentration for each radionuclide. The evaluation is conservative because the maximum concentration of each radionuclide occurs at a different time due to variations in transport time to Town Creek.

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

Locations of surface water users are listed in Table 2.4.1-202. An evaluation of effluent releases to surface waters is discussed in detail in Subsection 11.2.3.

BLN COL 2.4-5 The Fort Payne Water System municipal supply intake, described in Table 2.4.1-202, was used as the nearest potentially affected water intake in the effluent analysis release scenario.

2.4.14 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATION REQUIREMENTS

BLN COL 2.4-6 The grade elevation of the BLN are above the probable maximum 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.

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2.4.15 COMBINED LICENSE INFORMATION

2.4.15.1 Hydrological Description

BLN COL 2.4-1 This COL item is addressed in Subsection 2.4.1.

2.4.15.2 Floods

BLN COL 2.4-2 This COL item 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

BLN COL 2.4-3 This COL item is addressed in Subsection 2.4.11.5.

2.4.15.4 Groundwater

BLN COL 2.4-4 This COL item is addressed in Subsections 2.4.12, 2.4.12.3, and 2.4.12.5.

2.4.15.5 Accidental Release of Liquid Effluents into Ground and Surface Water

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BLN COL 2.4-5 This COL item is addressed in Subsections 2.4.12.3 and 2.4.13.

2.4.15.6 Emergency Operation Requirement

BLN COL 2.4-6 This COL item is addressed in Subsection 2.4.14.

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TABLE 2.4.1-201 (Sheet 1 of 2)
STRUCTURE ELEVATIONS

Structure	Elevation Opening	Elevation Grade
Nuclear Island (Safety-related)		628.6
Railcar Bay/Filter Storage Area door	628.6	
Annex Building		628.6
Temporary Electric Power Supply Room door	628.6	
Door to SO3 Stairs	628.6	
Door to SO4 Stairs	628.6	
Men’s Change Room door	628.6	
Corridor 40321 door	628.6	
Corridor 40311 door	628.6	
Access Area 40300 doors	628.6	
Containment Access Corridor Hatch and Door	635.8	
Diesel Generator Building		628.6
Diesel Generator Room A doors	628.6	
Diesel Generator Room B doors	628.6	
Combustion Air Cleaner Area A plenum	628.6	
Combustion Air Cleaner Area B plenum	628.6	
Radwaste Building		628.6
Mobile Systems Facility doors	628.6	
HVAC Equipment Room door	628.6	
Electrical/Mechanical Equipment Room door	628.6	
Turbine Building		628.6
Mobile Systems Facility doors	628.6	
Door to SO2 Stairs	628.6	
Aux Boiler Room door	628.6	
Motor Driven Fire Pump Room door	628.6	
Door to SO1 Stairs	628.6	
Turbine Building Grade Deck Room 20300	628.6	
Units 1 & 2 Buildings		629.0

TABLE 2.4.1-201 (Sheet 2 of 2)
BLNCOL 2.4-1
STRUCTURE ELEVATIONS

	Elevation	
Structure	Opening	Grade
Switchyard Building		616.0
Tech Support Center		628.6

a) Table 2.4.1-201 does not include penetrations between buildings and rooftop penetrations.

TABLE 2.4.1-202 (Sheet 1 of 3)

BLN COL 2.4-1

LOCAL SURFACE WATER USERS - GUNTERSVILLE WATERSHED AREA

Facility Name	Use Type	County, State	Distance (mi.)	Location (TRM and bank)	Maximum Use Rate (Mgd)	Monthly Consumption Rate (Mg/mo)	Source
Jasper Water Department	Public Supply	Marion, TN	[	]	1.048	31.44	Sequatchie River
South Pittsburg Water System	Public Supply	Marion, TN	[	]	1.1	33	Tennessee River
Bridgeport Utilities Board	Public Supply	Jackson, AL	[	]	1.86	55.8	Tennessee River
TVA Widows Creek Fossil Plant	Thermoelectric	Jackson, AL	[	]	1,079	32370	Tennessee River
Shaw Industries	Industrial	Jackson, AL	[	]	0.18	5.4	Bingers Creek
Smurfit-Stone Container Corporation	Industrial	Jackson, AL	[	]	9	270	Tennessee River
TVA Bellefonte Nuclear Plant[a]	Industrial	Jackson, AL	[	]	----	----	Tennessee River
Fort Payne Water System	Public Supply	Jackson, AL	[	]	5.0	150	Tennessee River
Fort Payne Water System	Public Supply	DeKalb	[	]	1.0	30	Big Willis Creek
Fort Payne Water System	Public Supply	DeKalb	[	]	5.41	162.3	Allen Branch
Scottsboro Water System	Public Supply	Jackson, AL	[	]	2.15	64.5	Tennessee River

TABLE 2.4.1-202 (Sheet 2 of 3)

BLN COL 2.4-1

LOCAL SURFACE WATER USERS - GUNTERSVILLE WATERSHED AREA

Facility Name	Use Type	County, State	Distance (mi.)	Location (TRM and bank)	Maximum Use Rate (Mgd)	Monthly Consumption Rate (Mg/mo)	Source
Scottsboro Water System	Public Supply	Jackson, AL	[	]	1.94	58.2	Tennessee River
Section & Dutton Water Boards	Public Supply	Jackson, AL	[	]	2.3	69	Tennessee River
Section Waterworks Board	Public Supply	DeKalb	[	]	7.2	216	Tennessee River
Christian Youth Camp[b]	Public Supply	Marshall, AL	[	]	----	----	Tennessee River
Guntersville State Park[c]	Irrigation	Marshall, AL	[	]	0.02	0.6	Guntersville Reservoir
DCNR - Guntersville State Park	Irrigation	Marshall, AL	[	]	0.02	0.6	Tennessee River
Albertville Municipal Utilities Board	Public Supply	Marshall, AL	[	]	10	300	Short Creek
Grant Waterworks Board	Public Supply	Marshall, AL	[	]	0.8	24	Guntersville Lake
Guntersville Water Works and Sewer Board	Public Supply	Marshall, AL	[	]	1.3	39	Tennessee River

TABLE 2.4.1-202 (Sheet 3 of 3)
BLN COL 2.4-1
LOCAL SURFACE WATER USERS - GUNTERSVILLE WATERSHED AREA

Facility Name	Use Type	County, State	Distance (mi.)	Location (TRM and bank)	Maximum Use Rate (Mgd)	Monthly Consumption Rate (Mg/mo)	Source
Guntersville Water Works and Sewer Board	Public Supply	Marshall, AL	**[**	**]**	"	"	Tennessee River
Arab Water Works Board	Public Supply	Marshall, AL	**[**	**]**	3.5	105	Guntersville Lake

Notes:

Mgd - Million gallons per day.

Mg/mo - Million gallons per month.

a) Current river usage is limited to fire protection needs.

b) Water usage not metered.

c) Estimated water usage.

BLN COL 2.4-1

TABLE 2.4.1-203
INVENTORY OF TENNESSEE RIVER WATERSHED WATER CONTROL STRUCTURES

Name	In Service Date	Owner	Type[a]	Tennessee River Mile	Drainage Area above dam (mi^2)	Flood Storage (1000 's of acre-feet)	Dam Height (ft.)	Dam Length (ft.)	Spillway Crest Elevation (ft. msl)	Normal Minimum Pool Elevation (ft. msl)	Normal Maximum Pool Elevation (ft. msl)	Generation Capacity (MW)
Fort Loudoun Dam	1943	TVA	CNER	602.3	9,550	111	125	4,190	815	807	813	155.6
Watts Bar Dam	1942	TVA	CNER	529.9	17,310	379	112	2,960	745	735	741	175.0
Chickamauga Dam	1940	TVA	CNER	471.0	20,790	345.3	129	5,800	685	675	683	160.0
Nickajack Dam	1967	TVA	CNER	424.7	21,870	31.5	81	3,767	635	632.5	634.5	104.0
Guntersville Dam	1939	TVA	CNER	349.0	24,450	263	94	3,979	595.44	593	595	140.4
Wheeler Dam	1936	TVA	CN	274.9	29,590	326.5	72	6,342	556.28	550.5	556	411.8
Wilson Dam	1924	TVA	CNPG	259.4	30,750	53.6	137	4,541	507.88	504.75	507.75	675.4
Pickwick Landing Dam	1938	TVA	CNER	206.7	38,820	417.7	113	7,715	418	408	414	240.2
Kentucky Dam	1944	TVA	CNER	22.4	40,200	4,008	206	8,422	375	354	359	223.1

a) Type: CNER – concrete and earth/rock filled, CN – concrete, CNPG – concrete gravity arch

(Reference 218)
mi^2 - square miles
ft. – feet
msl - mean sea level
MW – megawatts

TABLE 2.4.2-201 (Sheet 1 of 3)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT SOUTH PITTSBURG, TN (USGS STATION 03571850) 1917-1987

BLN COL 2.4-2

Water Year[(a)]	Date	Discharge (cfs)
1917	3/8/1917	320,000
1930	11/19/1929	172,000
1931	4/8/1931	125,000
1932	2/3/1932	192,000
1933	1/1/1933	241,000
1934	3/6/1934	215,000
1935	3/15/1935	175,000
1936	3/30/1936	241,000
1937	1/5/1937	209,000
1938	4/10/1938	136,000
1939	2/17/1939	189,000
1940	9/2/1940	87,400
1941	7/18/1941	57,000
1942	3/22/1942	77,300
1943	12/30/1942	231,000
1944	3/30/1944	193,000
1945	2/19/1945	124,000
1946	1/9/1946	231,000
1947	1/20/1947	191,000
1948	2/14/1948	216,000
1949	1/6/1949	194,000
1950	2/2/1950	180,000

TABLE 2.4.2-201 (Sheet 2 of 3)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT SOUTH PITTSBURG, TN (USGS STATION 03571850) 1917-1987

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1951	3/30/1951	155,000
1952	3/12/1952	141,000
1953	2/22/1953	120,000
1954	1/23/1954	199,000
1955	3/23/1955	122,000
1956	2/4/1956	168,000
1957	2/2/1957	217,000
1958	11/19/1957	195,000
1959	1/22/1959	116,000
1960	12/20/1959	114,000
1961	3/9/1961	183,000
1962	12/19/1961	188,000
1963	3/13/1963	216,000
1964	3/16/1964	137,000
1965	3/27/1965	184,000
1966	2/17/1966	124,000
1967	7/8/1967	132,000
1968	12/20/1967	168,000
1969	2/3/1969	158,000
1970	12/31/1969	217,000
1971	2/6/1971	111,000
1972	1/11/1972	129,700
1973	3/18/1973	315,000

TABLE 2.4.2-201 (Sheet 3 of 3)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT SOUTH PITTSBURG, TN (USGS STATION 03571850) 1917-1987

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1974	1/12/1974	205,000
1975	3/14/1975	189,000
1976	1/2/1976	82,200
1977	4/5/1977	206,000
1978	1/27/1978	128,000
1979	3/5/1979	181,300
1980	3/22/1980	170,000
1981	8/13/1981	59,100
1982	1/5/1982	142,400
1983	5/23/1983	150,000
1984	5/9/1984	267,000
1985	2/2/1985	107,000
1986	2/19/1986	76,200
1987	2/28/1987	153,000

a) Water Year runs from October 1 of prior year to September 30 of identified year
Gauge no longer operable, discontinued in 1987

(Reference 244)

TABLE 2.4.2-202 (Sheet 1 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[(a)]	Date	Discharge (cfs)
1867	3/11/1867	459,000
1874	5/01/1874	195,000
1875	3/01/1875	410,000
1876	12/31/1875	227,000
1877	4/11/1877	190,000
1878	2/25/1878	125,000
1879	1/15/1879	252,000
1880	3/18/1880	254,000
1881	12/03/1880	174,000
1882	1/19/1882	275,000
1883	1/23/1883	261,000
1884	3/10/1884	285,000
1885	1/18/1885	174,000
1886	4/03/1886	391,000
1887	2/28/1887	181,000
1888	3/31/1888	178,000
1889	2/18/1889	198,000
1890	3/02/1890	283,000
1891	3/11/1891	259,000
1892	1/17/1892	252,000
1893	2/20/1893	221,000
1894	2/06/1894	167,000
1895	1/12/1895	212,000
1896	4/05/1896	269,000
1897	3/14/1897	257,000
1898	9/05/1898	167,000

TABLE 2.4.2-202 (Sheet 2 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1899	3/22/1899	273,000
1900	2/15/1900	159,000
1901	5/25/1901	221,000
1902	1/02/1902	271,000
1903	4/11/1903	210,000
1904	3/25/1904	144,000
1905	2/11/1905	146,000
1906	1/26/1906	140,000
1907	11/22/1906	222,000
1908	2/17/1908	163,000
1909	6/06/1909	163,000
1910	2/19/1910	86,600
1911	4/08/1911	198,000
1912	3/31/1912	190,000
1913	3/30/1913	222,000
1914	4/03/1914	105,000
1915	12/28/1914	185,000
1916	12/20/1915	197,000
1917	3/07/1917	341,000
1918	2/02/1918	270,000
1919	1/05/1919	189,000
1920	4/05/1920	275,000
1921	2/13/1921	213,000
1922	1/23/1922	229,000
1923	2/07/1923	188,000
1924	1/05/1924	143,000

TABLE 2.4.2-202 (Sheet 3 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1925	12/11/1924	138,000
1926	4/16/1926	92,900
1927	12/29/1926	249,000
1928	7/02/1928	184,000
1929	3/26/1929	248,000
1930	11/19/1929	180,000
1931	4/08/1931	125,000
1932	2/01/1932	192,000
1933	1/01/1933	241,000
1934	3/06/1934	215,000
1935	3/15/1935	175,000
1936	3/29/1936	234,000
1937	1/04/1937	204,000
1938	4/10/1938	136,000
1939	2/17/1939	193,000
1940	9/02/1940	89,400
1941	7/18/1941	58,200
1942	3/22/1942	72,300
1943	12/30/1942	235,000
1944	3/30/1944	201,000
1945	2/18/1945	115,000
1946	1/09/1946	225,000
1947	1/20/1947	186,000
1948	2/14/1948	225,000
1949	1/06/1949	179,000
1950	2/02/1950	192,000

TABLE 2.4.2-202 (Sheet 4 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[(a)]	Date	Discharge (cfs)
1951	3/30/1951	140,000
1952	(b)	(b)
1953	2/22/1953	107,000
1954	1/22/1954	185,000
1955	3/23/1955	118,000
1956	2/04/1956	187,000
1957	2/02/1957	208,000
1958	11/19/1957	189,000
1959	1/23/1959	110,000
1960	12/20/1959	108,000
1961	3/09/1961	178,000
1962	12/18/1961	190,000
1963	3/13/1963	219,000
1964	3/16/1964	122,000
1965	3/26/1965	180,000
1966	2/16/1966	104,000
1967	7/08/1967	120,000
1968	12/23/1967	148,000
1969	2/03/1969	121,000
1970	12/31/1969	186,000
1971	2/07/1971	90,700
1972	1/11/1972	116,000
1973	3/18/1973	267,000
1974	1/11/1974	181,000
1975	3/14/1975	148,000
1976	1/28/1976	67,200

TABLE 2.4.2-202 (Sheet 5 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1977	4/05/1977	191,000
1978	1/28/1978	115,000
1979	3/05/1979	145,000
1980	3/21/1980	168,000
1981	2/12/1981	50,800
1982	1/04/1982	133,000
1983	5/21/1983	116,000
1984	5/9/1984	239,000
1985	2/02/1985	81,000
1986	2/18/1986	66,200
1987	2/27/1987	109,000
1988	1/21/1988	74,100
1989	6/21/1989	173,000
1990	2/19/1990	169,000
1991	12/23/1990	185,000
1992	12/04/1991	146,000
1993	3/24/1993	113,000
1994	3/28/1994	202,000
1995	2/18/1995	99,900
1996	1/28/1996	145,000
1997	3/04/1997	138,000
1998	4/19/1998	207,000
1999	1/24/1999	91,400
2000	4/05/2000	137,000
2001	2/18/2001	86,100
2002	1/24/2002	184,100

TABLE 2.4.2-202 (Sheet 6 of 6)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
CHATTANOOGA, TN (USGS STATION 03568000) 1867-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
2003	5/8/2003	241,000
2004	9/18/2004	160,000
2005	12/13/2004	153,000

a) Water Year runs from October 1 of prior year to September 30 of year identified

b) not reported

(Reference 244)

TABLE 2.4.2-203 (Sheet 1 of 2)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
GUNTERSVILLE, AL (USGS STATION 03573500) 1867-1938

BLN COL 2.4-2

Water Year[a]	Date	Gage Height[b] (ft.)	Discharge (cfs)
1867	3/13/1867	48	[c]
1905	2/12/1905	25.3	[c]
1906	1/27/1906	22.3	[c]
1907	11/24/1906	29.8	[c]
1908	2/19/1908	27.1	[c]
1909	3/14/1909	30.4	[c]
1910	5/27/1910	18.4	[c]
1911	4/10/1911	34	[c]
1912	4/02/1912	30.8	[c]
1917	3/10/1917	37.4	350,000
1924	4/21/1924	26.5	[c]
1925	1/15/1925	23.4	[c]
1926	1/23/1926	20	[c]
1927	12/31/1926	38.3	[c]
1928	7/04/1928	27	[c]
1929	3/28/1929	34.8	[c]
1930	11/18/1929	31	[c]
1931	4/09/1931	22.54	[c]
1932	2/04/1932	30.8	201,000
1933	1/03/1933	34.45	244,000

TABLE 2.4.2-203 (Sheet 2 of 2)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
GUNTERSVILLE, AL (USGS STATION 03573500) 1867-1938

BLN COL 2.4-2

Water Year[(a)]	Date	Gage Height[(b)] (ft.)	Discharge (cfs)
1934	3/08/1934	32.7	226,000
1935	3/17/1935	28.8	186,000
1936	4/02/1936	35.53	260,000
1937	1/06/1937	31.94	210,000
1938	4/11/1938	29.81	144,000

a) Water Year runs from October 1 of prior year to September 30 of year identified
Gauge no longer operable, discontinued in 1938

b) Datum = 546.31 feet above sea level NGVD29

c) not recorded

(Reference 244)

TABLE 2.4.2-204 (Sheet 1 of 4)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT WHITESBURG, AL (USGS STATION 03575500) 1925-2005

BLN COL 2.4-2

Water Year[(a)]	Date	Discharge (cfs)
1925	1/16/1925	134,000
1926	1/24/1926	114,000
1927	1/01/1927	283,000
1928	4/25/1928	170,000
1929	3/30/1929	231,000
1930	11/19/1929	210,000
1931	4/09/1931	127,000
1932	2/05/1932	208,000
1933	1/04/1933	236,000
1934	3/08/1934	224,000
1935	3/17/1935	186,000
1936	4/03/1936	282,000
1937	(b)	(b)
1938	4/11/1938	153,000
1939	2/18/1939	228,000
1940	2/20/1940	89,900
1941	4/08/1941	67,200
1942	3/22/1942	111,000
1943	12/31/1942	249,000
1944	3/30/1944	225,000
1945	2/20/1945	149,000
1946	1/09/1946	277,000

TABLE 2.4.2-204 (Sheet 2 of 4)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
WHITESBURG, AL (USGS STATION 03575500) 1925-2005

BLN COL 2.4-2

Water Year[(a)]	Date	Discharge (cfs)
1947	1/22/1947	243,000
1948	2/15/1948	269,000
1949	1/06/1949	272,000
1950	3/15/1950	213,000
1951	3/30/1951	249,000
1952	3/13/1952	180,000
1953	2/24/1953	157,000
1954	1/23/1954	258,000
1955	3/24/1955	173,000
1956	2/06/1956	230,000
1957	2/02/1957	293,000
1958	11/20/1957	268,000
1959	1/22/1959	130,000
1960	12/21/1959	136,000
1961	2/25/1961	234,000
1962	2/27/1962	252,000
1963	3/14/1963	285,000
1964	3/16/1964	199,000
1965	3/30/1965	244,000
1966	2/17/1966	152,000
1967	7/09/1967	128,000
1968	12/23/1967	191,000
1969	2/04/1969	213,000

TABLE 2.4.2-204 (Sheet 3 of 4)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT
WHITESBURG, AL (USGS STATION 03575500) 1925-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1970	1/02/1970	227,000
1971	2/28/1971	128,000
1972	1/12/1972	148,000
1973	3/19/1973	323,000
1974	12/28/1973	216,000
1975	3/15/1975	230,000
1976	1/04/1976	91,400
1977	4/07/1977	222,000
1978	11/30/1977	141,000
1979	3/06/1979	217,000
1980	3/22/1980	250,000
1981	4/05/1981	51,400
1982	1/05/1982	189,000
1983	4/07/1983	168,000
1984	5/10/1984	245,000
1985	2/02/1985	113,000
1986	2/20/1986	80,100
1987	3/01/1987	156,000
1988	1/22/1988	121,000
1989	6/23/1989	183,000
1990	2/17/1990	260,000
1991	12/24/1990	304,000
1992	12/04/1991	217,000

TABLE 2.4.2-204 (Sheet 4 of 4)
PEAK STREAMFLOW OF THE TENNESSEE RIVER AT WHITESBURG, AL (USGS STATION 03575500) 1925-2005

BLN COL 2.4-2

Water Year[a]	Date	Discharge (cfs)
1993	3/25/1993	176,000
1994	3/30/1994	268,000
1995	2/18/1995	164,000
1996	2/18/1996	189,000
1997	3/05/1997	187,000
1998	1/09/1998	(b)
1999	1/25/1999	(b)
2000	4/05/2000	193,000
2001	1/21/2001	121,000
2002	1/27/2002	207,000
2003	5/9/2003	292,000
2004	9/19/2004	204,000
2005	12/10/2004	253,000

a) Water Year runs from October 1 of prior year to September 30 of year identified

b) not reported

(Reference 244)

TABLE 2.4.2-205
MAJOR REGIONAL HISTORICAL FLOODS

BLN COL 2.4-2

Date	Elevation at the BLN (ft.)	Discharge (cfs)
Prior to current regulated conditions		
March 13, 1867[a]	610.8[b]	459,000[c]
March 1, 1875[c]	608.6[b]	410,000[c]
April 3, 1886[c]	607.2[b]	391,000[c]
March 10, 1917[a]	603.6[b]	350,000[a]
Under current regulated conditions		
March 18, 1973[d]	602.2[b]	315,000[d]
May 9, 1984[d]	< 602[b]	267,000[d]
May 8, 2003[c]	< 602[b]	241,000[c] - 292,000[e]

a) as reported at USGS Station 03573500 Guntersville, AL

b) estimated based on available data

c) as reported at USGS Station 03568000 Chattanooga, TN

d) as reported at USGS Station 03571850 South Pittsburg, TN

e) as reported at USGS Station 03575500 Whitesburg, AL

(Reference 244)

TABLE 2.4.2-206 (Sheet 1 of 2)
HMR 56 LOCAL INTENSE PROBABLE MAXIMUM
PRECIPITATION DEPTH DURATION

BLN COL 2.4-2

Duration		PMP
(min.)	(hr.)	(in.)
0	0	0
5	0.083	3.3
10	0.167	5.5
15	0.250	7.6
20	0.333	9.3
25	0.417	10.8
30	0.500	12.0
35	0.583	13.2
40	0.667	14.3
45	0.750	15.3
50	0.833	16.2
55	0.917	17.0
60	1	17.6
120	2	24.4
180	3	28.4
240	4	31.5
300	5	34.2
360	6	36.2
420	7	37.6
480	8	38.6
540	9	39.4

TABLE 2.4.2-206 (Sheet 2 of 2)
HMR 56 LOCAL INTENSE PROBABLE MAXIMUM PRECIPITATION DEPTH DURATION

BLN COL 2.4-2

Duration		PMP
(min.)	(hr.)	(in.)
600	10	40.0
660	11	40.5
720	12	41.0
780	13	41.4
840	14	41.8
900	15	42.2
960	16	42.5
1020	17	42.8
1080	18	43.1
1140	19	43.4
1200	20	43.7
1260	21	44.0
1320	22	44.3
1380	23	44.6
1440	24	44.9

TABLE 2.4.2-207
SITE DRAINAGE AREA DETAILS AND RESULTS OF THE EFFECTS OF LOCAL INTENSE PRECIPITATION

BLN COL 2.4-2

	Time of Concentration, Tc (min)	PMP Intensity, i (in./hr)	Area, A (ac.)	Runoff Coefficient, C	Flow, Q (cfs)	Water Surface Elevation (ft.)
A	11.41	32.0	9.44	1	302	(47 cfs)[(a)]
B	16.52	29.6	29.26	1	913 (866)[(b)]	627.51
C	16.52	29.6	8.25	1	1157 (244)[(c)]	626.77
D	6.70	37.5	8.72	1	327	623.90
E	5.00	39.6	1.01	1	47[(d)]	625.87
F	5.00	39.6	1.19	1	47	625.87

a) Drainage area A overflows into drainage area B and does not directly affect safety-related facilities. Only overflow to be added to drainage area B determined. No water surface elevation determined.

b) Value in parenthesis is the drainage area B contribution. Total flow including overflow from drainage area A is used to determine the resulting water surface elevation.

c) Value in parenthesis is the drainage area C contribution. Total flow including overflow from drainage area B is used to determine the resulting water surface elevation.

d) Flow for drainage area E conservatively assumed equal to flow for drainage area F.

BLN COL 2.4-2

TABLE 2.4.3-201 (Sheet 1 of 3)
PROBABLE MAXIMUM PRECIPITATION AND PRECIPITATION EXCESS

		Antecedent Storm		Main Storm	
Index No.[(a)]	Area[(a)]	Rain (in.)	Excess[(b)] (in.)	Rain (in.)	Excess[(c)] (in.)
1	Asheville	6.44	2.99	17.40	14.72
2	Newport, French Broad	6.44	4.04	18.50	16.51
3	Newport, Pigeon	6.44	4.04	19.30	17.31
4	Embreeville	6.44	4.04	15.10	13.11
5	Nolichucky Local	6.44	4.04	15.50	13.51
6	Douglas Local	6.44	4.86	17.10	15.88
7	Little Pigeon River	6.44	4.04	20.90	18.91
8	French Broad Local	6.44	4.19	18.60	16.81
9	South Holston	6.44	4.52	12.30	10.70
10	Watauga	6.44	4.04	13.30	11.31
11	Boone Local	6.44	4.04	14.10	12.11
12	Fort Patrick Henry	6.44	4.86	14.40	13.18
13	Gate City	6.44	4.86	12.30	11.08
14	Surgionsville Local	6.44	4.86	14.60	13.38
15	Cherokee Local below Surgoinsville	6.44	4.86	15.80	14.58
16	Holston River Local	6.44	4.52	17.10	15.50
17	Little River	6.44	4.04	21.50	19.51
18	Fort Loudoun Local	6.44	4.04	17.60	15.61
19	Needmore	6.44	2.99	21.20	18.52
20	Nantahala	6.44	2.99	21.50	18.82

BLN COL 2.4-2

TABLE 2.4.3-201 (Sheet 2 of 3)
PROBABLE MAXIMUM PRECIPITATION AND PRECIPITATION EXCESS

		Antecedent Storm		Main Storm	
Index No.[(a)]	Area[(a)]	Rain (in.)	Excess[(b)] (in.)	Rain (in.)	Excess[(c)] (in.)
21	Bryson City	6.44	2.99	19.10	16.42
22	Fontana Local	6.44	2.99	20.70	18.02
23	Little Tennessee Local – Fontana to Chilhowee Dam	6.44	2.99	24.00	21.32
24	Little Tennessee Local – Chilhowee to Tellico Dam	6.44	4.04	21.00	19.01
25	Watts Bar Local above Clinch River	6.44	4.04	15.80	13.81
26	Norris Dam	6.44	4.86	13.80	12.58
27	Coal Creek	6.44	4.52	14.60	13.19
28	Clinch Local	6.44	4.52	14.90	13.49
29	Hinds Creek	6.44	4.52	15.30	13.89
30	Bullrun Creek	6.44	4.68	15.70	14.29
31	Beaver Creek	6.44	4.52	16.10	14.69
32	Clinch Local (5 areas)	6.44	4.52	15.30	13.89
33	Local above mile 16	6.44	4.52	15.30	13.89
34	Poplar Creek	6.44	4.52	14.90	13.49
35	Emory River	6.44	4.52	13.10	11.69
36	Local Area at Mouth	6.44	4.52	14.90	13.49
37	Watts Bar Local below Clinch River	6.44	4.52	14.40	12.99
38	Chatuge	6.44	2.99	21.40	18.72
39	Nottely	6.44	2.99	19.10	16.42

BLN COL 2.4-2

TABLE 2.4.3-201 (Sheet 3 of 3)
PROBABLE MAXIMUM PRECIPITATION AND PRECIPITATION EXCESS

		Antecedent Storm		Main Storm	
Index No.[a]	Area[a]	Rain (in.)	Excess[b] (in.)	Rain (in.)	Excess[c] (in.)
40	Hiwassee Local	6.44	2.99	18.90	16.22
41	Apalachia	6.44	2.99	17.90	15.22
42	Blue Ridge	6.44	2.99	22.10	19.42
43	Ocoee No. 1 – Blue Ridge to Ocoee No. 1	6.44	4.04	18.30	16.31
44	Lower Hiwassee	6.44	4.19	15.20	13.41
45	Chickamauga Local	6.44	4.52	14.50	13.09
46	South Chickamauga Creek	6.44	4.35	12.30	10.89
47	Nickajack Local	6.44	4.52	11.70	10.29
48	Sequatchie	6.44	4.52	9.80	8.39
49	Guntersville N. Local	6.44	4.52	9.80	8.39
50	Guntersville S. Local	6.44	4.52	9.80	8.39
	Average above Guntersville Dam	6.44	4.20	15.56	13.80

a) Area Index No. corresponds to Figure 2.4.3-203 numbered areas.

b) Adopted antecedent precipitation index prior to antecedent storm, 1.0 in.

c) Computed antecedent precipitation index prior to main storm, 3.65 in.

BLN COL 2.4-2

TABLE 2.4.3-202 (Sheet 1 of 4)
UNIT HYDROGRAPH DATA

Unit Area No.	Name	Drainage Area (sq. mi.)	Duration (hr.)	Qp	C_p	T_p	W_{50}	W_{75}	T_B
1	French Broad River at Asheville	945	6	15,000	0.27	14	35	12	166
2	French Broad River, Newport to Asheville	913	6	35,000	0.53	12	12	7	108
3	Pigeon River at Newport	666	6	26,600	0.56	12	11	6	78
4	Nolichucky River at Embreeville	805	6	27,300	0.58	14	14	9	82
5	Nolichucky Local	378	6	10,600	0.40	12	16	9	87
6	Douglas Local	832	6	47,930	0.27	6	8	6	60
7	Little Pigeon River at Sevierville	353	6	15,600	0.62	12	10	6	102
8	French Broad River Local	207	6	7,500	0.51	12	11	8	60
9	South Holston	703	6	16,000	0.53	18	24	17	100
10	Watauga	468	6	17,700	0.53	12	13	7	84
11	Boone Local	669	6	22,890	0.16	6	13	8	90
12	Fort Patrick Henry	63	6	3,200	0.40	8	8	6	64
13	North Fork Holston River near Gate City	672	6	12,260	0.60	24	33	25	108

BLN COL 2.4-2

TABLE 2.4.3-202 (Sheet 2 of 4)
UNIT HYDROGRAPH DATA

Unit Area No.	Name	Drainage Area (sq. mi.)	Duration (hr.)	Qp	C_p	T_p	W_{50}	W_{75}	T_B
14	Surgoinsville Local	299	6	10,280	0.48	12	13	9	66
15	Cherokee Local Below Surgoinsville	554	6	18,750	0.48	12	14	7	66
16	Holston River Local	289	6	6,800	0.55	18	22	15	96
17	Little River at Mouth	379	4	11,730	0.68	16	14	8	96
18	Fort Loudoun Local	323	6	20,000	0.29	6	10	6	36
19	Little Tennessee River at Needmore	436	6	9,130	0.49	18	23	12	126
20	Nantahala	91	6	3,770	0.45	10	12	7	70
21	Tuckasegee River at Bryson City	655	6	26,000	0.43	10	12	7	58
22	Fontana Local	389	6	16,350	0.46	10	9	5	94
23	Little Tennessee River Local, Fontana-Chilhowee	406	6	16,900	0.58	12	9	5	84
24	Little Tennessee River Local, Chilhowee-Tellico Dam	650	6	17,000	0.61	18	21	11	72
25	Watts Bar Local above Clinch River	293	6	11,300	0.30	8	9	7	84

BLN COL 2.4-2

TABLE 2.4.3-202 (Sheet 3 of 4)
UNIT HYDROGRAPH DATA

Unit Area No.	Name	Drainage Area (sq. mi.)	Duration (hr.)	Qp	C_p	T_p	W_{50}	W_{75}	T_B
26	Norris Dam	2,912	6	43,300	0.07	6	15	8	118
27	Coal Creek	36.6	2	2,150	0.64	8	9	5	40
28	Clinch Local	22.25	2	1,350	0.10	2	8	5	34
29	Hinds Creek	66.4	2	3,620	0.68	9	7	5	54
30	Bullrun Creek	104	2	2,400	0.47	14	21	14	84
31	Beaver Creek	90.5	2	2,600	0.58	14	14	10	88
32	Clinch Locals (5 areas)	111.25	2	1,350	0.10	2	8	5	34
33	Local above mi. 16	37	2	4,490	0.95	6	4	3	46
34	Poplar Creek	136	2	2,800	0.61	20	25	13	88
35	Emory River at Mouth	865	6	34,000	0.37	9	13	8	87
36	Local area at Mouth	32	2	3,870	0.95	6	3	2	46
37	Watts Bar Local below Clinch River	427	6	16,300	0.36	9	9	7	84
38	Chatuge Dam	189	6	13,570	0.34	6	6	5	54
39	Nottely Dam	215	6	13,500	0.29	6	5	4	80
40	Hiwassee Local	564	6	13,800	0.36	12	18	12	124
41	Apalachia Local	50	6	2,900	0.54	9	6	4	90

BLN COL 2.4-2

TABLE 2.4.3-202 (Sheet 4 of 4)
UNIT HYDROGRAPH DATA

Unit Area No.	Name	Drainage Area (sq. mi.)	Duration (hr.)	Qp	C_p	T_p	W_{50}	W_{75}	T_B
42	Blue Ridge Dam	232	6	11,920	0.24	6	7	4	54
43	Ocoee No.1 to Blue Ridge	363	6	17,000	0.37	8	11	7	36
44	Lower Hiwassee	1,087	6	32,500	0.93	23	16	10	136
45	Chickamauga Local	780	6	32,000	0.38	9	14	7	36
46	South Chickamauga Creek	428	6	6,270	0.48	24	40	18	132
47	Nickajack Local	652	6	9,900	0.14	9	38	10	144
48	Sequatchie River	384	4	8,560	0.49	16	15	7	140
49	Guntersville North Local	1,041	6	22,400	0.40	15	20	11	138
50	Guntersville South Local	1,047	6	22,500	0.40	15	19	11	132

Q_p = Peak discharge, cfs
C_p = Snyder coefficient
T_p = Time from beginning of precipitation excess to peak of unit hydrograph, hrs
W_{50} = Width at 50% of peak discharge, hrs
W_{75} = Width at 75% of peak discharge, hrs
T_B = Base length of unit hydrograph, hrs

BLN COL 2.4-2

TABLE 2.4.4-201
SUMMARY OF FLOODS FROM POSTULATED SEISMIC FAILURE OF UPSTREAM DAMS

Failure Case	Elevation at Chickamauga Dam (ft.)		Elevation at Bellefonte Site (ft.)	
	Original Analysis	1998 Reassessment	Original Analysis	1998 Reassessment
1 Fontana, Hiwassee, Apalachia, and Blue Ridge Dams in the OBE with one-half PMF[a][b][c]	707.5	704.1	615.1	[d]
2. Norris, Cherokee, and Douglasin the SSE with 25-year flood[b][e]	706.9	702.0	612.7	[d]
3. Cherokee and Douglas Dams in the OBE with one-half PMF[b]	707.0	697.8	614.2	[d]
4. Fontana Dam in the OBE with one-half PMF[a][b][c]	[f]	699.8[g]	[f]	[d]
5. Norris, Douglas, Fort Loudoun, and Tellico Dams in the SSE with 25-year flood[b]	[f]	695.8	[f]	[d]
6. Norris Dam in the OBE with one-half PMF[b][h]	[f]	694.5	[f]	[d]

a) Coincident failure of Nantahala, Santeetlah, Cheoah, Calderwood, and Chilhowee Dams.

b) Resulting flood wave overtops and fails the West Saddle Dike at Watts Bar Dam.

c) Resulting flood wave overtops and fails Tellico Dam.

d) No elevations calculated. Resulting elevations would be significantly lower than determined by the original analysis.

e) Resulting flood wave overtops and fails Fort Loudoun Dam.

f) Not reported.

g) Fontana Dam failure was not considered a controlling case because simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams is more critical.

h) Resulting flood wave overtops and fails Melton Hill Dam.

BLN COL 2.4-2

TABLE 2.4.4-202
CUMULATIVE ANNUAL PROBABILITY OF EXCEEDANCE FOR SEISMICALLY-INDUCED DAM FAILURE SCENARIOS

Case 1 – Safe Shutdown Earthquake (SSE) Combined with 25-year Flood

SSE PGA Level	SSE Annual Probability of Exceedance[a]	25-year Flood Probability of Exceedance	Annual Exposure Window Probability[b]	Cumulative Annual Probability of Exceedance
0.18g	6.00E-05	4.00E-02	5.48E-03	1.32E-08

Case 2 – Operating Basis Earthquake (OBE) Combined with 500-year Flood or 1/2 Probable Maximum Flood (PMF)

OBE PGA Level	OBE Annual Probability of Exceedance[a]	500-year Flood Probability of Exceedance[c]	Annual Exposure Window Probability[c]	Cumulative Annual Probability of Exceedance
0.09g	2.20E-04	2.0E-03	5.48E-03	2.41E-09

a) The SSE of 0.18 g and OBE of 0.09 g correspond to the levels for these earthquake conditions in the original Bellefonte analysis which is consistent with the current Watts Bar and Sequoyah seismic design levels. The SSE and OBE probabilities are based on annual probability of exceedance for mean peak ground acceleration (100 Hz spectral value) shown in Bellefonte FSAR Subsection 2.5.2, Figure 2.5-274.

b) Annual Exposure Window Probability is the probability of the peak flood level, 2 days out of 365 days.

c) The return period for a 1/2 PMF is greater than 500 years; therefore, for comparison purposes the probability of the more likely 500 year flood is conservatively used here.

BLN COL 2.4-2

TABLE 2.4.4-203 (Sheet 1 of 2)
TVA DAMS RIVER MILE DISTANCES

River	Structure/River Mouth	River Mile[a]	Distance from BLN (mi.)
Tennessee River			
	Guntersville Dam	349	42.5
	BLN	**391.5**	**-**
	Nickajack Dam	424.5	33
	Chickamauga Dam	471	79.5
	Hiwassee River	499.5	108
	Watts Bar Dam	530	138.5
	Clinch River	568	176.5
	Little Tennessee River	601	209.5
	Fort Loudoun Dam	602	210.5
	Holston River	652	260.5
	French Broad River	652	260.5
Hiwassee River		0	108
	Ocoee River	34.5	142.5
	Apalachia Dam	66	174
	Hiwassee Dam	76	184
	Nottely River	92	200
	Chatuge Dam	121	229
Ocoee River		0	142.5
	Ocoee #1 Dam	12	154.5
	Ocoee #2 Dam	24	166.5
	Ocoee #3 Dam	29	171.5
	Toccoa River	38[b]	180.5
Toccoa River		0	180.5
	Blue Ridge Dam	15[b]	195.5
Nottely River		0	200
	Nottely Dam	21	221
Clinch River		0	176.5
	Melton Hill Dam	23	199.5
	Norris Dam	80	256.5
Little Tennessee River		0	209.5
	Tellico Dam	0.5	210

BLN COL 2.4-2

TABLE 2.4.4-203 (Sheet 2 of 2)
TVA DAMS RIVER MILE DISTANCES

River	Structure/River Mouth	River Mile[a]	Distance from BLN (mi.)
	Chilhowee Dam	33.5	243
	Calderwood Dam	43.5	253
	Cheoah Dam	51.5	261
	Fontana Dam	61	270.5
Holston River		0	260.5
	Cherokee Dam	52	312.5
French Broad River		0	260.5
	Douglas Dam	32	292.5

a) Approximated to the one-half river mile based on U.S. Geological Survey Quadrangles river mile designations.

b) Estimated river mile. River miles not provided for Toccoa River on U.S. Geological Survey Quadrangles.

TABLE 2.4.4-204 (Sheet 1 of 3)
FACTS ABOUT MAJOR TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Main River and Tributary Projects	River	State	Type of Dam[b]	Max Height[a] (ft.)	Length (ft.)	Drainage Area (sq. mi.)	Length of Lake (mi.)	Construction Start	Construction Completion
Kentucky	Tenn.	KY	CGE	206	8,422	40,200	184.3	7-1-38	9-14-44
Pickwick Landing	Tenn.	TN	CGE	113	7,715	32,820	52.7	3-8-35	6-23-38
Wilson[c]	Tenn.	AL	CG	137	4,535	30,750	15.5	4-14-18	9-12-25
Wheeler	Tenn.	AL	CG	72	6,342	29,590	74.1	11-21-33	11-9-36
Guntersville	Tenn.	AL	CGE	94	3,979	24,450	75.7	12-4-35	8-1-39
Nickajack[d]	Tenn.	TN	CGE	83	3,767	21,870	46.3	4-1-64	2-20-68
Chickamauga	Tenn.	TN	CGE	129	5,800	20,790	58.9	1-13-36	3-4-40
Watts Bar	Tenn.	TN	CGE	112	2,960	17,310	72.4	7-1-39	2-11-42
Ft. Loudoun	Tenn.	TN	CGE	122	4,190	9,550	55.0	7-8-40	11-9-43
Tims Ford	Elk	TN	E&R	170	1,470	529	34	3-28-66	11- -71
Apalachia	Hiwassee	NC	CG	150	1,308	1,018	9.8	7-17-41	9-22-43
Hiwassee	Hiwassee	NC	CG	307	1,376	968	22	7-15-36	5-21-40
Chatuge	Hiwassee	NC	E	144	2,850	189	13	7-17-41	12-9-54
Ocoee No. 1[c]	Ocoee	TN	CG	135	840	595	7.5	8- -10	1-10-12
Ocoee No. 2[c]	Ocoee	TN	RFT	30	450	516	-----	5- -12	10- -13
Ocoee No. 3	Ocoee	TN	CG	110	612	496	7	7-17-41	4-30-43

TABLE 2.4.4-204 (Sheet 2 of 3)
FACTS ABOUT MAJOR TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Main River and Tributary Projects	River	State	Type of Dam[b]	Max Height[a] (ft.)	Length (ft.)	Drainage Area (sq. mi.)	Length of Lake (mi.)	Construction Start	Construction Completion
Blue Ridge[c]	Toccoa	GA	E	167	1,000	232	10	11- -25[e]	7- -31
Nottely	Nottely	GA	E&R	184	2,300	214	20	7-17-41	1-10-56
Melton Hill	Clinch	TN	CG	103	1,020	3,343	44	9-6-60	7-3-64
Norris	Clinch	TN	CGE	265	1,860	2,912	72	10-1-33	7-28-36
Tellico	Little T.	TN	CGE	108	3,238	2,627	33.2	3-15-67	- -79
Fontana	Little T.	NC	CG	480	2,365	1,571	29	1-1-42	1-20-45
Douglas	French Broad	TN	CGE	202	1,705	4,541	43.1	2-2-42	3-21-43
Cherokee	Holston	TN	CGE	175	6,760	3,428	59	8-1-40	4-16-42
Fort Patrick Henry	S. Fork Holston	TN	CG	95	737	1,903	10.3	5-14-51	12-5-53
Boone	S. Fork Holston	TN	CGE	160	1,532	1,840	17.3	8-29-50	3-16-53
South Holston	S. Fork Holston	TN	E&R	285	1,600	703	24.3	8-4-47[f]	2-13-51
Watauga	Watauga	TN	E&R	318	900	468	16.7	7-22-46[f]	8-30-49
Great Falls[c] (Cumberland Valley)	Caney Fork	TN	CG	92	800	1,675	22	- -15	- -16

TABLE 2.4.4-204 (Sheet 3 of 3)
FACTS ABOUT MAJOR TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Main River and Tributary Projects	River	State	Type of Dam[b]	Max Height[a] (ft.)	Length (ft.)	Drainage Area (sq. mi.)	Length of Lake (mi.)	Construction Start	Construction Completion
Pumped Storage									
Raccoon Mountain	Tenn.	TN	E&R	230	8,080	-----	-----	7-6-70	1- -74

a) Foundation to operating deck.

b) Abbreviations: CG – Concrete gravity dams. CGE – Concrete gravity with earth embankments. E – Earthfill. E&R – Earth and rock fill. RFT – Rock-filled timber.

c) Acquired: Wilson by transfer from U.S. Corps of Engineers in 1933; Ocoee No. 1, Ocoee No. 2, Blue Ridge, and Great Falls by purchase from TEP Co. in 1939. Subsequent acquisition, TVA heightened and installed additional units at Wilson.

d) Nickajack Dam replaced the old Hales Bar Dam 6 miles upstream.

e) Construction discontinued early in 1926; resumed in March 1929.

f) Initial construction started February 16, 1942; temporarily discontinued to conserve critical materials during war.

TABLE 2.4.4-205 (Sheet 1 of 3)
STORAGE CHARACTERISTICS OF MAJOR TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Main River and Tributary Projects	Area of Lake at Full Pool (ac.)	Lake Elevation (ft. above msl)			Lake Volume (ac.-ft.)		Useful Controlled Storage (ac.-ft.)
		Minimum	Top of Gates	Full Pool[a]	At Minimum Elevation	At Top of Gates Elevation	
Kentucky	160,300	354	375	359	2,121,000	6,129,000	4,008,000
Pickwick Landing	43,100	408	418	414	688,000	1,105,000	417,000
Wilson[b]	15,500	504.5	507.88	507.5	582,000	641,000	59,000
Wheeler	67,100	550	556.3	556	720,000	1,071,000	351,000
Guntersville	67,900	593	595.44	595	379,700	1,052,000	172,300
Nickajack[c]	10,900	632	635	634	221,600	254,600	33,000
Chickamauga	35,400	675	685.44	682.5	392,000	739,000	347,000
Watts Bar	39,000	735	745	741	796,000	1,175,000	379,000
Ft. Loudoun	14,600	807	815	813	282,000	393,000	111,000
Tims Ford	10,700	860	895	888	294,000	617,000	323,000
Apalachia	1,100	1,272	1,280	1,280	48,600	57,500	8,900
Hiwassee	6,090	1,415	1,526.5	1,524.5	71,800	434,000	362,200
Chatuge	7,050	1,860	1,928	1,927	18,400	240,500	222,100
Ocoee No. 1[b]	1,890	816.9	837.65	837.65	53,500	87,300	33,800

TABLE 2.4.4-205 (Sheet 2 of 3)
STORAGE CHARACTERISTICS OF MAJOR TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Main River and Tributary Projects	Area of Lake at Full Pool (ac.)	Lake Elevation (ft. above msl)			Lake Volume (ac.-ft.)		Useful Controlled Storage (ac.-ft.)
		Minimum	Top of Gates	Full Pool[a]	At Minimum Elevation	At Top of Gates Elevation	
Ocoee No. 2[b]	-----	-----	1,115	1,115	-----	-----	-----
Ocoee No. 3	621	1,112	1,435	1,435	790	4,650	3,860
Blue Ridge[b]	3,290	1,590	1,691	1,690	12,500	196,500	184,000
Nottely	4,180	1,690	1,779	1,779	12,700	174,300	161,600
Melton Hill	5,690	790	796	795	94,500	126,000	31,500
Norris	34,200	930	1,034	1,020	290,000	2,555,000	2,265,000
Tellico	16,500	807	815	813	321,300	447,300	126,000
Fontana	10,640	1,525	1,710	1,708	295,000	1,448,000	1,153,000
Douglas	30,400	920	1,002	1,000	84,500	1,490,000	1,105,500
Cherokee	30,300	980	1,075	1,073	83,600	1,544,000	1,160,400
Fort Patrick Henry	872	1,258	1,263	1,263	22,700	26,900	4,200
Boone	4,400	1,330	1,385	1,385	45,000	193,400	148,400
South Holston	7,580	1,616	1,742	1,729	121,400	764,000	642,600
Watauga	6,430	1,815	1,975	1,959	52,300	677,000	624,700

BLN COL 2.4-2

TABLE 2.4.4-205 (Sheet 3 of 3)
STORAGE CHARACTERISTICS OF MAJOR TVA DAMS AND RESERVOIRS

Main River and Tributary Projects	Area of Lake at Full Pool (ac.)	Lake Elevation (ft. above msl)			Lake Volume (ac.-ft.)		Useful Controlled Storage (ac.-ft.)
		Minimum	Top of Gates	Full Pool[a]	At Minimum Elevation	At Top of Gates Elevation	
Great Falls[b] (Cumberland Valley)	2,100	780	805.30	805.30	14,600	51,600	37,000
Totals	638,353				8,621,490	23,732,359	15,110,860
Pumped Storage							
Raccoon Mountain	520	1,530	-----	1,672	2,000	37,800	35,400

a) Full pool elevation is the normal upper level to which the reservoirs may be filled. Where storage space is available above this level, additional filling may be made as needed for flood control.

b) Acquired: Wilson by transfer from U.S. Corps of Engineers in 1933; Ocoee No. 1, Ocoee No. 2, Blue Ridge, and Great Falls by purchase from TEP Co. in 1939. Subsequent acquisition, TVA heightened and installed additional units at Wilson.

c) Nickajack Dam replaced the old Hales Bar Dam 6 miles upstream.

TABLE 2.4.4-206 (Sheet 1 of 2)
FACTS ABOUT NON-TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Projects	River	Drainage Area (sq. mi.)	Distance from Mouth (mi.)	Maximum Height (ft.)	Length (ft.)	Area of Lake (ac.)	Length of Lake (mi.)	Total Storage[a] (ac.-ft.)	Construction Started
Major Dams									
Calderwood	Little Tenn	1,856	43.7	232	916	536	8	41,160	1928
Cheoah	Little Tenn	1,608	51.4	225	750	595	10	35,030	1916
Chilhowee	Little Tenn	1,976	33.6	91	1,373	1,690	8.9	49,250	1955
Nantahala	Nantahala	108	22.8	250	1,042	1,605	4.6	138,730	1930
Santeetlah	Cheoah	176	9.3	212	1,054	2,863	7.5	158,250	1926
Thorpe (Glenville)	West Fork Tuckasegee	36.7	9.7	150	900	1,462	4.5	70,810	1940
Minor Dams									
Bear Creek	East Fork Tuckasegee	75.3	4.8	215	740	476	4.6	34,711	1952
Cedar Cliff	East Fork Tuckasegee	80.7	2.4	165	600	121	2.4	6,315	1950

TABLE 2.4.4-206 (Sheet 2 of 2)
FACTS ABOUT NON-TVA DAMS AND RESERVOIRS

BLN COL 2.4-2

Projects	River	Drainage Area (sq. mi.)	Distance from Mouth (mi.)	Maximum Height (ft.)	Length (ft.)	Area of Lake (ac.)	Length of Lake (mi.)	Total Storage[a] (ac.-ft.)	Construction Started
Mission (Andrews)	Hiwassee	292	106.1	50	390	61	1.46	283	1924
Queens Creek	Queens Creek	3.58	1.5	78	382	37	0.5	817	1947
Wolf Creek	Wolf Creek	15.2	1.7	180	810	176	2.2	10,056	1952
East Fork	East Fork Tuckasegee	24.9	10.9	140	385	39	1.4	1,797	1952
Tuckasegee	West Fork Tuckasegee	54.7	3.1	61	254	9	0.5	183	1949
Walters (Carolina P&L)	Pigeon	455	38.0	200	870	340	5.5	25,390	1927

a) Volume at top of gates.

TABLE 2.4.7-201 (Sheet 1 of 3)
WATER TEMPERATURE DATA FOR THE TENNESSEE RIVER
AT SOUTH PITTSBURG, TN
(USGS STATION 03571850)

BLN COL 2.4-2

SAMPLE DATE	°F
9/12/1967	73.0
3/5/1968	42.8
8/26/1968	78.8
9/3/1968	78.8
11/27/1974	50.0
12/20/1974	44.6
1/30/1975	45.5
2/27/1975	46.4
3/27/1975	50.9
4/23/1975	56.3
5/21/1975	68.0
6/5/1975	71.6
7/21/1975	79.7
8/6/1975	79.7
9/11/1975	77.0
10/8/1975	65.3
11/6/1975	61.7
12/11/1975	50.0
1/20/1976	39.2
2/25/1976	48.2
3/18/1976	52.7
4/15/1976	58.1
5/6/1976	64.4
6/16/1976	73.4
7/15/1976	77.0
8/5/1976	78.8
9/15/1976	77.0
11/3/1976	57.2
11/23/1976	50.0
12/8/1976	46.4
1/12/1977	38.3
2/24/1977	45.3
3/24/1977	55.4
4/21/1977	65.3
5/26/1977	72.5
6/30/1977	80.6

TABLE 2.4.7-201 (Sheet 2 of 3)
WATER TEMPERATURE DATA FOR THE TENNESSEE RIVER
AT SOUTH PITTSBURG, TN
(USGS STATION 03571850)

BLN COL 2.4-2

SAMPLE DATE	°F
7/28/1977	82.4
8/30/1977	83.3
9/28/1977	76.1
10/20/1977	62.6
11/17/1977	59.0
1/26/1978	41.0
2/8/1978	36.5
3/9/1978	42.8
3/23/1978	50.0
4/6/1978	58.1
5/4/1978	60.8
6/21/1978	76.1
7/18/1978	83.3
8/3/1978	82.4
9/14/1978	80.6
10/11/1978	72.5
11/20/1978	62.6
12/14/1978	53.6
1/30/1979	42.8
2/28/1979	45.5
3/28/1979	50.9
5/3/1979	65.3
5/31/1979	68.0
7/13/1979	75.2
8/29/1979	77.9
10/16/1979	63.5
11/29/1979	54.5
12/18/1979	48.2
1/23/1980	46.4
2/27/1980	46.4
4/29/1980	61.7
5/29/1980	70.7
6/26/1980	77.0
8/27/1980	82.4
9/9/1980	82.4
10/30/1980	62.6

TABLE 2.4.7-201 (Sheet 3 of 3)
WATER TEMPERATURE DATA FOR THE TENNESSEE RIVER
AT SOUTH PITTSBURG, TN
(USGS STATION 03571850)

BLN COL 2.4-2

SAMPLE DATE	°F
11/24/1980	52.7
12/17/1980	50.0
2/19/1981	44.6
4/8/1981	58.1
5/28/1981	68.0
6/25/1981	80.6
7/22/1981	82.4
8/26/1981	79.7
9/16/1981	78.8
9/30/1981	73.4
10/30/1981	62.6
11/30/1981	55.4
12/16/1981	48.2
2/25/1982	48.2
4/21/1982	58.1
6/17/1982	77.0
8/12/1982	81.5
10/28/1982	62.6
1/6/1983	48.2
4/14/1983	56.3
7/28/1983	80.6
12/15/1983	50.0
2/29/1984	46.4
4/27/1984	59.0
7/18/1984	78.8
10/25/1984	69.8
1/25/1985	40.1
4/19/1985	62.6
10/23/1985	71.6
1/23/1986	42.8
4/17/1986	59.0
7/24/1986	80.6
3/10/1987	51.8
Min T, 2/8/1978	36.5
Max T, 8/30/1977 & 7/18/1978	83.3

(Reference 244)

TABLE 2.4.7-202
TVA MINIMUM MONTHLY WATER TEMPERATURES 2000-2006

BLN COL 2.4-2

Month	Nickajack Tailwater °F	Widows Creek Intake °F	Guntersville Tailwater °F
Jan	41.8	40.2	40.2
Feb	43.0	42.5	42.3
Mar	48.0	47.4	46.3
Apr	56.8	54.6	54.6
May	64.4	62.7	67.4
June	72.0	70.1	70.9
July	77.9	74.7	74.1
Aug	81.1	79.1	76.6
Sep	73.5	68.1	61.4
Oct	65.1	62.2	60.5
Nov	52.1	52.0	48.7
Dec	45.6	41.2	45.2

BLN COL 2.4-3

TABLE 2.4.11-201 (Sheet 1 of 2)
MINIMUM DAILY STREAMFLOW OBSERVED
ON THE TENNESSEE RIVER AT SOUTH PITTSBURG, TN
(USGS STATION 03571850)
1930-1987

Climatic Year[(a)]	Date	Minimum Daily Discharge, cfs
1930[(b)]	9/7/1930	5,950
1931	10/27/1931	5,350
1932	9/15/1932	6,940
1933	12/3/1933 & 12/4/1933	7,200
1934	9/28/1934	10,200
1935	10/8/1935 & 10/9/1935	6,460
1936	7/29/1936	10,400
1937	7/19/1937	12,200
1938	11/1/1938 & 11/2/1938	12,800
1939	1/30/1940 & 1/31/1940	6,500
1940	6/16/1940	7,410
1941	10/12/1941	7,940
1942	5/23/1942	9,110
1943	1/2/1944	13,000
1944	9/6/1944	17,200
1945	4/10/1945	15,900
1946	4/7/1946	14,100
1947	4/20/1947	13,700
1948	7/20/1948	16,500
1949	4/24/1949	19,100
1950	4/29/1950 & 4/30/1950	16,800
1951	7/4/1951	15,900
1952	11/16/1952	6,500
1953	11/1/1953 & 11/15/1953	2,900
1954	12/26/1954	5,500
1955	1/28/1956	4,600
1956	7/4/1956	13,400
1957	7/7/1957	10,200
1958	4/19/1958	9,200
1959	6/14/1959	4,500
1960	5/8/1960	10,700
1961	6/4/1961	10,200

BLN COL 2.4-3

TABLE 2.4.11-201 (Sheet 2 of 2)
MINIMUM DAILY STREAMFLOW OBSERVED
ON THE TENNESSEE RIVER AT SOUTH PITTSBURG, TN
(USGS STATION 03571850)
1930-1987

Climatic Year[a]	Date	Minimum Daily Discharge, cfs
1962	8/19/1962	13,800
1963	6/23/1963	8,800
1964	5/17/1964	13,500
1965	1/13/1966	10,400
1966	4/21/1966	8,400
1967	3/31/1968	6,840
1968	12/1/1968	6,930
1969	9/14/1969	8,350
1970	4/12/1970	8,240
1971	4/11/1971	15,500
1972	4/7/1972	20,200
1973	4/22/1973 & 10/28/1973	19,000
1974	9/21/1974	12,800
1975	9/28/1975	14,000
1976	5/1/1976	16,200
1977	8/21/1977	20,100
1978	11/8/1978	14,400
1979	2/29/1980	19,100
1980	3/1/1981	10,800
1981	11/22/1981	7,300
1982	4/23/1982	10,000
1983	11/13/1983	13,400
1984	1/26/1985	15,000
1985	9/29/1985	8,930
1986	5/4/1986	9,310
1987[b]	9/5/1987	11,100

a) Climatic Year – April 1 to March 31

b) Year 1930 incomplete, available data 7/1/1930 – 3/31/1931
Year 1987 incomplete, available data 4/1/1987 – 9/30/1987

(Reference 244)

BLN COL 2.4-3

TABLE 2.4.11-202
MINIMUM DAILY STREAMFLOW OBSERVED ON THE TENNESSEE RIVER AT CHATTANOOGA, TN (USGS STATION 03568000) AND WHITESBURG, AL (USGS STATION 03575500) 1988-2006

	Chattanooga, TN		Whitesburg, AL	
Climatic Year[a]	Date	Minimum Daily Discharge, cfs	Date	Minimum Daily Discharge, cfs
1988	5/25/1988	5740	6/11/1988	5930
1989	4/29/1989	6280	4/29/1989	8810
1990	4/24/1990	4540	6/26/1990	12,800
1991	6/9/1991	9800	10/13/1991	5400
1992	4/29/1992	5500	4/19/1992	5230
1993	5/30/1993	7870	9/12/1993	6810
1994	5/28/1994	9290	6/4/1994	8020
1995	5/6/1995	5740	5/6/1995	4850
1996	4/19/1996	8230	4/10/1996	7670
1997[b]	4/20/1997	10,300	5/24/1997 & 9/7/1997	12,400
1998[b]	2/27/1999	12,300	-	-
1999[b]	4/26/1999	7790	12/12/1999	2250
2000	5/7/2000	6820	2/4/2001	2300
2001	2/17/2002	4140	1/12/2002	631
2002	6/16/2002 & 7/21/2002	5000	4/27/2002	1100
2003[c]	3/28/2004	6020	4/2/2003	2720
2004[c]	5/18/2004	1090	3/27/2005	19,400
2005	5/15/2005	16,300	5/14/2005	12,800

a) Climatic Year – April 1 to March 31

b) Years 1997-1999 incomplete data from 10/1/1997 – 9/30/1999 for Whitesburg, AL gauge

c) Years 2003-2004 incomplete data from 10/1/2003 – 9/30/2004 for Whitesburg, AL gauge

BLN COL 2.4-3

TABLE 2.4.11-203
GUNTERSVILLE RESERVOIR TENNESSEE RIVER LOW FLOW VALUES, CFS, FOR 1, 7, AND 30 DAYS FOR SELECTED RETURN PERIODS[a]

	Return Period, years			
Duration, days	5	10	100	1,000
1	7,340	6,220	4,200	3,150
7	12,100	10,500	7,520	5,870
30	15,200	13,000	8,730	6,370

a) Low flow based on statistical analysis of data for USGS gauge on the Tennessee River at South Pittsburg, TN (USGS 03571850) from 1953 to 1987, and supplemented with interpolated data from USGS gauges on the Tennessee River at Chattanooga, TN (USGS 03568000) and Whitesburg, AL (USGS 03575500) from 1988 to 2005.

BLN COL 2.4-3

TABLE 2.4.11-204
MINIMUM DAILY STREAMFLOW OBSERVED ON THE TENNESSEE RIVER AT GUNTERSVILLE, AL (USGS STATION 03573500) 1930-1938

Climatic Year[a]	Date	Minimum Daily Discharge, cfs
1930[b]	9/8/1930	7,020
1931	10/28/1931	5,940
1932	9/16/1932	6,790
1933	11/2/1933	7,720
1934	9/19-22/1934 & 9/29/1934	11,800
1935	10/9/1935 &10/10/1935	6,640
1936	6/24-25/1936, 7/29/1936, 7/31/1936, & 8/27/1936	11,400
1937	10/17/1937	12,500
1939[b]	9/29/1938	17,500

a) Climatic Year – April 1 to March 31

b) Year 1930 incomplete, available data 5/1/1930 – 3/31/1931
Year 1938 incomplete, available data 4/1/1938 – 9/30/1938

(Reference 244)

BLN COL 2.4-4

TABLE 2.4.12-201 (Sheet 1 of 5)
WELL AND SPRING INVENTORY

Well Number[a]	Year Installed	Elevation[b] (ft. msl)	Well Depth (ft.)	Completion Zone	Comments
1	Unk	611	20	Unk	Private residential well[c]
2	Unk	621	Unk	Unk	Private residential well[c]
3	Unk	609	72	Unk	Private residential well[c]
4	Unk	602	Unk	Unk	Private residential well[c]
5	Unk	610	Unk	Unk	Private residential well[c]
6	Unk	600	Unk	Unk	Private residential well[c]
7	Unk	605	Unk	Unk	Private residential well[c]
8	Unk	608	Unk	Unk	Private residential well[c]
9	Unk	605	Unk	Unk	Private residential well[c]
10	Unk	605	Unk	Unk	Private residential well[c]
11	Unk	605	Unk	Unk	Private residential well[c]
12	Unk	629	172	Unk	Private residential well[c]
13	Unk	610	39	Unk	Private residential well[c]
14	Unk	623	33	Unk	Private residential well[c]
15	Unk	670	72	Unk	Private residential well[c]
16	Unk	629	102	Unk	Private residential well[c]

BLN COL 2.4-4

TABLE 2.4.12-201 (Sheet 2 of 5)
WELL AND SPRING INVENTORY

Well Number[a]	Year Installed	Elevation[b] (ft. msl)	Well Depth (ft.)	Completion Zone	Comments
17	Unk	619	34	Unk	Private residential well[c]
18	Unk	621	97	Unk	Private residential well[c]
19	Unk	637	70	Unk	Private residential well[c]
20	Unk	630	77	Unk	Private residential well[c]
21	Unk	620	70	Unk	Private residential well[c]
22	Unk	635	Unk	Unk	Private residential well[c]
23	Unk	617	55	Unk	Private residential well[c]
24	Unk	640	135	Unk	Private residential well[c]
25	Unk	630	131	Unk	Private residential well[c]
26	Unk	640	48	Unk	Private residential well[c]
27	Unk	640	200	Unk	Private residential well[c]
28	Unk	634	68	Unk	Private residential well[c]
29	Unk	630	72	Unk	Private residential well[c]
30	Unk	638	52	Unk	Private residential well[c]
31	Unk	615	Unk	Unk	Private residential well[c]
32	Unk	620	125	Unk	Private residential well[c]

BLN COL 2.4-4

TABLE 2.4.12-201 (Sheet 3 of 5)
WELL AND SPRING INVENTORY

Well Number[a]	Year Installed	Elevation[b] (ft. msl)	Well Depth (ft.)	Completion Zone	Comments
33	Unk	604	72	Unk	Private residential well[c]
34	Unk	639	116	Unk	Private residential well[c]
35	Unk	645	Unk	Unk	Private residential well[c]
S-1	N/A	637	Spring	N/A	Intermittent spring[d]
S-2	N/A	600	Spring	N/A	Intermittent spring[d]
WT1	1973	605.82	150.6	Bedrock[e]	Open boring, 455.9 – 597.2 msl
WT2	1973	625.74	150.4	Bedrock[e]	Open boring, 476.1 – 616.8 msl
WT3	1973	608.19	150.4	Bedrock[e]	Open boring, 457.2 – 599.6 msl
WT4	1973	598.99	150.2	Bedrock[e]	Open boring, 447.4 – 583.9 msl
WT5	1973	623.80	150.4	Bedrock[e]	Open boring, 473.4 – 614.7 msl
WT6	1973	611.76	150.4	Bedrock[e]	Open boring, 459.5 – 595.9 msl
B7	1978	602.30	82.8	Bedrock	Screened, 529.2 – 584.6 msl
B8	1978	605.10	85.0	Bedrock	Screened, 515.9 – 583.6 msl
W9	1984	605.80	13.7	Bedrock	Screened, 590.2 – 605.8 msl
W10	1984	603.20	11.6	Bedrock	Screened, 592.3 – 603.2 msl
W11	1984	599.10	12.0	Bedrock	Screened, 587.1 – 599.1 msl
W12	1990	622.95	34.3	Bedrock	[e]

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TABLE 2.4.12-201 (Sheet 4 of 5)
WELL AND SPRING INVENTORY

Well Number[(a)]	Year Installed	Elevation[(b)] (ft. msl)	Well Depth (ft.)	Completion Zone	Comments
W13	1990	607.60	29.2	Bedrock	(e)
W14	1990	659.05	40.9	Soil	(f)
W15	1990	648.68	20.4	Soil	(f)
W16	1990	638.39	28.4	Bedrock	(e)
W17	1990	626.60	15.0	Soil	(f)
W18	1990	652.02	23.0	Soil	(f)
W19	1990	615.81	24.6	Bedrock	(e)
W20	1992	635.12	37.3	Bedrock	(g)
W21	1992	629.82	50.0	Bedrock	(g)
W22	1992	631.80	43.4	Bedrock	(g)
W30	1996	605.86	43.6	Bedrock	(g)
W31	1996	616.81	18.2	Soil	(f)
W32	1996	625.95	33.0	Bedrock	(g)
W33	1996	677.70	37.3	Bedrock	(g)
P-1	2005	606.70	27.0	Bedrock	Screened, 594.70 – 579.70 msl
P-2	2005	620.91	40.3	Bedrock	Screened, 595.61 – 580.61 msl

BLN COL 2.4-4

TABLE 2.4.12-201 (Sheet 5 of 5)
WELL AND SPRING INVENTORY

Well Number[(a)]	Year Installed	Elevation[(b)] (ft. msl)	Well Depth (ft.)	Completion Zone	Comments
P-3	2005	604.41	24.3	Bedrock	Screened, 595.11 – 580.11 msl
P-4	2005	628.95	24.9	Bedrock	Screened, 619.05 – 604.05 msl

a) See Figure 2.4.12-203. This table does not include wells that have been abandoned prior to this report or wells installed after 2005.
b) Elevation at the ground surface (wells 1-35, springs S-1 and S-2) or top of well casing. Elevations were either obtained by reference or estimated from topographic maps.
c) Private water well data (wells 1-35) is from survey conducted in 1961. Additional private water wells may have been added and the existing ones changed; however, privately owned water supply wells are not required to be registered with the State of Alabama and no additional information related to well installations, well construction, water usage, groundwater level, or drawdown was available.
d) During field activities in 2006, flow was not observed from the two intermittent springs. With the exception of small intermittent seeps, no other springs were observed on the BLN site or in the surrounding area.
e) Monitoring well completed with solid surface casing, to the top of rock, and open borehole to the total depth of the well.
f) Monitoring well completed with slotted well screen in the soil above the top of rock. Completion information not available.
g) Monitoring well completed in bedrock below the soil zone. Completion information not available.

msl – Above mean sea level
Unk – Unknown
N/A – Not applicable

BLN COL 2.4-4

TABLE 2.4.12-202 (Sheet 1 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
1/10/1973	604.1	619.8	603.7	597.0	610.3	605.3
2/7/1973	603.9	617.9	603.2	596.3	610.3	604.4
3/28/1973	603.7	616.8	603.1	596.8	610.3	603.9
4/25/1973	603.4	613.8	601.4	595.6	606.7	599.5
5/23/1973	603.5	615.0	601.6	596.6	607.6	602.0
6/20/1973	603.2	615.0	601.9	596.0	607.9	602.0
7/18/1973	603.1	615.4	601.8	596.0	607.5	600.7
8/28/1973	603.9	622.0	604.9	597.7	615.5	609.9
9/19/1973	598.3	609.5	595.2	593.5	599.0	594.4
10/17/1973	598.1	609.1	594.8	593.3	598.1	594.4
11/14/1973	598.1	609.1	594.8	593.3	598.1	594.4
12/19/1973	602.3	611.0	598.0	594.5	602.1	597.5
1/21/1974	603.4	618.5	603.4	597.4	613.2	604.7
2/11/1974	603.7	618.1	603.1	597.3	612.6	604.5
3/11/1974	602.9	613.3	601.1	594.8	607.3	600.5
4/17/1974	603.3	616.6	602.6	595.7	611.4	603.5
5/29/1974	603.1	614.2	601.2	595.8	610.1	602.7
6/17/1974	601.8	611.1	598.0	594.7	605.0	597.0
7/15/1974	598.7	609.7	595.6	594.1	600.1	595.0
8/21/1974	597.6	608.8	594.6	593.4	598.7	594.1
9/23/1974	597.5	608.6	594.4	593.5	599.2	593.9
10/30/1974	597.6	609.1	594.9	592.6	601.8	593.9
11/13/1974	597.7	609.8	594.5	592.8	601.0	593.7
12/4/1974	602.0	612.8	598.8	595.2	605.1	600.2
1/10/1975	603.7	615.4	600.8	595.6	609.1	603.9
2/10/1975	603.9	616.6	601.3	596.4	609.1	605.2
3/5/1975	602.7	614.1	598.0	595.0	600.0	600.7
4/2/1975	603.6	615.4	599.3	598.1	598.4	603.4
5/9/1975	602.7	614.3	597.2	595.7	595.6	598.9
6/6/1975	602.8	613.3	596.4	595.1	598.7	596.9
7/7/1975	602.6	612.1	596.3	595.2	596.5	596.2
8/8/1975	602.1	612.5	596.8	594.5	597.4	597.4
9/3/1975	600.3	610.1	595.4	593.7	594.6	594.9
10/8/1975	603.6	615.9	599.5	596.2	599.2	601.2
11/5/1975	602.7	611.7	598.0	594.5	596.5	597.6
12/5/1975	602.9	612.9	599.1	595.3	597.6	601.5
1/15/1976	603.4	612.4	600.1	595.9	598.8	603.2
2/2/1976	603.4	610.8	599.4	595.4	597.8	601.3
3/8/1976	603.6	610.3	599.8	595.5	597.8	601.4
4/7/1976	603.6	610.3	599.8	595.3	598.1	601.4

BLN COL 2.4-4

TABLE 2.4.12-202 (Sheet 2 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
5/5/1976	603.3	608.6	599.2	595.4	597.7	599.2
6/1/1976	603.1	606.0	599.3	595.6	598.2	599.4
7/2/1976	603.5	605.5	599.7	595.6	598.4	599.9
8/3/1976	602.4	603.7	598.0	594.6	596.8	597.6
8/8/1976	602.8	605.2	599.2	594.7	599.8	598.4
9/9/1976	603.0	604.8	599.0	595.0	598.4	598.0
10/8/1976	602.8	605.2	599.2	594.7	599.8	598.4
11/10/1976	602.7	605.5	600.4	595.3	601.6	600.5
12/14/1976	603.1	600.9	600.4	595.9	605.3	602.5
1/13/1977	603.0	601.7	601.3	596.0	605.6	604.6
2/11/1977	602.4	600.2	600.0	594.9	603.6	601.5
3/21/1977	603.0	601.5	601.1	595.6	605.2	603.6
4/13/1977	603.1	602.6	601.5	596.1	605.0	603.4
5/12/1977	603.2	601.9	601.1	596.4	604.5	602.2
6/6/1977	602.8	602.3	599.5	595.0	603.7	599.2
7/6/1977	602.7	602.1	599.0	594.7	604.8	600.0
8/3/1977	603.8	603.1	600.1	595.3	603.6	599.3
9/7/1977	603.3	603.2	599.7	595.4	603.6	598.0
10/21/1977	603.0	603.8	600.3	595.1	603.4	600.7
11/10/1977	603.4	[a]	601.4	596.4	605.8	604.3
12/7/1977	603.4	[a]	601.4	596.7	606.1	604.0
1/4/1978	603.2	[a]	601.3	596.2	606.8	603.3
2/3/1978	603.4	[a]	601.3	596.0	605.8	603.5
3/1/1978	603.0	[a]	599.8	594.9	605.2	601.1
4/7/1978	602.9	610.7	600.1	595.0	604.7	601.3
5/16/1978	603.2	[a]	[a]	596.3	606.5	602.6
6/6/1978	603.3	[a]	[a]	595.1	605.0	599.3
7/12/1978	606.9	609.6	[a]	594.8	604.1	598.5
8/2/1978	602.9	[a]	[a]	594.6	603.8	598.4
9/15/1978	602.7	609.5	598.0	593.4	603.6	597.4
10/6/1978	602.9	610.1	598.3	593.9	605.1	597.9
11/7/1978	603.7	610.5	598.6	595.2	603.9	597.7
12/15/1978	603.3	612.7	598.6	595.6	606.6	600.4
1/23/1979	603.6	613.9	601.4	597.3	609.0	604.1
2/6/1979	603.3	610.5	600.6	595.5	608.9	604.1
3/15/1979	603.5	611.5	601.2	590.5	608.7	609.3
4/11/1979	603.5	611.4	601.3	596.3	607.8	609.2
5/11/1979	603.4	611.3	601.2	595.7	605.8	601.3

TABLE 2.4.12-202 (Sheet 3 of 7)
BLN COL 2.4-4
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
6/20/1979	603.1	611.3	599.5	594.9	605.5	601.3
7/6/1979	603.0	611.1	599.0	594.4	(a)	601.9
8/9/1979	603.2	611.4	599.5	594.5	(a)	602.1
9/7/1979	603.4	612.2	600.2	595.7	606.6	601.4
10/12/1979	603.4	611.9	600.1	595.7	606.4	601.1
11/20/1979	603.2	611.8	600.1	595.6	606.3	601.1
12/12/1979	603.2	611.7	599.8	595.4	606.2	600.8
1/16/1980	603.4	611.3	600.6	596.0	607.6	602.9
2/8/1980	603.4	611.2	600.6	595.8	607.6	602.9
3/14/1980	603.8	612.9	598.5	596.8	608.2	603.8
5/13/1980	603.4	612.3	597.6	595.8	607.2	601.5
6/18/1980	603.2	612.5	596.9	595.0	607.3	601.5
7/21/1980	603.0	612.3	596.3	594.0	606.2	598.8
8/13/1980	602.9	612.3	596.2	593.3	605.8	598.0
9/29/1980	603.8	(a)	597.0	595.3	606.5	600.5
10/15/1980	603.5	612.0	598.9	594.3	605.0	600.1
11/26/1980	603.7	613.2	601.0	597.2	604.3	599.5
1/9/1981	603.3	611.9	596.6	594.9	605.2	600.2
2/20/1981	603.7	(a)	597.8	597.1	609.7	603.4
3/10/1981	603.4	(a)	597.3	595.9	607.6	602.5
4/8/1981	603.4	(a)	600.5	596.1	608.0	602.8
5/6/1981	603.3	(a)	599.7	595.2	606.8	602.0
6/18/1981	603.5	611.6	600.8	595.5	607.4	601.8
7/8/1981	603.3	611.5	597.4	595.0	607.1	598.8
8/12/1981	603.4	(a)	597.3	595.0	607.0	598.5
9/25/1981	603.6	610.1	596.7	594.2	(a)	598.0
10/14/1981	603.5	610.1	595.5	593.9	605.0	598.1
11/17/1981	603.7	610.7	596.6	594.4	606.0	599.2
12/9/1981	603.8	611.1	597.4	594.4	605.3	600.8
1/18/1982	603.9	611.3	597.6	595.8	606.9	602.1
2/24/1982	603.8	610.2	598.2	596.0	606.9	602.6
3/25/1982	603.8	610.1	598.0	595.4	605.6	601.6
4/15/1982	603.8	611.5	597.9	595.0	605.3	600.6
5/21/1982	604.2	611.1	600.7	595.7	605.8	600.8
6/21/1982	604.1	610.6	598.9	594.9	607.1	598.7
7/21/1982	604.1	610.7	600.0	594.8	605.7	598.9
8/26/1982	604.1	612.7	599.1	594.3	606.7	598.7
9/23/1982	603.8	612.8	598.6	594.0	606.8	598.1

BLN COL 2.4-4

TABLE 2.4.12-202 (Sheet 4 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
10/12/1982	603.7	612.7	598.6	593.7	606.8	598.1
11/13/1982	(a)	614.0	599.3	594.5	609.0	599.5
12/13/1982	604.3	614.5	602.2	597.3	608.0	603.5
1/19/1983	604.2	614.0	601.0	596.7	607.7	603.1
2/8/1983	604.2	614.1	601.2	596.6	608.8	603.2
3/17/1983	604.1	614.9	601.2	595.4	606.4	601.8
4/18/1983	604.2	614.5	601.3	595.8	607.2	602.3
5/24/1983	604.3	614.8	602.1	597.8	606.6	603.1
6/29/1983	604.0	613.5	599.8	594.8	608.7	599.8
7/21/1983	604.3	614.1	599.9	594.3	606.9	598.6
8/26/1983	604.2	614.0	599.4	594.1	606.5	601.3
9/12/1983	604.3	614.0	599.0	594.4	607.1	598.1
10/20/1983	603.8	613.6	598.6	594.2	(a)	599.8
11/28/1983	(a)	615.2	601.7	597.2	(a)	602.2
12/21/1983	604.1	613.7	600.8	595.6	(a)	601.9
1/27/1984	604.8	614.4	601.7	596.5	608.0	603.1
2/27/1984	(a)	613.0	601.3	596.7	607.6	602.6
3/30/1984	604.2	613.6	601.8	596.8	608.4	603.2
4/26/1984	604.2	613.6	601.8	596.8	608.2	602.0
5/31/1984	604.4	613.3	600.7	595.6	608.8	600.9
6/25/1984	604.3	613.4	600.0	595.2	608.0	600.1
7/31/1984	(a)	614.1	601.0	597.1	608.7	601.9
8/30/1984	604.3	613.5	600.0	594.8	608.4	600.1
9/26/1984	603.8	611.8	598.9	594.4	606.2	598.6
10/26/1984	604.3	613.1	601.4	596.4	608.0	602.5
11/30/1984	604.3	613.7	601.3	596.7	608.1	602.9
12/18/1984	604.2	612.9	600.7	595.2	608.0	601.9
1/31/1985	(a)	612.5	601.3	596.4	608.3	602.8
2/14/1985	604.4	613.3	601.7	596.4	609.2	603.2
3/29/1985	604.4	612.4	601.1	595.9	608.0	602.3
4/24/1985	604.4	612.8	600.6	595.2	607.3	601.3
5/29/1985	(a)	612.6	599.9	595.0	607.7	600.3
6/28/1985	604.4	612.7	599.6	594.4	607.3	599.8
7/31/1985	604.5	613.5	599.8	595.1	607.6	600.4
8/28/1985	604.5	613.4	600.7	595.2	607.8	600.7
9/24/1985	604.0	612.0	598.8	593.7	607.0	598.9
10/24/1985	604.5	612.8	600.3	595.0	607.8	601.0
11/26/1985	604.4	613.7	599.8	594.7	608.0	600.4

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TABLE 2.4.12-202 (Sheet 5 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
1/2/1986	604.3	614.0	600.8	595.9	607.5	601.5
1/31/1986	604.0	613.0	600.6	595.4	607.3	601.4
2/28/1986	604.3	613.7	601.0	595.7	606.7	602.3
3/26/1986	604.4	613.2	600.9	595.7	607.7	602.3
4/29/1986	604.1	613.1	599.7	594.7	607.2	600.0
5/30/1986	604.6	614.5	601.8	596.9	610.8	602.7
6/24/1986	604.3	613.2	599.7	594.8	607.3	599.5
7/31/1986	604.4	614.0	598.9	594.1	606.3	598.8
8/29/1986	604.3	614.3	598.7	594.1	606.2	598.8
9/25/1986	604.4	614.4	599.4	594.6	606.8	599.2
10/30/1986	604.5	614.8	600.6	595.9	607.0	601.0
11/26/1986	[a]	615.4	602.3	597.8	608.8	603.3
12/31/1986	605.3	614.6	601.2	596.1	607.5	602.4
1/30/1987	604.7	616.5	601.6	596.4	607.8	603.0
2/27/1987	[a]	615.3	602.4	597.5	608.5	603.4
3/31/1987	[a]	613.9	601.7	596.9	608.0	602.5
4/29/1987	604.6	614.2	600.3	595.4	607.8	601.1
5/29/1987	604.6	614.2	599.6	595.3	607.7	599.9
6/24/1987	604.8	614.5	600.3	594.9	607.8	601.0
7/30/1987	604.3	614.1	598.9	594.8	607.1	598.6
8/31/1987	604.5	614.3	598.6	594.6	606.9	598.2
10/1/1987	604.7	614.3	599.3	595.0	607.2	599.9
10/28/1987	604.5	614.4	599.4	595.2	606.9	600.2
11/17/1987	604.6	614.6	600.0	596.4	606.2	600.1
11/19/1987	602.3	611.0	598.0	594.5	602.1	597.5
12/16/1987	604.8	614.4	600.6	596.3	606.6	600.9
1/20/1988	[a]	614.6	602.4	597.2	609.1	603.3
3/1/1988	605.0	614.2	600.7	595.2	606.8	601.6
3/30/1988	604.4	614.3	600.3	595.1	606.4	601.2
4/26/1988	605.0	614.5	600.9	595.8	606.4	601.7
6/10/1988	604.7	613.9	599.0	594.4	607.0	599.2
7/7/1988	604.7	613.6	599.1	594.6	607.4	600.0
8/17/1988	604.5	613.5	599.0	594.4	607.3	599.1
9/13/1988	604.9	614.2	601.5	597.0	607.7	601.5
10/17/1988	604.1	612.9	599.6	594.9	607.2	599.3
11/16/1988	604.5	613.3	601.4	595.9	607.0	601.6
12/12/1988	604.4	613.4	600.9	595.4	607.5	601.4
1/26/1989	604.6	613.2	601.4	595.9	607.8	602.3
2/22/1989	604.8	613.9	602.3	597.4	607.5	603.2

BLN COL 2.4-4

TABLE 2.4.12-202 (Sheet 6 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
3/13/1989	604.8	613.8	601.9	596.3	607.6	602.9
4/12/1989	604.9	614.1	602.3	596.8	608.3	603.1
5/24/1989	604.6	613.8	600.6	595.5	607.1	601.5
6/15/1989	(a)	614.3	601.8	597.2	607.6	602.2
7/19/1989	604.8	613.4	601.4	595.9	608.9	602.1
8/23/1989	604.6	612.6	599.6	595.0	606.7	600.0
9/19/1989	604.8	613.6	601.5	596.1	607.6	601.6
10/23/1989	604.5	612.9	600.2	595.3	607.8	600.7
11/13/1989	(a)	(a)	(a)	(a)	607.1	601.7
12/27/1989	604.6	612.9	601.4	595.7	606.9	601.9
01/31/90	604.83	614.06	602.36	597.07	607.90	603.50
02/27/90	604.79	613.74	602.01	596.77	608.12	602.73
03/21/90	604.88	614.21	602.35	597.41	608.16	603.38
04/13/90	604.70	613.45	601.50	596.20	607.50	602.15
05/23/90	604.73	613.69	601.55	596.03	607.70	602.06
06/29/90	604.38	612.35	599.09	606.35	606.35	598.34
07/17/90	604.55	613.19	599.95	595.04	607.25	600.79
08/16/90	604.34	612.25	599.58	594.95	607.03	599.94
09/07/90	604.15	612.58	599.30	594.50	607.50	599.95
10/18/90	604.42	612.70	599.37	594.96	607.40	599.47
11/19/90	604.54	613.10	600.50	595.52	607.35	600.40
12/27/90	(a)	614.69	602.40	597.63	607.57	603.47
01/25/91	604.48	613.30	601.70	596.10	607.35	602.36
02/27/91	605.00	614.00	602.65	597.25	607.60	603.45
03/14/91	(a)	613.75	602.43	596.89	608.30	603.00
04/22/91	604.96	613.45	602.04	596.91	607.85	602.30
05/29/91	604.90	613.80	600.95	595.50	608.40	601.50
06/24/91	604.70	613.10	600.20	595.55	607.25	600.65
07/19/91	604.73	613.30	600.15	595.70	607.55	600.67
08/28/91	604.70	613.00	600.05	595.60	605.25	600.80
09/20/91	604.45	612.70	599.08	595.60	606.70	599.65
10/22/91	604.22	612.74	598.74	594.94	607.50	597.83
11/14/91	603.82	611.94	598.07	594.94	605.90	597.26
12/17/91	604.96	(a)	602.19	596.94	608.35	602.86
01/22/92	604.67	(a)	601.39	596.19	608.10	601.86
03/16/92	604.69	(a)	601.60	596.08	608.20	601.86
10/29/92	(a)	612.91	(a)	(a)	(a)	(a)
11/05/92	(a)	614.14	(a)	(a)	(a)	(a)

BLN COL 2.4-4

TABLE 2.4.12-202 (Sheet 7 of 7)
HISTORIC GROUNDWATER ELEVATIONS

Date	WT-1 (ft. msl)	WT-2 (ft. msl)	WT-3 (ft. msl)	WT-4 (ft. msl)	WT-5 (ft. msl)	WT-6 (ft. msl)
11/12/92	(a)	613.70	(a)	(a)	(a)	(a)
11/20/92	(a)	613.00	(a)	(a)	(a)	(a)
11/25/92	(a)	613.95	(a)	(a)	(a)	(a)
12/04/92	(a)	613.20	(a)	(a)	(a)	(a)
12/10/92	(a)	613.00	(a)	(a)	(a)	(a)
12/18/92	(a)	614.05	(a)	(a)	(a)	(a)
01/05/93	(a)	613.70	(a)	(a)	(a)	(a)
01/14/93	(a)	614.53	(a)	(a)	(a)	(a)
01/19/93	(a)	613.75	(a)	(a)	(a)	(a)
02/05/93	(a)	612.96	(a)	(a)	(a)	(a)
02/19/93	(a)	613.32	(a)	(a)	(a)	(a)
03/02/05	605.21	614.44	602.39	596.89	(a)	603.16
05/04/05	604.72	612.74	601.43	595.83	606.40	601.48
07/28/05	604.47	612.11	599.99	595.04	606.65	598.95
09/21/05	603.65	611.42	598.34	594.58	605.06	597.26

a) Groundwater elevation data not available

msl - Above mean sea level.

TABLE 2.4.12-203 (Sheet 1 of 3)

BLN COL 2.4-4

GROUNDWATER MONITORING WELL INSTALLATION DETAILS

Monitoring Point	Reference Elevation (ft. msl)	Ground Elevation (ft. msl)	Well Depth (ft. bre)	Screen Length (ft.)	Top of Screen (ft. msl)	Btm of Screen[(a)] (ft. msl)	Boring Depth (ft. bgs)
MW-1201a	613.91	611.05	12.91	5.00	606.45	601.45	10.05
MW-1201b	613.78	611.04	77.81	10.00	546.42	536.42	75.07
MW-1201c	613.65	610.91	119.00	20.00	515.10	495.10	116.26
MW-1202a	617.52	614.99	15.42	5.00	607.55	602.55	12.89
MW-1202c	617.62	614.93	53.00	10.00	575.07	565.07	50.31
MW-1203a	621.93	619.02	12.57	5.00	614.81	609.81	9.66
MW-1203b	621.86	619.14	32.90	10.00	599.41	589.41	30.18
MW-1203c	621.70	619.04	121.00	20.00	521.15	501.15	118.34
MW-1204a	623.10	620.45	12.95	5.00	615.60	610.60	10.30
MW-1204b	623.16	620.48	53.20	10.00	580.41	570.41	50.52
MW-1204c	623.10	620.49	124.20	20.00	519.35	499.35	121.59
MW-1205a	629.42	627.04	13.00	5.00	621.87	616.87	10.62
MW-1205b	629.34	627.01	33.16	10.00	606.63	596.63	30.83
MW-1205c	629.14	626.89	49.11	10.00	590.48	580.48	46.86
MW-1206b	650.35	647.57	27.00	10.00	633.80	623.80	24.22
MW-1206c	649.95	647.40	52.80	10.00	607.60	597.60	50.25
MW-1207a	619.78	617.09	14.80	5.00	610.43	605.43	12.11
MW-1207b	619.80	617.24	21.00	5.00	604.25	599.25	18.44
MW-1207c	619.90	617.11	53.45	10.00	576.90	566.90	50.66
MW-1208a	617.33	614.79	18.73	10.00	609.05	599.05	16.19
MW-1208b	617.22	614.72	32.15	5.00	590.52	585.52	29.65

TABLE 2.4.12-203 (Sheet 2 of 3)
BLN COL 2.4-4
GROUNDWATER MONITORING WELL INSTALLATION DETAILS

Monitoring Point	Reference Elevation (ft. msl)	Ground Elevation (ft. msl)	Well Depth (ft. bre)	Screen Length (ft.)	Top of Screen (ft. msl)	Btm of Screen[(a)] (ft. msl)	Boring Depth (ft. bgs)
MW-1208c	617.26	614.69	57.95	10.00	569.76	559.76	55.38
MW-1209b	640.39	637.78	28.00	10.00	622.84	612.84	25.39
MW-1209c	640.44	637.84	53.05	10.00	597.84	587.84	50.45
MW-1210a	607.88	605.03	14.30	5.00	599.03	594.03	11.45
MW-1210b	608.01	605.04	33.79	10.00	584.67	574.67	30.82
MW-1210c	607.96	605.15	69.72	10.00	548.69	538.69	66.91
MW-1211a	618.87	615.89	11.53	5.00	612.79	607.79	8.55
MW-1211c	618.66	615.76	38.10	10.00	591.01	581.01	35.20
MW-1212a	607.00	603.98	15.28	5.00	597.17	592.17	12.26
MW-1212b	606.86	604.07	33.55	10.00	583.76	573.76	30.76
MW-1212c	606.79	603.94	64.00	10.00	553.24	543.24	61.15
MW-1213b	632.02	629.21	40.00	10.00	602.47	592.47	37.19
MW-1213c	632.20	629.42	50.00	10.00	592.65	582.65	47.22
MW-1214a	612.23	609.53	14.00	5.00	603.68	598.68	11.30
MW-1214b	612.09	609.74	22.80	5.00	594.74	589.74	20.45
MW-1214c	612.08	609.54	43.50	10.00	579.03	569.03	40.96
MW-1215a	635.64	632.79	13.25	5.00	627.84	622.84	10.40
MW-1215b	635.63	632.77	33.00	10.00	613.08	603.08	30.14
MW-1215c	635.60	632.79	52.50	10.00	593.55	583.55	49.69
MW-1216a	604.56	602.57	25.10	10.00	589.91	579.91	23.11
MW-1216c	604.64	602.23	63.00	10.00	552.09	542.09	60.59
MW-1217a	617.32	614.27	13.34	5.00	609.43	604.43	10.29

BLN COL 2.4-4

TABLE 2.4.12-203 (Sheet 3 of 3)
GROUNDWATER MONITORING WELL INSTALLATION DETAILS

Monitoring Point	Reference Elevation (ft. msl)	Ground Elevation (ft. msl)	Well Depth (ft. bre)	Screen Length (ft.)	Top of Screen (ft. msl)	Btm of Screen[a] (ft. msl)	Boring Depth (ft. bgs)
MW-1217b	617.10	614.15	33.30	10.00	594.25	584.25	30.35
MW-1217c	617.08	614.14	52.80	10.00	574.73	564.73	49.86
OW-1	623.33	620.55	82.89	20.00	560.89	540.89	80.11
OW-2	621.20	618.43	92.90	20.00	548.75	528.75	90.13
OW-3	622.89	620.13	82.87	20.00	560.47	540.47	80.11
OW-4	623.23	620.40	13.03	5.00	615.65	610.65	10.20
OW-5	621.20	618.34	12.80	5.00	613.85	608.85	9.94
OW-6	623.23	620.45	12.90	5.00	615.78	610.78	10.12
OW-7	617.46	614.78	52.85	10.00	575.06	565.06	50.17
OW-8	618.00	615.33	54.20	10.00	574.25	564.25	51.53
OW-9	615.58	613.10	52.70	10.00	573.33	563.33	50.22
OW-10	616.24	613.31	33.45	10.00	593.24	583.24	30.52
OW-11	616.39	613.71	32.92	10.00	593.92	583.92	30.24
OW-12	617.45	614.76	32.95	10.00	594.95	584.95	30.26

a) Bottom of screen includes 0.45 ft. (5.4 in.) for bottom cap and threads. Bottom of screen elevation = reference elevation - well depth + 0.45 ft.

msl - Above mean sea level.
bre - Below reference elevation (top of well casing).
bgs - Below ground surface.

BLN COL 2.4-4

TABLE 2.4.12-204 (Sheet 1 of 3)
GROUNDWATER AND SURFACE WATER ELEVATIONS

(ft. above msl)

Monitoring		2006									2007					Minimum		Maximum		
Point	**Notes**	**6/11**	**7/11**	**8/31**	**9/21**	**9/22**	**9/26**	**10/26**	**11/13**	**12/11**	**1/11**	**2/1**	**3/5**	**4/17**	**5/8**	**Date**	**Elevation**	**Date**	**Elevation**	**Change**
MW-1201a	(a,e)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
MW-1201b	(a)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
MW-1201c	(a)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
MW-1202a	(e)	605.93	605.57	605.80	605.30	606.04	610.30	607.78	607.89	605.12	610.69	609.60	608.00	607.92	605.12	12/11/06	605.12	1/11/07	610.69	5.57
MW-1202c	(d)	565.95	565.59	565.67	565.71	565.72	565.73	566.24	566.31	567.66	566.71	566.72	566.79	566.81	566.81	(n)	(n)	(n)	(n)	(n)
MW-1203a	(a,e)	Dry	609.70	610.10	610.00	610.01	609.99	609.74	609.90	610.28	610.72	611.18	611.15	611.23	611.77	(n)	(n)	(n)	(n)	(n)
MW-1203b	(m)	609.01	608.59	608.85	591.49(h)	593.73(h)	602.27(h)	609.51	609.55	608.54	610.06	609.93	610.08	610.00	609.35	(n)	(n)	(n)	(n)	(n)
MW-1203c	(b)	504.08	512.93	513.05	513.06	513.08	513.08	513.11	513.16	513.51	513.86	514.34	514.78	515.25	515.41	12/11/06	608.54	3/5/07	610.08	1.54
MW-1204a	(d,e)	Dry	610.55	611.75	611.98	612.00	612.11	612.62	613.00	613.38	613.74	614.21	614.58	614.68	614.65	(n)	(n)	(n)	(n)	(n)
MW-1204b	(a)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
MW-1204c	(m)	607.73	608.10	608.57	608.05	608.94	612.58	610.58	610.48	607.99	612.15	609.99	610.25	610.40	608.28	12/11/06	607.99	9/26/06	612.58	4.59
MW-1205a	(b,e)	618.21	617.51	617.31	617.23	617.23	617.21	617.09	617.09	617.08	617.07	617.08	617.08	617.06	617.06	(n)	(n)	(n)	(n)	(n)
MW-1205b	(f)	607.85	607.18	606.99	606.23	606.45	612.41	611.75	611.34	607.67	613.30	610.05	610.92	610.91	607.72	9/21/06	606.23	1/11/07	613.30	7.07
MW-1205c	(m)	607.12	607.14	607.95	606.91	610.36	613.29	611.15	610.98	607.18	612.90	611.61	610.59	610.59	607.41	9/21/06	606.91	9/26/06	613.29	6.38
MW-1206b	(f)	639.25	639.12	639.65	639.72	639.77	640.81	640.02	640.12	639.20	640.41	639.99	640.21	640.45	639.82	7/11/06	599.28	1/11/07	604.14	4.86
MW-1206c	(m)	629.48	632.01	633.93	634.28	634.35	634.64	635.61	636.06	635.92	636.03	635.93	635.39	634.60	634.23	9/21/06	605.77	1/11/07	610.20	4.43
MW-1207a	(b,e)	606.80	606.84	607.18	607.15	607.16	607.21	607.14	607.10	606.77	606.72	606.98	606.87	606.86	606.83	(n)	(n)	(n)	(n)	(n)
MW-1207b	(d)	602.80	605.46	607.67	607.67	607.70	607.69	607.63	607.60	607.17	607.04	607.28	607.14	607.29	607.30	(n)	(n)	(n)	(n)	(n)
MW-1207c	(m)	602.47	599.28	601.97	601.69	602.19	603.68	603.26	603.57	600.89	604.14	603.89	603.78	603.70	602.64	7/11/06	599.28	1/11/07	604.14	4.86
MW-1208a	(e)	607.18	606.12	605.80	605.77	606.06	608.29	608.93	609.19	607.23	610.20	608.85	609.32	609.37	607.52	9/21/06	605.77	1/11/07	610.20	4.43
MW-1208b	(m)	607.09	606.07	606.02	605.79	606.17	608.06	608.67	608.94	607.13	609.86	608.70	609.07	609.14	607.47	9/21/06	605.79	1/11/07	609.86	4.07
MW-1208c	(f)	607.08	604.33	605.57	605.41	605.44	606.05	608.03	608.56	607.52	608.79	608.84	608.52	607.70	606.39	7/11/06	604.33	2/1/07	608.84	4.51
MW-1209b	(m)	619.53	617.81	617.47	617.11	617.24	621.29	620.02	619.99	619.24	620.13	620.14	620.17	619.89	619.09	9/21/06	617.11	9/26/06	621.29	4.18

BLN COL 2.4-4

TABLE 2.4.12-204 (Sheet 2 of 3)
GROUNDWATER AND SURFACE WATER ELEVATIONS

(ft. above msl)

Monitoring Point	Notes	2006 6/11	7/11	8/31	9/21	9/22	9/26	10/26	11/13	12/11	2007 1/11	2/1	3/5	4/17	5/8	Minimum Date	Minimum Elevation	Maximum Date	Maximum Elevation	Change
MW-1209c	(b)	603.45	594.19	594.28	594.29	594.30	594.29	594.32	594.33	594.34	594.37	594.36	594.36	594.48	594.56	(n)	(n)	(n)	(n)	(n)
MW-1210a	(e)	597.47	597.63	597.28	596.34	596.37	599.91	600.08	601.20	600.05	602.42	602.14	601.98	602.23	600.25	9/21/06	596.34	1/11/07	602.42	6.08
MW-1210b	(m)	581.22	596.31	595.83	595.66	595.68	597.06	598.92	601.47	599.84	602.14	601.99	602.11	602.05	600.03	9/21/06	595.66	1/11/07	602.14	6.48
MW-1210c	(b)	543.25	543.30	543.38	543.41	543.41	543.41	543.49	543.58	543.64	543.71	543.75	543.78	543.79	543.86	(n)	(n)	(n)	(n)	(n)
MW-1211a	(b,e)	608.57	608.59	608.46	608.39	608.38	608.38	608.30	608.30	608.25	608.23	608.25	608.14	608.07	608.03	(n)	(n)	(n)	(n)	(n)
MW-1211c	(m)	609.96	608.55	608.81	608.66	608.64	610.31	610.88	610.70	610.36	611.91	610.41	610.74	609.93	609.00	7/11/06	608.55	1/11/07	611.91	3.36
MW-1212a	(e)	596.25	594.96	594.38	594.12	594.10	594.88	598.45	599.68	598.70	600.64	600.35	600.38	600.48	598.18	9/22/06	594.10	1/11/07	600.64	6.54
MW-1212b	98145.45 189	595.37	594.16	594.01	593.46	593.76	595.79	597.72	598.74	597.91	599.54	599.34	598.98	599.33	597.66	9/21/06	593.46	1/11/07	599.54	6.08
MW-1212c	(f)	591.05	594.07	593.86	591.68	593.66	593.97	595.67	596.99	595.90	596.74	598.36	598.84	598.56	598.25	9/21/06	591.68	3/5/07	598.84	7.16
MW-1213b	(m)	603.12	608.38	609.30	608.23	610.70	612.64	610.49	610.46	607.91	611.87	610.82	610.32	610.45	608.30	12/11/06	607.91	9/26/06	612.64	4.73
MW-1213c	(f)	607.75	608.30	609.22	608.14	610.80	612.53	610.55	610.50	607.88	611.90	611.25	610.33	610.37	608.23	12/11/06	607.88	9/26/06	612.53	4.65
MW-1214a	(e)	600.85	600.99	600.77	601.02	601.34	602.37	603.09	603.65	603.16	604.42	604.13	604.31	604.31	603.41	8/31/06	600.77	1/11/07	604.42	3.65
MW-1214b	(m)	600.08	596.62	599.42	598.83	599.57	601.07	600.41	601.17	598.76	602.94	602.90	603.00	602.82	601.68	7/11/06	596.62	3/5/07	603.00	6.38
MW-1214c	(f)	599.93	596.78	599.81	598.95	600.05	601.07	600.36	601.05	599.03	602.68	602.78	602.66	602.46	601.63	7/11/06	596.78	2/1/07	602.78	6.00
MW-1215a	(c,e)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
MW-1215b	(m)	613.98	613.61	613.72	613.76	613.85	615.66	616.02	615.41	614.11	616.60	614.71	615.05	615.10	614.14	7/11/06	613.61	1/11/07	616.60	2.99
MW-1215c	(f)	613.85	613.47	613.71	613.83	613.78	616.76	614.80	614.65	613.78	615.71	614.15	614.52	614.72	613.97	7/11/06	613.47	9/26/06	616.76	3.29
MW-1216a	(e)	597.24	595.22	594.70	594.34	594.47	596.21	597.76	598.86	597.88	599.80	599.68	599.32	600.30	597.88	9/21/06	594.34	4/17/07	600.30	5.96
MW-1216c	(m)	597.05	595.31	594.97	594.54	594.83	596.54	597.84	598.64	597.94	599.42	599.23	598.89	599.06	598.00	9/21/06	594.54	1/11/07	599.42	4.88
MW-1217a	(d,e)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	604.15	604.44	604.97	605.22	605.22	604.80	(n)	(n)	(n)	(n)	(n)
MW-1217b	(m)	599.90	598.08	600.07	599.97	600.42	601.79	601.38	601.69	599.23	602.31	602.14	602.09	601.94	600.84	7/11/06	598.08	1/11/07	602.31	4.23
MW-1217c	(b)	603.88	570.41	570.45	570.47	570.47	570.47	570.48	570.48	570.50	570.52	570.51	570.53	570.53	570.54	(n)	(n)	(n)	(n)	(n)
OW-1	(a)	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	(n)	(n)	(n)	(n)	(n)
OW-2	(b)	604.30	545.80	545.96	546.06	546.05	546.06	546.15	546.17	546.25	546.34	546.41	546.62	546.86	546.95	(n)	(n)	(n)	(n)	(n)

BLN COL 2.4-4

TABLE 2.4.12-204 (Sheet 3 of 3)
GROUNDWATER AND SURFACE WATER ELEVATIONS

(ft. above msl)

Monitoring Point	Notes	2006 6/11	7/11	8/31	9/21	9/22	9/26	10/26	11/13	12/11	2007 1/11	2/1	3/5	4/17	5/8	Minimum Date	Minimum Elevation	Maximum Date	Maximum Elevation	Change
OW-3	(d)	608.79	572.21	598.78	602.11	602.30	602.86	605.64	606.81	607.76	608.39	609.00	609.21	609.19	609.13	(n)	(n)	(n)	(n)	(n)
OW-4	(d,e)	Dry	Dry	Dry	Dry	Dry	617.52	614.67	614.47	613.55	612.95	612.77	612.47	612.05	611.96	(n)	(n)	(n)	(n)	(n)
OW-5	(d,e)	Dry	609.49	610.71	610.40	610.39	613.95	614.38	615.17	613.30	613.56	613.25	612.98	612.72	612.50	(n)	(n)	(n)	(n)	(n)
OW-6	(d,e)	615.89	615.92	616.04	615.92	616.24	616.21	615.92	616.02	615.86	616.20	616.31	615.92	615.95	615.93	(n)	(n)	(n)	(n)	(n)
OW-7	(b)	603.46	570.86	570.93	570.92	570.91	570.92	570.94	570.95	570.97	570.98	570.99	570.98	571.01	571.02	(n)	(n)	(n)	(n)	(n)
OW-8	(g)	599.82	598.18	600.70	600.41	600.85	602.35	601.92	602.22	599.65	602.79	602.43	602.55	602.45	601.33	7/11/06	598.18	1/11/07	602.79	4.61
OW-9	(d)	Dry	568.58	578.91	583.63	583.98	585.00	590.26	592.85	595.19	597.08	598.83	601.38	599.99	599.71	(n)	(n)	(n)	(n)	(n)
OW-10	(d)	Dry	587.17	595.35	597.00	597.16	598.29	599.50	600.49	599.84	601.29	602.09	600.46	601.21	600.59	(n)	(n)	(n)	(n)	(n)
OW-11	(g)	598.10	598.04	599.79	599.33	599.84	601.53	601.33	601.63	599.60	602.27	602.14	602.19	602.24	601.27	7/11/06	598.04	1/11/07	602.27	4.23
OW-12	(g)	599.70	598.32	600.27	600.07	600.43	601.91	601.47	601.80	599.36	602.39	602.15	602.15	602.10	600.96	7/11/06	598.32	1/11/07	602.39	4.07
SW-1		594.70	595.16	594.79	594.08	594.54	594.66	594.46	594.60	594.21	594.77	594.21	594.05	594.63	594.78	3/5/07	594.05	7/11/06	595.16	1.11
SW-2		594.62	595.07	593.69	593.97	594.49	594.70	594.37	594.58	594.17	594.70	594.09	593.95	594.55	594.66	8/31/06	593.69	7/11/06	595.07	1.38
SW-3		593.57	595.19	594.70	594.98	594.51	594.86	594.51	594.84	594.19	594.94	594.31	594.12	594.68	594.82	6/11//06	593.57	7/11/06	595.19	1.15
SW-4		594.47	595.18	594.65	594.92	594.54	594.81	594.51	594.68	593.98	594.94	594.11	594.06	594.57	594.81	9/21/06	593.94	7/11/06	595.18	1.24
SW-5		598.21	597.25	596.51	Dry	Dry	598.10	598.31	598.11	597.59	597.69	597.67	597.92	598.03	597.77	9/21/06	< 596.51 (k)	10/26/06	598.31	> 1.80 (k)
SW-6		600.54	600.30	600.34	600.31	600.49	600.62	600.60	600.44	600.39	600.64	600.69	600.39	600.64	600.62	7/11/06	600.30	2/1/07	600.69	0.39

Notes:

a) Dry, water in end cap only
b) Dry, water pooled in screen, no change observed
c) Dry, no water developed in well
d) Well exibited slow response during the monitoring period
e) Due to inconsistant availability of groundwater in the monitoring wells completed in the soil zone, soil zone groundwater potentiometric surface maps were not developed.
f) Groundwater elevation was consistant with another well in the cluster showing good response. Well not used for the potentiometric surface maps.
g) Due to proximity to MW-1217 cluster wells, observation wells OW-8, OW-11, and OW-12 water levels were not used for the groundwater potentiomentric surface maps.
h) MW-1203b water levels were taken following aqufer testing and were not fully recovered. Water levels were not used for the groundwater potentiomentric surface maps.
k) Water level in the construction holding pond was below the staff gauge base at 596.51 ft. (the surface elevation of the staff gauge base sediment).
m) Well used for evaluation of groundwater potentiometric surface. Basis for use provided in Table 2.4.12-205.
n) Not applicable due to Notes (a) through (d)

BLN COL 2.4-4

TABLE 2.4.12-205 (Sheet 1 of 3)
GROUNDWATER WELL RESPONSE

Well Number	Screened	Well Response	Used for Maps
MW-1201a	Soil	Dry, water in end cap only	No[(a)]
MW-1202a	Soil	Good response	No[(a)]
MW-1203a	Soil	Dry, water in end cap only	No[(a)]
MW-1204a	Soil	Slow response	No[(a)]
MW-1205a	Soil	Dry, water pooled in screen, no change observed	No[(a)]
MW-1207a	Soil	Dry, water pooled in screen, no change observed	No[(a)]
MW-1208a	Soil	Good response	No[(a)]
MW-1210a	Soil	Good response	No[(a)]
MW-1211a	Soil	Dry, water pooled in screen, no change observed	No[(a)]
MW-1212a	Soil	Good response	No[(a)]
MW-1214a	Soil	Good response	No[(a)]
MW-1215a	Soil	Dry, no water in well	No[(a)]
MW-1216a	Soil	Good response	No[(a)]
MW-1217a	Soil	Dry, water developed in wet months only	No[(a)]
OW-4	Soil	Slow response	No[(a)]
OW-5	Soil	Slow response	No[(a)]
OW-6	Soil	Slow response	No[(a)]
MW-1201b	Bedrock	Dry, water in end cap only	No
MW-1201c	Bedrock	Dry, water in end cap only	No
MW-1202c	Bedrock	Slow response	No
MW-1203b	Bedrock	Good response	**Yes**
MW-1203c	Bedrock	Dry, water pooled in screen, no change observed	No
MW-1204b	Bedrock	Dry, water in end cap only	No
MW-1204c	Bedrock	Good response	**Yes**
MW-1205b	Bedrock	Good response	No[(b)]
MW-1205c	Bedrock	Good response	**Yes**

BLN COL 2.4-4

TABLE 2.4.12-205 (Sheet 2 of 3)
GROUNDWATER WELL RESPONSE

Well Number	Screened	Well Response	Used for Maps
MW-1206b	Bedrock	Good response	No[(b)]
MW-1206c	Bedrock	Good response	**Yes**
MW-1207b	Bedrock	Slow response	No
MW-1207c	Bedrock	Good response	**Yes**
MW-1208b	Bedrock	Good response	**Yes**
MW-1208c	Bedrock	Good response	No[(b)]
MW-1209b	Bedrock	Good response	**Yes**
MW-1209c	Bedrock	Dry, water pooled in screen, no change observed	No
MW-1210b	Bedrock	Good response	**Yes**
MW-1210c	Bedrock	Dry, water pooled in screen, no change observed	No
MW-1211c	Bedrock	Good response	**Yes**
MW-1212b	Bedrock	Good response	**Yes**
MW-1212c	Bedrock	Good response	No[(b)]
MW-1213b	Bedrock	Good response	**Yes**
MW-1213c	Bedrock	Good response	No[(b)]
MW-1214b	Bedrock	Good response	**Yes**
MW-1214c	Bedrock	Good response	No[(b)]
MW-1215b	Bedrock	Good response	**Yes**
MW-1215c	Bedrock	Good response	No[(b)]
MW-1216c	Bedrock	Good response	**Yes**
MW-1217b	Bedrock	Good response	**Yes**
MW-1217c	Bedrock	Dry, water pooled in screen, no change observed	No
OW-1	Bedrock	Dry, water in end cap only	No
OW-2	Bedrock	Dry, water pooled in screen, no change observed	No
OW-3	Bedrock	Very slow response	No
OW-7	Bedrock	Dry, water pooled in screen, no change observed	No
OW-8	Bedrock	Good response	No[(c)]
OW-9	Bedrock	Very slow response	No

BLN COL 2.4-4

TABLE 2.4.12-205 (Sheet 3 of 3)
GROUNDWATER WELL RESPONSE

Well Number	Screened	Well Response	Used for Maps
OW-10	Bedrock	Very slow response	No
OW-11	Bedrock	Good response	No[(c)]
OW-12	Bedrock	Good response	No[(c)]

a) Due to inconsistent availability of groundwater in the monitoring wells completed in the soil zone, soil zone groundwater potentiometric surface maps were not developed.

b) Groundwater well elevation was consistent with another well in the cluster showing good response. Well not used for the groundwater potentiometric surface maps.

c) Due to proximity to MW-1217 cluster wells, observation wells OW-8, OW-11, and OW-12 water levels were not used for the groundwater potentiometric surface maps.

BLN COL 2.4-5

TABLE 2.4.12-206 (Sheet 1 of 2)
MONTHLY GROUNDWATER HYDRAULIC GRADIENT AND FLOW VELOCITY

Groundwater Velocity and Travel Time from BLN Unit 3 to Town Creek Embayment

Date	7/11/06	8/31/06	9/21/06	10/26/06	11/13/06	12/11/06	1/04/06	2/01/07	3/05/07	4/17/07	5/05/07
Elevation High (Eh)(ft.)	598.08	600.07	599.97	601.38	601.69	599.23	602.31	602.14	602.09	601.94	600.84
Elevation Low (El)(ft.)	595.18	594.65	593.94	594.51	594.68	593.98	594.94	594.11	594.06	594.57	594.81
Hydraulic Gradient ((Eh-El)/L)	1.81×10^{-3}	3.39×10^{-3}	3.77×10^{-3}	4.29×10^{-3}	4.38×10^{-3}	3.28×10^{-3}	4.61×10^{-3}	5.02×10^{-03}	5.02×10^{-3}	4.61×10^{-3}	3.77×10^{-3}
Velocity (V)(ft/day)	0.51	0.95	1.05	1.20	1.23	0.92	1.29	1.40	1.40	1.29	1.05
Travel Time (T)(yrs.)	8.63	4.62	4.15	3.64	3.57	4.77	3.40	3.12	3.12	3.40	4.15

Assumptions:

Hydraulic gradient is between MW-1217b (Eh) and SW-4 Town Creek embayment surface (El).
Pathway distance (L) = 1600 ft.
Hydraulic conductivity (K_h) = 3.95×10^{-3} cm/s
porosity (η) = 0.04.
Equation for velocity: $V = (K_h \times (E_H - E_L)/L)/\eta$ (Darcy equation for average linear velocity.)
Equations for travel time: T = L/V.
Conversions: 1 day = 86,400 sec.; 1 ft. = 30.48 cm; 1 year = 365.25 days.

BLN COL 2.4-5

TABLE 2.4.12-206 (Sheet 2 of 2)
MONTHLY GROUNDWATER HYDRAULIC GRADIENT AND FLOW VELOCITY

Groundwater Velocity and Travel Time from BLN Unit 4 to the Intake Structure Channel

	7/11/06	8/31/06	9/21/06	10/26/06	11/13/06	12/11/06	1/04/06	2/01/07	3/05/07	4/17/07	5/8/07
Elevation High (Eh)(ft.)	608.10	608.57	608.05	610.58	610.48	607.99	612.15	609.99	610.25	610.40	608.28
Elevation Low (El)(ft.)	595.07	593.69	593.97	594.37	594.58	594.17	594.70	594.09	593.97	594.55	594.66
Hydraulic Gradient ((Eh-El)/L)	5.01×10^{-3}	5.72×10^{-3}	5.42×10^{-3}	6.23×10^{-3}	6.12×10^{-3}	5.32×10^{-3}	6.71×10^{-3}	6.12×10^{-3}	6.27×10^{-3}	6.10×10^{-3}	5.24×10^{-3}
Velocity (V)(ft/day)	1.40	1.60	1.52	1.75	1.71	1.49	1.88	1.71	1.75	1.71	1.47
Travel Time (T)(yrs.)	5.07	4.44	4.70	4.08	4.16	4.78	3.79	4.16	4.06	4.17	4.85

Assumptions:

Hydraulic gradient is between MW-1204c (Eh) and SW-2 intake structure channel surface (El).
Pathway distance (L) = 2600 ft.
Hydraulic conductivity (K_h) = 3.95×10^{-3} cm/s
porosity (η) = 0.04.
Equation for velocity: $V = (K_h \times (E_H - E_L)/L)/\eta$ (Darcy equation for average linear velocity).
Equations for travel time: T = L/V.
Conversions: 1 day = 86,400 sec.; 1 ft. = 30.48 cm; 1 year = 365.25 days.

STD COL 15.7-1

TABLE 2.4.13-201 (Sheet 1 of 2)
AP1000 TANKS CONTAINING RADIOACTIVE LIQUID

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

STD COL 15.7-1

TABLE 2.4.13-201 (Sheet 2 of 2)
AP1000 TANKS CONTAINING RADIOACTIVE LIQUID

Tank	Location[a]	Nominal Tank Volume	Radioisotope Contents	Considerations / Features to Mitigate Release
WLS Monitor Tanks A, B, C	Auxiliary Building El. 66'-6" and 117'- 6"	15,000 gal	Effluent prepared for environmental discharge - much less than reactor coolant	Located in unlined room at lowest portion of Auxiliary Building
WLS Monitor Tanks D, E, F	Radwaste Building	15,000 gal	Effluent prepared for environmental discharge - much less than reactor coolant	Located in unlined room at grade level in curbed, non-seismic building
WLS Chemical Waste Tank	Auxiliary Building El. 66'-6"	8,900 gal	Less than reactor coolant	Located in unlined room at lowest portion of Auxiliary Building
WSS Spent Resin Storage Tanks	Auxiliary Building El. 100'	300 ft^3 (liquid volume will be much less)	Approximately reactor coolant	Located entirely inside Auxiliary Building; does not have any potential for foundation cracks to allow leakage directly to environment. Leakage would be to another room of Auxiliary Building.

a) Floor elevations are based on design plant grade of 100 ft as provided in the DCD.

BLN COL 2.4-5

TABLE 2.4.13-202
DISTRIBUTION COEFFICIENTS (K_d)

Isotope		B-1083/MW-1202a	B-1093/MW-1205a	B-1078/MW-1212a
Co^{60}	(cm^3/g)	9880 +/- 1041	9353 +/- 1160	10,836 +/- 1295
Cs^{137}	(cm^3/g)	7533 +/- 1065	6302 +/- 891	8377 +/- 1185
Fe^{55}	(cm^3/g)	4107 +/- 581	23,089 +/- 3265	3366 +/- 476
I^{129}	(cm^3/g)	1.6 +/- 0.2	24.8 +/- 3.7	11.7 +/- 1.7
Ni^{63}	(cm^3/g)	270 +/- 38	2858 +/- 378	1153 +/- 134
Pu^{239}	(cm^3/g)	> 1394	> 1504	> 2153
Sr^{90}	(cm^3/g)	126 +/- 17	104 +/- 14	92.5 +/- 12.5
Tc^{99}	(cm^3/g)	0.18 +/- 0.03	0.12 +/- 0.02	0.26 +/- 0.04
U^{235}	(cm^3/g)	289 +/- 41	117 +/- 16	47.1 +/- 6.7

BLN COL 15.7-1

TABLE 2.4.13-203 (Sheet 1 of 7)
LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Silver K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	The model default value is 0, which is the most conservative selection since it assumes no retardation during transport.
Barium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Bromine K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Cerium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Cobalt K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	10,837	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil
Chromium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Cesium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	8,377	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil

BLN COL 15.7-1

TABLE 2.4.13-203 (Sheet 2 of 7)
LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Iron K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	3,366	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil
Tritium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	The model default value is 0, which assumes no retardation during transport.
Iodine K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	11.7	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil
Lanthanum K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Manganese K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Molybdenum K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Niobium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	The model default value is 0, which is the most conservative selection since it assumes no retardation during transport.

TABLE 2.4.13-203 (Sheet 3 of 7)

BLN COL 15.7-1 LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Promethium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Rubidium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Rhodium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Ruthenium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	The model default value is 0, which is the most conservative selection since it assumes no retardation during transport.
Strontium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	92.5	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil
Technetium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0.26	A radionuclide-specific K_d value was measured by Argonne National Laboratory using BLN soil
Tellurium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	The model default value is 0, which assumes no retardation during transport.

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TABLE 2.4.13-203 (Sheet 4 of 7)
LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Yttrium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Zirconium K_d Coefficient (cm^3/g)	Radionuclide-specific retardation coefficient	0	A value of 0 was selected as most conservative since it assumes no retardation during transport.
Precipitation (meters per year)	Average quantity of precipitation per unit of area and per unit of time	7.200E-01	On-site data collected at BLN
Area of contaminated zone (square meters)	Area containing liquids released by the tank failure	5.298E+01	The contaminated soil area was assumed to be 2 meters in height with 0.45 porosity, thus an area of 7.279 square meters is required to contain 80% of the liquid effluent tank (22,400 gallons)
Runoff coefficient (unitless)	Coefficient (fraction) of precipitation that runoffs the surface and does not infiltration into the soil	8.400E-01	Site-specific value was determined to be 0.84

TABLE 2.4.13-203 (Sheet 5 of 7)
BLN COL 15.7-1 LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Contaminated zone total porosity (unitless)	Total porosity of the contaminated sample, which is the ratio of the soil pore volume to the total volume	4.500E-01	On-site data collected at BLN
Density of contaminated zone (g/cm^3)	Density of the contaminated soil impacted by the liquid tank failure	1.4416E+00	On-site data collected at BLN
Contaminated zone hydraulic conductivity (meters per year)	Flow velocity of groundwater through the contaminated zone under a hydraulic gradient	1.2465E+03	On-site data collected at BLN. The hydraulic conductivity value is based on the highest hydraulic gradient (i.e., fastest moving groundwater) measured at BLN.
Unsaturated zone soil density (g/cm^3)	Density of the unsaturated overburden soil	1.4416E+00	On-site data collected at BLN
Unsaturated zone hydraulic conductivity (meters per year)	Hydraulic conductivity that the unsaturated zone would have if saturated and subjected to a hydraulic gradient	1.2465E+03	On-site data collected at BLN

BLN COL 15.7-1

TABLE 2.4.13-203 (Sheet 6 of 7)
LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Density of saturated zone (g/cm^3)	Density of the saturated zone soil that transmits groundwater	1.4416E+00	On-site data collected at BLN
Saturated zone total porosity (unitless)	Total porosity of the saturated zone soil, which is the ratio of the pore volume to the total volume	4.00E-02	On-site data collected at BLN
Saturated zone effective porosity (unitless)	Ratio of the part of the pore volume where water can circulate to the total volume of a representative sample.	4.00E-02	The value is conservatively selected by have the effective porosity equal total porosity to achieve maximum groundwater movement
Saturated zone hydraulic gradient to surface water body (unitless)	Change in groundwater elevation per unit of distance in the direction of groundwater flow to a surface water body.	5.00E-03	On-site data collected at BLN. The value is conservatively selected as the highest hydraulic gradient measured at BLN

TABLE 2.4.13-203 (Sheet 7 of 7)
BLN COL 15.7-1 LISTING OF BLN DATA AND MODELING PARAMETERS SUPPORTING THE EFFLUENT HOLDUP TANK FAILURE

Soil Parameter	Parameter Description	Parameter Value[(a)]	Parameter Justification
Distance to the nearest surface water body (meters)	Distance to the nearest off-site surface water body that contributes to a potable drinking water source	362.23	The value is conservative selected by measuring the distance from the Unit 3 auxiliary building to the nearest point on Town Creek. The selection is conservative because this distance is the shortest distance from either the Unit 3 or Unit 4 auxiliary buildings to an off-site surface water body

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

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TABLE 2.4.13-204
RADIONUCLIDE CONCENTRATION AT NEAREST SURFACE WATER BODY THAT CONTRIBUTES TO A DRINKING WATER SOURCE IN AN UNRESTRICTED AREA DUE TO EFFLUENT HOLDUP TANK FAILURE

Detected Radionuclide	Radionuclide Concentration microcuries/ml	10 CFR 20 Appendix B Table 2 Column 2 microcuries/ml	Sum of Fractions Contribution[a]
Ag-110m	5.75E-10	6.00E-06	9.58E-05
Ce-144	2.53E-10	3.00E-06	8.43E-05
H-3	4.90E-05	1.00E-03	4.90E-02
Pr-144	2.53E-10	2.00E-05	1.27E-05
			Sum of Fraction Unity Rule Value
			4.92E-02

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

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL INFORMATION

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements

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STD DEP 1.1-1 Its Indicated in Section 2.0, this section is numbered to follow Regulatory Guide 1.206. The COL information items in DCD Subsections 2.5.1 through 2.5.6 are addressed in Subsection 2.5.6.

BLN COL 2.5-1 This section provides information on the geology, seismology, and geotechnical characteristics of the Bellefonte Units 3 and 4 (BLN) site. The section follows the standard format and content specifications of Regulatory Guide 1.206 (U.S. Nuclear Regulatory Commission, June 20, 2007).

A primary source of information for Section 2.5 is the Tennessee Valley Authority (TVA) report, Geotechnical, Geological and Seismological Evaluations for the Bellefonte Site (GG&S), prepared in 2006 by CH2MHill, Inc. in collaboration with Geomatrix Consultants (Reference 399). The GG&S report was prepared for the southern site located on the TVA Bellefonte property southwest of Units 1 and 2 (Figure 2.5-201). The southern site was later abandoned, but much of the geologic information presented in the report is thorough, current, and applicable to the present BLN site. Where appropriate, the text, tables and figures have been adopted directly from the GG&S report.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

This section describes information on the geological and seismological setting of the Bellefonte Nuclear Plant Units 3 and 4 (BLN) site and region. Regulatory Guide 1.208 describes the information and the level of investigation needed to confirm the suitability of a site for a nuclear facility. The guidance outlines four levels of investigation that generally increase in detail with proximity to the site. These include a radius of 200 miles, 25 miles, 5 miles, and 0.6 mile.

Several sources are used to develop the information summarized in this section. Bellefonte site-specific reports and documents including the Bellefonte Units 1 and 2 FSAR issued in June 1986 (Reference 201), a Final Environmental Impact Statement (FEIS) prepared in 1997 for the Bellefonte Conversion Project (Reference 202), and various construction documents were reviewed as part of the initial compilation effort. Extensive new data sets that have been compiled and interpreted for numerous site-specific and regional studies throughout the Central and Eastern United States (CEUS) in the time since completion of the Electric Power Research Institute - Seismicity Owners Group (EPRI-SOG) study in the late 1980s also were reviewed. These studies have used a variety of techniques to characterize the location, extent, and activity of tectonic features; the location, magnitude, and rates of seismic activity; and the general characteristics of the continental crust throughout the CEUS.

The information summarized in this section incorporates the information and findings of these studies as well as recent reports, maps, and articles published by state and federal agencies and in professional/academic journals. Additional unpublished data and information were obtained through communications with TVA personnel who are familiar with previous Units 1 and 2 studies, and individual researchers at university and state agencies. For the BLN investigation, additional site-specific documents were reviewed including historical topographic and geologic maps and construction photographs prepared for Bellefonte Units 1 and 2. Site-specific information developed during the BLN exploration program contributes to a large portion of the Site Geology (Subsection 2.5.1.2).

The information described in this section is organized in accordance with Regulatory Guide 1.206 and identifies any new information that could produce significant differences from the information used to develop the EPRI-SOG source model (Reference 203), which forms the starting point for the assessment of seismic hazard at sites in the CEUS (see discussion in Subsection 2.5.2). The EPRI-SOG study involved an extensive evaluation of the scientific knowledge concerning earthquake hazards in the CEUS by multi-discipline teams of experts in geology, seismology, geophysics, and earthquake ground motions. Regulatory Guide 1.208 specifies that the adequacy of the EPRI-SOG hazard results must be evaluated in light of more recent data and evolving knowledge pertaining to seismic hazard evaluation in the CEUS.

Subsection 2.5.1.1 describes the regional geologic and tectonic setting, focusing primarily on the 200-mi. radius. The EPRI-SOG seismic hazard analysis for the BLN site identified significant seismic sources at distances greater than 200 mi., particularly the New Madrid and Charleston seismic zones that were the source of large, geologically recent earthquakes. Recent information regarding the location, magnitude, and recurrence of these sources also is described in this subsection.

Subsection 2.5.1.2, Site Geology, describes the geology and structural setting of the 25-mi. radius, 5-mi. radius, and the 0.6-mi. radius. Site physiography and topography, geologic history, stratigraphy, lithology, structural geology, and engineering geology are discussed.

2.5.1.1 Regional Geology

This section describes the physiography, geologic history, and tectonic setting of the BLN site within a 200 mi. radius of the site. Topics reviewed include regional physiography, geomorphology, geologic history, stratigraphy, tectonics, structural geology, and seismology. A number of regional maps are presented, including physiography, geology, tectonics, paleogeography, terranes, structure, structure contour, magnetic and gravity anomalies, faults, and seismicity maps. A depth to basement map and a series of structural cross-sections provide information on thicknesses of Paleozoic strata. These materials present the same information as in an isopach map, therefore an isopach map is not included. In addition, relevant new information on potential seismic sources of significant more distant, large magnitude earthquakes in the New Madrid, Missouri, and Charleston, South Carolina, areas also is discussed.

Some of the information prescribed for this section in Regulatory Guide 1.206 instead is presented in other sections. Boring logs and aerial photographs are presented along with other site-specific information in Subsection 2.5.4. A pre-development aerial photograph Is shown in Figure 2.5-292. A map and summary of mineral and hydrocarbon extraction in the area are presented in Subsection 2.5.4.1.

2.5.1.1.1 Regional Physiography and Topography

The BLN site is located in the Browns Valley-Sequatchie Valley segment of the Cumberland Plateau section of the Appalachian Plateaus Province of the Appalachian Highlands Division (Reference 204). The area within a 200-mi. radius includes parts of five other physiographic provinces. These are: the Valley and Ridge Province; Blue Ridge Province; and Piedmont Province within the Appalachian Highlands to the east; the Interior Low Plateaus Province to the northwest; and the Coastal Plain Province to the southwest, south, and east (Figure 2.5-202). The following descriptions of the major physiographic provinces within the region is taken in part from the Units 1 and 2 FSAR (Reference 201), which relied extensively on descriptions provided by Fenneman (Reference 204) and Thornbury (Reference 205). A more detailed discussion of the Browns Valley-Sequatchie valley segment and physiography is provided in Subsection 2.5.1.2.1.

2.5.1.1.1.1 Appalachian Plateaus Physiographic Province

The Appalachian Plateaus Physiographic Province extends from northwestern New York to northwestern Alabama. From its maximum width of more than 200 mi., it begins to narrow in eastern Kentucky until it is barely 30-mi. wide in Tennessee. The width in Alabama is 50-mi. This province is essentially a broad syncline in rocks of Late Paleozoic age, bounded on all sides by escarpments that reflect the regional synclinal structure. The rock formations are nearly horizontal, a typical plateau structure, but the formations are so elevated and dissected that the landforms are in large part mountainous (Reference 206).

The Appalachian Plateaus Physiographic Province is divided into seven sections, three of which are within the 200-mi. radius boundary. Portions of the Kanawha, or unglaciated Allegheny Plateau, and the Cumberland Mountain sections are present in Kentucky and northern Tennessee. The Cumberland Plateau section, which includes the BLN site, is the southwestern most of the seven sections comprising the Appalachian Plateaus Province. In Tennessee and Alabama, the Cumberland Plateau section is generally underlain by the resistant Pottsville Formation of Pennsylvanian age, which consists of alternating beds of sandstone, siltstone, and shale with coal seams. Across the Tennessee River from the BLN site, the plateau is at an elevation of approximately 1400-ft. All elevations are referenced to NCVD 29 datum. Elevations increase to the north, and approach 2000-ft. near the Tennessee-Alabama border, and exceed 3000 ft. central and northern Tennessee, and 4000 ft. in Virginia.

The Cumberland Plateau section is bounded on the west by the Highland Rim section of the Interior Low Plateaus Physiographic Province. An escarpment that descends from 1500 to 1000-ft. elevation separates the Interior Low Plateaus and Appalachian Plateau physiographic provinces. This escarpment is approximately 1000-ft. high in Tennessee but gradually diminishes in height in Alabama south of Huntsville. The Cumberland Plateau section is bounded on the southwest by the Gulf Coastal Plain. Hills in the plateau may be capped by remnants of Coastal Plain sediment. On the east, the Cumberland Plateau section is bounded by the Valley and Ridge Physiographic Province. The eastern boundary of the Cumberland Plateau section is the Cumberland escarpment (described in Subsection 2.5.1.1.1.2), which marks the change from the broad open folds in the Cumberland Plateau to the close folding with marked faulting in the Valley and Ridge to the east. The straightness of the eastern Cumberland escarpment contrasts with the dissected character of the scarp on the west side of the Plateau.

2.5.1.1.1.2 Valley and Ridge Physiographic Province

The Valley and Ridge Physiographic Province to the east of the Appalachian Plateaus Physiographic Province extends for 1200-mi., from eastern New York to central Alabama. It ranges from 14 to 80-mi. in width, and is 40 to 50-mi. wide in Alabama and northwestern Georgia. The boundary between the Appalachian Plateaus and the Valley and Ridge is an abrupt topographic rise known as the Allegheny front in Pennsylvania and the Cumberland escarpment in Tennessee and Virginia. This escarpment is breached by the Pine Mountain thrust fault and the Sequatchie Valley fault and anticline, which are the westernmost of the Valley and Ridge thrust faults. Anticlinal valleys, anticlinal ridges, synclinal valleys, synclinal ridges, homoclinal valleys, and homoclinal ridges are six possible topographic expressions of the geologic structure commonly encountered in the Valley and Ridge Province (Reference 205). Folds are strongly compressed and the amount of faulting increases southward. This province is underlain by Paleozoic sedimentary formations totaling from (30,000 to 40,000-ft.) in thickness. Drainage in this province mainly shows a northeast-southwest flow. The physiographic boundary between the Valley and Ridge and the Blue Ridge coincides approximately with the northwestern limit of Precambrian basement rocks and late Precambrian rift-fill sedimentary and volcanic rocks in the hanging walls of Alleghanian thrust faults. In Alabama, ridges are generally approximately 1000-ft. in elevation and sometimes reach 1500-ft.; elevations in the northern part of the province sometimes exceed 4000-ft.

2.5.1.1.1.3 Blue Ridge Physiographic Province

The Blue Ridge Physiographic Province is bounded on the east by the Piedmont Physiographic Province and on the west by the Valley and Ridge Physiographic

Province. The Blue Ridge Physiographic Province is a deeply dissected mountainous area of numerous steep mountain ridges, intermontane basins, and trench valleys that intersect at all angles and give the area its rugged mountain character. The Blue Ridge contains the highest elevations and the most rugged topography in the Appalachian Mountain system of eastern North America. The North Carolina portion of the Blue Ridge, a part of which lies within the region, is about 200 mi. long and ranges from 25 to 15 to 55 mi. wide (Reference 207). Within North Carolina, 43 peaks exceed 6000 ft. in elevation and 82 peaks are between 5000 and 6000 ft.

The Blue Ridge is composed of complexly folded and faulted igneous (granitic) and metamorphic rocks. These rocks date to the Precambrian and Paleozoic and represent parts of the basement rock of the North American continent. Thomas (Reference 208) describes the Blue Ridge as an elongate external basement massif along which late Precambrian syn-rift sedimentary and volcanic rocks, as well as older basement rocks, have been translated and deformed by younger Appalachian compressional structures, especially large-scale Alleghanian (late Paleozoic) thrust faults.

2.5.1.1.1.4 Piedmont Physiographic Province

The Piedmont Province lies between the Coastal Plain and the Blue Ridge Mountains. The Piedmont is characterized by gently rolling, well-rounded hills and long, low ridges. Along the border between the Piedmont and the Coastal Plain, elevations range from 500 to 800-ft. To the west, elevations gradually rise to about 1700-ft. at the foot of the Blue Ridge. Most of the rocks in the Piedmont Province are gneiss and schist, with some marble and quartzite, and were derived by metamorphism of older sedimentary and volcanic rocks. Granite also is present. Some less intensively metamorphosed rocks, including considerable slate, occur along the eastern part of the province from southern Virginia to Georgia.

2.5.1.1.1.5 Interior Low Plateaus Physiographic Province

Northwest of the Appalachian Plateaus is the Interior Low Plateaus Province, which is about 300 by 300 mi. in size and covers most of central Tennessee and central Kentucky. Along its border with the Appalachian Plateaus Province (in Kentucky and Tennessee) is a west-facing escarpment that is underlain by sandstones of early Pennsylvanian age. Toward the center of this province are two large shallow basins called the Nashville Basin and the Lexington Plain. These two basins were formed by breaching and erosion of the Nashville and Jessamine Domes, respectively, along the Cincinnati Arch. From the Cincinnati Arch, the rocks dip gently toward the Appalachian Plateaus on the east and the Illinois Basin on the west (Reference 205). This province is underlain predominantly by Ordovician and Mississippian limestones on which a moderate karst topography is developed.

2.5.1.1.1.6 Coastal Plain Physiographic Province

South and southwest of the Appalachian Plateaus Physiographic Province is the East Gulf Coastal Plain Physiographic Province, which may be described largely in terms of its underlying rocks (Cretaceous and Eocene series). The inner boundary of the Coastal Plain Physiographic Province with the Appalachian Plateaus Physiographic Province is commonly called the Fall Line, but few rapids are produced where the central Alabama Coastal Plain rocks abut against Paleozoic sedimentary rocks (Reference 205). Geologically, the Fall Line represents the contact between the Cretaceous and younger sediments of the Coastal Plain and the older, crystalline rocks of the Piedmont. The Fall Line Hills is the belt of the East Gulf Coastal Plain that borders the Appalachian Plateaus Physiographic Province. Maximum elevations range from approximately 760 ft. in the western part to approximately 250 ft. along the southeastern boundary of the Fall Line Hills.

2.5.1.1.2 Regional Geologic History

The BLN site is located within the southern Appalachian orogen. Information from stratigraphic assemblages, known timing of the major faults, and times of plutonic intrusion and metamorphism have been used to reconstruct the plate tectonic history of the central and southern Appalachians. Details of these data sets and interpretations of the geologic history of the region are presented in a series of papers that describe the Appalachian-Ouachita orogen in the U.S. (Reference 209). Additional discussions of the tectonic framework and structural evolution of the Appalachian orogen in Alabama are outlined by Thomas (Reference 210). These papers and other recent publications as noted in the following text provide the basis for the following summary of the regional geologic framework.

The Appalachian orogen was built on the late Precambrian-early Paleozoic continental margin of North America. The Proterozoic Grenvillian crystalline rocks form the basement upon which many of the late Precambrian and younger stratigraphic packages that were ultimately involved in the Appalachian orogenies were deposited (Reference 211). Laurentian (Proterozoic North America) basement rocks underwent a granulite or at least an amphibolite metamorphism about 1.0 to 1.1 billion years before present (Ga) (Reference 212) during the Grenville orogeny.

Hatcher (References 211 and 213) describes the history of the orogen as a type example of one or more cycles of opening and closing of ocean basins. The process began following a period of crustal extension and rifting during the late Proterozoic that caused the separation of the North America and African plates and created the Iapetus (proto-Atlantic) Ocean. During rifting, the newly formed continental margin began to subside and an eastward thickening wedge of clastic sediments accumulated on the passive margin. Stratigraphic and sedimentologic analyses indicate that the Appalachian region subsequently experienced several compressional events: the Avalonian, Penobscotian, Taconic, Acadian, and Alleghanian orogenies (Figures 2.5-203 and 2.5-204). The processes of accretion of suspect and exotic terranes, together with terrane collision and ultimately continent-continent collision, resulted in construction of the Appalachian orogen. The major deformational events in the region are summarized as follows and are illustrated schematically in Figure 2.5-204. Additional details regarding the sequence of depositional and deformational events in northern Alabama are presented in Figure 2.5-205 and are summarized in Subsection 2.5.1.2.2.

The rifted margin of North America formed as the Iapetus and Theic-Rheic Oceans opened in the late Precambrian resulting in an irregular rift-transform margin in which basins of various depths developed (Figure 2.5-205). Lapetan rifting along the Blue Ridge dates from the interval 730 to 680 million years before present (Ma) (Reference 212). Ultimately this rifting led to the formation of oceanic crust and the opening of the Iapetus (proto-Atlantic) Ocean. The western rifted margin of Iapetus has been identified along the western side of the Appalachian orogen (Reference 214). The block-faulted basins to the west influenced the later configurations of thrusts that transported these deposits onto the North American craton. Deposition of the passive margin sequence followed breakup.

The Avalonian orogeny was a compressional episode in the late Proterozoic that produced calcalkaline plutonic rocks and a volcanic suite commonly described as an island arc. Late Proterozoic rifting during the Avalonian orogeny occurred between 650 and 570 Ma and was accompanied by deposition of non-marine to shallow marine sediments and volcanic deposits in grabens west of the Blue Ridge axis; while thick sequences of Precambrian turbidites and volcanics were deposited in listric fault-bounded basins on attenuated crust to the east (Reference 212). The Avalonian rocks in the southern Appalachians are found in the eastern part of the Piedmont Province and in the pre-Mesozoic basement beneath the Coastal Plain (Reference 212).

The early Cambrian-early Ordovician Penobscot orogeny represents the initial collision event in the Paleozoic that marks the beginning of the convergent phase in the closing of the Iapetus Ocean. Crustal convergence and accretion of micro-continents and intra-oceanic island-arc terrane that had developed in the proto-Atlantic ocean as a result of east-directed oceanic subduction and initial closing of the ocean basin occurred during this deformational episode.

The Taconic orogeny was a complex deformation episode that began earlier in the southern Appalachians than in the central and northern Appalachians (Reference 212). This early to middle Orodovician orogeny represents a major compressive episode caused by one stage of the closing of the Iapetus Ocean (Reference 212). The age of the Taconic event is estimated to be 450 to 480 Ma in the southern Appalachians (Reference 213). Uplift at a converging margin is indicated by the development of a prograding clastic wedge derived from orogenic uplift in Tennessee reentrant southwestward onto Alabama promontory (Figure 2.5-205). Hatcher (Reference 211) notes that a regional unconformity that developed on top of the Middle Ordovician carbonate bank may be the external vestige of the Taconic/Penobscot orogeny to the east, where thrust sheets loaded the continental margin and caused subsidence, forming a foredeep basin about 450

Ma in the southern Appalachians. Granitic plutons were formed during this deformational episode, probably above an eastward-dipping subduction zone, represented in remnant form by the Hayesville thrust sheet.

Later Acadian convergence and suturing of the Avalon (Carolina) terrane to North America in the Devonian Period to early Carboniferous Period produced another suite of granitic and mafic plutons and a large fault, the Central Piedmont suture, that welded the Carolina terrane to the ancient North American margin (Reference 211). This event resulted in a metamorphic event that spread across the Inner Piedmont and into the eastern Blue Ridge.

The Alleghanian orogeny occurred during the late Carboniferous Period and extended into the Permian Period. This orogeny is the most pervasive event to affect the central and southern Appalachians. This mountain building episode marks the collision of North America with Africa and represents the final convergent phase in the closing of the proto-Atlantic ocean. Alleghanian deformation and uplift of the southern and central Appalachians produced a large molasse deposit from Alabama to Pennsylvania, and folds and faults of the Valley and Ridge, and finally deformation of the molasse deposits of the Valley and Ridge and Cumberland-Allegheny Plateau (Reference 213). During this orogeny, the southern and central Appalachians were transported toward the North American craton as a huge composite crystalline thrust sheet-the Blue Ridge-Piedmont thrust sheet-that drove the foreland deformation in front of it (Reference 209). This collision resulted in a detachment of the ductile-brittle transition zone of the crust, propagating a thrust from the collision zone, which is probably under the Coastal Plain (Reference 211). The thrust sheet ramped into the rift-drift facies and platform sedimentary rocks along the leading edge and faults then propagated westward into the platform and early to late Paleozoic foreland sedimentary rocks (Reference 215). The master detachment in the frontal fold-thrust belt in the southern Appalachians is within the Lower Cambrian Rome Formation. Toward the west, faults propagated into higher detachments in the Ordovician, Mississippian, and Pennsylvanian rocks in parts of the southern Appalachians (Reference 215). The final phase of the Alleghanian deformation resulted in the development of dextral shear zones in the eastern Piedmont (Reference 216).

Crustal extension during early Mesozoic (late Triassic) time marked the opening of the Gulf of Mexico and Atlantic oceans. The early Mesozoic extensional episode gave rise to the Cenozoic Mid-Atlantic spreading center and development of the present passive trailing divergent continental margin along the Atlantic seaboard. This extensional period resulted in normal faulting and reactivation of structures and associated igneous activity within the Eastern continental margin south and east of the BLN site, but did not significantly affect the site (Reference 217).

Mesozoic and Cenozoic downwarping of the Gulf Coastal Plain has been imposed upon the Paleozoic structures of the Appalachian-Ouachita orogen (Reference 218). Post-Early Jurassic deposition along the present Atlantic Coast records transgression until late Cretaceous and possibly Paleocene time, followed by regression (complicated by smaller cycles of transgression) until the present (Reference 213). Tertiary regression is probably related to a change in the fundamental stress configuration in the crust along the continental margin, from dominant extension during the Mesozoic to compression related to ridge-push during the Tertiary to Recent (Reference 219). Within the study region, extensive areas of Cretaceous sediments were deposited in the Coastal Plain of Alabama, Tennessee, and Kentucky.

After the late Paleozoic, much of the eastern interior of the North American continent was above sea level, never to be inundated to the present time. There began a long episode of erosion, lasting into Cretaceous time in the Coastal Plains Province and up to the present throughout physiographic provinces of the Appalachian Highlands and Interior Plains Divisions (Figure 2.5-202). The present mountains result from Tertiary uplift and continued differential erosion of dissected Mesozoic and Tertiary surfaces as the crust readjusted isostatically to erosional unloading (Reference 219).

2.5.1.1.3 Regional Stratigraphy

Geologic formations within the 200 mi. radius are sedimentary rocks of Tertiary to Precambrian age and igneous and metamorphic rocks of Paleozoic to Precambrian age. A map showing the generalized stratigraphy within the 200 mi. radius is shown in Figure 2.5-206. A description of the general stratigraphy within the physiographic provinces in the region is provided as follows.

2.5.1.1.3.1 Appalachian Plateaus Physiographic Province

The Appalachian Plateaus Physiographic Province is underlain by Paleozoic sedimentary rocks (predominantly Mississippian and Pennsylvanian in age) that are nearly horizontal or gently folded. Rocks within this province generally are little deformed and have not been metamorphosed. Older rocks generally are exposed only in the crests of eroded anticlinal folds in the Cumberland Plateau section (e.g., the Sequatchie Valley and Big Wills Valley anticlines).

The following summary of the bedrock stratigraphy within this province is primarily from the Units 1 and 2 FSAR (Reference 201). A more detailed description of the stratigraphy at the site is provided in Section 2.5.1.2.3. A stratigraphic column in the Appalachian thrust belt in Alabama (including the Cumberland Plateaus and Valley and Ridge Provinces) is shown in Figure 2.5-207.

Sedimentary rocks from Permian to Cambrian in age are found within the Appalachian Plateaus Province. In Alabama, the Knox Group, the Chickamauga Formation, the Red Mountain Formation, the Bangor Limestone, and the Pottsville Formation comprise the majority of the bedrock in this province.

The Knox Group, which is 2500 to 3000-ft. thick, consists mostly of dolomite with some limestone and is late Cambrian to early Ordovician in age.

The Chickamauga Limestone of Middle Ordovician age is mainly alternating layers of limestone, siltstone, and shale, and is approximately 1400-ft. thick. As discussed in Subsection 2.5.1.2.3, the Chickamauga Limestone is subdivided into the Stones River Group, Nashville Group, and Sequatchie Formation, listed in order from older to younger.

The Red Mountain Formation of Silurian age is a shallow-marine clastic sequence that is composed of resistant sandstone, shale, and limestone. The formation is 200 ft. or more thick in northeastern Alabama. Overlying the Red Mountain Formation is a sequence of discontinuous variable shallow-marine facies and internal unconformities that includes the Devonian Frog Mountain Sandstone and Chattannooga Shale, and another resistant unit, the Mississippian Fort Payne Chert.

The Bangor Limestone of Mississippian age consists of thick-bedded, dark-bluish gray, crystalline and oolitic limestone. It ranges in thickness from about 100 to 700 ft.

The Pottsville Formation of Pennsylvanian age consists of alternating beds of sandstone, siltstone, and shale with coal seams. In Tennessee and Alabama, the entire width of the Cumberland Plateau is underlain by resistant Pottsville strata. In Alabama, the Pottsville Formation reaches a thickness of about 1200 ft.

Surficial deposits in the Appalachians including those found in the Cumberland Plateau section generally are only a few m thick, patchy, and difficult to date (Reference 220). Mills and Kaye (Reference 221) report occurrences of gravel on severely eroded remnants of high terraces that likely are of Quaternary age, but the age of these deposits is not well constrained. Inundation of many of the larger rivers by a system of large reservoirs (e.g., the Guntersville Reservoir) has obscured lower fluvial deposits and terrace surfaces. Many of the channels of the present drainages are eroded into bedrock. The Quaternary cover chiefly is composed of residual soils and soils modified from or derived from local parent material accumulated as alluvium and hillslope colluvium.

As noted in the Bellefonte Units 1 and 2 FSAR (Reference 201), the present course of the Tennessee River, the major drainage within the study region, includes a number of deflections that suggest adjustments by stream capture. Mills and Kaye (Reference 221) review previous hypotheses and present information on gravel locations that may provide constraints on possible former courses of the Tennessee River in the study region. The major course changes described in both the Units 1 and 2 FSAR (Reference 201) and Mills and Kaye (Reference 221) include: (1) west of Chattanooga, Tennessee, where the river leaves the Valley and Ridge Province and cuts through Walden Ridge; (2) near Guntersville, Alabama, where it leaves the southwestward-trending Sequatchie anticlinal valley and assumes a northwesterly course; and (3) near the juncture of the Alabama, Mississippi, and Tennessee borders, where it turns north to cross Tennessee and join the Ohio River. Mills and Kaye (Reference 221) cite studies that relate the northward diversion of the lower reaches of the river to crustal tilting caused by isostatic adjustment due to sea level change or crustal loading due to glaciation that could be as young as 1.13 Ma and other studies that suggest a minimum age of 5 to 6 Ma for the capture. They conclude, however, that there is not sufficient information on the distribution of Plio-Pleistocene deposits to decipher the detailed drainage history of the Tennessee River.

2.5.1.1.3.2 Valley and Ridge Physiographic Province

The Valley and Ridge Province is underlain primarily by Paleozoic sedimentary rocks that have been intensely folded and thrust faulted. The total thickness of Paleozoic sedimentary formations, which range in age from Cambrian to Permian, ranges from 30,000 to 40,000 ft. The Paleozoic section includes four major divisions: a basal, transgressive Cambrian clastic unit; a thick, extensive Cambrian-Ordovician carbonate-shelf facies; a thin, laterally variable shelf sequence of Ordovician to Lower Mississippian carbonate rocks, chert, and thin clastic units; and Upper Mississippian-Pennsylvanian synorogenic clastic-wedge rocks and Mississippian carbonate facies (Reference 218) (Figure 2.5-207).

The Quaternary record in the Valley and Ridge, like the rest of the Appalachians is thin, discontinuous, and difficult to date (Reference 220). Quaternary deposits include alluvial stream and fan deposits, and hillslope colluvium (Reference 220). Higher, older stream terraces are recognized and have been dated in this physiographic province to the north of the study region (e.g., the New River terraces in Virginia (Reference 222). Chapman (Reference 223) and Whisner et al. (Reference 293) provide descriptions of stream terrace deposits at varying locations within the Valley and Ridge Province. Along some rivers, the areal extent of old, highly weathered alluvium far exceeds the younger alluvium, suggesting that floodplains and low terraces were formerly more extensive than at present (Reference 220).

In northern Alabama, extensive alluvial terrace deposits are mapped in the Coosa River Valley in the Gadsden to Weiss Reservoir area (Etowah and Cherokee Counties) (Reference 224) (Figure 2.5-208). The alluvial and terrace deposits are preserved within a broad valley underlain by the Cambrian Conasauga Formation. Structural cross-sections and maps indicate that the Cambrian unit beneath the valley is a near horizontal thrust sheet, referred to as the Rome thrust (Reference 225) (see discussion in Subsection 2.5.1.1.4.2). The meandering river morphology is prominent where the widest part of the Rome thrust sheet is preserved. Downstream of the confluence of Big Canoe Creek and the Coosa River (about 10-mi. southwest of Gadsden), the valley narrows and the Coosa River takes a sharp bend to the south and cuts across the regional structural grain. Quaternary deposits are not shown on the State Geologic Map of Alabama (Reference 225) (Figures 2.5-208 and 2.5-209) downstream of this confluence. On the state Geologic Map of Alabama (Reference 226), the deposits are differentiated into alluvial and low terrace (Qalt) and high terrace (Qt) map units. The Qalt deposits are described as consisting of varicolored fine to coarse quartz sand containing clay lenses and gravel in places. Gravel is composed of quartz and chert pebbles and assorted metamorphic and igneous rock fragments in streams near the Piedmont. In areas of the Valley and Ridge Province, gravel is composed of angular to subrounded chert, quartz, and quartzite pebbles. The Qt deposits are described as varicolored lenticular beds of poorly sorted sand, ferruginous sand, silt, clay, and gravelly sand. Sand consists primarily of very fine to very coarse, poorly sorted quartz grains. The gravel is composed of quartz, quartzite, and chert pebbles.

Based on observations made during field reconnaissance investigations for this study, both the Qt and Qalt units appear to include multiple terrace surfaces. The highest surfaces as mapped between the town of Gadsden and the Weiss Reservoir range from elevations of about 600 to 670-ft. (based on contours shown on 1:100,000 and 1:250,000 scale maps). The most prominent surfaces appear to be at about 600-ft elevation, approximately 100-ft. above the present river level. Exposures of the high terrace gravels observed in and near Gadsden showed strong soil development with significant clay accumulation, strong mottling, and localized iron cementation. The age of these deposits is not known. Based on the regional denudation rate of 100-ft per million years (Reference 220) and the strong soil development, it is likely that the older terraces are on the order of hundreds of thousands to a million years old.

At one location in Gadsden (Field Stop KH9, Figure 2.5-208) subvertical features characterized by subvertical bands of alternating red and light brownish gray color were observed in the lower, more clay-rich, strongly mottled part of the soil. The features appeared to flare upward and become less distinct in the upper part of the mottled horizon and could not be traced into the upper 2 ft. of the soil. The contact between the mottled unit and gravelly sand does not show any apparent vertical displacement across the features. These features resemble non-tectonic soil weathering features (cutans) seen elsewhere in the Coastal Plain region of the southeastern U.S. (Reference 227).

2.5.1.1.3.3 Blue Ridge Physiographic Province

The Blue Ridge Physiographic Province consists of an allochthonous belt involving Precambrian (1.0 to 1.1 Ga) basement and younger rocks (Reference 228). The Blue Ridge is separated by a major fault system (Hayesville-Fries fault) into a western and an eastern block (Reference 228). The western block consists mainly of Grenville basement non-conformably overlain by Ocoee Series rocks, a cover sequence of Upper Proterozoic to Lower Cambrian sedimentary and rift-related rocks (Reference 215). The Ocoee basin was restricted to Tennessee. The Ocoee is conformably succeeded by the Chilhowee Group, a sequence of clean sandstones and shales that are more widespread than the Ocoee. These rocks overlap the basement along much of its extent and it is concluded to have been deposited in a post-rift environment (Reference 215). The eastern block consists of coeval metamorphosed turbidite sequences intercalated with mafic and ultramafic igneous rocks that are the same as those of the Inner Piedmont (Reference 229). Two small Grenville basement inliers, on the Tallulah Falls and Toxaway domes, also are present in the eastern Blue Ridge in the Carolinas and northeastern Georgia (Reference 215).

The Talladega belt in Alabama, which lies within the western block, was initially defined as a suspect terrane by Williams and Hatcher (References 230 and 231). It has since been shown to represent a more eastern (offshore) facies assemblage of the late Proterozoic to Devonian platform sequence that may have been deposited as fill in a strike-slip rhomb-graben basin near the North American shelf edge, and is, therefore, not an exotic terrane (Reference 215). The same rock assemblage may be present in the Murphy syncline farther northeast in southwest North Carolina.

The late Precambrian and Paleozoic metasedimentary and metavolcanic rocks become more intensely metamorphosed from west to east across the Blue Ridge Province, which separates platform rocks of the Valley and Ridge Province to the west from metavolcanic and metasedimentary rocks and intrusives of the Piedmont Province.

Surficial deposits in the unglaciated Blue Ridge Physiographic Province include alluvial stream terrace deposits, alluvial and debris-flow deposits, and hillslope colluvium (Reference 220). Relative-age mapping of alluvial fans in the Blue Ridge and adjacent Piedmont produces a map pattern of older and younger fan surfaces that has been used to infer the sequence of fan development (Reference 220). This mapping also shows that the relative abundance of young, intermediate, and old fan surfaces greatly vary from one area, suggesting episodic development. Comparison of data on fan surface heights and weathering rind thickness in two areas of the region suggest that downcutting and abandonment may take place at different rates. However, this conclusion is based on the assumption that the rate of weathering rind thickening on amphibolite is the same in the studied areas.

2.5.1.1.3.4 Piedmont Physiographic Province

In the Piedmont Province east of the Blue Ridge Province, the underlying rocks are mainly metamorphic (schists, gneisses, quartzites, and slates) and plutonic (granites, granodiroites, gabbros, peridotites, and dunites). The Piedmont Province is subdivided into a number of different zones based on differences in metamorphic grade and dominant lithology (Inner Piedmont, Charlotte belt, Carolina slate belt) (Reference 228). The Inner Piedmont is bounded by the Brevard zone to the west and the Central Piedmont suture to the east. Rocks of the Inner Piedmont consist of late Precambrian to early Paleozoic highly deformed sedimentary and mafic volcanic sequences regionally metamorphosed from upper greenschist to upper amphibolite facies. Small areas of Grenvillian rocks are exposed in windows through the thrust sheet (e.g., Pine Mountain) (Reference 231). The Charlotte and Carolina slate belts, to the east, are grouped as part of the late Precambrian-early Paleozoic Avalon terrane by Williams and Hatcher (Reference 230). Both belts contain a thick sequence of volcanic rocks and associated sedimentary rocks metamorphosed to greenschist grade in the slate belt and upper amphibolite grade in the Charlotte belt (Reference 228). The Kiokee belt is a belt of medium- to high-grade metamorphic and associated plutonic rocks between the Carolina slate belt on the northwest and the Belair belt on the southeast (Reference 215).

2.5.1.1.3.5 Interior Low Plateaus Province

In the Interior Low Plateaus Province to the west of the Appalachians Plateaus Province, the strata are relatively flat-lying and consist of sandstones, shales, and smaller amounts of limestones and dolomites, ranging in age from Ordovician to Cretaceous. The rock strata dip gently off the Jessamine and Nashville Domes, which developed along the axis of the Cincinnati Arch.

2.5.1.1.3.6 Coastal Plain Province

To the southwest lies the Coastal Plain Province, which is characterized by a sequence of Cretaceous and younger sediments over Paleozoic rocks. The Post-Paleozoic strata of the Atlantic and Gulf Coastal Plains are post-orogenic with respect to the Appalachian-Ouachita orogen and belong to two different tectonic regimes (Reference 232). The older Mesozoic rocks constitute fill of extensional fault-bounded basins and include sedimentary and volcanic components. These rocks of Triassic and early Jurassic age that are associated with rift-stage evolution of the present Atlantic and Gulf margin generally lie outside the 200 mi. radius, and are restricted to rift basins. Younger Mesozoic and Cenozoic strata are regionally continuous and represent shallow-marine onlap of the post-rift passive margin (Reference 232). The latter deposits are present within the 200-mi. radius (Figure 2.5-206).

2.5.1.1.4 Regional Tectonic Setting

The seismotectonic framework-the basic understanding of existing tectonic features and their relationship to the contemporary stress regime and seismicity-provides the basic underpinnings for assessments of seismic sources. In the EPRI-SOG study (References 233 and 203), seismic source models were developed based on the tectonic setting, the identification and characterization of "feature-specific" source zones, and the occurrence, rates, and distribution of historical seismicity. The EPRI models reflected the general state of knowledge of the geoscience community in the mid-1980s. The original seismic sources identified in the EPRI-SOG study are discussed in detail in the EPRI-SOG (Reference 203) report and are summarized in Subsection 2.5.2.2.1.

A second study conducted by Lawrence Livermore National Laboratory (LLNL) (Reference 234), which was a trial implementation project (TIP) of general guidance given in Senior Seismic Hazard Analysis Committee (SSHAC; Reference 235) for conducting a Level IV Probabilistic Seismic Hazard Assessment (PSHA), provided updated information for some of the seismic sources significant to the BLN site. A brief summary of the TIP study is provided in Subsection 2.5.2.2.2.

Subsequent to the EPRI-SOG and TIP studies, additional geological, seismological, and geophysical research has been completed at and near the BLN site. This section presents a summary of the current state of knowledge on the regional tectonic setting and highlights the more recent information that is relevant to the identification of seismic sources for the BLN site. The following sections describe the region in terms of the contemporary stress environment (Subsection 2.5.1.1.4.1), the primary tectonic features and seismic sources (Subsection 2.5.1.1.4.2), and significant seismic sources at distances greater than 200 mi. (Subsection 2.5.1.1.4.3). Historical seismicity is described in Subsections 2.5.1.1.2 and 2.5.1.2.2.

2.5.1.1.4.1 Contemporary Tectonic Stress

The BLN site lies within a compressive midplate stress province characterized by a relatively uniform compressive stress field with a maximum horizontal shear (SHmax) direction oriented northeast to east-northeast (NE to ENE) based on earthquake focal mechanisms, in situ stress measurements, borehole breakout data, and recent geologic features (References 236 and 237). Zoback and Zoback (Reference 236) note that although localized stresses may be important in places, the overall uniformity in the midplate stress pattern suggests a far-field source and that the orientation range coincides with both absolute plate motion and ridge push directions for North America. Richardson and Reding (Reference 238) also concluded, based on modeling of various tectonic processes using an elastic finite element analysis that distributed ridge forces are capable of accounting for the dominant ENE trend for maximum compression throughout much of the North American plate east of the Rocky Mountains.

In contrast to the stress domain map published by Zoback and Zoback (Reference 239), which was a primary reference used by the EPRI-SOG teams, the 1989 compilation shows general ENE compression extending to the Atlantic continental margin. Zoback and Zoback (Reference 236) concluded that a distinct Atlantic Coastal Plain stress province (characterized by northwest compression as inferred from the orientations of post-Cretaceous reverse faults in the Coastal Plain region and focal mechanisms in the northeastern U.S.) is not supported or justified by the available data.

Based on analysis of well-constrained focal mechanisms of North American midplate earthquakes, Zoback (Reference 240) concluded that earthquakes in the CEUS occur primarily on strike-slip faults dipping between 43° and 80°, with most in the 60° to 75° range. This analysis demonstrated that the CEUS earthquakes occur primarily in response to a strike-slip stress regime.

2.5.1.1.4.2 Regional Structures Within the 200-Mi. Radius

A tectonic map showing structures within a 200-mi. radius of the BLN site known at the time of the Units 1 and 2 licensing studies (Reference 201) and the EPRI-SOG study is shown in Figure 2.5-210. The concepts of suspect (allochthons) and exotic terranes, which were recognized at that time, have been more widely employed to decipher the accretionary history and tectonic evolution of the Appalachian orogen (see discussion in Subsection 2.5.1.1.2) and to define lithotectonic units (References 231, 211; 209; and 215). More recent tectonic maps and structural cross-sections at a regional scale for the Appalachian-Ouachita orogen and southern Appalachians are shown in Figures 2.5-211, 2.5-212, and 2.5-213, respectively. A map showing the major geologic and tectonic features and terrane boundaries of the southern Appalachians is shown in Figure 2.5-214.

Hatcher (Reference 219) defines lithotectonic subdivisions within the region. The westernmost lithotectonic province of the Appalachians as defined by Hatcher (Reference 219) is the Appalachian foreland, which includes the Cumberland-Allegheny Plateau and Valley and Ridge physiographic provinces. It is made up of two subdivisions: the Appalachian basin and a gently eastward-thickening miogeoclinal wedge of platform sedimentary rocks and syn-orogenic clastic wedges. The Appalachian basin may have formed as thrusts loaded the crust farther east, producing the basin and also the Cincinnati arch. Eastward, the Appalachian foreland fold-thrust belt in the region consists of a belt of Alleghanian imbricate thrusts and folds. East of the fold-thrust belt is the metamorphic core of the Appalachian orogen. Precambrian basement rocks, transported in external basement massifs, are present, along with continental margin or slope and rise sedimentary rocks in the western Blue Ridge. Farther east is the internal core of the Appalachians that includes the eastern Blue Ridge and Inner Piedmont physiographic provinces. Williams and Hatcher (References 230 and 231) refer to this belt as the Piedmont terrane. In more recent publications, it is shown as the Inner Piedmont belt (Figure 2.5-212). Metamorphic rocks of the northwestern part of the Inner Piedmont exhibit no Alleghanian deformation-except in the Brevard and Brookneal fault zones-but were translated northwestward on the Blue Ridge-Piedmont sole thrust and various splays (such as the Brevard fault) (Reference 211) (Figures 2.5-204 and 2.5-213). Metamorphism and plutonism accompanied Alleghanian faulting and penetrative deformation in the eastern Piedmont. The Carolina terrane (previously referred to as the Avalon terrane by Williams and Hatcher (References 230 and 231) includes the Charlotte and Carolina slate belts that are considered to be exotic or suspect terranes. These belts are interpreted to be island-arcs that were accreted to ancestral North America during the Acadian orogeny (Figure 2.5-204), but experienced regional metamorphism and presumably ductile deformation during the Taconic orogeny (Reference 241). The boundary structure between the Piedmont and Carolina terrane is referred to as the Central Piedmont suture (Reference 211). Along the eastern edge of the Piedmont in the Carolinas and Georgia is the Alleghanian Kiokee-Raleigh belt anticlinorium composed of middle to upper amphibolite-facies metamorphic rocks that contrasts with the older higher-grade rocks toward the west (Reference 219) (Figure 2.5-212).

During collision of North America with Africa during the Alleghanian orogeny, the southern and central Appalachians were transported toward the North America craton as a huge composite crystalline thrust sheet-the Blue Ridge-Piedmont thrust sheet-that drove the foreland deformation in front of it (Reference 215).

Evidence for the extent of the Alleghanian detachment beneath the Blue Ridge and Piedmont is derived from both geophysical and structural data. The acquisition and interpretation of the Consortium for Continental Reflection Profiling (COCORP) seismic reflection profiles across the southern Appalachians in the late 1970s to early 1980s (References 242 and 243) and subsequent interpretation of industry seismic data provided significant subsurface information to support a model for the development of the Appalachian thrust belt above a master décollement or detachment (Reference 219) and papers therein (Figure 2.5-213).

In central Alabama, the Paleozoic orogen plunges southwestward beneath postorogenic Mesozoic-Cenozoic strata of the Gulf Coast Plain (Figure 2.5-211). Data from oil wells drilled through the Coastal Plain sediment indicate that the orogenic belt curves westward through Mississippi and continues northwestward to the exposed Paleozoic structures in the Ouachita Mountains in Arkansas (Reference 210). The geometry and basin fill of the Black Warrior basin in the southwest part of the region (Figure 2.5-211) indicates a foreland basin related to the Ouachita fold-thrust belt rather than the Appalachian fold-thrust belt (Reference 218).

Comparison of Figure 2.5-210 to Figures 2.5-211 and 2.5-212 shows that the overall tectonic framework of the Appalachian region known at the time of the EPRI-SOG study has not changed with respect to the location of major mapped structural features. Additional information and analysis of subsurface data (e.g., industry seismic reflection profiles, deep wells) and seismicity data, however, provide an improved understanding of structures within the BLN 200-mi. radius, particularly with regard to the foreland Appalachian fold-thrust belt and possible relationships to subdetachment basement faults. The following sections: Section 'a' Appalachian Thrust Belt and Section 'b' Subdetachment Basement Faults describe these structures. More distant structures within the 200-mi. radius are described in Section 'c', Section 'd' presents a description of the characteristics of seismicity zones that may be associated with subdetachment faults within the Appalachian thrust belt region.

2.5.1.1.4.2.1 Appalachian Foreland Thrust Belt

The BLN site lies near the cratonward limit of the Appalachian detachment that underlies the Appalachian foreland thrust belt (Figures 2.5-212 and 2.5-213). The Appalachian Plateau (Cumberland Plateau Section), Valley and Ridge, and frontal part of the Blue Ridge physiographic provinces encompass the Appalachian foreland thrust belt (also referred to as the Alleghanian foreland thrust belt, the Appalachian fold-thrust belt, or Appalachian fold-and-thrust belt) and foreland basins (Reference 215). The southern Appalachian foreland thrust belt consists of a stack of mostly thin-skinned thrusts in an unconfined wedge configuration located above the Proterozoic basement and an eastern confined segment below the base of the Blue Ridge-Piedmont composite crystalline thrust (BRP) sheet that served as a rigid indenter that drove the foreland deformation (Reference 244).

In Alabama and Georgia, this thrust belt consists of late Paleozoic (Alleghanian), large-scale, northeast-striking, northwest-vergent thrust faults and associated folds bounded by undeformed strata in the Black Warrior foreland basin on the northwest and by the Talladega slate belt and Appalachian Piedmont on the southeast (Figure 2.5-215). The structural geometry and evolution of the thrust belt in Alabama and northeast Georgia is described by Thomas (Reference 245), Thomas and Bayona (Reference 225), and Bayona et al. (Reference 246). Using outcrop data from published geologic maps, detailed local mapping in key areas, and interpretation of seismic reflection profiles (contributed by sources in the petroleum industry), deep well data, and paleomagnetic data, they developed a series of strike-perpendicular balanced cross-sections and strike-parallel cross-sections (Figure 2.5-216). Representative strike-perpendicular cross-sections across the thrust belt in northern Alabama are shown in Figure 2.5-217. In these cross-sections, the Paleozoic strata are divided into four units: Unit 1, a basal weak unit (Lower and Middle Cambrian strata dominated by fine-grained clastic rocks, mostly Rome and Conasauga Formations); Unit 2, a regionally dominant stiff layer (Knox Group); Unit 3, a heterogeneous carbonate-siliciclastic Middle Ordovician-Lower Mississippian succession (this unit includes the Greensport-Sequatchie Formations); and Unit 4, Upper Mississippian-Pennsylvanian synorogenic foreland deposits. The regional detachment (décollement) is within the basal weak layer above Precambrian crystalline basement.

As illustrated in these cross-sections, the northwestern (frontal) part of the thrust belt is dominated by broad, flat-bottomed synclines and large-scale, northeast-trending asymmetric anticlines (Figure 2.5-217). The top of basement beneath the leading imbricate faults is shallow and flat, but it abruptly drops southeastward across basement faults into the Birmingham graben. The depth of the regional detachment as well as the amplitude of thrust ramps, increases abruptly southeast of the Big Canoe Valley fault and Peavine anticline, which are positioned over the down-to-the-southeast boundary fault system of the Birmingham graben. The two major structures closest to the BLN site, the Sequatchie Valley thrust and the Big Wills Valley thrust, are shallow imbricate faults with relatively small displacement compared to the structures to the southeast. The fold-and-thrust belt is bordered to the southeast by the large-scale, low-angle Talladega Front fault at the northwest boundary of Piedmont metamorphic rocks.

Surficial traces of the generally persistent strike-parallel structures in the overlying thrust sheet southeast of the frontal fault-related folds are interrupted by four distinct northwest-trending transverse zones (TZ), which are referred to from north to south as the Rising Fawn TZ, the Anniston TZ, the Harpersville TZ, and the Bessemer TZ (Reference 210) (Figure 2.5-215). Thomas (Reference 208), Thomas and Bayona (Reference 225), and Bayona et al. (Reference 246) describe the changes in deformation styles along-strike across the TZs. These include along-strike termination of structures, abrupt curve or offset in strike, abrupt change in plunge angle or direction, abrupt along-strike change in dip, abrupt along-strike changes in stratigraphic level of a thrust fault, and abrupt along-strike change in structural style. Thomas (Reference 210) notes that the cross-strike structural discontinuities that define the TZs are not lines, but rather are narrow bands (generally less than 12.5-mi. wide) that encompass observations across several different northeast-trending Appalachian structures. Bayona et al. (Reference 246) conclude that the along-strike changes in the thrust belt geometry are closely related to basement structural relief beneath the thrust belt. For example, tectonic thickening of graben-fill strata controls deformation southwest of the Anniston TZ (Gadsden mushwad in Sections D and E, Figure 2.5-217), whereas ramp and flat geometry is prevalent northeast of this TZ (Sections A, B, and C, Figure 2.5-216). Shallow, imbricate faults dominate the thrust belt in Georgia, where the top of basement dips gently to the southeast (Reference 246).

The culmination of the Alleghanian orogeny occurred in the late Paleozoic. There is no new information to suggest that the thrust faults within the Appalachian foreland thrust belt are capable tectonic structures as defined by Regulatory Guide 1.208 (Appendix A). Seismicity in the region occurs primarily within basement rocks below the regional detachment and first motion analyses indicate predominantly strike-slip focal mechanisms (see discussion in Subsection 2.5.1.1.4.2.4). Evidence for post-Cenozoic faulting or geomorphic evidence for Quaternary deformation in the region is not reported in the published literature (References 247 and 248).

2.5.1.1.4.2.2 Subdetachment Basement Faults

It was recognized at the time of the EPRI-SOG study that potential seismic sources may be present below the Appalachian detachment or décollement (Figure 2.5-213). Subsequent studies have focused on better defining the location and geometry of basement structures. Of significance in the southern Appalachian region are known or inferred large normal faults that originally formed along the passive margin of the late Proterozoic to early Paleozoic Iapetus Ocean. Compressional reactivation of favorably oriented Iapetan faults has been suggested as the causal mechanism for several seismically active regions in the southern Appalachians including Giles County, Virginia, and eastern Tennessee (References 249 and 214) (see discussion in Subsection 2.5.1.1.4.2.4). Bollinger and Wheeler (Reference 249) suggest that the steep eastward rise in the unfiltered Bouguer anomaly field is the eastern limit for the Iapetan normal faults and that most of the faults occur in the relatively intact continental crust of North America west of the gravity rise. This gravity rise, referred to as the Appalachian (Piedmont) gravity gradient, is interpreted to mark the transition from thick continental to less thick, and possibly more mafic (transitional), crust to the east (References 250, 251, and 252).

Based on published interpretations of deep seismic reflection profiles across parts of the Appalachians and the Coastal Plain, Wheeler (Reference 253) infers the southeastern boundary of preserved Iapetan faults to coincide with a narrow zone of intense thinning (ZIT) of Grenville crust that extends along the Appalachians coincident with the Appalachian gravity gradient. Wheeler (Reference 254) notes that reflection profiles within the ZIT and farther southeast show structures that disrupted or destroyed the Grenville crust and any Iapetan faults within it during Paleozoic compressional and Mesozoic extensional deformation. Bollinger and Wheeler (Reference 249) note that Iapetan normal faults likely decrease in size, abundance, and slip gradually and irregularly northwestward into the North American craton over a distance of perhaps 60 to 125 mi. The northwest boundary to Iapetan normal faults is based on the northwesternmost locations of known Iapetan faults, both seismic and currently aseismic (Reference 214). This boundary coincides approximately with the northwestward transition from a more seismically active continental rim to a generally less active cratonic interior (Figure 2.5-218).

Hatcher and Lemiszki (Reference 255), and Hatcher et al. (Reference 244) present a regional structure contour map on the basement surface beneath the Valley and Ridge and Blue Ridge and Piedmont of Alabama, Georgia, Tennessee, the Carolinas, and southwest Virginia (Figure 2.5-219). The basement surface is inferred from industry, academic, and U.S./state geological survey seismic reflection and surface geologic data, along with crustal seismic lines in the more internal parts of the orogen. The basement surface in this reconstruction dips gently southeast in the Tennessee embayment from Virginia to Georgia and contains several previously unrecognized rift-related large and small displacement Neoproterozoic-earliest Cambrian normal faults (Reference 244).

In Alabama and Georgia, more recent interpretation and analysis of industry data has provided a more detailed picture of the top of basement surface and sub-detachment basement faults (References 225, 246, and 256). The general depth to basement and the major basement structures that are interpreted from these data are shown in Figure 2.5-220. The figure shows that (1) along- and across-strike changes in thrust belt geometry are closely related to basement structural relief beneath the thrust belt; and (2) along-strike changes of the structural configuration of the top of basement are concentrated at northwest-striking basement faults, which offset northeast-striking basement faults (Reference 246). The northwest-striking basement fault separates domains of contrasting structural profiles of basement fault systems, differing elevation of top of basement, and differing thicknesses of the regional décollement-host weak layer in the lower part of the sedimentary succession above basement rocks (Reference 225).

Bayona et al. (Reference 246) interpret the westernmost basement fault shown on Figure 2.5-220 to lie witin a few miles of the BLN site. The mapped position of this fault is interpreted from seismic reflection data located approximately 33 mi. to the southwest and 23 mi. to the northeast of the BLN site (blue dots). The presence and exact location of this fault has not been confirmed with seismic or borehole data near the site. There are no seismicity alignments or surface geologic evidence to indicate that these faults have been reactivated in the current tectonic stress field.

2.5.1.1.4.2.3 Other Major Structures Within the 200-Mi. Radius

The major mapped faults and tectonic structures within the 200-mi. radius represent deformation that occurred most recently in the Paleozoic. Except for minor faults reported in Miocene deposits in Tennessee (Reference 293), see discussion in Subsection 2.5.1.1.4.2.4.2. Eastern Tennessee Seismic Zone), there is no reported evidence to indicate that any of these tectonic structures displace or deform late Cenozoic deposits or exhibit evidence for Quaternary deformation. Powell (in Reference 248) describes evidence for Cretaceous faulting and Cenozoic tectonism in the Appalachians of the eastern U.S. Cretaceous and younger faults are recognized within the Coastal Plain, Piedmont, Blue Ridge, and Valley and Ridge physiographic provinces, but none lie within the 200-mi. radius of the BLN site. The Paleozoic structures in the region, therefore, are not considered to be capable tectonic sources, as defined in Regulatory Guide 1.208, Appendix A. A description of the principal structures in each of the physiographic provinces, except the structures in the Cumberland Plateaus, Valley, and Ridge Provinces that are described in detail in Subsection 2.5.1.1.2, is provided as follows.

The Plateaus of the southern Appalachians contain a few large structures. The largest of these are the Pine Mountain thrust sheet and Sequatchie anticline-Cumberland Plateau overthrust (Figure 2.5-207). Important other smaller Plateau structures are the Lookout Valley (Peavine), Murphree Valley, and Wills Valley anticlines (Figure 2.5-213). These structures, which lie within the northwestern (frontal) part of the Appalachian fold-thrust belt (see discussion in Subsection 2.5.1.1.2), are explained in terms of a connected system of ramps and flats (Reference 257). Faults and folds are connected, in that steps in basal detachments give rise to ramp anticlines. In the Plateaus, these large-scale, northeast-trending asymmetric anticlines are separated by broad, flat-bottomed synclines (References 215 and 246). The Sequatchie and Big Wills Valley anticlines and associated faults, which lie within the 25-mi. radius are described in more detail in Subsection 2.5.1.2.4.

Three broad areas of the southern Appalachian Plateau are virtually undeformed. As described by Wiltschko (Reference 215) these are: (1) the broad region southeast of the Sequatchie anticline but northwest of the Lookout Valley, Wills Valley, and Murphrees Valley anticlines that is macroscoptically undeformed at the surface except for joints, although it is underlain by the basal detachment in the Rome Formation; (2) the area between the Pine Mountain thrust sheet and the Sequatchie anticline, which also is likely allochthonous, but is essentially undeformed except for jointing; and (3) the region northeast of the Pine Mountain thrust sheet, which exhibits minimal folding and no faulting.

The Blue Ridge Province is allochtonous; estimates of translation of the southern Appalachian Blue Ridge range from 156 to 175 mi. (References 242 and 258, respectively), placing the external massif onto platform or platform margin sediments. The Blue Ridge is differentiated from the Valley and Ridge on the basis of the appearance of Cambrian and Precambrian rocks in the thrust sheets, metamorphism southeast of the frontal thrust zone, and increased complexity of deformation (Reference 259). Blue Ridge rocks are mostly metamorphosed with grade increasing toward the southeast, and have been affected by more than one orogeny. The Blue Ridge may be divided into three subregions on the basis of the nature of exposed lithologies and bounding faults (Reference 229): (1) a western subregion of imbricate thrusts involving unmetamorphosed to low-grade rocks and some basement, transitional in structural style and degree of deformation to the Valley and Ridge Province on the west; (2) a central subregion containing most of the basement rocks, as well as metamorphic rocks of higher grade to the west; and (3) an eastern subregion bounded on the west by the Hayesville and Fries faults and involving medium- to high-grade metasedimentary and metavolcanic rocks (Reference 215).

In the southern Appalachians, the boundary between the Valley and Ridge and the Blue Ridge is a single fault or zone of faults. Westward-directed thrust faults of large displacement characterize the southern part of the Blue Ridge. There is considerable internal folding (e.g., Murphy syncline, Toxaway, Ela, and Bryson City domes) in addition to the thrusting. Much of the metamorphism, folding, and some of the faulting (Greenbrier, Allatoona-Hayesville faults, Shope, and others) clearly predate the Alleghanian orogeny (Reference 259). Several of the structures formed during earlier orogenies have been reactivated to various degrees during the Alleghanian orogeny. The Alleghanian BRP thrust sheet is bounded on the west by the Blue Ridge fault system, which comprises the Talladega (Alabama), Cartersville (Georgia), Great Smoky (northern Georgia-southern Tennessee), and Holston Mountain (northeastern Tennessee), and Blue Ridge (Virginia) faults (Reference 259).

The time of last motion of the Alleghenian faults of the Blue Ridge is younger than Mississippian, the youngest rock cut by any frontal Blue Ridge fault. Most of the internal deformation within the Blue Ridge attributed to the Alleghanian orogeny is brittle in nature; the thermal peak occurred earlier during the Taconic (Ordovician) orogeny (Reference 215).

Major structures within the Piedmont Physiographic Province include the Brevard fault, the Central Piedmont suture, and the Towaliga, Ocmulgee, and Modoc faults (Figure 2.5-212).

The Brevard fault defines the western boundary of the Piedmont Physiographic Province. The Brevard fault, a complex fault zone having a possible earlier history of dip-slip motion and an Alleghanian history of both dip- and strike-slip motion, is a major Alleghanian structure within the BRP thrust sheet (Reference 215). A summary of earlier studies and models for the origin and structure of the Brevard fault zone is provided by Wiltschko (Reference 215).

The boundary structure between the Piedmont and Carolina terrane, which is recognized in potential field data as well as surface geology, is referred to as the Central Piedmont suture (Reference 211). It is interpreted to have formed when the Carolina volcanic arc terrane was joined to North America during either the

Taconic or Acadian orogenies, and was tightly folded during either the same or a later event, then suitably oriented segments were reactivated by dextral slip (i.e., the Towaliga, the Ocmulgee, and the Goat Rock fault zones) during the Alleghanian orogeny (Reference 213).

Along the eastern edge of the Piedmont in the Carolinas and Georgia is the Alleghanian Kiokee-Raleigh belt anticlinorium composed of middle to upper amphibolite-facies metamorphic rocks that contrasts with the older higher-grade rocks toward the west (Reference 219). This belt is interpreted to be a micro-continent that was accreted to ancestral North America during the Taconic orogeny. The western boundary of the Kiokee belt is the Modoc zone, an east-northeast-trending plastic shear zone. The southeast-dipping, east-northeast-trending Augusta fault borders the southeast flank of the Kiokee belt. Alleghanian dextral strike-slip has been documented on the Modoc and the Augusta faults in the eastern Piedmont (Reference 215).

Faults within the Interior Low Plateaus Physiographic Province within the northwestern part of the 200 -mi. radius are part of the Rough Creek graben. Faults associated with the Rough Creek graben show strong evidence for initiation during Cambrian Iapetan phase rifting and reactivation during the mid-late Paleozoic Appalachian-Ouachita orogeny (References 260 and 261) (Figure 2.5-205). Mesozoic activity on the Rough Creek graben faults also is suggested by post-Permian displacements and regional correlation of extensional deformation associated with post-Permian to pre-Cretaceous rifting of the Pangea continental landmass (Reference 260). However, a lack of Mesozoic sediments in the Rough Creek graben and restriction of evidence for post-Permian deformation to the western portion of the Rough Creek graben, and a complex but moderately well-defined structural boundary limiting Mesozoic deformation to the west in the Fluorspar area are cited by Wheeler (Reference 262) as a paucity of evidence for Mesozoic reactivation of the Rough Creek graben.

Late Paleozoic orogenic structures exposed in the Appalachian Mountains of Alabama and the Ouachita Mountains of Arkansas extend beneath a cover of post-orogenic Mesozoic-Cenozoic strata in the Gulf Coastal Plain (Figure 2.5-211). In the eastern part of the region, post-Paleozoic erosion surface dips eastward beneath an eastward-thickening prism of post-orogenic Mesozoic and Cenozoic strata, and the strike of the post-Paleozoic surface approximately parallels the Appalachian strike (Reference 232). To the south, the overlap limit as well as the strike of the post-Paleozoic surface, curves west and northwest and crosses the Appalachian strike at a large angle. The post-Paleozoic surface dips generally toward the Gulf of Mexico beneath a thickening prism of Mesozoic and Cenozoic strata. In the Mississippian embayment of the Gulf Coastal Plain, Mesozoic-Cenozoic stata extend entirely across the Paleozoic Appalachian-Ouachita orogenic belt and cover Paleozoic rocks in the Black Warrior basin (Figure 2.5-206).

The Paleozoic Black Warrior basin is defined by a homocline dipping away from the craton and extending beneath the cratonward-directed frontal structures of the

Appalachian-Ouachita fold-thrust belt (Reference 218). The Black Warrior basin is bordered on the cratonward (north) side by the Nashville dome. A northwest-trending system of normal faults displaces the homocline down-to-southwest (Reference 218). On the southeast, the fault system intersects the front of the Appalachian fold-thrust belt at approximately 90°, and lateral ramps in some thrusts apparently are related genetically to the intersecting faults (Reference 218). The youngest rocks preserved in the Black Warrior basin (early Middle Pennsylvanian) are displaced by the faults.

2.5.1.1.4.2.4 Seismicity Zones within the 200-Mi. Radius

Higher rates of low- to moderate-magnitude earthquakes that are recognized in two regions of the Valley and Ridge Province of the southern Appalachians are referred to as the Giles County, Virginia, and the East Tennessee seismic zones (ETSZ). These two seismic zones, the Giles County, Virginia, and ETSZ were identified by several of the EPRI-SOG evaluation teams as distinct seismic source zones. Detailed studies of seismicity and potential field data that have been conducted since completion of the EPRI-SOG study provide new information regarding the characterization of these zones.

2.5.1.1.4.2.4.1 Giles County, Virginia, Seismic Zone (GCVSZ)

Earthquake foci at Giles County in southwestern Virginia define a tabular zone that strikes N44°E and dips steeply to the southeast within Precambrian basement beneath Appalachian thrust sheets (References 249 and 263). This zone, referred to as the Giles County, Virginia, seismic zone (GCVSZ), is about 25-mi. long, 6-mi. wide, and from 3 to 16-mi. deep (References 263 and 264). The zone is oriented at an angle of about 20° counter-clockwise to the east-northeasterly trend of the overlying, detached southern Appalachian structures (Valley and Ridge Province) and subparallel to the northeasterly trend of the central Appalachian structures in the northern part of the state (Reference 264). The largest known earthquake in the state, the 1897 Giles County earthquake (MMI = VIII, mb = 5.7), occurred within this zone near the Virginia-West Virginia border (Reference 265). This event has been reassessed as an m_b = 5 in the National Center for Earthquake Engineering Research (NCEER)-91 (Reference 330) and the U.S. Geological Survey (USGS) (Reference 331) earthquake catalogs. EPRI (Reference 268) concluded that the moment magnitude was **(M)** 5.9; this estimate was selected as the magnitude for the final catalog for the TVA dam safety seismic hazard assessment study (Reference 269) and this study.

The seismic energy from the GCVSZ appears to be released by predominantly strike-slip faulting that lies below the Appalachian detachment. Focal mechanisms of recent earthquakes exhibit mainly strike-slip motions on steeply dipping (>70°) planes that are right-lateral on the northerly striking nodal planes or left-lateral on the easterly striking nodal planes. The P-axes (maximum compressive stress axes) estimates are uniformly of a northeasterly (NNE to ENE) trend with subhorizontal inclination and are similar to the orientation of P-axes estimates elsewhere in the region. Based on an evaluation of the late Proterozoic and

Phanerozoic structural history of the surrounding region, (Reference 248) concluded that only Iapetan rifting could have produced a fault with an orientation and depth like those of the tabular zone of foci. From seismic reflection profile data, Gresko (Reference 270) interpreted a series of down-on-the-east, subdetachment faults, consistent with this hypothesis. Bollinger et al. (Reference 264) also state that it is likely that the release of seismic energy within the GCVSZ is the result of reactivation of one or more faults formed initially by extensional stresses during Precambrian time.

No capable tectonic sources have been identified within the GCVSZ, but evidence for possible differential uplift of Quaternary terraces near Pearlsburg (Reference 271) and a zone of small late Pliocene to early Quaternary age faults have been identified in southwestern Virginia in the area of the GCVSZ, near Pembroke (References 247, 272, 273, and 274).

These high-angle faults (the Pembroke faults) and a broad antiformal fold exposed in apparently young unconsolidated fluvial deposits have raised questions regarding the possibility of geologically recent tectonic faulting that may be related to seismic activity in this region (References 272, 275, and 276). The deformation is of latest Pliocene or Quaternary age based on the age of the deformed sediments that have been dated using cosmogenic Al-26 and Be-10 analysis (Reference 273). Law et al. (Reference 277) present three models to explain the formation of the fold and fault structures at this site: landsliding, solution collapse, and basement faulting of tectonic origin. Although some researchers have noted that the correlation between surface faults and sub-detachment seismogenic structures may be tenuous or completely lacking (References 273, 274, and 278) have concluded that a tectonic origin cannot be precluded based on the available data and interpretations. Crone and Wheeler (Reference 247) rate the faults as Class B[a] because it has not yet been determined whether the faults are tectonic or the result of solution collapse.

More recent geophysical and subsurface investigations of these structures (References 279, 280, and 281) provide additional constraints on the origin of the fold and faults. Robinson et al. (Reference 279) show that voids occur in the terrace sediments that may result from cavity collapse in the underlying limestone, and that no features occur in the limestone basement that correspond to the fold and graben structure in the terrace deposits. Williams et al. (Reference 280) map a linear depression in the limestone bedrock surface that corresponds to the graben in the terrace deposits, and they note that the fold and graben structure

a. Crone and Wheeler (Reference 247) define Class A features as those where geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin; Class B features as those where the fault may not extend deeply enough to be a potential source of significant earthquakes, or the currently available geologic evidence is not definitive to assign the feature to Class C or to Class A; and Class C features are those where geologic evidence is insufficient to demonstrate the existence of tectonic fault, or Quaternary slip, or deformation associated with the feature.

has a linear nature that is not consistent with formation due to a subcircular sinkhole. Law et al. (Reference 281) show that the nature of fine structure in some of the terrace deposits is consistent with sedimentation in a depression formed by limestone solution, followed by inversion to form the anticlinal structure. These observations appear to indicate that some or all of the observed deformation is non-tectonic (probably related to solution collapse) in origin. Surficial mapping by Anderson and Spotila (Reference 282) of fractures in bedrock outcrops shows that the orientation of many small fractures is not consistent with topography or with karst-related subsidence. They note that one set of northeast-trending fractures cross-cuts the regional structural trend, is oriented consistent with the trend of the underlying seismic zone, and may be a surface manifestation of rupture in the seismic zone. However, this field evidence does not provide any direct evidence for Quaternary displacement on these fractures. Therefore, definitive evidence for a capable tectonic source and for the recurrence of large earthquakes similar or larger than the 1897 Giles County earthquake is lacking.

2.5.1.1.4.2.4.2 Eastern Tennessee Seismic Zone (ETSZ)

The Eastern Tennessee seismic zone (ETSZ) is a well defined, northeasterly trending belt of seismicity, 187-mi. long by less than 60-mi. wide, within the Valley and Ridge and Blue Ridge physiographic provinces of eastern Tennessee and parts of North Carolina, Georgia, and Alabama (References 283, 284, 285, and 286). This area, which lies within the 200-mi. radius, is one of the most active seismic regions in the eastern United States. The largest recorded earthquakes in this zone are the 1973 M 4.6 Maryville, Tennessee, earthquake (mb 4.6) (References 216 and 264) and the recent April 2003 M 4.6 Fort Payne earthquake that occurred in northeast Alabama near the Georgia border (see discussion in Subsection 2.5.1.2). Focal depths of most earthquakes range from 3 to 14 mi., beneath detached Alleghanian thrust sheets (References 286 and 287). Focal mechanisms indicate strike-slip faulting on steeply dipping planes and a uniform regional stress field with horizontal maximum compression trending N70°E (Reference 286). Most mechanisms involve either right-lateral motion on north-south planes or left-lateral slip on east-west planes (Reference 288); Chapman et al., (Reference 286) also note that a smaller population shows right-lateral motion on northeasterly trending planes, parallel to the overall trend of the seismicity. They note that the seismicity is not uniformly distributed; rather epicenters form northeasterly trending en-echelon segments.

The earthquakes are associated with major potential field anomalies References 264, 283, 285, 286, 287, and 289) (Figure 2.5-221). The western margin of the ETSZ is associated with a prominent gradient in the total intensity magnetic field, the New York-Alabama (NY-AL) geophysical lineament (Reference 286). Alternative structural models have been postulated to explain the association of seismicity with these anomalies. Powell et al. (Reference 285) proposed that the ETSZ is an evolving seismic zone in which slip on north- and east-striking surfaces is slowly coalescing into a northeast-trending zone. They suggested that the ETSZ represents seismic activity that results from the regional stress field and is coalescing near the juncture between a relatively weak, seismogenic block (referred to as the Ocoee block by Johnston et al. (Reference 283) and the relatively strong crust to the northwest that may be strengthened by the presence of mafic rocks associated with an inferred Keweenawan-age (1100 million years old) rift (Reference 290). They note that the densest seismicity and the largest of the instrumentally located epicenters in the ETSZ generally lie close to and east of the NY-AL aeromagnetic lineament between latitudes 34.3°N and 36.5°N and west of the Clingman aeromagnetic lineament. They postulated that deformation within the ETSZ may evolve eventually into a thoroughgoing, strike-slip fault running along or near the entire northwest boundary of the Ocoee block in eastern Tennessee. Strike-slip motion would be consistent with both the sharp, apparently near-vertical nature of the boundary, as inferred from the aeromagnetic signature, and the orientation of the boundary in the contemporary stress field.

Based on detailed analyses of the pattern and focal mechanisms of earthquakes in the ETSZ, Chapman (Reference 291) and Chapman et al. (Reference 288) present a more refined picture of the nature of faulting in the region. Using a revised velocity structure model (Reference 292), focal mechanisms and hypocentral locations were updated. Statistical analysis of trends in the earthquake focal mechanisms suggests that earthquakes occur primarily by left-lateral strike-slip on east-west-trending faults and to a lesser degree by right-lateral slip on north- and northeast-trending faults. The hypocenters suggest possible east-west-trending fault sources are up to 30 to 60-mi. long and lie east of and adjacent to the NY-AL lineament. The analyses are consistent with a tectonic model in which seismogenic faulting is localized along a sharp contrast in crustal strength (competency) represented by the NY-AL lineament (Figures 2.5-221 and 2.5-222).

An alternative model to explain the localization of seismicity in the eastern Tennessee region is given by Long and Kaufmann (Reference 284). Based on an analysis of the velocity structure of the region, they conclude that the seismically active areas are not apparently constrained by the crustal blocks as defined by the NY-AL lineament, but rather their locations are determined by low-velocity regions at mid-crustal depths. They suggest that the data support the conjecture that intraplate earthquakes occur in crust that may be weakened by the presence of anomalously high fluid pressures. Their data suggest that only a portion of the NY-AL lineament is consistent with the contact between two crustal blocks having different properties.

Chapman et al. (Reference 286) conclude that the linear segments, and the locations of their terminations, may reflect the basement fault structure that is being reactivated in the modern stress field. They state that physical processes for reactivation of basement faults could involve a weak lower crust and/or increased fluid pressures with the upper to middle crust. There may be a marginal correlation between the seismicity and major drainage pattern and general topography of the region, suggesting a possible hydrological element linkage.

Detailed geologic studies focused on locating paleoseismic evidence of large magnitude prehistoric events have only been conducted in limited areas. Whisner et al. (Reference 293) investigated a 117-sq. mi. area within the most active part of the ETSZ and found no concrete evidence of large prehistoric earthquakes. They noted, however, two other sites that they are considering for further study. At the Gray fossil site in northeastern Tennessee, fractures and joints with little offset exist throughout Miocene clay units that are not inconsistent with the late Tertiary to Holocene stress field. Also, in the same region, apparent dewatering features, in particular a clay- and gravel-filled fracture in Miocene clay, are observed. Deformation is postulated to be related to either strong ground motion, or more likely, sinkhole collapse. Whisner et al. (Reference 293) also describe a site in Tellico Plains, Tennessee, that exhibits disturbed and folded sediments in an older landslide or terrace deposit beneath younger Tellico River alluvium. Deformation at this site may be the result of soft-sediment deformation and liquefaction related to a prehistoric earthquake, or alternatively, it could be the result of dewatering and folding at the toe of a prehistoric landslide. Based on the extent of weathering in cobbles, Whisner et al. (Reference 293) suggest that the older alluvium may be late Pleistocene or early Holocene in age.

2.5.1.1.4.3 Significant Seismic Sources at Distances Greater than 320 km (200 mi.)

The EPRI-SOG evaluation indicated that the seismic sources in the New Madrid, Missouri, and to a lesser degree, Charleston, South Carolina, regions were significant contributors to any hazard at the BLN site.

2.5.1.1.4.3.1 Seismic Sources in the New Madrid Region

The New Madrid region is the source of the 1811-1812 New Madrid earthquakes, which include the three largest earthquakes to have occurred in historical time in the CEUS. Extensive geologic, geophysical, and seismologic studies have been conducted since the 1989 EPRI SOG study (Reference 233) to characterize the location and extent of the likely causative faults of each of these earthquakes and to assess the maximum magnitude and recurrence of earthquakes in this region.

This more recent information is summarized in the Clinton Early Site Permit (ESP) application (Reference 294) and the Grand Gulf ESP application (Reference 295) through early 2004. For the BLN seismic source model, a modified version of the NRC approved Clinton ESP characterization of the New Madrid source zone was adopted for the PSHA. These modifications have to do with the recurrence model, presented in Subsection 2.5.2.4.4.1.3.

Since submittal of the Clinton ESP, several additional studies have been published related to the New Madrid seismic source. Those studies are summarized in Subsection 2.5.2.4.4.1, and do not change the Clinton characterization of the New Madrid Seismic Source.

2.5.1.1.4.3.2 Seismic Source in the Charleston, South Carolina, Region

The 1886 Charleston, South Carolina, earthquake was the largest earthquake occurring in historical time in the eastern U.S., and is considered to have a moment magnitude in the range of 6.8 to 7.5 (References 296, 297, 298, and 299). Based on the felt intensity reports defining the meizoseismal area (area of maximum damage) and the occurrence of continuing seismic activity (the Middleton Place Summerville seismic zone), the epicentral region of the 1886 earthquake is considered to be centered northwest of Charleston. Recent published and unpublished studies for information on the potential location and extent of the Charleston source and the maximum characteristic earthquake expected to occur on it are described as follows.

Several types of data provide constraints on the location and extent of the source fault(s) for Charleston-type earthquakes in the Atlantic Coastal Plain. Bollinger (Reference 300) reviewed the original interpretation of the meizoseismal area by Dutton (Reference 301) and concluded that the meizoseismal area of the 1886 Charleston earthquake forms an elliptical zone roughly 20-mi. wide (northwest-southeast) by 30-mi. long (northeast-southwest). This zone is centered northwest of Charleston near Middleton Place, and extends from Charleston to Jedburg, South Carolina. This region is characterized by ongoing seismicity in the so-called Middleton Place-Summerville seismic zone (MPSSZ) (Figure 2.5-223). Possible causative source faults for the Charleston earthquake within the meizoseismal region include the Woodstock and Ashley River faults, and Woodstock lineament (References 275, 302, and 303). Talwani (Reference 304) indicates that the northeast-trending Woodstock fault is cut and offset approximately 3 to 4 mi. near Summerville by the northwest-trending Ashley River fault (Figure 2.5-223). Talwani also suggests that the 1886 earthquake was associated with right-lateral strike-slip movement along the offset segments of the Woodstock fault and uplift along the Ashley River fault.

Marple and Talwani (Reference 305) and Talwani (Reference 306) describe a potential causative source for the earthquake that extends beyond the 1886 epicentral region. One possible extended source is the southern segment of the zone of river anomalies (ZRA) (ZRA-S in South Carolina) of the East Coast fault system (ECFS; Figure 2.5-224); (Reference 305). The ECFS is a 370-mi.-long north-northeast-trending inferred fault system that is based on a series of anomalous changes in fluvial geomorphology (ZRA), coincident with linear aeromagnetic anomalies and buried and surficial faults (Reference 305). The ECFS is divided into three segments, with the strongest geomorphic evidence for tectonic activity associated with the southernmost segment, ZRA-S (Figure 2.5-225).

Other features in the vicinity of the meizoseismal region of the 1886 earthquake that are considered potential sources of large-magnitude earthquakes include strike-slip faults that bound Mesozoic rift basins and inferred/mapped faults bounding regions of tectonic warping. Behrendt and Yuan (References 307 and 308) and Tarr et al. (Reference 309) note the association of the MPSSZ (and the meizoseismal region of the Charleston earthquake) with a buried Mesozoic basin in South Carolina. No specific evidence for reactivation of basin-boundary faults has been identified, except where those faults are coincident with the ZRA-S. Weems and Lewis (Reference 310) evaluate tectonic warping in the Charleston area from stratigraphic data and suggest that two northwest-trending faults (the Adams Run and Charleston faults) accommodate tectonic movement in a hinge zone (Figure 2.5-223). These authors indicate that slip on these inferred boundary faults and on the Ashley River and Woodstock faults may have caused the 1886 Charleston earthquake.

The spatial distribution of seismically induced liquefaction features along the Atlantic seaboard has been used to assess the location and timing of pre-1886 earthquakes (References 311, 312, 313, 314, 315, and 316) (Figures 2.5-225 and 2.5-226). These studies suggest that during the past 2,000 to ~6,000 years, large earthquakes (mb $\geq 5.8 \pm 0.4$) have been restricted to South Carolina (Figure 2.5-226).

Talwani et al. (Reference 314) and Talwani and Schaeffer (Reference 317) established a more precise chronology for paleoliquefaction events observed in the coastal plain sediments of South Carolina. Eight paleoliquefaction events have been identified to have occurred during the past 5800 years (Table 2.5-201). Six of these events appear to have resulted from earthquakes occurring on the same source as the Charleston earthquake. These six events (including the two most recent prehistoric events) appear to have been of similar magnitude to the 1886 earthquake, based on the similarities in the spatial distribution of generated liquefaction features (References 314, 315, and 317) (Figure 2.5-227).

Maximum magnitudes in the Charleston region are based largely on the analysis of intensity data from the 1886 earthquake sequence and to a lesser degree on magnitude assessments inferred from paleoliquefaction features. Johnston (Reference 297) suggested a preferred value of moment magnitude (**M**) 7.3 $\pm$ 0.26 for the 1886 earthquake. Earlier magnitude estimates (References 299 and 300) gave a body-wave magnitude (m_b) ranging from 6.6 to 6.9. In a recent approach, Bakun and Hopper (Reference 296) developed a method to directly invert intensity observations. They obtained an estimate of **M** 6.9 (6.4 to 7.2 at the 95th percent confidence level) for the 1886 earthquake.

An alternative approach for estimating the magnitude of the 1886 earthquake relies on back-calculation of ground motions from the liquefaction evidence (References 298, 318, and 319). Martin and Clough (Reference 298); conclude that the liquefaction evidence from the 1886 earthquake is consistent with an earthquake no larger than **M** 7.5, and possibly as small as **M** 7.0. Hu et al. (Reference 319) estimate magnitudes in the range of **M** 6.8 to 7.8 for paleoearthquakes attributed to the Charleston source. Leon et al., (Reference 320) reevaluated the prehistoric earthquake magnitudes and peak ground acceleration (pga) from the spatial distribution of paleoliquefaction features and in situ geotechnical data corrected for aging effects and estimated that the magnitude estimates for prehistoric events should be lowered about

0.9 magnitude units. They estimate that the prehistoric earthquakes that occurred during the past 6,000 years in the South Carolina Coastal Plain had moment magnitudes between approximately 5 and 7 and peak ground accelerations between about 0.15 and 0.30g when aging factors are considered (Table 2.5-202).

2.5.1.2 Site Geology

The following subsection presents a summary of geologic conditions of the 25-mi. radius, 5-mi. radius, and the 0.6-mi. radius. Site physiography and topography, geologic history, stratigraphy, lithology, structural geology, and engineering geology are discussed. The information presented is based on a review of previous Bellefonte Units 1 and 2 reports and documents, review of geologic literature, and the results of geotechnical and geologic field investigations conducted at the southern site, and at the Units 3 and 4 site (Figure 2.5-201). A more detailed discussion of the geological conditions beneath Category 1 structures is presented in Subsection 2.5.4.1.

2.5.1.2.1 Site Physiography and Topography

The BLN site is located in the Browns Valley-Sequatchie Valley segment of the Cumberland Plateau section of the Appalachian Plateaus Physiographic Province (Figure 2.5-202). The regional physiography has been discussed in Subsection 2.5.1.1.1.

The BLN site lies on the southeast side of the valley that separates Sand Mountain from the Cumberland Plateau (Figure 2.5-228). It is known as Browns Valley in Alabama. To the northeast in Tennessee it is known as the Sequatchie Valley. The Browns Valley-Sequatchie Valley extends northeast-southwest for approximately 140 mi., from Crab Orchard, Tennessee, to the vicinity of Blount Springs, Alabama. This valley was formed from erosion of the Sequatchie anticline (Figure 2.5-229). Where erosion breached the arch of thick sandstone and exposed the dolomite and limestone, an axial valley was developed. The valley is regionally bounded on the southeast by the prominent flank of Sand Mountain, which rises to about 1400 ft. above mean sea level (amsl). The highly dissected and irregular edge of the Cumberland Plateau, which rises to similar elevations, forms the northwestern flank of the valley. The present valley floor is in all respects like those of the folded Valley and Ridge Province to the east. Due to greater weathering of weaker rocks below the sandstone cover, the valley walls, which are bounded by escarpments, remain steep. The straightness of the valley merely reflects the straightness of the structural contours. Base-leveling of the upturned hard rocks on the flanks was never completed and these remain as low monoclinal ridges that are interrupted at intervals by gaps cut down to general level of the valleys.

The TVA Bellefonte property is located on the right bank of Guntersville Reservoir on the Tennessee River at river mile 391.5 in Jackson County, Alabama (Figure 2.5-201). At the site, the valley is approximately 5 mi. wide, and the

Tennessee River flows southwestward. The river has entrenched its course to about 570 ft. before impoundment of the reservoir in 1939. Normal pool level today is 595 ft.. The Units 3 and 4 power blocks lie within the gently rolling terrain of the river valley at an elevation of about 620 ft. Directly southeast of the Units 3 and 4 power blocks, within the BLN site, a low ridge, referred to as River Ridge, is developed in the more resistant beds of the southeastward-dipping Sequatchie and Red Mountain Formations (Figure 2.5-230). River Ridge separates the Units 3 and 4 power blocks from the Tennessee River/Guntersville Reservoir by a distance of about 3000 ft. and its crest stands at an elevation of about 800 ft. Gaps in the ridge are due to erosional development along normal dip joint systems, and no cross faulting is evident at these locations (Reference 201).

The BLN site slopes gently toward local drainages. Relief across the Units 3 and 4 power block construction zone is about 34 ft., sloping from 638 ft. at the south corner to 604 ft. at the north corner of the site. Surface drainage within the power block construction zone is north-northeast toward the Town Creek embayment via small creeks and ditches. A shallow divide just southeast of the power block construction zone separates this drainage from another that flows east toward the Tennessee River through a gap in the ridge at the intake structure.

Northwest of Units 3 and 4, the land slopes gently downward to the Town Creek embayment, the former valley of Town Creek now an arm of the Guntersville Reservoir. Quite typical of the area, the Town Creek embayment as well as the Mud Creek embayment to the northeast, show erosional development along the more soluble belts of the Lower Ordovician and Upper Cambrian strata. The BLN site is primarily underlain by Middle Ordovician strata of the Stones River Group. The Cambrian-Lower Ordovician Knox Group underlies the Stones River Group and outcrops to the northwest near the site boundary.

The bedrock at the BLN site is overlain by residual silts and clays, 5 to 40 ft. thick, derived from in-place weathering of the underlying rock. Drilling and excavation experience at the site and in adjacent areas shows that the residual soil transition through weathered rock to hard, unweathered bedrock can be gradual in the natural shallow subsurface profile in some places, or abrupt in other places. As shown on Figure 2.5-230, overburden has been disturbed by plant construction activities. Most of the BLN site lies in areas disturbed by the construction activities for Units 1 and 2. These areas include paved parking areas and roadways, and graded lands partially covered with placed fill.

2.5.1.2.2 Site Geologic History

The geologic history presented herein is an overview of the geologic history of the site summarized from the Units 1 and 2 FSAR (Reference 201) and from the Regional Geology Subsection 2.5.1.1. The overall tectonic framework of the region is outlined in Subsections 2.5.1.1.2 and 2.5.1.1.4. Generally, current understandings and thoughts on the geologic history of the area around the BLN site have not changed significantly since the Units 1 and 2 FSAR (Reference 201) was prepared. Changes in geologic thought and interpretation of past events deal more with the inferred details of the mechanics of the thrust faulting and folding of the bedrock units, and less with the ages of deformation or mapped positions of the bedrock units and structural features (see discussion in Subsection 2.5.1.1.4.1). Changes in interpretation that have occurred in the interim include the differentiation of the Ordovician limestones originally mapped as the Chickamauga Limestone in the site area into the Stones River Group, the Nashville Group, and the Sequatchie Formation, and the general recognition of the Chattanooga Shale as being of Devonian rather than Mississippian age (References 224 and 321). A stratigraphic column in the Appalachian thrust-belt region of Alabama is presented in Figure 2.5-207.

The earliest history of the area is recorded in the basement complex of metamorphosed rock that lies more than 1.5 mi. below land surface. Those rocks have been dated by K-Ar dating techniques and are reported as being from 750 to 1000 million years old (Neathery and Copeland, 1983, as reported by Reference 321). There is a gap in the geologic record between when the basement rocks were formed and when the near-surface sedimentary rocks exposed in the area were deposited. The oldest rocks visible at the surface or projected into the area from regional studies are of early Cambrian age. These rocks are about 500 million years old.

The geologic history of this area for the past half billion years can be generally broken into two primary episodes: the early history when marine and near-shore deposits of limestones, shales, and sandstones of Paleozoic age were deposited on top of the basement complex rocks, and the more recent history when the site generally was well above sea level and subjected to mostly erosional geologic conditions during the Mesozoic and Cenozoic Eras. There is a gap in the geologic record in north Alabama between the youngest Paleozoic rocks in the area, the Pottsville Formation of Pennsylvanian age, and the present. Elsewhere in Alabama, deposits assignable to the time represented by the later part of this gap are present. There are no deposits in Alabama or eastern Tennessee of Permian through early Cretaceous age other than a few intrusive igneous rocks assignable to Triassic age in east Alabama. The time represented by this gap in the record in northeast Alabama adjacent to the BLN site covers many millions of years and includes the period when the Paleozoic rocks at the site were thrust upward and westward to their present positions. The period of time since the last period of thrust faulting and mountain building in the early Mesozoic has been primarily one of erosion of the current land surfaces in north Alabama. Most deposits that might have been formed there during that time have been subsequently removed by erosion.

Episodes of uplift and erosion also occurred periodically during the Paleozoic Era, but the geologic record in this area for that time is mainly represented by marine rocks deposited in marine or near-shore marine environments. The periods of uplift and erosion are represented by erosional surfaces or unconformities in the stratigraphic record. The most significant of these unconformities roughly coincide with the breaks between the various geologic formations mapped in the area, although minor unconformities also occur within some of the rock units.

During early Paleozoic time, the part of North Alabama in which the BLN site lies was often covered by a shallow inland sea. The oldest rocks on top of the basement complex are shales and marine carbonate rocks such as limestones and dolomites that were deposited in the Cambrian and Ordovician Periods. Unconformities developed between rocks of Cambrian and early Ordovician age (Knox Group) and Middle Ordovician age (Chickamauga Limestone equivalent beds) suggest intervals of uplift and sub aerial erosion occurred. Volcanic activity in the Ordovician released ash and these formed thin beds of bentonite clay within the limestone. These clays are laterally continuous and widespread throughout north Alabama, Georgia, and Tennessee in rocks of this age.

Toward the later part of the Ordovician Period and into the Silurian, continental uplift and mountain building was associated with the uplift and erosion of the Nashville Dome and adjacent land masses, which in turn resulted in clay and sand being washed into and deposited within the area. These events are represented in the stratigraphic record by shales and sandstones. Unconformities between the rocks of differing age deposited during this time indicate periods of additional uplift and erosion accompanied by relative, local sea level rise and fall. Iron-rich sediments deposited during the Silurian Period (i.e., the Red Mountain Formation) indicate local environmental conditions changed significantly enough to allow primary deposition of iron-rich deposits in near-shore marine conditions.

Following the time when the Silurian system rocks were deposited at the site, significant uplift occurred and a regional unconformity developed before the deposition of the Devonian rocks. Sandstones and shales (i.e., the Frog Mountain Sandstone and Chattanooga Shale) deposited at this time indicate further inundation and adjacent landmass erosion. In places the deposition apparently continued relatively unabated into the Mississippian Period and less erosion of adjacent land occurred, as evidenced by thick deposits of Mississippian age limestones. Some apparently primary chert deposition in the middle Mississippian Period (Fort Payne Chert) indicates that environmental conditions were again altering and relatively unique, at least for some period of time. Cleaner limestones, containing less clastic material, overlying the Fort Payne indicate that the marine environments typical of shallow seas like those of the Cambrian and Ordovician Periods returned to the area. Shales and sandstones deposited elsewhere in north Alabama in the late Mississippian Period (i.e., the Floyd and Parkwood shales and Hartselle and Pride Mountain sandstones) indicate the relative sea level was beginning to drop and that significant erosion of nearby land masses was occurring.

Beginning at the end of the Mississippian Period and extending throughout the Pennsylvanian Period, the entire area of northeast Alabama and eastern Tennessee occupied the shore line area at the edge of the sea. Deposits represented in the site area include sandstones typical of beach deposition altering with stream deposits, near-shore muds and clays, and occasional coal beds. Apparently, the rise and fall of relative sea level in the area occurred in a cyclic pattern with altering periods of submergence and subsequent uplift and vegetation. This resulted in the layering of sandstones, shales, and coal beds typical of the Pottsville Formation.

Following the Pennsylvanian Period, there is no record of significant deposition of geologic units occurring in northeast Alabama through the present time. At that time, the Alleghanian orogeny was causing the thrusting and faulting observed today in the northeast to southwest trending valley and ridge system common to this part of the country. Associated mountain building caused the Appalachians to rise again. As the mountains rose, erosion began and that resulted in the beginning of Gulf and Atlantic Coastal Plain clastic deposition. This deposition began in the Mesozoic Era and generally continued through to today, although the general deposition was interrupted by periods of uplift and erosion and some gaps occur in the coastal plain record.

No geologic depositional evidence of the time between the Pennsylvanian through the Quaternary is preserved near the BLN site. The primary geologic processes during this time in the BLN area were erosional in nature. In the major river channels, erosion cut down to bedrock in most places and scoured the unconsolidated alluvial deposits away, leaving little geologic record for the last 135 to 150 million years of the earth's history. Away from the streams, thick residual soils developed in place over the carbonate units as a result of chemical weathering. Colluvial deposits developed to cover most hill slopes as the uppermost rock layers (generally sandstones that are resistant to weathering in this climate) slowly broke down or were undercut by erosion of softer underlying beds, and migrated down the slopes.

Karst processes affected areas underlain by carbonate rocks (limestone and dolomite) in the site region. Solution of the bedrock created a highly irregular bedrock surface beneath the residual soil. Deeper within the bedrock, groundwater solution enlarged fractures in the rock, forming underground drainage systems and caves. A few of these karst features were sites for very localized deposition, like the Gray Fossil Site in eastern Tennessee (Reference 293). The Gray Fossil Site preserved a late Miocene or Pliocene vertebrate fauna indicative of a forested ecosystem. More commonly, the karst features that developed on some bedrock units continued to enlarge and were significant factors in the erosional process. Within the Knox Group deposits northwest of the BLN site, large shallow closed depressions in the land surface, or sinkholes, show where significant karst development has occurred. Within the Stones River Group, no closed depressions are present, suggesting that much less karst development has occurred in that stratum. Small-scale karst features are documented over the Stones River Group and other limestone formations within the BLN site. A thorough discussion of the karst setting and documented karst features at the site is presented in Subsection 2.5.4.1.

2.5.1.2.3 Site Stratigraphy

This subsection presents the stratigraphic nomenclature for the site revised to conform to Geological Survey of Alabama standards (References 224 and 321).

The revised stratigraphic units are mapped over the 0.6-mi. radius, and each unit is described. Stratigraphic units include Paleozoic sedimentary rocks, Quaternary alluvium and colluvium, residual soils, and anthropogenic deposits.

2.5.1.2.3.1 Revised Stratigraphic Nomenclature

Regional geologic mapping has been performed since the late 1900s, with a 1926 detailed and comprehensive Geology of Alabama (Reference 322). The area stratigraphy described in the 1986 FSAR for Units 1 and 2 (Reference 201) was based on the 1926 Geology of Alabama (Reference 322) (Figure 2.5-229). Studies continuing through the twentieth century eventually resulted in revisions to formation names and correlations between rocks at the site and in adjacent parts of Alabama, Tennessee, and Georgia. In 1988, the Geological Survey of Alabama published a new Geologic Map of Alabama (Reference 224), which forms the basis of the stratigraphy used in the present study of the BLN site (Figure 2.5-228). The following paragraphs explain the differences between the stratigraphy used by the Bellefonte Units 1 and 2 FSAR (Reference 201) and that used in the present study.

The 1926 Geology of Alabama (Reference 322) is a detailed and comprehensive volume and map that has served as the authority on Alabama geology for many decades. It defines the Chickamauga Limestone to include all the rocks stratigraphically above the Knox Dolomite and below the Red Mountain Formation. Accordingly, the Bellefonte Units 1 and 2 FSAR (Reference 201) describes the site as being underlain by Chickamauga Limestone. The authors of Geology of Alabama, however, recognized that the Chickamauga was made up of several distinct units separated by unconformities. Their remarks, as they conclude the description of the Chickamauga Limestone, foreshadow future work,

> "Ultimately the several units of which the Chickamauga is composed will be accurately delimited and separately mapped and described and then the name will pass out of use." -Adams and others, 1926 (Reference 322)

In 1988 the Geological Survey of Alabama published a new geologic map of the state with updated information including new unpublished mapping in Jackson County (Reference 323). The new state map (Reference 224) and companion volume (Reference 321) established a revised stratigraphy that serves as the standard reference today. The new scheme divides the former Chickamauga Limestone within Sequatchie Valley into three units. The Stones River Group and Nashville Group comprise the Middle Ordovician strata, and the Sequatchie Formation comprises the Upper Ordovician strata. The Chickamauga Limestone is redefined to include only Middle Ordovician strata and the name is applied only in the western Valley and Ridge province of Alabama (References 224, 321, and 324) (Table 2.5-203). A second change in stratigraphic nomenclature is the general recognition of the Chattanooga Shale as being entirely of Devonian age. The Bellefonte Units 1 and 2 FSAR (Reference 201) summarizes the varying opinions and evidence to support a probable Devonian age for the shale, but stops short of ruling out a partially Mississippian age. The Geological Survey of

Alabama, however, assigns the Chattanooga Shale unequivocally to the Devonian based on published fossil evidence and radiometric ages (Reference 321).

The new state map (Reference 224) also revised the delineation of units in the site area, mapping parallel bands of Nashville, Sequatchie, and Red Mountain through the hills east of the site (Figure 2.5-228). The 1926 geologic map (Reference 322) had mapped only Red Mountain Formation in these hills, an interpretation adopted by the Bellefonte Units 1 and 2 FSAR (Reference 201) (Figure 2.5-229).

2.5.1.2.3.2 Stratigraphic Units

The stratigraphic units present in the BLN site 0.6-mi. radius include bedrock formations ranging in age from Cambrian to Pennsylvanian, and unconsolidated sands and gravels of Quaternary age (Table 2.5-204). No record of deposition from the Permian through the Tertiary Periods is known in this immediate area. The stratigraphic column includes those sedimentary rocks that crop out in the area or are likely expected to crop out nearby and are projected under the site (Figures 2.5-229 and 2.5-230). Several thousands of feet of bedrock are present at and underlying the site. In outcrop, the bedrock units underlying the area form generally parallel outcrop belts that strike northeast to southwest in the Sequatchie Valley. Quaternary alluvial and colluvial deposits overlie bedrock along the larger streams, on hillslopes and hollows, and are generally thin and of limited areal extent. Fill and spoils dating from the development of Units 1 and 2 in the 1970s and 1980s blanket large portions of the BLN site (Reference 201).

2.5.1.2.3.2.1 Paleozoic Units

Undifferentiated Knox Group rocks of Cambrian to Lower Ordovician age are the oldest rocks exposed at the BLN site. The Knox Group consists of dolomitic, siliceous, and cherty limestone, which are extensively weathered and covered in the area with thick cherty, red clay residuum that developed in place. The Knox Group rocks consist of the Copper Ridge Dolomite and Chepultepec Dolomite formations, which together are more than 2000-ft. thick (Reference 321). As a result of the extreme weathering of the dolomites, the formations cannot be distinguished in outcrop. Rocks assignable to the Knox Group crop out northwest of the BLN site along the axis of the Sequatchie anticline.

Little bedrock assignable to the Knox Group is visible in outcrop. Exposures consist mostly of deeply weathered residuum-consisting of reddish-brown clay with chert fragments and cobbles as much as tens of feet thick. Sinkhole features-closed depressions with internal drainage, occur commonly throughout the Knox Group outcrop belt-one relatively recent/apparently active small sinkhole or collapse feature was observed southwest of the BLN site on an adjacent land owner's property during aerial reconnaissance of the area. All other sinkholes noted from topographic maps of the area appear to be relatively inactive insofar as collapse is concerned-but they are directing surface water into the ground and subterranean groundwater flow is being induced in those areas. A further discussion of sinkholes and dissolution is presented in Subsection 2.5.4.1.

The Stones River Group of Middle Ordovician age unconformably overlies the Knox Group. The contact between the Knox and Stones River shown on the map is located based on the presence of reddish clay with cherty residuum in the field to the north of the plant road at the far western edge of the map, and on three boreholes near the railroad bridge that encountered the Knox Group at depth (Figures 2.5-229 and 2.5-230).

The Units 3 and 4 power block construction zone lies entirely upon limestone of the Stones River Group. The Stones River Group was originally defined in Tennessee where it was subdivided into five separate formations; however, in northern Alabama the Stones River Group is undifferentiated. It consists of medium to dark-gray thin to thick-bedded, fine-grained, dense limestone. Silty and argillaceous beds are locally present. It contains a zone of bentonite and bentonitic shales near the top. The Stones River Group has an approximate thickness at the site of 1050 ft., measured from cross section A-A' (Figure 2.5-231). Detailed descriptions of the rock underlying the plant footprint based on borehole logs are presented in Subsection 2.5.4.3.

An argillaceous and silty limestone unit within the Stones River Group, 60 to 70 ft. thick, was encountered in boreholes at the Units 3 and 4, Units 1 and 2, and southern sites (Figure 2.5-201). Its projected outcrop pattern is shown on the map and is labeled Osra (Figure 2.5-230). Within this unit, layers of dark gray argillaceous and silty dolomitic limestone, 0.5 to 3-in. thick, comprise forty percent or greater of the rock and alternate with layers of gray limestone. Downhole velocity logs show a marked decrease in velocity in this unit. Other argillaceous and silty intervals were encountered in boreholes within the Stones River Group, however, this particular unit was the thickest, and could be identified at all three sites. Near the Units 3 and 4 power blocks it is identified in the rock core as unit C; near the Units 1 and 2 power block it is identified in the rock core as unit 3 (Reference 201).

Outcropping on the slightly higher eastern edge of the valley at the base of the eastern hills outside the Units 3 and 4 power block construction zone, is the Nashville Group. Some service buildings for Units 1 and 2 sit on the Nashville Group, and the road to the intake structure crosses these rocks. The Nashville Group (map symbol Onv) conformably overlies the Stones River Group and consists of gray, fine-to medium-grained fossiliferous, locally silty and argillaceous limestones. Several bentonite beds are found in the lower part of the Nashville as well as in the upper Stones River Group. The bentonite beds represent volcanic ash that fell into the Ordovician seas, and as such, are important time markers within the Ordovician. The original ash layers have been partly altered to greenish bentonitic clay containing abundant crystals of primary feldspar. The boundary between the Stones River and Nashville Groups is placed at the T3 bentonite bed. The Nashville Group is 270 ft. thick.

Conformably overlying the Nashville Group is the Sequatchie Formation, of Upper Ordovician age. The Sequatchie Formation (map symbol Os) forms the core and crest of River Ridge. It consists of gray thin-bedded calcareous shale and mudstone with interbedded fossiliferous limestone and sandy, glauconitic bioclastic limestone. Beds correlative with the Leipers Limestone and Inman Formation comprise the lower beds of the Sequatchie Formation. Higher in the section, thick beds of gray micritic limestone outcrop in the headwalls of two small drainages located immediately southeast of the BLN site. Near the top of the section a distinctive maroon fossiliferous limestone, known as the Fernvale limestone is exposed in outcrop near the crest of the ridge, and was identified in boreholes in the intake structure area (Reference 201). A bed of ferruginous sandstone lies at the top of the formation and forms the crest of several highpoints in the ridge. Weathered residuum from the Fernvale limestone and overlying sandstone litters the hills and forms a deep, soft, reddish soil that mantles the hillslopes. The thickness of the Sequatchie Formation, based on boreholes drilled in the intake structure area, is approximately 240 ft.

The Silurian Red Mountain Formation unconformably overlies the Sequatchie Formation at the site, and crops out on the line of smaller hills between the crest of River Ridge and the reservoir. The Red Mountain Formation (map symbol Srm) is a shallow-marine clastic sequence that is composed of interbedded sandstone, siltstone, shale, and sandy limestone, with a few thin hematite beds. Unlike the rocks at the type locality for this formation near Birmingham, the beds of the Red Mountain Formation in northeastern Alabama are generally not red, and contain significant thicknesses of limestone (References 324 and 326). Fossiliferous limestone of the Red Mountain Formation is exposed in outcrop east of the site on the flanks of very low hills immediately adjacent to the reservoir. Rhythmically bedded sandy limestones and shales form a prominent cut-face adjacent to the barge dock. The Red Mountain Formation also crops out about two-mi. northwest of the site on the east side of Backbone Ridge. The Red Mountain Formation is approximately 100 ft. thick at the site.

2.5.1.2.3.2.2 Quaternary Units

Quaternary alluvium occurs along major and minor streams across northern Alabama. These deposits are typically thin and of limited areal extent. Quaternary alluvium is mapped at the site in two settings: (1) along the margins of the Guntersville Reservoir as remnant fluvial terrace deposits of the Tennessee River, and (2) filling small valleys where it has been deposited by small streams discharging from drainages in the hills. Alluvial deposits were mapped on the basis of topographic expression, and in the case of Tennessee River deposits, the presence of rounded clasts. More extensive Tennessee River alluvial deposits may be found beneath the waters of the Guntersville Reservoir with only the higher remnants exposed today above pool level.

Quaternary colluvium consists of weathered residuum transported by hillslope processes such as landslides, slopewash, and creep, and deposited at the base of hillslopes or in hollows on the hillslides. It is mapped primarily on the basis of topographic expression. These deposits are typically thin and of limited areal extent. Only larger bodies of colluvium are mapped.

A layer of predominantly residual soil averaging 10 to 15 ft. in thickness mantles the bedrock throughout the site area. The soils in Jackson County in the site area generally are grouped according to their topographic position (Reference 325) as: (1) soils of limestone valley uplands; (2) soils of sandstone plateaus; (3) soils of colluvial slopes; (4) soils of stream terraces; and (5) soils of first bottoms (bottom lands along drainages underlain by fluvial deposits subject to overflow). The areas classified as uplands and plateaus lie above the stream bottoms and consist of materials derived directly from the weathering and decay of the underlying rocks. Strictly residual soils are not common in the limestone valley uplands, as most soils are modified by or derived from parent material accumulated as alluvium or colluvium. Stream terraces are underlain by fluvial deposits that form benches adjacent to stream bottoms, but are not subject to flooding. Many of the higher terraces are severely eroded and mantled chiefly by residuum.

Descriptions of soils mapped on these surfaces and observations from the excavations indicate that the original deposits and soils have been eroded and these surfaces are mantled by a relatively thin veneer of residual soil formed from weathering of the in-place bedrock.

2.5.1.2.3.2.3 Anthropogenic Deposits

Construction taking place in the 1970s and 1980s for Units 1 and 2 resulted in large areas of fill and disturbed ground. Figure 2.5-230 shows the distribution of fill and disturbed ground based on: (1) a comparison of topographic contours from 1971 and 2006, (2) inspection of aerial photographs from the construction period, and (3) evaluation of TVA borrow and spoils areas map.

Areas marked "Fill" include areas of spoils, road fill, and undocumented fill outside the main developed areas (Figure 2.5-230). Mapped within this unit are borrow pits that were subsequently backfilled with spoils or used as landfills.

Areas marked "Disturbed Ground" include the existing Units 1 and 2 nuclear plant area east of the plant road and west of River Ridge, and the training facility (Figure 2.5-230). Much of this area was cut, filled, and graded for construction of the power plant. Patterns of cut and fill are too complex to delineate separately. Included are fill pads built for the reactors and other key structures, berms, waste ponds, areas that were graded and leveled for other buildings, and the switch yard.

2.5.1.2.4 Lithology of the Stones River Group

The Stones River Group is comprised of subunits that differ slightly from one another in composition and texture. Rock core from the 2006 Units 3 and 4 exploratory drilling program contain alternating beds of limestone to dolomitic limestone and argillaceous and silty limestones, with some cherty limestone. Within the Units 3 and 4 power block construction zone, six distinct lithologic units, designated units A through F were identified. Cross sections showing the six units are presented in Subsection 2.5.4.1. These units comprise a total thickness of

453 ft. within the 1050-ft. thick Stones River Group (Table 2.5-205). Downhole shear wave velocity measurements for eight of the boreholes, combined with examination of rock core photographs for all boreholes, formed the basis for identifying the units and correlating them throughout the construction zone. Geologists had previously established a sequence of stratigraphic units and marker beds in the boreholes drilled for the Bellefonte Units 1 and 2 in the 1970s (Reference 201). Table 2.5-205 presents a correlation of lithologic units between boreholes drilled for BLN Units 3 and 4 and those drilled previously for Units 1 and 2.

The six lithologic units are varieties of limestone that contain differing amounts of dolomite, clay, silt, and chert. Petrographic, chemical, and mineralogic analyses, combined with field descriptions, form the basis for the lithologic descriptions of each unit. Total carbonate in rock described as medium gray limestone varies from 68 to 96 percent, averaging 83 percent. Dolomite (ankerite) comprises up to one-third of this carbonate. Total carbonate in rock described as dark gray, argillaceous and silty (dolomitic) limestone varies from 32 to 68 percent.

Unit A (64'): Medium gray limestone (micrite and wackestone with a 1-foot-thick fossiliferous zone) with dark gray, wavy dolomitic laminae 0.01-to-0.05-ft. thick, and interbeds of dark gray argillaceous and silty limestone 0.1-to-0.3-ft. thick.

Unit B (121’): Medium gray limestone (micrite, packstone, and wackestone) with dark gray, wavy dolomitic laminae. Some 1-to-5-ft. thick beds of massive limestone and two prominent zones with interbeds, 0.05-to-0.3-ft. thick, of argillaceous and silty limestone.

Unit C (67'): Dark gray argillaceous and silty dolomitic limestone with interbeds of medium gray dolomitic limestone. In the upper 45 ft. of the unit, dark gray beds (50 percent) are thin 0.03 to 0.01 ft. and planar, and alternate with medium gray beds (50 percent) of similar thickness. A 5-foot section follows dominated by medium gray limestone (80 percent). The lower 20 ft. consists of about 40 percent dark gray beds and 60 percent medium gray beds, and the beds have irregular diffuse boundaries. Unit C is the most conspicuous and laterally continuous lithologic unit identified in the Middle Stones River Group.

Unit D (133'): Medium gray limestone (micrite, with few beds of packstone and wackestone) with dark gray, wavy dolomitic laminae. The upper 25 ft. of the unit contains scattered black chert nodules visible in the core. Chert nodules, up to 1.5 ft. in diameter, irregular and subrounded in shape, are also encountered in test pits dug in the residual soil developed over unit D where they float in the residual soil matrix. The base of unit is marked by a light olive-gray massive limestone, 3 ft. thick, with stylolites, located 5 ft. above the basal contact.

Unit E (20'): Medium gray limestone (micrite) with dark gray, wavy dolomitic laminae, and thin, planar interbeds of dark gray argillaceous and silty limestone.

Unit F (48'): Medium gray limestone (micrite) with dark gray, wavy dolomitic laminae.

2.5.1.2.5 Site Structural Geology

This section provides a review of the structural setting from existing information, including the Bellefonte Units 1 and 2 FSAR (Reference 201), supplemented by new structural information from the 2006 geologic mapping and exploration program for Units 3 and 4. New data include geologic mapping, and bedding attitudes measured from outcrops and computed from lithologic contacts in boreholes. Borehole televiewer (BHTV) discontinuity data provide additional data on the attitude of bedding and jointing. The site geologic map and cross-section (Figures 2.5-230 and 2.5-231) present basic structural information including bedding attitudes and outcrop pattern. A large-scale structure contour map of the Units 3 and 4 power block construction zone is presented in Subsection 2.5.4.1. A general site reconnaissance was performed to verify general structural interpretations of the area presented in the literature describing this part of Alabama and observations made on that trip also are included herein.

The BLN site lies within the northwestern (frontal) part of the Appalachian fold-thrust belt as described in Subsection 2.5.1.1.4.2. Two bedrock faults, the Sequatchie Valley and Big Wills Valley faults, are mapped within the 25-mi. radius, and one of these, the Sequatchie Valley fault, lies within the 5-mi. radius (Figure 2.5-228). Neither of these faults is considered to be a capable tectonic source, as defined in Regulatory Guide 1.208, Appendix A (see discussion in Subsection 2.5.3.6).

The BLN site is located on the gently dipping, i.e., about 15º to 25º dip, southeast limb of the Sequatchie anticline, shown in Figure 2.5-229. This asymmetrical anticline has a gently dipping southeast limb and a steeply dipping northwest limb. The axis of the Sequatchie anticline lies approximately 1.4 mi. northwest of the site. As documented in the FSAR (Reference 201), there is no intense folding or major faulting within the foundation bedrock of Bellefonte Units 1 and 2.

The strata strike N 45° E and dip 15° to 17° southeast within the Units 3 and 4 power block construction zone, based on borehole lithologic contacts and BHTV data. Dip decreases toward the Guntersville Reservoir. Dips measured in outcrops and computed from lithologic contacts in borings (Reference 201) are 12° near the intake structure, and 6° adjacent to the reservoir (Figures 2.5-230 and 2.5-231). Based on our interpretation of borehole logging and insitu geophysical surveys, the lithologic units within the Stones River Group are oriented consistently across the site and are not measurably deformed or offset by faulting. The bedding attitude is similar to that for the Units 1 and 2 site. At the Units 1 and 2 site the strata strike N39-40°E and dip 17° to the southeast (Reference 201).

The Sequatchie anticline is broken on the west by the Sequatchie Valley thrust fault, which at its closest point is 2.2 mi. northwest of the site (Figures 2.5-228 and

2.5-229). The fault dips to the southeast and is projected to be about 5000 ft. below the surface of the site (Reference 201). Additional data on the regional characteristics of the Sequatchie Valley thrust fault are presented in Subsection 2.5.3. In the site area, the fault juxtaposes limestones of Middle Ordovician age and the Fort Payne Chert of Mississippian age (Reference 201). No exposures of the main Sequatchie Valley thrust fault in the Bellefonte area were described in the FSAR for Bellefonte Units 1 and 2 (Reference 201), and none were observed during the field reconnaissance studies for the GG&S evaluation (Reference 399). Backbone Ridge, which is formed by the near-vertical resistant beds of Silurian and Mississippian age, marks the northwest limb of the Sequatchie anticline in the site area. At one location along Backbone Ridge, approximately 3.8 mi. southwest of the site, steeply west-dipping beds and numerous small faults that appear to be minor splays or backthrusts off the primary thrust fault are visible in a large excavation at the City of Scottsboro waste transfer facility. These faults, which have apparent displacements of only a few feet, are in the hanging wall less than 0.1 mi. from the mapped trace of the Sequatchie Valley thrust fault.

No evidence of faulting or shearing in the bedrock was observed in excavations for the Reactor, Auxiliary, and Control Building areas of Bellefonte Units 1 and 2 (Reference 201). Minor displacement that was observed in the northwest corner of Unit 1 QA Records Storage Vault was investigated by core drilling and recorded by surface mapping (Reference 201). The joints exhibited 3 in. of vertical offset with a strike of N89°E and an average dip of 64°. Three vertical coreholes and two inclined coreholes were drilled into the feature. The fault is described as 0.1-to-0.5 in. thick, sinuous in shape, and calcite filled. It terminates at a vertical joint. TVA concluded that the feature is not a significant fault, but is a joint that received minor displacement as part of the process that resulted in the entire joint set (Reference 201).

High angle joints are observed at both the BLN site. Televiewer data and observations of fractures in core within the Units 3 and 4 power block construction zone show a prominent joint set with a mean strike of N 35° E, nearly parallel to bedding strike, and a mean dip of 73° NW, approximately perpendicular to bedding dip. At the Bellefonte Units 1 and 2 site this same joint set occurs and is reported to strike N30-50° E and dip 70-80°NW. Two additional prominent joint sets are reported in the Bellefonte Units 1 and 2 FSAR (Reference 201), yet were not noted as prominent in the televiewer logs for the BLN site. One set strikes N80°E with dips ranging from 70°NW to near vertical, and the other set strikes N50°W to N80°W and is nearly vertical (Reference 201). Lineament orientations, reported in Subsection 2.5.3, suggest an additional high angle joint set may be present, striking N5°W to N25°E. These high angle joints may become loci for deep weathering as discussed in the later section on karst (Subsection 2.5.4.1).

Joints and fractures in the site area likely formed as a result of the thrusting and mountain building forces that created the Sequatchie anticline in late Paleozoic time. Most joints in the Appalachian Plateau rocks formed as a result of the primary compression forces with shortening in the NW-SE direction (Reference

215). Joints have two major trends in the southern Appalachian Plateau, one across and one parallel to the strike of major structures; two minor sets trend north-south and east-west (Reference 215). The joint trends observed at the site are consistent with these trends.

Lineaments were identified on topographic maps and aerial photographs of the site. These lineaments appear to be related to bedrock structure and jointing (see discussion in Subsection 2.5.3.1). Field investigations, consisting of surface geophysical surveys, test pits, cone penetrometer surveys, and drilling and coring along and adjacent to these lineaments, indicate the following:

- Some of the lineaments coincide with zones of increased depth of weathering or solution of the bedrock surface.
- Some lineaments may represent surface expressions of deeper seated geologic structures, such as high-angle joints, but no direct physical evidence for this has been observed.
- Some lineaments may represent areas where increased groundwater flow is concentrated in the near-surface bedrock.
- Some lineaments may be related to subtle lithologic differences in the southeasterly dipping Stones River Group bedrock
- Some southeast-northwest lineaments that cut across River Ridge appear to be part of a regional topographic fabric probably related to jointing caused by large-scale deformation.
- Some lineaments associated with large erosional gaps in River Ridge may represent the remnants of paleo-valleys, eroded along joints by Town Creek or a similar surface drainage active earlier in the Quaternary, but no direct evidence for this has been observed.

There is no geomorphic evidence to suggest differential uplift across any of the lineaments that intersect the site. In addition, there is no structural or stratigraphic evidence to suggest lateral displacement across any of the lineaments that intersect the site. There is no geomorphic evidence to indicate that any of the lineaments identified are associated with a capable tectonic source as defined by Regulatory Guide 1.208, Appendix A.

2.5.1.2.6 Site Engineering Geology Evaluation

An evaluation of engineering geology including geologic hazards at the BLN site was conducted. The review of geologic hazards was performed through a search of published maps and reports, by visual reconnaissance of the area, review of data from the BLN 2006 field exploration program, and discussions with TVA about geologic conditions and types of current and past industries in the area.

Based on review of the site geology, it is concluded that:

- Karst-related ground failure or subsidence due to underground dissolution of limestone is judged to be a geologic hazard to the BLN site, however, the hazard to the Units 3 and 4 power blocks is anticipated to be minor and thus can be mitigated during excavation and construction activities.The BLN site is underlain by limestone, a soluble rock associated in Alabama with underground drainage, caves, sinkholes, and irregular weathering of bedrock. Small-scale karst features are documented within the BLN site. Exploration data from the Units 3 and 4 power block construction zone show similar conditions to those at Units 1 and 2, where foundation conditions at excavation grade were excellent and isolated small cavities were grouted. A detailed discussion of the karst setting, identified karst features, and the potential for karst-related ground failure at the site is presented in Subsection 2.5.4.1.

- Earthquake activity with its resulting ground motion effects is judged to be a potential geologic hazard to the BLN site. The potential for tectonic surface deformation is judged to be negligible. Detailed discussions of vibratory ground motion and the potential for surface faulting at the BLN site are provided in Subsections 2.5.2. and 2.5.3, respectively. Historical and large earthquakes that could affect the site are presented in Subsection 2.5.2.2, and were factors for development of the site ground motion response spectrum.

- Slope failure, or landsliding is judged not to be a hazard to the site, except on the steep slopes of River Ridge. Slopes adjacent to Units 3 and 4 are generally flat except on the eastern boundary where very gentle slopes exist (Figure 2.5-230). Existing slopes in and proximal to the Units 3 and 4 power block construction zone exhibit no evidence of landslides, nor would landslides be expected to affect the safety-related structures, given the low slope angles. Small landslides do occur on the steeper slopes of River Ridge beneath outcrops of the Fort Payne Chert, but these slopes are directed eastward away from any safety-related structures. An assessment of the stability of slopes is presented in Subsection 2.5.5.

- Subsidence from human activities is judged not to be a hazard to the site. No groundwater withdrawal, petroleum production, or subsurface mining operations that could lead to subsidence are located near the site. There is no evidence of past subsurface mining activities at or near the BLN site. Coal mining in the region is primarily focused on the Pottsville Formation and occurs well to the northwest of the site (Reference 321). Quarry operations that remove limestone and chert from several locations along the Sequatchie Valley do not affect the BLN site. The closest quarries are located 2 to 5 mi. from the site (Reference 327).

- No significant weak zones of rock or adverse weak beds or zones of alteration occur within the bedrock. The Units 3 and 4 power blocks are underlain by hard, gently dipping, bedded limestone and argillaceous, silty limestone. Thin clay seams encountered in the Units 1 and 2 excavation were treated with conventional foundation preparation. Rock strength is documented in Subsection 2.5.4.2, and are evaluation of zones of alteration is made in Subsection 2.5.4.1.

- Zones of structural weakness, such as extensive fractured or faulted zones are not present, however, joints and bedding planes and minor shears are present. These discontinuities are judged to not be a hazard to the site. Subsection 2.5.5 describes the role of discontinuities in the stability of excavations and slopes.

- Ground failure and differential settlement due to liquefaction are judged not to be a hazard to the site. Residual soils are clayey silts and silty clays, soil types that are not typically susceptible to liquefaction. Detailed discussion of liquefaction potential is presented in Subsection 2.5.4.8.

- The effect of groundwater seepage into the excavation for the nuclear island is expected to be minor, and is judged not to be a hazard. Thorough evaluation of the groundwater conditions at the site is found in Subsections 2.4.12 and 2.5.4.6.

- No natural processes that might cause uplift are active at the site.

- No rocks or soils that might be unstable because of their mineralogy or their unstable physical or chemical properties are present, other than limestone which through dissolution hosts karst features.

- Unrelieved residual stresses are judged not to be a hazard at the site. Subsection 2.5.4.1.1 provides a discussion of residual stress.

These conclusions indicate that no geologic conditions were found at the site that would result in a hazard that could affect construction or operation of the safety-related facilities. It is judged that hazards due to karst weathering and earthquake activity can be satisfactorily mitigated using standard construction and engineering design methods.

2.5.2 VIBRATORY GROUND MOTION

BLN COL 2.5-2 This subsection of the referenced DCD is incorporated by reference with the following departures and/or supplements.

This subsection provides a detailed description of vibratory ground motion assessments, specifically the criteria and methodology for establishing the Ground Motion Response Spectra (GMRS) for the Bellefonte Nuclear Plant Units 3 and 4 (BLN) site. The section begins with a review of the approach in

Regulatory Guide 1.208, which satisfies the requirements set forth in Section 100.23, "Geologic and Seismic Siting Criteria," of Title 10, Part 100, of the Code of Federal Regulations (10 CFR 100), "Reactor Site Criteria." The GMRS for the BLN site was developed by adopting methodology consistent with the approach recommended in Regulatory Guide 1.208.

Following this introductory section, the remainder of the Subsection is presented as follows:

- Seismicity (Subsection 2.5.2.1)
- Geologic and Tectonic Characteristics of the Site and Region (Subsection 2.5.2.2)
- Correlation of Earthquake Activity with Seismic Sources (Subsection 2.5.2.3)
- Probabilistic Seismic Hazard Analysis (PSHA) and Controlling Earthquake (Subsection 2.5.2.4)
- Seismic Wave Transmission Characteristics of the Site (Subsection 2.5.2.5)
- Ground Motion Response Spectrum (GMRS) (Subsection 2.5.2.6).

Regulatory Guide 1.208 provides guidance on methods acceptable to the NRC to satisfy the requirements of the seismic and geologic regulation, 10 CFR 100.23, for assessing the appropriate Safe Shutdown Earthquake (SSE) ground motion levels for new nuclear power plants. Regulatory Guide 1.208 states that an acceptable starting point for this assessment at sites in the Central and Eastern United States (CEUS) is the Probabilistic Seismic Hazard Assessment (PSHA) conducted by the EPRI-SOG in the 1980s (References 203 and 233). The EPRI-SOG evaluation involved a comprehensive compilation of geological, geophysical, and seismological data, evaluations of the scientific knowledge concerning earthquake sources, maximum earthquakes, and earthquake rates in the CEUS by six multi-disciplinary teams of experts in geology, seismology, geophysics, and, separately, development of state of knowledge earthquake ground motion modeling, including epistemic and aleatory uncertainties. The uncertainty in characterizing the frequency and maximum magnitude of potential future earthquakes associated with these sources and the ground motion that they may produce was assessed and explicitly incorporated in the seismic hazard model.

Regulatory Guide 1.208 further specifies that the adequacy of the EPRI-SOG hazard results must be evaluated in light of more recent data and evolving knowledge pertaining to seismic hazard evaluation in the CEUS.

The GMRS was developed using the graded performance-based, risk-consistent method described in Regulatory Guide 1.208, *A Performance-based Approach to*

Define The Site-Specific Earthquake Ground Motion. The methodology for developing the GMRS is based on ASCE/SEI Standard 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities* (Reference 328). The method specifies the level of conservatism and rigor in the seismic design process such that the performance of structures, systems, and components of the plant achieve a uniform seismic safety performance consistent with the USNRC's safety goal policy statement. The ASCE/SEI Standard 43-05 approach is designed to achieve a quantitative safety performance goal (PF). The method is based on the use of site-specific mean seismic hazard and assumes that the seismic design criteria (SDC) and procedures contained in NUREG-0800 are applied in seismic source characterization (SSC) design.

The ASCE/SEI Standard 43-05 approach aims to conservatively assure a seismic safety target, or performance goal (PF), of mean 10^{-5} per year for SDC-5 SSCs. ANSI/ANS Standard 2.26-2004 Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design provides the criteria for selecting SDC and Limit State that establishes the Seismic Design Basis (SDB) for each SSC at a nuclear facility. The target mean annual performance goal for nuclear plants is achieved by coupling site-specific design response spectrum (DRS) with the deterministic SDC and procedures specified by NUREG-0800. The ASCE/SEI Standard 43-05 criteria for deriving a site-specific DRS are based on the conservative assumption that the SDC specified by NUREG-0800 achieve less than a 1 percent chance of failure for a given DRS. The conservatism of this assumption is demonstrated by analyses described in McGuire, et al. (Reference 329), which show plant level risk reduction factors ranging from about 20 to about 40 are attained by the USNRC's SDC. The method is based on the use of mean hazard results consistent with the recommendation contained in McGuire, et al. (Reference 329) and with the USNRC's general policy on the use of seismic hazard in risk-informed regulation.

2.5.2.1 Seismicity

BLN COL 2.5-1
BLN COL 2.5-2

The first step for evaluating seismic hazards at the BLN site involved an assessment of changes in seismicity for the site. The seismic history of the southeastern U.S. for the period from 1985 to present, as summarized in the existing earthquake catalogues, was evaluated to assess potential changes in the location, maximum magnitude, and frequency of earthquakes that could affect the BLN site. In addition, new information on historical earthquakes was identified and evaluated to update the existing information on the seismic setting of the BLN site. The development of an updated earthquake catalog for the project region is described in Subsection 2.5.2.1.1. Information on significant recent earthquakes and significant newly identified historical earthquakes is provided in Subsection 2.5.2.1.2. Figure 2.5-232 shows the combined independent earthquake catalog developed for this study. The earthquake catalog listing is provided in Appendix 2AA.

2.5.2.1.1 Earthquake Catalog

BLN COL 2.5-1
BLN COL 2.5-2

The data used to assess earthquake occurrence rates for the majority of seismic sources are the historical and instrumental earthquake record. An updated earthquake catalog of independent[b] earthquakes was developed for use in this study. This updated catalog was based on the independent earthquake catalog prepared for the 2004 TVA Dam Safety Seismic Hazard Assessment project (Reference 269) that hereafter, is referred to as the TVA Dam Safety catalog. The TVA Dam Safety catalog is a composite of the catalogs listed in Table 2.5-206, and covers the region from 31°N to 41°N and 75°W to 93°W. Additional catalogs that have become available after development of the TVA Dam Safety catalog have been evaluated and new information and new earthquakes have been incorporated to update the TVA Dam Safety catalog for the present study. The development of the TVA Dam Safety catalog is described in Subsection 2.5.2.1.1.1, and specific modifications to update that catalog for use in the BLN study are described in Subsection 2.5.2.1.1.2.

2.5.2.1.1.1 Development of the TVA Dam Safety Catalog

The TVA Dam Safety catalog was developed through comparisons of available earthquake catalogs covering the southeastern and central U.S. The initial catalog used in development of the seismic hazard mapping project was the updated independent earthquake catalog prepared by the USGS as part of their National Ground Motion Hazard Mapping project (hereafter referred to as the 2002 USGS catalog). The primary source for the 2002 USGS catalog is the NCEER-91 catalog (Reference 330) covering the period from 1627 to 1985. The NCEER-91 catalog was in turn based on the EPRI-SOG (Reference 203) catalog. In developing the 2002 USGS Catalog, the NCEER-91 catalog was supplemented with the catalogs from the Advanced National Seismograph System (ANSS), Southeast U.S. Seismic Network (SEUSSN), Center for Earthquake Research and Information (CERI), U.S. Geological Survey, Preliminary Determination of Epicenters (PDE), and Decade of North American Geology (DNAG), Mueller et al. (Reference 331); and Dr. Charles Mueller (References 267 and 332). The primary magnitude measure reported in these catalogs is body-wave magnitude m_b, which is considered to be equivalent to Nuttli magnitude, m_N, and to Lg-wave magnitude, m_{bLg}. The m_b values given in the NCEER-91 and the 2002 USGS catalogs were either converted from MMI (maximum Modified Mercalli Intensity) or MMI/FA (Felt

b. The PSHA formulation used in this study assumes that the temporal occurrence of earthquakes conforms to a Poisson process, implying independence between the times of occurrence of earthquakes. Thus, it is necessary to remove dependent events (such as foreshocks and aftershocks) from the earthquake catalog before estimating earthquake frequency rates. The only exceptions were modeling earthquake recurrence rates for seismic sources in the New Madrid and Charleston regions, where renewal models were used to estimate mean rates of occurrence of large earthquakes, and where (for New Madrid) a cluster model of multiple earthquake occurrences was used.

Area), or based on reliable instrumental magnitudes, in the order of increasing preference. Dependent earthquakes (i.e., foreshocks and aftershocks) were identified and removed from the catalog following the criteria of Gardner and Knopoff (References 267, 332, and 333). Additional information on the development of the 2002 USGS catalog (and its earlier 1996 version), including catalog, location, and magnitude authorities, conversion equations, treatment of significant earthquakes, is found in Mueller et al. (Reference 331).

The TVA Dam Safety catalog was developed from the 2002 USGS catalog as follows. All significant earthquake catalogs that covered parts or all of the area from 31°N to 41°N and 75°W to 93°W were obtained for comparison to the USGS 2002 catalog. These include catalogs from the EPRI (Reference 203), National Center for Earthquake Engineering Research (Reference 330), ANSS, SEUSSN, CERI, and National Earthquake Information Center (NEIC). The catalogs that were evaluated, area and time period of coverage, minimum magnitude considered, and source, are listed in Table 2.5-206. Each of these catalogs either excludes known non-tectonic events, such as mine blasts, collapses, reservoir-induced events, etc., or identifies them in the catalog listing. Additional listings of non-tectonic events (ANSS web site), known reservoir-induced events (Reference 334), and mine-related events (Reference 335) also were obtained for use in developing the TVA Dam Safety catalog.

Each of the additional catalogs was compared to the 2002 USGS catalog, and earthquakes that were not in the 2002 USGS independent catalog were identified. These events were evaluated for dependency with earthquakes in the 2002 USGS catalog. Essentially, all events listed in other catalogs, including NCEER-91, ANSS, SEUSSN, CERI, PDE, and DNAG, that were not in the 2002 USGS catalog, were judged to be dependent events (aftershocks or foreshocks), duplicate events, non-tectonic events, or were excluded because the magnitude was less than m_b 3.0 (the minimum magnitude of interest for developing earthquake occurrence parameters).

2.5.2.1.1.2 Modifications to the TVA Dam Safety Catalog for the BLN Catalog

The final BLN catalog covers a region (31° to 41°N and 75° to 93°W) extending more than 200 mi. in radius from the BLN site. This catalog was updated from the TVA Dam Safety catalog to include information on recent earthquakes, and to incorporate new information on location and magnitude of historical earthquakes and newly identified historical earthquakes. The specific sources of information that were used to update the TVA Dam Safety catalog are listed in Table 2.5-207. The new information includes data for 174 new historical earthquakes that are not included in the TVA Dam Safety catalog. A listing of the earthquakes in the BLN TVA, 2006 catalog is provided in Appendix 2AA. Newly discovered historic earthquakes added to the earthquake catalog come primarily from two sources: (1) a report on “new” historical earthquakes in the central U.S. by Metzger et al. (Reference 336), and (2) an unpublished listing of “new” historical earthquakes from locations throughout the study area compiled by the TVA in 2005 (Reference 337).

Metzger et al. (Reference 336) provided information for 103 newly identified earthquakes and 22 previously reported earthquakes occurring in the central U.S. during the time period from 1826 to 1899. Their information was obtained primarily through extensive review of microfilm records of historical newspapers. Metzger et al. (Reference 336) assessed moment magnitudes from MMI assessments following the approach of Johnston (Reference 297). The authors usually assigned epicenters near the center of highest intensity, although for some earthquakes, the epicenters were placed equidistant from lower intensity reports, or slightly closer to a town that reported aftershocks than another community with similar mainshock intensities but no reports of aftershocks.

Following the review of the historical earthquake catalog used for the TVA Dam Safety study and the preliminary updates to the catalog developed for the BLN study, the TVA (Reference 337) conducted research and developed information for additional new earthquakes occurring in the central and southeastern U.S. The TVA (Reference 337) study gathered new information on historic earthquakes primarily by performing keyword searches of online versions of historical newspapers. Keyword searches of online newspapers were used from the following sources:

- Ancestry.com (historic newspapers and Family and Local History sections (http://www.ancestry.com/search/);
- Brooklyn Daily Eagle (http://www.brooklynpubliclibrary.org/eagle/index.htm);
- Historic Missouri Newspaper Project (http://newspapers.umsystem.edu/Archive/skins/missourinavigator.asp?BP=OK&GZ=T&AW=1081184405843); and
- Colorado historical newspapers project (http://www.cdpheritage.org/newspapers/ about.html).

Other online listings of historical earthquakes consulted include:

- Pennsylvania earthquakes list (http://muweb.millersville.edu/~esci/geo/quake.html);
- Ohioseis list (http://www.dnr.state.oh.us/OhioSeis/html/eqcatlog.htm); and
- Maryland Geological Survey (http://www.mgs.md.gov/esic/brochures/earthquake.html).

The TVA review of historical online records resulted in identification of 152 “new” historic earthquakes during the time period from 1758 to 1923, as well as additional intensity reports and other information for several previously reported earthquakes.

The additional information for previously reported earthquakes from Bakun and Hopper (Reference 338) and TVA (Reference 337) was evaluated and was used to update the corresponding earthquake data. The data from Bakun and Hopper (Reference 338) includes estimates of moment magnitude and new assessments of the epicentral location of historical earthquakes based on a reassessment of the felt intensity data. The approach for estimating **M** from the intensity data appears robust and the estimates of **M** were judged to be appropriate for use in the project catalog.

Bakun and Hopper (Reference 338) use three approaches to select a new preferred epicentral location. The preferred epicentral locations that are based on the intensity data alone, either the isoseismal area or maximum intensity, also were judged to be more reliable than previous locations (which typically were based on fewer intensity reports), and were accepted for use in the BLN catalog. In the third approach, Bakun and Hopper (Reference 338) moved the preferred epicentral location to a known fault that is proximal to the intensity locations. This approach implies that a particular fault has ruptured, and in several cases, resulted in a significant relocation of the epicenter compared to the intensity data location (greater than 50 km [31 mi.]). Because this method for assigning the preferred epicentral location is based on a tectonic interpretation rather than based on the earthquake shaking data, it is not consistent with the methodology used to assign epicentral locations to other earthquakes in the catalog. Therefore, the initial location identified in Bakun and Hopper (Reference 338), which was based on intensity reports, was judged to be more appropriate and was adopted for use in updating the existing earthquake location. For several of these earthquakes, the preferred magnitude was adjusted to be consistent with the intensity-based epicentral location rather than the fault-based location.

With few exceptions, locations for the new earthquakes identified in TVA (Reference 337) were assigned to the town or city with the highest MMI or a point between two localities interpreted to have the same intensity. With the exception of a few earthquakes with multiple intensity reports, there is insufficient data at present to define felt areas for these newly identified earthquakes. Therefore, the primary information regarding the magnitude for the TVA (Reference 337) new earthquakes is the assumed maximum intensity.

For those earthquakes where a felt area could be identified, the intensity–area relationships of Johnston (Reference 297) were used to assign moment magnitude. For other earthquakes, maximum intensity data were used to develop estimates of m_b as follows. For new earthquakes from Reference 337, m_b was determined from the average of the following two relationships between maximum MMI and m_b magnitude and that used by Metzger et al. (Reference 336):

$m_b = 0.61 \times MMI_{max}+0.78$ (Reference 339) (2.5.2-1)

$m_b = 2.37+0.0466 \times (MMI_{max})^2$ (Reference 340) (2.5.2-2)

Metzger et al.'s (Reference 336) maximum intensity-based magnitudes were expressed as moment magnitudes. Because these moment magnitudes (developed from the Johnston (Reference 297) relationship between MMI and **M**) appear to be systematically higher than the moment magnitudes resulting from conversion of m_b (developed from the Venziano and Van Dyck (Reference 339), and Sibol et al. (Reference 340) equations) to **M** (based on Woods (Reference 341)), the magnitudes for the Metzger et al. (Reference 336) maximum MMI-based earthquakes were re-calculated as follows. The Metzger et al. (Reference 336) MMI-based moment magnitudes were converted to m_b by adding 0.36 units based on Woods (Reference 341) relationship between moment magnitude and m_b. In addition, new estimates of m_b were developed for each of these earthquakes using the two MMI – m_b equations listed previously. The average of the three m_b estimates is assigned as the best estimate of m_b for these earthquakes.

The resulting combined list of new earthquakes was reviewed for inclusion in the BLN project catalog as follows. Earthquakes that had an m_b of less than 3.0 or that were located outside of the area of the original TVA Dam Safety (31° to 41°N and 75° to 93°W) were excluded. The remaining new historical earthquakes were then reviewed to assess possible dependencies with other earthquakes in the existing catalog, and those that were confidently identified as dependent events were excluded. After removal of the dependent events, a total of 174 new historical earthquakes are included in the BLN catalog (all occurring between 1758 and 1923). In addition, 10 earthquakes occurring in 2004 and early 2005 were included in the BLN catalog. The BLN catalog is presented in Appendix 2AA. It presents epicenter coordinates, depth of focus, date, origin time, highest intensity, seismic moment, source mechanism, dimension and distance from the BLN site.

2.5.2.1.2 Recent Earthquakes and Historical Seismicity

The locations of newly identified historical earthquakes (pre-March 1985), and earthquakes occurring since March 1985 (post-EPRI-SOG, Reference 203) are compared to the spatial distribution of earthquakes included in EPRI-SOG evaluation in Figures 2.5-233 and 2.5-234, respectively. These figures show that there are no major differences in the spatial pattern of earthquakes for these three data sets. As noted in EPRI-SOG (Reference 203), the Charleston, South Carolina, New Madrid, and ETSZ are identified as the most seismically active zones in the central and southeastern U.S.

2.5.2.1.2.1 Recent Earthquakes

Three earthquakes of note (magnitude greater than m_{bLg} 4.0) have occurred within 200 mi. of the BLN site in the period post-1985. These are the March 27, 1987, Vonore, Tennessee, earthquake, the April 29, 2003, Fort Payne earthquake, and the 2004 earthquake near Braggville in west-central Alabama (Figure 2.5-234). Information on these earthquakes is summarized as follows and included in Appendix 2AA. Additional, previously identified significant historical earthquakes are described in the Units 1 and 2 FSAR (Reference 201).

March 27, 1987 m_{bLg} 4.2 (m_b 4.3) Vonore, Tennessee

The Vonore, Tennessee, earthquake occurred in eastern Tennessee approximately 32 mi. south of Knoxville. The USGS (Earthquake Hazards Program, U.S. Geological Survey Earthquake Search—Rectangular Area, http://wwwneic.cr.usgs.gov/neis/epic/epic_rect.html) lists the magnitude as m_{bLg} 4.2 and m_b 4.3. Minor damage, including cracked cinderblock walls, foundations, and chimneys, was reported over an 800-km^2 (309-sq. mi.) area, and the maximum MMI was VI (Reference 342). These authors also noted that the earthquake may have caused ground fissures along a ridge near Wellsville, but that the nature and time of origin of these features could not be conclusively determined. Focal mechanism solutions and the locations of aftershocks indicate that the earthquake occurred by right-lateral strike-slip on a north-south trending subvertical fault (Reference 342).

April 29, 2003 m_{bLg} 4.9 (M 4.6) Fort Payne, Alabama

The Fort Payne earthquake occurred in Dekalb County, in north-easternmost Alabama, near the Georgia border (Reference 343). The earthquake has a measured Lg wave magnitude (m_{bLg}) of 4.9 and a moment magnitude (**M)** of 4.6. The Fort Payne earthquake occurred at the southern end of the ETSZ, and is one of the strongest earthquakes to have occurred in Alabama and in the ETSZ in historic time. The earthquake, and the October 24, 1997, m_{bLg} 4.9 Escambia earthquake[c], are the two largest earthquakes to have occurred in the southeastern U.S. since 1985.

The Fort Payne earthquake caused minor damage, including damage to chimneys, cracked walls and foundations, broken windows, and collapse of a 9-m (29-ft.)-wide sinkhole. These examples of damage, and other reports of shaking correspond to a maximum MMI of VI (References 344 and 345). Based on reconnaissance in the epicentral area, no landslides were reported, and damage to chimneys was observed only for chimneys with masonry in poor/weakened condition. Other masonry, including chimneys in good condition, and several old masonry buildings did not appear to be damaged.

c. The Escambia earthquake occurred in southernmost Alabama at a distance greater than 200 mi. from the BLN site. Therefore, this earthquake is not considered in this analysis.

Studies of the earthquake focal mechanism indicate that the focal planes are subvertical and strike approximately north-south and east-west (Reference 343). The earthquake occurred at a depth of about 9.5 to 13 km (5.9 to 8.1 mi.) based on studies of the compression (P) waves (Reference 346).

Strong motion instruments located on the crests of Buford and Carters Dams in western Georgia, at distances of about 85 and 145 km (52.8 to 90.1 mi.), respectively, from the epicenter were triggered, possibly due to amplification in the earth embankment dams. The free-field (ground surface) and abutment instruments at both dams were not triggered (Reference 347). There are no strong motion instruments at the TVA Guntersville Dam near the BLN site. Strong ground shaking at the Sequoyah Nuclear Plant in eastern Tennessee apparently was slightly below the triggering threshold for the instrument.

November 7, 2004 m_b 4.4 (M 4.3) Braggville, Greene County, Alabama

An m_b 4.4 earthquake occurred near Braggville, in Greene County, central-western Alabama on November 7, 2004. The maximum MMI was V (Reference 344). Focal depth was shallow (3 km) and the focal mechanism indicated predominantly normal slip on NNE striking planes (Reference 343).

2.5.2.1.2.2 Historical Earthquakes

Several newly identified moderate magnitude (m_b > 4) historical earthquakes that occurred within 200 mi. of the BLN site are included in the updated catalog (Figure 2.5-233). The available information about these earthquakes is summarized as follows.

January 3, 1861 m_b 4.3 (M 3.9) North Carolina/Georgia Border Region

An earthquake of MMI V and m_b 4.3 (based on felt area) occurring on January 3, 1861, along the North Carolina/Georgia border region is identified as a new earthquake listing in NCEER-91 (Reference 330). This earthquake may correspond to an EPRI-SOG (Reference 203) listing for 1861 (no date) of m_b 2.5 and located about 130 km (80.1 mi.) north of the NCEER-91 location.

November 30, 1862 m_b 4.8 (M 4.7) Western Tennessee

An earthquake that was felt throughout the northern Mississippi Valley occurred on the morning of November 30, 1862. Metzger et al. (Reference 336) compiled newspaper reports and noted that MMI III effects were reported from Louisville, Kentucky, and St. Louis, Missouri; MMI IV effects were reported for Evansville, Illinois, and Cairo, Illinois; and MMI V effects were reported for Memphis, Tennessee. Metzger et al. (Reference 336) locates the epicenter in western

Tennessee between Memphis and Cairo, and assign a moment magnitude (**M**) of 4.7 to this earthquake.

March/April, 1874 m_b 4.4 (M 4.0) Williamson, Tennessee

An earthquake that was felt in Williamson County, Tennessee, was reported in the Franklin, Tennessee, Brooklyn Eagle newspaper on April 9, 1874 (Reference 337). The earthquake produced a large landslide, but the exact date of the earthquake was not reported. The maximum intensity was estimated as MMI VI to VII, however, because no additional reports of shaking were identified, the assigned magnitude is limited to m_b 4.4 (Reference 337), and it is possible that this is an overestimate of actual magnitude. Alternatively, the landslide may not have been of tectonic origin.

September 18, 1881 m_b 4.5 (M 4.2) Newnan, Georgia

An earthquake that was felt in Newnan, Georgia, late in the evening on September 17, 1881, (local time) was reported in the Atlanta Journal Constitution on September 20, 1881 (Reference 337). The shaking lasted for about 10 seconds, rattling houses and causing people to run outside. The maximum intensity was estimated as MMI VI, corresponding to m_b 4.5 (Reference 337).

October 5, 1899 m_b 4.4 (M 4.0) Smoky Mountains (Tennessee/North Carolina border)

A “severe shock” was felt in the Smoky Mountains along the Tennessee/North Carolina border early in the morning on October 5, 1899. As reported in the Fort Wayne News on October 6, 1899, the earthquake lasted for more than 10 seconds and caused an opening for several hundred feet along Abrams Creek (Reference 337). Although the local effects indicate a strong earthquake, no reports of this earthquake from surrounding regions (such as Knoxville, Tennessee) were identified. The maximum intensity is uncertain, and is estimated as MMI V to VIII. Because no reports of shaking were identified outside of the local area, the assigned magnitude is limited to m_b 4.4 (Reference 337).

June 9, 1910 m_b 4.2 (M 3.9) Dalton, Georgia

An earthquake that was felt in Dalton, Georgia, early in the evening on June 9, 1910 (local time) was reported in the Atlanta Journal Constitution on June 10, 1910 (Reference 337). The shaking lasted for a few seconds, shaking houses and causing people to run outside. The maximum intensity was estimated as MMI V to VI, corresponding to m_b 4.2 (Reference 337)

2.5.2.2 Geologic and Tectonic Characteristics of the Site and Region

BLN COL 2.5-2 As discussed previously, Regulatory Guide 1.208 specifies that recent information should be reviewed to evaluate if this information would indicate significant differences from the previous seismic hazard. Subsection 2.5.1.1.4 presents a summary of available geological, seismological, and geophysical data for the site at varying levels of detail; 320-km (200-mi.) radius, 40-km (25-mi.) radius, and 8-km (5-mi.) radius that provide the basis for evaluating seismic sources that contribute to the seismic hazard to the BLN site. This section presents a description of the seismic source characterizations from the EPRI-SOG (Reference 203) evaluation followed by a summary of general approaches and interpretations of seismic sources used in more recent seismic hazard studies. Subsections 2.5.2.4.1 and 2.5.2.4.3 present an evaluation of the new information relative to the EPRI-SOG (Reference 203) seismic source evaluations.

2.5.2.2.1 EPRI-SOG Source Evaluations

The EPRI-SOG evaluation completed in the late 1980s (Reference 203) involved assessments of the uncertainty in seismic source characterization in the CEUS by formal elicitation of six independent Earth Science Teams. The six teams were the Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants. Each team evaluated geologic, geophysical, and seismological data to evaluate seismic sources in the CEUS and provided detailed documentation of their assessments in separate volumes of the EPRI-SOG (Reference 203) evaluation. In the EPRI-SOG (Reference 203) evaluation, tectonic features that might be seismogenic were identified, and their probability of activity was assessed. The study first identified and defined criteria for assessing the activity of a feature. These criteria include attributes such as spatial association with large- or small-magnitude earthquakes, evidence of geologically recent slip, orientation relative to the regional stress regime, and others. The study also assigned a relative weight or relative value of each criterion in assessing the probability of activity. The seismic sources interpreted from the tectonic features (i.e., “feature-specific source zones”) were assigned a probability of activity equivalent to that of the features.

The seismic source evaluations were one element of the seismic hazard model inputs for a PSHA for nuclear plant sites in the CEUS (Reference 233). For the computation of hazard in the 1989 study, some of the seismic source parameters were modified or simplified from the original parameters defined by the EPRI-SOG (Reference 203) evaluation. The parameters used in final PSHA calculations are summarized in EPRI (Reference 233), which is the primary source for the EPRI-SOG seismic hazard model used in this study.

The seismic sources defined by each of the teams relative to the updated seismicity are shown in Figures 2.5-235, 2.5-236, 2.5-237, 2.5-238, 2.5-239, and 2.5-240. A screening criterion was implemented in the EPRI (Reference 233) seismic hazard calculations in that all sources with combined hazard less than

1 percent of the total hazard were excluded from the analysis. The sources that contributed 99 percent of the hazard at the BLN site are shown and labeled in these figures. The smaller inset figures show the complete set of seismic sources identified in the BLN site by each of the EPRI-SOG teams.

Tables 2.5-208A, 2.5-208B, 2.5-208C, 2.5-208D, 2.5-208E, and 2.5-208F summarize the significant sources that were included in the EPRI (Reference 233) seismic hazard analysis for the BLN site and list additional sources within the 200-mi.-radius that do not significantly contribute to the hazard at the site. The EPRI (Reference 233) evaluation indicated that the most significant contributors to hazard at the BLN site are the ETSZ, subdivisions of the crust around the ETSZ, and the New Madrid, Missouri, region (location of the 1811-1812 earthquakes). In addition, there is a minimal contribution from the Charleston, South Carolina, region sources (location of the 1886 earthquake).

2.5.2.2.2 LLNL-TIP Source Evaluations

A decade after the completion of the EPRI-SOG (Reference 203) evaluation, LLNL (Reference 234) conducted a Trial Implementation Project (TIP) of the SSHAC (Reference 235) guidance for a Level IV analysis. SSHAC (Reference 235) provides general guidance for conducting PSHA for important facilities and describes four levels of effort for quantifying epistemic uncertainty ranging from assessments by a single individual (Level I) to formalized elicitation of a panel of experts (Level IV). The EPRI-SOG (Reference 203) evaluation can be considered the prototype of a SSHAC Level IV study. The LLNL-TIP project focused on issues related to the development of seismic zonation and earthquake recurrence models. Participants in the project included a Technical/Facilitator/Integrator (TFI) team, a panel of five expert evaluators, and expert proponents and presenters. Preliminary implementations for two sites in the southeastern U.S., the Vogtle site in Georgia, which is affected by the issue of the Charleston earthquake, and the Watts Bar site in Tennessee, which is close to the ETSZ, were completed as part of the TIP study. Although focused primarily on process, the LLNL TIP study provided assessments for some of the seismic sources significant to the BLN site.

Seismic source models were developed for each of the five experts and through discussions at workshops, one-on-one interviews, and white papers, a set of common sources was identified as the basic building blocks for all the sources and alternative sources. The general boundaries of these common sources are shown in Figure 2.5-241. This minimum set of zones was then used to create the composite model of seismic sources that represented the range of feasible sources (Table 2.5-209). These sources included five basic alternative zones for both the East Tennessee and Charleston sources, three for the South Carolina-Georgia seismic zone, and alternative zones for background earthquakes for both the East Tennessee and Charleston regions. The probability of activity was defined as the probability of “existence” of a particular source zone.

2.5.2.2.3 2002 USGS Earthquake Hazard Mapping Source Characterization Model

As part of the 2002 USGS National Seismic Hazard Mapping program, updated seismic hazard maps for the conterminous U.S. were produced in 2002 (Reference 348). Input for revising the source characterization used in the 1996 hazard maps (Reference 349) was provided by researchers through a series of regional workshops. Key issues that were addressed in the updated source characterization included new information regarding the location, size, and recurrence of repeating large magnitude earthquakes in the Charleston and New Madrid source regions. Although the USGS program does not use formal expert elicitation and full uncertainty quantification, the resulting seismic hazard model provides information on the current understanding of the seismic potential of the study region and the catalog of recorded earthquakes.

The USGS source model and earthquake catalog (in body wave magnitude, m_b) developed by the USGS (Reference 331) are shown in Figure 2.5-242. The general approach used by the USGS for modeling distributed seismicity in the CEUS is to use a Gaussian kernel smoother to define the spatial distribution of future earthquakes based on the recorded locations of past earthquakes throughout the CEUS. No boundaries are placed on the locations of ruptures associated with the spatially smoother earthquake locations.

Two broad regions are defined with different maximum magnitudes in the USGS model: an extended margin zone (M_{max} =**M** 7.5) and a craton zone (M_{max} =**M** 7.0). In addition, the USGS source model includes an East Tennessee regional source zone, alternative fault line sources for repeating large magnitude earthquakes in the NMSZ, and alternative zones for a Charleston seismic source zone. The maximum magnitude probability distribution assigned to the New Madrid fault sources is **M** 7.3 (0.2), **M** 7.5 (0.2), **M** 7.7 (0.5), **M** 8.0(0.15)**.** For the Charleston source, the maximum magnitude probability distribution used was: **M** 6.8 (0.2), **M** 7.1 (0.2), **M** 7.3 (0.45), **M** 7.5 (0.15). The USGS model uses a mean recurrence time of 500 years and 550 years for repeating large magnitude earthquakes in the New Madrid and Charleston regions, respectively, and assumes a time-independent model.

2.5.2.2.4 2004 TVA Dam Safety Seismic Hazard Analysis Seismic Source Model

In 2004, Geomatrix Consultants completed regional and site-specific dam safety seismic hazard assessments for all of the TVA's major dams (Reference 269). As part of this study, Geomatrix developed a probabilistic seismic hazard model for the Tennessee Valley using a SSHAC Level II process. The project team was assisted by participatory review by an external peer review panel.

The study emphasized explicit incorporation of epistemic uncertainty through the use of logic trees in the PSHA. The source characterization effort was based on a review of published literature and discussion with active researchers. The study built upon previous studies including the EPRI-SOG (Reference 203) evaluation, the LLNL-TIP study (Reference 234), the USGS National Seismic Hazard Project (Reference 348), an EPRI-sponsored study to assess maximum magnitudes of earthquakes in stable continental regions (Reference 268), and the EPRI (Reference 350) CEUS ground motion project.

The seismic source model developed for the TVA Dam Safety study includes two types of sources, distributed seismicity sources and "fault-specific" sources of repeating large magnitude earthquakes. Two approaches were used to model the distributed seismicity sources: a zoneless approach similar to that used by the USGS to develop the 2002 hazard maps, and a seismotectonic zonation approach. Spatial smoothing of seismicity was employed in both approaches. Figures 2.5-243a, 2.5-243b, 2.5-243c and 2.5-243d show the alternative seismotectonic source zones defined by Geomatrix (Reference 269) in the vicinity of the BLN site.

"Fault-specific" sources were used to model repeating large earthquakes that have been identified in two specific regions, near Charleston, South Carolina, and the New Madrid region at the junction of Missouri, Kentucky, and Tennessee. The fault-specific sources were modeled as having some probability of being non-Poissonian processes, i.e. with time dependence.

2.5.2.3 Correlation of Earthquake Activity with Seismic Sources

As described in Subsection 2.5.2.2.1, the EPRI-SOG evaluation (Reference 203) examined the distribution of seismicity and grouped it into spatially delineated source zones using multiple criteria. The principal database for assessing earthquake activity is the historical and instrumental earthquake record. An updated catalog (Appendix 2AA) of independent historical and instrumental earthquakes covering the BLN site was developed for use in the BLN study (see discussion in Subsection 2.5.2.1.1).

The distribution of earthquake epicenters from the EPRI (pre-1985), the more recent (post-1985) instrumental events, and updated historical earthquakes for the site are shown in Figures 2.5-232, 2.5-233, and 2.5-234. Comparison of the updated earthquake catalog to the EPRI earthquake catalog yields the following conclusions:

- The updated catalog does not show any earthquakes within the site that can be associated with a known geologic structure. As described in Subsection 2.5.1, the majority of seismicity in the BLN site appears to be occurring at depth within the basement beneath the Appalachian décollement. The largest earthquake within a 25-mi. radius of the site, the 2003 **M** 4.6 Fort Payne earthquake which is likely a reactivated structure within the basement rock, but cannot be clearly associated with any of the major identified basement structures (Subsection 2.5.1.1.4.2.4.2); or mapped faults (Figure 2.5-294).

- The updated earthquake catalog (Appendix 2AA) adds several magnitude m_b 3 to 5 earthquakes in the time period covered by the EPRI-SOG catalog (principally prior to 1910). The effect of these additional events on estimated seismicity rates is assessed in Subsection 2.5.2.4.1.2.

- The updated earthquake catalog does not show a pattern of seismicity different from that exhibited by earthquakes in the EPRI-SOG catalog that would suggest a new seismic source in addition to those included in the EPRI-SOG characterizations.

- The updated earthquake catalog shows similar spatial distribution of earthquakes to that shown by the EPRI-SOG catalog, suggesting that no significant revisions to the geometry of seismic sources defined in the EPRI-SOG characterization is required.

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

This Subsection describes the PSHA conducted for the BLN site. Following the procedures outlined in Regulatory Guide 1.208, Subsections 2.5.2.4.1 and 2.5.2.4.2 discuss the significance of new information on seismic source characterization and ground motion characterization, respectively, that are potentially significant relative to the EPRI (Reference 233) seismic hazard model. Subsection 2.5.2.4.3 presents the results of PSHA sensitivity analyses used to test the impact of the new information on the seismic hazard. Using these results, an updated PSHA analysis was performed, as described in Subsection 2.5.2.4.4. The results of that analysis are used to develop uniform hazard response spectra (UHRS) and the identification of the controlling earthquakes (Subsection 2.5.2.4.4.5).

2.5.2.4.1 New Information Relative to Seismic Source Evaluations

Several factors may produce changes in the level of seismic hazard at the BLN site compared to what would be estimated based on the EPRI (Reference 233) evaluation. Seismic source characterization data and information that could affect the predicted level of seismic hazard include:

- Identification of a possible new seismic source in the site vicinity

- Changes in the characterization of the rate of earthquake occurrence for one or more seismic sources

- Changes in the characterization of the maximum magnitude for seismic sources.

2.5.2.4.1.1 Identification of Seismic Sources

Based on the review of new geological, geophysical, and seismological information that is summarized in Subsection 2.5.1, review of seismic source characterization models developed for post-EPRI seismic hazard analyses (Subsection 2.5.2.2), and comparison of the updated earthquake catalog to the EPRI evaluation (Subsection 2.5.2.3), no additional specific seismic sources have been identified.

As described in Subsection 2.5.1.1.4.2, additional information and analysis of subsurface data (e.g., industry seismic reflection profiles, deep wells) and seismicity data provides an improved understanding of structures within the BLN site 200-mi. radius, particularly with regard to the foreland Appalachian fold-thrust belt and possible relationships to subdetachment basement faults and zones of concentrated seismicity (e.g., ETSZ). However, the overall pattern of seismicity occurring on structures within the basement below the detachment was recognized at the time of the EPRI evaluation and the EPRI expert teams specified a variety of source geometries to represent the uncertainty in defining the source zone configurations.

Figures 2.5-244a and 2.5-244b compare the range of source zone geometries defined in the vicinity of the site by the EPRI expert teams (Figure 2.5-244a) and in subsequent studies (Figure 2.5-244b). The recent April 2003 **M** 4.6 Fort Payne earthquake is located at the southern extent of the concentrated seismicity that defines the ETSZ and is typical in both depth and focal mechanism to other earthquakes in the zone. The 2003 Fort Payne earthquake occurred just outside of the boundary of the East Tennessee seismic source zones defined by three of the EPRI expert teams and lies within the East Tennessee source zones (ETSZ) defined by the other three teams. This is also the case for more recent interpretations. The LLNL TIP (Reference 234) ETSZ does not include the 2003 Fort Payne earthquake (Table 2.5-209), but the USGS East Tennessee regional source zone (Reference 348) and alternative source zones included in the Geomatrix (Reference 269) hazard analysis do include the 2003 event. Therefore, the EPRI source zone interpretations are judged to adequately represent interpretations of the ETSZ.

The EPRI expert teams confined the location of events similar to the 1811-1812 earthquakes to the region of concentrated seismicity in the NMSZ. More recent seismic hazard analyses (e.g., References 269, 348, 349, and 351) also restrict the occurrence of similar size events to this region, often placing the events on fault-specific sources within the NMSZ. Thus, no modification of the EPRI New Madrid source configurations is needed. The more recent data have suggested more frequent occurrences for these events, as discussed in Subsection 2.5.2.4.1.2.

Seismic sources defined by the EPRI expert teams to represent possible locations for repeats of the 1886 Charleston earthquake were typically not included in the EPRI (Reference 233) hazard calculation for the BLN site because their contribution to the hazard was very small (< 1 percent). More recent data regarding the location and timing of repeating large magnitude earthquakes in the vicinity of Charleston, South Carolina, suggest alternative source configurations that fall within the range of EPRI source zone interpretations and (similar to New

Madrid), more frequent occurrence of these events. These new interpretations are considered in this study (see Subsections 2.5.2.4.3 and 2.5.2.4.4).

2.5.2.4.1.2 Earthquake Recurrence Rates

Subsection 2.5.2.1.1 describes the development of an updated earthquake catalog for the BLN project for a 200-mi. radius. This updated catalog includes modifications to the EPRI-SOG evaluation by subsequent researchers, the addition of earthquakes that have occurred after completion of the EPRI-SOG evaluation development (post March 1985), and identification of additional earthquakes in the time period covered by the EPRI-SOG evaluation for the project region (1758 to March 1985). The impact of the new catalog information was assessed by evaluating the effect of the new data on earthquake magnitude estimates and on earthquake recurrence estimates within the 200-mi. radius around the BLN site.

The earthquake recurrence rates computed in the EPRI-SOG (Reference 203) evaluation included a correction to remove bias introduced by uncertainty in the magnitude estimates for individual earthquakes. The bias adjustment was implemented by defining an adjusted magnitude estimate for each earthquake, m_b*, (Reference 339) and then computing the earthquake recurrence parameters by maximum likelihood using earthquake counts in terms of m_b*. The adjusted magnitude is defined by the relationship

$$m_b{}^* = m_b - \beta\sigma^2_{m_b|m_b\mathrm{instrumental}}/2 \qquad (2.5.2\text{-}3)$$

when m_b is based on instrumentally recorded m_b magnitudes and by the relationship

$$m_b{}^* = m_b + \beta\sigma^2_{m_b|X}/2 \qquad (2.5.2\text{-}4)$$

when m_b is based on other size measures *X*, such as maximum intensity, I_0, or felt area. The change in sign in the correction term from negative in Equation (2.5.2-3) to positive in Equation (2.5.2-4) reflects the effects of the uncertainty in the conversion from size measure *X* to m_b. Parameter ß is the Gutenberg-Richter *b*-value in natural log units. Values of the adjusted magnitude m_b* were computed for the earthquakes in the updated catalog using the assessed uncertainties in the magnitude estimates and a value of ß equal to 0.95×ln(10) based on the global *b*-value of 0.95 assigned to the CEUS by Frankel et al. (References 348 and 349). Values of $\sigma_{m_b|X}$ range from 0.56 for m_b estimated from maximum intensity, to 0.2 to 0.3 for m_b estimated from various measures of felt area, $\sigma_{m_b|m_b\mathrm{instrumental}}$ is typically set at 0.1. Figure 2.5-245 shows a histogram of the difference between the values of m_b* for the updated catalog and those given in the EPRI-SOG (Reference 203) evaluation for earthquakes within 200 mi. of the BLN site. The mean difference is essentially zero and the distribution of differences is relatively symmetric.

The EPRI-SOG (Reference 203) procedure for computing earthquake recurrence rates was based on a methodology that incorporated data from both the period of complete catalog reporting and the period of incomplete catalog reporting. For the period of incomplete reporting, a probability of detection, P^D, was defined that represented the probability that the occurrence of an earthquake would ultimately be recorded in the earthquake catalog for the region (Reference 339). The CEUS was subdivided into 13 “Completeness” regions that represented different histories of earthquake recording. Figure 2.5-246 shows the two completeness regions (3 and 4) that cover the area with 200 mi. of the BLN site.

The updated earthquake catalog includes a number of newly identified earthquakes for the time period covered by the EPRI catalog, reassessment of the sizes of previously identified events, and earthquakes that have occurred after completion of the EPRI evaluation. The event counts for the EPRI and updated catalogs are given in Table 2.5-210.

Most of the newly identified earthquakes within 200 mi. of the BLN site occurred in time periods identified in the EPRI evaluation as periods of incomplete catalog reporting ($P^D < 1.0$). Comparisons of the earthquake counts for these time periods suggest that inclusion of the newly identified earthquakes in the estimation of catalog completeness would likely yield values of P^D near unity for the period post 1860 within completeness regions 3 and 4 for the two lowest magnitude intervals: $3.3 \geq m_b^* > 3.9$ and $3.9 \geq m_b^* > 4.5$.

Figure 2.5-247 shows “Stepp” plots for the portions of EPRI-SOG (Reference 203) completeness regions 3 and 4 that lie within 200 mi. of the BLN site. The plot on the left shows the time variation of earthquake occurrence rates based on the EPRI-SOG catalog, and the plot on the right shows the occurrence rates based on the updated catalog. The observed rate of magnitude m_b 3.3 to 3.9 earthquakes begins to steadily decrease for times greater than 15 years before the end of the EPRI-SOG catalog (times before 1970) and the rate for m_b 3.9 to 4.5 earthquakes begins to decrease for times greater than 75 years before the end of the EPRI-SOG catalog (times before 1910). In contrast, the occurrence rates remain relatively constant back to approximately 1860 for these two magnitude intervals using the updated catalog. The time variation of the rate for earthquakes larger than m_b 4.5 shows somewhat erratic behavior due to the limited number of events.

The effect of the updated earthquake catalog on earthquake occurrence rates was assessed by computing earthquake recurrence parameters for the portions of EPRI completeness regions 3 and 4 that lie within 200 mi. of the site. The truncated exponential recurrence model was fit to the seismicity data using maximum likelihood. Earthquake recurrence parameters were computed using the EPRI catalog and equivalent periods of completeness and using the updated catalog and the updated equivalent periods of completeness. It was assumed that the probability of detection for all magnitudes is unity for the time period of March 1985 to March 2005. The resulting earthquake recurrence rates are compared in Figure 2.5-248. For completeness region 3, essentially the same earthquake recurrence parameters are obtained using the EPRI and updated catalog and equivalent periods of completeness. For completeness region 4, use of the updated earthquake catalog and equivalent periods of completeness result in lower earthquake occurrence rates.

On the basis of the comparisons shown in Figure 2.5-248, it is concluded that the earthquake occurrence rate parameters developed in the EPRI evaluation adequately represent the seismicity rates within 200 mi. of the BLN site based on more recent information.

The earthquake recurrence rate for the New Madrid and Charleston regions was also evaluated using results of paleoliquefaction studies. The results of studies of paleoliquefaction in the NMSZ (summarized in Subsection 2.5.1.1.4.3) have indicated that large earthquakes are more frequent than suggested by extrapolating the observed seismicity rates for small-to-moderate earthquakes up to large magnitudes ($m_b \geq 7$). Figure 2.5-249 compares the seismicity rates estimated from the updated earthquake catalog to the rate for large magnitude events estimated from paleoliquefaction data. The error bars attached to the updated catalog rates represent 90 percent confidence intervals estimated by relative likelihood from the observed earthquake counts within the Bechtel team source zone 30 (Figure 2.5-235), a typical EPRI New Madrid source. The hatched box represents the 90 percent confidence interval for the paleoliquefaction rate based on three earthquake sequences post 300 AD (e.g., Reference 352) and the solid circle indicates the rate used by Frankel et al. (Reference 348) in the USGS National Hazard Mapping project (500-year repeat time). The recurrence relationships shown in the figure indicate the mean and 15th to 85th percentile recurrence rates for New Madrid sources computed from the EPRI seismic source models. As shown in the figure, the EPRI recurrence rates are very consistent with the seismicity rates estimated from the updated earthquake catalog but underestimate the rate for large earthquakes based on paleoliquefaction data by approximately an order of magnitude. Based on a similar comparison, Exelon (Reference 294) concluded that the EPRI recurrence rates for large earthquakes in the NMSZ should be revised for PSHA calculations.

As discussed in Subsection 2.5.1.1.4.3, paleoliquefaction studies also have been conducted in the region of the 1886 Charleston, South Carolina, earthquake. The results of these studies have led to estimated repeat times for large earthquakes in the Charleston region of approximately 550 years (References 269, 348 and 349). This repeat time represents higher occurrence rates than obtained from the EPRI seismic hazard model (Reference 353).

2.5.2.4.1.3 Assessment of Maximum Magnitude

The four types of seismic sources that contribute to the hazard at the BLN site are (1) representations of the ETSZ, (2) the local host/background zone, (3) representations of the NMSZ, and (4) to a very minor extent, sources representative of the 1886 Charleston earthquake. Figures 2.5-250, 2.5-251, 2.5-252, and 2.5-253 show the maximum magnitude distributions for these sources. The top plot in each figure shows the composite of the distribution developed by the EPRI (Reference 233) expert teams in terms of the m_b magnitude scale, the magnitude scale used in the EPRI seismic hazard model. The bottom plot in each figure compares the composite EPRI maximum magnitude distribution to more recent assessments. These latter comparisons are made in terms of the moment magnitude scale, **M**. The composite m_b distributions were converted to moment magnitude using three equally weighted m_b – **M** relationships: by Reference 233,

$$m_b = -10.23 + 6.105\mathbf{M} - 0.7632\mathbf{M}^2 + 0.03436\mathbf{M}^3 \qquad (2.5.2\text{-}6)$$

by Atkinson and Boore (Reference 354)

$$\begin{aligned} \mathbf{M} &= -0.39 + 0.98m_b && \text{for } m_b \le 5.5 \\ \mathbf{M} &= 2.715 - 0.277m_b + 0.127m_b^2 && \text{or } (m_b > 5.5) \end{aligned} \qquad (2.5.2\text{-}7)$$

and by Johnston ().

$$\mathbf{M} = 1.14 + 0.24m_b + 0.0933m_b^2 \qquad (2.5.2\text{-}8)$$

The transformed composite EPRI maximum magnitude distributions are compared to distributions developed by Savy et al. (Reference 234), Frankel et al. (Reference 348), and Geomatrix (Reference 269).

Figure 2.5-250 summarizes the maximum magnitude assessments for sources representative of the ETSZ. The EPRI-SOG expert teams developed a broad uncertainty distribution for maximum magnitude for these sources. When transformed into moment magnitude, this distribution spans nearly the same range as more recent assessments of the distribution for maximum magnitude, and the distributions have modes at similar magnitudes. Frankel et al. (Reference 348) assigns a single value of **M** 7.5 to all of the extended crust region shown in Figure 2.5-242, including the ETSZ. The magnitude of the largest earthquake in the updated catalog that lies within these sources is m_b 5.2 (corresponding to events on August 31, 1861, and February 21, 1916).

Figure 2.5-251 summarizes the maximum magnitude assessments for sources that contain the BLN site (host zone) or represent local background sources that contribute to the hazard. The EPRI-SOG expert teams also developed a broad uncertainty distribution for maximum magnitude for these sources. When transformed into moment magnitude, the composite EPRI-SOG distribution again spans nearly the same range as more recent assessments, although it has a somewhat lower mode. Frankel et al. (Reference 348) assigns a single value of **M** 7.0 to all of the nonextended crust region shown in Figure 2.5-242, including the region around the BLN site. The largest historical earthquake in the updated catalog that lies within these sources is also m_b 5.2.

The comparisons in Figures 2.5-250 and 2.5-251 show that for both the ETSZ and host zone/local background sources, more recent assessments have tended to place more weight on higher magnitudes than the EPRI-SOG expert teams. However, no large historical or prehistorical earthquakes have been identified in these sources that would provide evidence for larger maximum magnitudes, and the EPRI-SOG maximum magnitude distributions for these sources do span the range of more recent assessments. Therefore, the EPRI-SOG maximum magnitude assessments for these sources are judged to be appropriate for use in PSHA calculations for the BLN site. The minimum values for a few of these distributions (local sources defined by Law and Woodward-Clyde) were adjusted to be consistent with the largest observed earthquake in these sources (e.g., changing the low-weighted lower value of m_b 4.2 to m_b 5.2).

The maximum magnitude assessments for New Madrid seismic sources are shown in Figure 2.5-252. The distributions defined by Frankel et al. (Reference 348) and Exelon (Reference 356) represent distributions for the “characteristic” earthquake. The distribution developed by Exelon (Reference 356) includes the ± ¼ magnitude variation in the characteristic magnitude defined in the characteristic magnitude distribution developed by Youngs and Coppersmith (Reference 357). More recent assessments of the size of characteristic New Madrid earthquakes are consistent with the EPRI-SOG evaluations of maximum magnitude for these sources.

The maximum magnitude assessments for Charleston, South Carolina, seismic sources are shown in Figure 2.5-253. The distributions defined by Frankel et al. (Reference 348), Geomatrix (Reference 269), and Savy et al. (Reference 234) essentially represent distributions for the “characteristic” earthquake. The distribution developed by Geomatrix (Reference 269) also includes the ± ¼ magnitude variation in the characteristic magnitude defined in the characteristic magnitude distribution developed by Youngs and Coppersmith (Reference 357). As was the case for New Madrid sources, more recent assessments of the maximum size of Charleston earthquakes are consistent with the EPRI-SOG evaluations.

2.5.2.4.1.4 Summary of Seismic Source Assessments

The following conclusions are obtained from the review of seismic source characterization data.

- No new seismic sources have been identified.
- The EPRI evaluation seismicity rates for sources within 200 mi. of the BLN site are consistent with seismicity rates defined using the updated earthquake catalog.
- The results of paleoliquefaction studies indicate that the frequency of large earthquakes in the New Madrid and Charleston source regions is higher than defined by the EPRI seismic hazard model.
- New data do not indicate a need to modify the EPRI evaluation maximum magnitude distributions for sources within 200 mi. of the BLN site, with the exception of adjusting the lower tails of the distributions for a few sources to reflect the largest earthquake known to have occurred in each source.

2.5.2.4.2 New Information Regarding CEUS Ground Motion Characteristics

The EPRI-SOG evaluation characterized epistemic uncertainty in earthquake ground motions by using three strong-motion attenuation relationships. These were the relationships developed by McGuire et al. (Reference 358), Boore and Atkinson (Reference 359), and Nuttli (Reference 360) combined with the response spectral relationships of Newmark and Hall (Reference 361). These relationships were based, to a large extent, on modeling earthquake ground motions using simplified physical models of earthquake sources and wave propagation. The random (aleatory) variability about the three sets of median attenuation relationships was modeled as a lognormal distribution with a standard deviation of 0.5 in units of the natural log of peak motion amplitude.

Estimating earthquake ground motions in the CEUS has been the focus of considerable research since completion of the EPRI-SOG evaluation. In particular, EPRI (Reference 350) conducted a study to characterize strong ground motion in the CEUS for application in PSHA for nuclear facilities. This study followed the SSHAC (Reference 235) guidelines for a Level III analysis. SSHAC (Reference 235) provided guidance on the appropriate methods to use for quantifying uncertainty in evaluations of seismic hazard. The product of the EPRI (Reference 350) study is a suite of ground motion relationships and associated relative weights that represent the uncertainty in estimating the median level of ground motion and its aleatory variability.

Figure 2.5-254 compares the EPRI (Reference 350) median attenuation relationships to those used in the EPRI-SOG evaluation. EPRI (Reference 350) defined the uncertainty in the median ground motions in terms of four ground motion “cluster” models. Each cluster represented a group of models based on a similar approach for ground motion modeling. The relationships shown in Figure 2.5-254 represent the median estimates of ground motions produced by the models within each cluster. The EPRI (Reference 350) models also use either the Joyner-Boore distance measure or the closest distance to rupture distance measure while the EPRI-SOG (Reference 203) ground motion models use hypocentral distance. In the comparisons shown in Figure 2.5-254, a hypocentral depth of 10 km (6 mi.) was used in conjunction with the EPRI-SOG ground motion models, consistent with their use in the EPRI (Reference 233) PSHA calculation. Depths to the top of the rupture of 5, 3, and 1 km (3, 2, and 0.6 mi.) were used for magnitudes m_b 5, 6, and 7, respectively, in computing the equivalent surface distance from EPRI (Reference 350) Cluster 3 models. The median models are generally consistent with the two spectral models used in the EPRI-SOG evaluation (References 358 and 359). All of the EPRI (Reference 350) median models predict lower levels of motion than obtained using the Nuttli (Reference 360)-Newmark and Hall (Reference 361) model.

In the Reference 350 representation of ground motion, the uncertainty in the median model for each ground motion cluster is defined by two additional models, one representing the 5th percentile of the uncertainty distribution for the median and one representing the 95th percentile. The range in these models defines the uncertainty range in the median ground motions. Figure 2.5-255 compares the composite range in median ground motions across all clusters for the EPRI (Reference 350) ground motions models with the EPRI-SOG attenuation relationships. For m_b 5 and 6, only models for Clusters 1, 2, and 3 are included in defining the range; Cluster 4 models are included in the range for m_b 7. The uncertainty range for the EPRI (Reference 350) peak acceleration relationships generally encompasses the three EPRI-SOG median relationships. However, for 1-Hz spectral acceleration (SA), the Nuttli (Reference 360)-Newmark and Hall (Reference 361) model lies outside of the uncertainty band for the EPRI (Reference 350) ground motion models.

The EPRI (Reference 350) study also developed an assessment of the aleatory variability about the median attenuation relationships. Figure 2.5-256 compares the EPRI (2004) Reference 350 assessments of aleatory variability (defined in terms of the standard deviation of ln [SA]) to the value used in the EPRI-SOG evaluation. The EPRI (Reference 350) assessments are significantly larger than those used in the EPRI-SOG evaluation.

The EPRI (Reference 350) study also developed an assessment of the aleatory variability about the median attenuation relationships. This assessment was re-evaluated and updated by Abrahamson and Bommer (Reference 362) because the EPRI (Reference 350) assessment tended to over-estimate the aleatory variability.

The purpose of the EPRI (Reference 350) and Abrahamson and Bommer (Reference 362) studies was to develop a current representation of the state of knowledge of ground motion estimation for regional hard rock site conditions in the CEUS for use in PSHA applications. The EPRI (Reference 350) median ground motion estimates, combined with the updated (Reference 362) aleatory variability models, are considered appropriate for use in calculating seismic hazard for the BLN site.

2.5.2.4.3 PSHA Revisions

This section describes the revisions to the model used in EPRI (Reference 233) that were identified as being important to PSHA results. Based on the assessments in Subsections 2.5.2.4.1 and 2.5.2.4.2, the following PSHA model adjustments were studied as part of PSHA sensitivity tests for the BLN site:

- Sensitivity to adjustment of the minimum value of maximum magnitude for a few EPRI-SOG sources upward to equal the largest known earthquake within the source zone, based on the updated BLN earthquake catalog.
- Sensitivity to new data relative to the occurrence of large earthquakes in the NMSZ.
- Sensitivity to new data relative to the occurrence of large earthquakes in the Charleston, South Carolina, region.
- Sensitivity to new ground motion models.
- Sensitivity to maximum magnitude values.

The first step in the analysis was to demonstrate that the EPRI (Reference 233) PSHA results could be reproduced. Table 2.5-211 compares the frequency of exceeding a range of ground motion levels computed using the Risk Engineering, Inc. FRISK88 software with the EPRI (Reference 233) results. For frequencies of exceedance greater than about 10^{-6}, the differences are generally less than 5 percent in terms of frequency of exceedance, which translates into approximately 2 percent in terms of ground motion level. This is acceptable agreement, given that independent computer programs were used in the calculations.

During the assessment of source contributions, it was discovered that the original EPRI-SOG input files did not include sources 4 and 4a for the Bechtel team and source 217 for the Law team (Table 2.5-212). The effect of adding these sources to the analysis is shown in Figure 2.5-257. The result is approximately a 3 to 5 percent increase in ground motion levels corresponding to mean hazard in the range of 10^{-4} to 10^{-5} and a 7 to 10 percent increase in ground motion levels corresponding to median hazard in the range of 10^{-4} to 10^{-5}. The values in subsequent analyses were computed using this corrected source list.

The first sensitivity analysis tests the effect of adjusting the EPRI-SOG maximum magnitude distributions to limit the lowest magnitude to be equal to the largest earthquake known to have occurred within each source. This represents a very small change in the inputs and the resulting effect on the hazard is negligible (<0.5 percent).

The next set of sensitivity analyses test the effect of incorporating sources of repeating large magnitude earthquakes at New Madrid and Charleston with return intervals of approximately 500 and 550 years, respectively, into the seismic hazard model. Ideally, the EPRI-SOG characterization of these sources should be updated to reflect the recent data. However, because of the large distance between these sources and the BLN site (> 200 mi.), what is of primary importance is the characterization of the size and frequency of the largest earthquakes. This is illustrated by the magnitude-distance deaggregation of the mean hazard from the EPRI-SOG model. Figure 2.5-258 shows the contributions to the mean hazard at ground motion exceedance levels of 10^{-4} and 10^{-5} deaggregated into 0.1 unit magnitude intervals and three distance intervals. The hazard from distances greater than 186 mi. is primarily from the ERPI-SOG New Madrid sources and is from earthquakes larger that m_b 6.5.

The simplest form of an updated source characterization is to just add sources of repeating earthquakes at New Madrid and Charleston to the existing EPRI-SOG characterization for those regions. This approach results in a small degree of “double counting” of the occurrence of large earthquakes as the EPRI-SOG source characterization includes large magnitude earthquakes in these areas, although with lower frequencies of occurrence. As indicated in Figure 2.5-249, the existing EPRI-SOG seismic source model for New Madrid adequately characterizes the frequency of earthquakes smaller than the estimated size of the 1811-1812 earthquakes (magnitudes less than approximately m_b 6.75). Therefore, a more appropriate update for use in calculating the hazard at the BLN site is to use the EPRI-SOG seismic source characterization to model the occurrence of these smaller earthquakes and to use more recent data to model the occurrence of large repeating earthquakes. This is accomplished by limiting the maximum magnitude for the EPRI-SOG seismic sources to m_b 6.75.

Sensitivity analyses were performed to examine both of these alternative approaches to updating the EPRI-SOG models for New Madrid and Charleston and the potential impact of double counting the occurrence rate of large earthquakes. First, seismic sources for repeating large earthquakes at Charleston and New Madrid were simply added to the EPRI-SOG seismic source model. The seismic source characterization developed by Exelon (Reference 356) for repeating earthquakes at New Madrid and by Geomatrix (Reference 269) for Charleston was used to characterize these sources. The magnitude of the repeating earthquakes at New Madrid and Charleston are shown in Figures 2.5-252 and 2.5-253, respectively (the Exelon, Reference 356, characterization is similar to the Geomatrix, Reference 269, characterization). Figure 2.5-259 shows the resulting mean hazard curves for the EPRI-SOG sources, the repeating large earthquakes at New Madrid and at Charleston, and the combined mean hazard. The repeating large earthquakes at New Madrid contribute to the 10-Hz motion hazard for exceedance frequencies between 10^{-2} and 10^{-4} and are the dominant contributor to the 1-Hz motion hazard for exceedance frequencies less than about 10^{-3}. Compared to the New Madrid source, the Charleston repeating earthquakes have only a very minor contribution to the hazard due to their smaller size and greater distance from the site. The inclusion of the updated source characterization for repeating large earthquakes at New Madrid results in a 1 to 10 percent increase in 10-Hz motions and a 100 to 150 percent increase in 1-Hz motions for mean frequencies of exceedance in the range of 10^{-4} to 10^{-5}. The effect on median hazard is somewhat smaller.

As discussed previously, a more appropriate simplified update of the EPRI-SOG characterization of New Madrid and Charleston that accounts for potential double counting of the occurrence of large earthquakes is to limit the maximum magnitude in the EPRI-SOG models for these sources to magnitudes smaller than the size of the repeating earthquakes and then add updated source characterization for the repeating earthquakes to the revised model. The revised update for New Madrid consists of setting the maximum magnitude for the EPRI-SOG New Madrid sources to m_b 6.75 and adding the seismic source model for larger New Madrid earthquakes developed by Exelon (References 356 and 294). A similar process was used to develop a simplified update of the seismic source characterization for Charleston sources, with the maximum magnitude for the EPRI-SOG sources limited to m_b 6.5 and the Geomatrix (Reference 269) characterization for large repeating earthquakes used to model reoccurrence of large earthquakes.

Figure 2.5-260 compares the hazard computed using the revised updated seismic source model for the New Madrid and Charleston sources to the hazard obtained by simply adding sources of repeating large earthquakes to the EPRI-SOG model. These results indicate that there is negligible effect (< 2 percent change in ground motion level) of “double counting” of large earthquakes in the range of 10^{-4} to 10^{-5} annual frequency of exceedance. The negligible impact of double counting is due to the large difference between the rate predicted by the EPRI-SOG models and the rate based on the results of recent paleoliquefaction studies (Figure 2.5-249).

The third PSHA sensitivity analysis evaluates the effect of replacing the three m_b-based ground motion attenuation models used in the EPRI-SOG (Reference 203) model with the new **M**-based ground motion attenuation models developed by EPRI (Reference 350). Figure 2.5-261 compares these hazard results for the EPRI-SOG (Reference 203) seismic source characterization. The three m_b-**M** relationships described in Subsection 2.5.2.4.1.3 were used to convert m_b magnitudes into moment magnitude for calculation of the hazard using the EPRI (Reference 350) ground motion models. The effect of using the updated ground motion models on the 10-Hz motion hazard is to produce a small increase in ground motion for an exceedance frequency of 10^{-4} (5 percent increase for mean hazard, 17 percent increase for median hazard) and larger increases in ground motions for lower exceedance frequencies (50 to 60 percent increase at

10^{-5} exceedance frequency). The larger ground motions at lower exceedance frequencies is due in part to the increased level of aleatory variability (greater standard deviation) in the EPRI (Reference 350) ground motion characterization compared to the value used in the EPRI-SOG (Reference 203) hazard model (see Figure 2.5-256). For 1-Hz motion hazard, use of the EPRI (Reference 350) ground motion model results in higher ground motions based on median hazard (24 to 40 percent increase in the range of 10^{-4} to 10^{-5} exceedance frequency) and lower ground motions based on mean hazard (33 to 44 percent decrease in the range of 10^{-4} to 10^{-5} exceedance frequency). The higher ground motions for median hazard is likely again due to larger aleatory variability in the EPRI 2004 (Reference 350) ground motion characterization. The lower ground motions for mean hazard is due to replacement of the Nuttli-Newmark Hall model (References 360 and 361) with models that produce lower median ground motions (see Figures 2.5-254 and 2.5-255).

An additional sensitivity analysis tested the effect of adjusting the EPRI maximum magnitude distributions to limit the lowest magnitude to be equal to the largest earthquake known to have occurred within each source. This represents a very small change in the inputs and the resulting effect on the hazard is negligible (<0.5 percent).

Sensitivity of PSHA results to the ground motion model and to the occurrence of large earthquakes in the NMSZ and the Charleston seismic zone are discussed in Subsection 2.5.2.4.4 below.

2.5.2.4.4 Updated PSHA

The revisions described in Subsection 2.5.2.4.3 identified three specific elements of the EPRI-SOG evaluations that are impacted by the new information and data. The areas that require revision are: (1) the characterization of the size and rate of the more frequently occurring large magnitude New Madrid events originating on the fault system that generated the 1811-1812 earthquake sequence; (2) the characterization of the source geometry, recurrence, and magnitude of repeating large magnitude earthquakes in the Charleston region (which has only a very minor impact on the site hazard); and (3) new ground motion models for the CEUS. The modifications to the EPRI-SOG seismic hazard model to incorporate these updates are discussed in the following sections. Note that, with the exception of the repeating large magnitude New Madrid and Charleston earthquakes, the seismicity parameters defined for the EPRI seismic sources are unchanged by new data and are found, consistent with Regulatory Guide 1.208, to be appropriate for use in the updated PSHA for the BLN site.

The first two revisions incorporated sources of repeating large magnitude earthquakes at New Madrid and Charleston with return intervals of approximately 500 and 550 years, respectively, into the seismic hazard model. Ideally, the EPRI characterization of these sources should be updated to reflect the recent data. However, because of the large distance between these sources and the BLN site

(> 200 mi.), what is of primary importance is the characterization of the size and frequency of the largest earthquakes. This is illustrated by the magnitude-distance deaggregation of the mean hazard from the EPRI model. Figure 2.5-258 shows the deaggregation of mean hazard at ground motion amplitudes corresponding to annual exceedance frequencies of 10-4,10-5, and 10-6. The hazard from distances greater than 200 mi. is primarily from large earthquakes in the New Madrid source.

Subsections 2.5.2.4.4.1 and 2.5.2.4.4.2 describe the models used for repeating large magnitude earthquakes in the New Madrid and Charleston seismic zones, respectively. These models include the recurrence rates and magnitudes.

The last revision replaced the ground motion attenuation models used in the Reference 233 model with the ground motion attenuation models developed in Reference 350 and the aleatory variability models developed by Reference 362. Subsection 2.5.2.4.4.3 summarizes the changes in ground motion models.

2.5.2.4.4.1 New Madrid Repeated Large Magnitude Earthquake Source

Characterization of the New Madrid source zone follows the model presented in the Clinton Early Site Permit (ESP) application (Reference 294), with one exception. For the New Madrid model recurrence rate calculation, the Clinton model used a time period of interest of 60 years whereas the Bellefonte model uses a time period of interest of 50 years. Both models assume 40-year plant lifetimes; however, the Bellefonte plant (Units 3 and 4) is expected to begin operation 10 years earlier than the Clinton plant. Thus, the time period of interest is 10 years shorter for Bellefonte relative to Clinton.

This discussion includes relevant new research published since the Clinton ESP was prepared. This new research does not change the Clinton characterization of the New Madrid seismic source.

Forte et al. (Reference 363) provide a new tectonic model for localizing strain in the new Madrid region involving descent of the ancient Farallon plate into the mantle. This new model helps explain large magnitude earthquakes in the New Madrid region, but does not provide additional information on the location, recurrence, or size of these earthquakes.

Recent research uses high precision GPS measurements to measure crustal motion within the New Madrid seismic zone. There is uncertainty as to the significance of data gathered to date (eg. References 364 and 365). However, the precision of velocity measurements is expected to increase as further measurements are made, such that these measurements eventually may be used to help delineate faults and determine present-day strain rates throughout the New Madrid seismic zone.

The principal seismic activity within the upper Mississippi embayment is interior to the Reelfoot rift along the NMSZ. Recent seismologic, geologic, and geophysical studies have associated faults within the NMSZ with large magnitude historical earthquakes that occurred during 1811-1812. Paleoliquefaction studies provide evidence that large magnitude earthquakes have occurred on these faults more frequently than the seismicity rates specified in the EPRI source characterizations. Figure 2.5-262 shows the locations of these sources relative to the BLN site.

The EPRI-SOG source characterizations, as they stand, adequately address the uncertainty related to location, magnitude, and frequency of earthquakes that may occur on other potential seismic sources in the region of the NMSZ, such as recently identified active faults along the northern and southern rift margins. Updating the EPRI-SOG seismic source evaluations for this study, therefore, focuses on the characterization of more frequent large magnitude events along the central fault system. The key source parameters are discussed in the following sections. The logic tree used to represent the uncertainty in the seismic source characterization model for the NMSZ central fault system is shown in Figure 2.5-263.

2.5.2.4.4.1.1 NMSZ Central Faults Source Geometry

Three fault sources are included in the updated characterization of the central fault system of the NMSZ: (1) the New Madrid South (NS) fault; (2) the New Madrid North (NN) fault; and (3) the Reelfoot fault (RF). The first three levels of the logic tree for these sources address the uncertainty in the research community regarding the location and extent of the causative faults that ruptured during the 1811-1812 earthquake sequence. This uncertainty is represented by alternative geometries for the NN, NS, and RF faults. These alternative geometries affect the distance from earthquake ruptures on these fault sources to the BLN site.

The locations of the faults that make up the New Madrid central fault system sources are shown in Figure 2.5-262 (inset A). For the New Madrid South fault (NS) source, two alternatives are considered, as described by Johnston and Schweig (Reference 366): (1) the BA/BL (BA/Bootheel lineament); and (2) the BA/BFZ (BA/Blytheville fault zone). Although modern seismicity is occurring primarily along the BFZ, Johnston and Schweig (Reference 366) present arguments suggesting that the BA/BL is the most likely location for the main NM1 (D1) event and that major NM1 (D1) aftershocks occurred on the BFZ (the northeast extension of the Cottonwood Grove fault). Therefore, slightly greater weight is given to BA/BL [0.6] (total length of 132 kilometers [80 mi.]) versus BA/BFZ [0.4] (total length of 115 km [69 mi.]).

Recent work by Guccione et al. (Reference 367) suggests that the Bootheel lineament is a Holocene-active fault with primarily right lateral displacement, Surficial mapping and corehole transects reveal a Holocene paleochannel (2.4 ka) displaced dextrally at least 13 m across the lineament and a Pleistocene fluvial sand (10.2 ka) displaced vertically about 3 m. These observations, along with documentation of liquefaction features along the Bootheel lineament and observation in cores of juxtaposed sediment types across the lineament, leads to the conclusion that the Bootheel lineament is an active fault that kinematically links the New Madrid North and South faults.

Two alternative total lengths are considered for the NN source. The first, which is given the highest weight [0.7], allows for rupture of the 60-km (36-mi.) fault segment (NN, Figure 2.5-262) as defined by Johnston and Schweig (Reference 366). Cramer (Reference 368) uses a similar value 59 km (35.4 mi.) as the length of his northeast arm. Concentrated seismicity defines the segment as ~40 km (24 mi.) long. Johnston (Reference 297), in modeling the source fault for the NM2 (J1) earthquake, extends the fault to the epicentral region of the 1895 Charleston, Missouri, earthquake (**M** 6.0-6.6), for a total length of 65 km (39 mi.). An alternative total length of 97 km (58 mi.) allows for the fault to extend north to include less well-defined seismicity trends noted by Wheeler (Reference 262). Wheeler et al. (Reference 369) and other researchers argue for a structural northern boundary to the rift in this region. The New Madrid northern extension (NNE, Figure 2.5-262) is not as well defined by seismicity as is the NN segment. Also, the recurrence interval of large magnitude earthquakes in the northern Mississippi embayment appears significantly longer than the recurrence interval for NMSZ earthquakes based on paleoliquefaction studies. Van Arsdale and Johnston (Reference 370) cite as evidence of a long recurrence interval (on the order of tens of thousands of years) the sparse seismicity, the lack of Holocene fault offsets in the Fluorspar Area fault complex along trend to the north, the presence of only minor Quaternary faulting, and the lack of discernable offset of the margins of Sikeston Ridge where it meets the NN. Given these observations, the longer (97 km [58 mi.]) fault length that includes the NN and NNE is given less weight [0.3].

Johnston and Schweig (Reference 366) conclude from historical accounts that the NM3 (F1) event occurred on the RF (Figure 2.5-264). Johnston and Schweig (Reference 366) identify three possible segments of the RF, a central 32-km (20-mi.)-long reverse fault defined by the RF scarp between the two northeast-trending strike-slip faults, a 35-km (22 mi.) -long segment (RS) that extends to the southeast, and a 40-km-long (24 mi.) segment west of the NN (Figure 2.5-264). Seismicity and geomorphic data indicate that the southeast segment is slightly shorter (25 to 28 km [16 to 17 mi.]) than indicated by (References 366, 371, and 372). Cramer (Reference 368) uses a total length of 60 kilometers for the RF. The alternative fault rupture scenarios of Johnston and Schweig (Reference 366) include rupture of a 40-km-long (24 mi.) northwest fault segment (Figure 2.5-264). Cramer (Reference 368) assigns a length of 33 km (21 mi.) to this segment, which he refers to as the west arm. Mueller and Pujol (Reference 372) note that this westerly arm is imaged as a vertical fault that terminates the Reelfoot thrust. They interpret the westerly arm as a left-lateral strike-slip fault kinematically linked to the Reelfoot thrust. Bakun and Hopper (Reference 338) suggest a preferred epicenter location at the northern end of the RS segment. Hough and Martin (Reference 373) show a slightly different geometry for the northwestern portion of the fault and do not interpret the historical 1811-1812 earthquake ruptures to have extended to the rift margin on the southeast (Figure 2.5-265). Two alternative fault geometries are included in this study: (1) the RF fault includes the NW, RF, and

RS segments as defined in Cramer (Reference 368) and (2) a shorter RF that extends from the intersection with the NN fault and extends to the southeastern end of the RF as shown by Hough and Martin (Reference 373) (Figure 2.5-265). The longer length is judged to be more consistent with displacements and magnitudes inferred for the NM3 event, and thus is given higher weight in the model.

2.5.2.4.4.1.2 NMSZ Central Faults Maximum Earthquake Magnitude

The next level of the logic tree addresses the maximum magnitude for earthquakes on the three New Madrid fault sources. As discussed previously in section (a), specific faults and seismicity lineaments have been proposed as the sources of the 1811-1812 and previous earthquakes. In addition, researchers have suggested that the sizes of prehistoric earthquakes associated with these sources are similar to the 1811-1812 earthquakes (e.g., Reference 374). The identification of fault sources and repeated large earthquakes of similar size is suggestive of the behavior of crustal faults in more active regions and many recent studies (e.g., References 269, 348, 349 and 351) have used the concept of "characteristic" earthquakes to characterize the behavior of the New Madrid seismic source. The characteristic earthquake concept is that a seismic source generates repeated large earthquakes of similar size at a frequency that is greater than obtained by extrapolating a Gutenberg-Richter recurrence relationship fit to the observed seismicity rate for smaller-magnitude earthquakes, as illustrated in Figure 2.5-249. These characteristic earthquakes represent the largest earthquakes produced by the source, and as such represent the maximum magnitude event. Using the concept of characteristic earthquakes, seismic source characterizations of the New Madrid seismic source zone typically consider the 1811-1812 earthquakes to represent the maximum earthquake for this source. Table 2.5-213 summarizes recent estimates of the magnitude of the New Madrid 1811-1812 mainshocks.

Bakun and Hopper (Reference 338) provide preferred estimates of the locations and moment magnitudes and their uncertainties for the three largest events in the 1811-1812 sequence near New Madrid. Their preferred intensity magnitude M_I, which is their preferred estimate of **M**, is 7.6 (6.8 to 7.9 at the 95 percent confidence interval) for the December 16, 1811, Event (NM1), 7.5 (6.8 to 7.8 at the 95 percent confidence interval) for the January 23, 1812, Event (NM2), and 7.8 (7.0 to 8.1 at the 95 percent confidence interval) for the February 7, 1812, Event (NM3). The intensity magnitude M_I is the mean of the intensity magnitudes estimated from individual MMI assignments. In their analysis, Bakun and Hopper (Reference 338) consider two alternative eastern North America (ENA) intensity attenuation models, which they refer to as models 1 and 3. As indicated in Table 2.5-213, these two models give significantly different results for larger magnitude earthquakes. Bakun and Hopper (Reference 338) state that because these models are empirical relations based almost exclusively on **M** < 6 calibration events "There is no way to confidently predict which relation better represents the MMI-distance data for **M** 7 earthquakes in ENA" (p. 66, Reference 338). They present arguments supporting their preference for model 3, but do not discount the results based on model 1.

Dr. Susan Hough (Reference 375) believes that there are insufficient data regarding the calibration of ENA earthquakes larger than **M** > 7 to rely strictly on ENA models as was done in Bakun and Hopper (Reference 338). She offers arguments to support **M** 7.6 (the size of the 2003 Bhuj earthquake) as a reasonable upper bound for the largest of the earthquakes in the 1811-1812 New Madrid earthquake sequence, which is more consistent with the estimates cited in Hough et al. (Reference 376) and Mueller et al. (Reference 377).

Mueller et al. (Reference 377) use instrumentally recorded locations of recent earthquakes (assumed by Mueller et al. to be aftershocks of the 1811-1812 sequence) and models of elastic stress change to develop a kinematically consistent rupture scenario for the mainshock earthquakes of the 1811-1812 New Madrid sequence. In general, the estimated magnitudes for NM1 and NM3 used in their analysis (**M** = 7.3 and **M** = 7.5, respectively) are consistent with those previously published by Hough et al. (Reference 376). Their results suggest that the mainshock Events NM1 and NM3 occurred on two contiguous faults, the strike-slip Cottonwood Grove fault and the Reelfoot thrust fault, respectively. The locations of the NM1 and NM3 Events on the Cottonwood Grove and RFs, respectively, are relatively well constrained. In contrast to the earlier Hough et al. (Reference 376) study that located the NM2 earthquake on the NN, they suggest a more northerly location for the NM2 Event, possibly as much as 200 km (124 mi.) to the north in the Wabash Valley of southern Indiana and Illinois. Hough et al. (Reference 378) also infer a similar more northerly location. Using Bakun and Wentworth's (Reference 379) method, Mueller et al. (Reference 377) obtain an optimal location for the NM2 mainshock at 88.43°W, 36.95°N and a magnitude of **M** 6.8. They note that the location is not well constrained and could be fit almost as well by locations up to 100 km (62 mi.) northwest or northeast of the optimal location. Mueller et al. (Reference 377) conclude that the three events on the contiguous faults increased stress near fault intersections and end points in areas where present-day microearthquakes have been interpreted as evidence of primary mainshock rupture. They note that their interpretation is consistent with established magnitude/fault area results, and do not require exceptionally large fault areas or stress drop values for the New Madrid mainshocks.

With respect to the location of the NM2 Event, Bakun and Hopper (Reference 338) also discuss the paucity of MMI assignments available for this earthquake to the west of the NMSZ and the resulting uncertainty in its location. They note that the two MMI sites closest to the NMSZ provide nearly all of the control on the location of this event and that, based on these two sites, a location northeast of their preferred site would be indicated. However, they conclude that the lack of 1811-1812 liquefaction observations in western Kentucky, southern Illinois, and southern Indiana preclude an NM2 location in those areas. Bakun and Hopper (Reference 338) follow Johnston and Schweig (Reference 366) in selecting a preferred location on the NN. Dr. Steve Obermeier confirmed that liquefaction features in the Wabash Valley region that would support the more northerly location preferred by Mueller et al. (Reference 377) are absent (Reference 380). He noted that he had looked specifically in the area cited in the Yearby Land account that was cited by Mueller et al. (Reference 377) and observed evidence for only small sand blows and dune sands, but did not see features of the size and origin described in that account.

The review of these new publications indicates that there still remain uncertainty and differing views within the research community regarding the size and location of the 1811-1812 earthquakes. In addition, Dr. Arch Johnston (Reference 381) indicates that the estimates of Johnston (Reference 297) are likely to be high by about 0.2 to 0.3 magnitude units. Based on this review of these articles and the communications with Drs. Bakun, Hough, and Johnston, the maximum magnitude for the New Madrid central fault system faults was defined as follows.

- Equal weight (one-third) is to be given to estimates based on Bakun and Hopper (Reference 338) and Hough et al. (Reference 376)/Mueller et al. (Reference 377), and the Johnston (Reference 381) revisions to Johnston (Reference 297)

- Results from both intensity attenuation relations (models 1 and 3) in the Bakun and Hopper (Reference 338) estimate are used. Based on Bakun and Hopper's preference for model 3, weights are assigned of 0.75 to model 3 and 0.25 to model 1

- In the case of the Hough et al. (Reference 376)/Mueller et al. (Reference 377) estimates and Hough (Reference 375) estimates, equal weight is assigned to the range of preferred values given for each earthquake.

The resulting characteristic magnitude distribution for each of the three faults is given in Table 2.5-214. Rupture sets 1 and 2 correspond to the revised Johnston (Reference 297) estimates, rupture sets 3 and 4 correspond to the Bakun and Hopper (Reference 338) estimates, and rupture sets 5 and 6 correspond to the Hough et al. (Reference 376) estimates.

As discussed in the following section, the present interpretation of the paleoearthquake data is that the two prehistoric earthquake ruptures that occurred before the 1811-1812 sequence also consisted of multiple, large magnitude earthquakes. Therefore, for this assessment, the event is considered to be rupture of multiple (two to three) of the fault sources shown in Figure 2.5-262. Furthermore, the arguments for the high versus low magnitude assessments for the individual faults are considered to be highly correlated. Therefore, six alternative sets of ruptures were produced from the distributions developed previously for each fault, as shown in the logic tree in Figure 2.5-263 and given in Table 2.5-214.

The magnitudes listed in Table 2.5-214 are considered to represent the size of the expected maximum earthquake rupture for each fault within the NMSZ. Following the development of the characteristic earthquake recurrence model by Youngs and Coppersmith (Reference 357), as modified by Youngs et al. (Reference 382), the size of the next characteristic earthquake is assumed to vary randomly about the expected value following a uniform distribution over the range of ±¼ magnitude units. This range represents the aleatory variability in the size of individual characteristic earthquakes. For example, given that the expected magnitude for the characteristic earthquake on the NS fault source is **M** 7.8, the magnitude for the next characteristic earthquake is uniformly distributed between **M** 7.55 and **M** 8.05.

2.5.2.4.4.1.3 NMSZ Central Faults Earthquake Recurrence

The best constraints on recurrence of repeated large magnitude NMSZ events result from paleoliquefaction studies throughout the New Madrid region and paleoseismic investigations of the RF scarp and associated fold. Based on studies of hundreds of earthquake-induced paleoliquefaction features at more than 250 sites, Tuttle et al. (Reference 374) conclude that: (1) the fault system responsible for the New Madrid seismicity generated temporally clustered, very large earthquakes in AD 900 ±100 and AD 1450 ±150 years as well as in 1811-1812; (2) given uncertainties in dating liquefaction features, the time between the past three events may be as short as 200 years or as long as 800 years, with an average of 500 years; and (3) prehistoric sand blows probably are compound structures, resulting from multiple earthquakes closely clustered in time (i.e., earthquake sequences).

A recent paleoliquefaction study in the northern part of the New Madrid seismic zone supports these conclusions (Reference 352). Six episodes of earthquake-induced liquefaction are associated with soil horizons containing artifacts and datable organic material. The oldest four episodes of liquefaction occurred around 2350 B. C (4350 ybp) +200 years and are interpreted to represent a cluster of earthquakes similar in size to the 1811-1812 New Madrid earthquakes. Two later episodes of liquefaction are documented to have occurred in A.D 300 (1700 ybp) +200 years, and A.D 1670. A New Madrid-type earthquake in 300 A.D. would be support and average recurrence time of 500 years.

The full paleoseismic record of the New Madrid seismic zone is reviewed by Guccione (Reference 383). The record includes evidence from paleoliquefaction, sediment rupture and deformation, fluvial response, and biotic response, Interdisciplinary approaches to paleoseismology have provided a well constrained catalog of Late Holocene earthquake events. Five well-dated large seismic events have occurred during the Late Holocene, and several less-well-dated events are documented during the Early and Middle Holocene.

Periodic channel perturbations in the Mississippi River across the Reelfoot fault are documented by Holbrook et al. (Reference 384), and assumed to correlate with seismic events that caused vertical displacements across the fault. Analysis of sequentially abandoned meander bends suggests that channel straightening events occurred upstream of the Reelfoot fault in the previously known A.D 900, event, documented by Kelson et al. (Reference 385). Another river-straightening event occurred between 4244 ± 269 ybp and 3620 ± 220 ybp. This research contributes evidence for activity on the Reelfoot fault in the middle Holocene.

Cramer (Reference 368) obtained a 498-year mean (440-year median) recurrence interval for New Madrid characteristic earthquakes based on a Monte Carlo sampling of 1,000 recurrence intervals using the Tuttle and Schweig (Reference 391) uncertainties as a range of permissible dates (± two standard deviations) for the two most recent prehistoric earthquakes (i.e., AD 900 ±100 and AD 1450 ±135). Assuming a lognormal distribution with a coefficient of variation of 0.5 for inter-arrival time, Cramer (Reference 368) obtained a 68 percent confidence interval for the mean recurrence interval of 267 to 725 years, and a 95 percent confidence interval of 162 to 1196 years (ranges for one and two standard deviations, respectively).

Exelon (Reference 294, Attachment 2 to Appendix B) presents a detailed assessment of the timing constraints on prehistoric New Madrid earthquakes and the development of occurrence rates for repeats of 1811-1812 earthquake sequence. The uncertainties in the ages of individual samples were used to constrain the timing of individual events. A Monte Carlo sample of 10,000 sets of time intervals between events was generated using these data. Two recurrence models were used to represent the occurrence of earthquake sequences, the commonly used Poisson (memoryless) model and a renewal model (one-step memory). The uncertainty in fitting these models to a sample of limited size (two closed time intervals, between 900 AD and 1450 AD and between 1450 AD and 1811-1812, and one open interval post 1812) together with the simulated distributions of time intervals provided uncertainty distributions on the recurrence rates for New Madrid sequences. For the renewal model, Exelon (Reference 294) used a lognormal distribution to represent the time between earthquakes. Exelon (Reference 356) repeated the analysis of the simulated time intervals between earthquake sequences using the Brownian Passage Time (BPT) model developed by Ellsworth et al. (Reference 387) and Matthews et al. (Reference 388) to represent the distribution of the time between earthquake sequences in the renewal model. Ellsworth et al. (Reference 387) and Matthews et al. (Reference 388) propose that the BPT model is more representative of the physical process of strain buildup and release on a seismic source than the other distribution forms that have been used for renewal models (e.g., the lognormal). Based on these arguments, the BPT model was used by the Working Group (Reference 389) to assess the probabilities of large earthquakes in the San Francisco Bay area.

Figure 2.5-266 shows the uncertainty distributions for the mean repeat time between New Madrid earthquake sequences obtained by Exelon (Reference 356). Application of the BPT model requires estimation of the aperiodicity coefficient α that defines the variability in the timing of individual events. Because of the very limited sample size, Exelon (Reference 356) did not estimate α from the simulated data. Instead, they utilized the distribution for α developed by the Working Group (Reference 389) of 0.3 (wt 0.2), 0.5 (wt 0.5), and 0.7 (wt 0.3). These alternative values were incorporated into the uncertainty model for the New Madrid repeating earthquake source (Figure 2.5-263).

Following the process used by Exelon (References 294 and 356), the occurrence rates for New Madrid large magnitude earthquake sequences were estimated using the distributions for mean repeat time shown in Figure 2.5-266. For the Poisson model, the occurrence rate is just the inverse of the mean repeat time. For the BPT-renewal model, an equivalent Poisson rate is obtained, allowing the exceedance rate from the New Madrid earthquake sequence to be added to the exceedance rate from all other sources. The equivalent Poisson rate, $\lambda_{renewal}$, is given by the expression:

$$\lambda_{renewal} = -1n[1 - P_{renewal}(\text{event in time } t_0 \text{ to } t_0 + \Delta t)] / \Delta t \qquad (2.5.2\text{-}9)$$

where t_0 is the present time measured from the date of the most recent event, Δt is the time period of interest, and $P_{renewal}()$ is the probability of the event occurring in the time interval Δt. The time period of interest, Δt, was taken to be 50 years. This is somewhat long for the typical life span of a nuclear power plant, but longer values of Δt produce larger values of the average rate. The renewal recurrence model, $P_{renewal}()$ is given by the expression:

$$P_{renewal}(\text{event in time } t_0 \text{ to } t_0 + \Delta t) = \frac{F(t_0 + \Delta t) - F(t_0)}{1 - F(t_0)} \qquad (2.5.2\text{-}10)$$

where $F()$ is the cumulative distribution for time between events.
Equation (2.5.2-10) gives the probability of a single event in time Δt while the equivalent Poisson rate (Equation 2.5.2-9) is based on the probability of one or more events. However, the probability of two or more in the renewal model case is negligible.

For the BPT model, $F()$ is given by:

$$\begin{aligned} F(t) &= \Phi[u_1(t)] + e^{2/\alpha^2}\Phi[-u_2(t)] \\ u_1(t) &= (\sqrt{t/\mu} - \sqrt{\mu/t})/\alpha \\ u_2(t) &= (\sqrt{t/\mu} - \sqrt{\mu/t})/\alpha \qquad (2.5.2\text{-}11) \\ f(t) &= \left(\frac{\mu}{2\pi\alpha^2 t^3}\right)^{1/2} \exp\left(-\frac{(t-\mu)^2}{2\mu\alpha^2 t}\right) \end{aligned}$$

where μ is the mean inter-arrival time (repeat time), α is the aperiodicity coefficient, and $\Phi(\ )$ is the standard normal cumulative probability function.

The uncertainty distributions for mean repeat time shown in Figure 2.5-266 were represented in the seismic hazard model by a five-point discrete approximation to a continuous distribution developed by Miller and Rice (Reference 390). Table 2.5-215 lists the discrete distributions for mean repeat time and the equivalent Poisson rates. The Poisson and renewal recurrence models are given equal weight (Figure 2.5-263). The renewal model is considered more appropriate on a physical basis for the behavior of characteristic earthquakes on active faults. The Working Group (Reference 389) applied weights of 0.7 and 0.6 to non-Poissonian behavior for the San Andreas and Hayward faults, respectively. For other, less active sources, they assigned a weight of 0.5 or less to non-Poissonian behavior. While the New Madrid faults are not plate boundary faults, they exhibit behavior that is similar to that expected for an active plate boundary fault. Equal weights represent maximum uncertainty as to which is the more appropriate model.

The paleoliquefaction data gathered in the New Madrid region indicate that the prehistoric earthquakes have occurred in sequences closely spaced in time relative to the time period between sequences, similar to the 1811-1812 sequence. Figure 2.5-267, taken from Tuttle et al. (Reference 374), shows the estimated earthquake sizes and event locations for the 1811-1812 sequence and the two previous sequences. These data indicate that the RF has ruptured in all three sequences, but the NN and NS sources may have produced earthquakes on the order of one magnitude unit smaller than the 1811-1812 earthquakes in previous sequences. Recent discussions with Dr. Tuttle (Reference 386) indicate that she considers that the difference between the size of the 1811-1812 earthquakes and those of the 900 and 1450 sequences are likely to be smaller than what was portrayed in Figure 6 of Tuttle et al. (Reference 374). As a result, Exelon (Reference 356) revised the model of Exelon (Reference 294) for New Madrid sequences to consist of two alternative models of rupture or earthquake sequences. In Model A, all ruptures are similar in size to the 1811-1812 earthquakes. In Model B one-third of the sequences are the same as Model A, one-third of sequences contain a smaller rupture of the NN, and one-third of sequences contain a smaller rupture of the NS. The difference in magnitude from the 1811-1812 ruptures was set to be no more than one-half magnitude unit, and no ruptures are allowed to be less than **M** 7. All three earthquakes were included in the hazard calculation in all rupture sequences. Model A (always full ruptures) is given a weight of two-thirds and Model B a weight of one-third, based on Dr. Tuttle's expression of the difficulties in estimating the size of the pre 1811-1812 ruptures and her judgment that the difference between the rupture sizes was likely smaller than proposed in Tuttle et al. (Reference 374).

The computation of the hazard from the New Madrid earthquake sequence uses the formulation outlined in Toro and Silva (Reference 351). The frequency of exceedance, $\nu(z)$, from the earthquake sequence is given by the expression:

$$v(z)_{characteristic} = \lambda_{sequence}\left[1-\prod_{i}\{1-P_i(Z>z)\}\right] \tag{2.5.2-12}$$

where $\lambda_{sequence}$ is the equivalent annual frequency of event clusters and Pi(Z > z) is the probability that earthquake *i* in the sequence produces ground motions in excess of level *z*.

The results of seismic hazard including the New Madrid earthquakes, as described here, are shown in Subsection 2.5.2.4.4.4.

2.5.2.4.4.2 Charleston Repeating Large Magnitude Earthquake Source

The 1886 Charleston, South Carolina, earthquake was the largest earthquake occurring in historical time in the eastern U.S., and is considered to have a moment magnitude in the range of 6.8 to 7.5 (References 297, 298, 299, and 338). Based on the felt intensity reports defining the meizoseismal area (area of maximum damage) and the occurrence of continuing seismic activity (the MPSSZ), the epicentral region of the 1886 earthquake is considered to be centered northwest of Charleston. Recent published and unpublished studies were reviewed during the BLN hazard evaluation for information on the potential location and extent of the Charleston source and the maximum characteristic earthquake expected to occur on it (see discussion in Subsection 2.5.1.1.4.3.2). These studies provide evidence that large magnitude earthquakes have occurred in the vicinity of Charleston more frequently than the seismicity rates specified in the EPRI (Reference 233) source characterizations. These studies also indicate that the source geometries specified in the EPRI evaluation do not adequately capture the full range of possible source geometries. An updated source characterization logic tree for repeating large magnitude Charleston earthquakes based on these new data is presented in Figure 2.5-268, and the basis for the alternative characterizations is described as follows.

2.5.2.4.4.2.1 Charleston Earthquake Source Geometry

The Charleston earthquake sources proposed in the EPRI evaluation (Reference 233; Figures 2.5-235, 2.5-236, 2.5-237, 2.5-238, 2.5-239, and 2.5-240) generally are centered on the meizoseismal area of the Charleston earthquake, with some zones extending northwest into central South Carolina and southeast, offshore of Charleston. Based on new information regarding the timing and distribution of paleoliquefaction in the South Carolina Coastal Plain (Figure 2.5-226), the LLNL TIP (Reference 234) interpretations limit the location of the Charleston source to the coastal plain area, or along the ZRA-S (Figure 2.5-241). The preferred alternative in the LLNL TIP study is for a localized source zone centered on the meizoseismal area and the Woodstock fault. The LLNL TIP model also includes an alternative rectangular zone that extends along the ZRA-S (Figures 2.5-241 and 2.5-242). The 2002 USGS source characterization considers both a regional source zone and a local source zone centered on the Woodstock fault and the southern part of the ZRA-S (Figure 2.5-242). Both alternatives are given equal weight (Reference 348).

Given the various interpretations and models reported in the recent literature for the location/extent of the source for the Charleston earthquake and other paleoearthquakes in coastal South Carolina, and for the location of a buried, potentially active fault system in South Carolina, this BLN study considers a range of models that encompass the likely extent of the Charleston-type source(s). Two approaches are used to locate the occurrence of Charleston-type earthquakes. The first approach (the fault source approach) considers the geologic features or structures identified within the meizoseismal zone of the 1886 earthquake (along with potential extensions of these features beyond the meizoseismal zone), to identify the location of the causative source of the 1886 earthquake and future repeating large magnitude earthquakes. The second approach does not specify a source fault or fault zone for the Charleston-type earthquakes. Instead, the source is constrained to a zone that is defined by the area of strong ground shaking associated with sites of paleoliquefaction.

Several types of data provide constraints on the location and extent of the source fault(s) for Charleston-type earthquakes in the Atlantic Coastal Plain (see discussion in Subsection 2.5.1.1.4.3.2). Two general fault sources have been postulated: the Ashley River/Woodstock faults in the meizoseismal area, and the north-northeast-trending zone of river anomalies (and associated faults and linear magnetic anomalies) referred to as the ZRA-S (Figures 2.5-223 and 2.5-224). The Woodstock fault and a fault zone localized along the southern part of the ZRA-S are included in the BLN source characterization. The Woodstock fault (and Woodstock lineament) is shown as a solid line in Figure 2.5-262. The alternative fault source used in the BLN study is identical to the USGS local source model (Reference 348) that includes the southern part of the ZRA-S in addition to the Woodstock/Ashley River faults. Two other postulated fault sources (the Adams Run and Charleston faults, Figure 2.5-223) are located within the general region of the Ashley River and Woodstock faults, but because these sources all are located at approximately the same distance from the BLN site, these faults were not evaluated as separate/specific sources for repeating Charleston large magnitude earthquakes in the BLN study. Three alternative areal source zones are included in the source characterization for the BLN study. These include the USGS 1996/2002 regional source zone (Reference 348 and 349); an areal source zone based on the locations of Mesozoic basins developed by Geomatrix (Reference 269); and a coastal zone defined by the LLNL-TIP study (Reference 234). The USGS 1996/2002 regional source zone for the Charleston source apparently was defined to include the ZRA-S and most of the paleoliquefaction sites along the South Carolina Coastal Plain. This source zone also encompasses parts of the Mesozoic basins in the coastal plain of South Carolina. The northwest margin of this areal source zone was not associated with any particular structure (Reference 349).

The alternative areal source zone that is based on the locations of Mesozoic Basins along the coastal plain region in South Carolina extends southwest to the Georgia border, and further eastward in the offshore region compared to the USGS regional source zone. The Mesozoic Basin source zone does not extend as far north as the extent of the ZRA-S (and the USGS regional source zone), but is consistent with the extent of paleoliquefaction features along the South Carolina Coastal Plain.

The third alternative is identical to the LLNL-TIP coastal zone (Reference 234) (Figure 2.5-241), which encompasses the three major centers of paleoliquefactions features identified in the South Carolina Coastal Plain. The centers of paleoliquefaction include one located northeast of Charleston at Georgetown, one centered at Middleton Place (northwest of Charleston), and one located southwest of Charleston at Bluffton (Figures 2.5-226 and 2.5-227).

The weights assigned to the alternative source geometries are summarized in the logic tree in Figure 2.5-268. The localized fault approach is strongly preferred to the areal source zone approach (weights of 0.67 and 0.33, respectively) based on the presence of potentially active faults in the Middleton Place-Summerville area and geomorphic evidence for Quaternary deformation along the ZRA-S. The Woodstock fault is preferred to the USGS Local Source Zone (ZRA-S) (weights of 0.67 and 0.33, respectively) because of the presence of a known fault in the epicentral region compared to the inferred fault along the ZRA-S. For the areal source zone approach, the USGS regional areal source and Mesozoic Basin areal source are preferred to the TIP coastal plain source (weights of 0.4, 0.4, and 0.2, respectively) because of the distribution of the paleoliquefaction features used to infer a source location is in part a function of the presence of susceptible deposits that are localized along the coast and, therefore, do not provide uniform coverage throughout the region.

2.5.2.4.4.2.2 Charleston Source Maximum Magnitude

Characterization of the source of repeating large earthquakes at Charleston was also performed by applying the concept of characteristic earthquakes. The interpretation of the sizes of prehistoric earthquakes is more uncertain here than at New Madrid, but the interpretations do not suggest that the prehistoric events were much, if any, larger than the 1886 earthquake. Therefore, the maximum (characteristic) earthquake magnitude for the Charleston source is taken as equal to the magnitude of the 1886 Charleston earthquake. Because there are uncertainties regarding the magnitude of this earthquake, published magnitude estimates were used in the BLN study to develop a maximum earthquake magnitude for the Charleston source (Table 2.5-216). Published estimates of the magnitude of the 1886 Charleston earthquake include that of Johnston (Reference 297), who suggested a preferred value of **M** 7.3 ± 0.26. This best estimate of **M** 7.3 is based on a weighted average of magnitude estimates from multiple regression relationships between the area encompassed by individual MMI levels and magnitude. These empirical relationships were developed using intensity and area data collected from eastern North America and SCRs worldwide. Specifically, the best estimate magnitude of **M** 7.3 is based on multiple regression relationships that maximize use of eastern North American data for MMI levels A_{felt}, A_{IV}, A_{V}, A_{VI}, supplemented by worldwide data for MMI levels A_{VII} and A_{VIII}. Further, the A_{felt} relationship was modified to lower the effect of distant outlying reports, and the A_{VII} and A_{VIII} relationships were corrected for wedge effects of the coastal plain sediments (References 297 and 392).

Earlier magnitude estimates (References 299 and 300) gave an m_b ranging from 6.6 to 6.9. Bollinger (Reference 300) and Nuttli et al. (Reference 299) used similar approaches that relate MMI data to m_b using intensity attenuation with distance. Bollinger estimated the magnitude as m_b 6.8, while Nuttli et al. estimated the magnitude as m_b 6.6. Nuttli (Reference 393) also used a relationship between the area of MMI IV and body-wave magnitude to estimate an m_b of 6.9 for the Charleston earthquake, concluding that a best estimate of the magnitude based on both techniques was m_b 6.7. These earlier estimates are represented with a mean value of m_b 6.75 ± 0.15 (Table 2.5-216).

In a new approach to estimating magnitude from MMI, Bakun and Hopper (Reference 338) developed a method to directly invert intensity observations to moment magnitude **M**. They obtained an estimate of **M** 6.9 (6.4 to 7.2 at the 95 percent confidence level) for the 1886 Charleston earthquake.

An alternative approach for estimating the magnitude of the 1886 earthquake relies on back-calculation of ground motions from paleoliquefaction evidence (Reference 298). In this approach, the threshold peak ground acceleration (PGA) required to cause ground deformation is estimated based on the intersection of the “layer curve effect” and the cyclic stress method for relating the percentage of a source layer that liquefies to the PGA. Martin and Clough (Reference 298) concluded that the liquefaction evidence was consistent with an earthquake no larger than **M** 7.5, and possibly as small as **M** 7.0. Their estimate is represented by a magnitude range of **M** 7.25 ± 0.25 (Table 2.5-216).

In recent studies of soils that liquefied during paleoearthquakes attributed to the Charleston source, Hu et al. (References 318 and 319) used the approach of Martin and Clough (Reference 298) to estimate the PGAs due to paleoearthquakes, and estimated magnitudes in the range of **M** 6.8 to 7.6 for these paleoearthquakes from the PGAs. More recent work by Leon et al. (Reference 320) to assess the liquefaction resistance of older soils indicates the preliminary magnitude estimates published by Hu et al. are too high, and that the best estimate magnitude for the largest paleoearthquakes (Events A and C’) are in the range of **M** 6.2 to 7.2 (Table 2.5-201).

The previous magnitude estimates were used to evaluate the maximum earthquake for the Charleston source. The estimate by Johnston (Reference 297) is deemed more reliable than the estimates of Bollinger (Reference 300) and Nuttli et al. (Reference 299), because the relationships used by Johnston were based on revised interpretations of the extent of MMI shaking levels (A_{felt}, A_{VII}, A_{VIII}) for the 1886 earthquake and on larger eastern North American and worldwide data sets. The new estimates from Bakun and Hopper (Reference 338) also are considered more reliable than the Nuttli and Bollinger estimates because they are based on larger data sets of MMI estimates. Furthermore, all three of these types of estimates are considered more reliable than the estimates based on liquefaction and paleoliquefaction data. Therefore, the Johnston (Reference 297) and Bakun and Hopper (Reference 338) estimates are assigned higher weight (0.25 and 0.35) than the Bollinger and Nuttli et al. estimates (0.2) and the Martin and Clough (Reference 298) and Leon et al. (Reference 320) estimates (0.1 each) (Table 2.5-216).

2.5.2.4.4.2.3 Charleston Earthquake Source Recurrence

The spatial distribution of seismically induced liquefaction features along the Atlantic seaboard has been used to assess the location and timing of pre-1886 earthquakes (Reference 317).

Talwani and Schaeffer (Reference 317) provide two scenarios for the recurrence of earthquakes in the South Carolina Coastal Plain, and Geomatrix (Reference 269) identified an alternative interpretation of one of Talwani's scenarios (Table 2.5-217). Scenarios 1 and 2 are based on the existence of one source located in the vicinity of Charleston; Scenario 3 is based on the existence of three sources, including one at Charleston, and one source to the north and one source to the south of Charleston. The paleoliquefaction data show that at least five earthquakes have occurred on the Charleston source. At least two additional earthquakes may have occurred on the northern or southern sources (Reference 317). Alternatively, all the paleoliquefaction features may result from earthquakes occurring on the Charleston source, for a total of six or seven earthquakes on the Charleston source (Table 2.5-217).

An important issue in estimating the recurrence interval for moderate to large magnitude earthquakes in South Carolina is the uncertainty in the timing of paleoliquefaction events. Eight or nine earthquakes (that resulted in liquefaction) are interpreted to have occurred in South Carolina based on the ages and one-sigma uncertainties shown in Table 2.5-217. Not all of these events necessarily represent different earthquakes at higher confidence levels; thus, the total number of earthquakes may be fewer. Specifically, one, two, or three events may have occurred during the period from 1600 to 2000 years before present (ybp). Similarly, one or two events may have occurred around 5000 to 6000 ybp.

Another issue with respect to estimating the recurrence of large magnitude earthquakes in South Carolina is the interval during which the record of paleoliquefaction is considered to be complete. The potential for liquefaction varies in response to the changes in groundwater levels along the coastal plain. Specifically, as groundwater levels are thought to have risen in response to the Holocene rise in sea level, the potential for liquefaction has increased during the Holocene (References 316 and 317).

Data summarized in Talwani and Schaeffer (Reference 317) indicates that worldwide, sea level was 10 m (33 ft.) below present mean sea level (msl) prior to about 6000 ybp, and was even lower prior to that time, such that liquefaction likely could not have occurred in sediments at the ground surface prior to 6000 ybp. Locally, along the South Carolina and Georgia coast, msl rose to a high stand of about -3 m (-10 ft.) to -1 m (-3 ft.) msl from ~5300 to 3500 ybp, falling to about -3 to -6 m (-10 to -20 ft.) msl from ~3500 to 2000 ybp, and then rising to present msl.

When groundwater levels were lower, liquefaction may not have occurred during large magnitude earthquakes. Thus, even if large magnitude earthquakes occurred, they may not be represented by paleoliquefaction features, and the paleoseismic record would be incomplete for periods of lower groundwater levels (References 316 and 317). Talwani and Schaeffer conclude that the paleoseismic record may be considered complete only for the past 2,000 years.

Based on review of the dates of paleoliquefaction events (References 247 and 317), the paleoliquefaction record likely is complete only for the past 2,000 years. Because the paleoseismic record does not appear to be complete for the period between 5800 and 2000 ybp, the recurrence intervals between older paleoliquefaction events likely is not representative of the recurrence times between paleoseismic events at Charleston. Therefore, for the recurrence assessment, two estimates are used for the completeness period for repeating large magnitude Charleston earthquakes, 2,000 years (weight of 0.9) and 6,000 years (weight of 0.1).

The recurrence scenarios are based on the time intervals between earthquakes in each model. For Scenarios 1 and 2, the paleoliquefaction is assumed to result from earthquakes occurring on the Charleston source, and the Charleston source is constrained to lie within the Charleston source zone. No separate northern or southern earthquake sources for the observed paleoliquefaction exist in these scenarios. The following paragraphs describe the event intervals for the shorter completeness period of 2,000 years.

For Scenario 1 in Table 2.5-217, the paleoliquefaction events that occurred at 1648 ybp and 1966 ybp (Events C and D) are assumed to represent two earthquakes on the Charleston source. Thus, there are four recurrence intervals for Charleston earthquakes in this scenario. The mean recurrence interval for Scenario 1 is 493 years. Because this scenario is not completely consistent with the observed distribution of paleoliquefaction sites for these two events, a low weight of 0.2 is assigned to Scenario 1.

For Scenario 2 in Table 2.5-217, only one paleoliquefaction event is assumed to have occurred in the period from 1600 ybp to 2000 ybp; this event occurred at 1683 ybp. Thus, there are three recurrence intervals for Charleston earthquakes in this scenario. This scenario is consistent with Scenario 2 as proposed by Talwani and Schaeffer (Reference 317). The mean recurrence interval for Scenario 2 is 562 years. This scenario is assigned a weight of 0.3 because the combined distribution of paleoliquefaction sites for this Event (C^1) is generally similar to the distribution of paleoliquefaction sites for Charleston Events A, B, and E.

For Scenario 3, the paleoliquefaction observed to the north at Georgetown and dated at about 1648 ybp is assumed to have resulted from an earthquake on a northern source. In addition, the paleoliquefaction observed to the south near Bluffton and dated at 1966 ybp is assumed to have resulted from an earthquake on a southern source. Thus, for this scenario, no earthquakes occur on the

Charleston source at 1648 ybp and 1966 ybp. This scenario is consistent with Scenario 1 as proposed by Talwani and Schaeffer (Reference 317). The annual frequency of earthquakes is based on the number of earthquakes estimated to have occurred during the past 2,000 years.

In Scenario 3, three earthquakes are interpreted to have occurred on the Charleston source (1886 or 113 ybp, 546 ybp, and 1021 ybp) during the past 2,000 years. There are two complete recurrence intervals (433 years and 475 years) and one incomplete recurrence interval in this scenario. Because the earthquake record (for earthquakes large enough to have caused liquefaction) is assumed to be complete for the past 2,000 years, the minimum time interval between the earthquake at 1021 ybp and the previous earthquake is about 1,000 years. It is necessary to develop an estimate of this time interval to account for the observation that no earthquake occurred at Charleston for a minimum time interval of 1,000 years.

The timing of the earthquake that occurred prior to the 1021 ybp earthquake is constrained to the period from 2000 ybp to 3548 ybp because the paleoliquefaction record is assumed to be complete for the past 2,000 years and because the next known (older) paleoearthquake occurred at approximately 3548 ybp (Table 2.5-217). A series of 10 possible dates at 180-year intervals between 2000 and 3600 ybp was selected to estimate the duration of the time interval between the 1021 ybp earthquake and the previous earthquake. The resulting distribution for the third time interval is combined with the two measured recurrence intervals (433 and 475 years) to estimate the annual frequency of occurrence of the characteristic earthquake. The mean recurrence interval for Scenario 3 is 513 years. Scenario 3 is favored by Talwani and Schaeffer (Reference 317, their Scenario 1); thus, this scenario is assigned a weight of 0.5 (Table 2.5-217).

For the alternative completeness period of 6,000 years, additional paleoearthquake recurrence intervals between events at 3548, 5038, and 5800 ybp are included in Scenarios 1 and 2, and intervals for events at 3548 and 5800 ybp are included in Scenario 3.

Only one paleoliquefaction event can be attributed to the potential northern earthquake source and to the potential southern earthquake source during the past 5,800 years; therefore, the frequency of earthquakes on these sources is much lower than for the Charleston source. Because the distribution of paleoliquefaction sites is much more limited for a potential northern and southern source, Talwani and Schaeffer (Reference 317) infer that the magnitude for these earthquakes is significantly smaller (~ **M** 6) than the magnitude for the Charleston source (~**M** 7). Alternatively, the limited distribution of paleoliquefaction features could result from a more distant large magnitude earthquake. This alternative source location is included in the BLN source model through the regional source zone approach (e.g., Mesozoic Basin source zone; Figure 2.5-262). Based on the inferred smaller magnitude and lack of multiple events, additional explicit northern and southern source areas for the second model are not included in the BLN hazard model. For this interpretation, modeling of recurrence based on seismicity in the Atlantic Coastal Plain adequately represents the recurrence of earthquakes at Bluffton and Georgetown.

The distribution of mean repeat times for repeating large magnitude Charleston earthquakes was developed using the process described in Subsection 2.5.2.4.4.1 for the repeating large magnitude New Madrid earthquakes. The data for event dates are given in Table 2.5-217. Data for prehistoric Event G and later events were used to simulate 10,000 sets of inter-arrival times for Charleston earthquakes. Figures 2.5-269a and 2.5-269b show the resulting distributions for mean repeat time developed for the six recurrence scenarios given in Table 2.5-217. The consideration of the alternative completeness period of ~6,000 years leads to longer mean repeat times.

The distributions of mean repeat time for repeating Charleston large magnitude earthquakes were also represented in the hazard analysis by five-point discrete approximations. Table 2.5-215 lists the distributions for mean repeat time and the equivalent Poisson rate obtained using Equation (2.5.2-9) for a time increment of 50 years from the present. The Poisson and renewal recurrence models are again given equal weight, and the three-point discrete distribution for α developed by the Working Group (Reference 389) was used (Figure 2.5-268).

2.5.2.4.4.3 Ground Motion Models

The updated PSHA was conducted using the representation of CEUS ground motions developed by EPRI (Reference 350). As indicated in previous sections, the aleatory variability model (Reference 362) was substituted for the original EPRI (Reference 350) aleatory variability model because the variability of the original model was determined to be too extreme.

In addition, the Cumulative Absolute Velocity (CAV) model quantified by Hardy et al. (Reference 394) was incorporated into the final seismic hazard calculations. The CAV model accounts for the damageability of ground motions caused by small magnitude earthquakes. Use of the CAV model is as addressed in Regulatory Guide 1.208.

2.5.2.4.4.4 PSHA Results

The PSHA update was conducted by combining the hazard from EPRI-SOG seismic sources with updated maximum magnitude distributions as described previously with the hazard from the repeating large magnitude earthquake sources at New Madrid and Charleston. Earthquakes occurring within the EPRI-SOG sources were treated as point sources, consistent with the EPRI-SOG evaluation, and the distance adjustment and additional aleatory variability factors discussed in Subsection 2.5.2.4.4.3 were applied. Repeating large magnitude earthquakes on the central New Madrid faults were assumed to rupture the entire fault, and the closest approach of the fault to the site was used as the distance to rupture. The Charleston source was simplified to be represented by the

Woodstock fault (see Figure 2.5-262), because this fault is near the center of the alternative geometries and this fault has the highest weight among alternative geometries (see Figure 2.5-268). Further, the Charleston source contributes only a small percentage of the hazard at BLN, as is discussed below. This simplification does not affect hazard results in a significant way. The distance adjustment factors of the EPRI (Reference 350) models were not applied in calculating the hazard from these fault sources because the fault ruptures were specifically defined. As discussed in Subsection 2.5.2.4.4.1, the large magnitude earthquakes occurring on the central New Madrid faults were treated as clustered events using Equation (2.5.2-12), with rates given by Poisson or renewal models.

Figures 2.5-270 and 2.5-271 compare mean hazard results (for 10 Hz and 1 Hz, respectively) for the EPRI (Reference 233) seismic source characterization using the original EPRI (Reference 233) ground motion models and using the updated ground motion models with the revised aleatory variability model (without CAV). This comparison uses only the original EPRI (Reference 233) seismic sources. The effect of using the updated ground motion models on the 10-Hz hazard is to produce a small increase in ground motion for an exceedance frequency of 10^{-4} (about 10 percent) and larger increases in ground motions for lower exceedance frequencies (about 50 percent increase for an exceedance frequency of 10^{-5}). The higher ground motion at lower exceedance frequency is due in part to the higher aleatory variability (greater standard deviation) in the Reference 362 ground motion characterization compared to the aleatory variability used in the Reference 233 study. For 1-Hz motion hazard, use of the updated ground motion model results in lower ground motions for the mean hazard (25 percent to 40 percent decrease in the range of 10^{-4} to 10^{-5} exceedance frequency). This decrease results from the Reference 350 median ground motion estimates that account for a double-corner seismic source model, which reduces estimates of ground motions for low frequencies.

As an illustration of the importance of various seismic sources considered in the hazard calculations, Figures 2.5-272 and 2.5-273 show mean seismic hazard curves for the Weston teams for its EPRI sources, for the NMSZ, and for the Charleston seismic zone, for 10 Hz and 1 Hz, respectively. The Weston team is used for this illustration because it has one of the simpler seismic source interpretations in the 200-mi. radius of the BLN site among the EPRI teams (see Figure 2.5-239). These seismic hazard results include the updated ground motions in Reference 350, and the Reference 362 aleatory variability model, and the CAV filter. It is clear that the NMSZ is the dominant contributor to low frequency hazard (Figure 2.5-273). For high frequencies, the local source (WGC-24) representing the ETSZ dominates seismic hazard for ground motions with annual frequencies of exceedance below about $7x10^{-5}$. The Charleston seismic source has a relatively minor contribution to hazard for all response spectrum frequencies.

Figures 2.5-274, 2.5-275, 2.5-276a, 2.5-276b, 2.5-277, 2.5-278, and 2.5-279 show the total hazard results (with updated NMSZ and Charleston models, and with updated ground motion models including CAV) for structural frequencies of 100 Hz (PGA), 25 Hz, 10 Hz, 5 Hz, 2.5 Hz, 1 Hz, and 0.5 Hz. These curves are for the mean hazard and the 15th, 50th, and 85th fractile hazard. One feature of these curves for high frequencies (PGA, 25 Hz and 10 Hz) is a slight change in slope around an annual frequency of 10^{-4}, particularly for the 85th fractile hazard curve. This change in slope occurs where the hazard contributions from the NMSZ and the local sources (e.g. the ETSZ) cross, and because hazard curves from these individual sources have different slopes at 10^{-4} (see, for example, Figure 2.5-272).

In summary, the PSHA sensitivity analyses indicate that both the updated characterization of repeating large magnitude earthquakes in the New Madrid region (and, to a minor extent, in the Charleston region) and the updated EPRI (Reference 350) ground motion characterization and the aleatory variabilities (Reference 362) lead to changes in the seismic hazard at the BLN site at frequencies of exceedance of 10^{-4} to 10^{-5} that are important to defining the GMRS ground motions.

2.5.2.4.4.5 Uniform Hazard Response Spectra for Rock and Identification of Controlling Earthquakes

PSHA calculations were performed for PGA and SA at frequencies of 25, 10, 5, 2.5, 1, and 0.5 Hz (spectral periods of 0.04, 0.1, 0.2, 0.4, 1.0, and 2.0 seconds, respectively). Figure 2.5-280 shows the UHRS for rock site conditions developed from these results using the ground motion levels for each spectral frequency corresponding to the mean and median 10^{-4}, 10^{-5}, and 10^{-6} annual frequencies of exceedance. PGA is plotted at a frequency of 100 Hz (a period of 0.01 second).

The magnitude and distance for earthquakes controlling the hazard were identified following the procedure outlined in Appendix D of Regulatory Guide 1.208. Figures 2.5-281, 2.5-282, 2.5-283, 2.5-284, 2.5-285, and 2.5-286 show the deaggregation of the mean hazard for high frequencies (HF, the average of 5 and 10 Hz), and at low frequencies (LF, the average of 1 and 2.5 Hz), for mean annual exceedance frequencies of 10^{-4}, 10^{-5}, and 10^{-6}, respectively.

Table 2.5-218 lists the magnitudes and distances for the controlling earthquakes computed for the mean annual frequencies of 10^{-4},10^{-5}, and 10^{-6}. For the low-frequency hazard, earthquakes with distances greater than 100 km (62 mi.) comprise more than 5 percent of the hazard (see Figures 2.5-281, 2.5-282, 2.5-283, 2.5-284, 2.5-285, and 2.5-286). The **M** and R values for 10^{-4} and 10^{-5} are similar, so nominal values of **M**=7.7 and R=360 km are chosen to represent low frequencies. For the high-frequency hazard, the overall hazard results indicate **M** and R values that generally are intermediate to the bimodal distribution that is evident in Figures 2.5-281, 2.5-282, 2.5-283, 2.5-284, 2.5-285, and 2.5-286. (Figure 2.5-285, the HF deaggregation for 10^{-6}, is an exception, showing almost all contributions to hazard coming from small, local earthquakes.) Mean **M** and R values for R<100 km were calculated and used to select nominal values of **M**=5.9 and R=20 km. These values represent the hazard from small, local earthquakes and are a reasonable choice to develop high-frequency spectral shapes. The use of mean **M** and R for events with R<100 avoids using values of **M** and R that are intermediate to small, local earthquakes and large, distant earthquakes. Such intermediate events do not contribute significantly to hazard at BLN, and representing them with **M** and R values would lead to a spectral shape that is less accurate than representing the small, local earthquakes specifically in terms of **M** and R.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The BLN site is a hard rock site. Approximately two thirds of the measured rock section exhibits shear wave velocities of 9200 fps or greater. Therefore the EPRI ground motion equations were used directly, without calculation of the site response. The ground motion response spectrum reflects this hard rock condition (Subsection 2.5.2.6).

The site stratigraphy, including thickness, seismic compressional (Vp) and shear wave (Vs) velocities are presented in Subsection 2.5.4.1. Because the nuclear island is located on rock, the bulk densities, soil index properties, shear modulus and damping variations with strain level and the water table information are not relevant to the analysis as described in Subsection 2.5.4.6. There is no nonlinear rock behavior, as described in Subsection 2.5.4.7.

BLN COL 2.5-2 2.5.2.6 Ground Motion Response Spectrum

BLN COL 2.5-3 This subsection presents the development of the GMRS applicable to the BLN site. The horizontal GMRS spectrum was developed using the approach described in ASCE/SEI Standard 43-05 (Reference 328) and Regulatory Guide 1.208. The vertical GMRS spectrum was developed using vertical/ horizontal spectral ratios recommended in McGuire et al. (Reference 395).

At the BLN site, rock layers dip gently 15°-17° to the southeast. These dips are less than 20°, thus can be considered approximately horizontal.

2.5.2.6.1 Horizontal GMRS Spectrum

Mean horizontal response spectra were computed for the controlling earthquakes defined in Table 2.5-218. Spectral shapes were derived using the recommendations of McGuire et al (Reference 345). The Low Frequency (LF) spectral shapes were anchored to the UHRS values at 0.5, 1, and 2.5 Hz, and the High Frequency (HF) spectral shapes were anchored to the UHRS values at 5, 10, 25, and 100 Hz. In between these frequencies, interpolation was used. The HF spectrum below 5 Hz was calculated by scaling the HF spectral shape to the 5 Hz spectral amplitude. The LF spectrum above 2.5 Hz and below 0.5 Hz was calculated by scaling the LF spectral shape to the 2.5 Hz and 0.5 Hz spectral amplitudes, respectively. These smooth mean response spectra are shown in Figures 2.5-287, 2.5-288, and 2.5-289 for the mean 10^{-4}, 10^{-5}, and 10^{-6} hazard levels, respectively. An envelope spectrum was constructed for each hazard level. These envelope spectra are listed in Table 2.5-219.

The approach in Reference 328 defines a GMRS in terms of the site-specific UHRS as:

$$GMRS = DF * UHRS, \qquad (2.5.2\text{-}14)$$

where UHRS is the site-specific UHRS, defined for Seismic SDC-5 at the mean 10^{-4} annual frequency of exceedance, and DF is the Design Factor defined based on the slope of the mean hazard curve between 10^{-4} and 10^{-5} mean annual frequency of exceedance. The procedure for computing the GMRS is as follows.

For each spectral frequency at which the UHRS is defined, a slope factor A_R is determined from:

$$A_R=SA(10^{-5})/SA(10^{-4}) \qquad (2.5.2\text{-}15)$$

where $SA(10^{-4})$ is the SA at a mean UHRS exceedance frequency of 10^{-4}/yr and $SA(10^{-5})$ is the SA at a mean UHRS exceedance frequency of 10^{-4}/yr. The DF at this spectral frequency is given by:

$$DF= 0.6(A_R)^{0.80} \qquad (2.5.2\text{-}18)$$

and

$$GMRS = max[SA(10^{-4}) \text{ x } max(1, DF), 0.45 \text{ x } SA(10^{-5})]$$

The derivation of DF is described in detail in Commentary to Reference 328 and in Regulatory Guide 1.208. Table 2.5-220 shows the values of A_R and DF calculated at each structural frequency and the resulting GMRS. The horizontal GMRS is plotted in Figure 2.5-290.

2.5.2.6.3 Vertical GMRS Spectrum

The vertical GMRS spectrum was developed by using vertical to horizontal (V/H) response spectral ratios. Table 4.5 in Reference 395 provides recommended V/H spectral ratios for CEUS rock site conditions. The ratios are given as a function of spectral frequency and horizontal peak acceleration level. The V/H ratios for PGA<0.2g apply to the 10^{-4} UHRS, and the V/H ratios for $0.2g<PGA\leq0.5g$ apply

to the 10^{-5} UHRS. The vertical UHRS for 10^{-4} and 10^{-5} were calculated by multiplying the horizontal UHRS for 10^{-4} and 10^{-5} by the appropriate V/H ratio. The vertical GMRS was then calculated using the performance-based procedure described in Subsection 2.5.2.6.1. Table 2.5-221 documents the calculation at each spectral frequency. For frequencies where V/H ratios were not available from Table 4.5 of Reference 395 V/H ratios were calculated using log-log interpolation from adjacent frequencies.

The resulting vertical GMRS is listed in Table 2.5-221 and is shown in Figure 2.5-290.The free field peak ground acceleration at the finished grade level is less than or equal to a 0.30 g SSE (GMRS), as demonstrated on Figure 2.5-290.

BLN COL 2.5-3 Comparison of site-specific GMRS to the Certified Seismic Design Response Spectrum (CSDRS) is addressed in Subsection 3.7.1.1.

2.5.3 SURFACE FAULTING

BLN COL 2.5-4 This subsection describes the evidence gathered to support discussions and conclusions for faulting or the absence of faulting at the Bellefonte Nuclear Plant, Units 3 and 4 (BLN).

After describing the investigations performed (Subsection 2.5.3.1) the following aspects of the geology and seismicity of the site region are discussed:

- Geologic evidence, or lack thereof, for surface deformation (Subsection 2.5.3.2)
- Earthquakes associated with capable tectonic sources (Subsection 2.5.3.3)
- Ages of most recent deformation (Subsection 2.5.3.4)
- Relationship between tectonic structures in the site area and regional structures (Subsection 2.5.3.5)
- Characterization of identified capable tectonic sources (Subsection 2.5.3.6)
- Identified zones of Quaternary deformation (Subsection 2.5.3.7)
- Potential for surface tectonic deformation at the site (Subsection 2.5.3.8)

A capable tectonic source is a tectonic structure that can generate both vibratory ground motion and tectonic surface deformation, such as faulting or folding at or near the earth's surface in the present seismotectonic regime (Regulatory Guide 1.208). Minor karst features related to dissolution preferentially occur along joints at the site and in the site area, but these features do not pose a surface rupture or displacement hazard. Additional description of karst features is provided in Subsection 2.5.4.1.

2.5.3.1 Geological, Seismological, and Geophysical Investigations

Geologic, seismological, and geophysical investigations have been performed at the BLN site which characterize Quaternary tectonics, structural geology, stratigraphy, paleoseismology, and geological history for the site. The results of these investigations, including site and regional geologic maps and profiles that illustrate lithology, stratigraphy, topography, and structure, are presented in Subsections 2.5.1 and 2.5.4. Investigations specifically relevant to the evaluation of surface faulting are presented in this subsection.

The following investigations relevant to the evaluation of surface faulting have been performed as part of the BLN site characterization study:

- Compilation and Review of Existing Data and Literature—The 1986 FSAR for Bellefonte Units 1 and 2 (Reference 201) provides detailed geologic maps and descriptions of the stratigraphy and structure within a 5-mi. radius of the site. Detailed subsurface information from the construction reports for Bellefonte Units 1 and 2 that described geologic structures observed in the foundation excavations also was reviewed, and personnel involved in the Bellefonte Units 1 and 2 site characterization studies were contacted. Current published maps and literature pertaining to the structure, tectonics, and stratigraphy of the region also were reviewed.

- Interpretation of Aerial Photography—Pre-construction and post-construction aerial photographs were obtained from the U. S. Geological Survey, TVA files, the TVA Map Store in Chattanooga, Tennessee, and the U. S. Department of Agriculture. The photographs of the site were examined to look specifically for evidence of tectonic or non-tectonic (e.g., karst or dissolution features) surface deformation. The 1966 photos and selected 1935 pre-inundation photos provide coverage along the Sequatchie Valley thrust in the site area. These photos were reviewed to assess the presence or absence of geomorphic features indicative of potential Quaternary activity along this thrust fault, the only mapped fault within 5 mi. of the site.

- Lineament Analysis—Lineaments are natural linear features in the landscape that in some cases may be related to surface faulting. A comprehensive lineament map was created for the BLN site to evaluate potential areas of surface faulting (Figure 2.5-291). The map is based on an interpretation of pre-construction detailed topography (1"=400') and pre-construction aerial photography. Two previous investigations had identified lineaments across the Bellefonte site based on aerial photography and 1:24,000-scale topographic mapping (Reference 396). These investigations were conducted in support of the Final Environmental Impact Statement (FEIS) for the proposed fossil fuel conversion project (Reference 202) and in the 2006 GG&S report for the proposed southern site (Reference 399). The lineament map presented in Figure 2.5-291 supersedes these earlier efforts.

The lineament evaluation focuses on the entire BLN site. Lineaments were identified and mapped from two types of data: pre-development topographic mapping and pre-development aerial photography. In 1971 before construction, a 1”-400’ topographic map was prepared as part of the initial Bellefonte Units 1 and 2 site investigation (Reference 397). This map was the primary source for the identification and mapping of topographic lineaments and drainage features. A second topographic map, created for the BLN Units 3 and 4 project in 2006, supplemented the pre-development topography, particularly in areas outside the developed areas where grading had not altered the original topography.

Color infrared (CIR) photography taken in 1973 shows drainage features and tonal lineaments more clearly than other available photography, including earlier black and white imagery (Reference 398) (Figure 2.5-292), thus it was the primary source for mapping of tonal features. Other types of imagery such as Side-Looking Airborne Radar (SLAR) and Light Detection and Ranging (LIDAR), were not available for this geographic area and time period (pre-development).

Each lineament is described with respect to its characteristics and possible genesis (Table 2.5-222). Two lineaments, lineaments #4 and #12, were located such that they could potentially affect the Units 3 and 4 power block construction zone (Figure 2.5-291) and were the subject of focused field investigations. Methods of investigation included vertical borings, angled borings, test pits, microgravity surveys, and cone penetrometer (CPT) surveys.

- Field Reconnaissance—Field reconnaissance was conducted as part of the BLN characterization activities. The initial field reconnaissance focused on review of the geology of the site within 1 km (0.6 mi.) and 8 km (5 mi.) of the power block construction zone. Photolineaments and karst features observed from the review of previous studies and aerial photographs were reviewed in the field. A reconnaissance along the Sequatchie Valley thrust fault and related fold within the site area also was conducted. The second field reconnaissance was conducted in conjunction with an aerial reconnaissance. The structures within the Appalachian fold and thrust belt within 25 mi. of the site, and in the epicentral region of the 2003 Fort Payne earthquake, located approximately 25 to 30 mi. from the site were the primary focus of this reconnaissance. The reconnaissance included a review of the Quaternary deposits mapped along the Coosa River in the Gadsen to Weiss Lake region, approximately 20 mi. from the epicentral region of the Fort Payne earthquake, and a historical landslide also in the epicentral region of this earthquake.

An additional aerial reconnaissance was flown in September of 2006 to look for any evidence of surface faulting that might affect the site, and to identify lineaments. The flight focused on a 5-mi. radius from the plant (Figure 2.5-293). The locations of several of the lineaments identified from aerial photography were confirmed. No geomorphic evidence of active surface faulting or deformation was noted.

- Discussions with Current Researchers in the Area—Researchers familiar with the structural and tectonic framework of the region were contacted. These researchers provided recent published and in-press publications for our review.

- Review of Seismicity Data - A comprehensive review of both instrumental as well as historical earthquakes was completed (See Subsection 2.5.2.1).

2.5.3.2 Geological Evidence, or Absence of Evidence, for Surface Deformation

Two bedrock faults, the Sequatchie Valley and Big Wills Valley thrust faults, are mapped in Paleozoic rocks within 25 mi. of the site, and one of these, the Sequatchie Valley thrust fault, is within 5 mi. of the site (Figures 2.5-208 and 2.5-293). Basement faults that may have influenced the development of these thrust faults are inferred from interpretation of seismic profile data. Descriptions of these faults are presented in Subsection 2.5.3.2.1, and a discussion of the evidence that indicates they are not capable tectonic sources is presented in Subsection 2.5.3.6.

As noted in the Bellefonte Units 1 and 2 FSAR (Reference 201) and the FEIS (Reference 202), there is no intense folding or major faulting of the bedrock at the Bellefonte Units 1 and 2 site and adjacent area. Small-scale fractures and one small fault were identified in the Bellefonte Units 1 and 2 foundation excavation exposures. The minor displacement observed was investigated by core drilling and recorded by surface mapping (References 201 and 400). Similar fractures and small-scale shears were observed in the rock core for the Units 3 and 4 investigation. Appendices for Subsection 2.5.4.3 present boring logs with descriptions of these features.

Lineaments are identified from examination of pre-development sources including 1971 topographic mapping and 1973 aerial photography. Lineaments are classified as either “topographic,” mapped from topographic expression, or “tonal,” mapped from tonal contrasts on aerial photography. A summary of the results of the lineament analysis is presented in Subsection 2.5.3.2.2.

Lineaments mapped within the BLN site are primarily associated with bedding and jointing in the bedrock. The most prominent lineaments identified either trend parallel to bedrock strike at N40-45°E, or trend approximately orthogonal to strike at N45-50°W. Additional lineaments, trending N5°W to N25°E are identified. Strike-parallel tonal lineaments are attributed to differences in soil, moisture, or vegetation associated with bedrock lithology. The trends of the major topographic lineaments are attributed to weathering and erosion along vertical joints (References 201 and 202). Karst weathering is enhanced along many of the lineaments.

There is no geomorphic or geologic mapping evidence to suggest that any of these lineaments are associated with a capable tectonic fault or pose a surface rupture hazard.

2.5.3.2.1 Geologic Structures in the 25-Mi. Radius

Key observations made from the literature review and field reconnaissance regarding the geologic structures within a 25-mi. radius are summarized as follows.

2.5.3.2.1.1 Sequatchie Anticline and Sequatchie Valley Thrust Fault

The Sequatchie anticline is the most northwesterly structure of the southern Appalachians (References 401 and 402). It is an elongated asymmetric anticline that extends 250 mi. from Morgan County, Tennessee, to Jefferson County, Alabama. The northwest limb of the anticline is steep along its entire length; and, a northwest-verging thrust fault extends along the northwest flank of the anticline from near its northeastern end 150 mi. southwestward. At its north end, the anticline is formed over a ramp linking the Cambrian Rome Formation with Pennsylvanian clastic rocks (Reference 403). The displacement on this fault decreases toward the south to a point about 70 mi. southwest of the Alabama-Tennessee state line where the fault disappears completely (Reference 404). The gently dipping southeast limb at the BLN site extends into the broad, flat-bottomed Coalburg syncline that underlies Sand Mountain.

Wells drilled to Precambrian rocks indicate that depth to basement rocks is approximately 8400 ft. beneath the Sequatchie anticline in northern Alabama, and stratigraphic observations in well and regional seismic data suggest that the structure merges into the regional detachment near the base of the Paleozoic cover sequence (References 402 and 404).

The Sequatchie Valley thrust fault is mapped along the northwest margin of Backbone Ridge, within 2.1 mi. of the power block construction zone at its closest distance (Figure 2.5-293). No exposures of the Sequatchie Valley thrust fault were observed during the reconnaissance for the GG&S study (Reference 399) and none are described in the mapping conducted as part of the Bellefonte Units 1 and 2 site characterization activities (Reference 201). Excavations at the Scottsboro waste transfer facility located approximately 3.6 mi. west of the BLN site along Backbone Ridge provide good exposures of steeply dipping strata and deformation (possibly backthrusts) in the hanging wall of the Sequatchie thrust. Backbone Ridge is formed where Fort Payne chert is preserved in the hanging wall of the thrust fault (Reference 405). Backbone Ridge terminates at the western margin of the Guntersville Reservoir along Mud Creek, approximately 3.5 mi. north of the BLN site. Northeast of Mud Creek, the Fort Payne chert is absent and the strong geomorphic expression of the steeply dipping beds is less apparent.

No deformation or geomorphic features indicative of potential Quaternary activity have been reported in the literature for this fault, and none were identified during aerial and field reconnaissance and air photograph interpretation.

2.5.3.2.1.2 Wills Valley Anticline and Thrust Fault

The Wills Valley anticline and associated thrust fault also lies within the northwestern (frontal) part of the Appalachian thrust belt, which is characterized by broad, flat-bottomed synclines and large-scale, northeast-trending, narrow asymmetric anticlines. The Wills Valley fault is located 17 mi. southeast of the BLN Site (Reference 405).

The Wills Valley thrust fault, like the Sequatchie Valley thrust fault, is of regional extent and merges into the regional detachment at a depth of about 10,000 ft. (References 404 and 406). The thrust fault crops out at the western margin of the Wills Valley on the eastern flank of Sand Mountain. No deformation or geomorphic features indicative of potential Quaternary activity have been reported in the literature for this fault, and none were identified during aerial and field reconnaissance.

2.5.3.2.1.3 Sub-detachment Basement Faults

Based on interpretation of seismic reflection profile data, Bayona and others (Reference 406) and Thomas and Bayona (Reference 256) identify faults within the basement below the detachment that appear to have controlled the location of the Sequatchie Valley and Wills Valley thrust faults and folds (Figure 2.5-220). The inferred subdetachment basement fault associated with the Sequatchie Valley thrust fault is shown to lie at depth, approximately 2 to 3 mi. southwest of the site. The fault location is based on correlations between picks on seismic lines that are located approximately 23 mi. and 33 mi. to the northeast and southwest of the BLN Site, respectively (Figure 2.5-220). The closest distance of the inferred basement fault in Wills Valley is approximately 19 mi. There are no seismicity alignments or surface geologic evidence to indicate that these faults have been reactivated in the current tectonic stress field (Figure 2.5-294). No deformation or geomorphic features indicative of potential Quaternary activity have been reported in the literature for these faults, and none were identified during aerial and field reconnaissance.

2.5.3.2.2 Results of Lineament Analysis

Using methods described in Subsection 2.5.3.1, a lineament analysis was conducted to identify and characterize lineaments within the BLN site, and to evaluate their significance relative to surface faulting, and weathering and erosion processes. Lineaments are shown on a 1971 topographic base map in Figure 2.5-291.

The overall pattern of lineaments identified during this investigation agrees with the results of previous studies including the 1997 FEIS (Reference 202) and the 2006 GG&S report (Reference 399). The overall orientations of the prominent lineaments in the site area are also consistent with the orientation of major joint sets observed in the Appalachian Plateau region (i.e., one across and one subparallel to the strike of major structures) (Reference 403) and with the orientation of joints mapped in the excavation exposures for Bellefonte Units 1 and 2 (Reference 201)

2.5.3.2.2.1 Classification and Origin of Lineaments

The lineaments mapped are divided into categories based on nature and possible origin. Lineaments shown are classified as either topographic or tonal lineaments. Topographic lineaments are those seen in the shape of the topography and include linear valleys, linear drainages, lines of saddles, and gaps in the hills. Tonal lineaments are those seen on the aerial photographs, and include contrasts in tone reflecting differences in vegetation, moisture, rock type, or soil characteristic that are linear in nature.

Most lineaments mapped, including tonal lineaments, are strike-parallel (N 40-45° E) and probably represent bedding contacts, or enhanced weathering and erosion along either bedding or prominent strike-parallel high-angle joints. These may be due to contrasts in the lithology of the bedrock, or the density of strike-parallel vertical joints in the bedrock. Linear valleys may owe their existence to deeper weathering and consequent erosion along less resistant beds of the limestone, either because of their lithology or a closer spacing of vertical joints. Strike-parallel lines of saddles are also a reflection of enhanced weathering and erosion of less resistant beds in the limestone.

Strike of bedding is also reflected in tonal features visible on aerial photography. Tonal contrasts reflect differences in soil type, depth, or moisture corresponding to bedrock lithology. In addition, lines of seeps or points of channel initiation can occur where a more argillaceous unit functions as an aquiclude, forcing groundwater to the surface (Figure 2.5-292).

Other lineaments are perpendicular to strike (N45-50°W) or at an angle to strike (N25°E, N5°W, N0°E, N15°W). These are seen in linear valleys, gaps, and drainage channels that cut across strike-parallel ridges. Most are less than 2000 ft. long, and probably represent weathering and erosion along other prominent high angle joints (Figure 2.5-291). Their limited length and lack of associated active-tectonic geomorphic features would be inconsistent with interpretation as active fault traces.

Two major lineaments, each more than 1 mi. in length, cut approximately perpendicular to strike and extend the entire width of the peninsula. Both curve somewhat and cross River Ridge in deep ravines. These lineaments also probably represent weathering and erosion along prominent joints, but based on their extent and shape, and the fact that they cut deep gaps through the hills, these lineaments are interpreted to be the remnants of paleo-valleys, eroded by Town Creek or a similar surface drainage active earlier in the Quaternary.

These two lineaments are part of a series of subparallel lineaments that is apparent along the Tennessee River Valley near the BLN site, and are part of the overall topographic fabric of the area. The location of these lineaments likely is related to large-scale structural deformation that resulted in joint formation in the limestone bedrock. These joints cause the bedrock to become weaker compared to surrounding rock, and also become areas of preferential groundwater flow. The result is that surface erosion and karst development occur preferentially along these features, resulting in the topographic expression. Karst weathering appears to be enhanced along their trend, as shown in Subsection 2.5.4.1.

Studies at the southern site documented enhanced karst weathering along strike-parallel lineaments (Reference 399). Seismic refraction and microgravity surveys showed localized depressions in the top of less-weathered bedrock (higher P-wave layer) that correlated with some of the more prominent lineaments at the site. These lineaments appear to represent either strike-oriented belts of dipping strata that may be prone to dissolution relative to surrounding strata, or cross-structural solutionally enlarged joints and/or fractures that have facilitated groundwater movement and weathering.

2.5.3.2.2.2 Investigation of Lineaments

Detailed field investigations were undertaken of two relatively short lineaments located within or adjacent to the Units 3 and 4 power block construction zone. Lineament #4 underlies the Unit 4 power block. Lineament #12 is located outside the power block construction zone but projects toward Unit 3.

Lineament #4 is located in the axis of a small linear valley and is mapped to coincide with the linear drainage channel that occupied it (Figure 2.5-291). The valley and drainage are approximately parallel to the strike of bedding and the lineament approximately follows the contact between lithologic units A and B. A comparison of 1971 and 2006 topography and inspection of aerial photography taken during construction show that this valley was filled in during construction of the parking lot for Bellefonte Units 1 and 2 in the 1970s and 1980s. Proposed power block Unit 4 directly overlies this lineament, the former valley and its drainage (Figure 2.5-291).

Data show that lineament #4 represents a zone of deeper weathering associated with a bedding contact and topographic lowland. Karstification is found to occur along this lineament, and to be especially deep (40 to 50 ft.) near the southwest corner of the Unit 4 power block (turbine building, away from the nuclear island), where cavities are encountered in borings and geophysical methods show a depression in the bedrock surface. Subsection 2.5.4.1 provides a detailed discussion regarding karst features associated with lineament #4. Lineament #4 is not coincident with a fault therefore is not a tectonic feature.

Lineament #12 is a topographic lineament following the trough of a linear valley that trends N 25 ^{o}E. It is 1,200 ft. in length, and is located parallel and adjacent to lineaments #11 and 13, each of similar length (Figure 2.5-291). It is located outside the power block construction zone, but projects toward the center of Unit 3.

Investigations show that Lineament #12 represents a zone of enhanced weathering and erosion associated with a high angle joint along which some minor shearing may have taken place. Based on available data, the shearing may be associated with deformation that created the Sequatchie anticline in late Paleozoic time. The joint does not continue into the power block construction zone.

The lineament identifies no geomorphic evidence that would indicate differential uplift or surface deformation (e.g., warping, tilting) associated with the lineaments that intersect or project toward the power block construction zone. This absence of differential uplift or surface deformation indicates that these lineaments are not related to capable tectonic faults.

2.5.3.2.2.3 Conclusions of Lineament Analysis

Lineaments mapped within the BLN site are primarily associated with bedding and jointing in the bedrock. Topographic lineaments are associated with enhanced weathering and erosion along bedding planes or bedrock joints, and with older drainage patterns. Tonal lineaments are expressions of differences in soil, moisture, or vegetation associated with bedding. Karst weathering is enhanced along many of the lineaments. There is no geomorphic or geologic mapping evidence to suggest that any of these lineaments is associated with a capable tectonic fault or pose a surface rupture hazard.

2.5.3.3 Correlation of Earthquakes with Capable Tectonic Sources

Historical earthquakes within 25 mi. of the site are not known to be associated with any mapped fault. Seismicity data from a full instrumental catalog compiled by the Southeastern U. S. Seismic Network (SEUSS) (Table 2.5-222, Reference 407) are plotted on Figure 2.5-294 along with mapped traces of surface faults. None of the earthquakes plotted within the 25-mi. radius are known to be associated with any mapped surface fault. Earthquake epicenters do not appear to be spatially related to these faults, and the earthquake depths are not compatible with the depths of these faults. Most earthquake focal depths are greater than 3 km (Table 2.5-223). The Sequatchie Valley and Big Wills Valley faults are thought to merge into the regional detachment at about 3 km (Reference 404) thus are too shallow to be the source of this seismicity.

Historical earthquakes in this area, including the M 4.6 Fort Payne earthquake in 2003, have been postulated to be associated with reactivated faults in the basement below the Appalachian detachment, however, no specific seismogenic basement faults have been identified (Subsection 2.5.1.1.4.2.4.2). There is no apparent correlation between the location of historical seismicity within 25 mi. of the site and inferred subdetachment basement faults (Figure 2.5-220). No capable tectonic sources are present that could extend to within 5 mi. of the site and cause surface deformation.

Potential seismogenic sources inferred from seismicity that is occurring in basement rocks below the detachment are considered in the characterization of alternative seismic sources included in the Probabilistic Seismic Hazard Analysis (PSHA) (Subsection 2.5.2).

2.5.3.4 Ages of Most Recent Deformation

As discussed in Subsection 2.5.3.2, none of the faults within the 40-km (25-mi.) radius or 8-km (5-mi.) radius exhibit evidence for Quaternary activity. The mapped surface faults formed during the culmination of the Alleghanian orogeny at the end of the Paleozoic. Hatcher and others (Reference 403) summarize evidence for the timing of deformation of the foreland during the Alleghanian orogeny. Deformation of the foreland affected rocks as young as Pennsylvanian and early Permian, and may have continued later. Foreland deformation may have ended as early as 286 to 266 million years (Ma), the age of the youngest deformed foreland unit, the Dunkard Group in Pennsylvania, Ohio, and West Virginia. The subdetachment basement faults are inferred to be Iapetan normal faults that likely formed initially during the late Proterozoic and early Paleozoic. These faults may have been reactivated during subsequent orogenies, but there is no evidence of surface deformation associated with reactivation of these faults in post-Alleghanian time.

2.5.3.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures

As described in Subsection 2.5.1.1.4.2.1, mapped surface bedrock faults within 40 km (25 mi.) of the BLN site are part of the regional Appalachian foreland fold-thrust belt that developed during the Alleghanian orogeny. The culmination of the Alleghanian orogeny occurred in the late Paleozoic. There is no new information to suggest that the thrust faults within the Appalachian foreland thrust belt are capable tectonic structures as defined by Regulatory Guide 1.208 (Appendix A). The subdetachment basement faults are inferred to represent the most cratonward of a zone of Iapetan normal faults.

2.5.3.6 Characterization of Capable Tectonic Structures

A ‘capable tectonic source’ as defined by the Nuclear Regulatory Commission is a tectonic structure that can generate both vibratory ground motion and tectonic surface deformation such as faulting or folding at or near the earth's surface in the present seismotectonic regime (Regulatory Guide 1.208, Appendix A). It is described by at least one of the following characteristics:

a. Presence of surface or near-surface deformation of landforms or geologic deposits of a recurring nature within the last approximately 500,000 years or at least once in the last approximately 50,000 years

b. A reasonable association with one or more moderate to large earthquakes or sustained earthquake activity that are usually accompanied by significant surface deformation

c. A structural association with a capable tectonic source that has characteristics of either item a or b (above), such that movement on one could be reasonably expected to be accompanied by movement on the other

The two mapped bedrock faults within the 40-km (25 mi.) radius of the BLN site, the Sequatchie Valley thrust fault, and the Big Wills Valley thrust fault, are judged not to be capable tectonic sources. This conclusion is based on the following information: (1) both faults merge into the regional detachment at depths of about 1.6 and 1.9 mi., respectively, and, thus, do not extend to the hypocentral depth at which moderate to large magnitude earthquakes typically nucleate; (2) northeast-trending thrust faults are not favorably oriented for reactivation in the contemporary stress field (northeast to east-northeast-directed maximum horizontal compression); (3) no evidence of Quaternary deformation is reported in the literature or was observed during field and aerial reconnaissance; and (4) instrumentally recorded seismicity, including the 2003 Fort Payne earthquake (the largest earthquake recorded in the Eastern Tennessee Seismic Zone [ETSZ]) generally occurs within basement rocks below the Paleozoic cover sequence overlying the regional Appalachian detachment (at depths of greater than 5 km [3 mi.]).

No historical earthquake in the site region has been known to cause faulting at or near the surface. Neither the Sequatchie Valley thrust fault nor the Big Wills Valley thrust fault are genetically or structurally related to any known capable tectonic source. The Bellefonte Units 1 and 2 FSAR (Reference 201) concludes that structurally related major northeast-trending faults within the Valley and Ridge Province are inactive based on: (1) detailed geologic mapping investigations throughout the province in which no evidence of active faulting since the Paleozoic is described, implied, or inferred; (2) dating of a sample from the Copper Creek fault near the Clinch River breeder reactor site by potassium-argon methods that indicated last movement occurred 280-290 million years ago (Ma); and (3) a core boring through the Missionary Ridge fault (at Chickamauga Dam) that indicated that material had recrystallized along the fault and core samples from the Tellico Project that showed the Knoxville fault as an unbroken sample at several locations.

The inferred sub-detachment basement faults within the 25-mi. radius also are judged not to be capable tectonic sources. There is no apparent association of seismicity with these faults (Figure 2.5-235), and there is no evidence of surface or near-surface Quaternary deformation to suggest that these faults are capable tectonic sources.

2.5.3.7 Designation of Zones of Quaternary Deformation in Site Region

No significant zones of Quaternary deformation that would require additional investigation are identified within 8 km (5 mi.) or 40 km (25 mi.) of the BLN site. No evidence for surface deformation at the site was observed during the field investigations, lineament analysis, or aerial reconnaissance. Based on review of historical Bellefonte Units 1 and 2 documents, and new mapping and subsurface investigations conducted for the BLN site, the lineaments mapped at the site likely are related to lithologic differences, solutionally enlarged joints, and/or fractures that have facilitated groundwater movement and weathering in the Stones River Group rocks (see discussion in Subsection 2.5.4.1). Small drainages and gullies appear to have localized along the zones of more weathered bedrock.

2.5.3.8 Potential for Surface Tectonic Deformation at the Site

The potential for tectonic deformation at the BLN Site is judged to be negligible. This conclusion is based on mapping of bedrock in the region (References 201 and 405) (Figure 2.5-228) that identified no evidence for surface faulting or deformation within BLN site area, and the absence of geomorphic features indicative of Quaternary deformation as reported in the previous reports and literature. Observations made during the field and aerial reconnaissance are consistent with this finding. The lineament analysis and field investigations conducted at the BLN site identified no geomorphic evidence that would indicate differential uplift or surface deformation (e.g., warping, tilting) associated with the lineaments that intersect or project toward the site. This absence of differential uplift or surface deformation indicates that these lineaments are not related to capable tectonic faults.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

BLN COL 2.5-1 This subsection presents information on the properties and stability of soils and rock that may affect the nuclear power plant facilities, under both static and
BLN COL 2.5-5 dynamic conditions, including vibratory ground motions associated with the Ground Motion Response Spectrum (GMRS). The discussion focuses on the stability of the materials as they influence the safety of seismic Category 1

facilities and presents an evaluation of the site conditions and geologic features that may affect the power plant structures or their foundations.

This subsection is organized into sub-subsections as presented in Regulatory Guide 1.206. These include:

- Geologic Features (2.5.4.1)
- Properties of Subsurface Materials (2.5.4.2)
- Foundation Interfaces (2.5.4.3)
- Geophysical Surveys (2.5.4.4)
- Excavations and Backfill (2.5.4.5)
- Groundwater Conditions (2.5.4.6)
- Response of Soil and Rock to Dynamic Loading (2.5.4.7)
- Liquefaction Potential (2.5.4.8)
- Earthquake Site Characteristics (2.5.4.9)
- Static Stability (2.5.4.10)
- Design Criteria (2.5.4.11)
- Techniques to Improve Subsurface Conditions (2.5.4.12).

2.5.4.1 Geologic Features

This subsection evaluates non-tectonic processes and features that may cause permanent ground deformation or foundation instability at the Bellefonte Nuclear Plant Units 3 and 4 (BLN) safety-related facilities. Processes and features evaluated include areas of actual or potential surface or subsurface subsidence, solution activity, uplift, or collapse, and the causes of these conditions, zones of alteration or irregular weathering profiles, zones of structural weakness, history of erosion and deposition, unrelieved residual stresses in bedrock, and rocks or soils that may be unstable due to their physical or chemical properties.

The area evaluated is shown on Figure 2.5-295. The Units 3 and 4 power block construction zone is defined by a rectangular boundary, approximately 1600-ft. long by 750-ft. wide. Exploration was focused within the construction zone, with a lower level of effort expended in the surrounding area. Figure 2.5-201 shows the extent of the entire BLN site.

As background for this subsection, descriptions, maps, and profiles of regional and site geology are previously presented in Subsections 2.5.1 and 2.5.2. Detailed descriptions of site geologic and geotechnical conditions encountered during the field investigations at the BLN site are presented in Subsections 2.5.4.2, 2.5.4.3, and 2.5.4.4.

2.5.4.1.1 Geologic History and Stress Conditions

This subsection reviews aspects of geologic history that are relevant to the potential for uplift and unrelieved residual stresses in the bedrock or soil. Information on site geologic history is summarized from Subsection 2.5.1.2.

The BLN site last experienced major tectonic uplift in the Pennsylvanian and Permian periods during the Alleghanian orogeny, which caused the folding and faulting observed today in the northeast to southwest trending valley and ridge system. At this time the Paleozoic strata on which the site is located were uplifted and gently folded. The history of the site since that time has been one of steady weathering and erosion. Thick residual soils developed in place over the carbonate rocks as a result of chemical weathering. Colluvial deposits developed to cover most hill slopes as the uppermost rock layers (generally sandstones that are resistant to weathering in this climate) slowly broke down or were undercut by erosion of softer underlying beds, and migrated down the slopes.

At present the site lies within a compressive midplate stress province characterized by a relatively uniform compressive stress field with a maximum horizontal shear (SHmax) direction oriented northeast to east-northeast (NE to ENE) based on earthquake focal mechanisms, in situ stress measurements, borehole breakout data, and recent geologic features (References 236 and 237).

Locally, the stress regime can be characterized as an unloading condition. Weathering and erosion have been relatively slow, allowing for gradual release of stress. Stress relief may be expressed in more closely spaced joints near the surface. There was no past glacial loading. These conditions are not conducive to high “locked in” residual stresses in the rock.

Gradual erosion of overburden combined with wetting and drying has resulted in preconsolidation of the overburden soils, a condition where the present overburden stresses are less than past overburden pressures at that depth. Soil test results show preconsolidation ratios of 5 to 9.1 (see Subsection 2.5.4.2.2.4), with higher values occurring in the upper 4 ft. of the soil. The fact that preconsolidation ratios are higher in the upper rather than the lower layers of soil suggests that wetting and drying, a more active process in the near-surface, may be a key factor in the overconsolidation of BLN soils.

There is no evidence of uplift occurring at the present time, and there are no geologic processes that are expected to lead to uplift at the site.

2.5.4.1.2 Soil and Rock Characteristics

This subsection presents information on the physical and chemical properties of the soil and rock, and an evaluation of zones of alteration and zones of potential weakness. The information is partly summarized from previous studies conducted at the site, but is primarily derived from the compilation and analysis of borehole and other site-specific information collected during the 2006 exploration program for Units 3 and 4.

Borehole data show that the bedrock within the power block construction zone is overlain by residual silts and clays, 5 to 40-ft. thick, derived from in-situ weathering of the underlying rock. Soil mineralogic analyses previously conducted on samples from monitoring wells throughout the BLN site show that the dominant mineralogy of the soil consists of clays, quartz, and calcite, with traces of iron oxides. Clay minerals include illite, kaolinite, montmorillonite, and muscovite (Reference 411). Drilling and excavation experience at the site and in adjacent areas shows that the transition from residual soil through weathered rock to hard, unweathered bedrock is gradual to abrupt.

Overburden soils within the power block construction zone have been disturbed by construction activities for Bellefonte Units 1 and 2, as shown on the 0.6-mile geologic map (Figure 2.5-230). Most of the Units 3 and 4 power block construction zone lies in areas disturbed by previous construction activities. These areas include paved parking areas and roadways, and graded lands partially covered with placed fill (Figure 2.5-295). A comparison of original (1971) and present-day (2006) topography within the Units 3 and 4 power block construction zone shows a complex of areas that are both higher and lower than original grade (Figure 2.5-296) (Reference 397). Borings drilled in areas now higher than original grade encounter fill materials. Only the north corner of the construction zone overlies original landscape with no discernible grade changes.

Original surface drainage indicated on Figure 2.5-296, was interpreted from historical aerial photographs and topographic maps. Both Units 3 and 4 partially overlie former swales in the original topography. These swales were occupied by first-order ephemeral drainages that drained northeast toward Town Creek.

Beneath the residual soil cover are the gently dipping beds of the Stones River Group. Folding of the bedrock is gentle throughout the BLN site and the Units 3 and 4 power block construction zone. Strike and dip are consistent across the construction zone, approximately N 45° E and 15° SE, respectively (Subsection 2.5.1.2.5). No deflection is present in structure contours that might indicate deformation is present (Figure 2.5-297). Bedding and jointing are the major discontinuities (Figure 2.5-298); however, minor discontinuous shears also occur, with slickensides on the faces of joints or bedding planes common. None of these discontinuities represent significant zones of weakness. However, they can provide pathways for water, localizing subsequent weathering and solution. Dilation of the upper zones of rock helps to open these discontinuities and facilitate the weathering process near the bedrock-soil interface. These weathered areas then can become potential zones of weakness, as discussed in Subsection 2.5.4.1.3.

The Stones River Group is comprised of subunits differentiated by composition and texture. Rock core from the Units 3 and 4 exploratory drilling program consists of interbedded limestone, dolomitic limestone, argillaceous and silty limestone, and cherty limestone. Within the Units 3 and 4 power block construction zone, six distinct lithologic units, designated units A through F, are identified and described. Together, units A through F are informally designated the “middle Stones River.” Beds above and below these six units are informally designated “upper Stones River” and “lower Stones River,” respectively. The middle Stones River comprises a total thickness of 453 ft.; the entire Stones River Group is 1050-ft. thick (Table 2.5-205).

Downhole shear wave velocity profiles for eight of the boreholes, combined with rock core photographs for boreholes, formed the basis for identifying the lithologic units and correlating them throughout the construction zone. A stratgraphic column showing a composite shear wave velocity profile for these six lithologic units is presented in Figure 2.5-299. These units correlate with similar lithologic units identified in rock core drilled for Bellefonte Units 1 and 2 as shown in Table 2.5-205 (Reference 201). Subsection 2.5.1.2.4 presents a summary description of each lithologic unit, based on field descriptions and petrographic, chemical, and mineralogic analyses.

Laboratory analyses of selected core samples from the Units 3 and 4 power block construction zone were performed to determine proper lithologic classification and rock characteristics. Analyses included petrographic, chemical, and mineralogical analyses.

Petrographic analysis of twenty-eight samples of rock core provided information on the formation and mineralogy of the Stones River Group underlying the Units 3 and 4 power block construction zone. The samples submitted for analysis were described in the field as limestone (including micrite, packstone, and wackestone types), shale, limestone with shale beds, limestone with wavy shale laminae, and chert. Table 2.5-224 lists the samples analyzed and their field descriptions. Figures 2.5-300, 2.5-301 and 2.5-302 show photomicrographs of six representative rock samples, cut into blocks and polished before preparation of the thin sections for petrographic analysis.

Petrographic results show that samples that were identified in the field as “limestone” are composed primarily of calcite with varying amounts of dolomite (Table 2.5-224). The dark-colored “wavy shale laminae” distributed within the limestone are mineralized with dolomite. Clay was noted in these laminae, but was seen as a minor constituent relative to dolomite. Other samples identified in the field as “shale” or “shale beds,” however, did contain significant percentages of clay and silt-sized quartz, and had retained some original sedimentary lamination. Still, dolomite and calcite were seen as the dominant minerals, thus it remained unclear whether these samples could be classified as shale, and further analysis was performed.

The visual estimation of the percentages of clay minerals in petrographic thin sections is problematic because of their small size, thus additional analyses were conducted to obtain a more quantitative estimate of the amount of clay in these samples. Thermogravimetric analysis (TGA) combined with X-ray diffraction was conducted on a subset of eleven of the original 28 samples (Table 2.5-224). The TGA method provides a quantitative estimate of the percentages of carbonate minerals in a sample, whereas X-ray diffraction identifies the minerals within a sample and gives their relative percentages based on the height of peaks on the diffractogram. The samples were selected to represent either the limestone portion of the sample (L) or a “shale bed” or “wavy shale laminae” (S).

The TGA results confirm that the samples identified as “limestone” are composed primarily of calcite and dolomite. The dolomite mineral is identified as ankerite, an iron-rich member of the dolomite group. Total carbonate percentages in the limestone samples vary from 68 to 96 percent, averaging 83 percent. Dolomite (ankerite) makes up from 0 to 36 percent of this carbonate (Table 2.5-224).

The shale samples again fall into two categories based on the TGA and X-ray diffraction analyses (Table 2.5-224). Those described as “wavy shale laminae” (BLN-2S, 3S) are dominated by dolomite (41 to 50 percent) and contain significant calcite (18 to 37 percent). The remaining percentage of non-carbonate minerals including clay (23 to 32 percent) is not enough for the material to be considered shale. X-ray diffraction shows the non-carbonate minerals to be quartz, muscovite, and pyrite. These results are consistent with the petrographic results, which also show high dolomite mineralization in these laminae. It is concluded that these wavy laminae are not shale and are instead impure dolomitic limestone (Figure 2.5-300).

By contrast, the samples described as “shale” or “shale beds” contain relatively less carbonate and more detrital materials, including clay and quartz (Table 2.5-224). In samples BLN-14S and BLN-10S, calcite and dolomite occur in approximately equal amounts and together comprise only 32 percent and 44 percent respectively of the rock. The remainder consists of quartz, muscovite, kaolinite, and pyrite. Given that quartz is more abundant in the samples than clay (kaolinite + muscovite) based on X-ray diffraction results, clay surely comprises less than 50 percent of the rock. Thus, these rocks also do not meet the requirements for shale (Reference 412). An appropriate rock name for the “shale” described in rock core in the field is suggested to be “argillaceous and silty limestone,” although specific beds may occur that could be classified as “calcareous shale,” or “calcareous siltstone.”

Petrographic, chemical, and mineralogic analyses result in the following lithologic classifications. Lithologies described as “shale,” “interbedded limestone and shale,” or “calcareous shale” in field logs are classified as “argillaceous and silty limestone.” Material described as “wavy shale laminae” in field logs are “wavy dolomitic laminae." Lithologies described as "limestone" may be either "limestone" or "dolomitic limestone."

Examination of textures and structures in petrographic thin sections show that the rock is fresh and compact without zones of alteration. Primary features in the rock such as fossils, pellets, and sedimentary laminations are generally intact and undeformed, indicating that tectonic uplift and folding has not resulted in any pervasive shearing, brecciation, or weakening of the rock (Figures 2.5-300, 2.5-301 and 2.5-302). Secondary features seen in thin section, such as stylolites, chert nodules, and dolomite mineralization, are indicative of diagenetic processes. These processes took place millions of years ago during burial and lithification, and include pressure solution that formed stylolites, and the migration of magnesium- and silica-rich fluids resulting in dolomite mineralization or replacement of carbonate with silica (chert) (Figures 2.5-300, 2.5-301 and 2.5-302).

In summary, the Units 3 and 4 power block construction zone is located on gently sloping topography underlain by a mantle of residual soil, over limestones of the Stones River Group. Lithology varies within the group and includes limestone, dolomitic limestone, and argillaceous and silty limestone, with minor chert-rich zones. Other than limestone which weathers primarily by dissolution (discussed in Subsection 2.5.4.1.3), no rocks or soils that might be unstable because of their mineralogy, lack of consolidation, water content, or potentially undesirable response to seismic or other events are present at the site. No zones of alteration in the limestone are present, but both primary sedimentary features and secondary diagenetic features are present. Rock is fresh, hard, and compact.

Zones of structural weakness, such as extensive fractured or faulted zones are not present; however, joints and bedding planes are present, along with minor discontinuous shears. These discontinuities are not themselves a source of weakness but may serve as pathways for water, along which weathering and dissolution of the limestone can take place.

2.5.4.1.3 Weathering Processes and Features

The BLN site is underlain by limestone, a rock type that weathers primarily by dissolution. Dissolution features, also termed karst features, are common throughout northern Alabama wherever limestone bedrock occurs. Jackson County contains extensive cave systems and sinkhole plains. A total of 1,526 caves are reported in the county (Figure 2.5-303) (Reference 413).

Investigations at the BLN site by TVA, both past and present, have not identified large-scale karst features (Reference 201). No natural sinkholes have been identified and no enterable caves have been located. Thick, pure limestones like the Tuscumbia, Monteagle, and Bangor Limestones that host large caverns elsewhere in Jackson County, do not occur at the site. Nevertheless, the underlying impure limestones of the Stones River Group are found to weather primarily by dissolution, and small-scale karst features are present.

Karst features at the BLN site are of a somewhat different character and smaller scale than highly karstified areas of northern Alabama. Factors such as relief, hydraulic gradient, and purity of the limestone beds have combined to produce a more subtle karst terrain.

The relief and hydraulic gradient at the BLN site are not favorable for the development of large cavern systems. In lowland areas like the BLN site, where limestone units have little relief, are relatively close to groundwater levels, and groundwater has relatively low hydraulic gradients, cave systems that can be entered and explored are not known. A map of the distribution of caves in Jackson County shows hundreds of caves in the adjacent highlands, but none within the Sequatchie Valley (Figure 2.5-303; Reference 413). Cave locations shown immediately east of the site are associated with the northeast-trending escarpment of Sand Mountain, approximately 1.5 miles east of the BLN site where the Mississippian Bangor and Monteagle Limestones crop out beneath the Permian sandstone cap.

Thick beds of pure limestones are not present at the BLN site. The limestone underlying the Units 3 and 4 power block construction zone belongs to the Ordovician Stones River Group and consists of beds of relatively pure limestone 80 to 100 percent carbonate) alternating with beds of argillaceous and silty limestones (30 to 80 percent carbonate). (See Subsection 2.5.4.1.2 for detailed lithology and mineralogy.) The presence of the impure limestone beds may inhibit development of larger conduits and favor smaller ones. Klimchouk and Ford (Reference 414) note that most caves are associated with bulk purities of greater than 90 percent carbonate. Argillaceous and silty limestones do not dissolve completely; they leave a residue that can clog incipient solution conduits. Pure limestone beds, while present in the Stones River Group, are interbedded with less pure rocks. Thus the size of conduits may be restricted by the thickness of these pure limestone beds. In addition, flow can be localized at the contacts with argillaceous interbeds.

A karst model was developed by TVA for the BLN site along with a review of previous groundwater studies at the site. A brief summary of this model is presented below. Additional discussion of the model with respect to groundwater at the site is presented in Subsection 2.4.1.2.

In the TVA model groundwater flow at the BLN site occurs within three different stratigraphic horizons: the “overburden,” the “epikarst” zone, and the deeper “karst” zone (Figure 2.5-304). The epikarst zone includes the upper weathered and dilated bedrock zone at the base of the overburden soils, characterized by differential weathering of the bedrock and stress relief dilation of the upper bedrock beds. The deeper karst zone correlates with deep bedrock. Water moves through rock via an integrated system of conduits following solutionally enlarged joints and bedding plane fractures. Rainwater percolates slowly downward from the surface to the successively deeper horizons, but may also flow laterally and independently within each horizon.

The Units 3 and 4 geological, geotechnical and geophysical exploration program was specifically developed to characterize the potential karst hazard, and incorporates a suite of different methods that proved successful in defining karst conditions for the Redstone Arsenal (References 415 and 416) and other sites in northeast Alabama. The results from the BLN investigation document that karst is not well developed in the Units 3 and 4 power block construction zone, and represents a low potential hazard. Specific features related to karst are documented and evaluated below. They include small cavities in rock boreholes, soft zones and cavities in soil, and an irregular bedrock surface beneath the soil mantle.

2.5.4.1.3.1 Cavities in Boreholes

Small cavities are commonly encountered in boreholes at the BLN site. At least one cavity, or void, was encountered in an average of 32 percent of exploratory boreholes drilled over the past thirty years at the BLN site. Table 2.5-225 presents the "hit rate" or number of boreholes that encountered at least one cavity, grouped by specific locations within the BLN site.

Most cavities were small and clustered near the top of rock. The 2006 BLN exploration program encountered a lower-than-average percentage of boreholes with cavities. Inside the Units 3 and 4 power block construction zone 21 percent of borings hit cavities, and outside the zone 19 percent of borings hit cavities (Table 2.5-225).

Table 2.5-226 lists and describes each cavity encountered in the 2006 BLN exploration program. Of the 57 cavities listed in the table, all but nine are described as clay-filled or soil-filled. Twenty-four are associated with a loss of water circulation. Most of the cavities encountered are small, 0.1 to 0.5 ft. in height, and clustered near the top-of-rock, 62 percent within 10 ft. and 84 percent within 20 ft. of top-of-rock (Figure 2.5-305). The largest cavity encountered within the construction zone is 4-ft. thick, located from 12.6 to 16.6-ft. below ground surface (bgs) in boring B-1076. The largest cavity, encountered outside the construction zone, is 8 ft.-thick, located from 20 to 28-ft. bgs in boring B-1072 adjacent to the south cooling tower (Figure 2.5-305). Appendix 2BB presents geotechnical boring logs.

Cavities in explored locations are clustered within 10 to 20 ft. of the top of rock, and decrease in frequency with depth. The distribution of deeper cavities may controlled in part by lithologic contacts within the bedrock. Figure 2.5-306 plots cavity occurrences in boreholes within the Units 3 and 4 power block construction zone, projected onto a cross section with lithologic unit contacts. This figure clearly shows the clustering of cavities near the top-of-rock and the lack of cavities beneath an elevation of 550 ft. (elevations are based on NAVD 88) (Foundation grade for Units 3 and 4 is 588.6 ft.). The more shallow cavities do not appear to be associated with any particular lithologic unit or contact. These are associated with the weathered or epikarst zone, close to the bedrock-soil interface. However, the deeper cavities may exhibit some alignment with bedding. Four cavities appear to be aligned along the contact between lithologic units A and B, and two cavities are located along the contact between units C and D. However, the dataset is small enough that these observations may not be significant.

At the Units 1 and 2 power block location, explored in the 1970s, 32 percent of borings encountered cavities (Table 2.5-225). Most cavities occurred in the upper ten feet of rock, and were removed during excavation. Photographs of the excavation (Figures 2.5-307 and 2.5-308) show competent rock without significant cavities at excavation grade. By contrast, the excavation for the intake structure, located approximately 3500-ft. east of Bellefonte Units 1 and 2 in a different rock formation (the Sequatchie Formation) and in a lower topographic position (within a gap in River Ridge), encountered a large cavity. Boreholes near the intake structure encountered cavities more frequently than any explored location (Table 2.5-225).

At the southern site, nine of 17 borings, or 53 percent, encountered cavities, (Table 2.5-225). Again, most were concentrated near top-of-rock, but occurred down to 94-ft. bgs. The higher percentage of boreholes with cavities at the southern site is attributed to its position in a topographic low, along the trend of a major lineament, and in line with a gap in River Ridge.

In summary, cavities encountered in exploratory boreholes at the Units 3 and 4 power block construction zone are relatively small and infrequent, occurring in an average of 21 percent of boreholes and are clustered near the top-of-rock. Of the three locations within the BLN site where exploratory boreholes fall within the middle Stones River Group, the Units 3 and 4 power block construction zone has the fewest borings that encountered cavities.

2.5.4.1.3.2 Cavities and Soft Zones within the Soil

Cavities and soft zones within the soil are common within the overburden residual soils above bedrock depressions or slots where soil has been piped down into the lower bedrock conduits via enlarged vertical fissures (Figure 2.5-304). Sowers (Reference 417) describes these zones in karst areas and notes that SPT values will decrease to a minimum, or to zero if a cavity is encountered, immediately above the bedrock within a slot. Two such features have been documented within the Units 3 and 4 power block construction zone.

The best documented feature is located at boring B-1051 at the southwest edge of the unit 4 power block (Figure 2.5-309). Boring B-1051 lies along lineament #4, a linear drainage oriented parallel to strike of the bedding. (See discussion of lineaments in Subsection 2.5.3.2.2) Boring B-1051 encountered stiff clays which began to soften at 27 ft. At 31 ft. the drill rods dropped to a depth of 36 ft. through a cavity, and from there to 43 ft. the drill penetrated soft, wet mud without reaching solid bedrock. (Boring logs are presented in Appendix 2BB; monitoring well logs are presented in Subsection 2.4.12.)

Additional borings drilled within 15 ft. of B-1051, encountered widely varying depths to bedrock, suggesting a complex bedrock-soil interface. Boring B-1051A encountered bedrock at 21 ft. then cored rock to 52 ft. Monitoring well MW-1213A encountered wet mud at 30 ft. and drilled through the mud to 44 ft. without hitting rock. The well completion log indicates that based on the consistency of the “soil” there may be a spring in the bottom of this feature. Well MW-1213B encountered rock at 20 ft., broke out into a cavity at 44 ft., and then drilled to 50 ft. through wet mud without encountering rock again. Well C encountered rock at 30 ft. and cored to 50 ft., ending in rock.

Boring B-1052 was drilled at an angle across the lineament toward B-1051 and appears to have encountered portions of the deeper bedrock drainage system. The boring encountered bedrock at 35 ft., and then encountered two cavities at depth, one at 45 ft. that was 1.1-ft. thick and a second one at 60-ft. depth that was 1.6-ft. thick.

Additional exploration of the B-1051 area helped define the geometry of the feature (Figure 2.5-309). Microgravity data show a depression in the bedrock surface 50 to 100-ft. wide. Seismic refraction data do not image this depression, perhaps because no seismic line crosses directly over it. Cone penetrometer (CPT) transects across the area show moderately deep soil, 15 to 18-ft. deep, that increased in thickness to 30 ft. suddenly near the center of the depression.

A second soft soil zone was encountered in a single borehole located in the north corner of the Units 3 and 4 power block construction zone. One of the 34 borings (SS-28) drilled for a 1987 soils investigation penetrated 30 ft. before encountering rock (Figure 2.5-310). SPT values dropped in the bottom of the hole to 3 and 0. Groundwater rose in the hole to 12 ft. from the surface. The other 33 borings encountered top of rock at about 10-ft. depth and were dry.

2.5.4.1.3.3 Irregular Bedrock Surface

Borehole data and seismic refraction data both indicate an irregular bedrock surface within the Units 3 and 4 power block construction zone. Borings encountered the top-of-rock variably at depths from 5 ft. to more than 43 ft. This irregular surface represents a dissolution weathering front, created by the slow downward movement of erosive water through the soil and rock. Depressions in the bedrock surface are places where groundwater is concentrated and dissolution has been most active, usually where joints or bedding planes allow water to drain downward.

Figure 2.5-310 presents two contour maps of the top-of-rock showing the shape of the bedrock surface using a 5-ft. contour interval. The left-hand map is based on auger and SPT refusal depths in boreholes. Included in the dataset are borings from the 2006 Units 3 and 4 exploration program and from the 1987 TVA study. The right-hand map is a 3D model of the seismic reflection data, and shows contours on the top of the 6,000 feet per second [fps] Vp layer, the layer found to most closely correlate with auger refusal. Approximate contacts between lithologic units at the bedrock surface are shown for reference.

The two maps on Figure 2.5-310 are similar in overall appearance, yet provide different detail. Both maps show a general slope of the bedrock surface northwest toward Town Creek, reflecting the slope of the overlying ground surface. Both also show low areas beneath the two power blocks, coincident with northeast-trending swales that were present in the original topography (Figure 2.5-296).

The contour map from borehole data shows two significant depressions in the bedrock surface not shown on the 3D seismic model, presumably because seismic lines did not cross them. Both these depressions were discussed above, and are areas where boreholes encountered very deep soils that become soft and wet near the bedrock interface. The depression at the southwest corner of the Unit 4 power block is centered on boring B-1051, and the depression at the north corner of the construction zone is centered on boring SS-28 of the TVA study.

Seismic profiles across the construction zone (Figure 2.5-311) show an undulating bedrock surface with some areas of smooth bedrock and other areas of pinnacles and depressions. The elevation of top-of-rock from auger refusal correlates well with the elevation of the 6,000 fps Vp layer, the green/blue contact, on the seismic 2D profiles. Warmer colors indicate progressively less weathered bedrock with depth.

Seismic refraction 3D models with detailed (2-ft.) contours show in greater detail the shape of the bedrock surface over the Units 3 and 4 power block construction zone. These models, presented as part of the subsection on geophysical surveys (Figures 2.5-312 and 2.5-313) depict the surfaces of the 6,000 fps Vp layer and the deeper 14,000 fps Vp layer. Both show a step in the bedrock surface coincident with the contact between lithologic units B and C. Both also show a complex pattern of high points and depressions in the bedrock surface. Altogether, the seismic reflection data, both 2D and 3D, suggest that the top-of-rock is more complex than the borehole data alone suggest, and that the depressions at B-1051 and SS-28 may not be unique.

Seismic reflection 3D-modeling also provides a “first look” at excavation conditions. For the Unit 4 nuclear island, both the top of the 6,000 fps Vp layer and the 14,000 fps Vp layer are above excavation grade of 589 ft. With the exception of any deep cavities or slots not imaged by the method, the Unit 4 nuclear island excavation extends below most cavities and the weathered rock.

Unit 3, however, is located downslope from Unit 4, thus the ground surface and underlying bedrock surface occur at lower elevations. Seismic refraction 3D modeling shows areas beneath the Unit 3 nuclear island where the top of the unweathered rock, or the 14,000 fps Vp layer, is below excavation grade of 589 ft. (A Vp of 14,000 fps corresponds to a Vs of 8000 fps, given a Poissons ratio of 0.26.) Sufficient excavation beneath Unit 3 removes weathered rock and establishes the foundation on hard rock (Figures 2.5-314, 2.5-312 and 2.5-313).

Seismic refraction surveys at the southern site identify two zones of relatively deeper weathering, termed the Eastern and Western Anomaly Zones, with the Eastern Anomaly Zone exhibiting the deepest and most extensive weathering. The Eastern Anomaly Zone contains two roughly strike-parallel troughs of deep weathering; one of these is 150 ft. in width and 90 ft. deep. Clay-filled cavities occur at depth within the rock in this zone.

The relatively deep and extensive weathering observed at the southern site may relate to its topographic position. The Eastern Anomaly Zone of the southern site lies on strike with and within the same rock units as the Unit 3 reactor area at the BLN site. However, the southern site lies along a major topographic lineament (Lineament #2 on lineament map, Figure 2.5-291) that extends from Town Creek through the gap in the ridge to the river. The lineament is a gentle trough in the topography through the lowlands, which then becomes a steep-walled gap as it passes through the ridge. This lineament may represent the eroded valley of a former Town Creek. If so, the southern site would lie along a former creek valley, and deeper weathering and karst dissolutioning might be expected.

Located at a higher topographic position and away from any gaps in the ridge, the Bellefonte Units 1 and 2 site was found during excavation mapping to exhibit relatively minor irregular weathering (Reference 201). During excavation "most of the rock containing cavities was … removed, leaving only isolated cavities at depth," (Reference 201). Cavities encountered at the base of the excavation were small and were grouted.

The Bellefonte Units 1 and 2 FSAR (Reference 201) describes the conditions encountered:

"Foundation rock was generally excellent at final grade….Weathering in the form of cavities was found in the excavated areas, and rarely did this type of weathering penetrate into the foundation elevations of the main plant. Most of the natural joints located in the main plant foundation were tight or opened by blasting and were not weathered."

The Bellefonte Units 1 and 2 FSAR (Reference 201) goes on to describe the treatment of cavities:

"All cavities were cleaned down to the minimum depth of two times the width. If the width increased with depth, the cavity was cleaned downward until a wedging effect was achieved. After the cleaning procedure was completed, the cavities were backfilled with concrete to the top of the surrounding rock."

"A grouting program was adopted in the powerhouse area, to treat slipped bedding planes, blast cracks, and the few cracks which develop along natural joints during blasting."

The Bellefonte Units 1 and 2 FSAR then describes the insertion of pipes into cracks and pressure grouting to a maximum of 5 psi, to seal these cracks.

Conditions at the excavation for reactor Units 3 and 4 are expected to be similar to those encountered at the excavation for reactor Units 1 and 2. The Bellefonte Units 1 and 2 power block area lies within the same lithologic units found in the Units 3 and 4 power block construction zone and southern sites, and is positioned between them, but is more similar to the northern site in its higher topographic position and its location some distance from both major gaps in River Ridge.

2.5.4.1.3.4 Conclusions Regarding Karst

Extensive review of existing data, historical construction photographs, and subsurface exploration data at the Unit 3 and 4 power block construction zone document minor karst features that are not expected to affect the stability of foundations at the Units 3 and 4 safety-related structures. Most rock containing cavities occurs within 10 to 20 ft. of the top-of-rock and is removed during excavation. Based on experience at the Bellefonte Units 1 and 2 excavation, rock conditions are excellent and minor cavities are remediated.

The Unit 3 power block is located downslope from Unit 4, thus the ground surface and underlying bedrock surface (top-of-rock) occur at lower elevations. Seismic refraction 3D modeling shows areas beneath Unit 3 where the top of the unweathered rock, or the 14,000 fps Vp layer, is below foundation grade of 588.6 ft. Sufficient excavation beneath Unit 3 foundation (elevation 588.6 ft.) removes weathered rock and establishes the foundation on hard rock.

An irregular bedrock surface exists beneath the residual soil mantle within the Units 3 and 4 power block construction zone. This surface represents a dissolution weathering front, created by the slow movement of erosive water through the soil and rock. Depressions in the bedrock surface exist where dissolution has been most active, usually where joints or bedding planes allow water to drain downward. Such depressions are visible in the seismic refraction profiles and are recorded in borehole logs. However, these exploration methods require confirmation with geologic mapping and exploratory probing during construction to detail the full depth and configuration of dissolution depressions and cavities. Remediation of Karst features will follow methods as described in Subsection 2.5.4.12, Techniques to Improve Subsurface Conditions.

2.5.4.1.4 Effects of Human Activities

Human activities such as mining, hydrocarbon extraction, and groundwater withdrawal have the potential to cause surface deformation. Underground mining tunnels can collapse; withdrawal of groundwater or hydrocarbons can result in subsidence of the ground surface. In some cases, lowering of the groundwater table may trigger sinkhole collapse.

Open-pit mines and quarries are present in Jackson County, extracting limestone, chert, sand, and gravel, primarily for the construction industry. A map of mineral resources of Jackson County is presented in Figure 2.5-315 (Reference 327). There have been no mining activities, either surface or subsurface, at the BLN site. The nearest quarry is a chert pit, located approximately 2 mi. from the center of the power block construction zone.

Hydrocarbon extraction is not conducted in northeastern Alabama. Major oil and gas and coalbed gas fields are located in western and southwestern Alabama. Figure 2.5-316 (Reference 419) shows the major fields in Alabama; the closest to the site is in the Black Warrior Basin.

Significant groundwater withdrawal is not occurring at the BLN site. Regardless, the hard limestone rock beneath the BLN site is not susceptible to intergranular subsidence.

In conclusion, there are no human activities at the site that could cause subsidence or collapse of the ground surface within the Units 3 and 4 power block construction zone.

2.5.4.1.5 Non-tectonic Surface Deformation

The BLN investigation did not encounter adverse geologic conditions in the Units 3 and 4 safety-related foundation explorations that pose a stability or safety hazard. Major safety-related structures are founded on fresh, hard bedrock, or on fill concrete placed over fresh, hard bedrock.

Dissolution and karst development is an active process in the Bellefonte area. Although no adverse conditions were identified in the Units 3 and 4 safety-related foundation investigations that pose a stability or safety hazard, minor dissolution and karst features are present, primarily in the upper 10 to 20 ft. of bedrock. This minor karst development is mitigated during construction, as needed, using standard foundation preparation techniques such as dental work, local grouting, and overexcavation, similar to the approaches used for Bellefonte Units 1 and 2 (Reference 201).

No significant zones of alteration or structural weakness are present. Bedrock contains bedding planes, joints, and minor shear planes, which do not significantly reduce rock strength.

There are no significant unrelieved stresses in the bedrock that could cause creep or rebound. Folding is gentle and there is no past glacial loading. Erosion rates are slow. These conditions are not conducive to “locked in” residual stresses.

Other than limestone which weathers primarily by dissolution, there are no rocks or soils that might be unstable because of their mineralogy, lack of consolidation, water content, or potentially undesirable response to seismic or other events.

There are no human activities, such as mining, hydrocarbon extraction, or groundwater withdrawal, which could cause subsidence or collapse.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 Introduction

BLN COL 2.5-6 This subsection presents a summary of the field investigation and subsurface material properties at the BLN site. The laboratory testing and sample control procedures are discussed as well. Refer to Subsection 2.5.4.3 for plot plans showing the boring and other field investigation locations and for sections of the subsurface conditions. Soil dynamic material properties are presented in Subsection 2.5.4.7.

The procedures used to perform field investigations for determining the engineering properties of soil and rock materials conform to Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants." Specifically, the following items discussed in Regulatory Guide 1.132 have been addressed:

1. The scope of the exploration program, including borings, test pits, and geophysical measurement locations, was planned using the guidelines presented in Appendix A of Regulatory Guide 1.132 and provided coverage in the power block areas, including the nuclear island and the adjacent non-safety related structures. The information obtained from the exploration program characterizes the subsurface conditions in the power block areas, and allows for the construction of detailed cross-sections through the areas of the nuclear island and adjacent structures, as discussed and illustrated in Subsection 2.5.4.3.

2. The field exploration program outside of the power block area confirms the subsurface conditions and geology as being consistent across the site as discussed in Subsection 2.5.4.1.2.

3. The field operations were conducted under the provisions of approved quality assurance plans and procedures. Field operations were conducted by experienced and qualified personnel. The boreholes were grouted upon completion, monitoring wells were capped, and boring locations were located by surveying methods following completion. The boring records included in Appendix 2BB contain coordinates, elevations and completion notes.

4. The field exploration and sampling methods were conducted in accordance with established procedures considering applicable industry standards.

5. Geophysical testing, consisting of seismic refraction, shear wave velocity measurements, suspension and down-hole logging, microgravity surveys, and natural gamma borehole surveys, was

performed. Refer to Subsection 2.5.4.4 for details of these investigations.

6. Detailed boring logs were prepared, and are presented in Appendix 2BB. Detailed logs of test pits were prepared and are presented in Appendix 2CC.

7. Groundwater investigations were conducted by observing water levels in borings, installing monitoring wells to different depths and measuring water levels periodically for a period of time after completion of the borings, and by performing an aquifer pump test. Refer to Subsections 2.4.12 and 2.5.4.6 for details.

8. Sample storage and retention was performed in accordance with appropriate quality procedures.

9. The soil samples and rock cores were photographed in the field prior to transporting them to the on-site storage area for further disposition.

The procedures used to perform laboratory investigations of soils and rocks conform to Regulatory Guide 1.138, “Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants”. Specifically, the following items discussed in Regulatory Guide 1.138 have been addressed:

1. The laboratories met the guidelines for space configuration, establishing controlled access testing areas, and adequate ventilation.

2. The facilities used calibrated equipment, traceable to a recognized standard, as described in EM 1110-2-1909 (Reference 420), which is referenced in Appendix A of Regulatory Guide 1.138. Lists of equipment used for testing were included with individual data packages.

3. Approved sample handling and storage protocol was followed prior to testing; chains of custody were used for sample shipment as discussed in Subsections 2.5.4.2.2 and 2.5.4.2.3.2.1.

4. Samples to be tested were initially identified based on a visual description in accordance with ASTM D2488 (Reference 421), and were selected to be representative of various soil and rock types found across the site. Damaged or otherwise inadequate samples were deleted from the testing program and replaced with a suitable sample chosen to represent the same area, soil type, strata as the original sample.

5. Remolded bulk samples were tested to represent the anticipated range of field moisture conditions.

6. Classification tests were performed on 91 samples to define the various soil types present across the site.

7. Both static and dynamic laboratory testing were performed in accordance with standard test procedures using calibrated equipment. In general, no deviations from standard test procedures were made; minor deviations, if any, are noted on individual test reports. The procedures used are discussed in Subsections 2.5.4.2.1 (static) and 2.5.4.7 (dynamic).

8. Rock samples for testing were selected to represent various rock strata and variations in composition.

The site exploration and field testing program is summarized in Tables 2.5-227, 2.5-228 and 2.5-229.

2.5.4.2.2 Soil Investigation: Scope and Methodology

The field exploration of in situ soils consisted of:

- Hollow-stem auger borings
- CME stabilized continuous sample tube boring
- Cone penetrometer test (CPT)
- Test pits
- Borings for undisturbed sampling

The hollow-stem auger borings with standard penetration testing were made using methods specified in ASTM D1586 (Reference 422). The purpose of these borings was to obtain disturbed soil samples for laboratory testing and to determine the standard penetration resistance, N, of the in-situ soils. The standard penetration testing was performed at 2.5-ft. intervals to top of rock. Disturbed samples from the split-spoon borings were sealed in glass jars after removal from the soil sampler and taken to MACTEC laboratories in Raleigh, North Carolina or Atlanta, Georgia for testing. Chemical tests were performed by Severn Trent Laboratories–St. Louis under subcontract to MACTEC.

Undisturbed soil sample borings were made to determine in situ soil conditions and obtain samples for laboratory testing. The soil samples were obtained in 3-inch diameter Shelby tubes using a fixed piston sampler following procedures of ASTM D1587 (Reference 423). The ends of the tubes were sealed immediately after removal from the boring to preserve the natural moisture content of the samples. Generally the undisturbed borings were located 5 ft. from selected split-spoon borings.

Plate load testing was not conducted because such testing is more appropriate for granular soils than for the clays present as the soil layer. Additionally, the nuclear island structures bear on rock, and plate load testing is not feasible for rock.

Selection of specimens for laboratory testing was such that it included samples from the full range of soil depths and the soil types encountered. The following laboratory tests were conducted in accordance with the applicable ASTM or other listed standard methods:

- Moisture content, ASTM D2216 (Reference 424)
- Atterberg limits, ASTM D4318 (Reference 425)
- Sieve and hydrometer analysis, ASTM D422 (Reference 426)
- Amount Finer than No. 200 (75mm), ASTM D1140 (Reference 427)
- Specific Gravity, ASTM D854 (Reference 424)
- Moisture-unit weight relationship, ASTM D698 (Reference 428) and ASTM D1557 (Reference 429)
- pH – EPA SW-846 9045d (Reference 430)
- Chloride and Sulfate – EPA-MCAWW 300.0A (Reference 431)

Laboratory testing parameters utilized in undisturbed soil testing were selected to duplicate or bracket anticipated field conditions. The following laboratory tests were conducted on undisturbed samples.

- Consolidated-undrained (R) triaxial compression tests (ASTM D4767) (Reference 432) with pore pressure measurements. (Representative of soil conditions after pore pressure equalization under the design loads).
- Unconfined compression tests (ASTM D2166) (Reference 433) at natural moisture content and density.
- Load-controlled consolidation tests (ASTM D2435) (Reference 434) using an oedometer

Relative density tests were not conducted because these tests are applicable only for granular soils and granular soils are not present in a significant quantity at the BLN site.

The soils encountered in the field investigation showed limited variability in composition and layering. Laboratory soil tests were performed on selected representative disturbed samples to determine the variation in engineering properties. These soil samples were obtained from 41 boring locations. These investigations reveal the presence of bedrock overlain by a relatively thin layer of residual soils (derived from weathering of the underlying rock) with some amount of fill. The thickness of this residual soil mantle varies from 4.9 ft. to 43.0 ft. across the site and averages a thickness of 15.3 ft. Of the 41 borings, only two borings had notably thicker overlying soil strata [B-1046 with 43.0 ft. and B-1051 with 23.6 ft.]. The average depth of soil excluding these borings was 14.3 ft.

2.5.4.2.2.1 Index Properties of In Situ Soils

The results of laboratory testing for index, classification and chemical corrosivity parameters are summarized in Table 2.5-230. A statistical overview of these findings is shown in Table 2.5-231.

Most of the soils sampled are fine-grained. Only three of the 88 soils classified were identified as coarse-grained soils according to the Unified Soil Classification System (USCS) (Reference 435). Plots of the Atterberg limits for the fine-grained soils tested are shown in Figure 2.5-317. Most soils plot above the A-Line and are classified as lean clays (CL) or fat clays (CH) in the USCS. Several samples plot below the A-line and are classified as elastic silts (MH). Most of the soils lie close to a line approximately parallel to the A-line. This is characteristic of a group of samples having the same geological origin. The majority of the soils are highly plastic clays (liquid limits are greater than 50) with several samples having both a high liquid limit (greater than 90 percent) and a high plasticity index (greater than 60 percent).

The fine-grained soils show a high degree of scatter in the natural moisture content, wn, with values ranging from 3.8 to 50.3 percent (Table 2.5-231 and Figure 2.5-318). With the exception of four samples, the natural water contents are below 34 percent with the mean value being 20.7 percent. In Figure 2.5-318, the minimum value of wn increases with increasing depth and forms a near linear lower boundary for wn. Data from the deeper soil borings (B-1046 and B1051) are consistent with this boundary. The average natural water content (20.7 percent) is lower than the average plastic limit of 25 percent. Because the plastic limit varies over a large range (15 percent - 35 percent), the natural water contents were scaled with respect to their Atterberg limits using the Liquidity Index. More than 60 percent of the samples have negative liquidity indices, indicating that these soils are dry of the plastic range (Figure 2.5-318).

Although the overlying soil layer is predominantly fine-grained, it contains variable amounts of sand and gravel (Table 2.5-231). Figure 2.5-319 shows the sand, gravel and coarse-grained fraction (sand and gravel) of the overlying soil. The percentage of coarse-grained material ranges from 1.6 to 49 percent with an average value of 19.4 percent. Significant scatter exists in both the amount of coarse-grained material and its distribution with depth. The content of coarse-grained material is not expected to influence the engineering behavior significantly. However, it may influence the capillarity of the soil. The specific gravity of the soils tested range from 2.59 to 2.98 with an average value of 2.76 (Table 2.5-231).

Swelling potential was evaluated for 18 clay samples for which hydrometer data was available. The activity of these clays varied from 0.5 to 1.4 with an average value of 0.9 (Table 2.5-231). The identification and classification of expansive soils as defined in Reference 436 is shown in Figure 2.5-320 with the data from the 18 clay samples. With the exception of two samples (B-1003 and B-1046), tested clays have high to very high volume change behavior.

The soils evaluated in the BLN study are similar in composition and properties with the soils reported in a previous study of soils near the BLN Units 3 and 4 area; soil constitutes the top 13 ft. of the profile and is composed of lean to fat clays with medium to very stiff consistency and groundwater at or below the elevation of the bedrock. The mean values for the soil properties reported in the previous study are:

- 25.5 percent natural water content
- 61 percent liquid limit
- 36 percent plasticity index
- 58 percent clay content (< 0.005 mm)
- 1 percent gravel
- 16 percent sand.

These mean values and the related ranges are consistent with those reported in the present investigation.

2.5.4.2.2.2 Moisture Unit Weight Relationship of In Situ Soil

Moisture-unit weight tests were run on 5 soil samples from three boring locations (B-1025, B-1055 and B-1088) using both standard and modified compactive effort to evaluate the difference in using these two efforts. The results of these tests are summarized in Table 2.5-232. The soil collected from a depth of 1–5 ft. at boring locations B-1025 and B-1055 consists of medium plasticity clays (CL according to USCS). The other tested samples were found to be high plasticity clays (CH). The optimum moisture contents were close to the natural moisture content of the soil (within ±2.6 percent). The maximum dry unit weights ranged from 99.5 pcf to 123.0 pcf. The plasticity indices of these soils are greater than 25 percent and the liquid limits are close to or in excess of 50 percent. It is likely that these soils have a high potential for volume change.

2.5.4.2.2.3 Shear Strength of In Situ Soil

Consolidated-undrained triaxial tests were conducted on low plasticity clay (CL) samples from depths of 4 to 6 ft. in boring B-1010 and on high plasticity clay (CH) samples from depths of 12 to 14 ft. in boring B-1017UDS. Samples were saturated using backpressure, and B values of 0.95 –0.98 were obtained. Shearing was conducted undrained at a rate of loading of 0.1 percent /min. Vertical load, vertical displacement, chamber pressure and pore pressures were recorded electronically during the loading phase. Tests were terminated at 17 percent strain.

Results of the consolidated-undrained triaxial tests are shown in Table 2.5-233. The initial saturation of the samples ranged from 78.6 to 95.5 percent. The average initial dry unit weight, average initial moisture content, average initial void ratio and total and effective strength parameters for the samples tested are shown in Table 2.5-233. Both CL and CH soils showed strength increase throughout the test. Most specimens showed an initial high modulus that decreased sharply at about 2 to 5 percent strain.

Pore pressures increased sharply during the initial stage of shearing (0 – 3 percent strain), then decreased steadily. The final pore pressures were negative for CL specimens tested under confinement stresses of 48.3 kPa and 96.5 kPa (7 and 14 psi) and for a CH specimen tested under a confinement stress of 48.3 kPa (7 psi). This is typical of overconsolidated soils. The low plasticity clay has an effective internal friction angle of 32.7° and zero cohesion intercept. The high plasticity clay has an effective internal friction angle of 26.8° with a cohesion intercept of 11.3 kPa (235 psf). The three samples of the low plasticity clay failed along well defined shear planes. The three high plasticity clay specimens tested showed some barreling and shear localization at failure.

Results of the unconfined compressive strength tests are shown in Table 2.5-234. The natural water content of both specimens is just within the plastic range and brittle type failure is observed. The unconfined compressive strength of the high plasticity clay was found to be 140.2 kPa (2929 psf) and 170.0 kPa (3551 psf) for the two samples tested.

The above findings were compared with shear strength tests reported in the Bellefonte Units 1 and 2 FSAR (Reference 201). The soils tested in both studies appear to be similar in composition and index properties. However, the reported strength parameters of the 1987 study show wider variation in internal friction angle for the same soil type and include some values considered uncharacteristically low.

2.5.4.2.2.4 Consolidation Properties of In Situ Soils

BLN COL 2.5-5
BLN COL 2.5-6

Oedometer consolidation tests were conducted with automated electronic recording of vertical displacements. A summary of consolidation tests is given in Table 2.5-235. Four of the five samples tested classify as high plasticity clays and one (B-1010-5UDS) as a clayey gravel (GC). Their liquid limits vary from 72 to 88 percent and have plasticity indices that range from 49 to 51 percent. The measured specific gravities range from 2.71 to 2.98. The samples were collected from depths ranging from 4 to 10 ft. The natural moisture contents are very close to the plastic limit and the soils are not fully saturated (saturation ranges from 81.9 to 99.1 percent). The soils are overconsolidated with overconsolidation ratios ranging from 5 to 9.3.

The coefficient of consolidation corresponding to each load increment is shown in Figure 2.5-321 for the samples tested. The coefficient of consolidation ranges between 0.22 to 118.4 m^2/day (0.02 to 11 ft^2/day). Sample B-1038 showed swelling at a vertical load of 95.8 kPa (2000 psf).

2.5.4.2.2.5 Design Parameters

Selection of design parameters was based on considerations of existing field conditions, various design loading conditions and the material properties. The selected design values are shown below.

Dry unit weight: CL: 16.3 kN/m^3 (104 pcf)

CH: 15.4 kN/m^3 (98 pcf) (average of seven samples)

GC: 15.9 kN/m^3 (101 pcf)

Total unit weight: CL: 19.6 kN/m^3 (125 pcf)

CH: 19.3 kN/m^3 (123 pcf) (average of seven samples)

GC: 19.8 kN/m^3 (126 pcf)

Specific Gravity: CL: 2.77 (average of two samples)

CH: 2.74 (average of nine samples)

GC: 2.98

Consolidated Undrained Strength – Effective Stress:

CL: ϕ '= 32°, c' = 0

CH: ϕ '= 26°, c' = 11.0 kPa (230 psf)

Consolidated Undrained Strength – Total Stress:

CL: = 11°, c = 81.4 kPa (1700 psf)

CH: = 0°, c = 71.8 kPa (1500 psf)

Unconfined Compressive Strength:

CH: qu=138.9 kPa (2900 psf) (lower value of two tests)

Consolidation:

CH: Cc = 0.23 (average of four tests)

Cr = 0.016 (average of four tests)

Pc = ranges from 5600 to 7500 psf

OCR = ranges from 5 to 9.3

Because the foundation for the safety-related structures (nuclear island) will bear on rock, the data developed from the field and laboratory soil testing programs was not used for the bearing capacity or settlement analysis of those structures. However, the data were used for the following purposes:

1. to evaluate the suitability of the soils to be excavated from the nuclear island area for use as structural fill and/or backfill behind below grade walls (see Subsection 2.5.4.5 for a detailed discussion).

2. to analyze the stability of temporary slopes around the nuclear island during construction (see Subsection 2.5.4.5.2.1 for a detailed discussion).

The test data are enveloped by the design in that the laboratory test results were needed to adequately characterize the on-site materials for use as structural fill and backfill behind the below grade walls of the nuclear island. Since the test results indicated that the majority of the on-site soils were high plasticity clays and silts that fell into the Class III and IV categories as defined in a previous study (Reference 201), it was determined that they were unsuitable for use for either application.

The slope stability analysis required the use of the field data in order to characterize the soils behind the temporary slope area, and to determine the groundwater level in the slope areas. The laboratory test data was needed to determine the strength characteristics of the soils that will comprise the slope. The slope stability analysis, including a discussion of the groundwater levels and strength characteristics used in the analysis, can be found in Subsection 2.5.4.5.2.1.

In determining the soil parameters to use in the analysis, the results from the current study were compared to the results from previous TVA studies (see Reference 201). The majority of the results from the three studies are reasonably consistent, and the minor variations were noted and discussed in the preceding subsections as appropriate. The design values presented above do not represent the high or low end of the range of values from any of the three studies individually, or any collective range of values. Rather, they are average values determined by viewing each individual set of data and choosing values that represent the soils anticipated to be present in the temporary slope area.

2.5.4.2.3 Rock Investigation: Scope and Methodology

BLN COL 2.5-6 The field investigation of in situ rock consisted of:

- Rock core borings of NQ and HQ size
- Goodman Jack testing
- Geophysical logging
- Seismic refraction data collection
- Microgravity data collection
- Packer pressure testing

Most of the soil borings were extended into rock using double-tube core barrels and rock coring techniques described in ASTM D2113 (Reference 437). The recovered core was carefully slid out of the core barrel into a supporting half-round tube, measured to determine recovery and Rock Quality Designation (RQD), then placed into labeled wooden core boxes. The field geologists described the rock core, prepared field descriptive logs and photographed the core.

Goodman Jack testing was performed in four boreholes at multiple depths to measure elastic modulus in situ using procedures of ASTM D4971 (Reference 438).

Geophysical logging was performed in 26 boreholes. The logging performed was PS Suspension logging, Borehole Televiewer and Natural Gamma. See Subsections 2.5.4.4.2 and 2.5.4.4.4 for discussion of procedures and results. Seismic refraction data was collected along 40 arrays as discussed in Subsection 2.5.4.4.1. Microgravity data was collected along 11 transects as discussed in Subsection 2.5.4.4.3.

Laboratory testing was performed on core samples of rock selected from 24 of the borings drilled during the subsurface investigation. Selection of the samples tested and the tests performed on the samples was done by MACTEC engineers with input from the William Lettis and Associates (WLA) Project Principal. Unconfined compressive strength testing was performed on 61 samples, and stress strain measurements were included for 21 of the samples tested.

The rock cores were controlled, prepared and tested in accordance with the following ASTM methods:

- Sample Preparation – Rock core (ASTM D4543) (Reference 439)
- Compressive Strength and Elastic Moduli (ASTM D7012) (Reference 440)
- Preserving and Transporting Rock Samples (ASTM D5079) (Reference 441)

Except for sample preparation, the tests were performed at MACTEC laboratories in Charlotte, North Carolina. Sample Preparation was performed by Haley and Aldrich Inc. under subcontract to MACTEC.

2.5.4.2.3.1 Laboratory Testing

2.5.4.2.3.1.1 Specimen Preparation – Rock Cores

Samples were prepared for testing using the methods of ASTM D4543 (Reference 439). This procedure specifies the methods for laboratory specimen preparation and determination of the length and diameter of rock core specimens and the conformance of the dimensions with established standards. The prepared cores were measured to determine the straightness of elements on the cylindrical surface, flatness of the specimen ends, and perpendicularity of end surfaces to the specimen axis.

2.5.4.2.3.1.2 Compressive Strength and Elastic Moduli – Rock Cores

The testing was done in accordance with ASTM D7012 (Reference 440). Under this method, the prepared specimen is placed in a loading frame and axial load is increased continuously on the specimen until peak load or failure of the specimen is obtained. To determine the elastic modulus, the specimen is instrumented with four strain gages (two mounted axially, two mounted laterally) prior to placement in the loading frame. Axial strain gages were 2 inches in length and lateral strain gages were 1 inch in length. Axial load and deformation (axial and lateral) readings are obtained as the load is applied to the specimen. Unconfined compressive strength is determined based on the cross-sectional area and the maximum recorded load applied to the specimen. The results are corrected for length to diameter (L/D) ratios differing from two units. Young's modulus (the slope of the stress-axial strain curve) and Poisson's ratio (ratio of lateral strain to axial strain) are calculated using the strain gage data from a portion of the data range generally between 40 and 60 percent of maximum stress. The specific data range for each core was individually selected based on visual review of the data. The selection utilized the average slope method over a range where both the axial and lateral stress-strain curves appeared most linear.

Two-inch axial strain gages were used for the cores. Two-inch gages were used to comply with the minimum axial strain gage length of 10 mineral grain diameters specified in ASTM D7012 (Reference 440).

2.5.4.2.3.2 Quality Assurance

2.5.4.2.3.2.1 Sample Control

Rock core samples were obtained from core borings made under the direct observation of a MACTEC or WLA rig geologist as part of the geotechnical exploration process. The rock core was carefully removed from the core barrel, photographed, measured and placed into wooden core boxes of appropriate size. Core boxes were labeled with identifying information, transferred to the lockable site temporary storage area, and inventoried into the sample inventory records.

The core boring field records were reviewed by MACTEC and WLA engineers and geologists, and sample intervals were identified for possible laboratory testing. Work instructions were issued listing samples to be removed from the site storage and shipped to the laboratories. In accordance with the work instructions, segments of the rock core samples were taken from the wooden core boxes, measured and placed into appropriate shipping containers with padding. The boring number, depth interval of the segment and the depth of the top of the segment was marked on the core using an indelible marker.

Samples were handled and transported or shipped to the appropriate laboratory location following handling methods in ASTM D5079 (Reference 441). The undisturbed samples were transported by MACTEC or WLA personnel in personal or company vehicles. Samples were shipped under chain-of-custody, and the receiving laboratory signed the chain-of-custody upon receipt. Samples were stored in the controlled laboratory environment in a secure location.

2.5.4.2.3.2.2 Testing Personnel and Equipment

Laboratory testing was conducted by personnel qualified under MACTEC's Quality Assurance Project Document using equipment calibrated in accordance with the MACTEC Quality Assurance Project Document.

2.5.4.2.3.3 In Situ Rock Mass Engineering Properties

The intact rock strength properties, determined as described in Subsection 2.5.4.2.3.1.2 and listed in the input portion of Table 2.5-236, were used in conjunction with field observations and evaluation of rock cores as described in the following subsections to determine the rock parameters used for static stability analysis.

In situ rock mass properties (shear strength, deformation modulus), for evaluation of the rock-embedded nuclear island basemat, are based on Hoek-Brown criterion (References 442 and 443) using the RocLab® program. In situ rock mass properties correspond to two bedrock types that are present at the foundation level and in the lower excavation walls for the nuclear island basemat; micritic limestone and argillaceous limestone (collectively referred to as “limestone”) of the Middle Stones River Group described in Subsection 2.5.4.1. The in situ limestone consists of generally blocky rock mass with a dominant and consistent bedding system.

The Hoek-Brown criterion is an empirically-based approach that develops non-linear shear strength envelopes for a rock mass, and accounts for:

- The strength-reducing influence of discontinuities (joints, bedding planes, faults)
- Mineralogy and cementation
- Rock origin (e.g., sedimentary or igneous)
- Level of induced disturbance from excavation/blasting
- Weathering

The Hoek-Brown method uses five input parameters to estimate rock mass strength:

- Unconfined Compressive Strength (UCS) of intact rock core samples.
- Material index (mi) related to rock mineralogy, cementation, and origin.
- Geological Strength Index (GSI) that factors the intensity and surface characteristics of rock mass discontinuities.
- Disturbance factor (D) related to the level of rock mass disturbance due to construction excavation and blasting.
- Young's laboratory modulus of intact rock core samples (Ei).

Rock mass field and laboratory data used as the basis for the GSI, Ei, and UCS were obtained by field and lab examination, borehole Optical Televiewer logs, and laboratory testing of rock core samples. Input parameters D and mi were estimated from the published empirical data/charts in Reference 442.

2.5.4.2.3.4 Selection of Inputs

2.5.4.2.3.4.1 Unconfined Compressive Strength (UCS)

Unconfined Compressive Strength (UCS) is based on laboratory testing of intact rock core samples (Subsection 2.5.4.2.3.2) obtained from exploratory borings made within the plant footprint and adjacent foundation excavation zone. The laboratory tests are segregated according to rock type: micritic limestone and argillaceous limestone. The UCS mean values, and one standard deviation ranges about the means, for each rock unit were used for input into the Hoek-Brown equations.

2.5.4.2.3.4.2 Geologic Strength Index (GSI)

Geotechnical borehole Rock Quality Designation measurements, UCS data, and borehole optical televiewer survey logs form the basis for developing Rock Mass Rating values for the in situ rock mass (References 442 and 443). These values are correlated with the micritic limestone and the argillaceous limestone. Values for Geologic Strength Index (GSI) were derived from the RMR classification values using the equation GSI = RMR-5 as recommended by Hoek (Reference 442). The derived GSI values are reported as mean and one standard deviation ranges about the mean for input into the Hoek-Brown equations.

2.5.4.2.3.4.3 Intact Young's Modulus (Ei)

Young's' Modulus values for intact rock core samples (Ei), based on laboratory testing of intact rock core samples from the borings within and adjacent to the nuclear island basemat footprints, were segregated according to rock type (micritic limestone and argillaceous limestone). Mean values and one standard deviation range about the mean, are input for the Hoek-Brown equations.

2.5.4.2.3.4.4 Excavation Disturbance “D” Factor

Excavation disturbance factor “D” is an estimated value based on the degree of disturbance in the rock mass developed from construction photographs of the Units 1 and 2 excavations. The value used in the calculations, 0.85, corresponds to very poor quality blasting in hard rock, resulting in severe local damage extending 6.6 to 9.8 ft into the surrounding rock mass. This assigned D value is conservative based on the Bellefonte Units 1 and 2 construction excavation photographs that show minimal, or only slight localized rock mass disturbance as a result of foundation excavation.

2.5.4.2.3.4.5 Material Indices

Material indices (mi) are based on chart values for similar rock types according to Hoek (Reference 442). The range in values for the hard, well-cemented micritic limestone and argillaceous limestone (mi range of 5 to 9) are in the range of those reported for softer and weaker rock types such as siltstone (mi = 7 ±2) and shale (mi = 6 ±2) providing a conservative assessment.

2.5.4.2.3.5 Hoek-Brown Equation Estimated Rock Mass Properties

Hoek-Brown calculations were performed for each rock type using estimated confining stress values of 2500 psf (120 kilopascals, kPa) and 5000 psf (239 kPa). These values bracket the estimated in situ stress range between the top of the excavated sound rock (2500 psf, 120 kPa) and the nuclear island basemat (5000 psf, 239 kPa) imparted by the rock and soil overburden. Estimated properties were calculated for both stress ranges assuming three rock mass conditions:

- Mean Rock Mass – resultant from mean input values for each parameter.
- Lower Bound Rock Mass – resultant from mean minus one standard deviation input values for each parameter.
- Upper Bound Rock Mass – resultant from mean plus one standard deviation input values for each parameter.

The resulting 12 Hoek-Brown curves represent a reasonable bracketed range for typical variations in the rock mass character. Figures 2.5-322 and 2.5-323 show the estimated shear strength envelope curves and in situ rock mass Young's modulus values. Key output parameters are summarized in Table 2.5-236.

Estimated rock mass Young's moduli were compared to in situ Goodman Jack tests that were performed at or near the nuclear island basemat elevations in three BLN COL borings within the footprint areas. The Goodman Jack tests estimated rock moduli ranging between about 889 to 5875 kips per in^2 (6129 to 40507 MPa); similar to, but higher than, the range of 481 to 1563 kips per in^2 (3316 to 10777 MPa) estimated for the global in situ rock mass by the Hoek-Brown method. The comparison shows good correlation between the independently derived rock mass estimates. The values resulting from the borehole Goodman Jack tests are somewhat higher because the borehole test intervals are small with fewer sampled discontinuities than the volume of rock mass considered in the Hoek-Brown analysis. Therefore, the Hoek-Brown results were selected for input for analysis of the nuclear island basemat bearing capacity, settlement, and sliding performance.

2.5.4.3 Foundation Interfaces

BLN COL 2.5-6 This subsection describes the relationship between site exploration, subsurface materials and the foundations of seismic Category 1 facilities. The information was developed on the basis of field explorations performed at the Unit 3 and 4 site and on laboratory tests performed on soil and rock samples obtained during the field exploration program. Field investigations performed at the adjacent Bellefonte Units 1 and 2 site in the 1970s and 1980s (Reference 201), and exploration at the southern site, shown on Figure 2.5-201 were also considered in this assessment, based on their close proximity and the similarity in geology at the three sites.

For the Units 3 and 4 investigation, a data collection plan was prepared which outlined the field program, including surface and subsurface exploration. The BLN program was conducted in 2006 using experienced and qualified geologists and engineers. Geotechnical data collected during the field and laboratory exploration program were analyzed and evaluated. The analysis included preparing tables and figures that represent interpretations of the subsurface geotechnical conditions beneath the power block construction zone.

2.5.4.3.1 Power Block Construction Zone Exploration

An exploration program of surface geophysics, in situ testing and subsurface drilling and sampling was conducted as shown in a comprehensive plan view on Figures 2.5-324 and 2.5-325. A description of the type, quantity, extent and purpose of these explorations is provided in Subsection 2.5.4.2. Figures 2.5-326 shows a series of primary and secondary rotary wash rock core borings. In situ tests, including borehole geophysics, Goodman Jack and packer permeability tests were conducted at locations shown on Figure 2.5-327. Test pit excavations and Cone Penetrometer test (CPT) probes were conducted at locations shown on Figure 2.5-328. Surface geophysical exploration, including seismic refraction and microgravity transects were performed at locations diagrammed on Figure 2.5-325, and discussed in Subsection 2.5.4.4. Groundwater monitoring points, included nested monitoring wells and pump test observation wells are shown on Figure 2.5-325.

2.5.4.3.2 Surrounding and Adjacent Structures Exploration

Exploration of facilities outside the power block construction zone was conducted. These explorations included profile borings for characterization and siting of monitoring wells, rotary wash borings adjacent to the existing cooling towers, and general yard borings to supplement and confirm previous exploration (Reference 201). In addition a single test pit (T-1415) was excavated northwest of Unit 3 and 4 power block construction zone. The exploration locations are shown on Figure 2.5-327.

2.5.4.3.3 Geotechnical Data Logs and Records

Several geotechnical data sets were used to compile the geotechnical figures contained in this subsection. Two of these are provided in Appendices; they include geotechnical soil and rock boring logs (Appendix 2BB), and logs of test pits (Appendix 2CC). A limited number of static CPT's were pushed, and are presented in Table 2.5-228.

As-built survey data and topographic surveys were used to prepare maps of the final geotechnical data exploration program as presented in Figures 2.5-324 and 2.5-327. The locations of exploratory borings, monitoring wells, test pits, and surface geophysical lines were recorded in digital format. These data were uploaded into a geographic information system (GIS). The GIS was used to prepare plan and profile drawings for this subsection.

Geotechnical borings and core box photographs as well as Natural Gamma, and P-S Suspension logging surveys were used to interpret the stratigraphic units presented in the geotechnical profiles, as discussed below in Subsection 2.5.4.3.5.

Laboratory test results of rock strength and locations of samples taken for petrography, mineralogy, and Dynamic Resonant Column/Torsion Shear tests are provided on the borehole summary sheets (Figures 2.5-329, 2.5-330, 2.5-331, 2.5-332, 2.5-333, 2.5-334, 2.5-335, 2.5-336, 2.5-337, and 2.5-338) and geotechnical cross sections (Figures 2.5-339, 2.5-340, and 2.5-341).

2.5.4.3.4 Borehole Summaries

The compilation of the geologic and geotechnical data collected from the field program is essential to interpret the subsurface conditions. Data including lithology, laboratory strength, P-S velocity and Natural-Gamma geophysical logging, Standard Penetration Test (SPT), Rock Quality Designation (RQD), and percent recovery were used to compile borehole summaries of key primary borings. An explanatory figure showing these data sources is included as Figure 2.5-329, followed by nine Borehole Summaries, Figures 2.5-330, 2.5-331, 2.5-332, 2.5-333, 2.5-334, 2.5-335, 2.5-336, 2.5-337, and 2.5-338. These summaries convey the integrated field, laboratory and geologic framework essential for creating profiles across the nuclear islands as discussed in Subsection 2.5.4.3.4.

2.5.4.3.5 Geotechnical Profiles

BLN COL 2.5-5 The borehole summaries were evaluated in the geologic context described in more detail in Subsections 2.5.1 and 2.5.4.1 to construct geotechnical profiles.
BLN COL 2.5-6 Three geotechnical profiles crossing the nuclear islands are presented; the locations of which are on Figure 2.5-324. Section A-A', B-B' and C-C' are shown

on Figures 2.5-339, 2.5-340, and 2.5-341. Geotechnical profile A-A' links the nuclear island of Unit 3 and 4 in a northwest to southeast direction. Geotechnical profiles B-B' and C'C' project through Units 3 and 4 respectively. These profiles include existing ground surface and nuclear island foundation grades for reference.

Below is a brief summary of the soil and rock conditions depicted on the profiles. A more thorough discussion of the site geology is presented in Subsections 2.5.1 and 2.5.4.1. Material properties are discussed in Subsection 2.5.4.2.

2.5.4.3.5.1 Soil

A surficial residual soil layer blankets the Units 3 and 4 power block construction zone. It represents the residuum of the weathering of the underlying limestone bedrock. In some instances a thin veneer of fill overlies the residual soil. Fill thickness and Standard Penetration Test (SPT) blow counts within the soil are not shown on the geotechnical profiles due to inappropriate map scale. The residual soil/top of rock contact, shown in gold on Figures 2.5-339, 2.5-340 and 2.5-341 is defined by hollow stem auger refusal.

2.5.4.3.5.2 Rock

As shown on Figure 2.5-339 bedrock beneath Units 3 and 4 is characterized by dipping carbonate bedrock strata of the geologic Stones River Group of Ordovician age. Six lithologic units were identified in the middle Stones River Group at the BLN site. They are shown on the profile and labeled as Units A through F. These units strike to the northeast and dip 15° to 17° to the southeast. The units primarily consist of micritic limestone (with intervals of packstone and wackestone), but also contain interbeds of argillaceous limestone. This argillaceous limestone is discussed in detail in Subsection 2.5.4.1.

2.5.4.3.5.3 Groundwater

Groundwater levels shown on the geotechnical cross-sections reflect field measurements recorded during the geotechnical drilling program. The levels reflect the stabilized measurements from the open borehole. Each of the boring logs shown on the profiles contains a reference to Note A for the recorded measurements. Monitoring well water level measurements are presented in Subsection 2.4.12 and a site-specific discussion on groundwater conditions is in Subsection 2.5.4.6.

2.5.4.4 Geophysical Surveys

BLN COL 2.5-6 This subsection presents the surface and borehole geophysical surveys that were conducted on the BLN site to characterize the subsurface conditions of soil and bedrock including dynamic properties and geologic features. The information obtained from these surveys was used in the development of site stratigraphy discussed in Subsection 2.5.1.2, evaluation of the potential for surface faulting presented in Subsection 2.5.3, and characterization of geologic features as provided in Subsection 2.5.4.1.

The investigations were conducted using methods described in Section 4.4 of Regulatory Guide 1.132. Planning and exploration layout and data collection was coordinated by project engineering geologists and geotechnical engineers.

Five types of geophysical surveys were performed at the BLN site:

- Seismic refraction surveys by MACTEC Engineering & Consulting, Inc
- Seismic cone penetrometer test (SCPT) shear wave velocity measurements in overburden soils by Gregg Insitu, Inc.
- Suspension and downhole logging tests by GeoVision, Inc.
- Microgravity surveys by Golder Associates Inc.
- Natural gamma borehole surveys by Norcal Geophysical Consultants, Inc.

This geophysical survey data was compared with geophysical survey results documented in the Units 1 and 2 FSAR (Reference 201) and in the 2006 TVA report (Section 2.5.4.4 in Geophysical Surveys).

2.5.4.4.1 Seismic Refraction Surveys

Field seismic refraction measurements were made in three phases at the BLN site. A precharacterization survey was conducted across the footprint of the power block footprints of Units 3 and 4, prior to the commencement of exploratory drilling and sampling. The pre-characterization survey included 2500 lineal feet of survey over 3 transects measured on 13 arrays. A second phase of seismic refraction was conducted after a portion of the exploratory drilling was complete, and the third near the end of the drilling program. The seismic refraction line surveys under phases 2 and 3 totaled 12,690 ft. of survey along 15 transects using 27 arrays. Locations of the survey lines are shown in Figure 2.5-342. The position and length of the survey lines were selected for the siting of secondary borings, stratigraphic interpolation between existing borings, to obtain compressional (Vp) wave velocity data for bedrock rippability, to profile the top of rock, to provide comparison of Vp velocities with borehole surveys, and to confirm the presence of potential lineaments at the site.

2.5.4.4.1.1 Seismic Refraction Survey Methods

Seismic refraction methods used to determine the seismic P-wave velocity structure of the subsurface included ASTM D5777 (Reference 444). This survey method involves generating seismic P-waves at the ground surface, which propagate through the soil and rock and were recorded by geophones at known distances from the source. Seismic investigations reported herein were conducted using a 24-channel digital enhancement seismograph. The energy source was an 8 gauge seismic shotgun (Betsy Gun). Shotpoints were in pre-drilled holes in either the landscaped areas or pavement. 2.5 Hz geophones were coupled to the ground surface using spikes coupled to the soil or pre-drilled holes in the pavement.

The data were analyzed by using a computer display of the arrival times to select compression wave arrival times. These were then input to a calculation program (SeisOpt® 2D™ [Reference 445]) that allows optimization of velocity analysis. Like the manual methods of analysis, the computer software uses the first break picks of the P-wave arrival times (observed data) to model the subsurface.

2.5.4.4.1.2 Seismic Refraction Survey Results

Velocity profiles of the refraction survey data indicate subhorizontal velocity intervals. Some irregularity is observed which most likely represents variable weathering on the top of limestone bedrock. Figures 2.5-343, 2.5-344 and 2.5-345 display the velocity panels the correspond with Unit 3 and 4 nuclear islands, as well as geotechnical Profiles A-A', B-B', and C-C'. Correlation with the microgravity surveys on other profiles are presented on Figure 2.5-346.

2.5.4.4.1.3 3-D Seismic Refraction Interpretation

Utilizing the extensive 2-D seismic refraction data, a set of 3-D models was developed within the Units 3 and 4 power block construction zone. A comparison of the seismic refraction velocity profiles with nearby boring data was made to correlate the top of weathered rock and the top of competent rock with seismic p-wave velocity layers. Four seismic velocities then were chosen for 3-D modeling: 4000, 6000, 8000, and 14,000 fps. Contour maps of the top of each velocity layer were made.

Seismic P-wave velocities of 6000 and 14,000 fps are most correlative with the top of weathered rock and the top of competent rock, respectively. The 6000 fps model is presented in Figure 2.5-312 and the 14,000 fps model is shown in Figure 2.5-313. Subsection 2.5.4.1.3.3 presents a thorough discussion of the interpretation of these results with respect to weathering of bedrock and karst.

2.5.4.4.2 Seismic Cone Penetrometer Test

One SCPT with seismic shear wave velocity test was performed at each of the Annex Buildings footprints of each BLN unit as shown on Figure 2.5-328. The CPT with seismic shear wave velocity testing allowed direct measurements of shear wave velocity in the residual soils. These measurements were made at 5 foot intervals. The shear wave velocity (Vs) provides information about small-strain stiffness while the penetration data provides information about large-strain strength. From interval shear wave velocity (Vs) and the mass density (ρ) of a soil layer, the dynamic shear modulus (G_o) of the soil can be calculated in a specific depth interval. The dynamic shear modulus (G_o) is a key parameter for the analysis of soil behavior in response to dynamic loading.

Results are summarized in Table 2.5-237. Each of the SCPT shear wave velocity tests was performed in residuum or fill soils above bedrock. The SCPT seismic test results summarized in Table 2.5-237 indicate that the S-wave velocity of the overburden soils at the BLN site ranges from 550 to 767 feet per second [fps].

The average S-wave velocity results from the downhole tests in the residuum at the Units 1 and 2 site ranged from 336 to 1217 fps, consistent with the seismic cone results from the BLN Site. Results from eight SCPT shear wave velocity tests at the southern site ranged from 354 to 1527 fps (Reference 399).

2.5.4.4.3 Suspension and Downhole Logging Tests

Compressional and Shear (P-S) Suspension logging tests were performed within eight soil and rock core holes advanced at the BLN site. Downhole methods were performed in the same boreholes as two of the P-S holes. The P-S logging was performed in rock core holes and in some cases offset soil borings. Table 2.5-238 provides a summary of the testing performed. Figure 2.5-326 indicates the locations of these tests. The objective of the suspension and downhole logging tests was to obtain S-wave and P-wave velocity values as a function of depth. The S-wave velocity values were used to determine whether the unweathered rock met the hard rock requirements for the site response analyses and SSE determination discussed in Subsection 2.5.2. A further discussion of the dynamic properties of soil and rock is presented in Subsection 2.5.4.7. Results of the suspension logging tests, shown on Figures 2.5-329, 2.5-330, 2.5-331, 2.5-332, 2.5-333, 2.5-334, 2.5-335, 2.5-336, 2.5-337, and 2.5-338) were also used in comparisons to data measured in rock at the Units 1 and 2 site. The Units 1 and 2 data included P-wave velocity results from geophysical logging with a Birdwell tool, as well as crosshole results to depths of nearly 100 ft. using explosive sources.

2.5.4.4.3.1 Test Methods

The downhole and P-S suspension logging services were performed in accordance with GEOVision Procedures.

Prior to performing the tests, each of the boreholes was advanced to final depth using HQ coring equipment. Steel casing was installed through the residuum or fill and weathered bedrock at each borehole to ensure borehole integrity. Water was maintained to within a few feet of ground surface in each borehole.

Continuous geophysical logs could not be collected in the soil and rock portions of some boreholes because of limitations on the hole sizes and preparation needs. To allow full coverage of the profile, logging was conducted in two adjacent boreholes. The original borehole provided logging for the rock. An adjacent borehole, designated with a PS suffix to the boring number, was drilled without soil sampling to rock, and a PVC casing was set, grouted in place and allowed to cure. After the grout had cured, rock coring was done to a nominal distance to assure the logging tools could fully image the soil section. Finally, the logging tools were used to collect data for the PS borehole. During processing, GeoVision combined the data from the two boreholes to provide a single borehole log.

The suspension logging tests were performed using an OYO Model 170 Suspension Logging Recorder and Probe. A seismic source is mounted near the base of the probe, and a pair of receivers are mounted approximately 3 ft. apart from one another, centered approximately 12-ft. above the source. The source generated a P-wave in the pore fluid near the base of the probe, which was converted to a S-wave and separate P-wave at the borehole wall. The shear wave traveled up along the wall, and the resulting compression wave was measured by the receiver pair. The S-wave and P-wave velocity for the interval between the receivers was then calculated based on the difference in wave arrival times.

Shear wave measurements were performed at 0.5-ft. intervals in each borehole, starting from approximately 15-ft. above the bottom of the boreholes. Tests were performed in bedrock below the depth of casing at each borehole (i.e., near the top of unweathered limestone).

2.5.4.4.3.2 P-S Test Results

The test results show relatively consistent shear wave (Vs) and compressional wave (Vp) velocities with depth. Below the base of weathered bedrock, most of the measured rock section exhibits Vs of 9200 fps or greater, and Vp of 14,000 fps or greater. Lower velocity levels correspond to argillaceous limestone intervals (described as shale on the boring logs). The Vs and Vp log responses have excellent agreement with the Natural Gamma logs as shown in Boring B-1034. A thorough discussion of soil and rock Vs profiles is presented in Subsection 2.5.4.7.

2.5.4.4.4 Microgravity Surveys

Microgravity measurements were obtained at the site to provide a secondary surface method for modeling the subsurface. Using similar techniques as used at the southern site provided comparative methods of analysis. The microgravity surveys consisted of 121 stations measured on 11 transects. The station spacing was 20 ft. on transects ML-1 through ML-6 coincident with selected seismic refraction survey transects. The remaining station spacing was 10 ft. on 3-D transects ML-7 through ML-11 located near Boring B-1051. The microgravity survey services were performed in general accordance with ASTM D6430 (Reference 446).

Microgravity measurements were made with a L & R Aliod 100 G-meter with a reading resolution of 5 microgals (μgal) and a standard deviation of less than 10 μgals. The average value of gravity on the Earth's surface is 980 milligals (mgals). Corrections for earth tides, free-air, latitude, Bougeur, and terrain corrections were applied to the readings.

Gravity data were collected at 121 stations along 11 transects. Five of the gravity transects overlapped with seismic refraction lines with stations on 20-foot centers. Six shorter transects with 10-foot station spacing were surveyed in the vicinity of boring B-1051 to provide a 3-D model of the subsurface. The gravity data were recorded in a field notebook and stored digitally in the instrument then downloaded each day to a laptop computer. The computer program GravMaster was used to process the gravity data.

In general the five gravity transects which overlap with selected seismic refraction surveys show good agreement with the seismic refraction data. The remaining six transects used for the 3-D surveys were critical in modeling a deeper weathering zone beneath the turbine building of Unit 4. Figure 2.5-309 shows the 3-D contours in the vicinity of B-1051.

2.5.4.4.5 Televiewer and Natural Gamma Surveys

Sixteen Borehole Televiewer (BHTV) and fifteen Optical Televiewer (OPTV) surveys were made in open boreholes at the Bellefonte site. In addition seven companion Natural-Gamma (N-Gamma) and sixteen Caliper test logs were conducted (Table 2.5-239). The surveys were advanced with winch wireline systems using multiple tool configurations.

Selected borehole logging suites were conducted at 17 separate locations (Figure 2.5-326). The BHTV measured borehole wall features using acoustic methods. The OPTV measured the same parameters using optical techniques in clear borehole fluid. The Caliper log measured borehole rugosity (roughness) and diameter, and the N-Gamma measured gamma intensity as a function of clay-bearing rock units. The logging was done in two separate mobilizations (phases) and in both vertical and inclined boreholes (Table 2.5-239).

The N-Gamma survey services were performed in accordance with ASTM D6274 (Reference 447). The BHTV and OPTV logging services were performed using procedures that used the Robertson Geologging Digital Optical Televiewer (OPTV) and the Technical Specification for the Robertson Geologging Hi-Resolution Acoustic Televiewer (HiRAT).

The results of the Natural Gamma are included on selected logs shown on Figure 2.5-330 and Figure 2.5-334. The response shown on the logs correlates relatively well with intervals of argillaceous limestone (kick to the right) and micritic limestone (kick to the left). These were confirmed with comparison of the core retrieved from the respective logs and were used in correlation of the stratigraphy across the BLN site.

2.5.4.5 Excavations and Backfill

BLN COL 2.5-7 This section discusses the excavation, backfill, and earthwork to be performed at the BLN site. The discussion includes the following elements:

1. The extent of Category I excavations, fills, and slopes.
2. Excavation methods and control of groundwater during excavation to preclude degradation of foundation materials.
3. Sources and properties of borrow and backfill materials.
4. Compaction specifications.
5. Quality Control (QC) programs related to foundation excavation, and protection and treatment of foundation subgrades.
6. Measures to monitor foundation heave and rebound.

2.5.4.5.1 Plans and Sections

Earthwork for Units 3 and 4 involves both cut and fill for overall site development, excavation for the Category I structures, and placement of backfill between the foundation walls and the sides of the excavations for the nuclear islands. Figure 2.5-347 illustrates the plan extent of the excavation and fill areas for the nuclear islands. Figures 2.5-348a and 2.5-348b illustrate the vertical extent of the excavations.

2.5.4.5.1.1 Overall Site

Figure 2.5-347 shows the general arrangement of the structures for both power block units within a construction zone measuring approximately 2650 ft. by 1770 ft. in plan dimensions. Subsection 2.4.1.1 defines the standard plant floor elevation/plant grade as 628.6 ft. (AP1000 design plant grade of elevation 100). From this elevation, the ground surface slopes down and away from the buildings to the northeast, northwest, and southwest quadrants to the limits of the construction zone, where cut or fill slopes extend to reach the existing ground. In the southeast quadrant, the ground surface outside the construction zone slopes up to match existing ground on a cut slope with a maximum height of approximately 160 ft.

The top edges of the fill embankments are no closer than 300 ft. to the nuclear island. As discussed further in Subsection 2.5.5.1.1, a failure of the fill embankment would be too far away from the nuclear island to cause an impact. In addition, the fill materials between the nuclear island and a distance of twice the embedment of the nuclear island (2 times 80 ft.) would not be affected by a failure at the edge of the fill area, and no loss of lateral confinement would occur. Thus there are no safety-related fill embankments or embankment foundations associated with the general site grading fills.

The existing soils where the fill embankments are constructed are shown by borings to be stiff clays that, based on experience, are capable of supporting the embankment heights shown on the plan without the need for special foundation designs.

2.5.4.5.1.2 Power Block

Site grading for the power block areas, which consist of the Turbine Building, Annex Building, Radwaste Building, Diesel Generator Building, and the Nuclear Island, involves approximate maximum depths of fill and cut of 20 ft. and 40 ft., respectively. Only the Nuclear Islands are Category I structures, and only these are discussed further. Considering a bottom of excavation grade at elevation 588.6 ft. for the Units 3 and 4 Nuclear Islands, the approximate depth of excavation for Unit 3 varies from 18 to 24-ft. below ground surface, and the approximate depth of excavation for Unit 4 varies from 26 to 33-ft. below the present ground surface.

During construction, temporary excavated slopes are made around the perimeter of the nuclear island footprints for construction of the basemat foundations. Figure 2.5-347 shows locations of two profiles through the nuclear island footprints that display the typical geometries and extents of basemat excavations. Figures 2.5-348a and 2.5-348b illustrate the excavation concept for the conditions at BLN. The side slopes of the excavation in rock are at an angle of 85° extending upward from the bottom of the excavation, and range in height from about 7 to 17 feet. These cuts locally may be higher where overexcavation is required below the basemat subgrade to provide sound rock embedment. A 10-foot wide bench is constructed at the top of rock (weathered rock) along the excavation perimeter, and the upper part of the excavation slopes made in residual soil above the rock are inclined at 1.5:1 (horizontal to vertical) up to the existing ground. The soil portions of the excavated slopes range in height between approximately 7 to 19 feet. The construction excavation slopes are maintained until completion of the nuclear island basemat foundation and lower structural walls.

2.5.4.5.2 Construction Excavation and Dewatering

The lateral extent of seismic Category I excavations is shown on Figure 2.5-347. There are no seismic Category I fills or cut slopes. The lateral and vertical extent of the seismic Category I excavations and of the fills is discussed in Subsection 2.5.4.1. Figures 2.5-348a and 2.5-348b show the vertical extent of the seismic Category I excavations.

2.5.4.5.2.1 Excavation Support

The soil overburden present across the site can be excavated using conventional methods such as scraper pans and track-mounted backhoes (trackhoes). In the

Nuclear Island areas, scraper pans may be used in the initial phases of excavation, where the excavated materials consist of residual soils that generally classify as a high plasticity clay (CH), with lesser amounts of medium plasticity clay (CL). When the depth of excavation makes it impractical for the scraper pans to exit the excavation, trackhoes could be used, with the material being loaded into trucks and hauled to an on-site stockpile, or placed as fill in non-structural or non-Category I areas. The excavation is staged, creating pads for the excavating equipment to work from as the excavation is extended deeper. Because of the clay soils and relatively small depths of soil excavation, the use of soil nailing is not needed, and the temporary slope method for slope retention is used. The soil excavation is sloped at a 1.5 (Horizontal): 1 (Vertical) inclination, as illustrated in Figures 2.5-348a and 2.5-348b, so lateral support is not required. Analyses of the temporary soil slopes show factors of safety greater than 2.0.

As discussed in Subsection 2.5.4.1.3, the weathered rock that is present in the transition zone between the soil overburden and coreable rock varies in thickness and relative density. In areas where the weathering is at an advanced stage, the material may be removed using conventional methods, such as a trackhoe with a toothed bucket. Some isolated ripping may be needed, and would likely be accomplished by an experienced equipment operator working the bucket into seams and joints and “prying” the material out.

When more resistant, less weathered materials are encountered, the conventional excavation methods described above are no longer effective. At that point, blasting or rock splitting methods will be used. General construction experience has shown that the quality of rock that requires blasting to be removed can be defined by a seismic P-wave velocity of 6,000 feet per second (fps) or greater.

Blasting is quicker than rock predrilling or line drilling, but would also result in more noise and vibrations, and a greater potential for damage to rock bearing surfaces. On the other hand, rock predrilling or line drilling would be slower than blasting, but reduces potential for overexcavation to remove damaged rock. The experience from construction of Bellefonte Units 1 and 2 shows that predrilling and line drilling were used for the near-vertical rock excavation sides with controlled blasting techniques for the foundation areas with normal and acceptable levels of damage that were readily addressed by inspection and repairs (Reference 201).

The rock is excavated unsupported, at an approximate 85° inclination from horizontal. Kinematic analyses using properties from the Hoek-Brown evaluations discussed in Subsection 2.5.4.2.3.3 and an average assumed interface friction value of 35°, bedding plane failure is not a viable failure mode. Movement of individual rock blocks is kinematically possible, but the number of frequency of such potential failures is believed to be low and could be addressed by localized excavation support, block removal or flattening the cut slope, all typical procedures for rock excavations. Based on the performance of the rock cut slopes during construction of Bellefonte Units 1 and 2, and current analysis discussed in Subsection 2.5.5, the slopes can perform satisfactorily at this inclination.

2.5.4.5.2.2 Dewatering

BLN COL 2.5-8 Based on information from a limited number of monitoring wells discussed in Subsection 2.4.12, and from water level observations during the boring program discussed in Subsection 2.5.4.6, the general groundwater level across the site is near the rock surface. In the proximity of Unit 3, groundwater was measured in boreholes at approximate elevations ranging from 601 to 596 ft., (elevations are referenced to the NAVD 88 datum), as shown in Table 2.5-241. In the proximity of Unit 4, groundwater was measured in boreholes at approximate elevations ranging from 612 to 601 ft. as shown in Table 2.5-242. The observed water levels were generally within 2 ft. above or below top of rock elevations.

Based on information gathered from monitoring wells screened above the rock surface, there is a water table trapped in the soil zone (perched water). In the Unit 3 area, the maximum perched groundwater level is about elevation 605 ft. At Unit 4, the maximum level is at about elevation 615 ft. Table 2.5-243 shows this information. Additional information regarding groundwater conditions can be found in Subsection 2.5.4.6.

Groundwater flow into the nuclear island excavations from isolated pockets of water, mainly associated with rock seams and joints, can seep into the excavation, and can accumulate with time if left unattended. As during the past construction, it is anticipated that groundwater infiltration can be managed by pumping from sump pits at the excavation low points.

The rocks that are present at the foundation bearing level are hard micritic and argillaceous limestones. Thin section analyses discussed in Subsection 2.5.4.1 do not indicate presence of significant clay or other minerals susceptible to degradation upon exposure to water. No observed degradation of exposed foundation rock was reported for Bellefonte Units 1 and 2 (Reference 201). Degradation of foundation materials due to groundwater infiltration is not anticipated.

2.5.4.5.3 Backfill

BLN COL 2.5-7 The current site grades are below the final site grades as shown on Figure 2.5-347. Fill is placed to reach the final site grades. The excavation adjacent to the nuclear island is defined as shown on Figures 2.5-348a and 2.5-348b. Fill concrete is placed between the basemat and the excavated rock slopes, and the remainder of the excavation is filled to plant/yard grade using fill derived from borrow sources that are to be identified and tested. Fill specifications are described in Subsection 2.5.4.5.4. The estimated quantity of this backfill is 55,000 cubic yards.

Potential sources of borrow material include on-site sources identified during previous and current explorations. This section discusses these materials and sources, compaction specifications, and Quality Control (QC) testing recommendations.

2.5.4.5.3.1 Materials and Sources

Several potential on-site sources of backfill material have been previously identified and investigated. These include the borrow areas explored during preparation of the Bellefonte Units 1 and 2, and the borrow area identified during preparation of a 1987 soils Investigation at the site. The Units 1 and 2 FSAR (Reference 201) borrow areas are located northeast, southwest, and south of the existing structures, while the 1987 area was located immediately north of Units 3 and 4. The investigations of these borrow areas included power and hand augers on relatively even spacing across the borrow areas, and multiple borings on a grid pattern across the site. Drilling was terminated in the soil overburden.

The current subsurface exploration consisted of 122 borings across the site, with a focus on the power block areas (see Figure 2.5-327). Except for two of the borings are in areas where new fill is required. Two of the outermost borings south and east of Unit 4 (B-1069 and B-1091) were performed in the southeast quadrant of the construction zone where cut is required, and could provide potential borrow materials. The approximate cut depth in the vicinity of these borings is above their respective auger refusal depth. Therefore, the excavated material at these two locations is soil overburden.

Several test borings were performed in the nuclear island areas to define the subsurface conditions and characterize the materials that would be excavated. These borings include the following:

- Unit 3: B-1000, B-1002 through B-1005, B-1007, B-1014, and B-1088.
- Unit 4: B-1034 and B-1035, B-1037, B-1040, B-1042, B-1044 and B-1045, B-1047, B-1049, and B-1053.

Based on the findings of these borings, the soil overburden depth varies from 5 ft. to 17-ft. below present ground surface for Unit 3 and from 8 ft. to 19 ft. below present ground surface for Unit 4. Weathered rock and rock are present below the soil overburden to the depths explored.

2.5.4.5.3.2 Material Properties

Numerous samples were tested during the previous studies to determine the engineering characteristics of the soil overburden. As a part of the current study, over 100 samples (split-spoon, thin-walled tube, and bulk) were tested. The testing during each of the studies included index (moisture content, particle size analysis, Atterberg limits), specific gravity, triaxial compression, one-dimensional consolidation, and moisture-density relationship (Proctor) testing. Subsection 2.5.4.2 discusses the testing and results in detail.

The test results indicate that the majority of the soils tested generally consist of medium to high plasticity clays and silts (designated as CH and MH per the Unified Soil Classification System – USCS). Lesser amounts of low to medium plasticity clays (CL) and silts (ML), and gravels (GC, GP, GW) are also present. A summary of the index properties of samples obtained from both Units 3 and 4 power block areas is shown in Table 2.5-240. The test results are representative of the properties obtained from testing performed on samples obtained from outside the power block areas, and are consistent with test results reported in the previous studies.

The soils from the previous on-site borrow areas were divided into four classes, based on classification and properties. The classes, and their USCS classification symbols, are as follows:

- Class I – SC
- Class II – CL
- Class III – CH
- Class IV – CH

Based on the laboratory test results from these investigations, Class III and IV soils would generally be considered undesirable for use as structural fill. Class I and Class II soils would be more suitable for use as structural fill because they exhibit lower plasticity, and contain more coarse-grained particles, than the Class III and IV soils. However, the Class III and IV soils, which are prevalent across the site and in the outlying borrow areas, may be used in the deeper areas of fill, in non-structural areas.

In structural areas, fill material consists of Class I, or better, soils. The geotechnical properties for Class I soils, as identified in the Units 1 and 2 studies are as follows:

- USCS Classification – SC
- Percent Fines – 34
- Liquid Limit (LL) – 28
- Plasticity Index (PI) – 11
- Standard Proctor Maximum Dry Density (MDD) – 1.93 g/cm3 (120.5 pcf)

- Standard Proctor Optimum Moisture Content (OMC) – 13.3%
- Angle of Internal Friction, φ (at OMC) – 23.4º
- Angle of Internal Friction, φ (at 2% above OMC) – 10.9º
- Cohesion, c (at OMC and 2% above OMC) – 66.0 KPa (0.69 tsf; 1,380 psf)

If off-site borrow material is needed to provide sufficient quantity of borrow to grade the site, then the off-site borrow material for the structural areas will meet the criteria outlined in Subsection 2.5.4.5.4.

Alternatively, rock fill resulting from the nuclear island excavation may be used as structural fill in the vicinity of the Category I structures. If used, rock fill is required to meet gradation requirements discussed in Subsection 2.5.4.5.4.1.2. Rock fill is placed to within no closer than 4 ft. to the base of a structure footing, and is placed in accordance with the guidelines presented in Subsection 2.5.4.5.4.1.2.

2.5.4.5.4 Recommended Backfill Material

When considering materials for use as backfill behind retaining walls and/or below grade walls, there are several characteristics that are desirable. The material should be free-draining, low plasticity, possess a relatively minor percentage of fines, and not be susceptible to shrink/swell. With the exception of the Class I soils from the Units 1 and 2 FSAR, the on-site soils generally do not exhibit these characteristics. Therefore, the on-site Class II through Class IV soils are not suitable for use as backfill behind the below grade walls of the nuclear islands.

In the space between the edge of the concrete basemat for the nuclear islands and the rock excavation, backfill material consists of lean concrete. In the space between the foundation walls and the soil excavation, the material to be used as backfill consists of Class I, soils or soils with lower percentage of fines and lower plasticity. The typical geotechnical properties of Class I soils were discussed in Subsection 2.5.4.5.3.2.

Using these properties as a guideline, backfill material should contain a maximum 35% fines (material passing the No. 200 sieve), with a maximum particle size of 2-inches or less. The material should have a Liquid Limit (LL) less than 35, and a Plasticity Index (PI) less than 15. Placement criteria and compaction specifications are presented in Subsection 2.5.4.5.4.1. Figures 2.5-348a and 2.5-348b illustrate the zone where Class I fill is required adjacent to the nuclear island walls.

Only isolated lenses and pockets of coarse-grained materials (Class I or better) soils are present on-site; therefore, suitable backfill materials are obtained from off-site sources. Off-site sources were not identified during the previous or current studies. Suitable sources can be identified at a later date. Sampling and testing is performed to demonstrate that the borrow source meets the above-defined criteria for backfill.

2.5.4.5.4.1 Compaction Requirements

2.5.4.5.4.1.1 Soil

Compacted fill outside of the wall backfill zone at the nuclear island areas is to be placed in horizontal 6-to 8-inch) lifts and compacted to a minimum of 98% of the maximum dry density obtained in accordance with ASTM Specification D698 (Standard Proctor) (Reference 428). The moisture content of fill at the time of placement should be within minus 1 percentage point to plus 2 percentage points of the optimum moisture content determined in the laboratory. In addition, soils containing more than 5 percent (by weight) fibrous organic materials, having a Liquid Limit (LL) greater than 35, Plasticity Index (PI) greater than 15, or Standard Proctor maximum dry density less than 1.6 g/cm^3 (100 pcf) should not be used for fill in structural areas. Fill for areas designated to receive Class 1 fill must meet the Class I fill requirements discussed in Subsection 2.5.4.5.4. Prior to the commencement of filling operations, samples of each fill material must be obtained and tested to determine that the index properties, maximum dry density, and optimum moisture content values meet the stated criteria.

Fills placed against sloping surfaces should be benched into the existing slope in horizontal layers to provide a satisfactory bond and to avoid planar surfaces of potential sliding. Materials should not be placed when either the fill material or the foundation surface is excessively wet, frozen, improperly compacted, or otherwise unsuitable.

Suitable compaction equipment in large areas consists of sheeps-foot or smooth-drum vibratory compactors, depending on the type of fill material that is being placed. In smaller confined areas, such as between the nuclear island foundation walls and the rock/soil cuts, smaller pieces of compaction equipment are more suitable due to maneuverability and the lower pressures that would be imparted on the adjacent wall. In backfill areas, maximum lift thicknesses may have to be limited to 6-inches in order to achieve proper compaction using the small equipment.

2.5.4.5.4.1.2 Rock Fill

Rock fill may be used as an alternative fill material below non-safety related structures. An exception to rock fill placement would be in the Turbine Building area, where rock fill would prohibit the installation of deep foundations. In that area, the fill should consist of Class I or better soils as described in Subsection 2.5.4.5.3.

Criteria for rock fill placement, compaction requirements, and fill constituents are presented below:

- Prior to rock fill placement, the exposed subgrade is evaluated by a qualified Geotechnical Engineer or his representative.

- Maximum rock particle size is to be limited to 18 inches to within 10 ft. of foundation grade, to 4-inches in the zone between 10-ft. below foundation grade and 4-ft. below foundation grade, and not used in the upper 4 ft. to avoid interference with possible utility construction.

- Rock fill lift thickness should be limited to a maximum of 18 inches.

- To achieve adequate compaction of rock fill, use of 6 to 8 passes of heavy construction equipment (i.e., Caterpillar D-6 or other larger or similar equipment) in two directions on the fill surface (half the passes in each perpendicular direction) is required. A test pad program to evaluate lift thickness, number of passes, and establish control methods is needed at the start of construction. Experience indicates that heavy self-propelled sheepsfoot rollers (similar to a Caterpillar 815), or vibratory drum rollers, when used in conjunction with normal spreading and hauling equipment, enhance fill performance. These rollers can better compact fine portions of the fill and tend to break-up some larger rock pieces.

- Rock fill must have adequate fines to effectively "choke" the larger rock pieces by filling the voids and open spaces. The larger rock pieces should lie flat and not overlap each other. The estimated percentage of soil in the fill should be limited to a maximum of 10 percent by volume.

Rock fill placement and compaction techniques would be closely monitored by a trained engineering technician, under the direction of the Geotechnical Engineer. The technical level personnel documents fill constituents, lift thickness, and compaction techniques.

2.5.4.5.4.2 Quality Control Testing

Following stripping of surficial materials, areas in the vicinity of the Category I structures that receive structural fill are evaluated for suitability by proofrolling with a 25 to 35 ton, four wheeled, rubber tired roller or similar approved equipment, such as a loaded tandem-axle dump truck. The proofroller makes at least four passes over the areas, with the last two passes perpendicular to the first two. Proofrolling is performed after a suitable period of dry weather to avoid degrading an otherwise acceptable subgrade. Proofrolling is observed by an engineering technician working under the supervision of the Geotechnical Engineer. Any areas that pump, rut or deflect excessively and continue to do so after several passes of the proofrolling equipment are evaluated by the Geotechnical Engineer. Proofrolling of temporary construction slopes is not required; these slopes are benched to provide bond between the slope and the fill.

Field density testing for fill is performed during fill placement by an experienced engineering technician working under the supervision of the Geotechnical

Engineer to measure the degree of compaction being obtained. A minimum of one density test is performed in general accordance with ASTM Specification D1556 (Reference 449) for each 2500 square feet of lift area with a minimum of two tests per lift.

Subgrade soils can deteriorate and lose their support capabilities when exposed to environmental changes and construction activity. Deterioration can occur in the form of freezing, formation of erosion gullies, extreme drying, exposure to moisture, or exposure for a long period of time to construction traffic. Subgrades that have deteriorated or softened must be proofrolled, scarified and recompacted (and additional fill placed, if necessary) immediately prior to placement of additional fill, or construction of a floor slab or pavement. Additionally, any excavations through the subgrade soils (such as utility trenches) are properly backfilled and compacted in thin loose lifts. Recompaction of subgrade surfaces and compaction of backfill are evaluated by performing field density tests to verify that adequate compaction is being achieved.

2.5.4.5.4.3 Quality Assurance Program

The Quality Assurance Program in place during design, construction, and operations phases is discussed in Section 17.5.

2.5.4.5.5 Foundation Excavation Monitoring

BLN COL 2.5-12 BLN COL 2.5-13 The foundation levels for the Category I structures result in bearing on rock. The exposed rock in side slopes as well as the bearing areas are examined in detail by a field geologist working under the supervision of a licensed geologist. Geologic maps of the excavation sides and the bearing surface are prepared to document the subgrade conditions, identify areas needing additional rock removal, placement of dental concrete or grout or installation of rock bolts for slope integrity or prior to placing concrete or a mud mat for subgrade protection. Subsection 2.5.4.12 provides further discussion of the improvement techniques.

BLN COL 2.5-13 Based on experience with construction of Bellefonte Units 1 and 2, there was no documented rebound or heave of the foundation excavation surface. Therefore, there are no plans to monitor foundation rebound and heave during construction of Units 3 and 4. Additional discussion regarding rebound and heave can be found in Subsection 2.5.4.10.

2.5.4.6 Groundwater Conditions

BLN COL 2.5-1 BLN COL 2.5-8 This subsection discusses the groundwater conditions at the site relative to the foundation stability for the safety-related facilities. The occurrence of groundwater and the history of groundwater fluctuations as presented in Subsection 2.4.12 are reviewed. The results of field permeability tests (hydraulic conductivity tests) are presented. No laboratory permeability tests were conducted because the soil zone is thin and consists of clays which by their nature have low permeability. Dewatering during construction and the analysis and interpretation of seepage and potential piping conditions during construction are presented.

2.5.4.6.1 Groundwater Occurrence

Information on groundwater conditions was collected from boreholes at the time of drilling and from monitoring wells with long-term water level readings. Monitoring wells were installed, at locations shown on Figure 2.4.12-212, in groups of two or three wells that penetrate to different depths. The wells were denoted A, B, or C depending on the depth of penetration with A being the shallowest. The A-series wells were installed with the screen interval in the soil, above the rock to allow checks of potential perched water. The groundwater elevations recorded for the monitoring wells over the whole area of the site exploration are shown on Figure 2.4.12-218. The groundwater elevations vary with the spatial distribution of the wells across the site and with differences in the depths of penetration. For any given location; however, groundwater elevations show minor fluctuations. Subsection 2.4.12 provides discussion of seasonal fluctuations and responses to flood events for the groundwater.

Water level data collected at the time of drilling or shortly thereafter from borings in the proximity of Unit 3 (B-1001, B-1002, B-1003, B-1004, B-1005, B-1007, B-1014, B-1021) are shown in Table 2.5-241. The data indicates groundwater levels ranging from elevation 601.3 to 595.7 ft., (elevations are referenced in the NAVD 88 datum). These elevations represent a range of 3.7-ft. above, to 6.2-ft. below, the top elevation of the rock. Figure 2.5-349 presents hydrographs of wells near Unit 3.

Water level data collected at the time of drilling or shortly thereafter from borings in the proximity of Unit 4 (B-1034, B-1037, B-1040, B-1042-I, B-1044, B-1045, B-1055) are shown in Table 2.5-242. The data indicates groundwater levels ranging from elevation 612.2 to 600.9 ft., corresponding to 7.4-ft. above to 2.8-ft. below the top elevation of the rock. Figure 2.5-350 presents hydrographs of wells near Unit 4.

Monitoring wells terminating at different depths indicate that independent and varied piezometric levels may exist in a given profile. Typical causes of this are perched water conditions and the nature of the joint connectivity in the bedrock. Piezometric levels measured in shallower wells (A-series), which were terminated in soil, are generally higher than those in deeper wells (B- and C-series), which were terminated in rock. Rock frequently has smaller interconnected porosity compared to soils, causing piezometric levels in rock to typically change more rapidly, and to a greater degree, than in soils.

The groundwater records from shallow wells screened only in the soil reflect perched groundwater conditions above the bedrock whereas those from deeper wells reflect groundwater conditions within the bedrock. Table 2.5-243 shows the highest and lowest groundwater records for shallow monitoring wells (A series) in the proximity of Units 3 and 4. Table 2.5-244 shows the highest and lowest groundwater records for deeper monitoring wells (B and C series) in the proximity of Units 3 and 4.

For the two A-series monitoring wells near Unit 3, the highest recorded groundwater elevation is 605.2 ft., which corresponds to 9.4-ft. above bedrock. In a different part of the year, the same well was recorded as being dry (i.e. groundwater level was lower than elevation 601.1 ft.). Based on these limited data, the perched groundwater near Unit 3 fluctuates between elevations lower than 601.1 ft. to elevation 605.2 ft. With respect to top of rock elevations, the perched groundwater is 3.4 to 9.4-ft. above the top of rock.

For the two A-series monitoring wells near Unit 4, the highest recorded groundwater elevation is 614.7 ft., and the lowest recorded level is 605.1 ft., corresponding to 4.5-ft. to 10.7-ft. above bedrock.

In the B- and C-series wells in the area of Unit 3, the groundwater levels in the rock range from elevation 496.5 to 602.3 ft. This range corresponds to 6.5-ft. above rock to 101.5-ft. below rock. In the same series wells in the area of Unit 4, the groundwater levels within the rock range in elevation from 565.6 to 613.3 ft. This range corresponds to about 1.5 ft. above the top of rock to 39.6-ft. below the top of rock.

2.5.4.6.2 Field Hydraulic Conductivity Testing

BLN COL 2.5-6 An aquifer pump test was performed at the site in September 2006. Subsection 2.4.12 contains detailed information on the test and results. The pumping well was located south of Unit 3, and several surrounding wells were monitored for drawdown during the test. During the 25-hour pumping phase, the total drawdown of the pumping well was approximately 8.4 ft. The closest surrounding wells experienced a rise in water levels during this phase, while two wells that were located further north of the pumping well experienced minor drawdowns. After an approximately 42-hour recharge period, the pumped well had an approximate 2-ft. rise in water level, and the immediate surrounding wells also experienced increased water levels. The furthest removed well, north of the pumped well, experienced a minor drawdown, similar in magnitude with the drawdown experienced during the pumping phase.

In-situ testing for hydraulic conductivity was conducted using double packer pressure testing in seven boreholes as indicated on Table 2.5-227. Several depth intervals in each borehole were tested. In nearly half of the depth intervals tested, an effective hydraulic conductivity could not be established because sustained flow was not observed under the test pressures. Inability to establish a sustained flow indicates that the amount of flow occurring through intact portions of the rock is insignificant. At depth intervals having sustained flow, the effective hydraulic conductivity ranged from 776.1 feet per year (fpy) to 4323.9 fpy. The hydraulic conductivity of the rock is controlled by the frequency and nature of jointing in the rock mass. The effective hydraulic conductivity measured at foundation elevation 588.6 ft. of Unit 3 is 649 fpy. Tests conducted near the foundation level of Unit 4 did not achieve a sustained flow, and no effective hydraulic conductivity could be determined.

2.5.4.6.3 Construction Dewatering

BLN COL 2.5-8 The excavation to reach foundations of the Units 3 and 4 nuclear Island structures extends into the rock. Water flow in the rock occurs only along existing joints and discontinuities. Seepage of water out of the excavation sides is expected to be slight and to vary around the excavation based on local jointing and fracturing in the rock. Seepage from the soil portions of the excavation slopes is expected to be slight due to the low hydraulic conductivity of the clay soils. Typically, water flow occurs in the zone at the interface between the soil and the rock. Construction experience with Units 1 and 2 showed that seepage did not impact the condition of the foundation rock, and did not impact the excavation slopes. Based on that experience and the similarity of the conditions for Units 3 and 4, no plans for quantitative analysis of seepage flow during construction are needed.

Dewatering for the construction can be accomplished efficiently by establishing and maintaining several low points during excavation. The base of the excavation is sloped toward the low points where sump pits are dug for the purpose of collecting and pumping water out of the excavation. This method of handling inflows was used successfully during construction of Bellefonte Units 1 and 2 (Reference 201). Lowering of the perched groundwater in the soils is not expected to cause settlement of adjacent ground because the soil overlying the bedrock is mostly composed of stiff overconsolidated clays and the amount of water level reduction is slight.

2.5.4.6.4 Groundwater Impacts on Foundation Stability

BLN COL 2.5-7 Units 3 and 4 are founded within hard, competent rock. Changes in groundwater level have no impact on the foundation settlement or bearing stability.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

BLN COL 2.5-6 This subsection provides a description of the response of soil and rock to dynamic loading including the following:

- Investigations of the effects of historic earthquakes on soil and rock, such as paleoliquefaction (Subsection 2.5.4.7.1).
- Compressional and Shear (P and S) wave velocity profiles from surface or in-hole geophysical surveys, including data and interpretation (Subsection 2.5.4.7.2).
- Results of dynamic laboratory testing of soil and rock samples (Subsection 2.5.4.7.3).
- Foundation conditions and uniformity (Subsection 2.5.4.7.4)
- Presentation of dynamic profiles (Subsection 2.5.4.7.5)

The dynamic properties for the site (seismic wave velocity, shear modulus, damping) for evaluation of earthquake site response were developed from extensive field measurements of rock and native residual soil in borings within the BLN Units 3 and 4 power block construction zone and laboratory dynamic testing of residual soil from select boring samples discussed in Subsection 2.5.4.7.3, and review of properties for existing fill reported in the Units 1 and 2 FSAR as described in Subsection 2.5.4.5. These data were compiled and statistically analyzed to develop a suite of dynamic velocity profiles to evaluate epistemic variability (uncertainty in the mean) in rock properties for general classification of the site (e.g., hard rock, DCD Subsection 2.5.4.5) and develop the site Ground Motion Response Spectra (GMRS; Subsection 2.5.2.6) for comparison with the Certified Seismic Design Response Spectra (CSDRS).

2.5.4.7.1 Prior Earthquake Effects and Geologic Stability

As discussed in Subsection 2.5.1, no active or potentially active faults or seismic deformation zones occur at the Bellefonte site. Geologic mapping and subsurface explorations discussed in Subsection 2.5.4.1 confirm that rock and soil materials at the Units 3 and 4 power block have not experienced seismically-induced ground failure (e.g., slope failure, liquefaction, lurching, subsidence) from historic or paleoearthquakes. Bedrock underlying the Units 3 and 4 power block construction zone consists of interbedded Paleozoic limestone and argillaceous limestone of the middle Stones River Group with uniform bedding that strikes to the northeast and dips consistently to the southeast at 15° to 17°. The uniform bedding was confirmed in the subsurface by a dense network of continuously-logged vertical and inclined rock core borings (to a maximum depth of 251 ft., and can be predictably traced between borings throughout the Units 3 and 4 power block construction zone as shown on cross sections in Subsection 2.5.4.3. Both the strike and dip, and thicknesses, of individual beds are consistent, and provide evidence for a lack of active deformation at the Power Block.

A review of available literature and field reconnaissance was conducted to determine the effects any previous earthquakes may have had on the site, including paleoliquefaction. Searches on available geologic and seismologic literature, and consultation with the Alabama Geologic Survey, indicated a lack of documented liquefaction or earthquake-induced ground failure at the site. During BLN investigation aerial reconnaissance and field mapping, no evidence of paleoearthquake-induced ground failure was found in the surficial deposits or exposed bedrock. Based on reviewed and collected data, there is no evidence of paleoearthquake-induced ground failure in soil or bedrock materials at, or near, the BLN site, supporting a low potential for earthquake-induced ground failure to occur in the native geologic deposits.

2.5.4.7.2 Field Dynamic Measurements

The following techniques were used to measure dynamic properties within the Units 3 and 4 power block construction zone:

- Borehole P-S seismic velocity suspension logging surveys in 13 borings ranging in depth between about 23 to 251 ft and including rock, native soil, and existing fill reported in Subsection 2.5.4.4.

- Borehole downhole seismic velocity surveys in two borings (boring B-1059 and B-1032) that also were surveyed with P-S suspension logging for independent comparison of velocities measured in rock reported in Subsection 2.5.4.4.

- Seismic Cone Penetrometer Test (SCPT) seismic velocity surveys made in two soundings in native soil and existing undocumented fill reported in Subsection 2.5.4.4.

- Surface refraction velocity surveys performed in a grid pattern with 10-foot geophone spacing throughout the Units 3 and 4 power block construction zone reported in Subsection 2.5.4.4.

2.5.4.7.3 Laboratory Dynamic Testing

The following laboratory testing technique was used to measure dynamic soil properties:

- Resonant Column/ Torsional Shear (RCTS) testing of shear modulus and damping of six undisturbed samples of native residual soil.

2.5.4.7.4 Foundation Conditions and Uniformity

The foundation conditions and geologic profile vary under the Units 3 and 4 power block construction zone. This variability is the result of alternating and shallowly dipping beds of limestone and argillaceous limestone, variations in the depth to the rock surface, differential residual soil thickness, and topographic variability, as described in Subsection 2.5.4.1. Additionally, fill placed over existing rock and/or residual soil raises the ground surface to the new plant grade elevation of 628.6 ft., North American Vertical Datum 1988 (NAVD88). The fill thickness generally varies between approximately 5 to 28 ft.

The Units 3 and 4 nuclear island basemats are at approximate elevation 588.6 ft., or 40 ft. below plant grade. As discussed in Subsection 2.5.4.10, the nuclear island basemat excavations are extended a minimum of 5 feet into sound, bedded limestone and argillaceous limestone that dip uniformly to the southeast at an inclination of about 15° to 17°.

Overexcavation below the basemat subgrade elevation may be required under portions of the basemat footprints to achieve the required sound rock embedment. The 5-ft. embedment recommendation extends foundations below possible weathered and/or dilated rock, and provides essentially uniform foundation conditions for the basemats. Variability in rock properties between the limestone and argillaceous limestone are not deemed significant because the strength and moduli of the relatively weaker argillaceous beds are still well above requirements for foundation bearing capacity, settlement, etc. as described in Subsection 2.5.4.10. Overexcavation areas can be filled with concrete up to basemat subgrade to provide a dense, coupled interface with sound rock.

The geologic conditions satisfy the definition of a “Uniform” hard rock site specified in the DCD Section 2.5.4.5 for the nuclear island basemats. The limestone bedrock is regularly bedded with a gentle dip 15° to 17° inclination, and individual beds exhibit substantial uniformity in conditions both along strike and dip throughout the Units 3 and 4 power block construction zone. The weathered top of rock is irregular with local variations in depth to top of rock on the order of about 3 to 10 ft. typically, but is globally quite flat without an overall sloping surface.

The groundwater table at the site occurs in the residual soils slightly above the bedrock surface. The groundwater table elevation is laterally variable based on groundwater measurements in both monitoring wells screened in the residual soils and the bedrock as described in Subsection 2.5.4.3. In the proximity of Unit 3, groundwater was measured in boreholes at approximate elevations ranging from 601 to 596 ft. In the proximity of Unit 4, groundwater is measured in boreholes at approximate elevations ranging from 612 to 601 ft. Based on information gathered from monitoring wells screened above the rock surface, there is a water table trapped in the soil zone (perched water). In the Unit 3 area, the maximum perched ground water level is about elevation 605 ft. At Unit 4, the maximum level is at about elevation 615 ft.

Borehole-specific velocity profiles, and the downhole velocity profiles, were consolidated onto the master stratigraphic profile correlating with geologic lithology. This was accomplished by translating the separate profiles "downdip" along key rock subunit marker beds (e.g., subunit "C" argillaceous limestone") to the corresponding position of the specific marker beds on the master profile. Figure 2.5-351 shows the resulting stratigraphic-velocity profile for the Vs rock data. Figure 2.5-339 shows a cross-section through the two nuclear islands showing geologic subsurface conditions.

2.5.4.7.5 Dynamic Profiles

This subsection presents the approach used to develop site-specific dynamic velocity profiles at the BLN site.

Dynamic velocity profiles were compiled and applied at two locations for evaluation of the potential range of dynamic properties and site ground motion characteristics of seismic Category I GMRS and safety-related structures (nuclear island structures) and at eight other locations outside of the nuclear island structures. The locations of base case dynamic velocity profiles developed for the BLN Units 3 and 4 are presented in Figure 2.5-352.

Two profiles are provided (Figure 2.5-353) that provide the hard rock foundation profiles (GMRS) beneath Units 3 and 4.

The base case dynamic velocity models of Units 3 and 4 are shown on Figures 2.5-354, 2.5-355, and 2.5-356.The base case suite defined for the BLN site considers variability of site conditions such as material thickness and lateral variability within foundation rock, and fill. The site GMRS are described in Subsection 2.5.2.6.

The existing residual soil, and fill placed over residual soil, are unconsolidated materials susceptible to cyclic "softening", but not liquefaction. Subsection 2.5.4.8 discusses the liquefaction evaluation for residual soil and fill. Dynamic shear modulus reduction and damping properties of the residual soil for site response were derived by laboratory RCTS testing of undisturbed Shelby samples of native residual soils performed at the University of Texas, Austin. Fill properties are based on assumed properties for Class I fills specified for the existing BLN Units 1 and 2. The selected test specimens included both upper and lower residual soil layers. The results from the RCTS tests are plotted on standard plasticity index-correlated Vucetic and Dobry (V&D) (Reference 450) shear modulus and damping curves developed for cohesive soils similar to the site residual soils, and shown on Figure 2.5-357. The plotted RCTS test data were visually compared against the V&D curves to select best fit curves to represent the upper residual soil, and lower residual soil. The plasticity index values for the best-fit curves were then compared against the measured plasticity indices for the test samples. The plasticity indices for the selected best-fit curves were in good agreement with the measured plasticity indices for the test samples, as shown on Table 2.5-245 and Table 2.5-246, providing verification of the suitability of the matching. The selected V&D curve for plasticity index of 15 percent was selected for the upper residual soil and a plasticity index of 50 percent for the lower residual soil. RCTS testing was performed at 1.0 and 4.0 times the effective confining stress of the samples, estimated from average units weights, groundwater conditions, and depth of samples. Based on review of the shear wave velocity matching between lab and field, and curve matching of data, the RCTS data sets from the 1.0 times confining stress test series were chosen to represent the site residual soil conditions.

Fill adjacent to the nuclear island walls is assumed to consist of borrow materials meeting the Units 1 and 2 FSAR specifications for Class I sand fill (SC) that includes a fines content of 34 percent, plasticity index less than 15 percent, and maximum field density of 120.5 pounds per cubic foot at optimum moisture content of 13.3 percent (Reference 201) Dynamic shear modulus reduction and damping properties for the assumed Class I fill are based on the standard EPRI (Reference 451) curves for sand at the corresponding depths.

2.5.4.8 Liquefaction Potential

2.5.4.8.1 Overview

BLN COL 2.5-9 In meeting the requirements of 10 CFR Parts 50 and 100, if the foundation materials at the site adjacent to and under Category I structures and facilities are saturated soils and the groundwater table is above bedrock, then an analysis of the liquefaction potential at the site is required. The need for a detailed analysis is determined by a study on a case-by-case basis of the site stratigraphy, critical soil parameters, and the location of safety-related foundations.

As discussed in Subsection 2.5.4.10, the Class I safety-related nuclear island basemat subgrades for Units 3 and 4, at approximate Elevation 588.6 feet (40 feet below plant grade) provides a minimum embedment of 5 feet into sound limestone and argillaceous limestone bedrock of the Paleozoic middle Stones River Group that is not susceptible to liquefaction. Plan maps, cross sections, and summary boring logs presented in Subsection 2.5.4.3 show the locations and rock foundation conditions of the Category I nuclear island basemats.

Within the Units 3 and 4 power block construction zone clay residual soil, averaging 5 to 15 feet thick, overlies the bedrock surface. The residual soils consist predominantly of stiff clay with fines content greater than 30 percent and Plasticity Index (PI) greater than 30, as described in Subsections 2.5.4.2 and 2.5.4.3. The groundwater table at the site occurs in the residual soils slightly above the bedrock surface. Fill adjacent to the nuclear island walls, and averaging 5 to 20 feet thick, is placed over the residual soil to raise the existing ground surface to plant grade at elevation 628.6 feet. Subsection 2.5.4.5 describes fill specifications that are in conformance with the Bellefonte Units 1 and 2 FSAR specifications for Class I sand fill (SC) that includes a fines content of

34 percent plasticity index less than 15 percent, and maximum field density of 120.5 pounds per cubic foot (at optimum moisture content of 13.3 percent). Shallow foundations for non-Category I plant facilities adjacent to the nuclear island (e.g. turbine building, annex building) are founded in fill placed over residual soil, or on piers or piles extended through the fill and residual soil to bear on the bedrock.

The clayey and stiff nature of the native residual soil and dense fill exhibit low susceptibility to liquefaction. A liquefaction screening assessment in conformance with Regulatory Guide 1.198 “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites” was performed to demonstrate the low liquefaction hazard associated with the residual soil and fill at the Units 3 and 4 power blocks, as discussed below.

2.5.4.8.2 Geologically-Based Liquefaction Assessment

As discussed in Subsection 2.5.3, no active or potentially active faults or seismic deformation zones occur at the Bellefonte site. In Subsections 2.5.4.1 and 2.5.1 geologic mapping and extensive subsurface explorations confirm that rock and soil materials at the Units 3 and 4 power blocks have not experienced seismically-induced ground failure (e.g., slope failure, liquefaction, lurching, and subsidence) from historic or paleoearthquakes. Therefore, the geologic setting and past performance indicate that liquefaction is not expected within the residual soils overlying bedrock. Furthermore, natural slopes surrounding the Units 3 and 4 power block area and yard fill are stable without evidence of past instability.

The geologic screening process described in Regulatory Guide 1.198 was applied to the BLN residual soil and the Class 1 SC fill. This process is based largely on work by Youd (Reference 452 and Reference 453), that show most liquefaction risk is associated with saturated, recent Holocene deposits of loose sand and silt and uncompacted fills (typically hydraulically-placed sandy fill). The BLN clay residual soil and fill do not fall within these categories of susceptible soil. Figure 2.5-358 is a geologically-based screening flow chart used to evaluate the BLN residual soil (screening process is limited to naturally-occurring deposits) that factors past performance, deposit age, percent granular material, and estimated Safe Shutdown Earthquake Peak Ground Acceleration range. This screening process indicates a very low susceptibility for liquefaction for the BLN residual soils.

2.5.4.8.3 Soil Texture-Based Liquefaction Assessment

A second screening method, involving quantitative evaluation of soil fines (clay and silt) content, Plasticity Index (PI), Liquid Limit (LL), and in situ water content was performed to provide an independent method to assess liquefaction potential of both residual soil and fill. The texture-based evaluation is consistent with Regulatory Guide 1.198 procedures, and state-of-industry criteria presented in Reference 454. Input for this analysis was derived from geotechnical laboratory index testing described in Subsection 2.5.4.2, and fill specifications for Bellefonte

Units 1 and 2 presented in the Bellefonte Units 1 and 2 FSAR, described above in Subsection 2.5.4.8.1. Laboratory testing included 76 samples from the residual soil and the following specific test methods:

- Mechanical grain size determinations
- Hydrometer grain size (clay versus silt) determinations
- Atterberg Indices (Plastic Limit, LL and PI)

Seventy five of the 76 tested samples (>98 percent) exhibit fines content greater than 35 percent, and most are classified as Lean to Fat Clay (CL-CH). This alone suggests a low potential for liquefaction. The other sample point was collected in gravel fill beneath asphalt, and removed during grading and excavation. Figure 2.5-359 is a liquefaction screening chart that provides further evaluation of residual soil and fill samples based on comparison of PI and LL ratios (PI:LL) against three zones correlating to liquefaction potential: Zone A Potentially Liquefiable; Zone B Marginal Zone; and a third non-designated zone of Low to No liquefaction potential. Test points with PI:LL ratios falling within Zones A or B are subject to additional screening based on the ratio between natural water content (WC) and 80 percent of the LL (0.8 LL), to screen the material as potentially liquefiable or not.

As shown on Figure 2.5-359, 65 of the 76 residual soil test points (~86 percent) fall clearly outside the chart zone of potential liquefaction, and therefore are considered to be non-liquefiable on the basis of their PI:LL ratios. Nine of the remaining test points (~12 percent) fall within the Zone B "Marginal" zone, but are screened out based on comparisons between WC and LL. Only two of the residual soil test points (<3 percent) fall within Potentially Liquefiable Zone A, but these are screened out based on comparison between WC and LL. Therefore, the texture-based screening chart method shows that the residual soil samples are non-liquefiable.

The fill placed between the nuclear island walls and the excavation sides results in a "sample" point indicated by the star symbol on Figure 2.5-359, and falls along the border between Zones A (Potential) and B (Marginal) based on PI:LL ratios. However, this material is screened out based on the WC to LL comparison, assuming a WC of about 15 percent correlating to the specified optimum water content for compaction (optimum moisture content plus two percent). The assumed WC of 15 percent is well below 0.8LL (22.4%), indicating that a margin exists with respect to possible future higher moisture content in the fill that could occur as the result of poor drainage. Therefore, the texture-based screening chart method shows that the fill is non-liquefiable.

The texture-based liquefaction screening provided independent confirmation of low liquefaction potential for the residual soil and fill.

2.5.4.9 Earthquake Site Characteristics

BLN COL 2.5-2 A performance-based site-specific Ground Motion Response Spectrum (GMRS) was developed in accordance with the methodology provided in Regulatory Guide 1.208. This methodology and the GMRS are provided in Subsection 2.5.2.6. The GMRS satisfies the requirements of 10 CFR 100.23 for development of a site-specific Safe Shutdown Earthquake (SSE) ground motion.

As recommended in Regulatory Guide 1.208, the following general steps were undertaken:

- Review and update the EPRI (1986) (Reference 455) seismic source model for the site region (200-mile radius).
- Update the EPRI (1989) (Reference 233) ground motion attenuation model using the EPRI (2004) (Reference 350) ground motion attenuation model.
- Perform sensitivity studies and an updated Probabilistic Seismic Hazard Analysis (PSHA) to develop rock hazard spectra and define the controlling earthquakes.
- Derive performance-based GMRS from the updated PSHA at a free field hypothetical outcrop of the top of competent material beneath the nuclear island.

The dynamic properties of soil and rock at the site were determined through a program of field exploration, laboratory testing and analysis as described in Subsections 2.5.4.2, 2.5.4.4 and 2.5.4.7. The Units 3 and 4 power block construction is located on rock with a shear wave velocity generally greater than 8000 fps.

2.5.4.10 Static Stability

BLN COL 2.5-10 The static stability of the Bellefonte Nuclear Plant, Units 3 & 4, nuclear island facilities was evaluated for foundation settlement, foundation bearing capacity, and lateral pressures against below grade walls. Evaluation of static stability was limited to the safety-related nuclear island facilities only. A discussion of bearing capacity, settlement, and lateral pressure evaluations is provided in Subsections 2.5.4.10.1 through 2.5.4.10.3. Foundation materials at the location of Bellefonte Nuclear Plants Unit 3 and Unit 4 consist of continuous rock and fill concrete placed on top of continuous rock. The fill concrete is used where the elevation of continuous rock is below the elevation of the nuclear island foundation. Additional discussion of the configuration of the foundations is presented in Section 3.8. The source and derivation of the rock engineering properties used for evaluation of the bearing capacity, potential rebound, settlement, and differential settlement are described in Subsection 2.5.4.2 and are shown in Table 2.5-236.

2.5.4.10.1 Bearing Capacity

The bearing capacity was evaluated at each unit using two independent methods. The methods used were:

- Method 1 - Ultimate Bearing Capacity using the Terzaghi approach based on the strength of the rock mass (Reference 456),
- Method 2 - Peck, Hanson & Thornburn, for allowable bearing pressure based on RQD of the rock (Reference 457).

Under Method 1, the ultimate bearing capacity is computed from the equation

$$q_{ult} = cN_c + 0.5\,\gamma B N_\gamma + \gamma D N_q$$ (Equation 6-1, Reference 456)

where

q_{ult} = the ultimate bearing capacity

γ = effective unit weight (i.e. submerged unit wt. if below water table) of the rock mass

B = width of foundation

D = depth of foundation below ground surface

c = the cohesion intercept for the rock mass

The terms N_c, N_γ, and N_q are bearing capacity factors given by the following equations:

$$N_c = 2N\phi^{1/2}(N\phi + 1)$$ (Equation 6-2a, Reference 456)

$$N_\gamma = N\phi^{1/2}\left(N_\phi^2 - 1\right)$$ (Equation 6-2b, Reference 456)

$$N_q = N_\phi^2$$ (Equation 6-2c, Reference 456)

$$N_\phi = \tan^2(45 + \phi/2)$$ (Equation 6-2d, Reference 456)

where

ϕ = angle of internal friction for the rock mass.

The value for ϕ was conservatively taken as 46°, the lower bound value for Unit A argillaceous limestone (the weaker of the two rock types) determined from Hoek-Brown analyses discussed in Subsection 2.5.4.2.3.4.

Method 2, the Peck, Hanson & Thornburn (Reference 457) method, is a widely used empirical design approach for determining the allowable bearing pressure to limit settlement in which bearing capacity is related to the Rock Quality Designation (RQD). The RQD is a measure of rock integrity determined by taking the cumulative length of pieces of intact rock greater than 4 inches long for the length of a core sampler advance and dividing by the length of the core sampler advance, expressed as a percentage. The average RQD for the Unit A argillaceous limestone is 87. For conservatism, a disturbance factor of 0.85 was applied to account for near surface blasting damage, resulting in an RQD of 74. Charts in Reference 457 were used to determine the bearing capacity.

Using the lower bound rock properties, both methods show bearing capacities well above the requirements in DCD Table 2-1 (8600 pounds per square foot [psf] for static and 35000 psf for dynamic). The calculated bearing capacities under both static and dynamic conditions are:

- Method 1; 251,000 psf, and
- Method 2; 236,000 psf.

BLN COL 2.5-7 A mud mat composed of lean, nonstructural concrete is placed between the prepared rock foundation bearing surface and the structural foundation mat (DCD Subsection 3.8.5.1). The mud mat has a specified concrete compressive strength of 17.4 MPa (2500 pounds per square inch) (Reference 458). Testing during construction is required to verify achievement of the specified compressive strength. A waterproof membrane is incorporated within the mud mat.

2.5.4.10.2 Resistance to Sliding

BLN COL 2.5-6 Resistance to sliding is normally computed by comparing the forces causing sliding to the resisting forces developed by friction between the foundation and the bearing material. For a soil site, a minimum friction angle of the soil underlying the mud mat needs to be 35° which yields a coefficient of friction of 0.7 as a minimum requirement. As noted in Subsection 2.5.4.10.1, the lower bound friction for the rock mass is 46°, which exceeds the minimum requirement. As an additional conservatism, the mat foundation is below the rock surface and the space between the edges of the mat foundation and the rock is filled with concrete. This approach provides an ability for the rock to aid in resisting lateral forces in addition to the base mat friction.

2.5.4.10.3 Rebound Potential

BLN COL 2.5-12 Rebound is evaluated by comparing the vertical change in stress due to removal of the soil and rock above the foundation level to the elastic properties of the material at and below the foundation level. The excavation depths below present ground level for BLN range from about 18 ft. to about 33 ft. The reduction in stress caused by this amount of soil and rock removal is less than about 217 kPa (31 psi). Because the elastic modulus of the rock, as reported in Subsection 2.5.4.2.3.2 is at least 3316 MPa (481000 psi), the elastic strain from removal is very small (217 kpa divided by 3316 MPa = 0.00006). Thus, the potential for significant rebound of the foundation rock is non-existent.

2.5.4.10.4 Settlement

2.5.4.10.4.1 Total Settlement

BLN COL 2.5-12 Estimates of post-construction settlement were calculated separately for Unit 3 and Unit 4 based on the theory of elasticity. Settlements were estimated by three
BLN COL 2.5-16 methods;

- use of the Boussinesq Equation (Reference 459)
- use of the Corps of Engineers Equation (Reference 456)
- use of the Steinbrenner Equation (References 460 and 461).

The calculations estimated settlement resulting from static loading of the nuclear island foundation bearing directly on rock or bearing on a depth of fill concrete in turn resting on rock. An equivalent area approach was used to model the nuclear island as one or more rectangular areas for purposes of estimating settlement.

The settlement methods listed above evaluate settlement by dividing the subsurface into layers with discrete elastic modulus values. The change in stress at the midpoint of a layer is calculated using elastic theory for loads applied to a semi-infinite half-space. The compression of each layer is computed as the result of dividing the applied stress increment by the elastic modulus to obtain an incremental strain, then multiplying the incremental strain by the layer thickness. The layer results are summed to obtain a total settlement. Variations in methods relate to the approach to obtain the layer modulus and the stress in the layer.

Because the rock mass is not intact, elastic moduli from laboratory tests on intact specimens must be reduced to reflect the rock mass character as discussed in Subsection 2.5.4.2.3.4. The reduced modulus and Poisson's ratio used to develop a subsurface model of the rock layers below the foundation were developed as described in Subsection 2.5.4.2.3.4 through use of the Hoek-Brown approach. The lower bound modulus value of the weakest rock layer was used in all cases to provide conservatism.

The reduced modulus values for continuous rock were used even though there may be instances where rock is expected to be removed and replaced with fill concrete. Reduced modulus values of the in-situ rock are lower than that of the fill concrete. This results in additional conservatism for the settlement estimate since the rock modulus values are used in place of fill concrete modulus values.

Bellefonte Nuclear Plant nuclear island structures are founded on rock and fill concrete, which does not incur sufficient settlement to disrupt the operation of the structure. The computed settlements from the methods used are less than about 0.20 in. The maximum estimated settlement is 0.18 in. beneath Unit 3 and 0.20 in beneath Unit 4. The magnitude of these settlement estimates are within the limits allowed by the DCD. This is consistent with expectations for a site utilizing rock to support the Nuclear Islands.

2.5.4.10.4.2 Differential Settlement

The rock layers forming the bearing for the nuclear island meet the criteria in DCD Table 2-1 for a uniform site; differential settlement is not a factor.

2.5.4.10.5 Lateral Earth Pressures

BLN COL 2.5-7
BLN COL 2.5-11

Lateral pressures develop against below grade nuclear island walls due to placement and compaction of soil backfill materials. The lateral pressures were developed based on the information listed below:

- The soil used to backfill adjacent to the walls of the Nuclear Islands has material properties as described in Subsection 2.5.4.5.3.2.
- Backfill soil adjacent to Nuclear Island walls (see Figures 2.5-348a and 2.5-348b) is compacted to 95 percent of the maximum dry density determined from the standard Proctor laboratory test performed in accordance with ASTM D698 (Reference 428).
- Backfill is compacted at water contents ranging from near the laboratory optimum value to no wetter than two percentage points above the optimum value. This results in an as-compacted initial degree of saturation of 80 percent or less.

- Light, hand-guided compaction equipment is used to compact the soil within 5 ft of the Nuclear Island walls. This avoids compaction-induced soil stresses against the wall.

- The Nuclear Island walls do not yield due to the lateral earth pressure applied to them. The at-rest pressure is the appropriate earth pressure to use for design of the walls.

- An at-rest earth pressure coefficient (K0) of 0.81 is appropriate for the Bellefonte Nuclear Plant backfill soils if compacted to 95 percent, 2 percent above the optimum moisture content.

- The Rankine earth pressure theory is used to compute the passive (ultimate) earth pressure.

Earth pressure coefficients for the at-rest and passive conditions determined using the methods described in Reference 462 are illustrated in Figures 2.5-360 and 2.5-361. Figure 2.5-360 shows the distribution of earth pressure from the soil backfill (at-rest condition), and, below the water table, the additional pressure caused by hydrostatic pressure. Figure 2.5-361 shows the soil passive pressure distribution. No hydrostatic pressure is included in the passive pressure because water has no shear strength and provides no additional passive resistance.

BLN COL 2.5-3 BLN COL 2.5-5 BLN COL 2.5-6 BLN COL 2.5-10

2.5.4.11 Design Criteria

Table 2.0-201 compares the DCD site parameter criteria and the site characteristics, including the following items:

- Average Allowable Static Bearing Capacity

- Maximum Allowable Dynamic Bearing Capacity for Normal Plus SSE

- Shear Wave Velocity

- Site and Structures conditions and geologic features

- Properties of the Underlying and Adjacent Subsurface Materials and Geologic Features

- Groundwater Level

- Lateral Variability of Foundation Bearing Material Stiffness

- Liquefaction Potential

Design of safety related foundations is based on the nuclear island foundation mat being supported by continuous rock or by fill concrete supported on continuous rock. Continuous rock is defined, for this purpose, as rock that is fresh to moderately weathered and has a Rock Qualify Designation of greater than 65%, based on boring logs. Rock descriptions and RQD values from the COL borings provide an initial estimated depth of removal to meet the criterion. Field examination of the excavation as discussed in Subsection 2.5.4.12 guide additional removal necessary to reach continuous rock. Where the elevation of continuous rock is below the elevation of the base of the foundation mat, fill concrete is placed between the continuous rock and the foundation mat. Fill concrete material meets the requirements for structural plain concrete as defined in DCD Subsection 2.5.4.6.3.

The design criteria used for static stability analyses are identified in Subsection 2.5.4.10 and are listed and compared to site parameters in Table 2.0-201. Discussion of assumptions, methods of analyses and conservatism in static stability analyses are included in Subsection 2.5.4.10.

Refer to Subsection 2.5.4.6 for Groundwater Level criteria, Subsection 2.5.4.8 for Liquefaction Potential and Subsection 2.5.4.7 for discussion of Shear Wave Velocity criteria.

Refer to Subsection 2.5.4.5.2.1 for slope stability design criteria.

Computer programs used in analyses were validated and verified by performing hand calculations to check computer output or by using published known solutions and running the program with the published inputs and comparing the computer output to the published solutions.

2.5.4.12 Techniques to Improve Subsurface Conditions

BLN COL 2.5-7

2.5.4.12.1 Introduction

The engineering challenges presented by karst in northern Alabama are well known and studied (Reference 463). The presence of karst features is known at the BLN site.

The rock at the foundation bearing level for the seismic Category I structures (nuclear island) is good quality, competent rock that is capable of supporting the structures with minor surface repairs to address local defects, if found. Information on the foundation configurations can be found in Section 3.8.5. Areas encountered during construction that require local improvement can be repaired using the techniques identified in the following subsections.

2.5.4.12.2 Mechanical Cleanup

Following completion of excavation to grade elevation loose, broken and displaced rock material is removed to the extent practical by mechanical means. Overhanging rock is removed. Weathered joints encountered are prepared as described in the following section. In the unlikely event that a large rock mass displaces along a discontinuity during blasting, evaluation on an individual basis is done to determine whether grouting, rock bolting/anchoring or removal is appropriate. The successful use of these repair and improvement methods during foundation construction of Bellefonte Units 1 and 2 is documented in Subsection 2.5.4.10 of the Bellefonte Units 1 and 2 FSAR (Reference 201)

2.5.4.12.3 Grouting and Concrete Dental Repair

Weathered discontinuities which are encountered during excavation of the foundation are cleaned a minimum of two times their width or if the joint widens with depth cleaned downward farther until a wedging effect can be achieved with fill concrete.

The rock properties used for bearing capacity and settlement analyses described in Subsection 2.5.4.10 were conservatively chosen, and include a reduction factor to account for blast damage to the rock during excavation. However, the rock mass properties can be improved by implementing a program of grouting to fill cracks formed, discontinuities widened, or stabilize rock blocks slightly displaced during blasting.

2.5.4.12.4 Rock Bolting

Rock bolting of selected rock blocks to prevent raveling or to stabilize large blocks loosened or slightly displaced during the excavation process is used when determined to be more appropriate than grouting. Details for grouting and rock bolting are provided in design criteria and construction specification documents.

2.5.4.12.5 Rock Anchors

Rock anchors are not expected; however, if used they are to be installed according to details provided in design criteria and construction specification documents.

2.5.4.12.6 Foundation Improvement Verification Program

Inspection and mapping of the completed excavations is accomplished through observation and examination by appropriately-qualified and trained project inspection personnel. Soundings, test holes, and similar measures are used to augment visual identification of areas needing repairs and to document that appropriate corrective measures have been completed. The quality assurance program in place during design, construction and operations phases is discussed in Section 17.5. Foundation improvement verification work will be conducted under that program. Milestones for implementation are not identified at this time because the construction planning has not yet designated milestones for this detailed activity.

2.5.5 STABILITY OF SLOPES

BLN COL 2.5-14 This section provides an evaluation of the stability of earth and rock slopes both natural and manmade whose failure could adversely affect the safety of the seismic Category 1 plant components. The plant design for BLN does not require a safety cooling, ultimate heat sink or related embankments. No safety related retaining walls, bulkheads, or jetties are required for the site.

BLN COL 2.5-15 No manmade earth or rock dams are present on the site that could adversely affect the safety of the nuclear power plant facilities. Potential failure of off-site dams is addressed in Subsection 2.4.4.

BLN COL 2.5-14 The plants are centrally sited on a broad, relatively level fill pad forming yard grade, as shown on Figure 2.5-362, and no natural or manmade slopes exist in proximity to the safety related nuclear islands that could pose a potential slope stability hazard to the safe operation of the plant. Additionally, no natural descending slopes, such as river banks or ridge slopes, exist around the perimeter of the BLN Units 3 and 4 plant yard area that could pose a potential encroachment or undermining hazard. As discussed in Subsection 2.5.4.8, the native residual soils, and fill, consist of clayey and/or compacted soils that are not prone to liquefaction. Therefore, a potential slope stability hazard does not exist under static or dynamic conditions that could adversely affect the Category 1 plant components.

Temporary cuts, below existing ground surface, are required for construction of the nuclear island basemat foundations. These cuts are backfilled up to the level plant grade, and will not pose a potential post-construction or operational slope stability hazard. This SAR section therefore presents a brief discussion of the permanent slopes, natural or manmade, while Subsection 2.5.4.5 briefly discusses the temporary slope stability of the construction cut slopes under static conditions.

2.5.5.1 Slope Characteristics

2.5.5.1.1 General Discussion

Based on the grades in the plant area as shown on Figure 2.4.2-202, no permanent cut slopes, or man-made fill slopes, exist that could compromise the operation of the safety-related plant facilities. The grading shown on Figure 2.5-362 of the BLN power block construction zone pad is generally level at about elevation 628.6 ft.for a minimum distance of over 500 ft.from the perimeter of the BLN nuclear islands. Fill slopes at the perimeter of the fill pad are limited in height to approximately 16 ft., and inclined at grades less than approximately 4:1 (horizontal to vertical). Existing graded or natural ground surface inclinations below or adjacent to the edge of the southwest, northwest, and northeast margins of the pad are relatively flat, and do not show evidence of past instability or potential unstable conditions as described in Subsection 2.5.4.1. The southeast margin of the pad extends to the toe of natural ridge slopes, a portion of which is steepened by excavation to extend the level pad southeastward. The steepest slope at the southeast pad margin is an 80-ft.high cut at an inclination of approximately 3:1 (horizontal to vertical). The toe of this cutslope is at least 950 ft. from the Unit 4 turbine building, and 1000 ft. from the Unit 4 nuclear island. The minimum separation distance between the plant and cutslope toe is over 10 times the slope height, providing a substantial safety buffer zone against possible slope failure under dynamic or static loading conditions. Therefore, this cut slope does not pose a potential safety hazard to the Unit 4 Category I Structures.

2.5.5.1.2 Exploration Program

Site investigations and subsurface geotechnical characterization used for the slope stability evaluation are presented in Subsections 2.5.4.1, 2.5.4.2, and 2.5.4.3. This information was used to evaluate possible slope stability hazards.

2.5.5.1.3 Groundwater and Seepage

A detailed discussion of groundwater conditions, including water levels and in situ rock mass transmissivity, is provided in Subsection 2.4.12. The groundwater characterization program included the installation of monitoring wells and performing pump tests on selected wells within the BLN Units 3 and 4 power block construction zone.

Pump tests described in Subsection 2.5.4.6 show that the rock is generally tight with low groundwater transmissivity. Groundwater occurs in and moves through joints and fractures in the rock mass, not through the intact rock. The excavation and dewatering will locally draw down the groundwater table around the excavation perimeter into the rock mass below the residual soil. Typical excavation dewatering procedures (e.g., sumps and pumps) will effectively control seepage during construction. Review of historic records indicate that similar procedures were effective during excavation for Units 1 and 2.

2.5.5.1.4 Slope Materials and Properties

Because the permanent slopes will not affect the seismic Category I structures and stability analyses were not performed, the selection of materials and properties was not necessary.

2.5.5.2 Design Criteria and Analyses

Because the permanent slopes do not affect the safety of the seismic Category I structures, design/performance criteria were not identified, and stability analyses were not performed.

2.5.5.3 Logs of Borings

The exploration program and the drilling and sampling procedures are discussed in Subsections 2.5.4.2 and 2.5.4.3. Boring logs of soil and rock borings in the vicinity of the excavations are included in Appendix 2BB and are discussed in Subsection 2.5.4.3. The boring logs provide the following information:

- Rock stratigraphy;
- Soil and rock engineering classification;
- Groundwater conditions;
- In situ rock mass condition and engineering properties (e.g., rock quality
- Designation, percent recovery, in situ modulus by Goodman jack testing, and rock discontinuities
- Soil in situ properties (Standard Penetration Test); and,
- Samples taken for laboratory geotechnical index and strength testing (soil and rock).

2.5.5.4 Compacted Fill

Specific sources of borrow material for the construction of the permanent fill slopes were not identified as part of the COL exploration. The on-site source for fill consists mainly of the nuclear island areas, as described in Subsection 2.5.4.5, and the cut indicated on Figure 2.4.2-202 at the southeast corner of the construction zone and from source areas identified previously in Subsection 2.5.4.5.3 of the FSAR (Reference 201). A discussion of use of onsite soils for backfill is presented in Subsection 2.5.4.5.

Subsection 2.5.4.5 contains the Quality Control and Quality Assurance requirements which in summary include engineering properties of the soil and rock materials, confirmation by laboratory testing, placement and compaction requirements, field density testing, monitoring, and record keeping.

STD DEP 1.1-1

2.5.6 COMBINED LICENSE INFORMATION

2.5.6.1 Basic Geologic and Seismic Information

BLN COL 2.5-1 This COL item is addressed in Subsections 2.5.1, 2.5.2.1, 2.5.4.1, and 2.5.4.6.

2.5.6.2 Site Seismic and Tectonic Characteristics Information

BLN COL 2.5-2 This COL item is addressed in Subsections 2.5.2 and 2.5.4.9.

2.5.6.3 Geoscience Parameters

BLN COL 2.5-3 This COL item is addressed in Subsections 2.5.2.6, 2.5.2.6.3, and 2.5.4.11.

2.5.6.4 Surface Faulting

BLN COL 2.5-4 This COL item is addressed in Subsection 2.5.3.

2.5.6.5 Site and Structures

BLN COL 2.5-5 This COL item is addressed in Subsections 2.5.4.1, 2.5.4.3.5, 2.5.4.2.2.4, 2.5.4.2.2.5, and 2.5.4.11.

2.5.6.6 Properties of Underlying Materials

BLN COL 2.5-6 This COL item is addressed in Subsections 2.5.4.2, 2.5.4.3, 2.5.4.4, 2.5.4.6.2, 2.5.4.7, 2.5.4.10.2, and 2.5.4.11.

2.5.6.7 Excavation and Backfill

BLN COL 2.5-7 This COL item is addressed in Subsections 2.5.4.5, 2.5.4.10.1, 2.5.4.10.5, and 2.5.4.12.

2.5.6.8 Groundwater Conditions

BLN COL 2.5-8 This COL item is addressed in Subsections 2.5.4.5.2.2 and 2.5.4.6.

BLN COL 2.5-9

2.5.6.9 Liquefaction Potential

This COL item is addressed in Subsection 2.5.4.8.

BLN COL 2.5-10

2.5.6.10 Bearing Capacity

This COL item is addressed in Subsections 2.5.4.10.1 and 2.5.4.11.

2.5.6.11 Earth Pressures

BLN COL 2.5-11 This COL item is addressed in Subsection 2.5.4.10.5.

2.5.6.12 Static and Dynamic Stability of Facilities

BLN COL 2.5-12 This COL item is addressed in Subsections 2.5.4.10.3 and 2.5.4.10.4.

2.5.6.13 Subsurface Instrumentation

BLN COL 2.5-13 This COL item is addressed in Subsection 2.5.4.5.5.

2.5.6.14 Stability of Slopes

BLN COL 2.5-14 This COL item is addressed in Subsection 2.5.5.

2.5.6.15 Embankments and Dams

BLN COL 2.5-15 This COL item is addressed in Subsection 2.5.5.

2.5.6.16 Settlement of Nuclear Island

BLN COL 2.5-16 This COL item is addressed in Subsection 2.5.4.10.4.

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TABLE 2.5-201
TIMING AND SOURCE OF LIQUEFACTION EVENTS IN SOUTHERN ATLANTIC COASTAL PLAIN

BLN COL 2.5-1

Liquification Episode	Age, Years B.P.	Scenario 1 Source	Scenario 1 Magnitude[a]	Scenario 2 Source	Scenario 2 Magnitude[a]
1886 AD	113	Charleston	7.3	Charleston	7.3
A	546±17	Charleston	7+	Charleston	7+
B	1021±30	Charleston	7+	Charleston	7+
C	1648±74	northern part	~6.0	-	-
C’	1683±70	-	-	Charleston	7+
D	1966±212	southern Part	~6.0	-	-
E	3548±66	Charleston	7+	Charleston	7+
F	5038±166	Northern Part	~6.0	Charleston	7+
G	5800±500	Charleston	7+	Charleston	7+

a) Magnitude is M_w; 1886 magnitude is from Johnston (Reference 297)
Source: Talwani and Schaeffer (Reference 317)

TABLE 2.5-202
ESTIMATED MAGNITUDES AND PEAK GROUND ACCELERATIONS OF PREHISTORIC EARTHQUAKE EPISODES IN SOUTH CAROLINA COASTAL PLAIN

BLN COL 2.5-1

	Estimated Magnitudes				Estimated Peak Ground Accelerations (g)	
	Talwani & Schaeffer (Reference 317)					
Episode	Empirical	Magnitude Bound	Hu *et al*. (Reference 319)	This Study	Hu et al. (Reference 319)	This Study
A	7+	7.0	7.4 to 7.6	6.2 to 7.0	0.16 to 0.18	0.14
B	7+	7.0	7.4 to 7.6	6.2 to 6.8	0.16 to 0.18	0.14 to 0.15
C	~6	6.3 to 6.8	6.3 to 7.0	5.1 to 6.4	0.21 to 0.28	0.20 to 0.29
C	7+	7.2	7.6 to 7.8	6.4 to 7.2	.016 to 0.17	0.14 to 0.15
D	~6	5.7			0.23 to 0.24	0.21 to 0.26
E	7+	7.0	6.8 to 7.0	5.6 to 6.4	0.31 to 0.42	0.30 to 0.53
F	~6		5.5 to 6.2	4.3 to 5.6	0.23 to 0.24	0.22 to 0.24
F’	7+		6.8 to 7.0	5.5 to 6.2		
G	7+	7.2				

Source: Leon et al. (Reference 320).

TABLE 2.5-203
COMPARISON OF SITE STRATIGRAPHIC UNITS, 1926 AND 1988

BLN COL 2.5-1

	1926 Geologic Map of Alabama	1988 Geologic Map of Alabama
Period	Unit	Unit
Silurian	Red Mountain Formation	Red Mountain Formation
Upper Ordovician	Chickamauga Limestone	Sequatchie Formation
Middle Ordovician		Nashville Group
		Stones River Group

TABLE 2.5-204
STRATIGRAPHIC COLUMN

BLN COL 2.5-1

AGE		Thickness	UNIT	DESCRIPTION
CENOZOIC	Quarternary	5 to 30 ft	Quaternary alluvium and colluvium.	Alluvium. Gravel, sand, silt, and clay deposited by streams along stream valleys. Colluvium: Poorly sorted weathered rock and soil deposited near the base of hillslopes by creep, slopewash, and landslides.
PALEOZOIC	Mississippian	200 ft [a]	Fort Payne Chert Maury Shale	Light gray, finely crystalline limestone and chert that occur in irregular beds and nodules. The base of the formation is marked by the Maury Formation, a greenish gray fissile shale.
PALEOZOIC	Devon-ian	20 ft [a]	Chattanooga Shale	Dark-colored organic shale with occasional sandstone beds near its base.
PALEOZOIC	Silurian	100 ft [a]	Red Mountain Formation	Interbedded siltstone, sandstone, and shale beds of varying color, and fossiliferous limestone, with few thin hematite beds
PALEOZOIC	Ordovician	240 ft [a]	Sequatchie Formation	Thin-bedded calcareous shale and mudstone with interbedded fossiliferous limestone and glauconitic bioclastic limestone. Includes the Leipers and Inman formations. The Fernvale limestone, a reddish sandy fossiliferous limestone, outcrops near the crest of River Ridge.
PALEOZOIC	Ordovician	270 ft [a]	Nashville Group	Medium- to dark-gray fine to medium-grained, crystalline, fossiliferous, locally silty and argillaceous, limestone. A 2.5 ft bed of bentonite (T3) marks the lower boundary.
PALEOZOIC	Ordovician	1050 ft [a]	Stones River Group	Medium- to dark-gray thick to thin-bedded fine-grained dense limestone, argillaceous and silty in part, locally fossiliferous or cherty. Contains a 70 ft zone of argillaceous and silty dolomitic limestone. Bentonite beds occur near the top.
PALEOZOIC	Cambrian to Ordovician	1476 to 4225 ft [b]	Knox Group	Dolomitic, siliceous, cherty limestones that are extensively weathered and are covered in the area with thick cherty, red clay residuum that developed in place. Sinkhole features are common in the outcrop belt.

a) Thicknesses are based on measurements made on site location cross sections constructed from borehole data and field outcrop mapping.

b) Thickness from Reference 321.

Notes: Double lines denote unconformities.

TABLE 2.5.205
BOREHOLE STRATIGRAPHY CORRELATION CHART

BLN COL 2.5-1

Lithology	Units 1 and 2 FSAR (Reference 201)	This Study (2007)		Vertical Thickness - ft	True Thickness - ft[(a)]
Limestone and shale, above the T3 bentonite	Unit 1 Chickamauga	Nashville		—	270
Limestone with argillaceous and silty interbeds	Unit 2 Chickamauga	Upper Stones River		110	106
Limestone with argillaceous and silty interbeds		Middle Stones River	Unit A	66	64
Limestone			Unit B	125	121
Argillaceous and silty dolomitic limestone	Unit 3 Chickamauga		Unit C	69	67
Limestone, with chert nodules in upper 25 ft.	Unit 4 Chickamauga		Unit D	138	133
Limestone with argillaceous and silty interbeds			Unit E	21	20
Limestone			Unit F	50	48
Limestone and dolomite, with argillaceous and silty interbeds		Lower Stones River		—	~490

a) True thickness = Vertical thickness · cos 15° (dip of beds)

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-206 (Sheet 1 of 2)
EARTHQUAKE CATALOGS FOR THE CENTRAL AND SOUTHEASTERN U.S. USED IN DEVELOPMENT OF THE TVA DAM SAFETY CATALOG

Catalog	Reporting Period	Minimum Magnitude (m_b or M)[a]	Comments
USGS (National Ground Motion Hazard Mapping Project)	1702 – 2001	3.0	Final independent catalog for CEUS (covers intermountain region and CEUS), from Chuck Mueller at USGS Denver. Documentation is Reference 331
ANSS (USGS/ NEIC)	1962 – present (March 1, 2005)	2.5	Entire U.S.; Includes all events from CERI (up through 2005) and SEUSSN (up through 2003) http://quake.geo.berkeley.edu/ anss/
SEUSSN	1698 – 2003	0.0	Southeastern U.S. http://www.geol.vt.edu/outreach/ vtso/
CERI	1974 – 2004 (January 1)	0.0	New Madrid Catalog; Central U.S. http:// folkworm.ceri.memphis.edu/
EPRI (Reference 203)	~1627 – 1985		Superceded by NCEER-91.
NCEER-91	1627 – 1985	3.0	Update of EPRI to eliminate non-tectonic events, prepare new magnitude estimates, etc.
NCEER-91 Update	1830 – 1906	3.0	Revisions to NCEER-91 from John Armbruster at Lamont Doherty. http://www.ldeo.columbia.edu/ ~armb/

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-206 (Sheet 2 of 2)
EARTHQUAKE CATALOGS FOR THE CENTRAL AND SOUTHEASTERN U.S. USED IN DEVELOPMENT OF THE TVA DAM SAFETY CATALOG

Catalog	Reporting Period	Minimum Magnitude (m_b or M)[a]	Comments
Johnston et al. Reference 268	~1700 – 1992	~4.0	Worldwide catalog. Used moment magnitude estimates for some events of M~4 to 5. No additional earthquakes.
Reinbold and Johnston Reference 409		~2.0	Appalachian region catalog. Additional earthquakes provided from this catalog by TVA.

a) The minimum magnitude indicates the minimum magnitude cut-off used when selecting data for each catalog (e.g., catalog search criteria), or the minimum magnitude of earthquakes in the catalog.

BLN COL 2.5-1
BLN COL 2.5-2

TABLE 2.5-207
NEW SEISMICITY DATA FOR THE CENTRAL AND SOUTHEASTERN U.S. USED IN DEVELOPMENT OF BLN EARTHQUAKE CATALOG

Catalog	Reporting Period	Minimum Magnitude (M)	Comments
Metzger et al. (2000) (Reference 336)	1826 – 1899	~3.3 (revised to 2.75)	New earthquakes and revisions to magnitudes (**M**) and locations of some earthquakes in the central U.S. (Reelfoot rift region). Some magnitudes and locations modified based on research by J. Munsey.
Bakun et al. (2003), Bakun and Hopper (2004) (References 410, 296 and 338)	1827 – 1938	3.7	Revised locations and magnitudes (**M**) for selected CEUS earthquakes. Some magnitudes and locations modified based on additional data reviewed for this project.
TVA (2005) (Reference 337)	1758 – 1923	2.6	New earthquakes identified for southeastern U.S. from available online newspaper and other sources. Data prepared by Jeff Munsey of TVA.
ANSS (USGS/ NEIC)	January 1, 2004 – March 1, 2005	2.5	Entire U.S.; Includes data from CERI and SEUSSN (?) http://quake.geo.berkeley.edu/anss/
CERI	January 1, 2004 – March 1, 2005	0.0	New Madrid Catalog; Central U.S. http://folkworm.ceri.memphis.edu/

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208A
BECHTEL SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
25	New York-Alabama Geopotential Lineament-Tennessee Segment	0.30	Yes
25A	New York-Alabama Geopotential Lineament	0.43	Yes
BZ0	New Madrid Region	1.00	Yes
BZ3	Northern Great Plains Region	1.00	Yes
BZ5	Southern Appalachians Region	1.00	Yes
BZ6	Southern Eastern Craton Region	1.00	Yes
27	Frankfort-Bucyrus Rift Zone	0.2	No
F	SE Appalachians	0.35	No
15	Rosman Fault	0.05	No
24	Bristol block Geopotential Trends	0.25	No
G	NW S. Carolina	0.35	No
32	Kentucky River Fault System	0.35	No
33	Rough Creek-Shawneetown Fault Zone	0.2	No
31	Reelfoot Rift	0.6	No
K	Southern Illinois	0.35	No
BZ4	Atlantic Coastal Region	1.00	No
	Sources Beyond 200-Mile Radius		
30	New Madrid	1.00	Yes
H	Charleston Area	0.5	No
N3	Charleston Faults	0.53	No

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208B
DAMES AND MOORE SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
4	Appalachian Fold Belt (mutually exclusive with 4A-4D)	0.35	No[(a)]
4A	Kink in zone that includes seismicity in Eastern Tennessee	0.65	No[(a)]
8	Eastern Marginal Basin (Default Source Zone for Zones 5, 6, and 7)	0.08	Yes
41	Southern Cratonic Margin (Default Source Zone for Zones 42, 43, and 46)	0.12	Yes
71	Indiana Illinois Block	0.05	Yes
10B	Default Zone (Default for Zones 10 and 11)	0.39	No
10	Nashville Dome	0.30	No
5	East Continent Gravity High	0.3	No
53	Southern Appalachian Mobile Belt	0.26	No
52	Charleston Mesozoic Rift	0.46	No
	Sources Beyond 200-Mile Radius		
21	New Madrid Compression Zone	0.75	Yes
54	Charleston Seismic Zone	0.70	Yes

a) This zone was not included in the EPRI (1989) (Reference 233) analysis. This source zone was a significant contributor in the Sequoyah and Watts Bar nuclear plant hazard results. The sensitivity of the results at the Bellefonte Site to inclusion of zones 4 and 4A in the analysis is discussed in Section 2.5.2.4.3.

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208C
LAW ENGINEERING SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
17	Eastern Basement	0.62	Yes
115	Indiana Block	1.00	Yes
217	Eastern Basement (Background)	0.38	No[(a)]
117	Mississippi Embayment (Background Zone)	1.00	No
1	East Continent Gravity High	0.32	No
8	Buried East Coast Mesozoic Basins	0.27	No
38 44 38 45	Mafic Plutons	0.43	No
107	Eastern Piedmont (Background Zone)	1.00	No
108	Brunswick (Background Zone)	1.00	No
	Sources Beyond 200-Mile Radius		
18	Postulated Faults in Reelfoot Rift	1.00	Yes
35	Charleston Seismic Zone	0.45	No

a) This zone was not included in the EPRI (1989) (Reference 233) analysis. The sensitivity of the results at the Bellefonte Site to inclusion of zone 217 in the analysis is discussed in Section 2.5.2.4.3.

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208D
RONDOUT ASSOCIATES SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
13	Southern NY-AL Lineament	1.0	Yes
25	Southern Appalachians	0.935	Yes
9	Eastern Tennessee	0.988	Yes
26	South Carolina Zone	1.0	Yes
50(C02)	Grenville Crust Background Source	1.0	Yes
5	East Continent Geophysical Anomaly	1.0	No
6	Central Tennessee	0.83	No
48	Tennessee/Illinois/Kentucky Lineament (TIKL) /Central Tennessee	0.874	No
52	Pre-Grenville Precambrian Craton (background)	1.0	No
49	Appalachian Crust (background)	1.0	No
27	Tennessee/Virginia Border	0.989	No
	Sources Beyond 200-Mile Radius		
1	New Madrid	1.0	Yes
24	Charleston, SC	1.0	No

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208E
WESTON GEOPHYSICAL SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
24	NY-AL Clingman Block	0.9	Yes
103	Southern Appalachian (background source)	1.0	Yes
106	South Central (background source)	1.0	No
104	Southern Coastal Plain (background source)	1.0	No
101	S. Ontario-Ohio-Indiana (background source)	1.0	No
26	South Carolina Seismic Zone (Part of 104)	0.86	No
	Sources Beyond 200-Mile Radius		
31	New Madrid	0.95	Yes
32	Reelfoot Rift Zone	1.0	Yes
25	Charleston South Carolina Seismic Zone	0.99	No

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-1

BLN COL 2.5-2

TABLE 2.5-208F
WOODWARD-CLYDE CONSULTANTS SEISMIC SOURCES

Source	Description	Probability of Activity (P_a)	Contributed to 99% of EPRI Hazard
	Sources Within 200-Mile Radius		
B39	Bellefonte Background Zone	1.0	Yes
31A	Blue Ridge (Combo 4,3 parts)	0.2	Yes
29	Central South Carolina Isostatic Gravity Saddle (Extended)	0.482	Yes
29A	Central South Carolina Isostatic Gravity Saddle (configuration #2)	0.482	
29B	Central South Carolina Isostatic Gravity Saddle (configuration #3)	0.436	
31	Blue Ridge (continuous)	0.2	No
44	New Madrid Loading Volume	0.7	No
	Sources Beyond 200-Mile Radius		
40	Central Disturbed Zone of the Reelfoot Rift	0.921	Yes
30	Ashley River and Woodstock Faults	0.438	No

Source: EPRI (1988) (Reference 203).

BLN COL 2.5-2

TABLE 2.5-209 (Sheet 1 of 3)
DESCRIPTION OF THE MINIMUM SET ZONES FOR THE LLNL - TIP STUDY

Earthquake Source Zone
1. General
Reference 234 presents six maps showing the source zones significant to Vogtle and eight showing the source zones for Watts Bar. The maps show the individual zone geometries and the spatial relationships among the zones. The maps are not intended to represent any particular source model scenarios (i.e., particular combinations of the zones); the scenarios are summarized in the logic trees presented in Reference 234.
A summary map showing the major source zone alternative boundaries is presented in Figure 2.5-241.
2. Charleston
• Zone IE is not shown. It coexists with IA and comprises two areas, which are coincident with the NE and SW areas of 1B
3. SC-GA Piedmont /Coastal Plain
• 3A and 3C are exclusive alternatives
3A-2 and 3A-3 represent fuzzy boundary of 3A. Possible combinations are: (3A-1) (3A-1) + (3A-2) (3A-I) + (3A-2) + (3A-3)
• 3B can exist without 3A or 3C
• 3B forms the background to 3A and 3C so the following combinations are possible: 3B 3A, (3B-3A) 3C, (3B-3C)
• Zone 7 forms the background to all Zone 3 alternatives and to Zone 6

TABLE 2.5-209 (Sheet 2 of 3)
DESCRIPTION OF THE MINIMUM SET ZONES FOR THE LLNL - TIP STUDY

Earthquake Source Zone
4. ETSZ There are five basic alternative zone definitions for the ETSZ, 4A, 4B, 4C, 4D, and 4E, all of which have the same overall bounding geometry as Zone 4A. • 4A-2 and 4A-3 represent a fuzzy boundary. Possible combinations are: (4A-I) + (4A-2) + (4A-3) (4A-1) + (4A-2) (4A-1) • Zone 4B is made up of two areas: the geometry of 4B-1 is identical to 4A-1 the geometry of 4B-2 is identical to (4A-2) + (4A-3) • possible combinations are: (4B-1) (4B-1) + (4B-2) • The geometry of Zone 4C is identical to (4A-1) + (4A-2) + (4A-3), within which the sources are defined as eight discrete faults • The geometry of Zone 4D is identical to (4A-1) + (4A-2) + (4A-3), within which the recurrence rate is inhomogeneous (rate spatial distribution determined by smoothing the seismicity map), rather than homogeneous as in each part of 4A, 4B, and 4E. • The bounding geometry of Zone 4E is identical to (4A-I) + (4A-2) + (4A-3), but has a graded boundary defined by three cylindrical sources (Bender).

TABLE 2.5-209 (Sheet 3 of 3)
DESCRIPTION OF THE MINIMUM SET ZONES FOR THE LLNL - TIP STUDY

Earthquake Source Zone
5. Appalachian/Central US • Zone 5 forms the background to the ETSZ, and comprises three areas. The alternative combinations are: (5-1), (5-2), (5-3) (5.1) + (5-2), (5-3) (5-1), (5-2) + (5-3) (5-1) - (5-2) + (5-3) • For all 4A alternative definitions for the ETSZ other than (4A-I) + (4A-2) + (4A-3) and for definition • (4B-1), seismicity in the remaining Zone 4 areas [(4A-2) or (4A-2) + (4A-3), (4B-2)] is included in Zone 5. • The Zone 5 alternatives can exist with or without a small, separate Giles County zone (not shown).

Source: Savy et al. (2002) (Reference 234).

TABLE 2.5-210 (Sheet 1 of 3)
EARTHQUAKE COUNTS FOR REGION WITHIN 200 MILES OF THE BELLEFONTE SITE

		Event Counts within 200 miles of BLN Site for Time Period:						
Magnitude Interval	Catalog	1625 – 780	1780 - 1860	1860 - 1910	1910 - 1950	1950 – 1975	1975 - March 1985	March 1985 - March 2005
Completeness Region 3								
3.3 m_b[a]< 3.9	P^{D}[a]	N/A[b]	N/A	0.182	0.489	0.76	1	
	EPRI	2	0	1	17	13	13	
	GG&S[c]	2	3	27	32	23	17	14
3.9 m_b[a]< 4.5	P^D	N/A	N/A	0.524	1	1	1	
	EPRI	0	1	5	14	11	2	
	GG&S	0	4	10	7	9	2	3
4.5 m_b[a]< 5.1	P^D	N/A	0.233	0.721	1	1	1	
	EPRI	0	1	1	2	3	0	
	GG&S	0	0	6	4	2	0	0

TABLE 2.5-210 (Sheet 2 of 3)
EARTHQUAKE COUNTS FOR REGION WITHIN 200 MILES OF THE BELLEFONTE SITE

		Event Counts within 200 miles of BLN Site for Time Period:						
Magnitude Interval	Catalog	1625 – 780	1780 - 1860	1860 - 1910	1910 - 1950	1950 – 1975	1975 - March 1985	March 1985 - March 2005
5.1 m_b[a]< 5.7	P^D	N/A	0.233	0.964	1	1	1	
	EPRI	0	0	0	2	0	0	
	GG&S	0	0	1	2	0	0	0
Completeness Region 4								
3.3 m_b[a]< 3.9	P^D	N/A	N/A	0.324	0.749	0.749	1	
	EPRI	0	0	0	3	0	1	
	GG&S	0	0	4	5	1	0	0
3.9 m_b[a]< 4.5	P^D	N/A	N/A	0.846	1	1	1	
	EPRI	0	1	1	2	0	0	
	GG&S	0	2	2	0	0	0	0

TABLE 2.5-210 (Sheet 3 of 3)
EARTHQUAKE COUNTS FOR REGION WITHIN 200 MILES OF THE BELLEFONTE SITE

		Event Counts within 200 miles of BLN Site for Time Period:						
Magnitude Interval	Catalog	1625 – 780	1780 - 1860	1860 - 1910	1910 - 1950	1950 – 1975	1975 - March 1985	March 1985 - March 2005
4.5 m_b[(a)]< 5.1	P^D	N/A	0.432	1	1	1	1	
	EPRI	0	1	1	0	0	0	
	GG&S	0	0	1	0	0	0	0
5.1 m_b[(a)]< 5.7	P^D	N/A	0.723	1	1	1	1	
	EPRI	0	0	0	0	0	0	
	GG&S	0	0	0	0	0	0	0

a) PD is probability of detection estimated in EPRI (1988) (Reference 203).

b) N/A catalog considered unusable for this time period in EPRI (1988) (Reference 203).

c) GG&S (Reference 399)

BLN COL 2.5-2

TABLE 2.5-211
VERIFICATION OF REPEATABILITY OF EPRI (1989) PSHA RESULTS

10 Hz hazard comparison

	Mean hazard comparison			Median hazard comparison			85% hazard comparison		
Amplitude	EPRI-SOG	REI		EPRI-SOG	REI		EPRI-SOG	REI	
(cm/sec)	hazard	hazard	% diff	hazard	hazard	% diff	hazard	hazard	% diff
1	1.32E-03	1.32E-03	-0.2%	7.66E-04	7.59E-04	-1.0%	2.83E-03	2.54E-03	-10.2%
5	5.63E-05	5.67E-05	0.6%	2.65E-05	2.66E-05	0.4%	1.06E-04	1.02E-04	-3.5%
10	8.54E-06	8.68E-06	1.7%	2.73E-06	2.82E-06	3.2%	1.31E-05	1.43E-05	9.1%
20	8.11E-07	8.30E-07	2.4%	1.13E-07	1.18E-07	4.0%	1.06E-06	1.07E-06	1.1%
30	1.56E-07	1.60E-07	2.8%	1.10E-08	1.08E-08	-1.5%	1.77E-07	1.78E-07	0.5%

1 Hz hazard comparison

Amplitude	EPRI-SOG	REI		EPRI-SOG	REI		EPRI-SOG	REI	
(cm/sec)	hazard	hazard	% diff	hazard	hazard	% diff	hazard	hazard	% diff
1	3.64E-03	3.69E-03	1.5%	1.42E-03	1.41E-03	-0.5%	9.52E-03	6.92E-03	-27.3%
5	3.77E-04	3.83E-04	1.5%	4.77E-05	4.96E-05	3.9%	1.07E-03	9.02E-04	-15.7%
10	1.16E-04	1.18E-04	1.5%	7.30E-06	7.67E-06	5.1%	3.49E-04	3.20E-04	-8.3%
20	2.29E-05	2.33E-05	1.6%	5.66E-07	5.75E-07	1.7%	6.14E-05	6.10E-05	-0.7%
40	2.51E-06	2.56E-06	2.2%	1.26E-08	1.59E-08	25.8%	4.12E-06	4.12E-06	0.0%

Note: “% diff” is based on more significant figures than are shown in table

Sources: EPRI - SOG (Reference 203); EPRI (1989) (Reference 233) REI (Risk Engineering, Inc.)

BLN COL 2.5-2

TABLE 2.5-212 (Sheet 1 of 3)
SOURCE CONTRIBUTIONS TO MEAN HAZARD IN EPRI SEISMIC HAZARD MODEL

	10 Hz Spectral Velocity				1 Hz Spectral Velocity			
				Bechtel				
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
25A[(a)]	4.23E-06	5.1	1.63E-07	2.7	1.05E-06	1.2	8.97E-08	1.3
25[(a)]	4.77E-06	5.8	1.81E-07	3.1	1.17E-06	1.3	9.90E-08	1.4
BZ6	3.04E-06	3.7	1.25E-07	2.1	1.14E-06	1.3	9.58E-08	1.3
BZ5	2.39E-05	28.9	1.19E-06	20.1	1.15E-05	12.8	7.64E-07	10.7
BZ3[(b)]	3.49E-05	42.1	4.04E-06	68.1	5.78E-06	6.4	7.24E-07	10.2
BZ0	9.23E-06	11.1	2.27E-07	3.8	9.11E-06	10.2	3.73E-07	5.2
30[(c)]	5.36E-07	0.6	1.83E-09	0	5.95E-05	66.4	4.89E-06	68.8
Total	8.28E-05	100	5.93E-06	100	8.96E-05	100	7.12E-06	100
				Dames and Moore				
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
71	7.95E-08	0.1	1.24E-09	0	1.37E-06	1.5	1.69E-07	1.4
54	2.07E-11	0	3.56E-15	0	1.65E-07	0.2	9.75E-10	0
41	4.25E-08	0.1	8.45E-10	0	6.40E-07	0.7	7.17E-08	0.6
21[(c)]	3.45E-07	0.6	1.05E-09	0	4.48E-05	49.5	3.30E-06	27.1
8	3.80E-07	0.6	3.47E-08	0.6	1.62E-07	0.2	3.65E-08	0.3
4A[(a)]	2.76E-05	46.1	1.89E-06	33.9	3.20E-05	35.3	5.89E-06	48.4
4[(b)]	3.08E-05	51.5	3.64E-06	65.4	1.12E-05	12.3	2.23E-06	18.3
Total	5.98E-05	100	5.57E-06	100	9.06E-05	100	1.22E-05	100

TABLE 2.5-212 (Sheet 2 of 3)
SOURCE CONTRIBUTIONS TO MEAN HAZARD IN EPRI
SEISMIC HAZARD MODEL

	10 Hz Spectral Velocity				1 Hz Spectral Velocity			
Law								
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
217	7.76E-07	2.1	9.52E-09	0.5	3.19E-08	0	8.43E-11	0
18[c]	3.97E-07	1.1	9.35E-10	0	6.30E-05	76.6	4.25E-06	67.1
17[a]	9.90E-06	27.3	5.63E-07	28.1	1.84E-05	22.4	1.96E-06	30.9
115[b]	2.51E-05	69.1	1.44E-06	71.6	7.07E-07	0.9	8.73E-09	0.1
Total	3.63E-05	100	2.01E-06	100	8.23E-05	100	6.34E-06	100
Rondout								
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
26	4.28E-08	0	1.28E-10	0	1.20E-06	0.8	2.34E-08	0.2
25[a]	7.21E-05	57	7.82E-06	52.1	3.77E-05	25.9	6.64E-06	45.8
13[b]	5.06E-05	40.1	7.15E-06	47.6	1.47E-05	10.1	2.08E-06	14.3
9	1.08E-06	0.9	1.20E-08	0.1	4.18E-06	2.9	2.39E-07	1.7
1[c]	3.89E-07	0.3	6.73E-10	0	8.70E-05	59.7	4.66E-06	32.1
50	8.38E-07	0.7	3.81E-08	0.3	4.28E-08	0	8.27E-10	0
Total	1.26E-04	100	1.50E-05	100	1.46E-04	100	1.45E-05	100
Woodward-Clyde								
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
40[c]	7.58E-07	0.8	6.89E-09	0.1	5.21E-05	54.6	5.08E-06	47
31A[a]	1.82E-05	19.9	1.31E-06	14.2	1.75E-05	18.3	2.84E-06	26.3
29A[d]	2.50E-08	0	1.78E-10	0	1.74E-06	1.8	1.24E-07	1.1
29[d]	5.05E-08	0.1	5.16E-10	0	1.89E-06	2.0	1.78E-07	1.6
B39[b]	7.63E-05	83.3	8.26E-06	89	2.62E-05	27.4	3.10E-06	28.7
Total	9.16E-05	100	9.28E-06	100	9.55E-05	100	1.08E-05	100

TABLE 2.5-212 (Sheet 3 of 3)
SOURCE CONTRIBUTIONS TO MEAN HAZARD IN EPRI SEISMIC HAZARD MODEL

	10 Hz Spectral Velocity				1 Hz Spectral Velocity			
				Weston				
Source	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**	**Mean 10^{-4}**	**%**	**Mean 10^{-5}**	**%**
32[(c)]	1.55E-08	0	2.05E-11	0	4.46E-06	4.6	2.00E-07	2.3
31[(c)]	1.22E-07	0.1	1.41E-10	0	3.77E-05	39.2	1.66E-06	19.3
32/C11	1.62E-08	0	1.83E-11	0	5.21E-06	5.4	1.78E-07	2.1
24[(a),(b)]	1.86E-04	92.3	2.11E-05	95.2	4.34E-05	45.2	5.71E-06	66.2
103/C19	7.63E-06	3.8	3.81E-07	1.7	3.18E-06	3.3	2.59E-07	3
103/C17	6.95E-06	3.5	6.83E-07	3.1	1.76E-06	1.8	1.95E-07	2.3
Total	2.01E-04	100	2.22E-05	100	9.61E-05	100	8.63E-06	100

a) East Tennessee seismic zone sources;

b) Host/background sources;

c) New Madrid sources;

d) Charleston sources

Source: EPRI-SOG (Reference 203)

TABLE 2.5-213
MAGNITUDE COMPARISONS FOR NEW MADRID 1811-1812 EARTHQUAKE SEQUENCE

Study	NM1	NM2	NM3
Johnston (1996) (Reference 213)	**M** 8.1 $\pm$ 0.3	**M** 7.8 $\pm$ 0.3	**M** 8.0 $\pm$ 0.3
Hough et al. (2000) (Reference 376)	**M** 7.2 to 7.3	**M** ~7.0[a] (located on the NN)	**M** 7.4 to 7.5
Mueller and Pujol (2001) (Reference 372)	-	-	**M** 7.2 to 7.4 (preferred **M** 7.2 to 7.3)
Bakun and Hopper (2004) (Reference 296)	**M** 7.6 (**M** 7.2 to 7.9) (preferred model 3)	**M** 7.5 (**M** 7.1 to 7.8) (preferred model 3)	**M** 7.8 (**M** 7.4 to 8.1) (preferred model 3)
	M 7.2 (**M** 6.8 to 7.9) (model 1)	**M** 7.2 (**M** 6.8 to 7.8) (model 1)	**M** 7.4 (**M** 7.0 to 8.1) (model 1)
Mueller et al. (2004) (Reference 377)	**M** 7.3	**M** 6.8 (located within the Wabash Valley of southern Illinois/ southern Indiana)	**M** 7.5
Johnston (Reference 381)	**M** 7.8-7.9	**M** 7.5-7.6	**M** 7.7-7.8

a) The estimated location and magnitude of this earthquake are revised in Mueller et al. (2004) (Reference 377) .

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TABLE 2.5-214
MAGNITUDE DISTRIBUTIONS FOR REPEATING LARGE MAGNITUDE NEW MADRID EARTHQUAKES

	Magnitude for Individual Faults (moment magnitude [M])			
Earthquake Rupture Set	New Madrid South	Reelfoot Thrust	New Madrid North	Weight
1	7.8	7.7	7.5	0.1667
2	7.9	7.8	7.6	0.1667
3	7.6	7.8	7.5	0.25
4	7.2	7.4	7.2	0.0833
5	7.2	7.4	7.0	0.1667
6	7.3	7.5	7.0	0.1667

BLN COL 2.5-2

TABLE 2.5-215 (Sheet 1 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
New Madrid Poisson	0.10108	160	6.26E-03
	0.24429	259	3.86E-03
	0.30926	407	2.46E-03
	0.24429	685	1.46E-03
	0.10108	1,515	6.60E-04
New Madrid Renewal, α = 0.3	0.10108	325	3.32E-03
	0.24429	401	9.96E-04
	0.30926	475	2.67E-04
	0.24429	562	4.98E-05
	0.10108	695	3.22E-06
New Madrid Renewal, α = 0.5	0.10108	310	4.87E-03
	0.24429	430	2.19E-03
	0.30926	559	8.81E-04
	0.24429	728	2.49E-04
	0.10108	1,008	2.72E-05
New Madrid Renewal, α = 0.7	0.10108	318	4.53E-03
	0.24429	494	2.28E-03
	0.30926	701	1.03E-03
	0.24429	986	3.35E-04
	0.10108	1,484	4.30E-05

TABLE 2.5-215 (Sheet 2 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 1 Poisson	0.10108	202	4.96E-03
	0.24429	298	3.36E-03
	0.30926	420	2.38E-03
	0.24429	625	1.60E-03
	0.10108	1,111	9.00E-04
Charleston Scenario 1 Renewal, $\alpha = 0.3$	0.10108	353	1.66E-04
	0.24429	418	2.59E-05
	0.30926	476	4.62E-06
	0.24429	541	6.35E-07
	0.10108	635	3.38E-08
Charleston Scenario 1 Renewal, $\alpha = 0.5$	0.10108	337	1.94E-03
	0.24429	435	6.73E-04
	0.30926	532	2.26E-04
	0.24429	650	5.79E-05
	0.10108	833	6.66E-06
Charleston Scenario 1 Renewal, $\alpha = 0.7$	0.10108	341	3.18E-03
	0.24429	479	1.44E-03
	0.30926	627	6.05E-04
	0.24429	817	1.93E-04
	0.10108	1,128	2.86E-05

TABLE 2.5-215 (Sheet 3 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 2 Poisson	0.10108	202	4.94E-03
	0.24429	311	3.22E-03
	0.30926	459	2.18E-03
	0.24429	714	1.40E-03
	0.10108	1,389	7.20E-04
Charleston Scenario 2 Renewal, $\alpha = 0.3$	0.10108	403	4.01E-05
	0.24429	480	4.09E-06
	0.30926	552	4.52E-07
	0.24429	634	3.49E-08
	0.10108	754	7.72E-10
Charleston Scenario 2 Renewal, $\alpha = 0.5$	0.10108	375	1.29E-03
	0.24429	499	3.29E-04
	0.30926	626	7.65E-05
	0.24429	783	1.21E-05
	0.10108	1,031	6.26E-07
Charleston Scenario 2 Renewal, $\alpha = 0.7$	0.10108	375	2.62E-03
	0.24429	553	9.36E-04
	0.30926	750	2.90E-04
	0.24429	1,010	5.92E-05
	0.10108	1,442	4.06E-06

TABLE 2.5-215 (Sheet 4 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 3 Poisson	0.10108	151	6.62E-03
	0.24429	245	4.08E-03
	0.30926	385	2.60E-03
	0.24429	649	1.54E-03
	0.10108	1,471	6.80E-04
Charleston Scenario 3 Renewal, $\alpha = 0.3$	0.10108	337	2.58E-04
	0.24429	418	2.59E-05
	0.30926	495	2.60E-06
	0.24429	586	1.57E-07
	0.10108	725	1.95E-09
Charleston Scenario 3 Renewal, $\alpha = 0.5$	0.10108	310	2.57E-03
	0.24429	437	6.59E-04
	0.30926	576	1.37E-04
	0.24429	756	1.66E-05
	0.10108	1,052	4.87E-07
Charleston Scenario 3 Renewal, $\alpha = 0.7$	0.10108	312	3.76E-03
	0.24429	495	1.31E-03
	0.30926	714	3.60E-04
	0.24429	1,018	5.63E-05
	0.10108	1,545	2.14E-06

TABLE 2.5-215 (Sheet 5 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 4 Poisson	0.10108	420	2.38E-03
	0.24429	575	1.74E-03
	0.30926	758	1.32E-03
	0.24429	1,020	9.80E-04
	0.10108	1,563	6.40E-04
Charleston Scenario 4 Renewal, $\alpha = 0.3$	0.10108	583	1.73E-07
	0.24429	658	1.64E-08
	0.30926	722	2.14E-09
	0.24429	791	2.36E-10
	0.10108	885	1.14E-11
Charleston Scenario 4 Renewal, $\alpha = 0.5$	0.10108	565	1.55E-04
	0.24429	680	4.07E-05
	0.30926	786	1.17E-05
	0.24429	907	2.76E-06
	0.10108	1,085	3.28E-07
Charleston Scenario 4 Renewal, $\alpha = 0.7$	0.10108	569	8.52E-04
	0.24429	731	3.25E-04
	0.30926	890	1.24E-04
	0.24429	1,080	3.84E-05
	0.10108	1,373	6.24E-06

TABLE 2.5-215 (Sheet 6 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 5 Poisson	0.10108	463	2.16E-03
	0.24429	649	1.54E-03
	0.30926	877	1.14E-03
	0.24429	1,191	8.40E-04
	0.10108	1,923	5.20E-04
Charleston Scenario 5 Renewal, $\alpha = 0.3$	0.10108	696	4.90E-09
	0.24429	783	3.05E-10
	0.30926	859	2.65E-11
	0.24429	942	1.81E-12
	0.10108	1,059	4.05E-14
Charleston Scenario 5 Renewal, $\alpha = 0.5$	0.10108	666	4.80E-05
	0.24429	807	9.08E-06
	0.30926	940	1.86E-06
	0.24429	1,093	2.98E-07
	0.10108	1,320	1.95E-08
Charleston Scenario 5 Renewal, $\alpha = 0.7$	0.10108	666	4.79E-04
	0.24429	869	1.41E-04
	0.30926	1,071	4.06E-05
	0.24429	1,316	8.89E-06
	0.10108	1,694	8.45E-07

TABLE 2.5-215 (Sheet 7 of 7)
EARTHQUAKE FREQUENCIES FOR REPEATING LARGE MAGNITUDE EARTHQUAKES

Recurrence Model	Weight	Mean Repeat Time (years)	Equivalent Annual Frequency
Charleston Scenario 6 Poisson	0.10108	463	2.16E-03
	0.24429	649	1.54E-03
	0.30926	877	1.14E-03
	0.24429	1,220	8.20E-04
	0.10108	1,923	5.20E-04
Charleston Scenario 6 Renewal, α = 0.3	0.10108	712	2.95E-09
	0.24429	801	1.71E-10
	0.30926	880	1.34E-11
	0.24429	967	8.05E-13
	0.10108	1,088	1.58E-14
Charleston Scenario 6 Renewal, α = 0.5	0.10108	682	3.98E-05
	0.24429	828	7.07E-06
	0.30926	965	1.38E-06
	0.24429	1,124	2.05E-07
	0.10108	1,360	1.21E-08
Charleston Scenario 6 Renewal, α = 0.7	0.10108	682	4.36E-04
	0.24429	893	1.21E-04
	0.30926	1,103	3.33E-05
	0.24429	1,358	6.85E-06
	0.10108	1,752	5.89E-07

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TABLE 2.5-216
MAGNITUDE COMPARISONS FOR 1886 CHARLESTON EARTHQUAKE IN CHARLESTON REGION

Reference Source	Approach for Magnitude Estimation	Weight for Approach	Magnitude	Assigned Weighting	Mean Magnitude (M)
Johnston, 1996 (Reference 297)	Felt Area for 1886 eq.; based on worldwide database	0.25	**M** 7.3 ± 0.26	0.185, 0.63, 0.185	7.3
Bollinger, 1977; Nuttli et al., 1979 (Reference 299 and 300)	Intensity distribution for 1886 eq.; based on U.S. data	0.2	m_b 6.75 ± 0.15 (~**M** 6.82 ± 0.22)[a]	0.185, 0.63, 0.185	6.8
Martin and Clough, 1994 (Reference 298)	Liquefaction data from 1886 eq.	0.1	**M** 7.25 ± 0.25	0.185, 0.63, 0.185	7.25
Bakun and Hopper, 2004 (Reference 296)	Intensity data for 1886 eq.; based on U.S. data	0.35	M_I 6.9[b]		6.9
Leon et al. (2005) (Reference 320)	Paleoliquefaction data from previous eqs. at/near Charleston[c]	0.1	M 7.0 ± 0.2[d]	0.185, 0.63, 0.185	7.0
				Weighted Mean	7.06
2002 USGS National Seismic Hazard Mapping Project (Reference 348)	Consideration of available magnitude estimates	--	**M** 6.9, 7.1, 7.3, 7.5	0.2, 0.2,0.45, 0.15	7.2

a) mb to M conversion based on Johnston (1996) (Reference 355) and Atkinson and Boore (1995) (Reference 354) (equal weight).

b) Mi – Intensity magnitude is considered equivalent to M (Bakun and Hopper, 2004). (Reference 296)

c) M – Magnitude based on magnitude bound method and Energy Stress method; assumed equal to M.

d) Magnitude based on magnitude estimates for largest paleoearthquakes at Charleston (1886 and Events A and C' in Leon et al., 2005) (Reference 320).

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TABLE 2.5-217
RECURRENCE SCENARIOS FOR CHARLESTON REPEATING LARGE MAGNITUDE EARTHQUAKES

			Recurrence Scenario 1 (0.2)[a]			Recurrence Scenario 2 (0.3)[b]			Recurrence Scenario 3 (0.5)[b]		
Paleo-Liquefaction Event[b]	Age (years before 1999 AD)[b]	Year of Event AD (BC)	Source	Magnitude (M)	Interval (years)[c]	Source	Magnitude (M)	Interval (years)[c]	Source	Magnitude (M)	Interval (years)[c]
1886 EQ	113	1886	Charleston	7.3[d]	>119	Charleston	7.3[d]	>119	Charleston	7.3[d]	>119
A	546 ± 17	1453	Charleston	7+	433	Charleston	7+	433	Charleston	7+	433
B	1021 ± 30	978	Charleston	7+	475	Charleston	7+	475	Charleston	7+	475
C	1648 ± 74	351	Charleston	7+	627	--	--	--	Northern	6+	?
C[1]	1683 ± 70	316	--	--	--	Charleston	7+	662	--	--	--
D	1966 ± 212	33	Charleston	7+	318	--	--	--	Southern	6+	?
E	3548 ± 66	(1549)	Charleston	7+	1582	Charleston	7+	1865	Charleston	7+	2527
F	5038 ± 166	(3039)	Charleston	7+	1490	Charleston	7+	1490	Northern	6+	?
G	5800 ± 500	(3801)	Charleston	7+	762	Charleston	7+	762	Charleston	7+	2252

a) Recurrence Scenario 1 developed by Geomatrix (2004) (Reference 269).

b) Data and recurrence scenarios 2 and 3 are from Talwani and Schaeffer (2001) (Reference 317).

c) Recurrence interval is for large magnitude earthquakes on Charleston earthquake source.

d) 1886 magnitude from Johnston (1996) (Reference 297).

TABLE 2.5-218
CONTROLLING EARTHQUAKES

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Struct. frequency	Annual Freq. Exceed.	Overall hazard		Hazard from R<100 km		Hazard from R>100 km	
		M	R, km	M	R, km	M	R, km
1 & 2.5 Hz	1E-4	7.5	330	5.9	23	7.7	360
5 & 10 Hz	1E-4	7.0	260	5.6	22	7.7	360
1 & 2.5 Hz	1E-5	7.4	290	6.0	18	7.7	360
5 & 10 Hz	1E-5	6.0	91	5.6	14	7.7	360
1 & 2.5 Hz	1E-6	7.1	220	6.2	15	7.8	360
5 & 10 Hz	1E-6	5.8	31	5.7	12	8.1	324
Nominal values chosen:				5.9	20	7.7	360

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TABLE 2.5-219 (Sheet 1 of 2)
MEAN UHRS AMPLITUDES FOR 10^{-4}, 10^{-5}, AND 10^{-6}

Freq	1E-4	1E-5	1E-6
100	0.138	0.523	1.476
90	0.151	0.579	1.641
80	0.174	0.673	1.916
70	0.209	0.816	2.335
60	0.253	1.002	2.882
50	0.297	1.192	3.448
45	0.316	1.274	3.697
40	0.330	1.343	3.908
35	0.340	1.396	4.078
30	0.346	1.434	4.205
25	0.346	1.451	4.275
20	0.342	1.352	3.881
15	0.323	1.184	3.276
12.5	0.304	1.063	2.866
10	0.276	0.911	2.374
9	0.272	0.864	2.222
8	0.265	0.811	2.055
7	0.255	0.751	1.872
6	0.243	0.684	1.674
5	0.227	0.610	1.459
4	0.202	0.512	1.227
3	0.177	0.433	0.962
2.5	0.161	0.393	0.826
2	0.144	0.362	0.750
1.5	0.120	0.311	0.635
1.25	0.104	0.274	0.555

TABLE 2.5-219 (Sheet 2 of 2)
MEAN UHRS AMPLITUDES FOR 10^{-4}, 10^{-5}, AND 10^{-6}

Freq	1E-4	1E-5	1E-6
1	0.0843	0.228	0.456
0.9	0.0796	0.226	0.457
0.8	0.0739	0.220	0.449
0.7	0.0674	0.210	0.432
0.6	0.0601	0.195	0.404
0.5	0.0520	0.176	0.367
0.4	0.0393	0.133	0.278
0.3	0.0272	0.0922	0.192
0.2	0.0157	0.0532	0.111
0.15	0.0103	0.0348	0.0727
0.125	0.00772	0.0261	0.0545
0.1	0.00531	0.0180	0.0375

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TABLE 2.5-220 (Sheet 1 of 2)
COMPUTATION OF HORIZONTAL GMRS

Freq	10^{-4} envelope	10^{-5} envelope	AR	DF	GMRS
100	0.138	0.523	3.787	1.741	0.240
90	0.151	0.579	3.840	1.761	0.265
80	0.174	0.673	3.876	1.774	0.308
70	0.209	0.816	3.916	1.788	0.373
60	0.253	1.002	3.960	1.804	0.457
50	0.297	1.192	4.011	1.823	0.542
45	0.316	1.274	4.039	1.833	0.578
40	0.330	1.343	4.071	1.844	0.608
35	0.340	1.396	4.105	1.857	0.632
30	0.346	1.434	4.144	1.871	0.647
25	0.346	1.451	4.189	1.887	0.654
20	0.342	1.352	3.951	1.801	0.616
15	0.323	1.184	3.665	1.696	0.548
12.5	0.304	1.063	3.494	1.632	0.497
10	0.276	0.911	3.296	1.558	0.431
9	0.272	0.864	3.180	1.514	0.411
8	0.265	0.811	3.062	1.469	0.389
7	0.255	0.751	2.943	1.423	0.363
6	0.243	0.684	2.819	1.375	0.334
5	0.227	0.610	2.691	1.325	0.300
4	0.202	0.512	2.537	1.264	0.255

TABLE 2.5-220 (Sheet 2 of 2)
COMPUTATION OF HORIZONTAL GMRS

Freq	10^{-4} envelope	10^{-5} envelope	AR	DF	GMRS
3	0.177	0.433	2.449	1.228	0.217
2.5	0.161	0.393	2.449	1.228	0.197
2	0.144	0.362	2.515	1.255	0.181
1.5	0.120	0.311	2.596	1.287	0.154
1.25	0.104	0.274	2.645	1.306	0.135
1	0.0843	0.228	2.702	1.329	0.112
0.9	0.0796	0.226	2.839	1.383	0.110
0.8	0.0739	0.220	2.975	1.435	0.106
0.7	0.0674	0.210	3.111	1.487	0.100
0.6	0.0601	0.195	3.246	1.539	0.0925
0.5	0.0520	0.176	3.385	1.591	0.0827
0.4	0.0393	0.133	3.385	1.591	0.0626
0.3	0.0272	0.0922	3.385	1.591	0.0433
0.2	0.0157	0.0532	3.385	1.591	0.0250
0.15	0.0103	0.0348	3.385	1.591	0.0164
0.125	0.00772	0.0261	3.385	1.591	0.0123
0.1	0.00531	0.0180	3.385	1.591	0.00844

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TABLE 2.5-221 (Sheet 1 of 2)
COMPUTATION OF VERTICAL GMRS

Freq	V/H factor PGA<0.2g	Vert 10^{-4}	V/H factor PGA 0.2 to 0.5g	Vert 10^{-5}	AR	DF	GMRS
100	0.78	0.108	1.00	0.523	4.856	2.124	0.235
90	0.82	0.124	1.02	0.601	4.835	2.117	0.270
80	0.87	0.150	1.09	0.734	4.879	2.132	0.330
70	0.89	0.186	1.13	0.921	4.940	2.153	0.414
60	0.89	0.226	1.14	1.140	5.044	2.190	0.513
50	0.86	0.256	1.12	1.341	5.238	2.257	0.603
45	0.85	0.267	1.10	1.405	5.260	2.264	0.632
40	0.83	0.273	1.04	1.399	5.123	2.217	0.630
35	0.79	0.270	0.98	1.369	5.067	2.198	0.616
30	0.77	0.265	0.94	1.343	5.066	2.197	0.604
25	0.75	0.260	0.88	1.277	4.915	2.145	0.575
20	0.71	0.244	0.83	1.116	4.583	2.028	0.502
15	0.69	0.223	0.79	0.933	4.191	1.888	0.420
12.5	0.68	0.207	0.77	0.819	3.958	1.803	0.373
10	0.67	0.185	0.75	0.683	3.689	1.705	0.316
9	0.67	0.182	0.75	0.648	3.560	1.657	0.302
8	0.67	0.177	0.75	0.608	3.428	1.608	0.285
7	0.67	0.171	0.75	0.563	3.294	1.557	0.266
6	0.67	0.163	0.75	0.513	3.156	1.505	0.245
5	0.67	0.152	0.75	0.457	3.012	1.450	0.220
4	0.67	0.135	0.75	0.384	2.840	1.383	0.187

TABLE 2.5-221 (Sheet 2 of 2)
COMPUTATION OF VERTICAL GMRS

Freq	V/H factor PGA<0.2g	Vert 10^{-4}	V/H factor PGA 0.2 to 0.5g	Vert 10^{-5}	AR	DF	GMRS
3	0.67	0.118	0.75	0.325	2.741	1.344	0.159
2.5	0.67	0.108	0.75	0.295	2.741	1.344	0.145
2	0.67	0.0964	0.75	0.271	2.816	1.373	0.132
1.5	0.67	0.0803	0.75	0.233	2.906	1.409	0.113
1.25	0.67	0.0694	0.75	0.206	2.960	1.430	0.0993
1	0.67	0.0565	0.75	0.171	3.024	1.454	0.0821
0.9	0.67	0.0533	0.75	0.169	3.178	1.513	0.0807
0.8	0.67	0.0495	0.75	0.165	3.331	1.571	0.0778
0.7	0.67	0.0452	0.75	0.157	3.482	1.628	0.0735
0.6	0.67	0.0403	0.75	0.146	3.634	1.684	0.0678
0.5	0.67	0.0348	0.75	0.132	3.789	1.742	0.0606
0.4	0.67	0.0264	0.75	0.100	3.789	1.742	0.0459
0.3	0.67	0.0182	0.75	0.0691	3.789	1.742	0.0318
0.2	0.67	0.0105	0.75	0.0399	3.789	1.742	0.0183
0.15	0.67	0.00689	0.75	0.0261	3.789	1.742	0.0120
0.125	0.67	0.00517	0.75	0.0196	3.789	1.742	0.00901
0.1	0.67	0.00355	0.75	0.0135	3.789	1.742	0.00619

BLN COL 2.5-4

TABLE 2.5-222 (Sheet 1 of 3)
LINEAMENT DESCRIPTIONS

No.	Type	Description
1	Linear valley	The axis of a broad linear valley identified from topographic contours and drainages oriented approximately N 5 ^{o}W. One 1500-ft. reach of the stream that drains this valley is strongly linear and prominent on aerial photography. The stream may have been artificially straightened at one time, but the stream has now carved small meander loops along the straight path.
2	Paleo-valley	A major lineament crossing strike, it curves through the southwestern gap in River Ridge, extends northwestward through a broad trough in the valley, then through the northwest trending arm of Town Creek.
3	Linear valley along strike	The axis of a strike-parallel linear valley located between the southwest front of River Ridge and the adjacent parallel low ridge. This valley is bright pink on the 1973 CIR photo, indicating vigorous growth of vegetation.
4	Linear valley along strike	The axis of a strike-parallel linear valley located beneath proposed reactor unit 4. A stream is visible in the valley on 1973 air photos. Lineament approx. 850-ft. in length.
5	Break-in-slope	The break-in-slope at the southwest front of River Ridge, parallel to strike.
6	Linear valley along strike	A linear valley, strike-parallel, similar to lineament 3 that lies between the southwest front of River Ridge and the adjacent parallel low ridge.
7	Linear valley	A strike-parallel lineament following the alignment of four saddles on adjacent low ridges.
8	Linear valley	A short lineament along the axis of a small linear valley oriented approximately N-S.
9	Linear valley	A short lineament along the axis of a small linear valley oriented approximately N-S.
10	Linear valley	A linear valley trending N 15 ^{o}W.

BLN COL 2.5-4

TABLE 2.5-222 (Sheet 2 of 3)
LINEAMENT DESCRIPTIONS

No.	Type	Description
11	Linear valley	A linear valley trending N 25 ^{o}E.
12	Linear valley	A linear valley trending N 25 ^{o}E. This lineament projects toward Reactor Unit 3.
13	Linear valley	A linear valley trending N 25 ^{o}E.
14	Paleo-valley	This major lineament curves across strike, following a low trough in the topography that begins at the gap in River Ridge near the intake channel, then continues northeastward through a reentrant in the Town Creek Embayment.
15	Linear valley	Linear valley across strike draining the north flank of River Ridge
16	Linear valley & gap	Linear valleys and gap cutting across strike through River Ridge
17	Linear valley & gap	Linear valleys and gap cutting across strike through River Ridge
18	Saddle alignment	Follows the alignment of four saddles on adjacent knobs on the northeast flank of River Ridge.
19	Break-in-slope	The break-in-slope at the southwest front of River Ridge in the intake channel area.
20	Linear valley & gap	Linear valley and gaps cutting across strike through River Ridge
21	Break-in-slope	The break-in-slope at the southwest front of River Ridge north of the intake channel.

BLN COL 2.5-4

TABLE 2.5-222 (Sheet 3 of 3)
LINEAMENT DESCRIPTIONS

No.	Type	Description
22	Line of seeps	Prominent line of seeps and wet ground along strike, visible on 1973 CIR photography. This lineament was identified at the southern site on black and white aerial photography. It was correlated with the “Western Anomaly Zone,” a linear zone of deep weathering identified by seismic refraction surveys. The lineament extends northeast through the north cooling tower, and appears to follow the upper contact of the argillaceous limestone unit, Osra or unit C.
23	Tonal	Tonal lineament visible on aerial photography, strike-parallel.
24	Tonal	Line of channel initiation points along the flank of a ridge, strike-parallel. Several small channels begin at a similar point.
25	Tonal	Tonal lineament visible on aerial photography, strike-parallel.
26	Tonal	Tonal lineament visible on aerial photography, strike-parallel.
27	Tonal	Tonal lineament visible on aerial photography, strike-parallel.
28	Tonal	Tonal lineament visible on aerial photography, strike-parallel.
29	Tonal	Tonal lineament visible on aerial photography, strike-parallel.

Note: See Figure 2.5-291 for mapped location of these lineaments.

BLN COL 2.5-4

TABLE 2.5-223 (Sheet 1 of 5)
SEISMICITY IN NORTHEASTERN ALABAMA AREA

State	Year	Latitude	Longitude	Depth (km)	Magnitude	Type*
GA	1981	34.5730019	-85.4349976	8.6	2.1	2
AL	1981	34.8250008	-85.8130035	6.3	1.6	2
TN	1982	35.1790009	-86.4290009	12.9	2.9	2
GA	1982	34.5750008	-85.4449997	6.5	2.5	2
TN	1982	35.0680008	-85.4459991		2.0	2
GA	1983	34.9150009	-85.5260010		2.3	2
GA	1983	34.9620018	-85.5120010		2.1	2
GA	1984	34.6619987	-85.3899994	10.9	2.1	2
AL	1984	34.6020012	-86.3040009	15.4	2.9	2
AL	1984	34.6059990	-86.3030014	8.6	1.0	2
AL	1984	34.7649994	-86.0350037	7.6	1.3	2
AL	1985	34.9420013	-86.1740036	9.6	0.9	2
AL	1985	34.9710007	-85.6740036	7.0	1.1	2
GA	1986	34.6759987	-85.4300003	13.0	0.0	2
TN	1986	35.1870003	-85.5100021	27.3	3.0	2
AL	1986	34.3019981	-85.4860001	18.7	1.5	2
AL	1986	34.3889999	-85.5380020	10.3	1.5	2
AL	1986	34.7410011	-85.9970016	17.1	1.8	2
AL	1987	34.8180008	-86.3160019	9.6	2.4	2
GA	1987	34.7330017	-85.3330002	9.9	2.4	2
GA	1987	34.5499992	-85.3320007	9.8	0.0	2
GA	1987	34.7599983	-85.3690033	9.5	0.5	2
GA	1988	34.5789986	-85.4660034	10.1	0.3	2
TN	1988	34.3050003	-85.4820023	13.8	1.4	2
TN	1988	35.0219994	-86.3249969	11.0	2.0	2
GA	1988	34.4970016	-85.5000000	11.9	1.2	2

TABLE 2.5-223 (Sheet 2 of 5)
BLN COL 2.5-4 SEISMICITY IN NORTHEASTERN ALABAMA AREA

State	Year	Latitude	Longitude	Depth (km)	Magnitude	Type*
AL	1988	34.3569984	-85.5070038	13.1	2.1	2
AL	1989	34.7589989	-86.4029999	17.9	1.3	2
GA	1989	34.3930016	-85.4039993	9.2	1.4	2
AL	1989	34.8310013	-85.9869995	22.4	0.8	2
GA	1989	34.4780006	-85.4380035	14.7	2.3	2
AL	1989	34.6689987	-86.0820007	7.7	1.6	2
AL	1989	34.3860016	-85.5339966	11.8	1.6	2
GA	1989	34.2330017	-85.4869995	10.6	1.4	2
GA	1990	34.4659996	-85.5210037	12.4	2.5	2
GA	1990	34.8429985	-85.3450012	10.8	1.8	2
AL	1990	34.3919983	-85.4950027	8.1	2.1	2
GA	1990	34.5390015	-85.4909973	6.0	2.9	2
AL	1990	34.4860001	-86.4039993	11.0	1.9	2
AL	1991	34.6269989	-86.2429962	12.7	1.8	2
GA	1991	34.9140015	-85.4830017	21.4	2.7	2
GA	1991	34.4930000	-85.4599991	8.4	2.0	2
GA	1991	34.7500000	-85.3649979	1.9	1.9	2
AL	1992	34.8699989	-86.3509979	8.2	2.1	2
GA	1992	34.5880013	-85.4430008	12.9	2.0	2
GA	1992	34.7630005	-85.3300018	7.4	1.0	2
TN	1992	35.1220016	-85.5479965	7.4	1.5	2
TN	1993	35.0299988	-85.5169983	12.6	1.6	2
TN	1993	35.0639992	-85.4459991	18.8	1.6	2
GA	1994	34.9329987	-85.4769974	11.9	2.3	2
GA	1994	34.9389992	-85.4970016	4.8	1.9	2
GA	1994	34.9690018	-85.4909973	24.3	3.2	1

BLN COL 2.5-4

TABLE 2.5-223 (Sheet 3 of 5)
SEISMICITY IN NORTHEASTERN ALABAMA AREA

State	Year	Latitude	Longitude	Depth (km)	Magnitude	Type*
GA	1994	34.9300003	-85.4779968	9.5	1.5	2
AL	1994	34.9580002	-86.1429977	8.8	2.3	2
AL	1994	34.5559998	-86.2919998	15.2	0.8	2
TN	1994	35.0620003	-85.4499969	13.9	0.7	2
TN	1994	34.7080002	-85.4739990	5.1	2.6	2
AL	1994	34.9290009	-85.7740021	10.6	1.2	2
AL	1997	34.6220016	-85.3529968	2.7	2.9	1
GA	1997	34.6049995	-85.3639984	5.4	1.6	2
AL	1997	34.9230003	-85.9879990		1.7	2
AL	1997	34.5779991	-85.9369965		2.3	2
AL	1997	34.5209999	-85.6539993	2.3	1.6	2
AL	1997	34.5330009	-85.6930008	8.2	0.8	2
AL	1997	34.5050011	-85.6279984	10.7	0.6	2
GA	1997	34.3940010	-85.4270020		2.0	2
AL	1997	34.5180016	-85.5899963	1.9	1.7	2
GA	1997	34.4710007	-85.5059967		2.4	2
AL	1998	34.4249992	-85.5540009		2.5	2
TN	1998	35.1230011	-85.7539978	6.7	1.8	2
AL	1998	34.4239998	-85.5640030		2.1	2
TN	1998	35.1629982	-85.7949982	5.0	1.7	2
AL	1998	34.6580009	-86.1610031		2.0	2
TN	1998	34.9959984	-86.2620010	6.2	1.5	2
AL	1998	34.6269989	-86.0950012		1.7	2
GA	1998	34.5900002	-85.4520035	2.8	2.2	2
TN	1999	35.0519981	-86.5009995	9.1	1.9	2
GA	1999	34.9589996	-85.4120026	0.6	1.9	2

TABLE 2.5-223 (Sheet 4 of 5)
BLN COL 2.5-4 SEISMICITY IN NORTHEASTERN ALABAMA AREA

State	Year	Latitude	Longitude	Depth (km)	Magnitude	Type*
TN	1999	35.1629982	-85.5709991	7.9	1.4	2
TN	1999	35.1819992	-85.3949966	15.8	1.7	2
GA	2999	34.848999	-85.3410034	12.1	2.2	2
TN	2000	35.0800018	-86.3610001		2.4	2
TN	2000	35.0489998	-86.3560028	8.6	2.0	2
TN	2000	34.6850014	-85.3619995	3.9	2.8	2
GA	2000	34.6940002	-85.3929977	3.5	1.7	2
GA	2000	34.6269989	-85.3949966	2.9	1.4	2
TN	2000	35.0519981	-85.5220032	18.5	1.7	2
TN	2000	35.132	-85.7350006	5.5	2.6	2
TN	2000	35.0270004	-85.7040024	3.6	2.0	2
TN	2000	35.0400009	-85.8239975	11.9	0.9	2
GA	2001	34.7480011	-85.4449997	2.4	1.7	2
AL	2001	34.4700012	-86.3450012	16.8	1.6	2
AL	2001	34.8470001	-85.4380035		3.2	2
AL	2001	34.7109985	-86.1589966	26.4	1.5	2
AL	2001	34.8619995	-85.8850021		2.3	2
GA	2001	34.5449982	-85.4599991	2.4	1.9	2
AL	2001	34.493	-86.1849976	10.6	1.7	2
GA	2001	34.6189995	-85.3499985	5.4	1.9	2
AL	2001	34.7099991	-86.2310028		3.9	1
GA	2002	34.5979996	-85.4759979	6.9	1.8	2
AL	1003	34.7290001	-86.2480011	15.4	1.9	2
AL	2003	34.4449997	-85.6200027	9.1	4.6	1
AL	2003	34.5620003	-85.6490021	3.1	1.9	2
AL	2003	34.4620018	-85.612999	1.9	2.5	2

TABLE 2.5-223 (Sheet 5 of 5)
BLN COL 2.5-4
SEISMICITY IN NORTHEASTERN ALABAMA AREA

State	Year	Latitude	Longitude	Depth (km)	Magnitude	Type*
AL	2003	34.480999	-85.6399994	12.3	2.5	2
AL	2003	34.4510002	-86.0579987	14.8	2.0	2
AL	2003	34.2529984	-85.822998	7.8	2.0	2
AL	2004	34.3610001	-85.5159988	9.7	2.0	2
GA	2004	34.6020012	-85.4540024		2.4	2
AL	2004	34.4710007	-85.6470032	6.1	2.2	2
GA	2004	34.6899986	-85.4189987	10.0	2.5	2
AL	2004	34.9350014	-86.1080017	8.7	2.0	2
TN	2005	35.1529999	-85.6429977	10.3	1.7	2
GA	2005	34.5740013	-85.5080032	16.4	1.7	2
GA	2005	34.4980011	-85.4990005	11.2	2.1	2
GA	2006	34.9370003	-85.4609985	14.2	2.9	2
GA	2006	34.6240005	-85.5080032	8.0	2.2	2
TN	2006	35.1710014	-85.4300003	11.0	1.1	2

Notes:

Full instrumental seismicity catalog from Virginia Tech Seismological Observatory (Reference 214) for area bounded by 34.2°N, 35.2°N, 85.3°W, and 86.3°W. These data are plotted on the Figure 2.5-294 "Seismicity Relative to Mapped Faults."

*Magnitude types:

1. mb from Lg wave data (Nuttli, Reference 418)
2. Md from duration or coda length
3. Other (eg. derived from intensity data, felt area, or unknown instrument)

BLN COL 2.5-1
BLN COL 2.5-6

TABLE 2.5-224 (Sheet 1 of 4)
PETROGRAPHIC AND MINERALOGIC RESULTS

Sample no.	Lithologic unit	Field description	Petrography visual estimate (%)					TGA analysis (%)				X-ray diffraction (order of peak height)					
			Calcite	Dolomite (Mg,Fe)	Quartz	Clay	Other	Sample no.	Calcite	Dolomite (Mg,Fe)	Calcite + Dolomite	Calcite	Dolomite (Mg,Fe)	Quartz	Muscovite	Kaolinite	Pyrite
BLN-1	LSR	Micrite+wavy shale laminae	2	91	<1	3	4	--									
BLN-2	F	Micrite+wavy shale laminae	90	5	0	1	4	BLN-2L	88	0	88						
								BLN-2S	37	41	77	1	2	3	4		5
BLN-3	E	Wackestone+ micrite	94	1	<1	2	3	BLN-3L	78	0	78						
								BLN-3S	18	50	68	2	1	3	5		4
BLN-4	D	Micrite+wavy shale laminae	89	<1	0	3	8	--									
BLN-5	D	Micrite+wavy shale laminae	75	23	0	0	2	BLN-5L	86	0	86						
BLN-6	D	Micrite+wavy shale lam	90	6	0	2	2	BLN-6L	92	0	92						
BLN-7	D	Micrite+wavy shale laminae	73	20	3	0	4	--									

BLN COL 2.5-1
BLN COL 2.5-6

TABLE 2.5-224 (Sheet 2 of 4)
PETROGRAPHIC AND MINERALOGIC RESULTS

Sample no.	Lithologic unit	Field description	Petrography visual estimate (%)					TGA analysis (%)				X-ray diffraction (order of peak height)					
			Calcite	Dolomite (Mg,Fe)	Quartz	Clay	Other	Sample no.	Calcite	Dolomite (Mg,Fe)	Calcite + Dolomite	Calcite	Dolomite (Mg,Fe)	Quartz	Muscovite	Kaolinite	Pyrite
BLN-8	D	Micrite+wavy shale+chert	40	17	40	0	3	--									
BLN-9	C	Limestone +shale bands	93	4	<1	1	2	--									
BLN-10	C	Shale	66	15	3	15	1	BLN-10S	22	22	44	3	2	1	4	5	6
BLN-11	B	Packstone	90	6	0	1	3	--									
BLN-12	D	Micrite+wavy shale+ chert	75	20	0	1	4	--									
BLN-13	C	Micrite+shale beds	77	20	0	0	3	--									
BLN-14	C	Micrite+shale beds	68	0	8	20	4	BLN-14S	14	18	32	3	2	1	4	5	6
								BLN-14L	33	36	68	1	3	2	4		5
BLN-15	B	Micrite+wavy shale bands	63	30	0	5	2	--									
BLN-16	B	Limestone	72	20	2	3	3	--									

BLN COL 2.5-1
BLN COL 2.5-6

TABLE 2.5-224 (Sheet 3 of 4)
PETROGRAPHIC AND MINERALOGIC RESULTS

Sample no.	Lithologic unit	Field description	Petrography visual estimate (%)					TGA analysis (%)				X-ray diffraction (order of peak height)					
			Calcite	Dolomite (Mg,Fe)	Quartz	Clay	Other	Sample no.	Calcite	Dolomite (Mg,Fe)	Calcite + Dolomite	Calcite	Dolomite (Mg,Fe)	Quartz	Muscovite	Kaolinite	Pyrite
BLN-17	B	Limestone	97	2	0	0	1	BLN-17L	96	0	96						
BLN-18	A	Micrite+wavy shale laminae	65	20	5	7	3	BLN-18L	75	0	75						
BLN-19	A	MIcrite+wavy shale laminae	84	10	3	0	3	--									
BLN-20	*	Micrite+wavy shale lam.	95	0	2	2	1	--									
BLN-21	*	Sparry micrite	93	0	2	3	2	--									
BLN-22	*	Micrite+wavy shale lam.	93	5	<1	1	1	--									
BLN-23	*	Micrite+wavy shale lam.	91	0	1	4	4	--									
BLN-24	*	Silty limestone	95	2	<1	1	2	--									
BLN-25	*	Silty limestone	96	0	0	1	3	--									

BLN COL 2.5-1
BLN COL 2.5-6

TABLE 2.5-224 (Sheet 4 of 4)
PETROGRAPHIC AND MINERALOGIC RESULTS

Sample no.	Lithologic unit	Field description	Petrography visual estimate (%)					TGA analysis (%)				X-ray diffraction (order of peak height)					
			Calcite	Dolomite (Mg,Fe)	Quartz	Clay	Other	Sample no.	Calcite	Dolomite (Mg,Fe)	Calcite + Dolomite	Calcite	Dolomite (Mg,Fe)	Quartz	Muscovite	Kaolinite	Pyrite
BLN-26	*	Micrite+wavy shale laminae	82	15	0	1	2	--									
BLN-27	Na	Green bentonite	0	<1	3	48	49**	--									
BLN-28	Na	Whitish bentonite	0	38	0	54	8**	--									

* Samples are from the southern site (Reference 399) where lithologic units were not established.

** Mostly potassium feldspar

-- Not tested

Na Nashville Group

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-225
FREQUENCY OF CAVITIES IN BOREHOLES BY LOCATION

Location[a]	Source	No. borings with cavities ≥0.1 ft. high	Total no. borings drilled	Percentage borings with cavities
Units 3 and 4 power block construction zone	Appendix 2BB	16	75	21%
Outside Units 3 and 4 power block construction zone	Appendix 2BB	9	47	19%
Units 1 and 2 power block area	Reference 201	27	85	32%
Intake channel area	Reference 201	17	23	74%
Southern site	Reference 399	9	17	53%
TOTAL boreholes		78	247	32%

Cavities in rock were encountered in a percentage of all boreholes throughout the BLN site, but are more frequent in some areas. This table shows the number of boreholes that encountered one or more cavities ≥0.1 ft. in height, at least 1 ft. below top-of-rock.

a) See Figure 2.5-201

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 1 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes
B-1001	13.4	14.0	1.1	0.8	Clay-filled cavity. Clay (10 YR 5/6), wet, stiff, medium plasticity, sand to gravel sized limestone clasts.
	15.4	16.1	6	0.85	Clay-filled cavity
	17.8	19.1	6	1.5	Clay-filled cavity
	19.8	20.5	8.4	0.8	Clay-filled cavity
	20.6	21.0	9.3	0.5	Clay-filled cavity. Rod drop 20.5-21.6. 21' water circulation lost, did not recover.
B-1002	27.8	28.0	11.4	0.2	Cavity. Poor to moderate fluid returns 51-120.5' bgs.
	33.4	33.5	16.9	0.1	Cavity. Poor to moderate fluid returns 51-120.5' bgs.
B-1008	18.3	20.5	6.4	2.2	Clay filled cavity. Lost water circulation 18.3-21.3' and 36.4-121.3' bgs. 90% water loss.
	23.9	24.0	10.9	0.1	Clay-filled cavity
B-1027	17.4	17.7	5.2	0.3	Cavity. Water circulation lost at 17'. Poor fluid return until 50' bgs.
B-1033	14.1	14.2	1.6	0.1	Clay-filled cavity.

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 2 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes
	14.6	14.7	2.1	0.1	Clay-filled cavity.
	19.0	19.2	6.6	0.2	Clay-filled cavity.
	19.3	19.5	6.8	0.2	Clay-filled cavity.
	19.7	19.9	7.2	0.2	Clay-filled cavity.
	30.1	30.2	17.6	0.1	Clay-filled cavity.
B-1036	18.0	18.1	2	0.1	Clay-filled cavity. Water circulation lost. 20.8-22' and 40.1' bgs to the bottom of the boring.
	18.2	18.3	2.2	0.1	Clay-filled cavity.
B-1038	17.9	18.0	5.2	0.1	Clay filled cavity.
B-1044	15.2	15.4	7.3	0.2	Cavity. Rod drop during drilling of ~1' at 15.2' bgs.
	63.0	63.1	55	0.1	Cavity.
B-1046	27.8	27.9	4.3	0.1	Calcareous clay infilled cavity. Lost water circulation at 25.4' bgs.
B-1050	16.2	16.9	1.7	0.7	Clay filled cavity. No water circulation throughout boring.

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 3 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes
	17.5	17.7	2.8	0.2	Clay filled cavity. No water circulation throughout boring.
	18.8	19.0	4.1	0.2	Clay filled cavity. No water circulation throughout boring.
B-1052	44.2	44.5	18.1	0.3	Cavity. No water circulation throughout boring.
	44.7	45.5	18.9	0.8	Cavity. No water circulation throughout boring.
	59.2	60.8	33.8	1.6	Clay filled cavity; (7.5YR 4/8), highly plastic.
B-1065	20.1	20.6	2.8	0.5	Clay-filled cavity.
	24.4	24.5	6.9	0.1	Clay-filled cavity.
	27.7	27.9	10.3	0.2	Clay-filled cavity.
B-1066	20.2	20.4	10.8	0.2	Clay-filled cavity. No water circulation throughout boring.
B-1069	31.3	31.4	22.0	0.1	Clay-filled cavity. Water circulation lost for first 26' bgs.
	33.4	33.5	24.1	0.1	Clay-filled cavity.
	34.5	34.6	25.2	0.1	Clay-filled cavity.
	37.5	37.6	28.2	0.1	Clay-filled cavity.

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 4 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes
	38.2	38.3	28.9	0.1	Clay-filled cavity.
B-1070	8.2	8.8	5.2	0.6	Clay-filled cavity.
B-1071	16.4	17.4	3.3	1	Soil-filled cavity. Rod drop at ~ 16' bgs. Water circulation lost at ~28' bgs.
B-1072	14.6	15.4	8.4	1.2	Soil-filled cavity.
	15.7	15.9	9.0	0.2	Soil-filled cavity.
	20.4	28.4	17.6	8	Soil-filled cavity.
	28.8	29.8	22.5	1	Soil-filled cavity. Water circulation lost at 45.5' bgs.
B-1074	16.3	17.6	11.2	1.3	Clay-filled cavity.
	20.1	21.8	15.2	1.6	Cavity.
	25.0	25.2	19.4	0.2	Cavity. Water circulation low at 26' bgs.
B-1076	12.6	16.6	6.3	4	Soil-filled cavity. Water circulation lost 13-50' bgs.
B-1077A	14.5	14.6	1	0.1	Clay-filled cavity.

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 5 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes
B-1080	34.2	34.3	24.2	0.1	Clay-filled cavity. No water circulation throughout boring.
B-1082	10.8	12.9	4.3	2.1	Soil-filled cavity; medium stiff.
	13.0	16.0	7	3	Soil-filled, medium stiff
B-1093	11.5	11.6	1.7	0.1	Clay-filled cavity. Slight loss of water circulation.
	11.7	11.8	1.9	0.1	Clay-filled cavity. Slight loss of water circulation.
	12.6	12.7	2.8	0.1	Clay-filled cavity. Slight loss of water circulation.
	12.8	12.9	3	0.1	Clay-filled cavity. Slight loss of water circulation.
B-1094	12.2	12.3	1.2	0.1	Clay-filled cavity. Poor to no water circulation through 16' bgs.
B-1095	14.4	14.8	1.3	0.4	Soil-filled cavity; medium stiff.

BLN COL 2.5-1
BLN COL 2.5-5

TABLE 2.5-226 (Sheet 6 of 6)
SUMMARY OF CAVITIES OBSERVED IN BORINGS

Borehole	Interval (bgs) Top	Interval (bgs) Bottom	Average Depth Below TOR (ft)	Thickness (ft)	Cavity Description and Drilling Notes

Notes:
bgs = below ground surface. TOR = top of rock.
Data are compiled from borehole logs, Appendix 2BB.
Criteria for identifying cavities in boring logs:
1) The cavity must be located at least 1 foot below the top of competent rock.
2) A cavity may be open, or partially to completely filled with soil or clay.
3) A cavity must be at least 0.1 foot-thick.
4) The following statements by the rig geologist indicate the presence of a cavity:
-The terms “void,” “cavity,” "soil," "clay," or "infill."
-The combination of the statements: “rod drop,” “short recovery,” and one or more “no fit” discontinuities, if the “lost” core was not recovered in later runs.

BLN COL 2.5-6

TABLE 2.5-227 (Sheet 1 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

Facility or Zone		Boring Number	Depth (ft)	Coordinates[3]		In-situ Testing				
				Northing	Easting	P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
Power Block AP1000 (Nuclear Island) and Adj. Structures										
	Unit 3	B-1000	185.9	1533129.7	628373.5					
		B-1001	176.5	1533081.9	628359.4		X		X	
		B-1001A	17.2	1533081.9	628359.4					
		B-1002	120.5	1532906.7	628471.5		X			X
		B-1003	120.4	1532838.5	628359.6		X			
		B-1004	150.2	1532950.0	628276.6					X
		B-1005	251.0	1532943.5	628407.1	X				
		B-1005 PS	26.2	1532936.7	628398.0	X				
		B-1006	176.0	1532755.0	628259.7	X				
		B-1006 UDD	13.1	1532752.4	628266.8					
		B-1007	120.0	1533041.2	628430.9					
		B-1008	121.3	1533048.9	628211.6		Attempted			X
		B-1009	75.8	1532957.3	628130.1					
		B-1010	74.0	1532836.3	628178.5					
		B-1011	75.3	1532610.4	628132.4		X			
		B-1012	50.0	1532678.0	628247.8					X
		B-1013	50.0	1532758.8	628315.6		X			
		B-1014	121.2	1532984.8	628354.3		X			
		B-1015	75.4	1533020.6	628276.3				X	

TABLE 2.5-227 (Sheet 2 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

BLN COL 2.5-6

			Coordinates[3]		In-situ Testing				
Facility or Zone	Boring Number	Depth (ft)	Northing	Easting	P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
Unit 3 (continued)	B-1015 UDD	15.0	1533026.4	628273.1					
	B-1016	20.7	1532867.3	628224.3				X	
	B-1017	50.6	1532937.4	628225.4					
	B-1017 UDS	18.0	1532938.0	628228.7					
	B-1017 UDD	15.0	1532937.2	628222.1					
	B-1018	16.5	1532827.3	628217.6					
	B-1019	35.0	1532716.2	628178.6					
	B-1020	35.0	1532590.2	628188.0					
	B-1021	35.4	1532997.4	628462.5					
Adjacent Structures	B-1022	76.1	1532633.2	628366.9					
	B-1023	60.1	1532853.9	628421.0			X		
	B-1024	175.4	1532702.4	628492.0					
	B-1025	75.0	1532823.2	628567.9					
	B-1025 UDS	11.6	1532828.3	628567.3					
	B-1026	50.0	1533363.2	628319.4					X
	B-1027	50.0	1532755.9	627608.5					Attempted
	B-1028	88.1	1532272.0	628314.6					
	B-1029	35.9	1533050.8	628111.8					
	B-1030	35.2	1532718.7	628698.8					
	B-1031	35.0	1532538.5	628081.8					
	B-1032 PS	36.2	1533056.2	628252.8	X				
	B-1032	175.6	1533054.2	628254.8	X				

BLN COL 2.5-6

TABLE 2.5-227 (Sheet 3 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

Facility or Zone	Boring Number	Depth (ft)	Coordinates[3] Northing	Easting	In-situ Testing P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
Unit 4	B-1033	121.5	1532169.8	628888.6					
	B-1034	249.5	1532442.6	629037.0	X	X		X	
	B-1034 PS	31.3	1532441.2	629046.3	X				
	B-1035	171.1	1532549.5	629112.9					
	B-1036	55.3	1532263.9	628856.1	X				
	B-1036A	175.5	1532263.7	628852.7					
	B-1036 UDD	13.8	1532264.0	628847.1					
	B-1037	149.7	1532329.0	628980.8					X
	B-1038	65.8	1532091.5	628812.5					
	B-1039	120.0	1532472.2	628859.9					X
	B-1040	119.7	1532554.0	629061.0					
	B-1041	120.3	1532544.9	628845.0					
	B-1042	124.3	1532377.2	628935.9		Attempted			
	B-1042A	23.0	1532377.2	628935.9					
	B-1043	2.0	1532423.8	628741.9				X	
	B-1043A	75.6	1532424.3	628756.8		X		X	
	B-1044	76.0	1532399.8	629086.6		X			
	B-1045	75.1	1532482.4	628986.7		X			
	B-1046	49.4	1532242.9	628945.3					X
	B-1047	120.0	1532622.2	628995.6					
	B-1048	76.3	1532210.9	628709.9		X		X	
	B-1048 UDD	17.3	1532209.8	628704.1					
	B-1049	128.0	1532539.2	628917.1			X		

TABLE 2.5-227 (Sheet 4 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

BLN COL 2.5-6

Facility or Zone	Boring Number	Depth (ft)	Coordinates[3]		In-situ Testing				
			Northing	Easting	P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
Unit 4 (continued)	B-1050	75.0	1532300.7	628780.2			X		
	B-1051	43.0	1532145.5	628731.1					
	B-1051A	51.9	1532159.7	628729.8					
	B-1052	79.0	1532330.9	628912.5					
	B-1052A	26.7	1532330.9	628912.5					
	B-1053	35.0	1532489.4	628910.4					
	B-1054	35.0	1532232.5	628800.2					
	B-1055	71.6	1532492.0	629080.8					
	B-1056	45.7	1532231.0	628882.6					
	B-1057	35.2	1532372.8	628836.1					
Adjacent Structures	B-1058	65.2	1532351.3	629027.4			X		
	B-1059	250.0	1532712.7	628706.2	X	X			X
	B-1060	176.3	1532444.1	629267.9	X				
	B-1060 PS	31.3	1532438.3	629268.5	X				
	B-1061	75.2	1532124.2	629092.1		X		X	
	B-1062	150.8	1532323.8	629378.9					
	B-1063	50.2	1531925.4	628928.3					
	B-1064	50.0	1532447.5	628574.8					
	B-1065	41.5	1532496.4	629393.6					
	B-1066	124.4	1532293.3	629209.1					
	B-1066 UDS	6.0	1532290.8	629211.7					
	B-1067	174.6	1532868.7	628839.9	X	X			
	B-1067 PS	23.5	1532872.1	628836.1	X	X			

BLN COL 2.5-6

TABLE 2.5-227 (Sheet 5 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

			Coordinates[3]		In-situ Testing				
Facility or Zone	Boring Number	Depth (ft)	Northing	Easting	P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
CCW Pipelines									
Unit 3	B-1068	50	1532280.8	627961.6					
Unit 4	B-1069	50	1531732.6	628733.3					
	B-1070	51	1530825.5	628348.8					
Cooling Tower									
Unit 3	B-1071	75.0	1531172.7	626508.3		Attempted			
Unit 4	B-1072	50.1	1530152.0	627504.4					
	B-1073	75.3	1530642.8	627276.0		Attempted			
	B-1074	50.0	1530685.8	627635.9					
	B-1075	75.0	1530444.8	627150.2					
General Site Coverage and Facilities	B-1076	50.0	1532710.0	628084.9					
	B-1077	15.8	1532870.6	626759.1					
	B-1077A	150.3	1532870.9	626755.1					
	B-1078	75.0	1533819.2	627721.5					
	B-1079	150.0	1533742.4	628277.0					
	B-1080	35.0	1532389.9	630169.0					
	B-1081	33.5	1532026.1	628657.8					
	B-1082	75.0	1532535.4	628502.6		X		X	
	B-1083	75.0	1532571.1	629135.1					X
	B-1084	50.3	1532586.4	628991.9					
	B-1085	6.9	1532807.8	627935.7					
	B-1085A	16.5	1532807.6	627930.7					

BLN COL 2.5-6

TABLE 2.5-227 (Sheet 6 of 6)
SUMMARY OF COMPLETED BORINGS AND IN SITU TESTING

			Coordinates[3]		In-situ Testing				
Facility or Zone	Boring Number	Depth (ft)	Northing	Easting	P-S Suspension and/or Downhole Logging	Borehole Televiewer	Goodman Jack	N-Gamma Logging	Packer Test
	B-1086	75.3	1533155.7	628520.7		X		X	
	B-1087	35.4	1532677.2	629071.3					
	B-1088	71.2	1533043.3	628497.2					
	B-1089	40.0	1531668.7	628473.1					
	B-1090	128.0	1533200.9	628455.8					
	B-1091	35.0	1531925.1	629403.7					
	B-1092	56.0	1532370.7	628902.4					
	B-1092 UDD	14.3	1532373.0	628904.0					
	B-1093	46.1	1532291.1	629224.6					
	B-1094	35.0	1533085.6	628571.3					
	B-1095	50.1	1532077.1	628968.9					
	B-1096	50.1	1532615.5	629217.0				X	
	B-1097	50.0	1531841.7	628482.3					
	B-1098	50.0	1532350.7	627838.6					
	B-1099	50.0	1533127.1	627932.7					

Notes:

1. See Figures 2.5-325, 2.5-326 and 2.5-328 for plan views of locations
2. NA = Not applicable, undisturbed soil boring only.
3. Coordinates referenced to Alabama Mercator East State Coordinate System and NAD 83(92) Horizontal Datum
4. Boring logs obtained in Appendix 2BB

BLN COL 2.5-6

TABLE 2.5-228
SUMMARY OF COMPLETED CONE PENETROMETER (CPT) SOUNDINGS

Facility or Zone	CPT Number	Coordinates Northing (ft)	Coordinates Easting (ft)	Depth (ft)
Power Block AP1000 (Nuclear Island) and Adm. Structures				
Unit 3	CPT-1301	1532891.3	628536.5	17.7
	CPT-1302	NP	NP	NP
	CPT-1303	1533043.3	628497.2	11.5
	CPT-1304	NP	NP	NP
Unit 4	CPT-1305	1532530.4	628922.0	11.3
	CPT-1306	1532444.2	629150.4	6.4
	CPT-1307	1532626.2	628940.0	11.3
	CPT-1308	NP	NP	NP
	CPT-1309	1532938.5	628231.8	20
Cooling Tower				
Unit 3 Tower	CPT-1310	1532141.1	628760.8	18.4
Unit 4 Tower	CPT-1311	1532144.3	628758.2	19.9
	CPT-1312	1532148.4	628754.9	20.7
General Site Coverage and Facilities				
	CPT-1313	1532152.1	628751.7	18.9
	CPT-1314	1532155.4	628747.8	18.4
	CPT-1315	1532159.3	628744.6	26.3
	CPT-1316	1532162.7	628741.0	33.1
	CPT-1317	1532166.2	628736.8	14.3
	CPT-1318	1532180.5	628749.6	20.2
	CPT-1319	1532177.0	628751.8	21.2
	CPT-1320	1532173.8	628755.1	15.6
	CPT-1321	1532170.5	628759.3	16.4

Notes

1. See Figure 2.5-328 for plan view of locations
2. Coordinates references to Alabama Mercator East State Coordinate System and NAD 83(92) Horizontal Datum
3. NP=Not performed

BLN COL 2.5-6

TABLE 2.5-229
SUMMARY OF COMPLETED TEST PITS

Facility or Zone		Test Pit Number	Coordinates Northing	Coordinates Easting
Power Block AP1000 (Nuclear Island) and Adj. Structures				
	Unit 3	T-1401	1532801	628112
		T-1402	1532812	628594
		T-1403	1533027.2	628350.6
	Unit 4	T-1404	1532443.4	629154.6
		T-1405	1532671	628884
		T-1406	1532248.2	628801.2
General Site Coverage and Facilities				
		T-1408	1532154	628209
		T-1409	1532050.2	628451.3
Test Pit location unallocated: Pending results of geologic mapping				
		T-1410	1532120.9	629096.8
		T-1411	1532089.9	628801.9
		T-1414	1532024.35	628679.7
		T-1415	1533686.79	627017.83
		T-1416	1533192.8	628473.26
		T-1417	1533167.3	628508.8

Notes:

1. See Figure 2.5-328 for plan view of locations
2. Coordinates referenced to Alabama Mercator East State Coordinate System and NAD 83(92) Horizontal Datum.

BLN COL 2.5-6

TABLE 2.5-230 (Sheet 1 of 5)
SOIL INDEX, CLASSIFICATION AND CHEMISTRY

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel [1] (%)	Sand [1] (%)	Fines [2] (%)	Silt [1] (%)	0.005 mm Clay [1] (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G_s	pH [3]	Chloride (mg/kg) (3), (6), (7)	Sulfate (mg/kg) (3), (6), (7)
B-1000	B-1000-3A	6-7.5	SPT	1.1	15.3	83.6	(9)	(9)	MH	23.4	59	25	(9)	(9)	(9)	(9)
B-1000	B-1000-5A	11-12.5	SPT	3.0	5.9	91.1			CH	23.3	95	68				
B-1002	B-1002-2B	3.5-5	SPT	0.0	8.6	91.4			CH	25.5	66	34				
B-1002	B-1002-3A,3B	6-7.5	SPT	0.0	14.7	85.3	28.5	56.8	CH	22.9	51	26				
B-1002	B-1002-4A	8.5-10	SPT						CH [8]	21.5	50	27				
B-1002	B-1002-5B	11-12.5	SPT	0.3	5.6	94.1			CH	25.9	68	45				
B-1003	B-1003-2	3.5-5	SPT	0.5	16.6	82.9			CL	16.4	26	10				
B-1003	B-1003-4	8.5-10	SPT	13.4	8.2	78.4			CL	22.3	44	23				
B-1003	B-1003-5	11-12.5	SPT	30.3	1.5	68.2	32.2	36.0	CL	16.0	44	23				
B-1004	B-1004-1	0-3.4	CME	5.6	18.9	75.5			CL	3.8	24	9				
B-1004	B-1004-2	3.4-8.2	CME	10.4	17.2	72.4			CL	15.2	48	23				
B-1004	B-1004-3	8.2-13.1	CME	1.0	5.0	94.0			CH	22.0	74	45				
B-1006	B-1006-3	6-7.5	SPT	2.0	38.2	59.8			CL	25.3	41	16				
B-1006	B-1006-4B	8.5-10	SPT						CL [8]	18.8	30	13				
B-1009	B-1009-3A	6-7.5	SPT	0.5	26.3	73.2			MH	22.0	51	20				
B-1009	B-1009-5A	11-12.5	SPT	5.1	37.7	57.2			CH	20.5	55	27				
B-1009	B-1009-6A	13.5-15	SPT	0.5	14.4	85.1			CH	21.6	81	56				
B-1010	B-1010-3UDS	4-6 [4]	UD	1.3	24.9	73.8	32.3	41.5	CL	19.3	48	27	2.70			
B-1010	B-1010-5UDS	7.5-9.5 [4]	UD	34.4	19.0	46.6	6.4	40.2	GC	29.1	74	49	2.98			
B-1011	B-1011-3	6-7.5	SPT	1.3	13.4	85.3			CH	24.0	77	53				
B-1011	B-1011-4	8.5-10	SPT	0.0	1.6	98.4			CL	17.9	34	16				
B-1012	B-1012-2	3.5-5	SPT	0.0	13.5	86.5			MH	25.2	63	28				
B-1012	B-1012-3	6-7.5	SPT							15.6				5.6	1.7[B,J]	ND [5]

BLN COL 2.5-6

TABLE 2.5-230 (Sheet 2 of 5)
SOIL INDEX, CLASSIFICATION AND CHEMISTRY

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel (1) (%)	Sand (1) (%)	Fines (2) (%)	Silt (1) (%)	0.005 mm Clay (1) (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G_s	pH (3)	Chloride (mg/kg) (3), (6), (7)	Sulfate (mg/kg) (3), (6), (7)
B-1012	B-1012-4	8.5-10	SPT	2.4	17.7	79.9			CH	23.4	68	46				
B-1013	B-1013-1B	1-2.5	SPT	5.3	12.4	82.3			CH	27.0	78	45				
B-1013	B-1013-2B	3.5-5	SPT	0.6	11.9	87.5			CL	18.0	28	12				
B-1015	B-1015-3UDS	4-6 (4)	UD	2.8	19.4	77.8	22.8	55.0	CH	20.4	78	44	2.78			
B-1015	B-1015-5UDS	7.5-9.5 (4)	UD	1.1	7.2	91.7	10.8	80.9	CH	21.7	98	77	2.69			
B-1017 UDS	B-1017UDS-1UDS	3.5-5.5 (4)	UD	0.4	20.2	79.4	33.4	46.0	CL	17.3	49	28	2.84			
B-1017 UDS	B-1017UDS-2UDS	8-10 (4)	UD	9.4	14.0	76.6	7.6	69.0	CH	27.7	115	82	2.71			
B-1017 UDS	B-1017UDS-3UDS	12-14 (4)	UD	0.9	14.5	84.6	19.8	64.8	CH	27.7	79	55	2.76			
B-1017	B-1017-4	8.5-10	SPT							16.3				5.7	1.7[B,J]	ND (5)
B-1019	B-1019-3A	6-7.5	SPT	0.4	12.8	86.8			CH	21.9	63	43				
B-1019	B-1019-6A	13.5-15	SPT	5.9	18.8	75.3			CH	17.8	69	45				
B-1020	B-1020-1A	1-2.5	SPT	72.9	16.1	11.0			GP-GC	6.7	38	19				
B-1020	B-1020-2	3.5-5	SPT	1.0	20.6	78.4			CL	28.3	42	21				
B-1020	B-1020-3A	6-7.5	SPT							18.4				6.8	6.0[J]	49.6
B-1023	B-1023-2	3.5-5	SPT	0.0	16.9	83.1			CH	26.6	62	36				
B-1023	B-1023-4A,4B	8.5-10	SPT	12.5	8.3	79.2	18.9	60.3	CH	25.8	73	48				
B-1025 UDS	B-1025UDS-1UDS	8-10 (4)	UD	0.5	12.8	86.7	19.3	67.4	CH	26.8	76	54	2.79			
B-1032	B-1032-1	1-2.5	SPT	0.0	19.0	81.0			MH	17.4	54	23				
B-1032	B-1032-2	3.5-5	SPT	0.0	18.2	81.8			CH	21.9	51	32				
B-1035	B-1035-2	2.5-4	SPT	3.2	24.7	72.1			CH	21.3	50	26				
B-1035	B-1035-3	5-6.5	SPT	0.0	4.4	95.6			CH	23.4	58	35				
B-1036	B-1036-4	8.5-11	SPT	2.7	22.6	74.7			CL	18.3	37	17				
B-1037	B-1037-2	3.5-5	SPT	4.4	21.5	74.1			CL	16.8	38	19				

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TABLE 2.5-230 (Sheet 3 of 5)
SOIL INDEX, CLASSIFICATION AND CHEMISTRY

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel [1] (%)	Sand [1] (%)	Fines [2] (%)	Silt [1] (%)	0.005 mm Clay [1] (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G_s	pH [3]	Chloride (mg/kg) [3], [6], [7]	Sulfate (mg/kg) [3], [6], [7]
B-1037	B-1037-4	8.5-10	SPT	0.0	8.6	91.4			CH	24.8	73	43				
B-1038	B-1038-3UDS	5-7 [4]	UD	19.7	24.1	56.2	19.0	37.2	CH	21.5	68	39	2.59			
B-1038	B-1038-5UDS	8.5-10.5 [4]	UD	0.0	6.8	93.2	20.5	72.7	CH	32.3	88	60	2.75			
B-1043A	B-1043A-2A,2B	3.5-5	SPT	1.3	19.2	79.5			CL	17.4	36	19				
B-1043A	B-1043A-4A	8.5-10	SPT	0.0	12.8	87.2	23.0	64.2	CH	30.0	73	47				
B-1044	B-1044-2B	3.5-5	SPT	1.0	18.7	80.3			CH	19.8	52	26				
B-1045	B-1045-1	1-2.5	SPT	0.6	13.7	85.7			CH	22.0	58	34				
B-1045	B-1045-3	6-7.5	SPT	17.6	31.4	51.0			CL	10.7	41	21				
B-1045	B-1045-6	13.5-15	SPT	0.0	13.8	86.2			CH	23.4	71	40				
B-1046	B-1046-4	8.5-10	SPT	0.2	20.8	79.0	36.0	43.0	CL	21.9	46	22				
B-1046	B-1046-7	16-17.5	SPT	3.5	9.1	87.4			CH	24.3	72	48				
B-1046	B-1046-8	18.5-20	SPT	6.1	8.7	85.2			CH	34.3	79	50				
B-1046	B-1046-9	21-22.5	SPT						CH [8]	39.8	84	54				
B-1047	B-1047-2	3.5-5	SPT	0.6	12.5	86.9			CH	23.1	59	33				
B-1047	B-1047-3	6-7.5	SPT	1.5	7.6	90.9			CH	25.6	66	38				
B-1048	B-1048-2	3.5-5	SPT	0.3	34.7	65.0			CH	22.7	54	26				
B-1048	B-1048-4	8.5-10	SPT	3.9	29.1	67.0			CH	20.7	51	24				
B-1048	B-1048-5	11-12.5	SPT							18.8				6.9	3.0[J]	ND [5]
B-1048	B-1048-7	16-17.5	SPT	0.0	2.0	98.0			CL	29.4	38	16				
B-1050	B-1050-1	1-2.5	SPT	2.9	30.8	66.3			CH	18.0	55	32				
B-1050	B-1050-2	3.5-5	SPT	1.8	29.2	69.0			CL	24.2	49	21				
B-1051	B-1051-3B	6-7.5	SPT	2.1	14.5	83.4			CH	20.8	57	30				
B-1051	B-1051-7	16-17.5	SPT	33.9	7.3	58.8			CH	16.3	72	45				

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TABLE 2.5-230 (Sheet 4 of 5)
SOIL INDEX, CLASSIFICATION AND CHEMISTRY

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel (1) (%)	Sand (1) (%)	Fines (2) (%)	Silt (1) (%)	0.005 mm Clay (1) (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G_s	pH (3)	Chloride (mg/kg) (3), (6), (7)	Sulfate (mg/kg) (3), (6), (7)
B-1051	B-1051-10B	23.5-25	SPT	1.6	2.6	95.8			CH	45.2	75	44				
B-1051	B-1051-13	36-37.5	SPT	27.7	8.4	63.9			CH	50.3	62	34				
B-1053	B-1053-1	1-2.5	SPT	1.8	16.1	82.1			CL	15.7	44	24				
B-1053	B-1053-2	3.5-5	SPT							10.2				7.2	0.72[B,J]	3.5[B]
B-1053	B-1053-3	6-7.5	SPT	0.3	12.3	87.4			CL	23.6	47	27				
B-1054	B-1054-2	3.5-5	SPT	2.0	20.6	77.4			CL	14.3	46	25				
B-1054	B-1054-3	6-7.5	SPT	0.1	25.9	74.0	21.6	52.4	CH	23.0	52	26				
B-1054	B-1054-5	11-12.5	SPT	49.9	37.0	13.1				6.7						
B-1057	B-1057-1A	1-2.5	SPT	0.0	12.5	87.5			CH	22.0	56	34				
B-1066 UDS	B-1066UDS-1UDS	4-6 (4)	UD	0.0	11.7	88.3	10.3	78.0	CH	30.7	82	56	2.84			
B-1071	B-1071-5	5.5-8.3	CME							12.8				6.7	0.42[B,J]	1.1[B]
B-1073	B-1073-3	6-7.5	SPT							21.2				6.4	2.8[J]	2.5[B]
B-1080	B-1080-3TOP	2.5-3.5	CME							6.1				7.6	0.36[B,J]	3.0[B]
B-1083	B-1083-2	5-10	CME							12.0				7.5	1.0[B,J]	0.65[B]
B-1086	B-1086-1B	1-2.5	SPT	0.0	12.1	87.9			CH	22.3	58	33				
B-1086	B-1086-2A	3.5-5	SPT							18.4				7.2	3.4[J]	1.5[B]
B-1087	B-1087-1	1-2.5	SPT							9.4				6.7	0.43[B,J]	7.7
B-1088	B-1088-1UDS	4-6 (4)	UD	0.0	10.3	89.7	21.4	68.3	CH	17.2	72	51	2.71			
B-1094	B-1094-1	1-2.5	SPT	2.4	22.2	75.4			CH	18.6	60	33				

(1) Due to computer roundoff, particle size fractions may total 100 ± 1.

(2) Fines include silt plus clay.

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TABLE 2.5-230 (Sheet 5 of 5)
SOIL INDEX, CLASSIFICATION AND CHEMISTRY

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel (1) (%)	Sand (1) (%)	Fines (2) (%)	Silt (1) (%)	0.005 mm Clay (1) (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G_s	pH (3)	Chloride (mg/kg) (3), (6), (7)	Sulfate (mg/kg) (3), (6), (7)

(3) Tests performed by STL - St. Louis, MO

(4) Depth interval shown reflects total pushed depth of UD tube.

(5) ND indicates analyte not detected at or above the Method Detection Limit

(6) B = Estimated Result. Result is less than Reporting Limit

(7) J = Method blank contamination. The associated method blank contains the target analyte at a reportable level

(8) USCS Classification estimated based on visual estimation that >50% passing #200 sieve.

(9) Blank cells indicate that not test for that parameter assigned.

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TABLE 2.5-231
RANGE OF GRAIN SIZE DISTRIBUTION AND PLASTICITY

a. Grain Size Distribution

	Depth of Soil Strata m (ft.)	Gravel (%)	Sand (%)	Coarse Fraction %	Fines (%)	Clay (0.005 mm) (%)	Clay (0.002 mm) (%)
Minimum	1.49 (4.9)	0.0	1.5	1.6	11*	36.0	25.0
Maximum	13.11 (43.0)	72.9	38.2	49	98.4	80.9	72.5
Average:	4.66 (15.3)	5.7	16.0	19.4	78.3	57.4	49.1
No. of Tests:	12.50 (41)	74	74	71	74	18	18

* Only 3 samples with % fines <50%

b. Plasticity

	Natural Moisture (%)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	Specific Gravity, G_s	LiquidityI ndex, LI	Activity
Minimum	3.8	24	15	9	2.59	-1.24	0.54
Maximum	50.3	115	35	82	2.98	0.66	1.44
Average:	20.7	59	25	35	2.76	-0.08	0.92
No. of Tests:	88	76	76	76	12	74	18

TABLE 2.5-232
BLN COL 2.5-6 COMPACTION CHARACTERISTICS USING STANDARD AND MODIFIED EFFORT

Source of Sample	Sample No.	Atterberg Limits: Liquid Limit (LL)	Atterberg Limits: Plasticity Index (PI)	Moisture-Density Relationship Data: Natural Moisture (%)	Moisture-Density Relationship Data: Maximum Dry Density kN/m^3 (pcf)	Moisture-Density Relationship Data: Optimum Moisture (%)
Boring B-1025UDS	B-1025UDS, 1-5'	46	26	11.5	18.70 (119.2)^A	12.9^A
Boring B-1055	B-1055, 1-5'	47	26	15.4	19.19 (122.3)^A	16.0^A
Boring B-1055	B-1055, 5-10'	52	28	18.7	16.03 (102.2)^B	20.1^B
Boring B-1088	B-1088, 1-5'	58	34	21.5	15.61 (99.5)^B	18.9^B
Boring B-1088	B-1088, 5-10'	54	32	11.7	19.29 (123.0)^A	11.0^A

A = ASTM D 1557-02 Method A Modified

B = ASTM D 698-00a Method A Standard

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TABLE 2.5-233
SUMMARY OF TRIAXIAL TESTS

Source of Sample	Sample No.	Sample Depth, m (ft)	USCS	Atterberg Limits: Liquid Limit, LL (%)	Atterberg Limits: Plasticity Index, PI (%)	Initial Dry Unit Weight, kN/m^3 (pcf)	Initial Moisture Content, %	Initial Void Ratio	Triaxial Test Data[a]: c, kPa (psf)	Triaxial Test Data[a]: Φ, (°)	Triaxial Test Data[a]: c', kPa (psf)	Triaxial Test Data[a]: Φ', (°)
B-1010	UDS3	1.2 - 1.8 (4-6)	CL	48	27	16.36 (104.3)	20.2	0.62	84.7 (1771)	11.4	0	32.7
B-1017 UDS	UDS3	3.6 - 4.3 (12-14)	CH	79	55	15.33 (97.7)	25.7	0.77	73.8 (1541)	1.6	11.3 (235)	26.8

a) Values shown are average of three test samples

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TABLE 2.5-234
UNCONFINED COMPRESSION TEST RESULTS

Source of Sample	Sample No.	Sample Depth m (ft)	USCS	Atterberg Limits: Liquid Limit, LL (%)	Atterberg Limits: Plasticity Index, PI (%)	Initial Dry Unit Weigh, kN/m^3 (pcf)	Initial Moisture Content, %	Strength [(a)] kPa (psf)
B-1025UDS	1UDS	2.4 - 3.0 (8-10)	CH	76	54	16.23 (103.6)	23.7	140.3 (2929)
B-1066UDS	1UDS	1.2 - 1.8 (4-6)	CH	82	56	15.33 (97.6)	26.8	170 (3551)

a) Maximum Deviator Stress

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TABLE 2.5-235
SUMMARY OF CONSOLIDATION TESTS

Source of Sample	Sample No.	Sample Depth, m (ft)	Specific Gravity	σ_v'kPa (psf)	Initial Dry Unit Weight, kN/m^3 (pcf)	Initial Moisture Content, %	Initial Void Ratio	Initial Satura-tion. %	Consolidation Test Results p_c kPa (psf)	C_c	C_r	OCR
B-1010	5UDS	2.3 - 2.9 (7.5-9.5)	2.98	512 (1070)	15.89 (101.1)	24.34	0.84	86.4	340 (7100)	0.183	0.018	6.6
B-1025UDS	1UDS	2.4 - 3.0 (8-10)	2.79	536 (1120)	98.22 (98.22)	26.46	0.78	95.1	268 (5600)	0.203	0.02	5
B-1038	5UDS	2.6 - 3.2 (8.5-10.5)	2.75	550 (1150)	14.63 (93.12)	30.42	0.75	99.1	359 (7500)	0.219	0.025	6.5
B-1066UDS	1UDS	1.2 - 1.8 (4-6)	2.84	292 (610)	15.46 (98.42)	24.19	0.80	85.6	273 (5700)	0.249	0.0066	9.3
B-1088	1UDS	1.2 - 1.8 (4-6)	2.71	287 (600)	15.68 (99.75)	21.02	0.70	81.9	263 (5500)	0.246	0.013	9.2

σ_v' = in situ overburden pressure; pc = preconsolidation pressure; Cc = Compression Index; Cr = Recompression Index;

OCR = Overconsolidation ratio

BLN COL 2.5-6

TABLE 2.5-236
HOEK-BROWN CRITERION INPUT AND OUTPUT FOR ROCK MASS FOUNDATION PROPERTIES

INPUTS	UCS (ksi)	GSI	mi	D	Ei (ksi)	OUTPUTS: Confining stress of 0.0524 ksi	Strength at Confining stress 0.0524 ksi: Rock Mass Shear Strength (ksi)	Mohr Envelope: c (ksi)	Mohr Envelope: Phi (degrees)	Rock mass Modulus: Erm (ksi)	Hoek Constants: a	Hoek Constants: s	Hoek Constants: mb	Rock mass Uniaxial Compressive strength (ksi)	Rock mass Global compressive strength (ksi)	Rock mass Tensile Strength (ksi)
Arg. Limestone; Mean	18.69	57	6	0.85	5383.71	Arg. Limestone; Mean	0.16	0.11	52	704.56	0.5	0.001	0.42	0.65	1.64	-0.06
Arg. Limestone; Upper Bound	27.58	61	7	0.85	5739	Arg. Limestone; Upper Bound	0.27	0.212	54	958.73	0.5	0.002	0.62	1.32	3.01	-0.11
Arg. Limestone; Lower Bound	9.8	52	5	0.85	5028.42	Arg. Limestone; Lower Bound	0.08	0.043	46	481.24	0.51	6E-04	0.25	0.23	0.66	-0.02
Micritic Limestone; Mean	21.55	60	8	0.85	7078.19	Micritic Limestone; Mean	0.20	0.144	56	1113.6	0.5	0.002	0.67	0.95	2.41	-0.07
Micritic Limestone; Upper Bound	24.14	63	9	0.85	8320.91	Micritic Limestone; Upper Bound	0.26	0.197	57	1562.83	0.5	0.003	0.9	1.35	3.18	-0.09
Micritic Limestone; Lower Bound	18.96	57	7	0.85	5835.48	Micritic Limestone; Lower Bound	0.02	0.105	53	763.68	0.5	0.001	0.48	0.66	1.79	-0.05

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TABLE 2.5-237
SEISMIC CPT SUMMARY

Seismic CPT Number	Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
CPT-1305	6.07	5.41	10.65		13.6			
	9.02	8.36	12.41	1.76	15.9	2.3	76.8	6.89
CPT-1309	6.07	5.41	10.65		5.6			
	20.01	19.35	21.42	10.77	25.2	19.6	549.5	9.8

Geophone Offset: 0.66 Feet

Source Offset: 9.17 Feet

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TABLE 2.5-238 (Sheet 1 of 2)
P-S/DOWNHOLE GEOPHYSICAL LOGGING SURVEY

Boring Number	Tool and Run Number	Depth Range (Feet)	Open Hole (Feet)	Depth to Bottom of Casing (Feet)	Sample Interval (Feet)	Date Logged
B-1005	SUSPENSION1	1.6–237.9	251.4	~11	1.6	7/10/2006
B-1005PS	SUSPENSION2	1.6–16.4	26.2	-	1.6	7/10/2006
B-1006	SUSPENSION	13.1–160.8	176	~13	1.6	7/11/2006
B-1032	SUSPENSION1	13.1–162.4	175.2	~21	1.6	7/10/2006
B-1032PS	SUSPENSION2	1.6–21.3	33.8	-	1.6	7/10/2006
B-1032	DOWNHOLE	4.5–170	175.2	~21	1.5 to 10	7/10/2006
B-1034	SUSPENSION1	14.8–237.9	250	14.6	1.6	7/11/2006
B-1034PS	SUSPENSION2	1.6–18.0	29.6	-	1.6	7/11/2006
B-1036A	SUSPENSION1	16.4–162.4	175.5	15.9	1.6	7/11/2006
B-1036	SUSPENSION2	1.6–26.2	55		1.6	7/11/2006
B-1059	SUSPENSION1	6.6–237.9	250	~12	1.6	5/25/2006
B-1059M	SUSPENSION2	3.3–21.3	150	NOTE: Data Not Used	1.6	5/25/2006
B-1059	DOWNHOLE	3–247	250	~12	1.5-10	5/26/2006

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TABLE 2.5-238 (Sheet 2 of 2)
P-S/DOWNHOLE GEOPHYSICAL LOGGING SURVEY

Boring Number	Tool and Run Number	Depth Range (Feet)	Open Hole (Feet)	Depth to Bottom of Casing (Feet)	Sample Interval (Feet)	Date Logged
B-1060	SUSPENSION1	3.3–164.0	176.3	~16	1.6	7/12/2006
B-1060PS	SUSPENSION2	3.3–19.7	30.3	15.8	1.6	7/12/2006
B-1067	SUSPENSION1	1.6–157.5	176.3	~7	1.6	7/12/2006
B-1067PS	SUSPENSION2	1.6–9.8	22.3	~7	1.6	7/12/2006

-casing not present

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TABLE 2.5-239
NATURAL GAMMA LOGGING SUMMARY

	DATE LOGGED	CALIPER	NATURAL GAMMA	OPTV	BHTV	COMMENT
B-1001	7/19/06	x	x	-	x	inclined borehole
B-1002	6/7/06	x	-	x	x	vertical borehole
B-1003	6/7/06	x	-	x	x	vertical borehole
B-1011	6/8/06	x	-	x	x	vertical borehole
B-1013	6/8/06	x	-	x	x	vertical borehole
B-1014	7/20/06	x	-	x	x	vertical borehole
B-1034	7/16/06	x	x	x	x	vertical borehole
B-1043	7/18/06	x	-	x	x	vertical borehole
B-1044	6/8/06	x	-	x	x	vertical borehole
B-1045	6/8/06	x	-	x	x	vertical borehole
B-1048	7/19/06	x	x	x	x	vertical borehole
B-1059	6/7/06	x	-	x	x	vertical borehole
B-1061	7/19/06	x	x	x	x	vertical borehole
B-1067	7/20/06	x	-	x	x	vertical borehole
B-1082	6/6/06	x	x	x	x	gamma log on 7-19-05
B-1086	7/20/06	x	x	x	x	vertical borehole
B-1096	7/20/06	-	x	-	-	vertical borehole

notes: x=completed geophysical log, -=not completed

BLN COL 2.5-7

TABLE 2.5-240
SUMMARY OF SOIL PROPERTIES - POWER BLOCK AREA

Material Type	Natural Moisture Content (%) Range (Avg.)	Liquid Limit (LL), % Range (Avg.)	Plasticity Index (PI), % Range (Avg.)	Maximum Dry Density Range /Average g/cm^3 (pcf)	Optimum Moisture Content (%)
CH	16.3-50.3 (24.5)	50-115 (68)	24-82 (42)	1.59-1.64 (99.5-102.2) [(a)] 1.97 (123.0) [(b)]	18.9-20.1 [(a)] 11.0 [(b)]
CL	3.8-29.4 (19.3)	24-49 (40)	9-28 (19)	1.91-1.96 (119.2-122.3) [(b)]	12.9-16.0 [(b)]
MH	17.4-23.4 (20.4)	54-59 (57)	23-25 (24)	N/A	N/A
ML	10.7	41	NP [(c)]	N/A	N/A
GC/GP	6.7-29.1 (17.9)	38-74 (56)	19-49 (34)	N/A	N/A

a) Standard Proctor (ASTM D698)

b) Modified Proctor (ASTM D1557)

c) NP = Non-plastic

d) Only one ML sample tested

BLN COL 2.5-1
BLN COL 2.5-8

TABLE 2.5-241 (Sheet 1 of 2)
GROUNDWATER DATA FROM BORINGS NEAR UNIT 3

		Top of Rock		Groundwater			
Boring #	Surface Elevation	Depth from surface (ft)	Elevation (ft)	Depth from surface (ft)	Elevation (ft)	Date of Record	Difference between Groundwater and Rock El[(a)] (ft)
B-1001-I	611.3	14.4	596.9	14.8	596.5	07/17/06	-0.4
				14.5	596.8	07/18/06	-0.1
B-1002	613.3	16.5	596.8	15.8	597.5	6/5/2006	0.7
				12.8	600.5	8/6/2006	3.7
B-1003	613.6	14.8	598.8	12.3	601.3	5/30/2006	2.5
				14.3	599.3	7/5/2006	0.5
B-1004	611.1	13.1	598	10.8	600.3	5/22/2006	2.3
				14.5	596.6	7/19/2006	-1.4
B-1005	608.9	11	597.9	13	595.9	6/10/2006	-2
				13.2	595.7	6/11/2006	-2.2
				11.2	597.7	8/4/2006	-0.2
B-1007	608.4	8.1	600.3	10.5	597.9	6/12/2007	-2.4

BLN COL 2.5-1
BLN COL 2.5-8

TABLE 2.5-241 (Sheet 2 of 2)
GROUNDWATER DATA FROM BORINGS NEAR UNIT 3

		Top of Rock		Groundwater			
Boring #	Surface Elevation	Depth from surface (ft)	Elevation (ft)	Depth from surface (ft)	Elevation (ft)	Date of Record	Difference between Groundwater and Rock El[(a)] (ft)
B-1014	609.1	6	603.1	12.2	596.9	8/6/2006	-6.2
B-1021	608.6	11.3	597.3	ND	ND	ND	ND
max			603.1	15.8	601.3		3.7
min			586.3	10.5	595.7		-6.2
average			597.8	13.1	597.9		-0.4

Notes:

ND= No data

a) Positive values mean groundwater level is above rock elevation and negative values mean it is below

BLN COL 2.5-1
BLN COL 2.5-8

TABLE 2.5-242 (Sheet 1 of 2)
GROUNDWATER DATA FROM BORINGS NEAR UNIT 4

		Top of Rock		Groundwater			Difference between Groundwater and Rock El[(a)] (ft)
Boring #	Surface Elevation	Depth from surface (ft)	Elevation (ft)	Depth from surface (ft)	Elevation (ft)	Date of Record	
B-1034	622	14	608	14.8	607.2	06/25/06	-0.8
				14.7	607.3	06/26/06	-0.7
				14.9	607.1	06/29/06	-0.9
				12.6	609.4	08/18/07	1.4
B1037	621.4	14	607.4	9.2	612.2	06/20/06	4.8
				14.3	607.1	06/21/06	-0.3
				13.4	608	08/17/06	0.6
B-1040	616.9	13.2	603.7	13.7	603.2	05/12/06	-0.5
				11.2	605.7	05/14/06	2
				9	607.9	06/28/06	4.2
				14.4	602.5	06/30/06	-1.2
				9	607.9	07/05/06	4.2
				7.2	609.7	07/14/06	6
				16	600.9	07/20/06	-2.8

BLN COL 2.5-1
BLN COL 2.5-8

TABLE 2.5-242 (Sheet 2 of 2)
GROUNDWATER DATA FROM BORINGS NEAR UNIT 4

Boring #	Surface Elevation	Top of Rock		Groundwater			Difference between Groundwater and Rock El[a] (ft)
		Depth from surface (ft)	Elevation (ft)	Depth from surface (ft)	Elevation (ft)	Date of Record	
B-1042-I	622	22.5	599.5	15.8	606.2	07/09/06	6.7
				15.1	606.9	07/11/06	7.4
				15.2	606.8	07/20/06	7.3
B1044	618.7	8	610.7	9.5	609.2	08/18/06	-1.5
B-1045	622.4	18.5	603.9	16.3	606.1	06/06/06	2.2
				16	606.4	07/20/06	2.5
B-1055	619.6	16	603.6	11.6	608	08/18/06	4.4
				11.8	607.8	08/20/06	4.2
				11.8	607.8	08/22/06	4.2
max			610.7	16.3	612.2		7.4
min			599.5	7.2	600.9		-2.8
average			605.3	12.9	607.0		2.3

a) Positive values mean groundwater level is above rock elevation and negative values mean it is below

BLN COL 2.5-1
BLN COL 2.5-8

TABLE 2.5-243
GROUNDWATER DATA FOR MONITORING WELLS TERMINATED IN SOIL

			Top of Rock[2]		Groundwater Record				
					Highest Value		Lowest Value		
General Location	Boring #	Surface Elevation	Depth from surface (ft)	Elevation (ft)	Elevation (ft)	Date of Record	Elevation (ft)	Difference between Highest Groundwater and Rock El (ft)	Difference between Lowest Groundwater and Rock El (ft)
	MW-1201 A	611.1	13.1	598	601.9	06/11/06	601.4	3.9	3.4
Unit 3	MW-1217 A	614.3	18.5	595.8	605.2	3/5/2007	<601.1 [3]	9.4	5.3
in-between Units 3 and Unit 4	MW-1203A	619.0	10.8	608.2	611.8	2/1/2007	<606.4 [3]	3.6	-1.8
	MW-1202A	614.99	14.99	600	610.7	1/11/2007	605.1	10.7	5.1
Unit 4	MW-1204A	620.45	10.25	610.2	614.7	3/5/2007	<607.5 [3]	4.5	-2.7
max				610.2	610.7		605.1	10.7	5.3
min				595.8	601.9		601.1	3.6	-2.7
average				602.4	608.9		604.5	6.4	1.9

Notes:
1. Recorded groundwater levels pertain to period of measurement from 6/11/06 to 5/8/07
2. Elevations of top of rock taken from adjacent borings:

MW-1201 -- B-1004 MW-1203 --B-1030 MW-1204 --B-1039

MW-1217 -- B-1006 MW-1202 --B-1083

3. Well was recorded to be dry for part of the year. Elevation shown is for the bottom of the well

BLN COL 2.5-8

TABLE 2.5-244
GROUNDWATER DATA FOR MONITORING WELLS TERMINATED IN ROCK

General Location	Boring #	Surface Elevation	Top of Rock[2] Depth from surface (ft)	Top of Rock[2] Elevation (ft)	Groundwater Record: Highest Value Elevation (ft)	Groundwater Record: Highest Value Date of Record	Groundwater Record: Lowest Value Elevation (ft)	Difference between Highest Groundwater and Rock El (ft)	Difference between Lowest Groundwater and Rock El (ft)
Unit 3	MW-1201 B	611.0	13.0	598	536.5		536.44	-61.5	-61.56
	MW-1201 C	610.9	12.9	598	496.5	10/26/06	496.5	-101.5	-101.5
	MW-1217 B	614.1	18.3	595.8	602.3	4/17/2007	599.20	6.5	3.4
	MW-1217 C	614.1	18.3	595.8	570.5		570.50	-25.3	-25.3
in-between Units 3 and Unit 4	MW-1203B	619.1	10.9	608.2	610.1		591.50	1.9	-16.7
	MW-1203C	619	10.8	608.2	515.5		504.10	-92.7	-104.1
Unit 4	MW-1202C	614.93	14.9	600	567.7		565.60	-32.3	-34.4
	MW-1204B	620.48	10.3	610.2	570.6		570.20	-39.6	-40
	MW-1204C	620.49	10.3	610.2	612.6		607.70	2.4	-2.5
max				610.2	612.6		607.7	6.5	3.4
min				595.8	496.5		496.5	-101.5	-104.1
average				602.7	564.7		560.2	-38.0	-42.5

Notes:
1. Recorded groundwater levels pertain to period of measurement from 6/11/06 to 5/8/07
2. Elevations of top of rock taken from adjacent borings:

MW-1201 -- B-1004	MW-1203 --B-1030	MW-1204 --B-1039
MW-1217 -- B-1006	MW-1202 --B-1083	MW-1205 --B-1093

3. Well was recorded to be dry for part of the year. Elevation shown is for the bottom of the well

BLN COL 2.5-6

TABLE 2.5-245 (Sheet 1 of 3)
RESONANT COLUMN TORSIONAL SHEAR LABORATORY RESULTS, PART 1

	Torsional Shear Tenth Cycle			Resonant Column Tests		
Sample Number	Cyclic Shear Strain %	G/Gmax	Damping Ratio %	Cyclic Shear Strain %	G/Gmax	Damping Ratio %
UTA-54-A 1.2ksf= 55kPa	8E-05	0.99	1.159	0.000108	1	3.01
	0.00016	1.01	1.183	0.00021	1	3.11
	0.00032	0.99	1.211	0.000415	0.99	3.24
	0.00056	0.98	1.269	0.000804	0.98	3.46
	0.00102	0.95	1.419	0.00143	0.95	3.76
	0.00213	0.9	1.712	0.00278	0.9	4.77
	0.0051	0.76	1.992	0.00514	0.82	6.39
	0.0124	0.59	2.511	0.00935	0.71	9.3
UTA-54-B 1.73ksf= 83kPa	0.00023	1	1.59	0.0001070	1.0000	2.640
	0.00045	1	1.48	0.0002970	1.0000	2.640
	0.0009	1	1.6	0.0005950	1.0000	2.650
	0.00181	1	1.66	0.0011900	1.0000	2.660
	0.00361	1	1.72	0.0023700	1.0000	2.680
	0.00735	0.99	1.82	0.0046900	0.9900	2.770
	0.0153	0.95	2.25	0.0086600	0.9800	2.960
				0.0164000	0.9500	3.700
				0.0284000	0.8600	4.070
				0.0421000	0.7700	5.290
				0.0747000	0.6100	7.930
				0.1500000	0.4000	11.500
				0.2270000	0.3100	15.340

BLN COL 2.5-6

TABLE 2.5-245 (Sheet 2 of 3)
RESONANT COLUMN TORSIONAL SHEAR LABORATORY RESULTS, PART 1

	Torsional Shear Tenth Cycle			Resonant Column Tests		
Sample Number	Cyclic Shear Strain %	G/Gmax	Damping Ratio %	Cyclic Shear Strain %	G/Gmax	Damping Ratio %
UTA-54-C 1.15ksf = 55kPa	0.0002	1	1.84	0.0000476	1.0000	2.980
	0.0004	1	1.9	0.0001240	1.0000	2.980
	0.00081	0.99	1.86	0.0002350	1.0000	3.020
	0.00164	0.97	1.94	0.0004690	1.0000	3.040
	0.00342	0.95	2.19	0.0009330	0.9900	3.070
	0.007	0.91	2.75	0.0018500	0.9900	3.130
	0.0159	0.8	4.41	0.0036100	0.9800	3.250
				0.0065000	0.9400	3.520
				0.0121000	0.8800	4.150
				0.0213000	0.7800	5.680
				0.0345000	0.6500	7.380
				0.0642000	0.4800	10.160
				0.1170000	0.3200	12.490
				2.72E-01	0.1500	15.600
UTA-54-D 1.73ksf = 83kPa	0.00055	1	1.42	0.0001660	1.0000	2.040
	0.00109	1	1.43	0.0004480	1.0000	2.070
	0.00218	1	1.49	0.0008090	1.0000	2.110
	0.00438	1	1.52	0.0016200	1.0000	2.130
	0.00881	0.99	1.58	0.0032300	1.0000	2.160
	0.0181	0.96	1.81	0.0064300	0.9900	2.190
	0.0408	0.86	2.99	0.0127000	0.9800	2.350
				0.0226000	0.9500	2.370
				0.0406000	0.8800	3.010
				0.0693000	0.7400	4.430
				0.1050000	0.6000	5.330

BLN COL 2.5-6

TABLE 2.5-245 (Sheet 3 of 3)
RESONANT COLUMN TORSIONAL SHEAR LABORATORY RESULTS, PART 1

	Torsional Shear Tenth Cycle			Resonant Column Tests		
Sample Number	Cyclic Shear Strain %	G/Gmax	Damping Ratio %	Cyclic Shear Strain %	G/Gmax	Damping Ratio %
UTA-54-E 1.15ksf= 55kPa	0.0004	1	1.95	0.000226	1	3.24
	0.0008	1	2.13	0.000459	1.01	3.21
	0.0016	1	2.03	0.000916	1	3.31
	0.00322	0.99	2.12	0.00183	1	3.33
	0.00651	0.98	2.23	0.00363	1	3.58
	0.0135	0.95	2.54	0.00719	0.99	3.62
	0.03	0.86	3.82	0.0132	0.97	3.76
				0.0247	0.91	4.12
				0.0441	0.79	5.5
				0.0672	0.67	7.16
				0.119	0.5	10.17
				0.227	0.3	12.15
UTA-54-F 1.15ksf= 55kPa	0.00016	1	1.18	0.0000137	1.01	2.78
	0.00033	1	1.48	0.0000288	1	2.79
	0.00065	1	1.56	0.0000579	1	2.76
	0.00131	0.99	1.6	0.000113	1	2.8
	0.00266	0.98	1.85	0.000207	1	2.82
	0.00561	0.93	2.41	0.000412	1	2.92
	0.0125	0.83	3.74	0.000816	0.99	2.93
				0.00161	0.99	2.9
				0.0031	0.97	3.02
				0.00555	0.93	3.31
				0.0103	0.88	4.04
				0.0177	0.77	5.42
				0.0273000	0.6600	6.750
				0.0527000	0.4900	8.460
				0.1010000	0.3200	10.310

BLN COL 2.5-6

TABLE 2.5-246
RESONANT COLUMN TORSIONAL SHEAR LABORATORY RESULTS, PART 2

Sample	Boring Number	Sample Depth	Confining Pressure	Vs at 1x v (fps)	Vs at 4x v (fps)	Plasticity Index
UTA-54-A	B-1036	9.4-10	1.2 ksf	1122	1362	22
UTA-54-B	B-1017	14.5-14.8	1.7ksf	856	897	50
UTA-54-C	B-1092	10-Aug	1.15ksf	940	1034	24
UTA-54-D	B-1017	17.6-18	1.73ksf	588	656	43
UTA-54-E	B-1006	9.6-10	1.15ksf	642	664	28
UTA-54-F	B-1048	9.6-10	1.15ksf	1011	1111	35

APPENDIX 2AA
EARTHQUAKE CATALOG

BLN COL 2.5-1

BLN COL 2.5-2 The updated earthquake catalog prepared for the project constitutes this appendix. The development of this catalog is described in Subsection 2.5.2.1. This catalog was used to select the final catalog of earthquakes occurring within ~200 mi of the BLN site. The catalog data is unchanged from its original form in the TVA 2006 GG&S Report (Reference 399 of Section 2.5), Appendix I, Table 1-1.

The headings for the data in the table are described below:

ID No - Project assigned identification number

Date - Year, Month, Day in Coordinated Universal Time (UTC)

Time - Hour, Minute, Second in Coordinated Universal Time (UTC)

Lat - Latitude (•North)

Long - Longitude (•West negative)

Dep (km) - Hypocentral depth in kim

GG&S mb* - m_b adjusted for bias due to uncertainty

GG&S mb - m_b

EPRI-SOG rmb - m_b adjusted for bias due to uncertainty

EPRI-SOG emb - m_b

GG&S Sig mb - Uncertainty in m_b (Sigma m_b)

GG&S Type - Category for earthquakes:

- EPRI, from EPRI-SOG (1988)
- Added, newly identified earthquakes added to EPRI-SOG catalog (occurring from 1758 to February 1985)
- Post, earthquakes occurring post-EPRI-SOG catalog (May, 1985 to February, 2005)

EPRI-SOG-UNID - EPRI assigned identification number

EPRI-SOG Lat - Latitude (•North)

EPRI-SOG Long - Longitude (•West negative)

EPRI-SOG-IO - Maximum intensity

EPRI-SOG Flag - earthquake dependency:

- MAIN, mainshock with dependent events;
- blank, mainshock with no associated dependent events;
- [number], EPRI UNID of mainshock

GG&S R (km) - Distance from Bellefonte GG&S site in km

APPENDIX 2BB
GEOTECHNICAL BORING LOGS

BLN COL 2.5-1
BLN COL 2.5-5
BLN COL 2.5-6

This appendix contains geotechnical boring logs that are the basis for discussion in relevant sections of 2.5. The logs are of soil and rock borings and represent a record of subsurface conditions at the BLN site.

The appendix contains the logs of 122 borings and key to symbols and descriptions of rock logs, soil logs and, general notes.

APPENDIX 2CC
TEST PIT LOGS

BLN COL 2.5-6 This appendix contains the entire set of test pit logs and material descriptions for the BLN site. This data was provided by MACTEC.