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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 Section 2.1.

2.0 SITE CHARACTERISTICS

LNP SUP 2.0-1 Chapter 2 describes the characteristics and site-related design parameters of Levy Nuclear Plant Units 1 and 2 (LNP). The site location, characteristics, and parameters, as described in the following sections, are provided in sufficient detail to support a safety assessment of the proposed site:

- FSAR Section 2.1 — Geography and Demography
- FSAR Section 2.2 — Nearby Industrial, Transportation, and Military Facilities
- FSAR Section 2.3 — Meteorology
- FSAR Section 2.4 — Hydrologic Engineering
- FSAR Section 2.5 — Geology, Seismology, and Geotechnical Engineering

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

LNP site. An irregularly shaped area that will be comprised of the following site components: the plant site, the pipeline corridor, transmission line corridors, site access roads, and the intake structure and pumphouse. The LNP site is located within Levy County (Figure 2.1.1-201). The LNP site is approximately 1257 hectares (ha) (3105 acres [ac.]) in size.

Vicinity. The area from the centerpoint of the LNP power block footprint to a 9.7-km (6-mi.) radius. The vicinity includes a much larger tract of land than the LNP site. The vicinity is located within Levy, Citrus, and Marion counties. For discussions within FSAR Section 2.5, vicinity is defined in accordance with RG 1.208 as a 40-km (25-mi.) radius.

Region. The area from the centerpoint of the LNP power block footprint to an 80-km (50-mi.) radius. The LNP site is located in a rural, sparsely populated area. For discussions within FSAR Section 2.5, region is defined in accordance with RG 1.208 as a 320 km (200 mi.) radius.

Table 2.0-201 provides a comparison of site-related design parameters for which the AP1000 plant is designed and site characteristics specific to the LNP 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 of the LNP. The fourth column denotes the section or table in the LNP FSAR where these data are presented. The last column indicates whether or not the site characteristic is bounded by the AP1000 DCD site parameters. "Yes" indicates the site characteristic falls within the parameter, while "No" indicates it does not. Where a "No" is indicated, justification is provided in the FSAR reference. Control room atmospheric dispersion values, expressed as Chi/Q for all applicable accident analyses, are presented in Table 2.0-202. All of the control room values fall within the AP1000 DCD Acceptance Criteria.

LNP SUP 2.0-1

Table 2.0-201 (Sheet 1 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Air Temperature				
Maximum Safety [a]	115°F dry bulb / 86.1°F coincident wet bulb	105.1°F dry bulb / 78.7°F coincident wet bulb (Tallahassee); 104.4°F dry bulb / 82.3°F coincident wet bulb (Jacksonville). Values are 100-year return estimates of 2-hour duration, 0% exceedance values.	FSAR Subsection 2.3.1.2.7	Yes
	86.1°F wet bulb (non-coincident) [h]	85.5°F wet bulb (non-coincident) (Tampa, 100-year return estimate of 2-hour duration, 0% exceedance values).	FSAR Subsection 2.3.1.2.7	Yes
Minimum Safety [a]	-40°F	3°F (Tallahassee,100-year return period)	FSAR Subsection 2.3.1.2.7	Yes
Maximum Normal [b]	101°F dry bulb / 80.1°F coincident wet bulb	95°F dry bulb / 78°F coincident wet bulb (Jacksonville)	FSAR Subsection 2.3.1.2.7	Yes
	80.1°F wet bulb (non-coincident) [c]	80°F wet bulb (non-coincident) (Tampa).	FSAR Subsection 2.3.1.2.7	Yes
Minimum Normal [b]	-10°F	24°F (Tallahassee)	FSAR Subsection 2.3.1.2.7	Yes
Wind Speed				
Operating Basis	145 mph (3-second gust); importance factor 1.15 (safety), 1.0 (non-safety); exposure C; topographic factor 1.0	120 mph (3-second gust, 50-year recurrence)(importance factor 1.0 [non-safety]; exposure C; topographic factor 1.0) 128 mph (3-second gust, 100-year recurrence)(importance factor 1.15 [safety]; exposure C; topographic factor 1.0).	FSAR Subsection 2.3.1.2.2	Yes
Tornado	300 mph	300 mph	FSAR Subsection 2.3.1.2.2	Yes
	Maximum pressure differential of 2 lb/in^2	2 lb/in^2	FSAR Subsection 2.3.1.2.2	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 2 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Seismic				
CSDRS	CSDRS free field peak ground acceleration of 0.30 g with modified Regulatory Guide 1.60 response spectra (see Figures 5.0-1 and 5.0-2.). The SSE is now referred to as CSDRS. Seismic input is defined at finished grade except for sites where the nuclear island is founded on hard rock. If the site-specific spectra exceed the response spectra in Figures 5.0-1 and 5.0-2 at any frequency, or if soil conditions are outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed. This evaluation will consist of a site-specific dynamic analysis and generation of in-structure response spectra at key locations to be compared with the floor response spectra of the certified design at 5 percent damping. The site is acceptable if the floor response spectra from the site-specific evaluation do not exceed the AP1000 spectra for each of the locations or the exceedances are justified. The HRHF envelope response spectra are shown in Figure 5.0-3 and Figure 5.0-4 defined at the foundation level for 5 percent damping. The HRHF envelope response spectra provide an alternative set of spectra for evaluation of site-specific GMRS. A site is acceptable if its site-specific GMRS falls within the AP1000 HRHF envelope response spectra. Evaluation of a site for application of the HRHF envelope response spectra includes consideration of the limitation on shear wave velocity identified for use of the HRHF envelope response spectra. This limitation is defined by a shear wave velocity at the bottom of the basemat equal to or higher than 7,500 ft/sec, while maintaining a shear wave velocity equal to or above 8,000 ft/sec at the lower depths. [d]	Peak ground acceleration: 0.069 g horizontal 0.051 g vertical GMRS peak ground acceleration defined at 100 Hz. Ground Response Spectra: At LNP 1 and LNP 2: The horizontal and vertical GMRS are bounded by the CSDRS (Figure 2.5.2-296).	FSAR Subsections 2.5.2.6 and 3.7	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 3 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Fault Displacement Potential	No potential fault displacement considered beneath the seismic Category I and seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the seismic Category I and seismic Category II structures.	The potential for tectonic deformation at the LNP site is negligible.	FSAR Subsection 2.5.3.8	Yes
Soil				
Average Allowable Static Bearing Capacity	The allowable bearing capacity, including a factor of safety appropriate for the design load combination, shall be greater than or equal to the average bearing demand of 8,900 lb/ft^2 over the footprint of the nuclear island at its excavation depth.	Allowable Static Bearing Capacity for LNP 1 and LNP 2: 108,000 psf	FSAR Subsection 2.5.4.10.1.2	Yes
Dynamic Bearing Capacity for Normal Plus SSE	The allowable bearing capacity, including a factor of safety appropriate for the design load combination, shall be greater than or equal to the maximum bearing demand of 35,000 lb/ft^2 at the edge of the nuclear island at its excavation depth, or Site-specific analyses demonstrate factor of safety appropriate for normal plus safe shutdown earthquake loads.	Allowable Dynamic Bearing Capacity for LNP 1 and LNP 2: 108,000 psf	FSAR Subsection 2.5.4.10.1.2	Yes
Shear Wave Velocity	Greater than or equal to 1,000 ft/sec based on minimum low-strain soil properties over the footprint of the nuclear island at its excavation depth.	Materials below nuclear island subgrades have V_S greater than 1000 ft/sec.	FSAR Subsection 2.5.4.4.2	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 4 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Lateral Variability	Soils supporting the nuclear island should not have extreme variations in subgrade stiffness. This may be demonstrated by one of the following: 1. Soils supporting the nuclear island are uniform in accordance with Regulatory Guide 1.132 if the geologic and stratigraphic features at depths less than 120 feet below grade can be correlated from one boring or sounding location to the next with relatively smooth variations in thickness or properties of the geologic units, or 2. Site specific assessment of subsurface conditions demonstrates that the bearing pressures below the nuclear island do not exceed 120% of those from the generic analyses of the nuclear island at a uniform site, or 3. Site specific analysis of the nuclear island basemat demonstrates that the site specific demand is within the capacity of the basemat. As an example of sites that are considered uniform, the variation of shear wave velocity in the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint and 40 feet beyond the boundaries of the nuclear island footprint meets the criteria in the case outlined below:	The nuclear islands will be founded on a 10.7 m (35 ft.) thick roller compacted concrete (RCC) bridging mat, overlaying the Avon Park Formation. Average V_s is greater than 2500 ft/sec for every layer below the nuclear island. LNP 1: Dip is approximately 2 degrees. Beneath the RCC bridging mat, one geologic unit is uniformly present to depths beyond 120 ft. below grade. This is consistent across all boreholes within the nuclear island footprint. Properties, particularly shear wave velocity, can vary within the geologic unit, but they vary smoothly and by less than 15 percent between boreholes. Because of the presence of the 10.7 m (35 ft.) thick RCC bridging mat, and the relative uniformity of the geologic unit below the RCC bridging mat, the site specific demand is within the capacity of the AP1000 basemat.	FSAR Subsection 2.5.4.10.3 FSAR Subsection 2.5.4.4.2.1.1 FSAR Subsection 2.5.4.4.2.1.2	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 5 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
	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.	LNP 2: Dip is approximately 2 degrees. Beneath the RCC bridging mat, one geologic unit is uniformly present to depths beyond 120 ft. below grade. This is consistent across all boreholes within the nuclear island footprint. Properties, particularly shear wave velocity, can vary within the geologic unit, but they vary smoothly and by less than approximately 20 percent between boreholes. Because of the presence of the 10.7 m (35 ft.) thick RCC bridging mat, and the relative uniformity of the geologic unit below the RCC bridging mat, the site specific demand is within the capacity of the AP1000 basemat.		
Liquefaction Potential	No liquefaction considered beneath the seismic Category I and seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the seismic Category I and seismic Category II structures.	Material beneath and adjacent to the nuclear island will be non-liquefiable. Some of the material in the passive resistance wedge, adjacent to the nuclear island, will be removed and replaced. Roller Compacted Concrete is used to support the nuclear island and is a zero-slump concrete with high flyash content, compacted by vibratory rollers. This material is non-liquefiable. Surface soils adjacent to the nuclear island will be removed or improved. Adjacent structures are supported on deep foundations.	FSAR Subsection 2.5.4.8	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 6 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters		LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Minimum Soil Angle of Internal Friction	The minimum soil angle of internal friction is greater than or equal to 35 degrees below the footprint of nuclear island at its excavation depth. If the minimum soil angle of internal friction is below 35 degrees, a site-specific analysis shall be performed using the site-specific soil properties to demonstrate stability.		Not applicable: Soils beneath the foundation for the nuclear islands will be excavated and replaced with RCC. A waterproofing membrane will be located between the RCC and the mudmat, meeting AP1000 DCD requirements of ≥0.55 static coefficient of friction.	Not applicable	Not applicable
Limits Of Acceptable Settlement Without Additional Evaluation [i]	Differential Across Nuclear Island Foundation Mat	0.5 in. in 50 ft.	<0.25 in. in 50 ft. (projected)	FSAR Subsection 2.5.4.10.3	Yes (projected)
	Total for Nuclear Island Foundation Mat	6 in.	< 1 in. (projected)		
	Differential Between Nuclear Island and Turbine Building [j]	3 in.	< 1 in. (projected)		
	Differential Between Nuclear Island and Other Buildings [j]	3 in.	< 1 in. (projected)		
Missiles					
Tornado	4000-lb. automobile at 105 mph horizontal, 74 mph vertical		4000-lb. automobile at 105 mph horizontal, 74 mph vertical	DCD Subsection 3.5.1.4	Yes
	275-lb., 8-in. shell at 105 mph horizontal, 74 mph vertical		275-lb., 8-in. shell at 105 mph horizontal, 74 mph vertical	DCD Section 3.5	Yes
	1-in.-diameter steel ball at 105 mph in the most damaging direction		1-in.-diameter steel ball at 105 mph in the most damaging direction	APP-GW-GLR-020, “Wind and Tornado Site Interface Criteria,” Westinghouse [e]	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 7 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Flood Level	Less than plant elevation 100 ft.	DCD plant elevation of 100 ft. = 51 ft. NAVD88 or 52 ft. NGVD29 (nominal plant grade floor elevation)	FSAR Subsection 2.4.1.1	Yes
		The maximum water elevation in Lake Rousseau from a PMF is 29.7 ft. NAVD88.	FSAR Subsection 2.4.3.5	
		The maximum water surface elevation in the lower Withlacoochee River associated with a postulated failure of the Inglis Dam during a PMF is 24.65 ft. NGVD29.	FSAR Subsection 2.4.3.6	
		The maximum total (surge and wave action) water elevation from a PMH is 49.78 ft. NAVD88 or 50.78 ft. NGVD29.	FSAR Subsection 2.4.5.4.9	
Groundwater Level	Less than plant elevation 98 ft.	DCD groundwater elevation of 98 ft. = 49 ft. NAVD88 or 50 ft. NGVD29.	FSAR Subsection 2.4.12.5	Yes
		Surficial monitoring wells MW-15S (LNP 1) and MW-13S (LNP 2) recorded groundwater elevations (March, June, September, and December 2007), which ranged from 37.88 to 42.05 ft. NAVD88 and 37.66 to 41.94 ft. NAVD88, respectively.		

LNP SUP 2.0-1

Table 2.0-201 (Sheet 8 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

	AP 1000 DCD Site Parameters	LNP Site Characteristics	LNP Site Characteristic Reference	Bounding Yes/No
Plant Grade Elevation	Less than plant elevation 100 ft., except for portion at a higher elevation adjacent to the annex building	The nominal plant grade floor elevation is 51 ft. NAVD88 or 52 ft. NGVD29, which corresponds to AP1000 elevation of 100 ft. The actual plant grade will be lower and will vary to accommodate site grading, drainage, and local site flooding. Therefore, DCD plant elevation of 100 ft. = 51 ft. NAVD88 or 52 ft. NGVD29.	FSAR Subsection 2.4.1.1	Yes
Precipitation				
Rain	20.7 in./hr [1-hr 1-mi^2 PMP]	19.6 in./hr	FSAR Subsection 2.4.2.3	Yes
Snow / Ice	75 lb/ft^2 on ground with exposure factor of 1.0 and important factor of 1.2 (safety) and 1.0 (non-safety)	The 50-year recurrent Ground Snow Load for all monitoring stations is zero; therefore, estimations of the weight of snowpack are not necessary for the LNP site.	FSAR Subsection 2.3.1.2.3	Yes
Atmospheric Dispersion Values X/Q (f)				
Site Boundary (0-2 hours)	≤ 5.1 x 10^{-4} sec/m^3	5.08 x 10^{-4} sec/m^3	Table 2.3.4-201	Yes
Site Boundary (annual average)	≤ 2.0 x 10^{-5} sec/m^3	1.90 x 10^{-5} sec/m^3	Table 2.3.4-201	Yes
Low population zone boundary				Yes
0-8 hours	≤ 2.2 x 10^{-4} sec/m^3	9.70 x 10^{-5} sec/m^3	Table 2.3.4-201	Yes
8-24 hours	≤ 1.6 x 10^{-4} sec/m^3	7.19 x 10^{-5} sec/m^3	Table 2.3.4-201	Yes
24-96 hours	≤ 1.0 x 10^{-4} sec/m^3	3.75 x 10^{-5} sec/m^3	Table 2.3.4-201	Yes
96-720 hours	≤ 8.0 x 10^{-5} sec/m^3	1.48 x 10^{-6} sec/m^3	Table 2.3.4-201	Yes
Population Distribution				
Exclusion area (site)(g)	0.5 miles	The minimum distance from the effluent release boundary to the exclusion area boundary is 1340 m (4396 ft. or 0.83 mi.), except for LNP 1's ESE sector which has a minimum distance of 1247 m (4091 ft. or 0.77 mi.).	FSAR Subsection 2.1.1.2	Yes

LNP SUP 2.0-1

Table 2.0-201 (Sheet 9 of 9)
Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics

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

b) The maximum normal value is the 1-percent seasonal exceedance temperature. The minimum normal value is the 99-percent seasonal exceedance temperature. The minimum temperature is for the months of December, January, and February in the northern hemisphere. The maximum temperature is for the months of June through September in the northern hemisphere. The 1-percent seasonal exceedance is approximately equivalent to the annual 0.4-percent exceedance. The 99-percent seasonal exceedance is approximately equivalent to the annual 99.6-percent exceedance.

c) The non-coincident wet bulb temperature is applicable to the cooling tower only.

d) With ground response spectra as given in Figures 3.7.1-1 and 3.7.1-2 of the AP1000 DCD.

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 more conservative design.

f) For AP1000, the terms “site boundary” and “exclusion area boundary” are used interchangeably. Thus, the X/Q values specified for the site boundary applies whenever a discussion refers to the exclusion area boundary.

g) Exclusion area (site) for the LNP is defined as two overlapping circles centered on the reactor building of each unit. The radius of each circle is 1340 m (4396 ft.). The overall shape of the LNP exclusion area boundary is defined by the outermost boundary of each unit’s circle. The EAB for LNP 1 was modified in the east-southeast direction. Atmospheric dilution factor (Chi/Q) calculations support the modification of the EAB to follow the property line in the east-southeast sector.

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

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

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

°F = degrees Fahrenheit
CSDRS = certified seismic design spectra
FIRS = foundation input response spectrum
ft. = foot
ft/sec = feet per second
g = unit of measure of acceleration of gravity
GMRS = ground motion response spectrum
HRHF = hard rock high frequency
Hz = hertz
in. = inch
lb. = pound
lb/ft^2 = pound per square foot
lb/in^2 = pound per square inch
lb/m^2 = pound per square meter
m = meter
mph = miles per hour
NAVD88 = North American Vertical Datum of 1988
NGVD29 = National Geodetic Vertical Datum of 1929
PMF = probable maximum flood
PMH = probably maximum hurricane
PMP = probable maximum precipitation
RCC = roller compacted concrete
sec/m^3 = seconds per cubic meter
SSE = safe shutdown earthquake
V_S = shear wave velocity

LNP SUP 2.0-1

Table 2.0-202 (Sheet 1 of 2)
Comparison of Predicted LNP Control Room Chi/Q Values with AP1000 DCD Acceptance Criteria

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

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	**Ground Level Contain-ment Release Points[(c)]**	Ground Level Contain-ment Release Points	**PORV and Safety Valve Releases[(d)]**	PORV and Safety Valve Releases	**Condenser Air Removal Stack[(g)]**	Condenser Air Removal Stack	**Steam Line Break Releases**	Steam Vent	**Fuel Handling Area[(e)]**	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	LNP	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	LNP
0 - 2 hours	**3.0E-03**	1.7E-03	1.5E-03	**6.0E-03**	4.3E-03	**2.0E-02**	1.0E-02	**6.0E-03**	1.7E-03	**2.4E-02**	1.1E-02	**6.0E-03**	1.3E-03	1.0E-03
2 - 8 hours	**2.5E-03**	1.0E-03	8.4E-04	**3.6E-03**	3.5E-03	**1.8E-02**	5.7E-03	**4.0E-03**	1.4E-03	**2.0E-02**	6.1E-03	**4.0E-03**	8.3E-04	6.4E-04
8 - 24 hours	**1.0E-03**	4.5E-04	3.7E-04	**1.4E-03**	1.2E-03	**7.0E-03**	2.7E-03	**2.0E-03**	6.4E-04	**7.5E-03**	3.0E-03	**2.0E-03**	3.7E-04	3.0E-04
1 - 4 days	**8.0E-04**	4.5E-04	3.8E-04	**1.8E-03**	1.2E-03	**5.0E-03**	2.1E-03	**1.5E-03**	5.9E-04	**5.5E-03**	2.3E-03	**1.5E-03**	3.4E-04	2.6E-04
4 - 30 days	**6.0E-04**	3.6E-04	3.0E-04	**1.5E-03**	9.9E-04	**4.5E-03**	1.3E-03	**1.0E-03**	4.7E-04	**5.0E-03**	1.5E-03	**1.0E-03**	2.7E-04	2.0E-04

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

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	**Ground Level Contain-ment Release Points[(c)]**	Ground Level Contain-ment Release Points	**PORV and Safety Valve Releases[(d)]**	PORV and Safety Valve Releases	**Condenser Air Removal Stack[(g)]**	Condenser Air Removal Stack	**Steam Line Break Releases**	Steam Vent	**Fuel Handling Area[(e)]**	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	LNP	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	LNP
0 - 2 hours	**1.0E-03**	3.7E-04	3.8E-04	**1.0E-03**	3.4E-04	**4.0E-03**	8.3E-04	**2.0E-02**	3.2E-03	**4.0E-03**	8.0E-04	**6.0E-03**	3.3E-04	3.2E-04
2 - 8 hours	**7.5E-04**	2.4E-04	2.5E-04	**7.5E-04**	2.8E-04	**3.2E-03**	4.8E-04	**1.8E-02**	1.8E-03	**3.2E-03**	4.7E-04	**4.0E-03**	2.2E-04	2.1E-04
8 - 24 hours	**3.5E-04**	1.1E-04	1.1E-04	**3.5E-04**	1.3E-04	**1.2E-03**	2.3E-04	**7.0E-03**	7.8E-04	**1.2E-03**	2.2E-04	**2.0E-03**	1.0E-04	1.0E-04
1 - 4 days	**2.8E-04**	1.1E-04	1.1E-04	**2.8E-04**	1.2E-04	**1.0E-03**	2.2E-04	**5.0E-03**	6.9E-04	**1.0E-03**	2.1E-04	**1.5E-03**	9.8E-05	9.4E-05
4 - 30 days	**2.5E-04**	8.9E-05	9.1E-05	**2.5E-04**	1.0E-04	**8.0E-04**	1.8E-04	**4.5E-03**	5.3E-04	**8.0E-04**	1.8E-04	**1.0E-03**	7.8E-05	7.5E-05

LNP SUP 2.0-1

Table 2.0-202 (Sheet 2 of 2)
Comparison of Predicted LNP Control Room Chi/Q Values with AP1000 DCD Acceptance Criteria

Notes:

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

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

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

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

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

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 handling area relief panel opens on high temperature. These dispersion factors are used for the fuel handling accident occurring outside containment and for evaluating the impact of releases associated with spent fuel pool boiling.

g) 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.

Chi/Q = atmospheric dilution factor
HVAC = heating, ventilation, and air conditioning
sec/m^3 = second per cubic meter

2.1 GEOGRAPHY AND DEMOGRAPHY

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

STD DEP 1.1-1 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.

2.1.1 SITE LOCATION AND DESCRIPTION

LNP COL 2.1-1

2.1.1.1 Specification of Location

The Levy Nuclear Plant Units 1 and 2 (LNP) site is located in Levy County, Florida (Figure 2.1.1-201). This is a large, primarily rural area located southwest of Gainesville and west of Ocala and approximately 15.5 kilometers (km) (9.6 miles [mi.]) northeast of the Crystal River Energy Complex, an energy facility also owned by Florida Power Corporation doing business as Progress Energy Florida, Inc. (PEF) (Figure 2.1.1-201). While there are small communities and clusters of homes in the region, the area is sparsely populated. As shown on Figure 2.1.1-201, the nearest towns from the LNP site are Inglis and Yankeetown, which are located 6.6 km (4.1 mi.) southwest and 12.9 km (8.0 mi.) southwest from the site, respectively; the Gulf of Mexico is located approximately 12.8 km (7.9 mi.) west of the LNP site, and Lake Rousseau lies about 4.8 km (3.0 mi.) to the south.

The LNP site is approximately 1257 hectares (ha) (3105 acres [ac.]). Much of the LNP site, in particular the reactor locations, has been in intensive silviculture production for over a century.

The reactor building and generating facilities would lie within a nuclear exclusion area, to which access would be controlled. Table 2.1.1-201 describes the location of each reactor building. The Levy Nuclear Plant Unit 1 (LNP 1) is the southernmost reactor and the Levy Nuclear Plant Unit 2 (LNP 2) is the northernmost reactor. The site is located on the Yankeetown SE U.S. Geological Survey (USGS) quadrangle map 7.5-minute series (Reference 2.1-201). Table 2.1.1-202 lists this and the USGS quadrangle maps of the surrounding area.

2.1.1.2 Site Area Map

The LNP site is shown on Figure 2.1.1-202, Exclusion Area Boundary (EAB) and Low Population Zone (LPZ), and Figure 2.1.1-203, LNP Exclusion Area Boundary Plan. These plans show the principal plant structures, the exclusion area, and the major roads and transportation routes in the area. No private, residential, industrial, institutional, or commercial structures are located currently on-site. The LNP site area is described in Final Safety Analysis Report (FSAR) Subsection 2.1.1.1. The plant property line is shown as the site boundary on Figure 2.1.1-203.

The LNP EAB, as shown on Figures 2.1.1-202 and 2.1.1-203, is defined as two overlapping circles centered on the reactor building of each unit. The radius of each circle is 1340 meters (m) (4396 feet [ft.]). The overall shape of the LNP EAB is defined by the outermost boundary of each unit's circle. The EAB for LNP 1 was modified in the southeast direction. Atmospheric dilution factor (Chi/Q) calculations provided in FSAR Section 2.3 support the modification of the EAB to follow the property line in this quadrant. Figures 2.1.1-202 and 2.1.1-203 show the modified EAB with respect to the property boundary. The LNP site is located within a much larger tract of land that includes the LNP EAB and surrounding lands.

The major highway located near the LNP site is U.S. Highway 98/U.S. Highway 19. Figure 2.1.1-202 shows the transportation routes in the region of the LNP site. Interstate 75 (I-75) is the closest interstate, which is located approximately 45 km (28 mi.) to the east of the LNP site. At its nearest point, U.S. Highway 98/U.S. Highway 19 is located approximately 1974 m (6477 ft.) from the center of the LNP site (Figure 2.1.1-203).

No active railroads are located within the 8-km (5-mi.) radius of the LNP site. Two railroad lines, an abandoned track, and an active commercial line are located within 16 km (10 mi.) of the LNP site. FSAR Section 2.2 describes these lines in further detail.

The Withlacoochee River is located south of the LNP site and extends in an east-west direction (Figure 2.1.1-201). The river is not used for commercial traffic and is classified by the Florida Department of Environmental Protection (FDEP) as an outstanding surface water body (Reference 2.1-202). The Cross Florida Barge Canal (CFBC) and the CFBC By-Pass Spillway are located approximately 5.2 km (3.2 mi.) to the south of the LNP site (shown on FSAR Figure 2.2.2-201). The Lower Withlacoochee River, from the Gulf of Mexico to the CFBC By-Pass Spillway, is classified by FDEP as "special waters" under the outstanding Florida waters category and is used for recreation (References 2.1-203 and 2.1-204).

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The boundary lines of the restricted area of a nuclear power plant include the protected area (as defined in 10 Code of Federal Regulations [CFR] 20.1003). The protected area would be a fenced area surrounding the power block. The protected area would be guarded and access granted only to authorized personnel. See FSAR Section 13.6 and the Security Plan for additional information.

The 10 CFR 20 “Standards for Protection Against Radiation” and Appendix I to 10 CFR 50 describe effluent release limits to ensure that 1) the concentrations of radionuclides in gaseous effluent at the EAB do not exceed the limits set forth in Table 2, Column 1 of Appendix B to 10 CFR 20; 2) the annual average concentrations of radionuclides in liquid effluent at the point of discharge do not exceed the limits set forth in Table 2, Column 2 of Appendix B to 10 CFR 20; and 3) the cumulative liquid and gaseous radionuclide releases do not result in exposures to individuals outside the EAB in excess of the limits set forth in Appendix I to 10 CFR 50. Figure 2.1.1-203 shows the combined EABs for LNP 1 and LNP 2, and Table 2.1.1-203 shows the distance from the centerpoint of the LNP site to the LNP EAB for each major compass direction. Because of the shape of the overlapping EABs, the distance from the centerpoint of the LNP site to the outermost boundary of the LNP EAB ranges from 1341 to 1493 m (4398 to 4897 ft.).

The Liquid and Gaseous Waste Processing Systems are discussed in FSAR Sections 11.2 and 11.3, respectively. These radioactive releases are within the limits set forth in 10 CFR 20 and 10 CFR 50.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

Figure 2.1.1-202 shows the LNP EAB and LPZ. As defined by 10 CFR 100.21(a), this subsection describes the applicant’s legal rights to establish authority to determine all activities, including exclusion and removal of personnel and property from the area. The EAB is defined by 10 CFR 100.3, as “the area surrounding the reactor, in which the reactor licensee has the authority to determine all activities including exclusion or removal of personnel and property from the area.” All lands within the EAB are owned by PEF. The LPZ is defined by 10 CFR 100.3, “as the area immediately surrounding the EAB which contains residents, the total number and density of which are such that there is a reasonable probability that appropriate protective measures could be taken in their behalf in the event of a serious accident.”

No mineral rights have been leased within the EAB, and there are no surface or subsurface rights for mineral mining associated with the LNP site.

2.1.2.2 Control of Activities Unrelated to Plant Operation

According to the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.206, there are no commercial activities located within the EAB that are unrelated to plant operations (Figure 2.1.1-203).

2.1.2.3 Arrangement for Traffic Control

The following measures will be implemented if it becomes necessary to control traffic entering the EAB:

- Access control will be established by Plant Security/Local Law Enforcement personnel on public roads where they intersect with the EAB to limit access to the area to authorized personnel.

Additional information regarding evacuation management is provided in the 2007 publication “Levy Nuclear Station Development of Evacuation Time Estimates” (Reference 2.1-205); updated information may be found in later revisions of that document.

2.1.2.4 Abandonment and Relocation of Roads

The EABs for LNP 1 and LNP 2 lie within the LNP site boundary (Figure 2.1.1-202). Therefore, the EABs for LNP 1 and LNP 2 should not affect any public roads or bridges. The project will not require the abandonment or relocation of any public roadways.

2.1.3 POPULATION DISTRIBUTION

2.1.3.1 Population within 10 Miles

Based on the 2000 U.S. Census, the total residential population within 16 km (10 mi.) of the LNP site was estimated to be 17,457 persons, as shown in Table 2.1.3-201. The population within the 16-km (10-mi.) radius of the LNP site varies from the population reported in the Emergency Plan, because 16-km (10-mi.) emergency planning zones and population radius methodologies differ slightly. The significant population groupings (for example, cities and towns) within 16 km (10 mi.) of the LNP site are shown on Figure 2.1.3-201, which also shows a sector chart divided into radii for 0 to 16 km (0 to 10 mi.). The sector chart was used in determining population distribution as described in the following subsections. The current plan includes the installation of two AP1000 units. The center of the distance between the two reactor buildings was assumed to be the centerpoint for the radii and sector grid. The radii were expanded by half of the distance between the two reactor buildings for LNP. The two reactor buildings are centered at the following coordinates:

LNP 1 Latitude: 29° 04’ 20.25” Longitude: -82° 37’ 12.94”

LNP 2 Latitude: 29° 04’ 29.62” Longitude: -82° 37’ 16.68”

The distance between the centerpoint of the reactor buildings for LNP 1 and LNP 2 is 289.5 m (950 ft. or 0.2 mi.) (Figure 2.1.1-203). Half of this distance, or 144.8 m (475 ft. or 0.1 mi.), was used to extend the radii in the grid sectors. For example, the 1.6-km (1-mi.) radius was extended to 1.7 km (1.1 mi.) to provide adequate coverage of LNP 1 and LNP 2 while maintaining compliance with guidance provided in NRC Regulatory Guide 1.206.

Residential and transient population distribution within the sectors have been summarized and are provided in Table 2.1.3-201. The table indicates that a majority of the population live in the eastern sectors, 8 to 16 km (5 to 10 mi.) from the site. The southwest and west-southwest sectors include the cities and towns of Inglis (population of 1491), located 6.6 km (4.1 mi.) southwest, and Yankeetown (population of 629), located 12.9 km (8.0 mi.) west-southwest. Data from the 2000 U.S. Census and a geographic information system (GIS) were used to determine the sector population distribution. Populations were calculated using census blocks, the smallest unit of data collected by the U.S. Census Bureau. There were approximately 759 census blocks within the 16-km (10-mi.) radius of the site. For population calculations, it was assumed that the 2000 U.S. Census population data were evenly distributed throughout a census block. Using this assumption, the GIS was used to determine the percent area of a census block contained in a particular sector. The percent area of the census block was then used to calculate the portion of the census block population within that sector. For example, if a sector contained 50 percent of a census block, it was assumed that the sector also contained 50 percent of the census block population.

Transient populations were calculated and included in the population estimates. These transient populations are defined in FSAR Subsection 2.1.3.3.

Population projections for 10-year increments up to 80 years from the 2000 U.S. Census for population within the 16-km (10-mi.) radius are included in Table 2.1.3-202. County projection information was collected from the Bureau of Economic and Business Research (BEBR) CD-ROM, “Detailed Population Projections by Age, Sex, Race, and Hispanic Origin for Florida and Its Counties.” The population projections are based on the expected population percent change rates (percent change) between 2000 and 2010, 2010 and 2020, and 2020 and 2030 (Reference 2.1-206). The percent change was estimated for each county, and the expected population change rate for the 10-year increments between 2030 and 2080 were assumed to be the average of the estimated percent change for the three periods between 2000 and 2030. The county percent change rates were then used to project populations using the U.S. Census Bureau data for each census block within the county. Population projections for each sector were calculated using the same method described above, assuming even distribution throughout the census block.

2.1.3.2 Population Between 10 and 50 Miles

Based on the 2000 U.S. Census, the total residential population between 16 km (10 mi.) and 80 km (50 mi.) of the LNP site was estimated to be 884,089 persons, as shown in Table 2.1.3-203. The significant population groupings (for example, cities and towns) within the region (80 km [50 mi.]) are shown in Figure 2.1.3-202, which also shows a sector chart divided into radii for 16 to 80 km (10 to 50 mi.).

Residential population distributions within the sectors for the 16- to 80-km (10- to 50-mi.) radii have been summarized and provided in Table 2.1.3-203. Table 2.1.3-203 indicates that a majority of the residential population is concentrated in the north-northeast, south, and east sectors; however, a significant portion of the resident population is in the eastern sectors. The U.S. Census Bureau data from the 2000 U.S. Census and the GIS were used to determine the sector population distribution, as described in FSAR Subsection 2.1.3.1.

Population projections for 10-year increments up to 80 years from the latest U.S. Census, for population between the 16-km (10 mi.) and 80-km (50 mi.) area of the LNP site, are included in Table 2.1.3-204. The population projections are based on the expected population percent change between 2000 and 2010, 2010 and 2020, and between 2020 and 2030. Population projections were obtained from the BEBR CD-ROM, “Detailed Population Projections by Age, Sex, Race, and Hispanic Origin for Florida and Its Counties” (Reference 2.1-206). The methodology described in FSAR Subsection 2.1.3.1 was used to forecast populations within the 16- to 80-km (10- to 50-mi.) region.

2.1.3.3 Transient Population

Transient populations were calculated and included in the population estimates. The following categories were used in estimating the transient population for each sector in the 0- to 16-km (0- to 10-mi.) radius:

- **Seasonal Population**. The GIS was used to collect information reported in the 2000 U.S. Census on seasonal and vacation home usage within the 16-km (10-mi.) radius. A standard housing occupancy factor of 2.49 people per house was used to estimate transient population for seasonal housing (Reference 2.1-207).

- **Transient Business Population***.* For businesses located within the 16-km (10-mi.) radius, the employees for major employers were assumed to be included in the transient population estimates. A list of the major employers and total number of employees was obtained from the Economic Development offices for Levy, Citrus, and Marion counties (References 2.1-208, 2.1-209, and 2.1-210). Major employers were defined as those with more than 100 employees.

- **Hotel/Motel Population**. Hotels and motels located within the 16-km (10-mi.) radius were identified using the GIS. The GIS data were sorted based on distance from the centerpoint of the two reactor units. Total room numbers were obtained by phone surveys, and one person was assumed to occupy each room on a given night.

- **Recreation Areas**. Major recreational areas were identified within the 16-km (10-mi.) radius of the LNP site, as shown on Figure 2.1.3-203. Total projected occupancy estimates collected for major recreational areas were used in the transient population estimates and are presented in Table 2.1.3-205.

- **Special Populations (Schools, Hospitals, Nursing Homes, and Correctional Facilities)**. The GIS was used to determine schools, hospitals, nursing homes, and correctional facilities located within the 16-km (10-mi.) radius. Telephone interviews were conducted to identify occupancy estimates for hospitals, nursing homes, and correctional facilities located within the 16-km (10-mi.) radius.

- **Festivals**. No major festivals are held within the 16-km (10-mi.) radius that would affect the transient population estimates. The annual Nature Coast Civil War Reenactment is held on the Crystal River Quarry property and is attended by approximately 7300 people; however, this 3-day event is not included in transient population estimates because of its short duration (Reference 2.1-211).

- **Migrant Workers**. Migrant worker populations were calculated using average statewide statistical information supplied by the U.S. Department of Agriculture (USDA) 2002 Agricultural Census (Reference 2.1-212).

The following categories were used in estimating the transient population for each sector in the 16- to 80-km (10- to 50-mi.) radius:

- **Seasonal Population**. The methodology described for the 16-km (10-mi.) radius was used to determine seasonal population for the 80-km (50-mi.) radius.

- **Transient Business Population**. For businesses located within the 80-km (50-mi.) radius, no net change was assumed to occur in population. This assumption was based on the large radial area and reasonable judgment that the number of workers commuting into the 80-km (50-mi.) area is the same as the number of workers commuting out of the 80-km (50-mi.) area on a daily basis.

- **Hotel/Motel Population**. The GIS was used to collect information on the location and number of hotels, motels, inns, and bed and breakfast establishments within the 80-km (50-mi.) radius. Based on the large area and reasonable judgment, the average hotels, motels, inns, and bed and breakfast establishments were assumed to contain 75, 25, 10, and 5 rooms, respectively. To estimate transient population, one person was assumed to occupy each room on a given night.

- **Recreation Areas**. Recreation areas were defined to be public recreation areas where usage patterns are tracked based on parking permits or other entrance fees. Major recreational areas are shown on Figure 2.1.3-203 with corresponding occupancy numbers provided in Table 2.1.3-205.

- **Special Populations (Schools, Hospitals, Nursing Homes, and Correctional Facilities)**. Based on the large area and reasonable judgment, no net change in special population was assumed to occur within the 80-km (50-mi.) radius. The U.S. Census was assumed to include university students living in dormitories and apartments, residents of correctional facilities, and long-term residents of nursing homes, hospitals, and other institutions, as part of the census survey for residential totals. Staff and residents temporarily placed in hospitals, nursing homes, and other institutions are likely to live within the 80-km (50-mi.) radial area; therefore, special populations would not contribute to transient population estimates within the region.

- **Festivals**. Several large festivals and sporting events occur in the 80-km (50-mi.) area. However, these festivals occur throughout the year causing the transient population to vary on a daily basis. Any additional transient population would be small in comparison and short in duration.

- **Migrant Workers**. The methodology described for the 16-km (10-mi.) radius was used to determine migrant worker population for the 80-km (50-mi.) radius.

2.1.3.4 Low Population Zone

The LPZ, shown on Figure 2.1.1-202, is the area immediately surrounding the exclusion area encompassed by two circles of 4.8-km (3-mi.) radius centered on each of the reactor buildings for the LNP 1 and LNP 2. In accordance with the guidance provided in Regulatory Guide 1.206, the LPZ was established such that the distance from the boundary of the LPZ to the nearest population center (described in FSAR Subsection 2.1.3.5) that exceeds 25,000 residents is at least one and one-third times the distance from the reactors to the outer boundary of the LPZ. The population distribution of the LPZ is shown in the first three columns of Table 2.1.3-201, which includes the permanent residents and transients. Peak daily population estimates are assumed to be the resident plus transient population within the LPZ. Population distribution was determined based on a single radius of 4.99 km (3.1 mi.) to include the distance between LNP 1 and LNP 2. The number and density of residents in the LPZ are low, which will enable effective evacuation procedures to be followed in the event of a serious accident (Table 2.1.3-201).

The LPZ was selected to provide reasonable probability that appropriate protective measures could be taken on behalf of the permanent and transient residents. Figure 2.1.1-201 shows the highway network around the site and in the surrounding area. The roads and highways within the area will be the primary transportation routes for evacuation. A topographic map of the area is provided in FSAR Figure 2.2.2-201.

The determination of the LPZ is further explained in FSAR Section 2.3.

No facilities or institutions were identified within the LPZ. Nearby industrial, transportation, and military facilities are further described in FSAR Section 2.2.

2.1.3.5 Population Center

A population center is described in 10 CFR 100.3 as a densely populated center where there are about 25,000 inhabitants or more. The closest such center with the largest population is Ocala, Florida, which is located 48.4 km (30.1 mi.) east-northeast of the site (Table 2.1.3-206). The land use between the site and Ocala is primarily rural with some scattered residential. In 2000, Ocala's population was 45,622 (Reference 2.1-213). This distance was determined from the corporate boundary that satisfies the 10 CFR 100.11 criteria that the population center be at least one and one-third times the distance from the outer boundary of the LPZ or, in this case, approximately 4.8 km (3 mi.). Table 2.1.3-206 shows the 2000 populations, distances, and directions from the site of cities, towns, and villages within approximately 80 km (50 mi.) of the site. Figure 2.1.1-201 shows major population centers within 80 km (50 mi.) of the site, which are also included in Table 2.1.3-206. Transient population was not considered in establishing the population center. As noted in Tables 2.1.3-201, 2.1.3-202, 2.1.3-203, and 2.1.3-204, the population within 80 km (50 mi.) of the LNP site is projected to change through 2080.

2.1.3.6 Population Density

The current and projected residential and transient population densities in the 0- to 16-km (0- to 10-mi.) and 16- to 80-km (10- to 50-mi.) areas surrounding the LNP site are presented on Figures 2.1.3-204, 2.1.3-205, 2.1.3-206, 2.1.3-207, 2.1.3-208, and 2.1.3-209. Most of the area within the 16-km (10-mi.) radius of the site is rural, with a population density in 2000 of 81 people per square mile (ppsm) (Table 2.1.3-207). The area between 16 and 80 km (10 and 50 mi.) of the site is the most densely populated. In 2000, the residential and transient population within the 0- to 32-km (0- to 20-mi.) area was approximately 123,067 persons, with an average population density of 97 ppsm (as shown in Table 2.1.3-207). The average population densities projected for the years 2010, 2015, and 2020 are shown in Table 2.1.3-207. The projected Combined License approval date is 2010. The projected population distribution and population density for the year 2010 and 5 years after the approval date are shown in Table 2.1.3-207 and on Figures 2.1.3-205, 2.1.3-206, 2.1.3-208, and 2.1.3-209.

STD DEP 1.1-1

2.1.4 COMBINED LICENSE INFORMATION FOR GEOGRAPHY AND DEMOGRAPHY

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

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LNP COL 2.1-1

Table 2.1.1-201
Coordinates of Reactors

Reactor Unit	Latitude	Longitude	State Plane Northing	State Plane Easting	UTM Zone 17N Northing	UTM Zone 17N Easting
1	29 04 20.25	-82 37 12.94	1723097.07	458028.56	3217078.80	342285.36
2	29 04 29.62	-82 37 16.68	1724045.25	457701.88	3217368.62	342188.25

Notes:

UTM = Universal Transverse Mercator

LNP COL 2.1-1

Table 2.1.1-202
U.S. Geological Survey Quadrangle Maps

USGS Quad ID	USGS Quad Name	State Name
28082-H5	Crystal River	Florida
29082-A4	Dunnellon	Florida
28082-H4	Holder	Florida
29082-B6	Lebanon Station	Florida
28082-H6	Red Level	Florida
29082-B4	Romeo	Florida
29082-B5	Tidewater	Florida
29082-A6	Yankeetown	Florida
29082-A5	Yankeetown SE	Florida

Source: Reference 2.1-201

LNP COL 2.1-1

Table 2.1.1-203
Minimum Distance from the LNP to the Exclusion Area Boundary (EAB) for Each Major Compass Direction

	Distance[a]	
Sector	**Meters**	**Feet**
N	1484	4867
NNE	1492	4759
NE	1400	4593
ENE	1341	4398
E	1382	4534
ESE	1359	4460
SE	1436	4712
SSE	1493	4897
S	1484	4867
SSW	1492	4759
SW	1400	4593
WSW	1341	4398
W	1382	4534
WNW	1436	4712
NW	1476	4841
NNW	1493	4897

Notes:

a) The distances were obtained from Figure 2.1.1-203.

The minimum distance in any direction from each reactor to an exclusion area boundary is approximately 1340 meters (m) (4396 feet [ft.]). The measurements reported in this table represent the distances from the centerpoint of Units 1 and 2 to the outermost boundary of the exclusion area boundaries at the centerpoint of each sector (Figure 2.1.1-203).

Distance measurements were provided in standard units; unit conversion to metric units may contain rounding differences.

E = east
N = north
S = south
W = west

LNP COL 2.1-1

Table 2.1.3-201 (Sheet 1 of 2)
2000 Resident and Transient Population within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
North-Residential	0	5	35	67	18	11	**136**
North-Transient	3	12	11	16	20	168	**230**
North North East-Residential	0	4	14	14	8	270	**310**
North North East-Transient	3	7	11	16	20	168	**225**
North East-Residential	1	1	6	10	5	806	**829**
North East-Transient	3	7	11	16	20	137	**194**
East North East-Residential	1	0	0	0	4	1066	**1071**
East North East-Transient	3	7	11	16	20	126	**183**
East-Residential	1	2	2	0	11	2300	**2316**
East-Transient	3	7	11	16	20	1234	**1291**
East South East-Residential	2	7	11	45	90	2725	**2880**
East South East-Transient	3	7	11	16	22	281	**340**
South East-Residential	2	7	31	322	294	1582	**2238**
South East-Transient	3	7	11	16	40	1187	**1264**
South South East-Residential	2	7	27	48	277	2474	**2835**
South South East-Transient	3	7	11	22	36	309	**388**
South-Residential	2	7	13	16	44	1455	**1537**
South-Transient	3	7	11	16	34	1004	**1075**
South South West-Residential	2	5	49	419	33	102	**610**
South South West-Transient	3	7	11	18	37	305	**381**
South West-Residential	2	8	55	499	599	210	**1373**
South West-Transient	3	7	11	16	30	1009	**1076**
West South West-Residential	2	11	26	142	239	736	**1156**
West South West-Transient	3	7	11	16	20	479	**536**
West-Residential	1	5	3	7	22	8	**46**
West-Transient	3	7	11	16	20	421	**478**

LNP COL 2.1-1

Table 2.1.3-201 (Sheet 2 of 2)
2000 Resident and Transient Population within 16 Km (10 Mi.)

	km 0-1.6	1.6-3.2	3.2-4.8	4.8-6.4	6.4-8.1	8.1-16.1	Total for Sector
	mi. 0-1	1-2	2-3	3-4	4-5	5-10	
West North West-Residential	0	2	4	4	1	6	**17**
West North West-Transient	3	7	11	16	20	168	**225**
North West-Residential	0	2	4	5	5	3	**19**
North West-Transient	3	7	11	16	20	168	**225**
North North West-Residential	0	2	22	18	35	7	**84**
North North West-Transient	3	7	11	16	20	168	**225**
Residential Total	**18**	**75**	**302**	**1616**	**1685**	**13,761**	**17,457**
Cumulative Total (Residential plus Transient)	**66**	**192**	**478**	**1880**	**2084**	**21,093**	**25,793**

Notes:

To account for the difference in distance between each LNP unit and the LNP centerpoint, 0.16 km (0.1 mi.) was added to each radial distance to conservatively adjust the population data. The totals are subject to rounding differences.

km = kilometer
mi. = mile

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 1 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
North-Residential							
2005 Population	0	5	39	73	20	11	**148**
2010 Population	0	6	43	82	22	14	**167**
2015 Population	0	6	47	90	24	14	**181**
2020 Population	0	7	51	97	26	17	**198**
2030 Population	0	8	58	111	29	20	**226**
2040 Population	0	9	69	130	34	23	**265**
2050 Population	0	10	82	153	40	26	**311**
2060 Population	0	12	97	181	47	30	**367**
2070 Population	0	14	115	214	56	36	**435**
2080 Population	0	16	136	252	66	42	**512**
North-Transient							
2005 Population	3	13	12	18	22	185	**253**
2010 Population	4	15	14	20	25	207	**285**
2015 Population	4	16	15	22	27	226	**310**
2020 Population	5	18	17	24	30	245	**339**
2030 Population	6	20	19	27	34	277	**383**
2040 Population	7	24	22	32	40	328	**453**
2050 Population	8	28	26	38	47	388	**535**
2060 Population	9	33	31	45	56	459	**633**
2070 Population	11	39	37	53	66	543	**749**
2080 Population	13	46	44	63	78	642	**886**
North North East-Residential							
2005 Population	0	4	15	15	9	297	**340**
2010 Population	0	5	17	17	9	327	**375**
2015 Population	0	5	18	18	10	356	**407**
2020 Population	0	6	20	20	10	384	**440**
2030 Population	0	7	22	22	11	434	**496**
2040 Population	0	8	26	26	13	511	**584**
2050 Population	0	9	30	31	15	600	**685**
2060 Population	0	11	35	36	17	706	**805**
2070 Population	0	13	41	42	20	832	**948**
2080 Population	0	15	48	49	23	979	**1114**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 2 of 12) Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
North North East-Transient							
2005 Population	3	8	12	18	22	192	**255**
2010 Population	4	9	14	20	25	217	**289**
2015 Population	4	10	15	22	27	240	**318**
2020 Population	5	11	17	24	30	263	**350**
2030 Population	6	12	19	27	34	301	**399**
2040 Population	7	14	22	32	40	366	**481**
2050 Population	8	17	26	38	47	445	**581**
2060 Population	9	20	31	45	56	541	**702**
2070 Population	11	24	37	53	66	658	**849**
2080 Population	13	28	44	63	78	800	**1026**
North East-Residential							
2005 Population	1	1	7	11	6	939	**965**
2010 Population	1	1	7	12	6	1060	**1087**
2015 Population	1	1	8	13	7	1168	**1198**
2020 Population	1	1	8	14	7	1304	**1335**
2030 Population	1	1	9	16	8	1515	**1550**
2040 Population	1	1	11	19	9	1859	**1900**
2050 Population	1	1	13	22	11	2292	**2340**
2060 Population	1	1	15	26	13	2842	**2898**
2070 Population	1	1	18	31	15	3513	**3579**
2080 Population	1	1	21	37	18	4345	**4423**
North East-Transient							
2005 Population	3	8	12	18	22	156	**219**
2010 Population	4	9	14	20	25	177	**249**
2015 Population	4	10	15	22	27	196	**274**
2020 Population	5	11	17	24	30	214	**301**
2030 Population	6	12	19	27	34	245	**343**
2040 Population	7	14	22	32	40	298	**413**
2050 Population	8	17	26	38	47	362	**498**
2060 Population	9	20	31	45	56	440	**601**
2070 Population	11	24	37	53	66	535	**726**
2080 Population	13	28	44	63	78	650	**876**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 3 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km	0-1.6	1.6-3.2	3.2-4.8	4.8-6.4	6.4-8.1	8.1-16.1	Total for Sector
mi.	0-1	1-2	2-3	3-4	4-5	5-10	
East North East-Residential							
2005 Population	1	0	0	0	4	1255	**1260**
2010 Population	1	0	0	0	5	1443	**1449**
2015 Population	1	0	0	0	5	1609	**1615**
2020 Population	1	0	0	0	6	1786	**1793**
2030 Population	1	0	0	0	7	2071	**2079**
2040 Population	1	0	0	0	8	2576	**2585**
2050 Population	1	0	0	0	9	3207	**3217**
2060 Population	1	0	0	0	11	4006	**4018**
2070 Population	1	0	0	0	13	4999	**5013**
2080 Population	1	0	0	0	15	6235	**6251**
East North East-Transient							
2005 Population	3	8	12	18	22	144	**207**
2010 Population	4	9	14	20	25	163	**235**
2015 Population	4	10	15	22	27	180	**258**
2020 Population	5	11	17	24	30	197	**284**
2030 Population	6	12	19	27	34	225	**323**
2040 Population	7	14	22	32	40	274	**389**
2050 Population	8	17	26	38	47	333	**469**
2060 Population	9	20	31	45	56	405	**566**
2070 Population	11	24	37	53	66	492	**683**
2080 Population	13	28	44	63	78	598	**824**
East-Residential							
2005 Population	1	2	2	0	12	2706	**2723**
2010 Population	1	2	2	0	13	3111	**3129**
2015 Population	1	2	2	0	14	3472	**3491**
2020 Population	1	2	2	0	15	3845	**3865**
2030 Population	1	2	2	0	17	4446	**4468**
2040 Population	1	2	2	0	20	5537	**5562**
2050 Population	1	2	2	0	23	6909	**6937**
2060 Population	1	2	2	0	27	8617	**8649**
2070 Population	1	2	2	0	32	10,749	**10,786**
2080 Population	1	2	2	0	38	13,411	**13,454**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 4 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
East-Transient							
2005 Population	3	8	12	18	22	1400	**1463**
2010 Population	4	9	14	20	25	1577	**1649**
2015 Population	4	10	15	22	27	1734	**1812**
2020 Population	5	11	17	24	30	1891	**1978**
2030 Population	6	12	19	27	34	2151	**2249**
2040 Population	7	14	22	32	40	2592	**2707**
2050 Population	8	17	26	38	47	3123	**3259**
2060 Population	9	20	31	45	56	3763	**3924**
2070 Population	11	24	37	53	66	4534	**4725**
2080 Population	13	28	44	63	78	5463	**5689**
East South East-Residential							
2005 Population	2	8	12	50	99	3045	**3216**
2010 Population	2	9	14	55	111	3396	**3587**
2015 Population	2	10	15	60	121	3692	**3900**
2020 Population	2	11	17	65	132	4005	**4232**
2030 Population	2	12	19	73	150	4505	**4761**
2040 Population	2	14	22	86	177	5324	**5625**
2050 Population	2	17	26	102	209	6302	**6658**
2060 Population	2	20	31	120	246	7466	**7885**
2070 Population	2	24	37	143	291	8870	**9367**
2080 Population	2	28	44	168	344	10,514	**11,100**
East South East-Transient							
2005 Population	3	8	12	18	24	319	**384**
2010 Population	4	9	14	20	27	359	**433**
2015 Population	4	10	15	22	29	395	**475**
2020 Population	5	11	17	24	32	430	**519**
2030 Population	6	12	19	27	36	489	**589**
2040 Population	7	14	22	32	43	589	**707**
2050 Population	8	17	26	38	51	710	**850**
2060 Population	9	20	31	45	60	855	**1020**
2070 Population	11	24	37	53	71	1030	**1226**
2080 Population	13	28	44	63	84	1241	**1473**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 5 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
South East-Residential							
2005 Population	2	8	34	356	331	1759	**2490**
2010 Population	2	9	38	395	367	1964	**2775**
2015 Population	2	10	41	432	399	2126	**3010**
2020 Population	2	11	45	468	431	2315	**3272**
2030 Population	2	12	52	529	484	2604	**3683**
2040 Population	2	14	61	622	573	3062	**4334**
2050 Population	2	17	71	734	678	3609	**5111**
2060 Population	2	20	84	867	802	4260	**6035**
2070 Population	2	24	99	1023	949	5039	**7136**
2080 Population	2	28	117	1208	1123	5944	**8422**
South East-Transient							
2005 Population	3	8	12	18	45	1333	**1419**
2010 Population	4	9	14	20	50	1482	**1579**
2015 Population	4	10	15	22	55	1613	**1719**
2020 Population	5	11	17	24	59	1745	**1861**
2030 Population	6	12	19	27	67	1961	**2092**
2040 Population	7	14	22	32	79	2320	**2474**
2050 Population	8	17	26	38	93	2745	**2927**
2060 Population	9	20	31	45	110	3248	**3463**
2070 Population	11	24	37	53	130	3843	**4098**
2080 Population	13	28	44	63	154	4547	**4849**
South South East-Residential							
2005 Population	2	8	30	53	311	2766	**3170**
2010 Population	2	9	32	59	345	3082	**3529**
2015 Population	2	10	35	64	376	3352	**3839**
2020 Population	2	11	37	69	406	3628	**4153**
2030 Population	2	12	42	77	455	4078	**4666**
2040 Population	2	14	50	90	538	4815	**5509**
2050 Population	2	17	58	106	638	5691	**6512**
2060 Population	2	20	68	125	755	6728	**7698**
2070 Population	2	24	81	147	893	7964	**9111**
2080 Population	2	28	95	173	1056	9411	**10,765**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 6 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km mi.	0-1.6 0-1	1.6-3.2 1-2	3.2-4.8 2-3	4.8-6.4 3-4	6.4-8.1 4-5	8.1-16.1 5-10	Total for Sector
South South East-Transient							
2005 Population	3	8	12	24	40	347	**434**
2010 Population	4	9	14	27	45	386	**485**
2015 Population	4	10	15	29	49	420	**527**
2020 Population	5	11	17	32	53	454	**572**
2030 Population	6	12	19	36	60	510	**643**
2040 Population	7	14	22	43	71	603	**760**
2050 Population	8	17	26	51	84	713	**899**
2060 Population	9	20	31	60	99	844	**1063**
2070 Population	11	24	37	71	117	999	**1259**
2080 Population	13	28	44	84	138	1182	**1489**
South-Residential							
2005 Population	2	8	14	17	49	1627	**1717**
2010 Population	2	9	16	19	53	1807	**1906**
2015 Population	2	10	17	20	57	1966	**2072**
2020 Population	2	11	19	22	62	2126	**2242**
2030 Population	2	12	22	25	69	2388	**2518**
2040 Population	2	14	26	29	81	2817	**2969**
2050 Population	2	17	30	33	95	3327	**3504**
2060 Population	2	20	35	39	110	3928	**4134**
2070 Population	2	24	42	46	129	4648	**4891**
2080 Population	2	28	50	53	152	5492	**5777**
South-Transient							
2005 Population	3	8	12	18	38	1128	**1207**
2010 Population	4	9	14	20	42	1254	**1343**
2015 Population	4	10	15	22	46	1365	**1462**
2020 Population	5	11	17	24	50	1476	**1583**
2030 Population	6	12	19	27	56	1658	**1778**
2040 Population	7	14	22	32	66	1962	**2103**
2050 Population	8	17	26	38	78	2321	**2488**
2060 Population	9	20	31	45	92	2746	**2943**
2070 Population	11	24	37	53	109	3249	**3483**
2080 Population	13	28	44	63	129	3844	**4121**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 7 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
South South West-Residential							
2005 Population	2	6	53	460	36	112	**669**
2010 Population	2	6	61	515	39	124	**747**
2015 Population	2	7	66	561	42	134	**812**
2020 Population	2	7	73	610	45	145	**882**
2030 Population	2	8	83	690	50	164	**997**
2040 Population	2	9	98	816	57	192	**1174**
2050 Population	2	11	115	965	66	224	**1383**
2060 Population	2	13	135	1138	77	261	**1626**
2070 Population	2	15	160	1345	90	310	**1922**
2080 Population	2	18	189	1587	105	362	**2263**
South South West-Transient							
2005 Population	3	8	12	20	41	343	**427**
2010 Population	4	9	14	22	46	381	**476**
2015 Population	4	10	15	24	50	415	**518**
2020 Population	5	11	17	26	54	449	**562**
2030 Population	6	12	19	29	61	505	**632**
2040 Population	7	14	22	34	72	597	**746**
2050 Population	8	17	26	40	85	706	**882**
2060 Population	9	20	31	47	101	835	**1043**
2070 Population	11	24	37	56	119	988	**1235**
2080 Population	13	28	44	66	141	1169	**1461**
South West-Residential							
2005 Population	2	9	60	551	661	236	**1519**
2010 Population	2	10	67	615	737	263	**1694**
2015 Population	2	11	72	670	803	287	**1845**
2020 Population	2	12	79	731	869	309	**2002**
2030 Population	2	14	89	826	983	347	**2261**
2040 Population	2	17	105	973	1160	410	**2667**
2050 Population	2	20	123	1148	1368	484	**3145**
2060 Population	2	24	145	1359	1614	573	**3717**
2070 Population	2	28	170	1605	1906	679	**4390**
2080 Population	2	33	199	1895	2251	803	**5183**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 8 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km mi.	0-1.6 0-1	1.6-3.2 1-2	3.2-4.8 2-3	4.8-6.4 3-4	6.4-8.1 4-5	8.1-16.1 5-10	Total for Sector
South West-Transient							
2005 Population	3	8	12	18	33	1133	**1207**
2010 Population	4	9	14	20	37	1260	**1344**
2015 Population	4	10	15	22	40	1372	**1463**
2020 Population	5	11	17	24	44	1483	**1584**
2030 Population	6	12	19	27	50	1666	**1780**
2040 Population	7	14	22	32	59	1971	**2105**
2050 Population	8	17	26	38	70	2332	**2491**
2060 Population	9	20	31	45	83	2759	**2947**
2070 Population	11	24	37	53	98	3264	**3487**
2080 Population	13	28	44	63	116	3862	**4126**
West South West-Residential							
2005 Population	2	13	29	155	264	811	**1274**
2010 Population	2	13	32	174	296	907	**1424**
2015 Population	2	15	35	189	323	986	**1550**
2020 Population	2	15	38	206	353	1074	**1688**
2030 Population	2	17	43	233	401	1211	**1907**
2040 Population	2	20	51	275	473	1428	**2249**
2050 Population	2	24	60	325	557	1686	**2654**
2060 Population	2	28	71	382	660	1991	**3134**
2070 Population	2	33	84	451	780	2355	**3705**
2080 Population	2	39	99	532	918	2780	**4370**
West South West-Transient							
2005 Population	3	8	12	18	22	533	**596**
2010 Population	4	9	14	20	25	594	**666**
2015 Population	4	10	15	22	27	648	**726**
2020 Population	5	11	17	24	30	702	**789**
2030 Population	6	12	19	27	34	791	**889**
2040 Population	7	14	22	32	40	936	**1051**
2050 Population	8	17	26	38	47	1107	**1243**
2060 Population	9	20	31	45	56	1309	**1470**
2070 Population	11	24	37	53	66	1548	**1739**
2080 Population	13	28	44	63	78	1831	**2057**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 9 of 12) Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
West-Residential							
2005 Population	1	5	3	7	25	9	**50**
2010 Population	1	6	3	8	27	9	**54**
2015 Population	1	6	3	8	30	10	**58**
2020 Population	1	7	3	9	32	10	**62**
2030 Population	1	8	3	10	36	11	**69**
2040 Population	1	9	3	11	41	12	**77**
2050 Population	1	10	3	12	49	14	**89**
2060 Population	1	12	3	13	57	16	**102**
2070 Population	1	14	3	15	67	18	**118**
2080 Population	1	16	3	17	79	21	**137**
West-Transient							
2005 Population	3	8	12	18	22	464	**527**
2010 Population	4	9	14	20	25	518	**590**
2015 Population	4	10	15	22	27	566	**644**
2020 Population	5	11	17	24	30	614	**701**
2030 Population	6	12	19	27	34	694	**792**
2040 Population	7	14	22	32	40	821	**936**
2050 Population	8	17	26	38	47	971	**1107**
2060 Population	9	20	31	45	56	1148	**1309**
2070 Population	11	24	37	53	66	1358	**1549**
2080 Population	13	28	44	63	78	1606	**1832**
West North West-Residential							
2005 Population	0	2	4	4	1	7	**18**
2010 Population	0	2	5	4	1	7	**19**
2015 Population	0	2	5	4	1	8	**20**
2020 Population	0	2	6	4	1	8	**21**
2030 Population	0	2	7	4	1	9	**23**
2040 Population	0	2	8	4	1	11	**26**
2050 Population	0	2	9	4	1	13	**29**
2060 Population	0	2	11	4	1	15	**33**
2070 Population	0	2	13	4	1	18	**38**
2080 Population	0	2	15	4	1	21	**43**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 10 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
West North West-Transient							
2005 Population	3	8	12	18	22	185	**248**
2010 Population	4	9	14	20	25	207	**279**
2015 Population	4	10	15	22	27	226	**304**
2020 Population	5	11	17	24	30	245	**332**
2030 Population	6	12	19	27	34	277	**375**
2040 Population	7	14	22	32	40	328	**443**
2050 Population	8	17	26	38	47	388	**524**
2060 Population	9	20	31	45	56	459	**620**
2070 Population	11	24	37	53	66	543	**734**
2080 Population	13	28	44	63	78	642	**868**
North West-Residential							
2005 Population	0	2	4	6	6	3	**21**
2010 Population	0	2	5	6	6	3	**22**
2015 Population	0	2	5	7	7	3	**24**
2020 Population	0	2	6	7	7	3	**25**
2030 Population	0	2	7	8	8	3	**28**
2040 Population	0	2	8	9	9	3	**31**
2050 Population	0	2	9	11	11	3	**36**
2060 Population	0	2	10	13	13	3	**41**
2070 Population	0	2	12	15	15	3	**47**
2080 Population	0	2	14	18	18	3	**55**
North West-Transient							
2005 Population	3	8	12	18	22	185	**248**
2010 Population	4	9	14	20	25	207	**279**
2015 Population	4	10	15	22	27	226	**304**
2020 Population	5	11	17	24	30	245	**332**
2030 Population	6	12	19	27	34	277	**375**
2040 Population	7	14	22	32	40	328	**443**
2050 Population	8	17	26	38	47	388	**524**
2060 Population	9	20	31	45	56	459	**620**
2070 Population	11	24	37	53	66	543	**734**
2080 Population	13	28	44	63	78	642	**868**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 11 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

km / mi.	0-1.6 / 0-1	1.6-3.2 / 1-2	3.2-4.8 / 2-3	4.8-6.4 / 3-4	6.4-8.1 / 4-5	8.1-16.1 / 5-10	Total for Sector
North North West-Residential							
2005 Population	0	2	24	20	39	8	**93**
2010 Population	0	2	27	22	43	8	**102**
2015 Population	0	2	29	24	47	9	**111**
2020 Population	0	2	32	26	51	9	**120**
2030 Population	0	2	36	30	58	10	**136**
2040 Population	0	2	42	35	69	11	**159**
2050 Population	0	2	49	41	81	13	**186**
2060 Population	0	2	58	49	96	15	**220**
2070 Population	0	2	68	58	113	17	**258**
2080 Population	0	2	80	68	133	20	**303**
North North West-Transient							
2005 Population	3	8	12	18	22	185	**248**
2010 Population	4	9	14	20	25	207	**279**
2015 Population	4	10	15	22	27	226	**304**
2020 Population	5	11	17	24	30	245	**332**
2030 Population	6	12	19	27	34	277	**375**
2040 Population	7	14	22	32	40	328	**443**
2050 Population	8	17	26	38	47	388	**524**
2060 Population	9	20	31	45	56	459	**620**
2070 Population	11	24	37	53	66	543	**734**
2080 Population	13	28	44	63	78	642	**868**
2005 Population							
Residential Total	18	83	330	1778	1873	15,591	**19,673**
Cumulative Total (Residential plus Transient)	66	216	522	2074	2314	23,823	**29,015**
2010 Population							
Residential Total	18	91	369	1983	2080	17,525	**22,066**
Cumulative Total (Residential plus Transient)	82	241	593	2312	2577	26,721	**32,526**
2015 Population							
Residential Total	18	99	398	2160	2266	19,192	**24,133**
Cumulative Total (Residential plus Transient)	82	265	638	2521	2805	29,240	**35,551**

LNP COL 2.1-1

Table 2.1.3-202 (Sheet 12 of 12)
Resident and Transient Population Projections within 16 Km (10 Mi.)

	km mi.	0-1.6 0-1	1.6-3.2 1-2	3.2-4.8 2-3	4.8-6.4 3-4	6.4-8.1 4-5	8.1-16.1 5-10	Total for Sector
2020 Population								
Residential Total		18	107	436	2348	2453	20,968	**26,330**
Cumulative Total (Residential plus Transient)		98	290	708	2742	3045	31,866	**38,749**
2030 Population								
Residential Total		18	119	494	2654	2767	23,816	**29,868**
Cumulative Total (Residential plus Transient)		114	319	798	3097	3437	36,120	**43,885**
2040 Population								
Residential Total		18	137	582	3125	3263	28,591	**35,716**
Cumulative Total (Residential plus Transient)		130	371	934	3650	4053	43,232	**52,370**
2050 Population								
Residential Total		18	161	680	3687	3851	34,400	**42,797**
Cumulative Total (Residential plus Transient)		146	444	1096	4310	4782	51,820	**62,598**
2060 Population								
Residential Total		18	189	800	4352	4546	41,457	**51,362**
Cumulative Total (Residential plus Transient)		162	522	1296	5089	5651	62,186	**74,906**
2070 Population								
Residential Total		18	222	945	5139	5370	50,050	**61,744**
Cumulative Total (Residential plus Transient)		194	621	1537	6008	6674	74,720	**89,754**
2080 Population								
Residential Total		18	258	1112	6061	6340	60,383	**74,172**
Cumulative Total (Residential plus Transient)		226	724	1816	7093	7882	89,744	**107,485**

Notes:

To account for the difference in distance between each LNP unit and the LNP centerpoint, 0.16 km (0.1 mi.) was added to each radial distance to conservatively adjust the population data. The totals are subject to rounding differences.

km = kilometer; mi. = mile

LNP COL 2.1-1

Table 2.1.3-203 (Sheet 1 of 2) 2000 Resident and Transient Population between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
North-Residential	637	5551	8364	11,512	**26,064**
North-Transient	141	267	303	845	**1556**
North North East-Residential	2646	7754	21,826	156,599	**188,825**
North North East-Transient	146	323	3560	3251	**7280**
North East-Residential	2242	3503	11,136	6797	**23,678**
North East-Transient	306	748	986	706	**2746**
East North East-Residential	7762	32,043	58,111	6919	**104,835**
East North East-Transient	473	1716	3219	1384	**6792**
East-Residential	5920	34,574	65,253	17,122	**122,869**
East-Transient	2383	771	1242	1451	**5847**
East South East-Residential	6607	5148	22,170	60,649	**94,574**
East-South-East-Transient	975	1239	1701	4065	**7980**
South East-Residential	24,287	28,151	11,061	17,376	**80,875**
South East-Transient	1333	3370	2159	3959	**10,821**
South South East-Residential	17,636	11,629	25,828	18,790	**73,883**
South South East-Transient	3082	1978	2650	5179	**12,889**
South-Residential	10,602	4087	31,161	90,824	**136,674**
South-Transient	8684	1567	1708	1174	**13,133**
South South West-Residential	199	0	0	0	**199**
South South West-Transient	330	27	0	0	**357**
South West-Residential	0	0	0	0	**0**
South West-Transient	3	0	0	0	**3**
West South West-Residential	0	0	0	0	**0**
West South West-Transient	0	0	0	0	**0**
West-Residential	0	510	0	0	**510**
West-Transient	7	233	0	0	**240**

LNP COL 2.1-1

Table 2.1.3-203 (Sheet 2 of 2)
2000 Resident and Transient Population between 16 and 80 Km (10 and 50 Mi.)

km mi.	16-32 10-20	32-48 20-30	48-64 30-40	64-80 40-50	Total for Sector
West North West-Residential	2	1093	476	238	**1809**
West North West-Transient	74	1453	380	101	**2008**
North West-Residential	62	726	1202	5258	**7248**
North West-Transient	141	234	4152	3168	**7695**
North North West-Residential	453	907	11,875	8811	**22,046**
North North West-Transient	141	234	1841	1394	**3610**
Residential Total	**79,055**	**135,676**	**268,463**	**400,895**	**884,089**
Cumulative Total (Residential plus Transient)	**97,274**	**149,836**	**292,364**	**427,572**	**967,046**

Notes:

To account for the difference in distance between each LNP unit and the LNP centerpoint, 0.16 km (0.1 mi.) was added to each radial distance to conservatively adjust the population data. The totals are subject to rounding differences.

km = kilometer
mi. = mile

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 1 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km	16-32	32-48	48-64	64-80	Total for Sector
mi.	10-20	20-30	30-40	40-50	
North-Residential					
2005 Population	696	6109	9260	12,757	**28,822**
2010 Population	778	6805	10,173	13,966	**31,722**
2015 Population	844	7414	10,945	15,017	**34,220**
2020 Population	918	8049	11,758	16,050	**36,775**
2030 Population	1038	9096	13,018	17,691	**40,843**
2040 Population	1219	10,713	15,105	20,465	**47,502**
2050 Population	1430	12,620	17,534	23,699	**55,283**
2060 Population	1685	14,873	20,402	27,469	**64,429**
2070 Population	1989	17,558	23,755	31,863	**75,165**
2080 Population	2343	20,697	27,702	37,001	**87,743**
North-Transient					
2005 Population	155	295	336	941	**1727**
2010 Population	174	324	375	1049	**1922**
2015 Population	190	350	409	1142	**2091**
2020 Population	206	375	443	1235	**2259**
2030 Population	233	416	498	1386	**2533**
2040 Population	276	483	588	1636	**2983**
2050 Population	326	561	695	1931	**3513**
2060 Population	385	651	821	2280	**4137**
2070 Population	455	756	970	2691	**4872**
2080 Population	538	877	1146	3177	**5738**
North North East-Residential					
2005 Population	2907	8580	24,118	172,975	**208,580**
2010 Population	3251	9586	26,129	187,350	**226,316**
2015 Population	3530	10,474	27,859	199,699	**241,562**
2020 Population	3850	11,387	29,588	212,061	**256,886**
2030 Population	4355	12,883	32,213	230,725	**280,176**
2040 Population	5133	15,253	36,690	262,668	**319,744**
2050 Population	6042	18,080	41,795	299,001	**364,918**
2060 Population	7123	21,425	47,622	340,460	**416,630**
2070 Population	8425	25,413	54,270	387,657	**475,765**
2080 Population	9936	30,128	61,850	441,450	**543,364**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 2 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16 - 32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
North North East-Transient					
2005 Population	166	364	4017	3591	**8138**
2010 Population	189	407	4489	3889	**8974**
2015 Population	209	444	4901	4145	**9699**
2020 Population	229	482	5314	4402	**10,427**
2030 Population	262	542	5981	4789	**11,574**
2040 Population	319	645	7118	5453	**13,535**
2050 Population	388	768	8471	6209	**15,836**
2060 Population	472	914	10,081	7070	**18,537**
2070 Population	574	1088	11,997	8051	**21,710**
2080 Population	698	1295	14,277	9168	**25,438**
North East-Residential					
2005 Population	2532	4119	13,003	7531	**27,185**
2010 Population	2859	4740	14,828	8225	**30,652**
2015 Population	3144	5291	16,445	8821	**33,701**
2020 Population	3444	5847	18,120	9438	**36,849**
2030 Population	3937	6766	20,829	10,392	**41,924**
2040 Population	4756	8443	25,723	12,019	**50,941**
2050 Population	5745	10,535	31,812	13,945	**62,037**
2060 Population	6962	13,147	39,387	16,226	**75,722**
2070 Population	8437	16,408	48,790	18,919	**92,554**
2080 Population	10,225	20,483	60,488	22,127	**113,323**
North East-Transient					
2005 Population	349	853	1125	784	**3111**
2010 Population	396	967	1258	858	**3479**
2015 Population	438	1068	1373	921	**3800**
2020 Population	479	1170	1488	984	**4121**
2030 Population	548	1339	1671	1084	**4642**
2040 Population	666	1628	1995	1251	**5540**
2050 Population	810	1979	2382	1444	**6615**
2060 Population	985	2406	2844	1667	**7902**
2070 Population	1197	2925	3395	1925	**9442**
2080 Population	1455	3556	4053	2222	**11,286**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 3 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
East North East-Residential					
2005 Population	9139	37,729	68,427	8144	**123,439**
2010 Population	10,515	43,428	78,736	9372	**142,051**
2015 Population	11,732	48,506	87,958	10,461	**158,657**
2020 Population	12,998	53,635	97,213	11,572	**175,418**
2030 Population	15,045	62,086	112,532	13,397	**203,060**
2040 Population	18,741	77,482	140,456	16,713	**253,392**
2050 Population	23,383	96,733	175,374	20,865	**316,355**
2060 Population	29,195	120,782	219,002	26,060	**395,039**
2070 Population	36,436	150,808	273,471	32,537	**493,252**
2080 Population	45,490	188,343	341,558	40,628	**616,019**
East North East-Transient					
2005 Population	557	2021	3791	1630	**7999**
2010 Population	641	2326	4363	1876	**9206**
2015 Population	716	2598	4874	2096	**10,284**
2020 Population	791	2871	5384	2315	**11,361**
2030 Population	915	3323	6231	2679	**13,148**
2040 Population	1143	4150	7782	3346	**16,421**
2050 Population	1428	5183	9719	4179	**20,509**
2060 Population	1783	6473	12,138	5219	**25,613**
2070 Population	2227	8084	15,160	6518	**31,989**
2080 Population	2781	10,096	18,934	8141	**39,952**
East-Residential					
2005 Population	6969	40,704	76,846	20,245	**144,764**
2010 Population	8016	46,848	88,407	23,363	**166,634**
2015 Population	8930	52,316	98,764	26,154	**186,164**
2020 Population	9920	57,861	109,196	28,954	**205,931**
2030 Population	11,502	66,987	126,408	33,592	**238,489**
2040 Population	14,318	83,611	157,718	42,125	**297,772**
2050 Population	17,856	104,384	196,898	52,874	**372,012**
2060 Population	22,303	130,355	245,866	66,396	**464,920**
2070 Population	27,834	162,766	306,976	83,374	**580,950**
2080 Population	34,755	203,267	383,384	104,772	**726,178**

LNP COL 2.1-1

**Table 2.1.3-204 (Sheet 4 of 12)
Resident and Transient Population Projections
between 16 and 80 Km (10 and 50 Mi.)**

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
East-Transient					
2005 Population	2806	908	1463	1845	**7022**
2010 Population	3230	1045	1683	2211	**8169**
2015 Population	3608	1167	1880	2537	**9192**
2020 Population	3986	1290	2077	2863	**10,216**
2030 Population	4613	1493	2404	3411	**11,921**
2040 Population	5761	1865	3002	4559	**15,187**
2050 Population	7195	2329	3749	6094	**19,367**
2060 Population	8986	2909	4682	8146	**24,723**
2070 Population	11,223	3633	5848	10,889	**31,593**
2080 Population	14,017	4537	7304	14,555	**40,413**
East South East-Residential					
2005 Population	7417	6044	30,162	77,446	**121,069**
2010 Population	8240	6907	37,235	93,326	**145,708**
2015 Population	8985	7692	43,698	107,638	**168,013**
2020 Population	9725	8503	50,197	121,952	**190,377**
2030 Population	10,948	9832	61,330	146,236	**228,346**
2040 Population	12,968	12,226	87,177	197,776	**310,147**
2050 Population	15,370	15,272	124,127	267,851	**422,620**
2060 Population	18,228	19,165	176,938	363,253	**577,584**
2070 Population	21,672	24,087	252,557	493,502	**791,818**
2080 Population	25,729	30,373	360,879	671,463	**1,088,444**
East South East-Transient					
2005 Population	1122	1524	2092	5170	**9908**
2010 Population	1269	1789	2457	6194	**11,709**
2015 Population	1400	2019	2773	7107	**13,299**
2020 Population	1530	2250	3090	8020	**14,890**
2030 Population	1745	2632	3614	9556	**17,547**
2040 Population	2122	3396	4664	12,774	**22,956**
2050 Population	2580	4382	6019	17,075	**30,056**
2060 Population	3137	5655	7767	22,824	**39,383**
2070 Population	3815	7297	10,023	30,509	**51,644**
2080 Population	4639	9416	12,934	40,781	**67,770**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 5 of 12) Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km	16-32	32-48	48-64	64-80	Total for Sector
mi.	10-20	20-30	30-40	40-50	
South East-Residential					
2005 Population	27,227	31,575	14,057	23,351	**96,210**
2010 Population	30,230	35,046	16,755	28,631	**110,662**
2015 Population	32,895	38,145	19,220	33,461	**123,721**
2020 Population	35,570	41,256	21,687	38,329	**136,842**
2030 Population	39,943	46,325	25,894	46,649	**158,811**
2040 Population	47,205	54,781	35,184	65,801	**202,971**
2050 Population	55,815	64,795	48,181	93,089	**261,880**
2060 Population	65,976	76,599	66,413	132,007	**340,995**
2070 Population	78,078	90,668	92,129	187,654	**448,529**
2080 Population	92,322	107,229	128,494	267,238	**595,283**
South East-Transient					
2005 Population	1497	3785	2637	4920	**12,839**
2010 Population	1664	4208	3078	5800	**14,750**
2015 Population	1812	4581	3458	6580	**16,431**
2020 Population	1959	4954	3838	7359	**18,110**
2030 Population	2201	5567	4465	8655	**20,888**
2040 Population	2604	6587	5709	11,280	**26,180**
2050 Population	3081	7794	7300	14,701	**32,876**
2060 Population	3645	9222	9334	19,160	**41,361**
2070 Population	4313	10,911	11,935	24,972	**52,131**
2080 Population	5103	12,910	15,260	32,546	**65,819**
South South East-Residential					
2005 Population	19,789	13,060	29,743	21,737	**84,329**
2010 Population	21,986	14,517	33,501	24,551	**94,555**
2015 Population	23,922	15,806	36,884	27,085	**103,697**
2020 Population	25,890	17,101	40,267	29,613	**112,871**
2030 Population	29,091	19,220	45,838	33,764	**127,913**
2040 Population	34,403	22,743	55,568	41,102	**153,816**
2050 Population	40,687	26,917	67,368	50,040	**185,012**
2060 Population	48,121	31,856	81,674	60,924	**222,575**
2070 Population	56,946	37,724	99,063	74,205	**267,938**
2080 Population	67,351	44,652	120,152	90,388	**322,543**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 6 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
South South East-Transient					
2005 Population	3462	2251	3016	6041	**14,770**
2010 Population	3848	2520	3376	6847	**16,591**
2015 Population	4189	2759	3697	7574	**18,219**
2020 Population	4530	2999	4017	8301	**19,847**
2030 Population	5090	3392	4543	9493	**22,518**
2040 Population	6022	4065	5444	11,639	**27,170**
2050 Population	7125	4871	6524	14,270	**32,790**
2060 Population	8430	5837	7818	17,496	**39,581**
2070 Population	9974	6995	9368	21,451	**47,788**
2080 Population	11,801	8382	11,226	26,300	**57,709**
South-Residential					
2005 Population	11,888	4582	35,916	105,711	**158,097**
2010 Population	13,188	5095	40,462	119,626	**178,371**
2015 Population	14,369	5545	44,592	132,217	**196,723**
2020 Population	15,521	6006	48,655	144,817	**214,999**
2030 Population	17,430	6754	55,404	165,460	**245,048**
2040 Population	20,597	7985	67,206	202,504	**298,292**
2050 Population	24,352	9441	81,528	247,823	**363,144**
2060 Population	28,775	11,175	98,894	303,355	**442,199**
2070 Population	34,057	13,242	120,027	371,408	**538,734**
2080 Population	40,260	15,679	145,678	454,841	**656,458**
South-Transient					
2005 Population	9754	1783	1969	1369	**14,875**
2010 Population	10,843	1996	2220	1552	**16,611**
2015 Population	11,804	2186	2445	1717	**18,152**
2020 Population	12,765	2375	2670	1882	**19,692**
2030 Population	14,343	2686	3039	2152	**22,220**
2040 Population	16,970	3219	3688	2638	**26,515**
2050 Population	20,078	3857	4475	3234	**31,644**
2060 Population	23,756	4622	5430	3965	**37,773**
2070 Population	28,107	5539	6589	4861	**45,096**
2080 Population	33,255	6638	7996	5960	**53,849**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 7 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km mi.	16-32 10-20	32-48 20-30	48-64 30-40	64-80 40-50	Total for Sector
South South West-Residential					
2005 Population	222	0	0	0	**222**
2010 Population	246	0	0	0	**246**
2015 Population	267	0	0	0	**267**
2020 Population	288	0	0	0	**288**
2030 Population	323	0	0	0	**323**
2040 Population	380	0	0	0	**380**
2050 Population	447	0	0	0	**447**
2060 Population	527	0	0	0	**527**
2070 Population	622	0	0	0	**622**
2080 Population	734	0	0	0	**734**
South South West-Transient					
2005 Population	371	30	0	0	**401**
2010 Population	412	34	0	0	**446**
2015 Population	449	37	0	0	**486**
2020 Population	485	40	0	0	**525**
2030 Population	545	45	0	0	**590**
2040 Population	645	53	0	0	**698**
2050 Population	763	63	0	0	**826**
2060 Population	903	75	0	0	**978**
2070 Population	1068	89	0	0	**1157**
2080 Population	1264	105	0	0	**1369**
South West-Residential					
2005 Population	0	0	0	0	**0**
2010 Population	0	0	0	0	**0**
2015 Population	0	0	0	0	**0**
2020 Population	0	0	0	0	**0**
2030 Population	0	0	0	0	**0**
2040 Population	0	0	0	0	**0**
2050 Population	0	0	0	0	**0**
2060 Population	0	0	0	0	**0**
2070 Population	0	0	0	0	**0**
2080 Population	0	0	0	0	**0**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 8 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
South West-Transient					
2005 Population	3	0	0	0	**3**
2010 Population	4	0	0	0	**4**
2015 Population	4	0	0	0	**4**
2020 Population	5	0	0	0	**5**
2030 Population	6	0	0	0	**6**
2040 Population	7	0	0	0	**7**
2050 Population	8	0	0	0	**8**
2060 Population	9	0	0	0	**9**
2070 Population	11	0	0	0	**11**
2080 Population	13	0	0	0	**13**
West South West-Residential					
2005 Population	0	0	0	0	**0**
2010 Population	0	0	0	0	**0**
2015 Population	0	0	0	0	**0**
2020 Population	0	0	0	0	**0**
2030 Population	0	0	0	0	**0**
2040 Population	0	0	0	0	**0**
2050 Population	0	0	0	0	**0**
2060 Population	0	0	0	0	**0**
2070 Population	0	0	0	0	**0**
2080 Population	0	0	0	0	**0**
West South West-Transient					
2005 Population	0	0	0	0	**0**
2010 Population	0	0	0	0	**0**
2015 Population	0	0	0	0	**0**
2020 Population	0	0	0	0	**0**
2030 Population	0	0	0	0	**0**
2040 Population	0	0	0	0	**0**
2050 Population	0	0	0	0	**0**
2060 Population	0	0	0	0	**0**
2070 Population	0	0	0	0	**0**
2080 Population	0	0	0	0	**0**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 9 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
West-Residential					
2005 Population	0	561	0	0	**561**
2010 Population	0	625	0	0	**625**
2015 Population	0	681	0	0	**681**
2020 Population	0	740	0	0	**740**
2030 Population	0	836	0	0	**836**
2040 Population	0	982	0	0	**982**
2050 Population	0	1158	0	0	**1158**
2060 Population	0	1365	0	0	**1365**
2070 Population	0	1608	0	0	**1608**
2080 Population	0	1893	0	0	**1893**
West-Transient					
2005 Population	8	257	0	0	**265**
2010 Population	9	287	0	0	**296**
2015 Population	10	314	0	0	**324**
2020 Population	11	340	0	0	**351**
2030 Population	12	385	0	0	**397**
2040 Population	14	455	0	0	**469**
2050 Population	17	538	0	0	**555**
2060 Population	20	636	0	0	**656**
2070 Population	24	752	0	0	**776**
2080 Population	28	889	0	0	**917**
West North West-Residential					
2005 Population	2	1206	528	261	**1997**
2010 Population	2	1340	582	291	**2215**
2015 Population	2	1461	630	313	**2406**
2020 Population	2	1584	684	344	**2614**
2030 Population	2	1793	763	384	**2942**
2040 Population	2	2116	892	446	**3456**
2050 Population	2	2493	1039	517	**4051**
2060 Population	2	2943	1219	608	**4772**
2070 Population	2	3474	1423	709	**5608**
2080 Population	2	4096	1664	826	**6588**

LNP COL 2.1-1

Table 2.1.3-204 (Sheet 10 of 12)
Resident and Transient Population Projections between 16 and 80 Km (10 and 50 Mi.)

km mi.	16-32 10-20	32-48 20-30	48-64 30-40	64-80 40-50	Total for Sector
West North West-Transient					
2005 Population	82	1602	421	112	**2217**
2010 Population	91	1789	467	124	**2471**
2015 Population	99	1955	508	134	**2696**
2020 Population	108	2121	549	145	**2923**
2030 Population	122	2399	618	162	**3301**
2040 Population	144	2837	727	190	**3898**
2050 Population	170	3355	856	223	**4604**
2060 Population	201	3967	1007	261	**5436**
2070 Population	238	4691	1185	306	**6420**
2080 Population	281	5547	1395	358	**7581**
North West-Residential					
2005 Population	67	801	1321	5843	**8032**
2010 Population	75	892	1476	6451	**8894**
2015 Population	82	973	1608	6994	**9657**
2020 Population	88	1058	1746	7540	**10,432**
2030 Population	101	1197	1970	8435	**11,703**
2040 Population	117	1414	2323	9871	**13,725**
2050 Population	137	1668	2735	11,551	**16,091**
2060 Population	162	1970	3222	13,531	**18,885**
2070 Population	191	2329	3802	15,839	**22,161**
2080 Population	224	2752	4479	18,542	**25,997**
North West-Transient					
2005 Population	155	258	4598	3523	**8534**
2010 Population	174	288	5104	3889	**9455**
2015 Population	190	315	5555	4215	**10,275**
2020 Population	206	341	6005	4541	**11,093**
2030 Population	233	386	6755	5080	**12,454**
2040 Population	276	456	7950	5950	**14,632**
2050 Population	326	539	9357	6969	**17,191**
2060 Population	385	637	11,012	8163	**20,197**
2070 Population	455	753	12,960	9562	**23,730**
2080 Population	538	890	15,253	11,200	**27,881**

LNP COL 2.1-1

**Table 2.1.3-204 (Sheet 11 of 12)
Resident and Transient Population Projections
between 16 and 80 Km (10 and 50 Mi.)**

km / mi.	16-32 / 10-20	32-48 / 20-30	48-64 / 30-40	64-80 / 40-50	Total for Sector
North North West-Residential					
2005 Population	501	998	13,160	9828	**24,487**
2010 Population	556	1115	14,811	11,031	**27,513**
2015 Population	606	1217	16,248	12,078	**30,149**
2020 Population	659	1323	17,729	13,144	**32,855**
2030 Population	745	1496	20,175	14,914	**37,330**
2040 Population	877	1767	24,076	17,804	**44,524**
2050 Population	1034	2084	28,734	21,248	**53,100**
2060 Population	1217	2463	34,322	25,379	**63,381**
2070 Population	1441	2911	41,008	30,318	**75,678**
2080 Population	1695	3438	48,992	36,241	**90,366**
North North West-Transient					
2005 Population	155	258	2049	1551	**4013**
2010 Population	174	288	2299	1741	**4502**
2015 Population	190	315	2520	1908	**4933**
2020 Population	206	341	2741	2076	**5364**
2030 Population	233	386	3112	2357	**6088**
2040 Population	276	456	3710	2810	**7252**
2050 Population	326	539	4422	3350	**8637**
2060 Population	385	637	5271	3993	**10,286**
2070 Population	455	753	6283	4760	**12,251**
2080 Population	538	890	7490	5674	**14,592**
2005 Population					
Residential Total	89,356	156,068	316,541	465,829	**1,027,794**
Cumulative Total (Residential plus Transient)	109,998	172,257	344,055	497,306	**1,123,616**
2010 Population					
Residential Total	99,942	176,944	363,095	526,183	**1,166,164**
Cumulative Total (Residential plus Transient)	123,060	195,212	394,264	562,213	**1,274,749**
2015 Population					
Residential Total	109,308	195,521	404,851	579,938	**1,289,618**
Cumulative Total (Residential plus Transient)	134,616	215,629	439,244	620,014	**1,409,503**

LNP COL 2.1-1

**Table 2.1.3-204 (Sheet 12 of 12)
Resident and Transient Population Projections
between 16 and 80 Km (10 and 50 Mi.)**

km mi.	16-32 10-20	32-48 20-30	48-64 30-40	64-80 40-50	Total for Sector
2020 Population					
Residential Total	118,873	214,350	446,840	633,814	**1,413,877**
Cumulative Total (Residential plus Transient)	146,369	236,299	484,456	677,937	**1,545,061**
2030 Population					
Residential Total	134,460	245,271	516,374	721,639	**1,617,744**
Cumulative Total (Residential plus Transient)	165,561	270,262	559,305	772,443	**1,767,571**
2040 Population					
Residential Total	160,716	299,516	648,118	889,294	**1,997,644**
Cumulative Total (Residential plus Transient)	197,961	329,811	700,495	952,820	**2,181,087**
2050 Population					
Residential Total	192,300	366,180	817,125	1,102,503	**2,478,108**
Cumulative Total (Residential plus Transient)	236,921	402,938	881,094	1,182,182	**2,703,135**
2060 Population					
Residential Total	230,276	448,118	1,034,961	1,375,668	**3,089,023**
Cumulative Total (Residential plus Transient)	283,758	492,759	1,113,166	1,475,912	**3,365,595**
2070 Population					
Residential Total	276,130	548,996	1,317,271	1,727,985	**3,870,382**
Cumulative Total (Residential plus Transient)	340,266	603,262	1,412,984	1,854,480	**4,210,992**
2080 Population					
Residential Total	331,066	673,030	1,685,320	2,185,517	**4,874,933**
Cumulative Total (Residential plus Transient)	408,015	739,058	1,802,588	2,345,599	**5,295,260**

Notes:

To account for the difference in distance between each LNP unit and the LNP centerpoint, 0.16 km (0.1 mi.) was added to each radial distance to conservatively adjust the population data. The totals are subject to rounding differences.

km = kilometer
mi. = mile

LNP COL 2.1-1

Table 2.1.3-205 (Sheet 1 of 2)
Recreational Areas within 80 Km (50 Mi.) of the LNP

Area	Average Daily Attendance	Daily Capacity	Average Percent Utilization	Projected Capacity	Approximate Distance[a] and Direction to LNP
Cedar Key Museum State Park	56	884	6.3%	908	42.3 km (26.3 mi.) E
Cedar Key Scrub State Park	46	216	21.3%	352	37.5 km (23.3 mi.) SE
Crystal River Archaeological State Park	52	488	10.7%	588	18.2 km (11.3 mi.) N
Crystal River Preserve State Park	748	NA	NA	NA	9.0 km (5.6 mi.) NE
Dade Battlefield Historic State Park	51	980	5.2%	980	66.5 km (41.3 mi.) NW
Devil’s Millhopper State Park	122	480	25.4%	480	73.2 km (45.5 mi.) S
Dudley Farm Historic State Park	44	260	16.9%	260	64.8 km (40.3 mi.) S
Fanning Springs State Park	770	1010	76.2%	1318	63.9 km (39.7 mi.) SE
Fort Cooper State Park	68	1018	6.7%	1302	41.3 km (25.7 mi.) NW
Goethe State Forest	5 [b]	NA	NA	NA	2.6 km (1.6 mi.) S
Homosassa Springs Wildlife State Park	895	6464	13.8%	6464	30.1 km (18.7 mi.) N
Lake Griffin State Park	97	622	15.6%	904	73.7 km (45.8 mi.) W
Manatee Springs State Park	367	2536	14.5%	2544	55.8 km (34.7 mi.) SE
Marjorie Harris Cross Carr Florida Greenway	82 [c]	NA	NA	NA	5.2 km (3.2 mi.) S
Marjorie Kinnan Rawlings Historic State Park	55	120	45.8%	120	63.2 km (39.3 mi.) SW
Ocala National Forest	NA	NA	NA	NA	63.7 km (39.6 mi.) W

LNP COL 2.1-1

Table 2.1.3-205 (Sheet 2 of 2)
Recreational Areas within 80 Km (50 Mi.) of the LNP

Area	Average Daily Attendance	Daily Capacity	Average Percent Utilization	Projected Capacity	Approximate Distance and Direction[(a)] to LNP
Paynes Prairie Preserve State Park	533	2820	18.9%	2850	57.3 km (35.6 mi.) SW
Rainbow Springs State Park	541	1775	30.5%	1835	16.9 km (10.5 mi.) W
San Felasco Hammock Preserve State Park	157	816	19.2%	1616	71.6 km (44.5 mi.) S
Silver River State Park	629	1074	58.6%	1602	56.7 km (35.2 mi.) W
Wacasassa Bay State Park	72	208	34.6%	280	9.5 km (5.9 mi.) E
Withlacoochee State Forest	1869	NA	NA	NA	22.5 km (14.0 mi.) W
Yulee Sugar Mill Ruins Historic State Park	87	288	30.2%	288	32.1 km (20.0 mi.) N
TOTAL	7346	22,059	-	-	-

Notes:

a) Distances were obtained from Figure 2.1.3-203.

b) Attendance is estimated based on the amount of fees paid. Due to the open access of multiple entrances of the forest, many people do not pay fees, and are therefore not accounted for in attendance estimate.

c) Attendance reported for the portion of the Greenway to the west of Lake Rosseau.

E = east
km = kilometer
mi. = mile
N = north
NA = Data not available (due to open access in these recreation areas, capacity information is unavailable).
S = south
W = west

Sources: References 2.1-214, 2.1-215, 2.1-216, 2.1-217, 2.1-218, 2.1-219, 2.1-220, 2.1-221, 2.1-222, 2.1-223, 2.1-224, 2.1-225, 2.1-226, 2.1-227, 2.1-228, 2.1-229, 2.1-230, and 2.1-231

LNP COL 2.1-1

Table 2.1.3-206 (Sheet 1 of 3)
2000 Population of Cities and Communities within an 80-Km (50-Mi.) Radius

Place Name	Total Population in 2000	Distance Kilometers	Distance Miles	Direction
ALACHUA	5932	77.6	48.2	NNE
ARCHER	1282	51.6	32.1	NNE
BAYPORT	24	58.6	36.4	S
BELL	349	79.3	49.2	NNW
BELLEVIEW	2554	54.3	33.8	E
BEVERLY HILLS	8317	23.5	14.6	SE
BRONSON	981	41.6	25.8	N
BROOKRIDGE	3141	59.4	36.9	SSE
BROOKSVILLE	7250	61.9	38.4	SSE
BUSHNELL	2160	67.1	41.7	SE
CEDAR KEY	775	40.9	25.4	W
CENTER HILL	951	77.2	48.0	SE
CHIEFLAND	1996	50.2	31.2	NNW
CITRUS SPRINGS	4159	16.6	10.3	ESE
COLEMAN	697	61.6	38.3	ESE
CROSS CITY	1839	79.2	49.2	NW
CRYSTAL RIVER	3339	19.2	11.9	S
DUNNELLON	1919	15.8	9.8	E
FANNING SPRINGS	668	64.8	40.2	NNW
FLORAL CITY	4889	47.8	29.7	SE
FRUITLAND PARK	3197	73.4	45.6	ESE
GAINESVILLE	95605	71.18	44.2	NNE
HAWTHORNE	1400	77.4	48.1	NE
HERNANDO	8415	30.7	19.1	SE
HERNANDO BEACH	2150	67.2	41.8	S
HOMOSASSA	2263	32.4	20.2	S
HOMOSASSA SPRINGS	12750	30.6	19.0	S
HORSESHOE BEACH	202	76.5	47.5	WNW

LNP COL 2.1-1

**Table 2.1.3-206 (Sheet 2 of 3)
2000 Population of Cities and Communities
within an 80-Km (50-Mi.) Radius**

Place Name	Total Population in 2000	Distance Kilometers	Distance Miles	Direction
HUDSON	12724	79.0	49.1	S
INGLIS	1491	6.6	4.1	SW
INVERNESS	6725	38.7	24.0	SE
ISTACHATTA	61	56.8	35.3	SE
LACOOCHEE	1172	80.4	50.0	SSE
LADY LAKE	11,678	70.1	43.5	ESE
LAKE PANASOFFKEE	3445	62.2	38.6	SE
LECANTO	4738	27.3	17.0	SSE
LEESBURG	15884	78.0	48.4	ESE
MASARYKTOWN	881	71.9	44.7	SSE
MICANOPY	623	58.2	36.2	NE
NEWBERRY	3331	63.6	39.5	N
NORTH BROOKSVILLE	1479	59.3	36.9	SSE
OCALA	45622	48.4	30.1	ENE
OKAHUMPKA	204	79.3	49.3	ESE
OTTER CREEK	109	31.5	19.6	NNW
REDDICK	567	52.4	32.6	NE
RIDGE MANOR	4122	76.6	47.6	SE
SILVER SPRINGS SHORES	6554	59.8	37.2	E
SOUTH BROOKSVILLE	1339	63.9	39.7	SSE
SPRING HILL	69196	65.9	41.0	S
SUGARMILL WOODS	6479	39.5	24.5	SSE
TIMBER PINES	5817	67.0	41.7	S
TRENTON	1548	62.9	39.1	NNW
WEBSTER	812	75.4	46.9	SE
WEEKI WACHEE	9	62.1	38.6	S
WEEKI WACHEE GARDENS	1162	60.0	37.3	S
WILDWOOD	4031	61.1	38.0	ESE

LNP COL 2.1-1

**Table 2.1.3-206 (Sheet 3 of 3)
2000 Population of Cities and Communities within an 80-Km (50-Mi.) Radius**

Place Name	Total Population in 2000	Distance		Direction
		Kilometers	Miles	
WILLISTON	2304	38.7	24.1	NNE
YANKEETOWN	647	12.9	8.0	WSW

Notes:

E = east
N = north
S = south
W = west

Source: Reference 2.1-213

LNP COL 2.1-1

Table 2.1.3-207
Estimated and Projected Residential and Transient Population Density within 80 Km (50 Mi.) of the LNP (People per Square Mile)

Year	People per Square Mile 0 to 16 km (0 to 10 mi.)	0 to 32 km (0 to 20 mi.)	0 to 80 km (0 to 50 mi.)
Year 2000	81	97	126
Year 2010	102	123	170
Year 2015	117	136	184
Year 2020	121	146	201

Notes:

To account for the difference in distance between each LNP unit and the LNP centerpoint, 0.16 km (0.1 mi.) was added to each radial distance to conservatively adjust the population data.

km = kilometer
mi. = mile

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

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

LNP COL 2.2-1
LNP COL 3.3-1
LNP COL 3.5-1

As outlined in the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.206, information was compiled on major manufacturing plants, chemical plants, refineries, storage facilities, mining and quarrying operations, military bases, missile sites, transportation routes and facilities, oil and gas pipelines, drilling operations and wells, and underground gas storage facilities within 8 kilometers (km) (5 miles [mi.]) of the Levy Nuclear Plant Units 1 and 2 (LNP) site. In the case where there were no facilities located within 8 km (5 mi.) of the LNP, information was collected on nearby facilities, and the study radii expanded to include the closest facility within each category. A collection of electronic resources (websites), an Environmental Data Resources, Inc. (EDR) report, and the State of Florida geographical information system (GIS) data clearinghouse were used to compile the information. Facilities that may manufacture, store, or transport materials that may be toxic, flammable, or explosive, such as chlorine, ammonia, compressed or liquid oxygen, or propane were identified based on the information provided in the EDR report and the Florida Geographic Data Library (FGDL). Information was also gathered on military firing and bombing ranges and any nearby flight, holding, and landing patterns.

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 Final Safety Analysis Report (FSAR) Section 2.2. This is being done to accommodate the incorporation of Regulatory Guide 1.206 numbering conventions for FSAR Section 2.2.

2.2.1 LOCATIONS AND ROUTES

LNP COL 2.2-1
LNP COL 3.3-1
LNP COL 3.5-1

The LNP is located in the southern part of Levy County, Florida, east of U.S. Highway 19/98 (State Route [SR] 55) and near the cities and towns of Inglis, Yankeetown, and Crystal River. Figure 2.2.1-201 shows the site location and surrounding region. Other nearby cities and towns include Lebanon, Tidewater, Dunnellon, Otter Creek, Chiefland, Bronson, and Fanning Springs, as presented on Figure 2.2.1-201. Topographic features of the surrounding region are shown on FSAR Figure 2.3.2-222.

The area immediately adjacent to the LNP site to the north is primarily state-owned forest land, known as the Goethe State Forest (see FSAR Figure 2.1.3-203). The Department of Agriculture and Consumer Services, Division of Forestry, manages the Goethe State Forest through timber management, wildlife management, ecological restoration, and outdoor recreation, such as picnicking, hiking, biking, fishing, wildlife viewing, overnight camping, and horseback riding (Reference 2.2-201).

No major industrial activities are located within the 8-km (5-mi.) radius of the LNP site (Figure 2.2.1-202). Topographic features of the 8-km (5-mi.) radius are shown on FSAR Figure 2.3.2-221. FSAR Subsection 2.2.2.1 provides a description and map showing the location and distance of commercial and industrial facilities in relation to the LNP site. The majority of the industrial development in the 80-km (50-mi.) radius of the LNP site is located in the urbanized areas of Levy, Marion, and Citrus counties. This development is discussed further in FSAR Subsection 2.2.2.8.

No active quarrying or mining facilities are located within the 8-km (5-mi.) radius of the LNP site. Gulf Rock, Inc., is an inactive mine located 6.3 km (3.9 mi.) from the LNP site (Figure 2.2.1-203). Inglis Mine, which is owned by Citrus Mining and Timber, is an active limestone mine located approximately 9.7 km (6 mi.) from the LNP site (Reference 2.2-202). Citrus Mining and Timber, and Cemex USA plan to expand the quarry from 136 to 327 hectares (ha) (335 to 809 acres [ac.]). The local community, however, opposes the expansion and is concerned that the mine is located in an environmentally sensitive area (Reference 2.2-202).

Holcim (US), Inc., operates the Crystal River quarry located outside of the 8-km (5-mi.) radius (Figure 2.2.1-203) and is a supplier of Portland and blended cements and related mineral components (Reference 2.2-203). The facility is located in Citrus County south of the Cross Florida Barge Canal (CFBC) (Reference 2.2-204). The facility is a crushed and broken limestone facility and has an air permit (Reference 2.2-205). Fourteen additional mining or quarrying facilities are located within a 40-km (25-mi.) radius of the LNP site (Figure 2.2.1-203).

Plum Creek Timberlands, L.P., is planning a mining operation, Titan Mines – Phase 2, within 8 km (5 mi.) of the LNP site, approximately 1.6 km (1 mi.) west of U.S. Highway 19 (Figure 2.2.1-203) (Reference 2.2-206). Plum Creek Timberlands, L.P., recently received a general construction modification permit for the 1618.7-ha (4000-ac.) tract of land they own. The proposed mining operation is to encompass 3.9 ha (9.7 ac.) of the 1618.7-ha (4000-ac.) tract (Reference 2.2.-206).

In addition to Orlando and Tampa, which are located beyond the 80-km (50-mi.) radius, Gainesville and Ocala are two major transportation hubs for central Florida that are located within the region (Figure 2.2.1-201). Gainesville and Ocala are served by rail lines, as well as major interstates and highways that serve local and interstate traffic. These highways and interstates are described in further detail in FSAR Subsection 2.2.2.5.

No airports or private airstrips are located within the 8-km (5-mi.) radius of the LNP site (Figure 2.2.1-204). J.R.'s private airstrip and the Crystal River Power Plant Heliport are located within a 16-km (10-mi.) radius of the site. Airport operations are described in further detail in FSAR Subsection 2.2.2.7.

No active military facilities are within 8 km (5 mi.) of the LNP site (Figure 2.2.1-205). Florida National Guard, Company B, 3rd Battalion, 20th Special Forces Group and the 690th Military Police Company National Guard are the only significant military facilities located within an 80-km (50-mi.) radius of the LNP site, as presented on Figure 2.2.1-205. Florida National Guard, Company B, 3rd Battalion, 20th Special Forces is located in Brooksville, Florida and is 67.6 km (42 mi.) from the site. The 690th Military Police Company National Guard is located in Crystal River, Florida, adjacent to the Crystal River Airport, and is 24.5 km (15.2 mi.) from the site.

No active railroads are located within the 8-km (5-mi.) radius of the LNP site. Two railroad lines, an abandoned track and an active line, are located within 16 km (10 mi.) of the LNP site. FSAR Subsection 2.2.2.6 describes these lines in further detail.

The Withlacoochee River is located south of the LNP site and extends in an east-west direction (Figure 2.2.1-201). The river is not used for commercial traffic and is classified by the Florida Department of Environmental Protection (FDEP) as an outstanding surface water body (Reference 2.2-207). The CFBC is located approximately 8 km (5 mi.) to the south of the LNP site (Figure 2.2.1-201). The CFBC is primarily used for recreational boating, with minor barge traffic to and from the Inglis Mine. FSAR Subsection 2.2.2.4 describes the surrounding waterways in further detail.

2.2.2 DESCRIPTIONS

Based on the EDR report, electronic resource review, and an aerial photo survey, no significant industrial activities, as described in Regulatory Guide 1.206, are located within the 8-km (5-mi.) radius of the LNP site. Some commercial automotive service, parts, storage, and gas stations are located within the surrounding area (Reference 2.2-208).

2.2.2.1 Description of Facilities

The following potentially hazardous commercial and industrial facilities are located within 8-km (5-mi.) of the LNP site (Figure 2.2.2-201):

- One Tier 2 facility (discussed further in FSAR Subsection 2.2.2.2). Tier 2 facilities store or manufacture hazardous materials and submit a hazardous chemical inventory report to state and local agencies (Reference 2.2-208).
- Three leaking underground storage tanks (LUST).
- Eleven underground storage tanks (UST).
- Two aboveground storage tanks (AST).

- Eight wastewater (WW) sites.
- Three Resource Conservation and Recovery Information System (RCRIS) small quantity generators (SQG).
- Two emergency response notification systems (ERNSs) (ERNS reports releases of oil and hazardous substances).
- One Integrated Compliance Information System (ICIS) (ICIS is a system that supports the information needs of the national enforcement and compliance program and the National Pollutant Discharge Elimination System [NPDES] program).
- One clandestine drug lab (CDL) (law enforcement agencies located chemicals or other items that indicate the presence of a CDL or dump site).

Table 2.2.2-201 presents detailed information on the facilities listed in this subsection, including the company associated with each facility. The locations of identified hazards are shown on Figure 2.2.2-201. Metal Industries, Inc., is a Superfund Site located approximately 19.3 km (12 mi.) from the LNP site (References 2.2-209 and 2.2-210). A site inspection completed in 1988 indicated the U.S. Environmental Protection Agency (USEPA) plans no further remedial action (Reference 2.2-209).

2.2.2.2 Description of Products and Materials

No manufacturing facilities that use or store hazardous products are located within the 8-km (5-mi.) radius of the LNP site (Figure 2.2.2-201). A Tier 2 facility (the Town of Inglis water treatment plant [WTP]) is located approximately 4.8 km (3 mi.) from the LNP site and stores/uses hazardous chemicals. Tier 2 facilities are those that store or manufacture hazardous materials (Reference 2.2-208). Table 2.2.2-202 presents the chemicals and the quantities stored/used at the Town of Inglis WTP. Florida Public Utilities is located on the east side of U.S. Highway 19 approximately 5.5 km (3.4 mi.) south of the LNP site (Reference 2.2-210). This facility, located in the Town of Inglis, provides propane gas and has three tanks on-site. One tank has a storage capacity of 113,563 liters (30,000 gallons) and each of the other two tanks can store 68,137 liters (18,000 gallons). No other explosives are located at this facility.

On-site chemical storage that supports plant operation is discussed in the Westinghouse Electric Company, LLC, AP1000 Design Control Document for the certified design as amended (DCD) Subsection 6.4.4. Transported materials are discussed in FSAR Subsection 2.2.3.2.1.

2.2.2.3 Description of Pipelines

Based on information from the EDR, no petroleum pipelines are present within the 8-km (5-mi.) radius of the LNP site (Reference 2.2-208). Three underground natural gas pipelines are located on the eastern side of U.S. Highway 19 alongside the remaining rail bed from the abandoned railroad track. The pipelines run parallel to U.S. Highway 19, approximately 1769 m (5803 ft.) to the west-northwest of the LNP site.

The three natural gas pipelines consist of 20.3-centimeter (cm) (8-inch [in.]), 76.2-cm (30-in.) and 91.4-cm (36-in.) diameter pipe, which are owned by FGT. The 20.3-cm (8-in.) pipeline is buried to a minimum of 0.9 meters (m) (3 feet [ft.]) below ground surface (bgs), and is 2123 m (6966 ft.) west of the LNP site. The pipeline has a maximum pressure of 912 pounds per square inch (psi). The 76.2-cm (30-in.) pipeline is buried a minimum of 0.9 m (3 ft.) bgs. The pipeline has a maximum pressure of 1200 psi and is located 1769 m (5803 ft.) west-northwest of the LNP site. The 91.4-cm (36-in.) pipeline is buried a minimum of 0.9 m (3 ft.) bgs. The pipeline has a maximum pressure of 1333 psi and is located 1757 m (5763 ft.) west-northwest of the LNP site. There are no plans to carry any other product in the pipelines except for natural gas. The locations of the 20.3-cm (8-in.), 76.2-cm (30-in.), and 91.4-cm (36-in.) pipelines with respect to the safety-related structures of the LNP are shown in Figure 2.2.2-202.

2.2.2.4 Description of Waterways

Five waterways are located within the 8-km (5-mi.) radius of the LNP site. The waterways include Ten Mile Creek, which connects to Cow Creek and the Gulf of Mexico, Spring Run Creek, which extends to the Gulf of Mexico, Lake Rousseau, the CFBC, and Withlacoochee River (Reference 2.2-211). Lake Rousseau's main channel is 4.3 to 5.2 m (14 to 17 ft.) deep, the CFBC is 3.7 m (12 ft.) deep, and Withlacoochee River is 3 m (10 ft.) deep (References 2.2-212, 2.2-213, and 2.2-214).

Recreational boating within the 8-km (5-mi) radius is likely to be associated with Cow Creek, Lake Rousseau, the CFBC, and Withlacoochee River. Inglis Lock was constructed from 1964 to 1970. The lock measures 26 m (84 ft.) by 183 m (600 ft.) and was constructed from heavy steel (Reference 2.2-215). The Inglis Lock was operated after the termination of the CFBC, until delayed maintenance activities resulted in the decommissioning of the lock in 1999 (Reference 2.2-216). The CFBC was renamed the Marjorie Harris Carr Cross Florida Greenway and is now used for recreational boating (see FSAR Figure 2.1.3-204) (Reference 2.2-217). The Inglis Mine utilizes the section of the barge canal to the west of U.S. Highway 19. The Inglis Mine has a slip on the northern side of the CFBC that is used for periodic shipments of limestone. The Inglis Mine is located outside of the 8-km (5-mi.) radius of the LNP site (Figure 2.2.1-203).

2.2.2.5 Description of Highways

The major highway located near the LNP site leading to Gainesville and Ocala include U.S. Highway 19/98 (SR 55). Figure 2.2.1-201 shows the transportation routes in the region of the LNP site. Interstate 75 (I-75) is the closest interstate, which is located approximately 45 km (28 mi.) to the east of the LNP site.

At its nearest point, U.S. Highway 19/98 (SR 55) is located approximately 1974 m (6477 ft.) from the center of the LNP site (Figure 2.2.2-201). The average annual daily traffic (AADT) counts at the four closest monitoring points within the 8-km (5-mi.) radius of the LNP site range from 1600 (Site 340086–SR 121, 0.32 km (0.2 mi.) northeast of SR 55) to 8600 (Site 340069–SR 55 at the southern city limits of Inglis) cars per day (Reference 2.2-218). This highway is mainly used for local traffic and local commodity deliveries.

2.2.2.6 Description of Railways

Two railroad lines are located within 16 km (10 mi.) of the LNP site. The lines include an abandoned track with only the rail bed remaining, which is located northeast of the site and north of SR 336, and an active railroad line operated by CSX Transportation, Inc. (CSX), which is located southeast of the LNP site. The CSX line runs from the city of Crystal River northeast to the city of Dunnellon.

In accordance with NRC Regulatory Guide 1.206, further analysis of the CSX rail segment was not conducted, as it lies outside of the 8-km (5-mi.) radius of the LNP site.

2.2.2.7 Description of Airports

No airports are within the 8-km (5-mi.) radius of the LNP site (Figure 2.2.1-204). J.R.'s private airstrip is 10.1 km (6.3 mi.) from the LNP site, and the Crystal River Power Plant Heliport is 14.5 km (9 mi.) from the site. Nine public airports and 48 private airports or airstrips are located between the 16-km (10-mi.) and 80-km (50-mi.) radii of the LNP site, but the locations of the private airports have limited facilities. No further analysis was performed on the private airports or airstrips. The nine public airports and their distances from the plant are shown below:

- Crystal River Airport (Citrus County)–23.3 km (14.5 mi.)
- Marion County Dunnellon Airport (Marion County)–23.8 km (14.8 mi.)
- Williston Municipal Airport (Levy County)–34.1 km (21.2 mi.)
- Ocala International Airport (Marion County)–40.1 km (24.9 mi.)
- George T. Lewis Airport (Levy County)–42.7 km (26.2 mi.)
- Inverness Airport (Citrus County)–42.7 km (26.2 mi.)

- Hernando County Airport (Hernando County)–68.6 km (42.6 mi.)
- Gainesville Regional Airport (Alachua County)–76.1 km (47.3 mi.)
- Cross City Airport (Dixie County)–78.7 km (48.9 mi.).

Approximately 50 aircraft are based at the Crystal River Airport (43 single-engine, 5 multi-engine, 1 helicopter, and 1 glider airplane), with approximately 100 aircraft operations per day (49 percent local general aviation [49 flights]; 49 percent transient general aviation [49 flights]; 1 percent air taxi aviation [1 flight]; and less than 1 percent military [1 flight]) (Reference 2.2-219). Future plans for the airport include a 1524-m (5000-ft.) extension of the east-west runway to be completed within the next 4 to 5 years. This improvement is designed to make aircraft landings safer and will not increase traffic. No aircraft accidents or collisions have occurred at Crystal River Airport that have resulted in fatalities or that have been considered serious accidents. Only minor landing mishaps that did not result in property damage have been reported by airport operations.

Approximately 52 aircraft are based at Marion County Dunnellon Airport (42 single-engine, 5 multi-engine, and 5 ultralights), with approximately 41 aircraft operations per day (80 percent local general aviation [33 flights] and 20 percent transient general aviation [8 flights]) (Reference 2.2-220). Future plans for the airport include rehabilitation of the two existing runways to accommodate slightly larger general aviation and corporate aircraft. An increase in traffic is not expected. Two accidents occurred in the past 3 years at Marion County Dunnellon Airport.

Approximately 36 aircraft are based at Williston Municipal Airport (27 single-engine, 3 multi-engine, 2 jet planes, 2 helicopters, and 2 ultralights), with approximately 45 aircraft operations per day (30 percent local general aviation [14 flights] and 70 percent transient general aviation [31 flights]). Skydiving also takes place from the Williston Municipal Airport (Reference 2.2-221). Williston Municipal Airport will be constructing new hanger storage and anticipates a 20 percent growth in operations. No aircraft accidents or collisions have occurred at Williston Municipal Airport that have resulted in fatalities or that have been considered serious accidents. Only minor landing mishaps that did not result in property damage have been reported by airport operations.

The closest large-scale public airport to the LNP site is the Ocala International Airport (Figure 2.2.1-204). Ocala International Airport maintains 155 aircrafts used for general aviation with approximately 110,000 operations annually. No plans to expand the runways are projected for the near future at Ocala International Airport; however, within in the next 10 to 15 years, the airport plans to expand. The airport was not able to provide accident information.

George T. Lewis Airport, also known as the Cedar Key Airport, is located on an island 1.6 km (1 mi.) west of Cedar Key and is owned by Levy County (Reference 2.2-222). The airport is public, does not have service staff, and has very light operations. George T. Lewis Airport has no aircraft types or operations data and has no plans to expand. The main function of this airport is to serve the resort and recreation activities at Cedar Key (Reference 2.2-222).

The Inverness Airport maintains 35 total aircraft with approximately 14,965 annual operations (25 single-engine, 1 twin-engine, 3 helicopters, 1 ultralight, and 5 military aircraft) (Reference 2.2-227).

The Hernando County Airport maintains166 total aircrafts with approximately 72,500 annual operations (125 single-engine, 16 twin-engine, 8 jets, 15 helicopters, and 2 ultralights). Currently, the airport is extending one of the runways and will eventually move both of them in the near future. No major accidents have been reported.

Approximately 135 aircrafts are based at the Gainesville Regional Airport, with 93,502 annual operations. Helicopters for the Gainesville Police and Alachua County Sheriff's Department are also housed at this airport, in addition to operating a flight school. Additional growth for the airport will be associated with the Eclipse 500. In the past 20 years, one accident resulting in a fatality has been reported.

The Cross City Airport maintains 11 total aircrafts with approximately 17,885 annual operations (9 single-engine, 1 jet, and 1 helicopter) (Reference 2.2-228).

Table 2.2.2-203 describes the types of aircraft and flying patterns for aircraft-associated airports within the region. According to the Federal Aviation Administration (FAA), there are no temporary flight restrictions (TFR) within 32 km (20 mi.) of the LNP site (Reference 2.2-223). The outer boundaries of five airways are routed within 2 mi. of the LNP site: V7-521, VR1006, J119, Q110-116-118 and Q112 (Figure 2.2.1-204).

2.2.2.8 Projections of Industrial Growth

The LNP site is located in the southern part of Levy County immediately east of U.S. Highway 19/98 (SR 55). The site is primarily timber and currently undeveloped. The Goethe State Forest is located to the northeast, and the surrounding area is undeveloped agricultural land or sparsely populated rural residential land use. Some commercial automotive service, parts, storage, and gas stations are located within 8 km (5 mi.) of the site. These facilities are primarily located along U.S. Highway 19 and County Route 40. Because Levy County is primarily rural, the majority of the industrial development in an 80-km (50-mi.) radius of the LNP site is located in the urbanized areas of Marion, and Citrus counties. Personal communication with the Levy County Planning Department indicates that no industrial growth is planned within an 8-km (5-mi.) radius of the project site.

Industrial development within a 16-km (10-mi.) radius of the LNP site is primarily concentrated in Inglis along County Route 40 and U.S. Highway 19, and is limited to metal fabrication, automotive repair shops, and several mining operations. Mines within the 16-km (10-mi.) radius of the LNP site include the Inglis Mine, located north of the CFBC; Holcim (US), Inc., located south of the CFBC; and Crystal River Quarry located in the community of Red Level. Gulf Rock Mine is located northwest of the LNP site and is inactive (Figure 2.2.1-203).

The LNP site is located in the southern portion of Levy County. Citrus County is located to the south and Marion County is located to the east. Table 2.2.2-204 shows the largest employers in Citrus, Levy, and Marion counties. The largest employers are within the utilities, education, and healthcare sectors (References 2.2-224, 2.2-225, and 2.2-226).

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 subsection. General Design Criterion 4, “Environmental and Missile Design Basis,” of Appendix A, “General Design Criteria for Nuclear Power Plants,” to 10 Code of Federal Regulations (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

In accordance with Regulatory Guide 1.206, C.I.2.2.3.1, design basis events are defined as those accidents that have a probability of occurrence on the order of 10^{-7} per year or greater, and have potential consequences serious enough to affect the safety of the plant to the extent that the guidelines in 10 CFR 100 could be exceeded. The expected rate of occurrence exceeding the guidelines in 10 CFR 100 (on the order of magnitude of 10^{-6} per year) is acceptable if, when combined with reasonable qualitative arguments, the realistic probability can be shown to be lower.

As presented in FSAR Subsection 2.2.2, there are no major industrial facilities within 8 km (5 mi.) of the plant site that could adversely affect the safety of the nuclear facility. A review of the materials transported or stored within this radius indicates that the only sources that present a potential hazard are road transportation of potentially explosive material, failures of the nearby natural gas pipelines, and the accidental release of toxic materials from the nearby water treatment plant. These sources are evaluated below.

2.2.3.2 Explosions

2.2.3.2.1 Transportation of Explosives

Potential sources of explosions from nearby activities described in FSAR Subsection 2.2.2 are limited to an explosion in highway transport.
U.S. Highway 19/98 is located to the west of the center of the site and its nearest approach to the site is approximately 1974 m (6477 ft.), as presented in Figure 2.2.2-201. This highway is mainly used for local traffic and local commodity deliveries only. There is no indication that explosives are transported past the project site by this highway (Reference 2.2-218).The major highway through the area is I-75 located approximately 45 km (28 mi.) east of LNP. This is the main north-south route through the area for commodity traffic. Other corridors, such as rail, water, and air, do not pose a potential hazard to LNP. There is no rail traffic within an 8-km (5-mi.) radius of LNP. Water traffic is presently limited to pleasure and/or fishing boats in the five navigable waterways near the site. There are no military facilities within 8 km (5 mi.) of the site. There are several small airports located 10.1 to 42.2 km (6.3 to 26.2 mi.) from the project site, none of which support long-range air traffic.

The method for establishing the safe distances for explosive materials can be based on a level of peak positive incident overpressure below which no significant damage would be expected. Per Regulatory Guide 1.91, this is conservatively established as 1 psi, defined by the following quantity distance relationship.

$$R \geq kW1/3$$

Where, R is the distance in feet from the exploding charge of W pounds (lb.) of trinitrotoluene (TNT) (50,000 lb. being the maximum probable hazardous solid cargo for a single highway truck). With R in feet and W in pounds, k = 45.

This results in a distance of 505 m (1658 ft.) for a pressure of 1 psi, which is well below the separation distance of 1974 m (6477 ft.) from U.S. Highway 19/98.

Therefore, there are no adverse effects on LNP due to the transport of explosives via roadway.

2.2.3.2.2 Stationary Explosives

The Florida Public Utilities facility, located 5.5 km (3.4 mi.) from LNP in Inglis, provides propane gas and has three tanks on-site. The facility houses one 113,562-liter (30,000-gallon) tank and two 68,137-liters (18,000-gallon) tanks. Due to the relatively small quantity and the large separation distance from LNP, the facility does not pose a potential hazard to LNP.

2.2.3.2.3 Nearby Gas Pipeline

There are three natural gas pipelines in the area of LNP as shown on Figure 2.2.2-202. There is a 20.3-cm (8-in.) pipeline lateral that splits from a 76.2-cm (30-in.) pipeline 2123 m (6966 ft.) northwest of LNP and runs to the north-northeast from LNP. It normally operates at or below 912 psi. Because the 20.3-cm (8-in.) pipeline is smaller and a greater distance from the plant, the accident analysis for this pipeline is considered to be bounded by the accident analysis for the 91.4-cm (36-in.) pipeline described below and does not require additional consideration.

The 76.2-cm (30-in.) pipeline is located to the west of the site and runs from the south-southwest to the north-northeast. It is 1769 m (5803 ft.) from the nearest location of the LNP safety-related structures at its closest approach west-northwest of the site as shown on Figure 2.2.2-201, and has a maximum operating pressure of 1200 psi.

The 91.4-cm (36-in.) pipeline runs parallel to the 76.2-cm (30-in.) pipeline, is located to the west of the site and runs from the south-southwest to the north-northeast. It is 1757 m (5763 ft.) from the nearest location of the LNP safety-related structures at its closest approach west-northwest of the site as shown on Figure 2.2.2-201, and has a maximum operating pressure of 1333 psi.

The distance between compressor stations for the 6.2-cm (30-in.) and 91.4-cm (36-in.) lines is 104.9 km (65.2 mi.) and the distance between isolation valves is 14.3 km (8.9 mi.) (first section), 31.2 km (19.4 mi.) (second section), 29.6 km (18.4 mi.) (third section), and 18.4 km (11.4 mi.) (fourth section). The impact of a postulated rupture of the two larger pipelines was evaluated with respect to LNP. The analysis includes the following assumptions:

- Unconfined vapor explosions of natural gas are not considered credible events. Therefore, deflagration of a natural gas/air mixture is taken as the limiting case. In terms of plant safety, this is considered as assuring that a mixture within the flammable limits is not present near the safety-related structures.

- The release rate due to a double-ended circumferential rupture of the 76.2-cm (30-in.) line is conservatively taken as a constant rate of 14,280 lb. per second from each side of the break. The release rate due to a double-ended circumferential rupture of the 91.4-cm (36-in.) line is conservatively taken as a constant rate of 23,726 lb. per second from each side of the break. These values assume that all of the released natural gas is vapor and exits the failed pipeline at sonic velocity.

- The postulated breach is modeled as a continuous plume release with Gaussian dispersion characteristics. The evaluation considered Pasquill stability categories C through G and wind speeds from 1 to 15 m per second (3.3 to 49.2 ft. per second). Pasquill stability categories are described in Regulatory Guide 1.23, Revision 1.

- Credit was taken for plume rise in accordance with Regulatory Guide 1.194.

Based on these assumptions, the maximum downwind concentration was determined to be less than 1 percent at LNP. This is well below the lower flammability limit (LFL) for natural gas of 4.8 percent in air. The downwind concentration was estimated as a function of stability classes C, D, F and G and wind speeds varying from 1 to 15 m per second (3.3 to 49.2 ft. per second). The results demonstrate that the maximum distance of the frontal boundary of flammable concentration (4.8 percent) from the pipeline is 1400 m (4594 ft.) for stability category D and a wind speed of 15 m per second (49.2 ft. per second). The majority of flammable portion of the gas cloud will be even closer to the pipeline and, therefore, farther from the LNP site. This results in minimum separation distance from the leading edge of a potentially flammable cloud to the site critical structures of 356 m (1169 ft.).

The heat intensity for a sustained jet fire at the break location was determined to be no more than 300 Btu/hr/ft^2 (equivalent to solar heat flux on the ground) at a distance of 1120 m (3677 ft.) from the 91.4-cm (36-in.) pipeline.

The potential overpressure from the deflagration of the vapor cloud at the closest point of approach (356 m [1169 ft.] from the site critical structures) is considered negligible (less than 1 psi).

Therefore, there are no adverse effects due to the unlikely rupture of the gas pipelines at their closest location to LNP.

2.2.3.3 Toxic Chemicals

As previously noted, there is no rail or barge traffic within 8 km (5 mi.) of LNP. The road transportation corridors within 8 km (5 mi.) of LNP include the following routes. U.S. Highway 19/98, located 1.9 km (1.2 mi.) west of LNP, is mainly used for local traffic and local commodity deliveries only. Four county roads are shown on Figure 2.2.2-201: County Road 40, 4.5 km (2.8 mi.) south; County Road 40A, 4.8 km (3.0 mi.) southwest; SR 336, 6.8 km (4.2 mi.) east-northeast; and County Road 337, 7.7 km (4.8 mi.) northeast of LNP. None of these roadways are considered to carry regular heavy truck traffic. Due to the lack of major industries in the area, significant commodity traffic on U.S. Highway 19/98 is expected to be minimal, with the preferred route for north-south commodity flow to be via I-75, which is 45.1 km (28 mi.) east of LNP (Reference 2.2-218). Therefore, there are no adverse effects to LNP likely due to the transportation of toxic materials.

Stationary hazardous chemical sources within 8 km (5 mi.) of LNP are limited to the Inglis WTP located 4.8 km (3 mi.) from LNP (Reference 2.2-208). The quantities stored at the plant are listed in Table 2.2.2-202. As shown in Table 2.2.2-202, the quantities stored are small and are not significant sources of airborne contamination even in the event of an accidental failure of the storage containers. Therefore, there are no sources of toxic chemicals within 8 km (5 mi.) of LNP that could pose a threat to LNP.

There are no toxic gas release event hazards identified for the LNP site from hazardous chemicals that are outside the scope of the DCD, as identified in DCD Table 6.4-201.

2.2.3.4 Fires

Fires originating from accidents at any facilities or transportation routes identified above do not have the potential to endanger the safe operation of LNP because the distances between potential accident locations and LNP are greater than 1.6 km (1 mi.). The closest potential source of a significant fire is the 76.2-cm (30-in.) natural gas line at 1769 m (5803 ft.) from LNP. An evaluation of the heat flux from a prolonged fire at the gas line results in a heat flux comparable to the maximum solar heat flux on the surface of the earth (approximately 300 British thermal units per hour per square foot) at about 883.9 m (2900 ft.) from the pipeline. In addition, the LNP main control room heating, ventilation, and air conditioning (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 LNP operators.

2.2.3.5 Collision with the Intake Structure

This subsection is not applicable, as LNP is not located on a navigable waterway with commercial traffic.

2.2.3.6 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.

2.2.3.7 Effects of Design Basis Events

There are no design basis events identified in FSAR Subsection 2.2.3 that require mitigating actions to be undertaken to eliminate or lessen the likelihood and severity of potential accidents. The consequences of the events in FSAR Subsection 2.2.3 do not cause design basis events that could result in significant impact to the ability of LNP to continue operation or to safely shut down.

STD DEP 1.1-1

2.2.4 COMBINED LICENSE INFORMATION FOR IDENTIFICATION OF SITE-SPECIFIC POTENTIAL HAZARDS

LNP COL 2.2-1

This COL item is addressed in FSAR Subsections 2.2.1, 2.2.2, and 2.2.3.

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LNP COL 2.2-1

Table 2.2.2-201 (Sheet 1 of 2)
Facilities within 8 Km (5 Mi.) of the LNP Site

Facility Type	Facility Name	Substance Stored
TIER 2	TOWN OF INGLIS – WTP	See Table 2.2.2-202
WW	FLYNN - INGLIS VILLAS APARTMENTS WASTE WATER TREATMENT FACIITY	Wastewater
WW & RCRIS-SQG	RISHER'S AUTO PARTS – SERVICE & STORAGE	Wastewater; other material not reported
WW	TOWN OF INGLIS – REVERSE OSMOSIS CONCENTRATE	Wastewater
WW	LAKE ROUSSEAU SAFARI CAMPGROUND	Wastewater
WW	RIVER LODGE RECREATIONAL VEHICLE PARK	Wastewater
WW	INGLIS MAIN DAM – GRADING MODIFICATIONS	Stormwater
WW & ICIS	FORESTRY YOUTH TRAINING CENTER WASTE WATER TREATMENT FACILITY	Wastewater; other material not reported
WW	NATURE COAST LANDING – PHASE 3	Stormwater and wastewater
UST & LUST	DAVES SUPER SERVICE	Diesel, unleaded and leaded gas
UST	SUPER STOP FOOD STORE	Unleaded gas
UST	LIL CHAMP FOOD STORE #6274	Kerosene
UST & LUST	CIRCLE K #7229	Unleaded gas
UST	BEASLEY TIRE CO	Unleaded and leaded gas
UST	CHEVRON-INGLIS	Diesel, unleaded gas, and waste oil
UST	LIL CHAMP FOOD STORE-HUNTLEY	Unleaded and leaded gas
UST	INGLIS TOWN-MAINT	Leaded gas
UST & AST	ALLENS BAIT & SEAFOOD	Fuel
UST	HAWTHORNE RICHARD	Fuel Oil
UST	GENE BABBIT	Unleaded gas

LNP COL 2.2-1

Table 2.2.2-201 (Sheet 2 of 2)
Facilities within 8 Km (5 Mi.) of the LNP Site

Facility Type	Facility Name	Substance Stored
LUST	KWIK STOP #4	Not reported
AST	SOUTHERN BELL	Diesel
RCRIS-SQG	HUNTLEY JIFFY FOOD STORES #274	Not reported
RCRIS-SQG	MIKEY TOWING & USED AUTO PARTS	Not reported
ERNS	910 EAST HWY 40	Not reported
ERNS	11333 NORTH HONEY JORDAN POINT	Not reported
CDL	63 RIVERSIDE DR	Not reported

Notes:

AST = aboveground storage tank
CDL = clandestine drug lab (law enforcement agencies located chemicals or other items that indicate the presence of a CDL or dumpsite)
ERNS = Emergency Response Notification System (ERNS reports releases of oil and hazardous substances)
ICIS = Integrated Compliance Information System (ICIS is a system that supports the information needs of the national enforcement and compliance program and the National Pollutant Discharge Elimination System (NPDES) program.
LUST = leaking underground storage tank
RCRIS-SQG = Resource Conservation and Recovery Information System – small quantity generators
Tier 2 = facilities that store or manufacture hazardous materials and submit a hazardous chemical inventory report to state and local agencies
UST = underground storage tank
WTP = water treatment plant
WW = wastewater (Existing, permitted facility/site for which effluent, reclaimed water or wastewater residual discharge into the environment and/or monitoring is taking place.)

Source: Reference 2.2-208

LNP COL 2.2-1

**Table 2.2.2-202
Chemicals Found at the Tier 2 Facility:
Town of Inglis Water Treatment Plant**

Chemical	Actual Quantity Stored On-Site
Calcium Hydroxide (hydrated lime in powdered form)	18[a]
Sulphuric Acid	50[b]
Chlorine (as bleach solution) Sodium Hypochlorite	Less than 850[b]
Phosphoric acid	100[b]
Phosphonic Acid	100[b]
Sodium Hydroxide	700[b]
Hydrogen Peroxide (< 52%)	35[b]

Notes:

a) Measured in tons.

b) Measured in gallons.

LNP COL 2.2-1

Table 2.2.2-203
Public Airports within 80 Km (50 Mi.) of the LNP Site

Airport	Approx. Distance to Site	Operations	Length and Orientation of Runway	Types of Aircraft Using the Facility	Aircraft Based on the Field	Flying Patterns Associated with the Airport
Marion County Dunnellon Airport	14.8 mi.	Average 41/day (14,965/year)	1) 4941 feet Oriented northeast & southwest 2) 4702 feet Oriented east-west	Local and transient aviation	52	Left hand traffic
Crystal River Airport	14.5 mi.	Average 100/day (36,500/year)	1) 4555 feet Oriented east-west 2) 3020 feet Oriented north-south	Local and transient aviation, air taxi, and military	50	Left hand traffic
Williston Municipal Airport	21.2 mi.	Average 44/day (16,060/year)	1) 6690 feet Oriented northeast-southwest 2) 4330 feet Oriented northwest-southeast	Local and transient aviation, and skydiving	36	Left hand traffic
Ocala International Airport	24.9 mi.	Average 110,000/year	1) 7400 feet Oriented north-south 2) 3000 feet Oriented east-west	Local and transient aviation	155	Left hand traffic
George T. Lewis/Cedar Key Airstrip	26.2 mi.	Very light operations. This strip is unmanned and public. No operations data available	2400 feet Orientated north-south	Local and transient aviation	0	Left hand traffic
Hernando County Airport	42.6 mi.	72,500/year	1) 5015 feet Oriented north-south 2) 7001 feet Orientated east-west	Local and transient aviation	166	Left hand traffic
Inverness Airport	26.2 mi.	14,965/year	5000 feet Oriented north-south	Local and transient aviation	35	Right/Left hand traffic
Gainesville Regional Airport	47.3 mi.	93,502/year	1) 7503 feet Orientated northwest 2) 4147 feet Orientated east-southeast	Local and transient aviation	135	Left hand traffic
Cross City Airport	48.9 mi	17,885/year	1) 5005 feet Oriented northeast - southwest 2) 5001 feet Oriented northwest to southeast	Local and transient aviation	11	Left hand traffic

Sources: References 2.2-210, 2.2-219, 2.2-220, 2.2-221,2.2-222, 2.2-227, and 2.2-228

LNP COL 2.2-1

Table 2.2.2-204
Largest Employers in Citrus, Levy, and Marion Counties

Company	Specialization	Employment
Citrus County		
1. Progress Energy	Utility	1100
2. Citrus County School Board	Education	1000
3. Citrus Memorial Hospital	Healthcare	1000
4. Seven Rivers Community Hospital	Healthcare	500
5. Pro-Line Boats	Boat Manufacturer	250
6. Citrus County Sheriff's Department	Law Enforcement	250
7. Spring Lodge 378	Business Services	100
8. Service Zone, Inc.	Business Consulting	100
9. Citrus County Detention Facility	Correctional Institution	100
10. Cypress Creek Correctional Facility	Correctional Institution	100
Levy County		
1. Levy County School Board	Education	876
2. Monterey Boats	Boat Manufacturer	495
3. Wal-Mart	Supermarket	467
4. White Industries	Construction	200
5. Williston Health Care Center, Inc.	Healthcare	197
6. D&B Construction	Construction	150
7. A&N Corporation	Vacuum Fitting	120
8. Williston Holding Company	Financial Holding Company	111
9. Central Florida Electric Co-Op, Inc.	Utility	93
10. V.E. Whitehurst	Construction	83
Marion County		
1. Munroe Regional Medical Center	Healthcare	2100
2. Emergency One, Inc.	Fire Apparatus Manufacturing	1309
3. Ocala Regional Medical Center	Healthcare	983
4. ClosetMaid	Wire Shelving Manufacturing	915
5. K-Mart Corporation	Distribution	650
6. Cingular Wireless	Customer Support Center	500
7. Lockheed Martin	Defense Contractor	500
8. Mark IV Automotive-Dayco Ocala	Automotive Parts Manufacturing	476
9. Swift Transportation Company, Inc.	Trucking	440
10. Class 1	Wire Harness Manufacturing	390

Sources: References 2.2-224, 2.2-225, and 2.2-226

2.3 METEOROLOGY

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

The meteorological parameters associated with the region surrounding the Levy Nuclear Plant Units 1 and 2 (LNP) site and the site itself, as described within this section, are bounded by the site parameters specified in Table 2-1 of the DCD and as compared in Section 2.0 of this Final Safety Analysis Report (FSAR).

2.3.1 REGIONAL CLIMATOLOGY

LNP COL 2.3-1

This subsection describes the general climate surrounding the LNP. Also included in this subsection is a summary of the regional meteorological conditions that provide a basis for the design and operating conditions of the Levy Nuclear Plant Unit 1 (LNP 1) and the Levy Nuclear Plant Unit 2 (LNP 2). A climatological summary of normal and extreme values of relevant meteorological parameters is presented for the first-order National Weather Service (NWS) stations or Automated Surface Observing System (ASOS) stations located in Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida. Figure 2.3.1-201 shows the locations of these meteorological observation stations with respect to the LNP site. Additional information regarding regional climatology was derived from various documents, which are referenced in the text below.

2.3.1.1 General Climate

The LNP site is located near the geographical west central portion of Florida in the gulf coast region. Five first-order meteorological observation stations are located within the general area surrounding the LNP site. The locations of these stations, which are all in Florida, and their distances from the LNP site are presented in Table 2.3.1-201. The Gainesville station is approximately 76 kilometers (km) (47 miles [mi.]) to the north-northeast of the LNP site; the Jacksonville station is 181 km (112 mi.) to the northeast; the Orlando station is 146 km (91 mi.) to the east-southeast; the Tallahassee station is 222 km (138 mi.) to the northwest, and the Tampa station is 125 km (78 mi.) to the south of the site. These fully instrumented meteorological stations are “first-order” meteorological observing stations, continuously recording a complete range of meteorological parameters. The observations are recorded continuously, either by automated instruments or by human observer, for the 24-hour period from midnight to midnight. The LNP site is located in Florida’s North Central state climate division of the National Climatic Data Center (NCDC). (Reference 2.3-229)

Climatological data for the general area surrounding the LNP site were obtained from several sources containing statistical summaries of historical meteorological data for these meteorological observation stations The references used to characterize the climatology include the following:

- Gale Research Company, *Climates of the States*, Third Edition (Reference 2.3-201).
- Gale Research Company, *Weather of U.S. Cities*, Fourth Edition (Reference 2.3-202).
- "Local Climatological Data, Annual Summary with Comparative Data" for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida, as published by the National Oceanic and Atmospheric Administration (NOAA) National Climatic Data Center (NCDC) (References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207).

The climatology of central Florida is characterized by mild winters and long, warm, and humid summers. Low temperatures are typically about 10 degrees Celsius (°C) (50 degrees Fahrenheit [°F]) in the winter and 21.1°C (70°F) during the summer. Afternoon highs range from the low 70s (°F) in the winter to the low 90s from June through September. Invasions of cold northern air can produce an occasional cool winter morning. Freezing temperatures typically occur one or two mornings per year during December, January, and February. In some years no freezing temperatures occur. Temperatures rarely fail to rise into the 60s (°F) on even cooler winter days. Temperatures above the low 90s (°F) are generally uncommon in the summer because of the afternoon sea breezes and thunderstorms. Information on prevailing wind speed and direction for the region is contained in FSAR Subsection 2.3.2.1.1. An outstanding feature of the climate is the summer thunderstorm season. Most thunderstorms occur in the late afternoon hours from June through September. The resulting sudden drop in temperature (associated with evaporative cooling) from about 32.2°C (90°F) to around 21.1°C (70°F) makes for a pleasant change. Between a dry spring and a dry fall, approximately 60 percent of the annual rainfall occurs during the summer months. Snowfall is very rare. Measurable snowfall in the area (more than $^{1}/_{4}$ inch) has occurred only a few times in the last 100 years. Although the surface of Florida is largely sandy in nature, the presence of prolific vegetation throughout the area is expected to preclude the occurrence of dust or sand storms. Given the generally flat and low elevation of the topography near the coast, the area is vulnerable to tidal surges. Tropical storms have threatened the area on a few occasions most years. The greatest risk of hurricanes has been during the months of June and October. Many hurricanes, by replenishing the soil moisture and raising the water table, do far more good than harm. The heaviest recorded rains during a 24-hour period have typically been associated with hurricanes.

Table 2.3.1-202 presents a summary of historical climatological observations from the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa meteorological observation stations.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Basis

2.3.1.2.1 Thunderstorms, Hail, and Lightning

The local climatological data (LCD) summaries for the cities in the area surrounding the LNP site indicate that thunderstorms have been observed on an average of 75.7 days per year in Gainesville (23-year period of record [POR]), 67.5 days per year in Jacksonville (59-year POR), 80.6 days per year in Orlando (50-year POR), 81.2 days per year in Tallahassee (59-year POR), and 81.3 days per year in Tampa (59-year POR). The LCD summaries for these cities also indicate that thunderstorms occur most frequently during the months of June, July, and August in all five locations. Gainesville averaged 14 days of thunderstorms in June, 18 days in August, and 16 days in July. Jacksonville averaged 12 days in June, 16 days in August, and 14 days in July. Orlando averaged 15 days in June, and 18 days in July and August. Tallahassee averaged 14 days, 19 days, and 16 days in June, July, and August, respectively. Tampa averaged 14 days in June, and 20 days July and August. Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa averaged five or more thunderstorm days per month from May through September and less than three days per month from October through April. A thunderstorm is normally recorded only if thunder is heard at the weather observation station. It is reported on a regularly scheduled observation if thunder is heard within 15 minutes preceding the observation (Reference 2.3-208). Otherwise, special observations are recorded as a thunderstorm whenever thunder is heard.

A severe thunderstorm is defined by NOAA as a thunderstorm that possesses one or more of the following characteristics (Reference 2.3-209):

- Winds of 50 knots (58 miles per hour [mph]) or more.
- Hail 1.9 centimeters (cm) (0.75 inch [in.]) or more in diameter.
- Thunderstorms that produce tornadoes.

Severe thunderstorms producing hail events with hail greater than 1.9 cm (0.75 in.) or more in diameter have been recorded since 1950. Forty-five events were reported in Levy County during the period from January 1, 1950, to November 30, 2008. Four storms resulted in reported property and crop damage (Reference 2.3-210). The number of reported hail events has increased significantly over time, primarily as a result of increased reporting efficiency and confirmation skill and the possible overlooking of storms in the early years of data collection. Additionally, the increase in urbanization over the past 50 years has effectively resulted in an increase in the number of reported storms, if for no other reason than there are more targets damaged by hail and thunderstorms in an urban area than in a rural area. As a result, there is a higher frequency of reported storms in urban areas than in rural areas. While 45 hail storms were reported in Levy County over the period of 1950 to 2008, the more recent storm reports

(Reference 2.3-209) indicate that there is a greater frequency of reported storms in more recent years.

The frequency of lightning flashes per thunderstorm day over a specific area can be estimated using Equation 2.3.1-1, which takes into account the distance of the location from the equator (Reference 2.3-211):

$$N = (0.1 + 0.35\sin\theta)(0.40 \pm 0.20) \qquad \text{Equation 2.3.1-1}$$

where

N = Number of flashes to earth per thunderstorm day per square kilometer (km^2)

θ = Geographical latitude

For the LNP site, the most northern boundary of the site is located at approximately 29.07° north latitude. The frequency of lightning flashes (N) is predicted to range from 0.054 to 0.162 flashes per thunderstorm day per km^2. The value 0.162 is used as the most conservative estimate of lightning frequency in the following calculations:

The average annual number of thunderstorm days in the area (i.e., as reported at the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa observation stations) is 77.26. This results in a predicted mean frequency of 12.52 lightning flashes per km^2 per year, as calculated below:

$$\frac{0.162\,flashes}{(thunderstorm-day)(km^2)} \times \frac{77.26\,thunderstorm-days}{year} = \frac{12.52\,flashes}{(km^2)(year)}$$

The total area of the LNP site is approximately 1257 hectares (ha) (3105 acres [ac.]). Hence, the predicted frequency of lightning flashes within the area of the LNP property is 157 per year, as calculated below:

$$\frac{12.52\,flashes}{(km^2)(year)} \times 12.57 km^2 = \frac{157\,flashes}{(year)}$$

The exclusion area boundary (EAB) for LNP 1 and LNP 2 is a radius of 1340 meters (m) (4396 feet [ft.]) around each unit. This is considered to be the approximate operational area of the LNP site. The predicted frequency of lightning flashes in the LNP exclusion area of a single reactor can be calculated as follows:

$$\frac{12.52\,flashes}{(km^2)(year)} \times 5.64 km^2 = \frac{70.6\,flashes}{(year)}$$

Therefore, the predicted number of lightning flashes in the immediate vicinity of LNP 1 and LNP 2 is predicted to be 70.6 per year.

2.3.1.2.2 Tornadoes and Severe Winds

Based on a 57.25-year period of record, the average number of tornadoes reported in Florida was 50.8 per year (Reference 2.3-212). A summary of the tornadoes reported in Florida is provided in Table 2.3.1-203, which summarizes, by tornado intensity, all tornadoes during the period from January 1, 1950, to March 31, 2007 (Reference 2.3-212). The storm intensities reported in the table are based on the original Fujita (as opposed to the recently introduced Enhanced-Fujita [E-F]) Tornado Scale. Both scales are used to estimate wind speeds associated with the amount of damage observed after the storm event, as opposed to actual measured wind speeds. During this period, the numbers and types of tornadoes reported in Florida were:

- 150 (F)
- 1559 (F0)
- 819 (F1)
- 327 (F2)
- 42 (F3)
- 4 (F4)
- 0 (F5)
- 1043 (Waterspouts)

These totals equate to an average of 27 F0, 14 F1, 6 F2, less than 1 F3, less than 1 F4, and 0 F5 tornadoes reported in Florida per year.

During the same period (January 1, 1950, to March 31, 2007), a total of 21 tornadoes were reported in Levy County. The number of reported tornadoes for Levy County and nine adjacent counties surrounding the LNP site are summarized in Table 2.3.1-204 using the original Fujita scale. A total of 336 tornadoes were reported during the period of record for the 10 counties (Levy, Dixie, Gilchrist, Alachua, Marion, Lake, Sumter, Citrus, Hernando, and Pasco) surrounding the LNP site (Reference 2.3-212). The worst reported tornado in Levy County, an F2, occurred in March 1993. This tornado resulted in three fatalities, 10 injuries, and a published damage estimate of $50 million. Table 2.3.1-205 summarizes the number of tornadoes in Florida by year and the (original) Fujita Tornado Scale Category for the period from January 1, 1950, to March 31, 2007. During this period, there were a maximum of 116 tornadoes in 1997 and a minimum of six in 1950.

Based on a statistical analysis of tornado occurrences in the United States over a 70-year period, Fujita (Reference 2.3-213) concluded that the indicated increase in tornado occurrences was primarily a result of increased reporting efficiency and confirmation skill and that F0- and F1-class tornadoes were typically overlooked during the early data collection years. Additionally, research conducted by Grazulis (as reported by Gaya et al.) concluded that the increase in urbanization over the past 50 years has effectively resulted in an increase in the number of reported tornadoes, if for no other reason than there are more targets destroyed or damaged by a tornado in an urban area than in a rural area (Reference 2.3-214). As a result, there is a higher frequency of reported incidents in urban areas than in rural areas.

The probability of a tornado strike for the LNP site can be calculated using an empirical relationship such as the following:

$$P_s = \bar{n}\left(a/A\right)$$

where

P_s = Probability that a tornado will strike a particular location during a 1-year interval.

$\bar{n}$ = Average number of tornadoes per year (i.e., equal to 5.8 for the ten-county area containing and surrounding the LNP site, as calculated from Table 2.3.1-204).

a = Average individual tornado area, equal to 0.81 square kilometers (km^2) (0.314 square miles [$mi.^2$]) for the LNP site, as calculated from Table 2-14 in NUREG/CR-4461, Rev. 2.

A = Total area of concern (e.g., 1° square between 29° and 30° latitude and 82° and 83° longitude) is equal to approximately 10,709.8 km^2 (4183.5 $mi.^2$).

Using this equation, the tornado strike probability for the LNP site, P_s, is estimated to be 0.000439, which corresponds to a return frequency of once in 2280 years.

Waterspouts, which are similar to tornadoes, have been observed to occur only over very large bodies of water, such as the Gulf of Mexico. Waterspouts are recorded in the NCDC Storm Event Database (Reference 2.3-212), and a review of the database indicated that approximately 1043 waterspouts have been reported in the state of Florida during the period from January 1, 1950, to March 31, 2007.

Design-basis tornado parameters have traditionally been based on the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.76 and other NRC published documents that have stated that the probability of occurrence of a tornado that exceeds the design-basis tornado should be less than about 1.0E-7 per year per nuclear power plant. The NRC's original Regulatory Guide 1.76 delineates maximum wind speeds of 386 kilometers per hour (km/h) (240 mph) to 579 km/h (360 mph), depending on the region of the United States in which the site is located. More recent evaluations have resulted in recommendations for reduced design-basis tornado wind conditions. American National Standards Institute (ANSI)/American Nuclear Society (ANS) 2.3 recommends a maximum tornado wind speed of 418 km/h (260 mph) and a tornado recurrence of 1.0E-6 per year (Reference 2.3-215). Although this standard has not been endorsed by the NRC, the NRC staff has endorsed and recommended the use of a maximum tornado wind speed of 483 km/h (300 mph) in the design of evolutionary and passive advanced light water reactors (ALWR) for sites east of the Rocky Mountains.

The determination of a design-basis tornado for a specific area of the United States is not design specific, but is location specific. In other words, for a given geographic location, a tornado with specific properties is related to an acceptable mean recurrence interval. This conclusion is unrelated to the reactor type. The maximum wind speed of 483 km/h (300 mph) for sites east of the Rocky Mountains, along with other associated parameters, have previously been evaluated and accepted by the NRC staff as an appropriate design-basis tornado.

The NRC re-evaluated the available tornado data in NUREG/CR-4461, Revision 1. The NRC study was based on a tornado data tape prepared by the National Severe Storm Forecast Center that contains 30 years' worth of data, including the data for approximately 30,000 tornadoes that occurred during the period from 1954 through 1983. Wind speed values associated with a tornado having a mean recurrence interval of 1.0E-7 per year were estimated to be about 322 km/h (200 mph) for states west of the Rocky Mountains and 483 km/h (300 mph) for states east of the Rocky Mountains.

Other characteristics associated with a maximum wind speed of 483 km/h (300 mph) have been identified by NRC for a wind speed of 483 km/h (300 mph); that is, rotational speed of 386 km/h (240 mph), maximum translational speed of 97 km/h (60 mph), radius of maximum rotational speed of 46 m (150 ft.), pressure drop of 2.0 pounds per square inch (psi), and rate of pressure drop of 1.2 psi per second (psi/sec).

Because actual measurement of site-specific tornado parameters is not practical, the site characteristics for tornado parameters have historically been based on the best available information, which has generally been reflected in the NRC guidance for the design-basis tornado (i.e., NRC Regulatory Guide 1.76). Further, NUREG/CR-4461, Revision 1, represents better available information than Regulatory Guide 1.76, and the latest NRC position on design basis tornadoes is based on the information in NUREG/CR-4461, Revision 1. This is further supported by NRC's Draft Guidance 1143, as follows:

- Rotational velocity = 386 km/h (240 mph).
- Maximum translational velocity = 97 km/h (60 mph).
- Maximum wind speed = 483 km/h (300 mph).

- Radius of maximum rotational velocity = 46 m (150 ft.).
- Total pressure drop = 13.8 kilopascals (kPa) (2.0 psi).
- Rate of pressure drop = 8.3 kilopascals per second (kPa/sec) (1.2 psi/sec).

The design parameters for the Westinghouse Electric Company, LLC (Westinghouse), AP1000 Reactor (AP1000) meet these criteria, as noted in Subsection 3.3.2.1 of Westinghouse Electric Company, LLC, Design Control Document for the certified design as amended (DCD). However, it is noted that NRC's most recent guidance on "Design Basis Tornadoes and Tornado Missiles for Nuclear Power Plants" is provided in Revision 1 of Regulatory Guide 1.76, Revision 1 published in March 2007. The revised guidance is based on the E-F scale rather than the original Fujita scale and provides for lower design-basis tornado characteristics than were previously specified in NRC's guidance. The current Regulatory Guide 1.76, Revision 1 guidance is as follows:

- Rotational velocity = 82 meters per second (m/s) (184 mph).
- Maximum translational velocity = 21 m/s (46 mph).
- Maximum wind speed = 103 m/s (230 mph).
- Maximum rotational velocity radius = 45.7 m (150 ft.).
- Pressure drop total = 83 millibars (mb) (1.2 psi).
- Pressure drop rate = 37 mb per second (mb/s) (0.5 psi/sec).

These parameters are NRC's published design-basis tornado parameters for the region surrounding the LNP site. They are less stringent than the proposed design criteria for the AP1000 units that will be used for LNP 1 and LNP 2. However, since the maximum site characteristics for wind speed and pressure drop associated with the guidance in NRC's Draft Regulatory Guide 1143 are higher than those in Regulatory Guide 1.76, Revision 1, the Draft Regulatory Guide 1143 values will be used as the maximum site characteristics for comparison with the DCD site parameters in FSAR Table 2.0-201.

Observed peak gust wind speeds at the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa stations were previously identified in Table 2.3.1-202. The observed peak gust wind speeds were reported, without distinction, as the higher of the peak gust, 3-second gust or 5-second gust for each station. As indicated in the table, the observed peak gust wind speeds at these stations were 103 km/h (64 mph), 124 km/h (77 mph), 169 km/h (105 mph), 134 km/h (83 mph), and 98 km/h (61 mph), respectively.

In addition to the observed peak gust wind speeds in the region that are presented in Table 2.3.1-202, a 3-second gust wind speed that represents a 100-year return period for the region has been established at 224 km/h (139 mph). The 3-second gust wind speed is based on the American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) 7-05, “Minimum Design Loads for Buildings and Other Structures” (Reference 2.3-216). The 3-second gust wind speed was obtained from the Engineering Weather Data (EWD) compact disc (CD) published by NOAA for the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa weather stations (Reference 2.3-217). The maximum published 3-second gust wind speed based on severe winds for these stations is 209 km/h (130 mph) (Orlando and Tampa) and is represented as the 50-year return 3-second gust at 10 m (33 ft.) above the ground. A conversion factor to estimate the 100-year return period for this value is provided in Table C6-7 of the reference document, “Conversion Factors for Other Mean Recurrence Intervals.” The conversion factor for a 100-year return period is 1.07, resulting in a 3-second gust wind speed in the region of 224 km/h (139 mph).

The 100-year return period 3-second gust wind speed for the region is based on observed values that were reported at the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa weather stations. The DCD site characteristic “basic” wind speed is a theoretical value extrapolated from basic wind speed plots provided in the ASCE guidance. The following paragraph provides a discussion of the site characteristic “basic” wind speed for the LNP site. For consistency with the methodology provided in the DCD, the theoretical wind speed values determined by this methodology are reported as the site characteristic operating basis (3-second gust, 50-year and 100-year recurrence) wind speed values in Table 2.0-201.

The DCD specifies the design operating-basis wind as the “basic” wind. This is defined as the “basic” wind speed of 145 mph based on the most severe location identified in ASCE 7-98, “Minimum Design Loads for Buildings and Other Structures.” This wind speed is the 3-second gust speed at 33 feet above the ground in open terrain (ASCE 7- 98, Exposure C). The ASCE “basic” wind speed is estimated from a plot of basic wind speeds provided as Figure 6-1B of the ASCE 7-05 document (i.e., a more recent version of ASCE 7-98). By following the procedure described in the DCD (i.e., using Figure 6-1B of the ASCE 7-05 reference document) the LNP site characteristic basic wind speed is 120 mph. This value is bounded by the DCD design value of 145 mph. A 1.07 scaling factor was also used to factor this number to a 100-year recurrence value (probability of occurrence of 0.01) of 128 mph, which is also bounded by the DCD design value.

2.3.1.2.3 Heavy Snow and Severe Glaze Storms

Winter weather events are defined as the occurrence of measurable precipitation in the form of snow, sleet, freezing rain, or cold rain. Large-scale cyclone and frontal activity is responsible for some winter precipitation; however, this is usually in the form of rain.

Trace amounts of snowfall do occur in Florida, but measurable snowfalls are extremely rare (typically less than 0.25 inch) and occur only a few times in most locations in Florida, as indicated in Table 2.3.1-202. The record snowfall in the region was at Jacksonville, where 3.81 cm (1.5 in.) of snow fell in February of 1958. The 50-year recurrent Ground Snow Load for the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa stations is reported by the EWD data as zero (Reference 2.3-217).

Subsection C.I.2.3.1.2 of NRC Regulatory Guide 1.206 and Interim Staff Guidance (ISG) DC/COL ISG-07, "Interim Staff Guidance on Assessment of Normal and Extreme Winter Precipitation Loads on the Roofs of Seismic Category I Structures," suggests that applicants identify winter precipitation events as site characteristics and site parameters for determining normal and extreme winter precipitation loads on roofs of Seismic Category I structures. Based on the historical record snowfall for the region and the estimated 50-year recurrent Ground Snow Loads (which are essentially zero), the estimations of normal and extreme winter precipitation events and the resulting normal and extreme winter precipitation roof loads are not necessary for the LNP site since they are not considered to be significant.

2.3.1.2.4 Hurricanes

Hurricanes have made landfall on both the Atlantic and Gulf of Mexico coastlines of Florida. From 1899 through 2002, Florida received 60 direct hits from hurricanes, an average of 0.57 storms per year. This accounts for 35.7 percent of all hurricanes affecting the entire U.S. coastline during the 104-year period. Florida has a coastline length of approximately 2172.6 km (1350 statute mi.), resulting in an average distance between landfalls of 36.2 km (22.5 mi.). Tropical storms (a storm with sustained winds of 39 – 73 mph) affect Florida with greater frequency than hurricanes. Florida has experienced 79 tropical storms in the same period — an average of 0.76 storms per year (Reference 2.3-218).

From 1899 through 2007, Florida has experienced 150 hurricanes and tropical storms. Of the 150 storms, 85 are tropical storms, 19 are Category 1, 19 are Category 2, 19 are Category 3, six are Category 4, and two are Category 5 hurricanes (Reference 2.3-218). Table 2.3.1-206 summarizes the number of tropical storms and hurricanes in Florida by year and the Saffir-Simpson Scale Category for the period from 1899 to 2007. The NOAA Coastal Services Center reports that during the 157-year period between 1851 and 2007, 21 hurricanes rated Category 1-5 have passed within 50 nautical miles of the LNP site, and 45 hurricanes rated Category 1-5 have passed within 100 nautical miles of the LNP site. Based on the reported number of hurricanes passing within the vicinity of the LNP site, the annual frequency of hurricanes is estimated to be 0.13 and 0.29 storms per year within 50 and 100 nautical miles of the LNP site, respectively. (Reference 2.3-231)

An additional review of the NOAA Coastal Services Center Website (information available at www.maps.csc.noaa.gov/hurricanes/) indicates that during the period of 1851 to 2007, 45 hurricanes rated Category 1-5 have passed within 100 nautical miles of the LNP site. This included a total of 10 Category 3 hurricane tracks and 1 Category 4 hurricane track. Using information collected from the NOAA Coastal Services Center, a maximum wind speed of 125 knots (144 mph) was observed on August 13, 2004, during Hurricane Charley.

Hurricanes deteriorate rapidly as they move onshore as a result of increased frictional drag and loss of energy. Once onshore, the increased frictional effects have a tendency to turn the winds inward toward the hurricane's center. This results in decreased surface wind speeds but enhanced low-level convergence and greater vertical velocities that are capable of producing intense rainfall and isolated tornadoes. The LNP site lies approximately 9.7 km (6 mi.) from the nearest coastline. The major effects from hurricanes on the area are expected to be high winds, heavy precipitation, and potential flooding due to storm surges.

2.3.1.2.5 Normal Operating Heat Sink Design Parameters

Mechanical draft cooling towers provide a heat sink during normal operation of LNP 1 and 2. The AP1000 reactor does not rely on site service water as a safety grade ultimate heat sink; therefore, this subsection establishes the meteorological design parameters for the mechanical draft cooling towers during normal operation, including any extreme meteorological conditions that could be encountered during operation of the plant. The primary controlling meteorological parameters for the cooling capacity of a mechanical draft cooling tower based system are wet and dry bulb temperatures. Table 2.3.1-207 provides a summary of statistically significant dry and wet bulb temperatures that are used to define the design temperatures at the LNP site, as obtained from the Jacksonville, Tallahassee, and Tampa meteorological observing stations. These data were obtained from the 30-year (1961-1990) Solar and Meteorological Surface Observation Network (SAMSON) database (Reference 2.3-219) and from the 24-year (1973-1996) NOAA EWD database (Reference 2.3-217).

As discussed in NRC's Regulatory Guide 1.27, the meteorological conditions resulting in the maximum evaporation and drift loss are considered to be the worst 30-day average combination of the controlling parameters — namely, the wet bulb temperature and the coincident 30-day average dry bulb temperature for the same period. Based on an evaluation of the historical meteorological data presented in Table 2.3.1-207, the site characteristic maximum 30-day running average wet bulb temperatures are 24.9°C (76.8°F), 24.8°C (76.6°F), and 25.5°C (77.9°F), respectively, for the Jacksonville, Tallahassee, and Tampa meteorological observing stations. The coincident 30-day average dry bulb temperatures for the same period are 28.1°C (82.6°F), 28.3°C (82.9°F), and 28.6°C (83.5°F), respectively.

As also discussed in NRC's Regulatory Guide 1.27, the meteorological conditions resulting in minimal water cooling would be the worst-case combination of the controlling parameters; namely, the maximum 1-day and 5-day average wet bulb temperatures and the corresponding 1-day and 5-day average coincident dry bulb temperatures for the same period. Based on an evaluation of the historical meteorological data presented in Table 2.3.1-207, the site characteristic maximum 5-day running average wet bulb temperatures for the 30-year period from 1961 to 1990 are 26.5°C (79.7°F), 26.1°C (79.0°F), and 26.9°C (80.4°F), for the Jacksonville, Tallahassee, and Tampa meteorological observing stations, respectively. The coincident 5-day running average dry bulb temperatures are 31.0°C (87.8°F), 30.9°C (87.6°F), and 30.2°C (86.4°F), respectively. The site characteristic maximum 1-day running average wet bulb temperatures are 27.7°C (81.9°F), 27.0°C (80.6°F), and 27.6°C (81.7°F), respectively, and the coincident 1-day running average dry bulb temperatures for the same period are 31.2°C (88.2°F), 32.1°C (89.8°F), and 31.0°C (87.8°F), respectively. The site characteristic wet bulb temperatures that were exceeded less than 1 percent of the time were 26.0°C (78.8°F), 25.8°C (78.4°F), and 26.1°C (79°F) for the Jacksonville, Tallahassee, and Tampa stations, respectively. The maximum wet bulb temperatures recorded for Jacksonville, Tallahassee, and Tampa during this period were 30.3°C (86.5°F), 30.4°C (86.7°F), and 29.5°C (85.1°F), respectively.

Because modern cooling towers have almost no drift losses, this is not considered to be a critical design parameter. Site wind velocities and direction are considered in designing the mechanical draft cooling towers to minimize any recirculation of air and vapor exiting the towers and to provide adequate tower capacity should any recirculation occur.

2.3.1.2.6 Inversions and High Air Pollution Potential

Weather records from many U.S. weather stations have been analyzed by Hosler (Reference 2.3-220), Holzworth (Reference 2.3-221), and Holzworth (Reference 2.3-222) with the objective of characterizing atmospheric dispersion potential. The expected seasonal frequencies of inversions based below 152 m (500 ft.) for Tampa, which is 125 km (78 mi.) to the south of the LNP site, are shown in Table 2.3.1-208. The extent of vertical mixing is a major factor in the determination of atmospheric diffusion characteristics. Low-level temperature inversions inhibit vertical mixing. As shown in Table 2.3.1-208, the inversion frequency in Tampa averaged 28 percent in summer season and 37 percent in winter season (Reference 2.3-220).

In general, mixing depths are characterized by a diurnal cycle of nighttime minimum and daytime maximum depths. The nighttime minimum is the result of surface radiational cooling. This cooling produces stable conditions, frequently coupled with low-level temperature inversions or isothermal layers. Daytime maximums are the result of surface heating, which produces instability and convective overturning through a larger portion of the atmosphere. When daytime (maximum) mixing depths are shallow (low inversion heights), air pollution potential is considered to be greatest. Mean monthly mixing depths for Tampa are shown in Table 2.3.1-209. The lowest mean monthly mixing depth occurs in January (730 m [2395 ft.]) and the greatest mean mixing depth occurs in May (1410 m [4625 ft.]) (Reference 2.3-221).

The LNP site is located in Levy County, which is currently designated by the U.S. Environmental Protection Agency (EPA) as being in attainment of the national ambient air quality standards (NAAQS) (Reference 2.3-223). The Florida Department of Environmental Protection (DEP), in collaboration with local environmental programs, operates a network of ambient air quality monitoring stations throughout the state. There are 13 monitoring stations in the geographic area surrounding the LNP site. The monitoring stations are located in Alachua, Citrus, Lake, Marion, and Pasco counties. There are no monitoring stations located within Levy County. These stations monitor for various NAAQS criteria pollutants (i.e., ozone, particulate matter of 2.5 micrometers (μm) and smaller [$PM_{2.5}$], particulate matter of 10 μm and smaller [PM_{10}], sulphur dioxide [SO_2], and carbon monoxide [CO]) (Reference 2.3-224, Reference 2.3-225).

2.3.1.2.7 Ambient Air Temperatures

A summary of the ambient air temperatures at the major meteorological observing stations surrounding the LNP site (i.e., Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa) is provided in Table 2.3.1-210 for the following frequencies of occurrence of dry and wet bulb temperature:

Maximum Temperatures:

- 0-percent Occurrence
- 0.4-percent Occurrence
- 1.0-percent Occurrence
- 2.0-percent Occurrence
- “Maximum Safety” (DCD Site Parameter)
- “Maximum Normal” (DCD Site Parameter)

Minimum Temperatures:

- 97.5-percent Occurrence
- 99.0-percent Occurrence
- 99.6-percent Occurrence
- 100-percent Occurrence
- “Minimum Safety” (DCD Site Parameter)
- “Minimum Normal” (DCD Site Parameter)

The “Maximum Safety” temperatures in Table 2.3.1-210 were developed using over 50 years of temperature observations and statistical regression techniques to estimate temperatures for a 100-year period. Observed temperature data and statistics were obtained from NOAA EWD data (Reference 2.3-217), NOAA SAMSON data (Reference 2.3-219), and the ASHRAE fundamentals handbook (Reference 2.3-226). The results are provided for comparison with the AP1000 DCD site parameters as listed in DCD Table 2-1. A discussion of each of the DCD site parameters for air temperature is provided in the subsections below.

2.3.1.2.7.1 Maximum Safety Dry Bulb and Coincident Wet Bulb Temperature

This DCD site parameter is represented by a single data pair consisting of a maximum dry bulb temperature of 115°F (minimum of 2 consecutive hours), and a coincident (same 2-hour period) wet bulb temperature of 86.1°F. The estimated Maximum Safety 100-year recurrent dry bulb and coincident wet bulb temperature data pairs shown in Table 2.3.1-210 are 104.4/82.3, 105.1/78.7, and 98.7/78.1°F respectively for Jacksonville, Tallahassee, and Tampa. When compared with the DCD site parameter data pair of 115/86.1°F, the maximum estimated regional site temperatures are seen to be bounded by the DCD site parameter, with the maximum dry and wet bulb components being well below the Maximum Safety DCD limits.

2.3.1.2.7.2 Maximum Safety Wet Bulb Temperature (Non-Coincident)

This DCD site parameter is represented by a maximum wet bulb temperature of 86.1°F that exists for a minimum of 2 consecutive hours. The estimated Maximum Safety 100-year recurrent non-coincident wet bulb temperature in the region (85.5°F, Tampa) does not exceed the DCD site parameter value of 86.1°F. Although higher wet bulb temperatures are reported in Table 2.3.1-210 (0 percent Occurrence values), those values are not representative of a consecutive 2-hour period.

2.3.1.2.7.3 Maximum Normal Dry Bulb and Coincident Wet Bulb Temperature

This DCD site parameter is represented by a single data pair consisting of a maximum dry bulb temperature of 101°F, in combination with a coincident (same hour) wet bulb temperature of 80.1°F. The Maximum Normal temperatures in Table 2.3.1-210, which are based on 0.4 percent annual exceedance temperatures, are well below the Maximum Normal DCD site parameter of 101°F dry bulb/80.1°F coincident wet bulb, with the highest observed values being 95°F dry bulb/78°F wet bulb (Jacksonville).

2.3.1.2.7.4 Maximum Normal Wet Bulb Temperature (Non-Coincident)

This DCD site parameter is represented by a maximum wet bulb temperature of 80.1°F, excluding the highest 1 percent of values. The highest Maximum Normal wet bulb temperature in Table 2.3.1-210 is 80°F (Tampa).

2.3.1.2.7.5 Minimum Safety Dry Bulb Temperature

This DCD site parameter is represented by a minimum dry bulb temperature of -40°F that exists for a minimum of 2 consecutive hours. The estimates of Minimum Safety temperatures that are provided in Table 2.3.1-210 are well above the DCD site parameter, with the lowest estimated Minimum Safety dry bulb temperature being only 3°F (Tallahassee).

2.3.1.2.7.6 Minimum Normal Dry Bulb Temperature

This DCD site parameter is represented by a minimum dry bulb temperature of -10°F. The Minimum Normal temperatures in Table 2.3.1-210, which are based on 99.6-percent annual exceedance temperatures, are well above the DCD site parameter of -10°F dry bulb. The lowest observed Minimum Normal dry bulb temperature at any of the observing stations is only 24°F (Tallahassee).

2.3.1.3 Effects of Global Climate Change on Regional Climatology

Global trends in various meteorological and geophysical parameters are currently the subject of much discussion in both the scientific community and in the media. While it may be evident (and expected) that changes in the averages of certain meteorological parameters are occurring over time (i.e., such as temperature and precipitation), it is also evident and generally acknowledged that the prediction of any such changes are difficult if not impossible to reliably predict. Even the most reliable climate change models are not capable of accurately predicting design basis extremes in weather patterns. A discussion of public concerns or speculations about climate change would not add to the resolution of these issues, nor would a discussion of changes in average global trends, because these data cannot be reviewed on a site-specific basis with any degree of accuracy or reliability. It is relatively easy to demonstrate that an increase in the average value of temperature (or precipitation) at a given location is much more likely to be a result of numerous increases in temperatures (or precipitation) in the "normal range" rather than increases in extreme values, because a change in a select number of extreme values will essentially have no measurable effect on longer term average values. Therefore, the information presented in this subsection of the FSAR is focused on the extreme meteorological conditions that will facilitate a plant design that will operate within these safety margins throughout the projected plant life of 40 to 60 years. This is accomplished by identifying historical extremes and projecting, in a scientifically defensible manner, the potential effects weather will have on the safety and operation of the LNP.

2.3.2 LOCAL METEOROLOGY

LNP COL 2.3-2 An on-site meteorological monitoring system has been in operation at the LNP site since February 1, 2007. The on-site tower is located approximately 1.4 km (0.9 mi.) west southwest of the location of LNP 1 and LNP 2 and consists of a 60.4-m (198-ft.) guyed, open-latticed design. The location of the tower is shown on Figure 2.3.3-201. The base of the tower is at approximately 13.7 m (45 ft.) above mean sea level (msl). Local meteorological monitoring results and summaries of the parameters monitored by the on-site system are described and presented in this subsection. A more detailed description of the on-site meteorological monitoring system and operational program is provided in FSAR Subsection 2.3.3.

The POR of on-site meteorological measurements is the 2-year period from February 1, 2007, to January 31, 2009.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

2.3.2.1.1 Wind Summaries

Hourly wind speed and direction measurements at the LNP site for the 2-year POR were used to prepare monthly and annual average joint frequency distributions of wind speed and wind direction by Pasquill Stability Category for the 10-m (33-ft.) and 60-m (197-ft.) levels of the on-site meteorological tower. The wind speed categories presented in the joint frequency distributions correspond to the 11 wind speed categories recommended in Regulatory Guide 1.23, Revision 1.

The lower-level (10-m [33-ft.]) wind direction and wind speed are summarized by individual Pasquill stability category (i.e., A through G) and for the “All Stability” category in Tables 2.3.2-201, 2.3.2-202, 2.3.2-203, 2.3.2-204, 2.3.2-205, 2.3.2-206, 2.3.2-207, and 2.3.2-208 for the 2-year POR. Additionally, the lower-level wind direction and wind speed are summarized monthly for the POR for the “All Stability” category in Tables 2.3.2-209, 2.3.2-210, 2.3.2-211, 2.3.2-212, 2.3.2-213, 2.3.2-214, 2.3.2-215, 2.3.2-216, 2.3.2-217, 2.3.2-218, 2.3.2-219, and 2.3.2-220. For the first year POR, graphical illustrations of the wind roses of wind speed and direction for the lower-level tower measurements are shown on Figure 2.3.2-201 (all stabilities, 1-year POR) and on Figures 2.3.2-202, 2.3.2-203, 2.3.2-204, 2.3.2-205, 2.3.2-206, 2.3.2-207, 2.3.2-208, 2.3.2-209, 2.3.2-210, 2.3.2-211, 2.3.2-212, and 2.3.2-213 (all stabilities, by month). It is noted that the information in Tables 2.3.2-208 to 2.3.2-220 indicates a relatively high frequency of “calm” winds at the 10-meter level (i.e., 18.8 percent of the total observations). A review of the hourly meteorological data indicated that, during the 1-year period of record, nearly all of the observed winds at the 10-meter level were observed to be in the range of “greater than 0” to less than 0.4 m/s (0.9 mph). Wind directions associated with these measurements do not reflect the characteristics of calm wind conditions in that the directions are not highly variable or abruptly changing, as would be expected during true calm (stagnant) conditions. The very low wind speeds observed at the 10-meter level are believed to be attributable to the height of the local forest canopy and its corresponding frictional influence on wind speeds at the 10-meter level. The wind speeds are representative of this site and the local area surrounding the LNP.

The upper-level (60-m [197-ft.]) wind direction and wind speed data are summarized by individual Pasquill stability category (i.e., A through G) and for the "All Stability" category in Tables 2.3.2-221, 2.3.2-222, 2.3.2-223, 2.3.2-224, 2.3.2-225, 2.3.2-226, 2.3.2-227, and 2.3.2-228 for the POR. Additionally, the upper-level wind direction and wind speed are summarized monthly for the POR for the "All Stability" category in Tables 2.3.2-229, 2.3.2-230, 2.3.2-231, 2.3.2-232, 2.3.2-233, 2.3.2-234, 2.3.2-235, 2.3.2-236, 2.3.2-237, 2.3.2-238, 2.3.2-239, and 2.3.2-240.

Graphical wind roses of wind speed and direction from the nearby Tampa, Gainesville, and Tallahassee airports are also provided for comparison with the on-site wind measurements described above. Figures 2.3.2-214, 2.3.2-215, and 2.3.2-216 illustrate these wind roses for the 5-year period from January 1, 2001 through December 31, 2005. It is noted that the wind roses for the Tampa and Gainesville observing stations are most similar to the LNP on-site annual wind rose (Figure 2.3.2-201) in that there is a notable east-west bias in the results, which is most likely attributable to the diurnal influence of sea breeze effects. These effects are much more distinct in the on-site data where a strong east-west wind direction is evident in the data. The Tallahassee wind rose is seen to exhibit more of a north-south bias in the results, which is also believed to be attributable to sea breeze influences, which is consistent with the proximity of the station to the east-west shoreline in that part of the state.

2.3.2.1.2 Ambient Temperature

Ambient temperature from the on-site monitoring system is measured at both the 10-m (33-ft.) and 60-m (197-ft.) levels, and differential temperature (used in determining wind stability classification) is measured between the 10-m (33-ft.) and 60-m (197-ft.) levels of the tower. The maximum temperature recorded by the system for the first year of on-site data was 34.6°C (94.3°F), and the minimum temperature was -3.9°C (25.0°F). A summary of the on-site temperature information, by month and for the first year of onsite data is presented in Table 2.3.2-241. Based on the maximum and minimum temperature observations in the table, the diurnal temperature range of the on-site temperatures during this period is approximately 20-22 degrees in the fall, winter, and spring seasons and approximately 14-17 degrees in the summer and early fall seasons. The on-site temperature measurements are consistent with the long-term regional observations from Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, which are also summarized in Table 2.3.2-241.

2.3.2.1.3 Dew-Point Temperature

Dew-point temperature is used as a measure of the absolute humidity in the air. It is the temperature to which air must be cooled to reach saturation/condensation, assuming pressure and water vapor content remain constant. The on-site composite monthly and annual dew-point measurements for the first year of on-site data are compared with regional long-term observations from the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville stations in Table 2.3.2-242. The observed on-site dew-point temperatures are consistent with and generally bounded by the long-term regional observations of dew-point temperatures.

2.3.2.1.4 Atmospheric Moisture

2.3.2.1.4.1 Relative Humidity

Maximum relative humidity usually occurs during the early morning hours, and minimum relative humidity is typically observed in the mid-afternoon. For the annual cycle, the lowest relative humidities occur in mid-spring, with the summer months typically exhibiting the highest relative humidities. Table 2.3.2-243 summarizes relative humidity observations from the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville meteorological observing stations.

2.3.2.1.5 Precipitation

The total precipitation observed at the LNP meteorological monitoring station during the first year of on-site monitoring was 109.09 cm (42.95 in.). Table 2.3.2-244 compares average monthly and annual precipitation measurements at the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville meteorological observation stations with the monthly and annual precipitation measurements from the LNP on-site meteorological monitoring station. The region displays some variance in total monthly and annual precipitation between stations from month-to-month and year-to-year, and the wettest period of the year is typically the summer, with approximately twice the monthly totals in those months as compared to winter months. The one year of on-site precipitation data presented here are considered to be consistent with and generally bounded by the long-term regional observations from the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville meteorological observing stations when compared with long-term periods of record at those locations (Table 2.3.2-244). Based on a review of the regional precipitation data, it appears to be reasonably representative of the site area; and there is no reason to expect that on-site measurements of precipitation would be significantly different.

2.3.2.1.6 Fog

Fog is an aggregate of minute water droplets suspended in the atmosphere near the surface of the earth. According to international definition, fog reduces visibility to less than 1.0 km (0.62 mi.). According to United States observing practice, ground fog is a fog that hides less than 60 percent of the sky and does not extend to the base of any clouds that may lie above it. Ice fog is fog composed of suspended particles of ice; it usually only occurs in high latitudes in calm, clear weather at temperatures below -28.9°C (-20°F) and increases in frequency as temperature decreases (Reference 2.3-227).

Table 2.3.2-245 summarizes the occurrence of fog at the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville meteorological observation stations. Heavy fog (i.e., visibility less than or equal to 0.4 km [0.25 mi.]) has been observed at Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville an average of 15.3, 46.5, 18.0, 49.8, and 39.3 days per year, respectively (Table 2.3.2-245). The greatest number of fog days typically occurs during the months of December through February. However, fog can be a very localized phenomenon, and the information provided in Table 2.3.2-245 is used as a regional estimate for fog occurrence. Based on a review of regional fog observations, they appear to be reasonably representative of the site area; and there is no reason to expect that on-site observations of naturally occurring fog would be significantly different. Given that the air quality of Florida is considered to be good, smog (generally considered to be a combination of fog and air pollution episodes) is not expected to occur in the region at any time.

2.3.2.1.7 Atmospheric Stability

A joint frequency distribution of wind speed, wind direction, and atmospheric stability is used in conjunction with a dispersion model to estimate the average rate of dispersion of routine and potential accidental radioactive releases. For the LNP site, joint frequency distributions have been generated from on-site data using the vertical temperature gradient and the variability of the horizontal wind to estimate atmospheric stability, as recommended in NRC Regulatory Guide 1.23, Revision 1. As previously noted, joint frequency distributions of wind speed, wind direction, and atmospheric stability measured at the LNP site for the period from February 1, 2007, to January 31, 2009, are provided in a series of 40 tables, beginning with Table 2.3.2-201 and ending with Table 2.3.2-240.

Based on the two years of meteorological data collected on the LNP site, temporal variations within the individual stability categories are relatively small. Almost 50 percent of all hours fall into either neutral (D) or slightly stable (E) stability categories. More that 25 percent of all hours fall into the stable (F) and extremely stable (G) stability categories. Extremely unstable (A), moderately unstable (B), and slightly unstable (C) categories combined occurred approximately 25 percent of the total hours. These distributions of stability category are generally consistent with what would be expected for this region and the high predominance of A through E stability is considered to be conducive to very good atmospheric dispersion conditions during the majority of the hours of the day.

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

The construction and operation of the LNP has the potential to influence the local micrometeorology in the immediate vicinity of the LNP site. These effects could occur as a result of minor changes to the topography and vegetation resulting from land clearing and the construction of additional buildings and supporting infrastructure, as well as the use of mechanical draft cooling towers for system heat rejection to the atmosphere. Changes to the local topography are not expected to be significant because the topography of the site and region is essentially flat. There should be no effect on diffusion characteristics in the area except in the immediate vicinity of the buildings. The use of mechanical draft cooling towers for system heat rejection will result in visible moisture plumes from the cooling tower during certain atmospheric conditions The amount of condensation of evaporated water vapor, and thus the formation of visible plumes from the cooling towers, are expected to be greatest during winter months when ambient air temperatures are cool.

Icing conditions caused by the freezing of condensed water vapor from cooling tower plumes could occur on vertical surfaces (such as buildings and equipment) and on horizontal surfaces (such as roadways) in the immediate vicinity of the cooling towers. However, given the climate in central Florida, these types of conditions are expected to occur only on rare occasions and only at on-site locations. Because of the large distances from the locations of the cooling towers to areas of public access (such as roadways), the potential for fogging and icing conditions at off-site locations is expected to be very small.

2.3.2.2.1 Topographical Description

The LNP site and surrounding region is relatively flat, with no significant terrain features that will otherwise be expected to adversely or unusually impact natural dispersion downwind of the plant. Figures 2.3.2-217, 2.3.2-218, 2.3.2-219, and 2.3.2-220 show cross sectional plots of elevation versus distance from the LNP center for each of 16 directional sectors. Figure 2.3.2-221 shows the existing topographic features within an 8-km (5-mi.) radius of the LNP. The area surrounding the LNP site is relatively flat, and no significant terrain modifications are expected during and after construction of the LNP. Figure 2.3.2-222 shows topographic features within an 80-km (50-mi.) radius of the LNP site, which is noted to be generally flat in all directions.

2.3.2.2.2 Fogging and Icing Effects Attributable to Cooling Tower Operation

As discussed in FSAR Subsection 2.3.2.2, the operation of the LNP will result in significant heat dissipation to the atmosphere.

Ground level fogging and icing impacts attributable to cooling tower operation are not expected to be significant at the LNP site. Although ground level fogging could occur in the immediate vicinity of the mechanical draft cooling towers, those events would only be expected at on-site locations and under relatively cold and moist atmospheric conditions and when building wake and downwash effects (i.e., from the cooling tower structures or from nearby plant structures) have an adverse influence on the dispersion of the cooling tower plumes.

The greatest frequency of occurrence of extended visible plumes from the cooling towers will likely occur during periods of high humidity when restricted visibility occurs naturally. At the Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville meteorological observing stations, naturally occurring fog with visibilities of less than 0.4-km (0.25-mi.) have been reported an average of 15 to 50 days per year (see Table 2.3.2-245).

Ice formation on structures or at ground level is not expected to occur to any significant extent in the vicinity of the LNP site. A summary of the climatological records from the Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa meteorological observing stations in Table 2.3.1-202 indicates that the average number of days of below freezing ambient temperatures in the region is only 3 days in Orlando and Tampa, 12 days in Gainesville. At the Jacksonville and Tallahassee stations, which are farther to the north, the average annual number of below freezing days is 18 and 34, respectively. There are no large safety-related plant structures or other nearby structures that are expected to be adversely affected by icing from the cooling tower plumes under any meteorological conditions that could reasonably be expected to occur.

2.3.2.2.3 Assessment of Heat Dissipation Effects on the Atmosphere

Mechanical draft wet cooling towers are used to dissipate heat to the atmosphere from LNP 1 and LNP 2 during normal operation. Although the cooling towers are not expected to have a significant influence on local meteorological conditions, under some circumstances there could be limited periods of time when visible cooling tower plumes may extend short distances from the cooling towers, possibly being visible from selected off-site locations. However, because of the large size of the project site, periods of cooling tower plume visibility will be very limited.

Under full power generation, it is expected that the cooling towers will evaporate up to 28,040 gallons per minute (gpm) (106,130 liters per minute [l/min]) of water, depending on weather conditions. Under most meteorological conditions, the discharge will condense upon leaving the tower, and the length of the visible plume will depend on the temperature and humidity of the atmosphere. Colder and more humid weather is typically conducive to longer plumes. There is also a very small potential for the occurrence of fogging and icing at or near ground level under certain meteorological conditions, primarily in the immediate vicinity of the cooling towers. Most of the time, visible plumes can be expected to extend only a short distance from the tower and then disappear by evaporation.

EPA's CALPUFF dispersion model was used to evaluate cooling tower plume behavior and to estimate the frequency of occurrence and length of visible cooling tower plumes (Reference 2.3-228). The analysis of cooling tower plume behavior was performed under the assumption of full load operation, with maximum heat dissipation to the atmosphere. The maximum potential system heat rejection rate to the cooling towers is 7.63E09 British thermal units per hour (Btu/hr) per unit, which was assumed to be a bounding value for purposes of the analysis. The physical and operating characteristics of the cooling towers for each of the two banks of towers (i.e., one bank of towers for each generating unit, LNP 1 and LNP 2) are as follows:

Number of cells	44
Orientation of cells	2x22
Length	362.8 m (1190 ft.)
Width	292.6 m (97 ft.)
Height	17.1 m (56 ft.)
Fan diameter	10.0 m (32.8 ft.) (per cell)
Circulating water flow rate	531,100 gpm (2,010,187 l/min)
Drift rate	0.0005 percent
Heat rejection rate	7.63E09 Btu/hr

The analysis of cooling tower plume behavior was performed using 1 year of hourly surface meteorological data (2003) from the Gainesville, Florida, observing station. The results of the CALPUFF analysis are summarized in Table 2.3.2-246. The table summarizes the predicted plume heights and lengths for all hours (excluding existing ambient fog conditions where no change in ambient conditions would result), as well as the predicted plume heights and lengths for daylight hours only. The table presents the information for each season (winter, spring, summer, and fall), as well as for the annual average. Plume height and length are provided for various distance categories of height and length. The results of the analysis indicate that visible plumes from the LNP cooling towers will remain very close to the towers, primarily on-site and within 100 m (328 ft.) of the cooling towers. Only a very small percentage of visible plumes are predicted to extend beyond 100 m (328 ft.) from a location midway between the cooling towers, with plumes greater than 1000 m (3280 ft.) predicted to occur less than approximately 2 percent of the time. During daylight hours, the frequency of occurrence is predicted to be less than 1 percent of the time. It is noted that the property boundary closest to the cooling towers is approximately 854 m (2800 ft.) to the west, and the nearest public road (U.S. Highway 19) is approximately 1.4 km (0.9 mi.) to the west of the nearest cooling tower bank (as illustrated in FSAR Figure 2.1.1-203). Based on this analysis, the expected frequency of occurrence of visible cooling tower plumes that will leave the property is expected to be very small, and the frequency of occurrence of visible plumes in the vicinity of the nearest public road is expected to be very infrequent. In addition to the plume visibility assessment, the analysis also indicated that there were no predicted occurrences of ground level fogging or icing beyond 1000 m (3280 ft.) of the cooling towers. Since the closest public road (US Highway 19) is located 1.4 km (0.9 mi.) from the nearest cooling tower, fogging or icing is not predicted or expected to occur in the vicinity of any public roadway.

No synergistic effects of cooling tower plumes mixing with plant radiological or any other releases are expected to occur. Any gaseous effluents released from the plant during operation would be expected to occur at different elevations and at locations other than the cooling towers. Also, any such releases would be at or near ambient temperature and no significant plume rise would occur. Because the cooling tower plume would be at a different elevation than the elevation of any radiological releases, the potential for the mixing of the plumes is expected to be minimal.

A very small fraction of the water circulating through the cooling towers will be carried into the plume as small water droplets, as evidenced by the manufacturer's specified drift loss rate of 0.0005 percent. The maximum amount of drift loss from the towers, with both units operating at maximum load, would therefore only be approximately 5.3 gpm (20.0 l/min). A small amount of dissolved and suspended solids will be contained in the drift leaving the towers, which can be expected to result in a small amount of solid particle deposition to the surface, primarily in close proximity to the plant.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Design and operating bases, such as tornado parameters and temperature and precipitation extremes are statistics that, by definition and necessity, are based on long-term regional records. Although data collected by the LNP on-site meteorological monitoring system is representative of site conditions, only one year of on-site data is available. Therefore, long-term regional data are considered most appropriate for use in establishing conservative estimates of climatological extremes. Therefore, the design and operating basis conditions were based on regional meteorological data, as previously described in FSAR Subsection 2.3.1.

2.3.3 ONSITE METEOROLOGICAL MEASUREMENT PROGRAMS

LNP COL 2.3-3 The LNP on-site meteorological measurement program began in February 2007 with the installation of a 60.4-m (198-ft.) guyed, open-latticed meteorological tower. The tower has been used to monitor meteorological parameters at two levels above ground level, and has operated continuously since it was first installed. Table 2.3.3-201 shows the current elevations of the operational sensors for all monitored parameters for both the lower and upper monitoring levels. Figure 2.3.3-201 shows a topographical map of the area and the location of the meteorological tower with respect to the LNP site and LNP 1, and LNP 2. The area surrounding the tower is generally covered with low-level vegetation (less than approximately 0.9 m [3 ft.] in height and indigenous to the central region of Florida) within several hundred feet of the tower in all directions. In the immediate vicinity of the tower base and within the security fence, gravel has been used as a means of controlling weeds. The presence of this gravel is not extensive and is not expected to have an influence on the parameters measured on the tower. The location of the LNP meteorological tower is ideally situated for use in support of the LNP Combined License Application (COLA). Therefore, the monitoring results obtained from the tower will be used to characterize the on-site meteorological conditions for the LNP site. The topography of the area, as discussed in FSAR Subsection 2.3.2, is essentially flat with no significant terrain variations that would influence or otherwise affect dispersion. Topographical cross sections of the region are provided on Figures 2.3.2-217, 2.3.2-218, 2.3.2-219, and 2.3.2-220, which show the topographical changes by direction from the center of the site out to a distance of 80 km (50 mi.) of the LNP site.

Two years of continuous and consecutive meteorological data from the on-site tower for the period from February 1, 2007, through January 31, 2009, are submitted with this COLA in the electronic format recommended in Appendix A of Regulatory Guide 1.23, Revision 1. These data are also used for the determination of short- and long-term diffusion estimates, as described in FSAR Subsections 2.3.4 and 2.3.5.

The planned operational meteorological monitoring program will be a continuation of the pre-operational program. The pre-operational meteorological program for LNP 1 and LNP 2 meets the guidance provided in Regulatory Guide 1.23, Rev. 1. The pre-operational monitoring program for LNP 1 and LNP 2 is planned to be continued as the operational program for both units. Given that the existing program is planned to be continued during operation, both programs are described jointly in the following sections.

2.3.3.1 Instrumentation

The meteorological tower was first installed and began operation on the LNP site in February 2007 in support of the development and licensing of the LNP. The on-site tower is located approximately 1.4 km (0.9 mi.) west-southwest of the LNP reactors as shown on Figure 2.3.3-201. There are no structures of any significance within 1 mile of the tower. There are two banks of mechanical draft cooling towers for LNP 1 and LNP 2, with each bank approximately 17.1 m (56 ft.) high, 29.6 m (97 ft.) wide, and 181.4 m (595 ft.) long. The orientation of the cooling towers relative to the meteorological tower is illustrated on Figure 2.3.3-201. There are no structures or vegetation in the area surrounding the tower that are within 10 obstruction heights as recommended in Regulatory Guide 1.23, Revision 1. The base of the meteorological tower is at an elevation of approximately 13.7 m (45 ft.) msl. A weatherproof National Electrical Manufacturer’s Association (NEMA)-type enclosure mounted to the tower houses the system datalogger and remote access equipment. The information monitored on the tower is routinely accessed, downloaded, and archived remotely. The system is powered by a self-contained solar power generator and communications with the tower are achieved through a modem that accesses a cellular telephone network. The monitoring system is compliant with all applicable requirements of Regulatory Guide 1.23, Revision 1, as summarized in Table 2.3.3-202.

Progress Energy evaluated potential impacts attributable to heat and moisture sources, specifically the LNP cooling towers, with respect to the location of the meteorological tower. The cooling system for the LNP Units 1 and 2 consists of two banks of mechanical draft cooling towers located approximately 600 m (1,968 ft) to the northeast of the meteorological tower, as shown on Figure 2.3.3-201. A graphical illustration of the first full year of meteorological data collected at the LNP site (February 1, 2007 through January 31, 2008) is provided in Figure 2.3.2-201. There is a notable east-west bias in the wind directions, which is attributable to the diurnal influence of classic sea breeze effects. Winds blowing from the northeast (in the direction of the meteorological tower) have been observed to occur approximately 9 percent of the time as indicated in Figure 2.3.2-201. The results of comprehensive cooling tower plume modeling analyses performed by Progress Energy indicated that visible plumes greater than 500 m in length can be expected to occur less than 3 percent of the time. Given that the observed frequency of northeast winds (i.e., in the direction of the meteorological monitoring tower) is only 9 percent, the frequency of occurrence of visible plumes extending to the monitoring tower would be less than 0.3 percent of the time. In addition, visible plume heights were found to be between 0 and 200 m approximately 63 percent of the time, with the majority of those expected to be at the upper end of that range based on buoyancy effects immediately upon release from the cooling tower. Based on these evaluations, the LNP cooling tower plumes are not expected to have a significant or measurable effect on the meteorological measurements made by the LNP meteorological monitoring system.

2.3.3.1.1 Wind Systems

Wind speed and direction is measured at the 10-m (32.8-ft.) and 60-m (196.8-ft.) levels. Lower-and upper-level wind speeds are recorded by sensors mounted on 3.7-m (12-ft.) retractable booms to minimize tower shadow effects. Wind direction, wind speed, and wind direction variance (sigma theta) are monitored at both the lower and upper levels of the tower.

2.3.3.1.2 Temperature Systems

Ambient temperature and delta-T are monitored at both the 10-m (32.8-ft.) and 60-m (196.8-ft.) levels of the tower. Two channels of differential temperature are monitored simultaneously between the lower and upper levels. The temperature probes are mounted in aspirated shields attached to a 2.5-m (8-ft.) retractable boom. Dew-point temperature is measured at the 10-m (32.8-ft.) level of the tower.

2.3.3.1.3 Precipitation and Solar Radiation Systems

Precipitation and solar radiation are monitored near ground level by sensors near the base of the tower.

2.3.3.1.4 Maintenance and Calibration

The equipment is checked and calibrated on a routine basis and in accordance with NRC guidance. Accumulated system data are routinely analyzed for inconsistent or erratic data, including a comparison with appropriate meteorological data obtained from other local or regional meteorological observation stations. In order to achieve the required level of system reliability (i.e., annual data recovery targets), the following maintenance program is followed:

- Calibrate datalogger input channels semiannually.
- Calibrate or replace wind sensors with National Institute for Standards and Technology (NIST)-traceable calibrated sensors semiannually.
- Calibrate precipitation monitoring device (rain gauge) semiannually.
- Calibrate or replace barometric pressure, dew-point temperature, and solar radiation channel sensors with NIST-traceable calibrated sensors annually.
- Check the two ambient/differential temperature channels for deviations. Temperature sensors are thermistors purchased with NIST-traceable calibration documentation. Thermistors are inherently stable (100-month drift less than 0.01°C) and routine replacement is therefore not necessary. Deviation between the two ambient/differential temperature channels is performed daily, which provides an early warning of a problem with one of these channels. During semi-annual maintenance and calibration activities, measurements from the temperature sensors are compared with test probes that are calibrated every 6 months to ensure proper sensor operation.
- The guy wires and the tower anchors are inspected on an annual basis.

2.3.3.1.5 Data Reduction

Data from the LNP datalogger system are retrieved via a remote connection through a cellular telephone link. Using a host computer, an off-site meteorological consultant retrieves the meteorological data from the datalogger on a daily basis (except weekends and holidays). The retrieved data are reviewed for potential problems and then checked for consistency with data obtained from an Automatic Weather Observing Station operated by the municipality of Ocala, as well as data from the nearby Crystal River Energy Complex. Erroneous data are discarded prior to insertion into the historical site database. The edited and reviewed 15-minute averaged data are then stored on electronic media.

The routine computer outputs include the following information:

- Summaries of data listing maximum temperature, minimum temperature, average temperature, barometric pressure, precipitation, solar radiation, and dew-point temperature as daily and monthly averages.
- Totals of hourly precipitation, and hourly averages of barometric pressure, ambient temperature, differential temperature, dew-point temperature, upper- and lower-level wind direction and wind speed, upper- and lower-level wind direction variance (sigma theta), Pasquill stability classes (as calculated in accordance with a procedure outlined in NRC's Regulatory Guide 1.23, Revision 1), and accumulated solar radiation (langlies per minute).
- Averages of all parameters in 15-minute increments except precipitation, which is displayed as a 15-minute total value.
- Distributions of joint wind frequency (as outlined in NRC's Regulatory Guide 1.23, Revision 1) for both upper and lower levels showing average wind speeds and number of unrecovered data hours.

2.3.3.1.6 Accuracy of Measurements

Table 2.3.3-202 summarizes the accuracy of the measurements of the monitored parameters and the criteria upon which the accuracies are based. The accuracy of the meteorological monitoring system during the 2-year POR of on-site data described in FSAR Subsection 2.3.2 is consistent with the requirements of NRC Regulatory Guide 1.23, Revision 1.

2.3.4 SHORT-TERM DIFFUSION ESTIMATES

2.3.4.1 Objective

LNP COL 2.3-4 Conservative estimates of the local atmospheric dilution factors (Chi/Q) for LNP 1 and LNP 2 were made using an atmospheric dispersion model and on site meteorological data for the period from February 1, 2007, through January 31, 2009. These data were prepared using 12 wind speed categories (including a calm wind category) and these data were formatted for use in NRC's PAVAN dispersion model. The wind speed categories are the same as recommended in NRC Regulatory Guide 1.23, Revision 1, except for the addition of a 0.4 m/s category for calms. The 0.4 m/s limit for this category corresponds to the manufacturer's stated instrument threshold wind speed. This is an exception to

NRC Regulatory Guide 1.23, Revision 1, which provides guidance for the use of 11 wind speed categories (plus calms). This change was made because of the high frequency of observed light winds at the LNP site and the low starting speed of the instrumentation. While almost no true “calm” winds were observed (i.e., no detectable wind speed) during the period of record, approximately 19 percent of all observed winds were assigned to the lowest wind speed category.

2.3.4.2 Chi/Q Estimates Using the PAVAN Computer Code and On-Site Data

The PAVAN computer code was used to calculate short-term accident Chi/Q values for the LNP 1 and LNP 2 EAB and low population zone (LPZ) distances of 1340 m (4396 ft.) and 4830 m (3 mi.), respectively. The LNP EAB, which was previously discussed in FSAR Section 2.1, is illustrated on FSAR Figure 2.1.1-203. The predicted LNP 1 and LNP 2 Chi/Q values are compared in Table 2.3.4-201 to the acceptance criteria established in DCD Subsection 15A and listed in DCD Table 15A-5 (values reproduced in the table).

The maximum predicted Chi/Q values were determined in accordance with guidance provided in NRC Regulatory Guide 1.145 for the 0.5 percent maximum sector Chi/Q and the 5 percent direction independent Chi/Q. Regulatory Guide 1.145 recommends that the set of 2-hour Chi/Qs be calculated for each windspeed-stability class combination. Next, the set of 2-hour Chi/Qs are ordered by frequency of occurrence and a cumulative probability distribution is constructed for each sector or for the entire site depending on the analysis. PAVAN, in its implementation of Regulatory Guide 1.145, interpolates or extrapolates the distributions to determine the Chi/Q value that occurs 0.5 percent of all time in each sector. This process can result in unreasonable Chi/Qs when extrapolations are made, especially with a large number of calms.

To establish more representative Chi/Q values, the maximum possible Chi/Q is assigned as the sector value whenever the PAVAN-extrapolated 0.5 percent sector-dependent Chi/Q is larger than the maximum possible Chi/Q. The maximum possible 2-hour Chi/Q is the largest value calculated for the recorded meteorological conditions in the sector. This occurs for calm conditions at very stable, class G, conditions with windspeeds of 0.4 m/s or less. The PAVAN model calculated 0.5 percent sector-dependent value is retained when the value is interpolated from the set of sector Chi/Qs. A 0.5 percent interpolated Chi/Q value will always be less than the maximum possible Chi/Q in the sector.

A similar analysis is performed at 5 percent for the direction independent Chi/Q around the entire site. However, the maximum possible Chi/Q will also be bounding for the 5 percent direction independent Chi/Q.

Input to the PAVAN model consisted of the following information:

- Meteorological Data: Joint frequency distribution of wind speed, wind direction, and atmospheric stability for 16 standard azimuthal sectors. Period of record February 1, 2007, to January 31, 2009 (Table 2.3.4-202).
- Wind Sensor Height: Lower – 10 m (33 ft.).
- Delta-Temperature Heights: 10 – 60 m (33 – 197 ft.).
- Number of Wind Speed Categories: 12 (including calm category).
- Minimum Building Cross Section: 2730 square meters (m^2) (29,385 square feet [$ft.^2$]) (DCD Figure 3.8.2-1).
- Containment Height: 43.9 m (144 ft.) (DCD Figure 3.8.2-1).
- Release Height: 10 m (33 ft.) (ground level default height).

Based on the locations of LNP 1 and LNP 2 with respect to the meteorological tower, the atmospheric diffusion parameters, sigma y and sigma z, are not expected to be unduly influenced by the meteorological or topographical conditions in the vicinity of the LNP site. Therefore, no modifications were made to the atmospheric dispersion parameters, sigma y and sigma z.
The results of the PAVAN analysis are summarized in Table 2.3.4-203 for the EAB and Table 2.3.4-204 for the LPZ.

2.3.4.3 Chi/Q Estimates for Short-Term Diffusion Calculations

The results from the Chi/Q analysis show that building wake effects have very little influence on predicted Chi/Q values, particularly for very short averaging periods. The results for the 0- to 2-hour 5 percent values at the EAB (Table 2.3.4-203) are not influenced by building wake effects. For averaging periods greater than 2 hours, the 5-percent values at the LPZ are slightly higher without building wake effects. These values are used for all further LNP COLA evaluations and analyses and are shown in Table 2.3.4-204.

2.3.4.4 Control Room Diffusion Estimates

Conservative estimates of the site-specific Chi/Q for the LNP 1 and LNP 2 control room were made using an atmospheric dispersion model and on-site meteorological data. The meteorological data consists of hourly data, covering the period from February 1, 2007, through January 31, 2009. Each record of the hourly data contains a location identifier, Julian day, hour, lower level (10 m) direction, lower level speed, stability class, upper level (60 m) direction, and upper level speed.

NRC's ARCON96 computer code was used to calculate short-term accident Chi/Q values for the LNP 1 and LNP 2 control room. The predicted LNP 1 and LNP 2 Chi/Q values are compared in Table 2.3.4-206 to the acceptance criteria established in DCD Subsection 15A and listed in Table 15A-6 of the DCD (values reproduced in the table).

The maximum predicted Chi/Q values were determined in accordance with NRC Regulatory Guide 1.194.

Input to the ARCON96 model other than the site-specific meteorological data consisted of the data provided in DCD Table 15A-7 and DCD Figure 15A-1 and FSAR Table 2.3.4-207 and FSAR Figure 2.1.1-203. DCD Table 15A-7 provides the release and receptor elevations and the horizontal distance between the release and receptor points. DCD Figure 15A-1 shows the orientation of the AP1000 and the locations of the release and receptor points. FSAR Figure 2.1.1-203 shows the plant layout on the site, including the true north direction. Plant north is given as 45 degrees clockwise. Table 2.3.4-207 provides the site-specific release/receptor azimuthal angles for input to ARCON96.

2.3.5 LONG-TERM DIFFUSION ESTIMATES

2.3.5.1 Objective

LNP COL 2.3-5 Estimates of long-term Chi/Q and relative deposition (D/Q) were made using a straight-line Gaussian model, consistent with the requirements of NRC Regulatory Guides 1.109 and 1.111. The objective was to calculate Chi/Q and D/Q values at the following locations in each of the 16 primary directions, including:

- EAB (as described in FSAR Subsections 2.1.2 and 2.3.4.2).
- LPZ (as measured from the site centerpoint).
- Distance to nearest milk cow.
- Distance to nearest milk goat.
- Distance to nearest garden.
- Distance to nearest meat animal.
- Distance to nearest residence.
- Distances of 0.8, 1.2, 1.6, 2.4, 3.2, 4.0, 4.8, 5.6, 6.4, 7.2, 8.0, 12.0, 16.0, 22.5, 32.0, 40.0, 48.0, 56.0, 64.0, 72.0, and 80.0 km (0.5, 0.75, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 7.5, 10.0, 15.0, 20.0, 25.0, 30.0, 35.0, 40.0, 45.0, and 50.0 mi.) from the LNP site.

The distances listed above (i.e., distances to nearest milk cow, milk goat, garden, meat animal, and resident) were measured from the midpoint between LNP 1 and LNP 2.

FSAR Subsection 2.3.5.2 provides additional information on the calculations and results of long-term Chi/Q estimates for the LNP site.

2.3.5.2 Calculations

The calculations of Chi/Q and D/Q values at the locations and distances listed above were made using NRC's XOQDOQ computer program using 2 years of hourly, on-site meteorological data.

Assumptions used in the analysis are summarized below:

- Meteorological Data Source – LNP on-site meteorological tower.
- Period of Record – February 1, 2007, to January 31, 2009.
- Wind Reference Level – 10 m (33 ft.).
- Stability Calculation – Delta-Temperature (10- and 60-m [33- and 197-ft.] tower levels).
- Release Type – Ground level.
- Release Height – 10 m (33 ft.).
- Building Wake Effects – Included (see FSAR Subsection 2.3.4.2).
- For sectors containing nearest milk cow, milk goat, garden, meat animal, and residence, it was assumed that if these did not exist within 8 km (5 mi.) of the LNP site, 8 km (5 mi.) was assumed as the location of the receptor.

Based on the location of LNP 1 and LNP 2 with respect to surrounding topography, the atmospheric diffusion parameter, sigma z, is not expected to be significantly influenced by topographical conditions; therefore, no modifications were made to this atmospheric dispersion parameter. The site is also located approximately 12.8 km (7.9 mi.) from the Gulf of Mexico and the regime of horizontal or vertical dispersion is not expected to be influenced by internal thermal boundary layer effects attributable to the land-sea interface.

The distances that were used in the calculations using the XOQDOQ model ranged from a minimum of 2576 meters (1.6 miles) to a maximum of 8049 meters (5 miles), as described in Table 2.3.5-201. The wind speed associated with the lowest wind speed category used in the analysis was 0.4 m/s (0.9 mph) as described in FSAR Subsection 2.3.2.1.1. The use of the lowest wind speed category in the calculation over the entire distance of transport corresponds to maximum travel times in the range of approximately 1.8 to 5.5 hours. While there is a sea-breeze driven diurnal variation in wind directions that is evident in the meteorological observations (with predominant east-west directions), these travel times are both feasible and consistent with the use of the XOQDOQ straight-line trajectory dispersion model in this application."

The results of the long-term Chi/Q and D/Q have been summarized in Tables 2.3.5-201, 2.3.5-202, 2.3.5-203, and 2.3.5-204. Table 2.3.5-201 contains the Chi/Q calculations for routine releases, and Table 2.3.5-202 contains D/Q calculations for routine releases accounting for deposition effects. Table 2.3.5-203 contains Chi/Q calculations based on radioactive decay with an overall half-life of 2.26 days for short-lived noble gases. Table 2.3.5-204 contains Chi/Q calculations based on radioactive decay with an 8-day half-life for all iodines released to the atmosphere.

Based on these analyses, the established site characteristic value for the maximum average annual dispersion factor at the EAB is a value of 1.9E-05 sec/m^3 for any given sector (based on west-southwest sector; refer to Table 2.3.5-201).

2.3.6 COMBINED LICENSE INFORMATION

2.3.6.1 Regional Climatology

LNP COL 2.3-1 This COL Item is addressed in FSAR Subsection 2.3.1.

2.3.6.2 Local Meteorology

LNP COL 2.3-2 This COL Item is addressed in FSAR Subsection 2.3.2.

2.3.6.3 Onsite Meteorological Measurements Program

LNP COL 2.3-3 This COL Item is addressed in FSAR Subsection 2.3.3.

2.3.6.4 Short-Term Diffusion Estimates

LNP COL 2.3-4 This COL Item is addressed in FSAR Subsection 2.3.4.

2.3.6.5 Long-Term Diffusion Estimates

LNP COL 2.3-5 This COL Item is addressed in FSAR Subsection 2.3.5.

2.3.7 REFERENCES

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LNP COL 2.3-1

Table 2.3.1-201
Regional Meteorological Observation Station Locations

Station	Latitude (N)		Longitude (W)		Distance from LNP Site[a]	Direction from LNP Site[a]
	Degree	Minute	Degree	Minute	km (mi.)	(Compass)
Gainesville, FL	29	41	-82	16	76 (47)	NNE
Jacksonville, FL	30	29	-81	41	181 (112)	NE
Orlando, FL	28	26	-81	19	146 (91)	ESE
Tallahassee, FL	30	23	-84	21	222 (138)	NW
Tampa, FL	27	57	-82	32	125 (78)	S

Notes:

a) See Figure 2.3.1-201

N = north
E = east
S = south
W = west
km = kilometer
mi. = mile

Sources: References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-1

Table 2.3.1-202 (Sheet 1 of 3)
Climatological Data from Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida

	Station									
Parameter	Gainesville	POR (years)	Jacksonville	POR (years)	Orlando	POR (years)	Tallahassee	POR (years)	Tampa	POR (years)
Location										
Distance from LNP Site (mi.)	47		112		91		138		78	
Direction from LNP Site	NNE		NE		ESE		NW		S	
Elevation Above Mean Sea Level (ft.)	134		26		90		57		8	
Temperature										
Average Annual Observed (°F)	68.7	25	68.8	59	72.5	54	67.6	59	72.3	74
Maximum Observed (°F)	108 (7/2000)	23	105 (7/1942)	65	102 (5/1945)	64	103 (7/2000)	46	99 (6/1985)	60
Minimum Observed (°F)	10 (1/1985)	23	7 (1/1985)	65	19 (1/1985)	64	6 (1/1985)	46	18 (12/1962)	60
Normal Degree days/year (heating)	1143	30	1354	30	580	30	1604	30	591	30
Normal Degree days/year (cooling)	2659	30	2627	30	3428	30	2551	30	3482	30
Relative Humidity (%)										
Annual average at 7 A.M.	93	30	91	30	91	30	91	30	88	30
Annual average at 1 P.M.	59	30	58	30	56	30	55	30	59	30
Wind										
Annual average speed (mph)	6.3	23	6.8	23	7.9	23	5.6	23	7.1	23

LNP COL 2.3-1

Table 2.3.1-202 (Sheet 2 of 3)
Climatological Data from Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida

	Station									
Parameter	Gainesville	POR (years)	Jacksonville	POR (years)	Orlando	POR (years)	Tallahassee	POR (years)	Tampa	POR (years)
Wind (continued)										
Prevailing direction	ENE	23	NE	39	N	41	N	31	ENE	38
Peak Gust Speed (mph)[(a)]	64 (9/2004)	8	77 (7/1998)	10	105 (8/2004)	10	83 (9/1990)	10	98 (5/1979)[b]	11
Direction[(a)]	NE	8	W	10	ESE	10	ESE	10	NNW	11
Precipitation (in.)										
Annual average	48.36	30	52.34	30	48.35	30	63.21	30	44.77	30
Monthly maximum	16.45 (9/2004)	23	19.36 (9/1949)	65	19.57 (7/1960)	64	20.12 (7/1964)	46	20.59 (7/1960)	60
Monthly minimum	T (10/1987)	23	0.04 (12/1956)	65	T (12/1944)	64	T (10/1987)	46	T (3/2006)	60
24-hour maximum	6.16 (9/1988)	23	10.17 (9/1950)	65	9.67 (9/1945)	64	10.13 (7/2001)	46	12.11 (7/1960)	60
Maximum annual	58.37 (2004)	23	79.63 (1991)	30	67.85 (1994)	30	104.18 (1964)	46	67.71 (1997)	30
Snowfall (in.)										
Annual average	0.0	30	0.0	30	0.0	30	0.0	30	0.0	30
Monthly maximum	T (4/1997)	15	1.5 (2/1958)	60	T (5/1997)	34	1.0 (12/1989)	36	0.2 (1/1977)	60
Maximum 24-hour	T (4/1997)	15	1.5 (2/1958)	60	T (5/1997)	34	1.0 (12/1989)	36	0.2 (1/1977)	60

LNP COL 2.3-1

Table 2.3.1-202 (Sheet 3 of 3)
Climatological Data from Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida

Parameter	Station									
	Gainesville	POR (years)	Jacksonville	POR (years)	Orlando	POR (years)	Tallahassee	POR (years)	Tampa	POR (years)
Mean Annual (number of days)										
Precipitation ≥ 0.01 in.	125.4	30	115.9	30	117.0	30	113.5	30	104.3	30
Snow, sleet, hail ≥ 1.0 in.	0.0	30	0.0	30	0.0	30	0.0	30	0.0	30
Heavy fog (visibility 0.25 mi. or less)	46.5	23	39.3	43	18.0	39	49.8	43	15.3	43
Maximum temperature ≥ 90°F	89.5	30	78.4	30	108.7	30	92.2	30	90.0	30
Minimum temperature ≤ 32°F	11.7	30	18.3	30	2.7	30	34.4	30	2.7	30

Notes:

a) Reported wind speeds are the higher of peak gust, 3-second gust, or 5-second gust.
b) See National Institute of Standards and Technology (NIST) database of peak gust wind speeds.

°F = degrees Fahrenheit
N = north
E = east
S = south
W = west
ft. = foot
in. = inch
mi. = mile
mph = miles per hour
POR = period of record
T = trace amount

Sources: References 2.3-201, 2.3-202, 2.3-203, 2.3-204, 2.3-205, 2.3-206, 2.3-207, and 2.3-230

LNP COL 2.3-1

Table 2.3.1-203
Summary of Reported Tornado Occurrences in Florida

Tornado Intensity (Fujita Tornado Scale)	Number of Reported Occurrences January 1, 1950, to March 31, 2007
F	150
F0	1559
F1	819
F2	327
F3	42
F4	4
F5	0
Waterspouts	10

Notes:

F = Fujita tornado scale intensity was not available for these storm events.
F0 = 40 – 72 mph
F1 = 73 – 112 mph
F2 = 113 – 157 mph
F3 = 158 – 206 mph
F4 = 207 – 260 mph
F5 = 261 – 318 mph

mph = miles per hour

Source: Reference 2.3-212

LNP COL 2.3-1

Table 2.3.1-204
Summary of Reported Tornado Occurrences in Levy and Surrounding Counties

County	F0	F1	F2	F3	F4	F5	Number of Reported Occurrences (1950 to 2007)
Levy	11 (1)	7	2	0	0	0	21
Dixie	2	1	0	0	0	0	3
Gilchrist	1	0	2	0	0	0	3
Alachua	18 (1)	12	8	0	0	0	39
Marion	22	22	9	1	0	0	54
Lake	17 (3)	18	7	3	0	0	48
Sumter	6 (1)	1	3	1	0	0	12
Citrus	30 (1)	11	2	1	0	0	45
Hernando	25	7	0	0	0	0	32
Pasco	51 (5)	17	6	0	0	0	79

Notes:

These statistics are based on the reporting periods between January 1, 1950, and March 31, 2006. Numbers listed in parentheses indicate tornadoes reported without a Fujita scale intensity (“F” instead of “F0”).

F0 = 40 – 72 mph
F1 = 73 – 112 mph
F2 = 113 – 157 mph
F3 = 158 – 206 mph
F4 = 207 – 260 mph
F5 = 261 – 318 mph

Source: Reference 2.3-212

LNP COL 2.3-1

Table 2.3.1-205 (Sheet 1 of 3)
Reported Tornado Occurrences in Florida: January 1950 to March 31, 2007

Year	F0	F1	F2	F3	F4	F5	Waterspouts	Total
1950	(1)	3	2	0	0	0	0	6
1951	(1)	4	4	0	0	0	0	9
1952	1	6	3	0	0	0	0	10
1953	5 (2)	8	6	0	0	0	0	21
1954	5 (2)	2	3	1	0	0	0	13
1955	1 (6)	3	2	0	0	0	0	12
1956	3 (1)	0	4	2	0	0	0	10
1957	3 (4)	4	6	0	0	0	0	17
1958	2 (6)	9	4	5	1	0	0	27
1959	2 (5)	5	5	1	0	0	0	18
1960	6 (4)	10	10	1	0	0	0	31
1961	3 (6)	7	8	0	0	0	0	24
1962	3 (1)	6	7	1	0	0	0	18
1963	(7)	10	11	0	0	0	0	28
1964	2 (8)	14	12	1	0	0	0	37
1965	1 (2)	4	4	1	0	0	0	12
1966	4 (10)	9	4	0	3	0	0	30
1967	(6)	5	9	3	0	0	0	23
1968	7 (14)	27	13	0	0	0	0	61
1969	6 (21)	20	11	0	0	0	0	58
1970	2 (23)	10	12	1	0	0	0	48
1971	16	25	16	1	0	0	0	58
1972	18	27	30	2	0	0	0	77
1973	16	18	16	0	0	0	0	50
1974	24	28	6	0	0	0	0	58
1975	42	55	6	1	0	0	0	104
1976	54	13	0	0	0	0	0	67
1977	26	8	1	0	0	0	0	35
1978	62 (1)	21	9	1	0	0	0	94

LNP COL 2.3-1

Table 2.3.1-205 (Sheet 2 of 3)
Reported Tornado Occurrences in Florida: January 1950 to March 31, 2007

Year	F0	F1	F2	F3	F4	F5	Waterspouts	Total
1979	39 (13)	31	7	0	0	0	0	90
1980	20	38	1	1	0	0	0	60
1981	31	25	4	1	0	0	0	61
1982	31	32	8	0	0	0	0	71
1983	19	50	23	1	0	0	0	93
1984	19	10	1	1	0	0	0	31
1985	20	14	1	1	0	0	0	36
1986	40	10	3	0	0	0	0	53
1987	36	7	2	0	0	0	0	45
1988	27	14	5	1	0	0	0	47
1989	60	9	3	0	0	0	0	72
1990	51	6	0	0	0	0	0	57
1991	46	10	0	0	0	0	0	56
1992	46	11	2	1	0	0	0	60
1993	36 (2)	12	4	0	0	0	1	57
1994	41 (1)	5	1	0	0	0	4	52
1995	67 (3)	14	4	0	0	0	5	93
1996	58	7	5	0	0	0	0	70
1997	88	25	2	1	0	0	0	116
1998	70	32	7	6	0	0	0	115
1999	49	11	1	0	0	0	0	61
2000	65	12	1	0	0	0	0	78
2001	53	14	6	0	0	0	0	73
2002	36	8	0	0	0	0	0	44
2003	35	8	2	0	0	0	0	45
2004	77	26	4	0	0	0	0	107
2005	45	9	2	0	0	0	0	56
2006	33	5	4	0	0	0	0	42
2007 [(a)]	7	3	0	5	0	0	0	15

LNP COL 2.3-1

Table 2.3.1-205 (Sheet 3 of 3)
Reported Tornado Occurrences in Florida: January 1950 to March 31, 2007

Year	F0	F1	F2	F3	F4	F5	Waterspouts	Total
Total	1559 (150)	819	327	42	4	0	10	2911
Average	26.88 (2.59)	14.12	5.64	0.72	0.07	0	0.17	50.19

Notes:

Numbers listed in parentheses indicate tornadoes reported without a Fujita scale intensity (“F” instead of “F0”).

F0 = 40 – 72 mph
F1 = 73 – 112 mph
F2 = 113 – 157 mph
F3 = 158 – 206 mph
F4 = 207 – 260 mph
F5 = 261 – 318 mph

a) Data for 2007 is through March 31, 2007.

Source: Reference 2.3-212

LNP COL 2.3-1

Table 2.3.1-206 (Sheet 1 of 3)
Reported Tropical Storm and Hurricane Occurrences in Florida
Period of Record: 1899 to 2007

Year	Tropical Storm	Cat 1	Cat 2	Cat 3	Cat 4	Cat 5	Total
1899	2	0	0	0	0	0	2
1900	1	0	0	0	0	0	1
1901	3	0	0	0	0	0	3
1902	1	0	0	0	0	0	1
1903	0	0	1	0	0	0	1
1904	1	0	0	0	0	0	1
1906	2	1	1	0	0	0	4
1907	2	0	0	0	0	0	2
1909	3	0	0	1	0	0	4
1910	0	0	0	1	0	0	1
1911	0	1	0	0	0	0	1
1914	1	0	0	0	0	0	1
1915	1	1	0	0	0	0	2
1916	1	1	1	0	0	0	3
1917	0	0	0	1	0	0	1
1919	1	0	0	0	1	0	2
1920	1	0	0	0	0	0	1
1921	0	0	0	1	0	0	1
1924	0	2	0	0	0	0	2
1925	0	1	0	0	0	0	1
1926	0	0	1	0	1	0	2
1928	1	0	1	0	1	0	3
1929	0	0	0	1	0	0	1
1930	1	0	0	0	0	0	1
1932	2	0	0	0	0	0	2
1933	1	1	0	1	0	0	3
1934	2	0	0	0	0	0	2
1935	0	0	1	0	0	1	2
1936	2	0	0	1	0	0	3

LNP COL 2.3-1

**Table 2.3.1-206 (Sheet 2 of 3)
Reported Tropical Storm and Hurricane Occurrences in Florida
Period of Record: 1899 to 2007**

Year	Tropical Storm	Cat 1	Cat 2	Cat 3	Cat 4	Cat 5	Total
1937	3	0	0	0	0	0	3
1938	1	0	0	0	0	0	1
1939	0	1	0	0	0	0	1
1940	1	0	0	0	0	0	1
1941	1	0	1	0	0	0	2
1944	0	0	0	1	0	0	1
1945	1	1	0	1	0	0	3
1946	1	1	0	0	0	0	2
1947	2	1	0	0	1	0	4
1948	1	0	1	1	0	0	3
1949	0	0	0	1	0	0	1
1950	2	0	0	2	0	0	4
1951	1	0	0	0	0	0	1
1952	1	0	0	0	0	0	1
1953	4	1	0	0	0	0	5
1956	0	1	0	0	0	0	1
1957	2	0	0	0	0	0	2
1959	2	0	0	0	0	0	2
1960	0	0	0	0	1	0	1
1964	0	0	3	0	0	0	3
1965	1	0	0	1	0	0	2
1966	0	1	1	0	0	0	2
1968	1	0	1	0	0	0	2
1969	1	0	0	0	0	0	1
1970	1	0	0	0	0	0	1
1972	0	1	0	0	0	0	1
1975	0	0	0	1	0	0	1
1976	1	0	0	0	0	0	1

LNP COL 2.3-1

Table 2.3.1-206 (Sheet 3 of 3)
Reported Tropical Storm and Hurricane Occurrences in Florida
Period of Record: 1899 to 2007

Year	Tropical Storm	Cat 1	Cat 2	Cat 3	Cat 4	Cat 5	Total
1979	0	0	1	0	0	0	1
1981	1	0	0	0	0	0	1
1983	1	0	0	0	0	0	1
1984	1	0	0	0	0	0	1
1985	3	0	1	0	0	0	4
1987	0	1	0	0	0	0	1
1988	1	0	0	0	0	0	1
1990	1	0	0	0	0	0	1
1992	0	0	0	0	0	1	1
1994	3	0	0	0	0	0	3
1995	2	0	1	1	0	0	4
1996	1	0	0	0	0	0	1
1997	1	0	0	0	0	0	1
1998	1	1	1	0	0	0	3
1999	1	1	0	0	0	0	2
2000	2	0	0	0	0	0	2
2001	2	0	0	0	0	0	2
2002	1	0	0	0	0	0	1
2003	0	0	0	0	0	0	0
2004	1	0	1	1	1	0	4
2005	2	0	1	2	0	0	5
2006	2	0	0	0	0	0	2
2007	1	0	0	0	0	0	1
Total	85	19	19	19	6	2	150

Notes:
Tropical Storm = 39 – 73 mph
Category 1 = 74 – 95 mph
Category 2 = 96 – 100 mph
Category 3 = 111 – 130 mph
Category 4 = 131 – 155 mph
Category 5 = greater than 155 mph

Sources: Reference 2.3-218, 2.3-232, 2.3-233, 2.3-234, 2.3-235, and 2.3-236

LNP COL 2.3-1

Table 2.3.1-207
Summary of Wet and Dry Bulb Temperature Observations

	Jacksonville		Tallahassee		Tampa	
	Wet Bulb (°C)	**Dry Bulb (°C)**	**Wet Bulb (°C)**	**Dry Bulb (°C)**	**Wet Bulb (°C)**	**Dry Bulb (°C)**
Highest Running Average Wet Bulb (with Coincident Dry Bulb)						
30-Day Average	24.9	28.1	24.8	28.3	25.5	28.6
5-Day Average	26.5	31.0	26.1	30.9	26.9	30.2
1-Day Average	27.7	31.2	27.0	32.1	27.6	31.0
Maximum Ambient Dry Bulb (with Coincident Wet Bulb)						
0% Exceedance	26.0	39.4	27.7	39.4	25.4	36.7
1% Exceedance	26.9	33.5	27.2	33.7	26.3	32.6
Minimum Ambient Dry Bulb (with Coincident Wet Bulb)						
100% Exceedance	-15.3	-13.9	-15.7	-14.4	-8.8	-7.2
99% Exceedance	-1.1	0.0	-3.3	-2.2	2.8	4.4
Maximum Ambient Wet Bulb (with Coincident Dry Bulb)						
0% Exceedance	30.3	33.9	30.4	31.7	29.5	34.4
1% Exceedance	26.1	31.1	26.1	31.1	26.7	31.1

Notes:
NA = Coincident data not available
Periods of Record: 1973 – 1996 and 1961 – 1990
°C = degrees Celsius

Sources: References 2.3-217 and 2.3-219

LNP COL 2.3-1

Table 2.3.1-208
Seasonal Frequencies of Inversions below 152 m (500 ft.) in Tampa, Florida

	Percent Frequency of Inversions Based below 152 m (500 ft.)				
Season	**0300 GMT**	**1500 GMT**	**0000 GMT**	**1200 GMT**	**All Times**
Winter	69	17	28	60	37
Spring	59	1	7	52	30
Summer	62	8	14	57	28
Fall	63	2	25	76	38

Notes:

GMT = Greenwich mean time
m = meter
ft. = foot

Source: Reference 2.3-220

LNP COL 2.3-1

Table 2.3.1-209
Mean Monthly Mixing Depths at Tampa, Florida

Month	Depth (m)
January	730
February	950
March	940
April	1310
May	1410
June	1360
July	1310
August	1290
September	1270
October	1290
November	1000
December	810

Notes:

m = meter

Source: Reference 2.3-221

LNP COL 2.3-1

Table 2.3.1-210 (Sheet 1 of 2)
Ambient Dry and Wet Bulb Temperature Observations for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida

Maximum and Minimum Dry Bulb Temperatures (with Coincident Wet Bulb Temperatures) (°F)

	Gainesville		Jacksonville		Orlando		Tallahassee		Tampa	
	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb
Maximum Temperatures										
0% Occurrence	(f)	(f)	103	79	(f)	(f)	103	82	98	78
0.4% Occurrence	94	77	95	78	93	77	95	77	93	78
1.0% Occurrence	92	77	93	78	92	76	93	76	91	78
2.0% Occurrence	91	76	91	76	91	76	92	76	90	77
“Maximum Safety” [(a)]	(e)	(e)	104.4	82.3	(e)	(e)	105.1	78.7	98.7	78.1
“Maximum Normal” [(b)]	94	77	95	78	93	77	95	77	93	78
Minimum Temperatures										
97.5% Occurrence	38	36	39	35	45	42	32	30	45	42
99.0% Occurrence	33	31	32	30	40	37	28	26	40	37
99.6% Occurrence	29	27	28	26	36	33	24	23	36	33
100% Occurrence	-	-	7	4	-	-	6	4	19	16
“Minimum Safety” [(c)]	4	NA	4	NA	9	NA	3	N/A	12	N/A
“Minimum Normal” [(d)]	29	27	28	26	36	33	24	23	36	33
Period of Record (yrs)	30		52		30		53		55	

LNP COL 2.3-1

Table 2.3.1-210 (Sheet 2 of 2)
Ambient Dry and Wet Bulb Temperature Observations for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida

Maximum Wet Bulb Temperatures (with Coincident Dry Bulb Temperatures) (°F)

	Gainesville		Jacksonville		Orlando		Tallahassee		Tampa	
	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb
0% Occurrence	(f)	(f)	87	93	(f)	(f)	87	89	85	94
0.4% Occurrence	80	88	80	90	80	88	80	89	80	88
1.0% Occurrence	79	87	79	88	79	87	79	88	80	88
2.0% Occurrence	78	86	78	87	78	86	78	87	79	87
“Maximum Safety” [(a)]	(e)	NA	84.7	NA	(e)	NA	84.2	NA	85.5	NA
“Maximum Normal” [(b)]	79	NA	79	NA	79	NA	79	NA	80	NA

Notes:
a) “Maximum Safety” temperatures are 100-yr estimates based on indicated POR and regression analyses.
b) “Maximum Normal” temperatures are based on the 0.4-percent annual occurrence temperatures from a 30-year POR.
c) “Minimum Safety” temperatures are 100-year estimates based on a 30-year POR.
d) “Minimum Normal” temperatures are based on the 99.6-percent annual occurrence temperatures from a 30-year POR.
e) “Maximum Safety” values not developed for these stations.
f) “0% Occurrence” values not available from published data.
°F = degrees Fahrenheit; NA = Not Applicable or Not Available

Sources: References 2.3-217, 2.3-219, and 2.3-226

LNP COL 2.3-2

Table 2.3.2-201
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category A

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5-1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1.1-1.5	0	1	0	2	1	0	1	1	0	1	0	3	2	4	1	0	17
1.6-2.0	4	2	2	4	4	0	3	2	3	1	1	5	2	5	6	0	44
2.1-3.0	8	22	16	15	18	9	3	4	2	5	20	21	30	12	11	17	213
3.1-4.0	8	11	30	34	28	11	3	8	7	6	43	106	98	11	13	19	436
4.1-5.0	3	9	11	35	42	13	0	1	3	18	38	77	53	4	6	14	327
5.1-6.0	0	0	7	18	19	1	0	0	3	6	11	19	32	2	0	0	118
6.1-8.0	0	0	0	4	6	0	0	0	0	1	1	1	10	2	0	0	25
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	3
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	23	45	66	112	118	34	10	16	18	38	114	232	230	40	37	50	1183

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 0

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-202
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category B

Speed (m/s)	Wind Direction (Blowing From) [(a)]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5-1.0	0	0	2	0	1	0	0	0	0	0	0	1	0	0	0	0	4
1.1-1.5	4	4	5	5	2	0	1	2	4	1	0	2	3	1	3	4	41
1.6-2.0	3	11	9	12	8	4	2	8	5	2	3	5	5	6	5	8	96
2.1-3.0	20	21	41	25	34	16	16	16	2	9	33	39	54	15	24	23	388
3.1-4.0	18	21	34	49	59	34	14	6	7	7	34	70	72	5	9	12	451
4.1-5.0	6	8	23	25	29	6	2	0	1	10	8	19	32	4	1	5	179
5.1-6.0	0	1	10	11	4	3	0	0	2	6	2	3	4	0	0	0	46
6.1-8.0	0	0	0	2	0	1	0	0	0	5	0	0	4	2	0	0	14
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	51	66	124	129	137	64	35	32	21	40	80	139	174	33	42	52	1219

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 0

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-203
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category C

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5-1.0	3	1	0	0	0	0	1	1	0	0	0	1	1	2	3	2	15
1.1-1.5	7	7	6	7	11	3	4	4	6	10	4	6	8	7	7	7	104
1.6-2.0	9	22	10	14	18	15	12	8	10	9	11	14	12	11	6	16	197
2.1-3.0	30	37	39	53	55	24	23	14	18	16	37	53	77	22	13	22	533
3.1-4.0	8	14	43	52	49	24	11	13	10	13	19	53	74	3	3	9	398
4.1-5.0	2	8	21	27	29	11	3	2	2	8	11	14	18	1	0	4	161
5.1-6.0	0	2	6	7	6	1	0	0	3	10	4	0	3	0	0	1	43
6.1-8.0	0	0	0	2	1	0	0	0	0	3	0	1	1	0	0	0	8
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	2
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	59	91	125	162	169	78	54	42	49	69	86	142	196	46	32	61	1461

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 1

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-204
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category D

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	1	0	0	3	1	0	1	2	1	3	0	1	1	2	1	17
0.5-1.0	23	10	18	16	22	14	13	12	15	17	15	17	10	9	12	8	231
1.1-1.5	40	36	39	26	24	22	31	25	30	20	37	37	27	26	34	27	481
1.6-2.0	50	54	80	60	73	31	31	21	17	28	42	52	61	35	34	35	704
2.1-3.0	102	112	197	196	142	94	51	32	48	59	73	147	198	44	32	54	1581
3.1-4.0	42	73	127	118	113	46	40	18	22	68	39	95	83	11	24	25	944
4.1-5.0	19	30	50	69	52	25	10	8	27	48	27	29	24	12	8	7	445
5.1-6.0	0	1	13	20	22	9	1	1	12	27	12	11	18	2	0	3	152
6.1-8.0	0	0	1	0	4	1	0	0	4	11	8	9	10	1	0	0	49
8.1-10.0	0	0	0	0	0	0	0	0	0	1	1	0	4	0	0	0	6
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	276	317	525	505	455	243	177	118	177	280	257	397	436	141	146	160	4610

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 49

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-205
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category E

Speed (m/s)	Wind Direction (Blowing From) (a)																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	6	5	5	4	3	2	4	4	4	3	3	2	3	3	5	2	58
0.5-1.0	21	60	72	72	62	42	56	53	28	25	34	37	35	52	39	17	705
1.1-1.5	34	82	133	147	133	83	53	35	38	26	47	49	39	32	21	27	979
1.6-2.0	40	51	127	134	126	58	46	14	38	19	19	31	32	15	28	30	808
2.1-3.0	61	82	101	123	131	62	42	17	30	34	9	22	26	12	35	30	817
3.1-4.0	8	15	11	17	23	10	3	1	17	6	3	12	7	2	4	8	147
4.1-5.0	1	0	3	5	1	3	1	2	6	1	0	3	1	5	1	2	35
5.1-6.0	0	0	0	0	0	0	0	0	1	0	0	2	0	0	0	0	3
6.1-8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.1-10.0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	171	295	452	502	479	260	205	126	162	115	115	158	143	121	133	116	3553

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 227

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-206
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category F

Speed (m/s)	Wind Direction (Blowing From) [(a)]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	5	8	6	18	8	7	8	7	8	8	3	1	4	7	7	1	106
0.5-1.0	21	34	74	109	100	40	32	28	16	16	22	14	25	29	20	19	599
1.1-1.5	29	26	39	119	103	43	12	10	8	7	11	9	6	5	13	18	458
1.6-2.0	15	10	5	31	44	14	3	2	2	0	1	2	2	1	2	10	144
2.1-3.0	1	2	0	0	7	1	0	0	1	2	3	1	3	0	1	0	22
3.1-4.0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	1	0	3
4.1-5.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.1-6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.1-8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	71	80	124	277	262	105	55	47	35	33	41	27	41	42	44	48	1332

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 643

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-207
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, Category G

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	5	1	16	36	34	6	7	7	6	0	3	1	3	5	6	2	138
0.5-1.0	19	15	32	107	97	49	22	14	11	9	2	4	5	8	27	16	437
1.1-1.5	8	2	7	42	32	10	2	3	2	1	1	1	3	2	3	8	127
1.6-2.0	3	0	0	5	3	2	0	0	0	1	0	0	0	0	1	1	16
2.1-3.0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	2
3.1-4.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4.1-5.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.1-6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.1-8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	35	18	55	190	166	69	31	24	19	11	6	6	11	15	37	27	720

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 2303

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-208
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	16	15	27	58	48	16	19	19	20	12	12	4	11	16	20	6	319
0.5-1.0	87	120	198	304	282	145	124	108	70	67	73	74	76	100	101	62	1991
1.1-1.5	122	158	229	348	306	161	104	80	88	66	100	107	88	77	82	91	2207
1.6-2.0	124	150	233	260	276	124	97	55	75	60	77	109	114	73	82	100	2009
2.1-3.0	222	276	394	412	387	208	135	83	101	125	175	283	388	105	116	146	3556
3.1-4.0	84	134	245	270	272	125	71	46	63	100	139	336	335	32	54	73	2379
4.1-5.0	31	55	108	161	153	58	16	13	39	85	84	142	128	26	16	32	1147
5.1-6.0	0	4	36	56	51	14	1	1	21	49	29	35	57	4	0	4	362
6.1-8.0	0	0	1	8	11	2	0	0	4	20	9	11	25	5	0	0	96
8.1-10.0	0	0	0	0	0	0	0	0	0	2	1	0	9	0	0	0	12
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	686	912	1471	1877	1786	853	567	405	481	586	699	1101	1231	438	471	514	14078

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 3223

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-209
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	2	0	2	6	0	2	1	0	0	0	1	1	2	1	5	2	25
0.5-1.0	8	6	8	21	14	9	8	6	6	0	10	6	6	19	15	8	150
1.1-1.5	12	14	10	22	24	12	1	4	7	4	12	6	6	16	7	7	164
1.6-2.0	11	7	8	12	29	8	9	2	2	4	11	10	10	7	10	9	149
2.1-3.0	23	13	18	16	32	17	13	10	5	16	18	16	35	16	13	16	277
3.1-4.0	4	5	15	3	12	9	6	7	9	27	21	34	27	5	13	5	202
4.1-5.0	0	1	5	2	6	6	1	2	7	14	6	8	16	6	5	2	87
5.1-6.0	0	0	0	0	0	0	0	0	5	12	6	5	17	0	0	0	45
6.1-8.0	0	0	0	0	0	0	0	0	0	5	2	0	3	0	0	0	10
8.1-10.0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	60	46	66	82	117	63	39	31	41	83	87	86	122	70	68	49	1110

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 256

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-210
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: March (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [(a)]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	2	1	2	6	7	0	0	0	0	0	1	0	2	0	1	2	24
0.5-1.0	10	3	20	32	26	12	7	2	3	6	2	3	6	11	8	7	158
1.1-1.5	5	10	16	31	44	13	6	0	3	2	2	3	5	6	5	6	157
1.6-2.0	9	10	13	43	33	16	11	5	2	4	0	4	14	9	4	3	180
2.1-3.0	22	10	19	41	57	24	21	5	16	10	4	14	26	8	7	11	295
3.1-4.0	3	7	27	42	40	16	11	8	2	8	3	16	37	4	3	6	233
4.1-5.0	8	5	11	19	28	17	6	3	8	8	7	8	11	1	0	5	145
5.1-6.0	0	0	4	1	9	3	0	1	9	13	3	0	9	0	0	1	53
6.1-8.0	0	0	0	0	3	0	0	0	1	12	0	0	5	1	0	0	22
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	6	0	0	0	6
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	59	46	112	215	247	101	62	24	44	63	22	48	121	40	28	41	1273

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 190

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-211
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: April (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	1	0	1	3	2	1	2	3	0	3	0	1	0	3	3	0	23
0.5-1.0	5	14	12	17	33	10	7	7	4	2	2	4	7	15	14	4	157
1.1-1.5	13	9	12	10	11	15	11	4	4	1	4	9	12	9	6	10	140
1.6-2.0	16	6	16	12	11	9	17	6	8	2	3	9	8	3	11	12	149
2.1-3.0	20	10	37	36	39	10	15	14	11	11	12	16	28	15	20	11	305
3.1-4.0	2	8	26	33	27	8	3	5	12	7	9	32	50	4	7	12	245
4.1-5.0	2	3	9	19	2	0	0	0	4	12	29	21	26	2	2	9	140
5.1-6.0	0	0	4	4	1	0	0	0	1	7	7	1	6	1	0	0	32
6.1-8.0	0	0	0	0	0	0	0	0	1	2	2	0	7	3	0	0	15
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	3
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	59	50	117	134	126	53	55	39	45	47	68	93	147	55	63	58	1209

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 175

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-212
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: May (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	2	3	4	2	2	1	2	0	1	2	0	3	4	2	0	28
0.5-1.0	1	5	6	30	14	7	2	3	3	7	5	8	5	10	9	3	118
1.1-1.5	7	7	17	44	23	16	7	1	6	5	14	7	7	9	7	4	181
1.6-2.0	2	6	9	22	28	9	12	2	5	2	7	10	12	4	3	4	137
2.1-3.0	8	16	22	30	52	20	9	4	4	7	9	36	34	4	7	8	270
3.1-4.0	1	4	14	32	57	24	3	2	1	1	14	57	43	3	2	7	265
4.1-5.0	0	0	7	27	45	10	0	0	1	4	8	23	25	0	0	0	150
5.1-6.0	0	0	4	13	19	1	0	0	0	0	9	7	13	0	0	0	66
6.1-8.0	0	0	0	5	6	0	0	0	0	0	0	8	2	0	0	0	21
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	19	40	82	207	246	89	34	14	20	27	68	156	144	34	30	26	1236

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 227

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-213
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: June (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	1	1	1	4	7	3	2	3	1	1	2	0	0	1	2	0	29
0.5-1.0	6	11	26	39	44	21	23	17	13	11	15	12	3	5	1	3	250
1.1-1.5	17	2	11	24	24	18	15	14	9	6	11	16	18	6	2	9	202
1.6-2.0	5	4	12	23	17	24	9	11	12	7	14	14	8	6	3	6	175
2.1-3.0	7	4	25	27	25	17	5	4	11	2	18	36	56	13	5	6	261
3.1-4.0	2	4	9	11	12	4	3	1	3	2	14	55	50	4	0	1	175
4.1-5.0	0	0	3	4	0	0	0	0	0	0	2	23	16	1	0	0	49
5.1-6.0	0	0	0	0	0	0	0	0	0	0	0	13	1	0	0	0	14
6.1-8.0	0	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0	2
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	38	26	87	132	129	88	57	50	49	29	76	170	152	36	13	25	1157

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 281

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-214
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: July (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	1	2	2	6	4	3	4	4	7	3	1	1	0	0	0	0	38
0.5-1.0	5	7	11	16	26	24	32	30	9	16	12	12	13	3	10	5	231
1.1-1.5	2	8	6	8	22	17	15	21	19	12	23	21	8	5	14	6	207
1.6-2.0	9	4	4	13	10	9	7	9	12	13	14	15	11	14	6	7	157
2.1-3.0	6	7	8	13	18	15	7	6	4	9	27	53	53	7	5	7	245
3.1-4.0	0	3	5	2	6	4	3	2	0	3	17	58	36	1	1	2	143
4.1-5.0	0	2	5	0	3	0	0	0	0	0	4	35	5	1	0	0	55
5.1-6.0	0	0	2	0	0	0	0	0	0	0	0	1	0	0	0	0	3
6.1-8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	23	33	43	58	89	72	68	72	51	56	98	196	126	31	36	27	1079

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 381

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-215
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: August (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [(a)]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	0	2	2	4	3	2	4	6	2	2	0	1	0	0	0	28
0.5-1.0	8	10	8	27	29	21	18	17	12	12	10	8	9	5	4	4	202
1.1-1.5	4	9	11	12	35	18	7	10	9	11	11	16	6	4	10	6	179
1.6-2.0	3	3	5	13	25	15	9	6	4	4	7	15	14	7	3	5	138
2.1-3.0	1	9	10	25	27	31	16	10	7	6	19	38	53	6	2	18	278
3.1-4.0	9	4	3	18	25	11	11	1	1	3	10	47	27	1	9	7	187
4.1-5.0	2	1	0	7	4	0	5	2	0	3	3	12	15	5	5	0	64
5.1-6.0	0	0	0	1	4	0	0	0	2	0	3	2	3	0	0	0	15
6.1-8.0	0	0	0	0	2	0	0	0	0	0	4	1	4	0	0	0	11
8.1-10.0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	2
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	27	36	39	105	155	99	68	50	41	42	70	139	132	28	33	40	1104

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 380

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-216
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: September (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	5	5	7	8	3	2	1	1	1	0	1	0	0	0	2	0	36
0.5-1.0	8	26	36	18	18	13	6	5	2	3	1	3	2	5	4	3	153
1.1-1.5	5	22	33	40	26	14	15	6	8	5	2	7	9	5	0	2	199
1.6-2.0	10	22	29	21	19	2	5	3	3	4	1	3	10	5	3	6	146
2.1-3.0	23	51	47	65	31	15	7	4	8	3	7	15	30	9	4	8	327
3.1-4.0	4	9	35	39	47	14	6	1	1	5	3	10	19	0	3	5	201
4.1-5.0	5	1	27	30	26	11	1	0	0	1	0	0	0	0	0	0	102
5.1-6.0	0	0	9	10	9	0	0	0	2	0	0	0	0	0	0	0	30
6.1-8.0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	60	136	224	231	179	71	41	20	25	21	15	38	70	24	16	24	1195

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 239

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-217
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: October (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	2	2	2	6	3	0	2	0	0	1	0	0	0	4	3	0	25
0.5-1.0	11	15	30	20	19	6	2	8	2	1	1	5	1	5	10	3	139
1.1-1.5	20	26	43	55	17	7	6	4	7	4	0	4	1	3	7	5	209
1.6-2.0	11	20	48	45	24	5	2	2	6	2	1	4	5	7	10	12	204
2.1-3.0	11	45	92	58	36	18	7	6	3	6	2	4	17	8	13	12	338
3.1-4.0	4	34	51	45	19	8	3	1	2	3	10	4	11	0	3	4	202
4.1-5.0	0	24	28	33	28	4	0	0	0	0	2	1	4	0	1	1	126
5.1-6.0	0	1	6	15	8	1	0	0	0	0	0	0	1	0	0	0	32
6.1-8.0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	59	167	300	278	154	49	22	21	20	17	16	22	40	27	47	37	1276

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 193

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-218
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: November (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	4	1	0	0	0	2	0	0	1	2	1	2	1	14
0.5-1.0	11	8	13	28	13	4	3	4	3	3	4	2	7	7	15	8	133
1.1-1.5	17	19	30	26	11	6	7	3	2	2	4	6	5	5	9	16	168
1.6-2.0	24	24	55	21	7	2	3	3	4	7	3	8	5	5	9	13	193
2.1-3.0	48	54	51	38	11	5	9	3	10	15	11	14	24	6	11	19	329
3.1-4.0	10	28	27	21	6	3	2	1	13	12	7	7	14	2	2	6	161
4.1-5.0	7	5	6	12	3	0	0	0	6	4	3	4	0	1	1	8	60
5.1-6.0	0	2	2	4	0	0	0	0	1	3	0	1	1	0	0	0	14
6.1-8.0	0	0	0	0	0	0	0	0	2	0	1	0	0	0	0	0	3
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	117	140	184	154	52	20	24	14	43	46	33	43	58	27	49	71	1075

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 335

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-219
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: December (2007 and 2008 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	2	3	1	7	0	1	2	0	0	1	0	0	1	0	0	18
0.5-1.0	8	5	16	40	27	10	8	7	5	4	7	5	5	7	7	4	165
1.1-1.5	12	16	22	55	58	19	4	3	5	9	10	9	4	6	4	12	248
1.6-2.0	7	23	23	23	48	17	11	3	3	4	8	7	10	3	8	8	206
2.1-3.0	23	20	24	28	39	23	20	7	7	11	23	18	15	2	8	17	285
3.1-4.0	13	10	10	14	15	17	14	13	8	7	14	9	10	6	9	5	174
4.1-5.0	0	7	1	1	1	4	3	4	2	8	7	0	8	3	0	0	49
5.1-6.0	0	0	0	0	0	3	0	0	0	1	1	3	4	1	0	0	13
6.1-8.0	0	0	0	0	0	1	0	0	0	0	0	1	1	0	0	0	3
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	63	83	99	162	195	94	61	39	30	44	71	52	57	29	36	46	1161

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 305

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-220
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: January (2008 and 2009 Combined Hours of Occurrence)
Lower Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	2	0	2	8	8	0	3	0	3	1	1	0	1	1	0	1	31
0.5-1.0	6	10	12	16	19	8	8	2	8	2	4	6	12	8	4	10	135
1.1-1.5	8	16	18	21	11	6	10	10	9	5	7	3	7	3	11	8	153
1.6-2.0	17	21	11	12	25	8	2	3	14	7	8	10	7	3	12	15	175
2.1-3.0	30	37	41	35	20	13	6	10	15	29	25	23	17	11	21	13	346
3.1-4.0	32	18	23	10	6	7	6	4	11	22	17	7	11	2	2	13	191
4.1-5.0	7	6	6	7	7	6	0	2	11	31	13	7	2	6	2	7	120
5.1-6.0	0	1	5	8	1	6	1	0	1	13	0	2	2	2	0	3	45
6.1-8.0	0	0	0	2	0	0	0	0	0	1	0	0	3	1	0	0	7
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	102	109	118	119	97	54	36	31	72	111	75	58	62	37	52	70	1203

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 261

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-221
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category A

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5-1.0	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	2
1.1-1.5	1	0	1	0	1	0	1	1	1	1	0	2	2	2	1	1	15
1.6-2.0	1	1	4	2	1	0	2	1	1	0	0	2	0	2	1	0	18
2.1-3.0	5	6	5	5	12	3	2	3	1	2	4	13	6	8	13	6	94
3.1-4.0	8	17	17	19	16	7	2	0	2	2	18	24	23	6	5	4	170
4.1-5.0	3	12	22	17	23	7	2	5	3	4	35	55	70	8	7	15	288
5.1-6.0	5	4	10	21	26	7	1	3	6	7	16	60	65	4	9	9	253
6.1-8.0	3	5	14	35	32	9	0	2	3	14	45	53	46	4	7	10	282
8.1-10.0	0	0	0	5	8	0	0	1	1	6	7	2	16	4	1	0	51
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	6	3	0	0	9
TOTAL	26	45	73	104	120	33	10	16	18	37	125	211	234	41	44	45	1182

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 0

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-222
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category B

Speed (m/s)	Wind Direction (Blowing From) [(a)]															
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
0.5-1.0	3	1	0	2	2	0	0	0	1	1	0	0	0	1	0	0	11
1.1-1.5	1	2	4	1	7	1	1	1	5	1	0	1	3	0	1	3	32
1.6-2.0	4	4	4	6	3	4	2	4	3	1	4	4	1	3	4	6	57
2.1-3.0	13	11	25	15	16	11	11	12	2	1	8	12	14	7	11	10	179
3.1-4.0	14	11	26	30	35	18	6	7	2	8	29	32	40	8	17	10	293
4.1-5.0	17	10	26	30	45	13	11	4	3	4	21	37	58	4	10	8	301
5.1-6.0	4	10	14	21	19	10	4	3	3	8	8	27	29	2	2	5	169
6.1-8.0	4	3	23	18	17	4	1	1	2	7	13	14	24	2	2	3	138
8.1-10.0	0	0	3	4	0	1	0	0	1	7	1	1	2	0	0	0	20
>10.0	0	0	0	0	0	0	0	0	0	3	0	0	4	2	0	0	9
TOTAL	60	53	125	127	144	62	36	32	22	41	84	128	175	29	47	45	1210

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 0

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-223
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category C

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	0	0	0	0	0	0	1	0	1	0	1	0	0	0	0	3
0.5-1.0	1	2	1	1	4	2	2	1	3	4	4	1	1	5	3	2	37
1.1-1.5	4	5	6	4	8	3	5	1	3	5	2	3	5	1	5	5	65
1.6-2.0	7	10	9	14	12	5	7	3	3	7	6	6	7	4	3	4	107
2.1-3.0	13	25	15	26	30	20	12	8	13	11	11	22	18	17	9	21	271
3.1-4.0	18	17	33	36	40	11	14	9	7	13	25	32	51	12	5	13	336
4.1-5.0	4	10	16	34	32	15	12	11	7	9	17	36	58	4	4	3	272
5.1-6.0	3	8	21	22	22	10	2	6	3	6	11	15	32	3	2	6	172
6.1-8.0	2	5	25	23	16	5	2	3	2	9	11	11	15	5	1	4	139
8.1-10.0	0	1	1	2	2	0	0	0	1	11	5	1	1	0	0	1	26
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	3
TOTAL	52	83	127	162	166	71	56	43	42	76	92	128	191	51	32	59	1431

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 6

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-224
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category D

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	1	2	0	0	0	2	0	0	2	1	0	1	0	0	0	1	10
0.5-1.0	6	11	8	7	5	9	6	4	11	11	13	7	5	3	9	5	120
1.1-1.5	11	10	8	10	18	12	11	10	16	11	11	8	8	7	8	5	164
1.6-2.0	15	15	15	13	17	11	14	14	11	14	11	13	17	9	15	9	213
2.1-3.0	40	48	63	68	67	34	28	26	24	17	45	53	52	29	27	38	659
3.1-4.0	60	67	102	98	74	55	39	33	20	20	50	83	121	43	24	25	914
4.1-5.0	40	58	128	108	74	39	33	18	19	33	37	78	121	17	19	17	839
5.1-6.0	38	49	96	98	91	26	29	9	24	49	39	47	60	17	16	24	712
6.1-8.0	22	66	88	87	78	26	18	27	32	78	37	48	28	18	20	25	698
8.1-10.0	1	6	13	12	15	5	5	8	10	39	24	14	16	9	7	2	186
>10.0	0	0	1	0	0	0	0	0	3	8	9	8	16	1	0	0	46
TOTAL	234	332	522	501	439	219	183	149	172	281	276	360	444	153	145	151	4561

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 53

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-225
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category E

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2
0.5-1.0	3	3	3	3	2	0	2	4	5	2	5	0	4	0	1	3	40
1.1-1.5	4	2	2	5	5	4	4	1	7	5	2	5	6	2	1	1	56
1.6-2.0	8	3	6	5	2	6	10	3	10	6	2	7	8	4	2	2	84
2.1-3.0	16	23	23	33	34	33	32	24	23	21	31	26	38	21	17	15	410
3.1-4.0	30	57	81	98	79	62	53	66	48	23	33	47	64	39	20	21	821
4.1-5.0	40	76	173	222	147	97	93	65	62	31	32	35	55	39	26	32	1225
5.1-6.0	38	86	136	155	125	59	47	29	33	18	5	15	18	25	33	36	858
6.1-8.0	10	32	21	18	11	10	10	12	25	6	4	15	9	9	19	18	229
8.1-10.0	0	0	0	1	2	0	2	0	1	0	0	4	1	4	2	0	17
>10.0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
TOTAL	149	283	445	540	407	271	253	204	214	113	115	154	203	143	121	128	3743

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 20

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-226
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category F

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1
0.5-1.0	2	0	1	1	5	3	1	2	3	2	3	2	2	4	0	0	31
1.1-1.5	2	0	0	2	2	4	5	6	4	1	0	2	4	3	3	0	38
1.6-2.0	3	2	3	4	2	7	5	8	2	4	7	5	5	4	3	4	68
2.1-3.0	4	11	13	29	16	18	15	25	18	18	24	11	26	14	11	7	260
3.1-4.0	15	9	28	41	30	35	55	28	30	15	15	24	33	33	12	14	417
4.1-5.0	13	21	61	85	62	65	51	29	22	13	13	34	26	30	27	9	561
5.1-6.0	22	29	51	106	74	44	33	5	9	2	5	9	13	23	20	22	467
6.1-8.0	19	12	4	12	20	3	0	5	3	0	1	1	0	4	4	15	103
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	80	84	161	280	211	179	165	108	92	55	68	88	109	115	80	71	1946

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 18

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-227
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, Category G

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	1	1	0	0	0	0	0	0	0	2	0	0	1	0	5
0.5-1.0	3	1	2	2	4	3	7	5	7	4	4	7	3	5	6	8	71
1.1-1.5	6	2	7	4	7	5	5	8	5	9	10	7	8	8	6	8	105
1.6-2.0	10	4	11	10	6	8	15	10	14	10	22	11	16	14	10	7	178
2.1-3.0	28	14	23	39	36	47	40	35	32	33	30	36	37	29	35	22	516
3.1-4.0	26	22	35	55	53	32	53	56	40	27	46	47	42	41	47	36	658
4.1-5.0	42	31	46	72	60	62	37	37	23	15	18	37	48	43	50	44	665
5.1-6.0	50	29	33	64	59	44	15	13	9	9	3	1	18	25	37	48	457
6.1-8.0	59	15	18	36	22	19	3	0	11	5	0	0	5	2	22	60	277
8.1-10.0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	224	118	176	283	247	220	176	164	141	112	133	148	177	167	214	233	2933

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 77

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-228
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February 1, 2007, to January 31, 2009
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) (a)																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	1	4	1	1	0	2	0	1	3	2	1	4	0	0	1	1	22
0.5-1.0	18	18	15	16	23	17	18	16	30	25	29	17	15	18	19	18	312
1.1-1.5	29	21	28	26	48	29	32	28	41	33	25	28	36	23	25	23	475
1.6-2.0	48	39	52	54	43	41	55	43	44	42	52	48	54	40	38	32	725
2.1-3.0	119	138	167	215	211	166	140	133	113	103	153	173	191	125	123	119	2389
3.1-4.0	171	200	322	377	327	220	222	199	149	108	216	289	374	182	130	123	3609
4.1-5.0	159	218	472	568	443	298	239	169	139	109	173	312	436	145	143	128	4151
5.1-6.0	160	215	361	487	416	200	131	68	87	99	87	174	235	99	119	150	3088
6.1-8.0	119	138	193	229	196	76	34	50	78	119	111	142	127	44	75	135	1866
8.1-10.0	1	7	17	24	27	6	8	9	14	63	37	22	36	17	10	3	301
>10.0	0	0	1	0	0	0	0	0	3	12	9	8	29	6	0	0	68
TOTAL	825	998	1629	1997	1734	1055	879	716	701	715	893	1217	1533	699	683	732	17006

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 174

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-229
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: February (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	2
0.5-1.0	0	2	1	2	1	3	0	1	1	0	2	0	0	1	0	0	14
1.1-1.5	1	0	3	2	1	2	1	0	2	1	2	6	4	0	2	0	27
1.6-2.0	2	2	3	7	3	4	0	1	1	2	4	2	2	3	2	2	40
2.1-3.0	10	10	8	12	8	6	6	5	3	4	10	7	13	15	17	10	144
3.1-4.0	22	15	13	23	14	5	4	6	6	11	26	23	32	23	15	23	261
4.1-5.0	21	27	24	13	32	24	22	13	5	9	15	33	39	21	19	15	332
5.1-6.0	11	18	11	17	32	28	18	10	8	20	9	11	20	15	27	17	272
6.1-8.0	8	2	4	8	20	9	2	12	15	24	10	11	15	12	17	15	184
8.1-10.0	0	0	0	0	0	0	0	1	3	15	6	5	14	2	1	0	47
>10.0	0	0	0	0	0	0	0	0	0	4	0	0	1	0	0	0	5
TOTAL	75	76	67	84	111	81	53	49	44	90	84	100	140	92	100	82	1328

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 9

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-230
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: March (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	2
0.5-1.0	0	1	1	2	0	1	0	1	1	0	3	1	1	0	4	1	17
1.1-1.5	2	1	2	1	4	1	4	0	3	1	0	1	2	1	2	2	27
1.6-2.0	1	3	3	5	3	6	5	1	3	2	0	0	0	1	3	3	39
2.1-3.0	10	11	19	16	24	15	7	4	3	3	4	8	12	5	5	13	159
3.1-4.0	14	10	19	27	39	24	13	7	3	6	4	11	29	18	8	10	242
4.1-5.0	14	17	41	76	70	29	31	10	5	4	3	9	29	17	11	11	377
5.1-6.0	23	13	26	57	93	30	27	1	14	4	2	6	22	10	7	13	348
6.1-8.0	10	1	14	21	34	15	5	11	9	15	6	4	8	5	4	8	170
8.1-10.0	0	0	1	0	3	1	3	2	8	21	6	2	8	0	0	1	56
>10.0	0	0	0	0	0	0	0	0	1	4	0	0	11	1	0	0	17
TOTAL	74	57	126	205	270	123	95	37	50	60	28	42	122	58	44	63	1454

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 9

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-231
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: April (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
0.5-1.0	1	0	1	1	1	1	1	0	1	0	0	1	0	1	0	0	9
1.1-1.5	0	0	1	0	2	1	0	2	3	0	1	0	1	1	0	1	13
1.6-2.0	6	3	3	1	1	2	3	2	1	1	3	1	1	1	2	3	34
2.1-3.0	8	10	11	20	15	9	15	7	11	3	7	9	10	10	6	3	154
3.1-4.0	20	7	43	30	30	11	16	12	6	9	11	14	17	19	17	14	276
4.1-5.0	18	11	34	28	27	19	26	31	16	8	13	20	62	23	18	19	373
5.1-6.0	14	2	16	32	32	18	21	15	10	10	7	21	43	20	14	16	291
6.1-8.0	11	9	19	21	5	0	2	2	5	15	32	7	29	9	13	14	193
8.1-10.0	0	0	2	0	0	0	0	0	0	5	6	0	4	4	1	0	22
>10.0	0	0	0	0	0	0	0	0	1	2	1	0	7	4	0	0	15
TOTAL	78	44	130	133	113	61	84	71	54	53	81	73	174	92	71	70	1382

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 2

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-232
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: May (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1
0.5-1.0	1	0	1	0	3	1	1	0	0	3	1	1	0	0	2	1	15
1.1-1.5	0	1	1	2	4	0	2	3	3	2	2	2	5	0	0	1	28
1.6-2.0	1	2	2	2	3	3	3	3	4	2	1	1	3	3	1	1	35
2.1-3.0	3	10	7	9	20	8	6	10	3	2	6	8	19	11	10	7	139
3.1-4.0	7	16	22	38	33	20	9	6	4	2	21	26	38	12	5	5	264
4.1-5.0	5	11	25	56	71	31	19	10	7	5	17	34	41	12	13	11	368
5.1-6.0	2	2	19	59	91	25	17	7	4	3	20	37	33	13	5	15	352
6.1-8.0	4	2	15	39	50	12	0	0	0	4	19	41	24	3	2	3	218
8.1-10.0	0	0	0	9	11	0	0	0	0	0	7	6	2	0	0	0	35
>10.0	0	0	0	0	0	0	0	0	0	0	0	5	0	0	0	0	5
TOTAL	23	44	93	214	286	100	57	39	25	23	94	161	165	54	38	44	1460

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 3

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-233
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: June (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	1	0	1	0	1	0	0	0	0	0	0	0	0	0	0	3
0.5-1.0	4	4	2	0	1	1	3	2	4	3	5	3	5	2	2	0	41
1.1-1.5	5	0	1	6	4	6	3	2	5	10	3	8	7	1	3	2	66
1.6-2.0	5	2	9	6	5	9	14	10	6	12	5	9	7	8	4	4	115
2.1-3.0	12	8	24	16	29	21	16	23	24	22	19	29	22	9	7	5	286
3.1-4.0	14	6	24	27	34	20	35	29	35	14	20	41	45	11	5	9	369
4.1-5.0	2	4	23	50	45	27	32	20	16	6	19	42	56	9	2	2	355
5.1-6.0	1	2	4	13	8	9	6	1	5	1	5	25	33	8	0	0	121
6.1-8.0	0	3	3	2	1	0	1	2	0	1	8	34	15	3	0	0	73
8.1-10.0	0	0	0	1	1	1	0	0	0	0	0	3	0	0	0	0	6
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	43	30	90	122	128	95	110	89	95	69	84	194	190	51	23	22	1435

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 5

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-234
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: July (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	2
0.5-1.0	4	3	2	4	3	2	4	2	9	3	3	6	2	6	3	5	61
1.1-1.5	8	5	4	1	4	4	6	6	7	10	7	2	9	6	6	2	87
1.6-2.0	9	5	5	8	3	5	11	5	10	8	18	13	7	11	10	6	134
2.1-3.0	14	10	6	10	14	19	21	31	19	18	37	38	35	17	22	19	330
3.1-4.0	4	5	6	18	25	19	31	60	36	10	42	59	55	17	11	4	402
4.1-5.0	3	0	7	13	11	10	9	21	30	5	25	71	55	8	9	3	280
5.1-6.0	0	2	3	2	5	0	5	1	3	1	7	34	25	2	3	1	94
6.1-8.0	1	1	11	0	2	1	0	1	0	0	4	16	5	1	1	0	44
8.1-10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	43	31	44	56	67	60	87	127	114	56	143	239	193	68	66	40	1434

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 30

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-235
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: August (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	0	2
0.5-1.0	2	2	1	1	4	1	4	1	5	1	2	0	1	3	1	1	30
1.1-1.5	2	3	3	2	5	4	4	6	7	3	2	4	1	5	3	3	57
1.6-2.0	3	0	9	7	5	4	4	9	5	5	6	8	5	4	7	1	82
2.1-3.0	6	4	10	32	27	28	19	12	23	17	28	25	23	19	7	13	293
3.1-4.0	4	6	10	27	42	29	40	17	15	13	23	63	63	22	10	4	388
4.1-5.0	1	3	7	19	27	45	22	16	12	7	20	53	43	3	5	3	286
5.1-6.0	6	1	4	21	14	15	8	6	4	2	6	28	12	2	1	8	138
6.1-8.0	3	8	2	4	11	0	9	5	1	1	3	9	10	1	5	7	79
8.1-10.0	0	1	0	1	4	0	1	2	2	1	7	3	1	4	8	0	35
>10.0	0	0	0	0	0	0	0	0	0	1	6	1	6	0	0	0	14
TOTAL	27	28	46	114	139	126	111	74	74	52	103	195	165	63	47	40	1404

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 31

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-236
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: September (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1
0.5-1.0	1	1	1	1	5	1	0	2	4	4	1	0	0	2	0	6	29
1.1-1.5	3	2	3	4	7	2	4	2	3	2	0	1	2	2	2	2	41
1.6-2.0	5	4	4	7	1	4	2	3	5	3	3	3	7	3	2	3	59
2.1-3.0	5	19	18	21	17	10	18	13	13	7	5	10	15	7	11	7	196
3.1-4.0	10	41	46	41	18	21	21	14	9	6	7	15	31	13	7	5	305
4.1-5.0	10	41	84	88	41	31	12	7	2	3	3	9	32	10	8	4	385
5.1-6.0	4	33	48	71	49	15	3	1	1	4	1	2	6	1	4	5	248
6.1-8.0	2	14	38	31	31	10	4	1	2	1	0	1	0	0	0	3	138
8.1-10.0	1	0	8	3	2	0	0	0	0	0	0	0	0	0	0	0	14
>10.0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1
TOTAL	41	155	251	267	171	94	64	43	40	30	20	41	93	38	34	35	1417

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 17

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-237
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: October (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0	0	2
0.5-1.0	2	0	1	2	0	0	1	1	0	1	3	0	2	0	2	0	15
1.1-1.5	1	1	2	0	1	2	2	1	3	1	0	0	0	2	2	1	19
1.6-2.0	3	5	4	1	6	1	5	1	2	1	0	6	7	1	2	3	48
2.1-3.0	15	12	8	24	16	18	4	5	2	3	0	2	3	7	15	10	144
3.1-4.0	7	19	46	43	39	22	14	17	12	7	2	2	14	11	14	7	276
4.1-5.0	3	25	88	98	39	5	7	4	13	6	9	0	15	12	12	8	344
5.1-6.0	24	44	98	113	35	2	3	2	4	3	6	0	8	3	8	20	373
6.1-8.0	19	44	42	53	23	4	3	0	0	1	3	1	4	0	1	14	212
8.1-10.0	0	5	5	6	6	1	0	0	0	0	0	0	1	0	0	0	24
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	74	155	294	340	165	55	39	31	37	23	23	12	54	36	56	63	1457

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 12

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-238
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: November (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [(a)]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5-1.0	2	1	0	1	3	0	1	0	1	1	0	1	0	0	1	2	14
1.1-1.5	2	2	4	2	5	0	1	2	2	1	2	0	2	2	3	4	34
1.6-2.0	8	8	4	4	2	0	1	0	3	2	0	2	5	3	2	5	49
2.1-3.0	20	19	21	17	7	4	5	2	3	2	7	10	9	7	8	23	164
3.1-4.0	19	29	33	40	8	9	10	7	2	9	8	8	20	10	11	23	246
4.1-5.0	33	41	64	50	16	8	16	10	7	14	6	15	26	14	16	23	359
5.1-6.0	34	42	75	48	5	7	0	3	10	17	11	4	12	10	20	21	319
6.1-8.0	24	26	15	29	3	1	0	1	24	14	2	4	3	1	16	34	197
8.1-10.0	0	1	1	0	0	0	0	0	1	3	0	0	0	0	0	0	6
>10.0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0	3
TOTAL	142	169	217	191	49	29	34	25	54	64	37	44	77	47	77	135	1391

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 19

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-239
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: December (2007 and 2008 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
<0.5	0	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	2
0.5-1.0	1	2	1	2	1	5	2	4	2	5	6	3	2	0	2	2	40
1.1-1.5	1	3	3	4	8	4	1	0	3	1	4	0	2	2	1	3	40
1.6-2.0	1	2	5	5	9	2	4	2	0	2	6	3	8	1	1	0	51
2.1-3.0	9	10	17	22	27	16	12	7	3	9	14	15	14	9	8	6	198
3.1-4.0	23	22	39	46	31	25	18	13	8	9	20	11	13	12	15	10	315
4.1-5.0	22	23	33	59	35	50	32	14	10	13	21	9	17	8	11	14	371
5.1-6.0	13	11	20	31	40	35	22	15	9	13	4	4	13	7	11	14	262
6.1-8.0	6	8	5	8	5	6	6	8	3	7	7	2	10	4	4	6	95
8.1-10.0	0	0	0	0	0	1	0	3	0	2	4	1	2	1	0	0	14
>10.0	0	0	0	0	0	0	0	0	0	0	1	2	1	0	0	0	4
TOTAL	76	82	123	177	156	144	97	67	38	61	87	50	82	44	53	55	1392

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 24

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-240
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence)
Period of Record: January (2008 and 2009 Combined Hours of Occurrence)
Upper Wind Level, All Categories

Speed (m/s)	Wind Direction (Blowing From) [a]																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
<0.5	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	3
0.5-1.0	0	2	3	0	1	1	1	2	2	4	3	1	2	3	2	0	27
1.1-1.5	4	3	1	2	3	3	4	4	0	1	2	4	1	1	1	2	36
1.6-2.0	4	3	1	1	2	1	3	6	4	2	6	0	2	1	2	1	39
2.1-3.0	7	15	18	16	7	12	11	14	6	13	16	12	16	9	7	3	182
3.1-4.0	27	24	21	17	14	15	11	11	13	12	32	16	17	14	12	9	265
4.1-5.0	27	15	42	18	29	19	11	13	16	29	22	17	21	8	19	15	321
5.1-6.0	28	45	37	23	12	16	1	6	15	21	9	2	8	8	19	20	270
6.1-8.0	31	20	25	13	11	18	2	7	19	36	17	12	4	5	12	31	263
8.1-10.0	0	0	0	4	0	2	4	1	0	16	1	2	4	6	0	2	42
>10.0	0	0	0	0	0	0	0	0	0	0	0	0	3	1	0	0	4
TOTAL	129	127	148	94	79	87	48	64	76	134	109	66	78	56	74	83	1452

Notes:

Data represent the number of hours a condition occurred.

Number of Calm Hours: 13

a) Wind direction: E = east; N = north; S = south; W = west

m/s = meters per second

LNP COL 2.3-2

Table 2.3.2-241
Summary of Mean Daily Maximum and Minimum Ambient Air Temperatures (°F)

	LNP On-Site[(a)]		Tampa		Gainesville		Orlando		Tallahassee		Jacksonville	
Month	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**
January	66.6	46.0	70.5	50.9	66.5	42.8	70.4	48.9	63.9	39.8	65.1	42.6
February	67.0	46.5	70.8	51.5	69.7	45.9	72.9	51.4	67.1	42.3	67.9	45.3
March	76.9	55.1	75.5	56.4	75.0	50.4	77.4	55.8	73.2	47.7	73.7	50.4
April	77.0	55.8	81.1	61.5	80.0	54.6	82.5	60.4	80.0	53.4	79.8	56.2
May	85.9	62.4	87.2	68.0	84.6	60.7	87.5	66.4	86.6	61.8	86.0	63.7
June	88.3	70.0	89.0	72.4	89.1	68.2	89.9	71.5	90.4	68.9	89.7	70.0
July	88.4	72.0	90.1	74.5	90.9	71.4	91.1	73.3	91.4	71.7	91.9	72.8
August	89.6	72.8	90.3	74.6	90.1	71.4	90.9	73.7	91.0	71.7	90.9	72.7
September	86.9	70.6	88.4	72.7	87.1	69.0	88.9	72.5	88.0	68.3	87.1	70.1
October	82.5	67.6	84.0	66.3	81.2	60.5	83.6	65.9	80.9	57.0	80.1	60.6
November	73.6	51.3	76.9	57.7	74.6	51.8	77.7	57.9	72.3	47.0	73.0	50.9
December	72.9	52.0	72.0	52.5	67.8	44.3	72.1	51.6	65.5	41.2	66.5	44.3
Annual	79.7	60.3	81.3	63.3	79.7	57.6	82.1	62.4	79.2	55.9	79.3	58.3
Period of Record (years)	1		74		25		50		59		59	

Notes:
a) LNP on-site data are for the period from February 1, 2007, to January 31, 2008.
°F = degrees Fahrenheit

Sources: LNP on-site data; References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-2

Table 2.3.2-242
Summary of Mean Dew-Point Temperatures (°F)

Month	LNP On-Site[(a)]	Tampa	Gainesville	Orlando	Tallahassee	Jacksonville
January	47.4	52.3	46.2	51.5	42.1	44.9
February	46.0	54.0	48.4	53.0	44.7	47.4
March	53.0	57.0	52.2	55.8	48.9	51.4
April	53.9	60.2	56.3	58.9	53.9	55.8
May	60.2	66.4	63.3	65.4	62.4	63.4
June	69.4	72.0	70.2	71.4	69.7	70.5
July	72.2	73.7	72.6	73.1	72.6	72.9
August	72.7	74.1	72.7	73.5	72.4	73.1
September	70.5	72.5	70.3	72.2	68.3	70.7
October	67.1	66.0	62.7	66.1	58.7	62.8
November	52.9	60.0	55.5	60.0	51.5	55.1
December	55.0	54.2	48.1	53.8	44.2	47.4
Annual	61.4	63.5	59.9	62.9	57.5	59.6
Period of Record (years)	1	23	23	23	23	23

Notes:

a) LNP on-site data is for the period from February 1, 2007, to January 31, 2008.

°F = degrees Fahrenheit

Sources: LNP on-site data; References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-2

Table 2.3.2-243
Summary of Diurnal Relative Humidity (%)

	Tampa				Gainesville				Orlando				Tallahassee				Jacksonville			
Month	**01:00**	**07:00**	**13:00**	**19:00**	**01:00**	**07:00**	**13:00**	**19:00**	**01:00**	**07:00**	**13:00**	**19:00**	**01:00**	**07:00**	**13:00**	**19:00**	**01:00**	**07:00**	**13:00**	**19:00**
January	85	87	60	74	89	90	61	76	86	89	57	70	85	87	58	72	86	88	59	76
February	84	87	57	70	88	90	56	69	85	89	53	64	54	87	54	64	85	88	55	71
March	83	87	55	68	89	91	53	64	85	90	51	62	86	89	51	60	86	89	52	68
April	82	86	52	64	88	91	51	62	85	88	48	60	87	90	47	56	86	89	49	65
May	82	85	54	64	90	91	50	63	87	89	50	64	89	91	51	60	88	90	53	68
June	85	86	60	70	93	93	59	74	90	91	58	73	91	92	56	68	90	90	59	75
July	86	88	64	74	94	94	63	78	91	92	59	76	93	94	61	74	91	91	60	76
August	88	90	65	76	94	96	64	80	92	93	60	78	93	95	61	76	92	93	62	80
September	88	91	63	76	94	96	64	81	92	93	61	79	91	93	58	75	93	94	65	83
October	87	90	58	73	92	94	62	81	89	91	57	76	88	91	53	72	92	93	61	84
November	86	89	59	74	92	93	61	82	89	91	57	75	89	90	56	77	91	92	59	84
December	86	88	61	75	91	91	62	81	88	90	59	74	87	88	58	77	88	90	61	82
Annual	85	88	59	72	78	93	59	74	88	91	56	71	89	91	55	69	89	91	58	76
Period of Record (years)	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30

Sources: References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-2

Table 2.3.2-244
Summary of On-Site and Regional Precipitation Observations (in.)

Month	LNP On-Site[(a)]	Tampa	Gainesville	Orlando	Tallahassee	Jacksonville
January	3.04	2.15	3.18	2.23	4.43	3.10
February	5.74	2.99	3.17	2.73	4.80	3.44
March	3.02	3.11	4.10	3.49	6.06	3.81
April	1.22	2.04	2.54	2.29	3.75	2.96
May	0.45	2.66	2.30	3.37	4.30	3.31
June	5.85	6.59	6.94	7.51	7.14	6.04
July	5.12	7.54	6.45	7.38	8.43	6.51
August	6.21	7.89	6.67	6.67	7.18	7.14
September	4.02	6.48	5.18	5.87	5.64	7.98
October	5.47	2.42	2.93	3.15	3.17	4.00
November	0.77	1.57	2.04	2.01	3.35	1.95
December	2.04	2.28	2.39	2.28	4.20	2.68
Annual	42.95	47.72	47.89	48.98	62.45	52.92
Period of Record (years)	1	74	25	54	59	59

Notes:

a) LNP on-site data is for the period from February 1, 2007, to January 31, 2008.

in. = inch

Sources: LNP on-site data; References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-2

Table 2.3.2-245
Average Number of Days of Fog Occurrence

Month	Tampa	Gainesville	Orlando	Tallahassee	Jacksonville
January	3.9	5.9	3.2	6.6	5.3
February	2.5	4.7	2.6	5.0	3.6
March	1.9	3.4	1.7	5.2	3.1
April	0.7	2.7	1.0	4.6	2.5
May	0.2	3.6	1.1	4.8	3.0
June	0.3	2.7	0.7	2.6	1.6
July	0.2	2.3	0.4	2.3	1.2
August	0.2	2.3	0.6	2.4	2.0
September	0.2	3.5	0.8	2.0	2.2
October	0.7	4.0	1.0	3.1	3.4
November	1.9	5.2	1.8	5.0	5.3
December	2.6	6.2	3.1	6.2	6.1
Annual	15.3	46.5	18.0	49.8	39.3
Period of Record	43	23	39	43	43

Sources: LNP on-site data; References 2.3-203, 2.3-204, 2.3-205, 2.3-206, and 2.3-207

LNP COL 2.3-2

Table 2.3.2-246 (Sheet 1 of 2)
Predicted Visible Cooling Tower Vapor Plume Heights and Lengths for LNP 1 and LNP 2

		Winter		Spring		Summer		Fall		Annual[a]	
		hours	%	hours	%	hours	%	hours	%	hours	%
All Hours (Except Existing Fog/Calm)		1573	17.96%	1410	16.10%	1114	12.72%	1500	17.10%	5597	63.90%
Plume Height (m)											
>0	<200	1573	17.96%	1396	15.94%	1062	12.12%	1477	16.86%	5508	62.88%
>200	<400	0	0.00%	14	0.16%	25	0.29%	16	0.18%	55	0.63%
>400	<500	0	0.00%	0	0.00%	12	0.14%	6	0.07%	18	0.21%
>500		0	0.00%	0	0.00%	15	0.17%	1	0.01%	16	0.18%
Plume Length (m)[b]											
>0	<100	1573	17.96%	1326	15.14%	918	10.48%	1411	16.11%	5228	59.69%
>100	<300	0	0.00%	29	0.33%	67	0.76%	27	0.31%	123	1.40%
>300	<500	0	0.00%	6	0.07%	5	0.06%	5	0.06%	16	0.18%
>500	<1000	0	0.00%	12	0.14%	46	0.53%	12	0.23%	70	0.80%
>1000	<1500	0	0.00%	9	0.10%	20	0.23%	8	0.10%	37	0.42%
>1500	<5000	0	0.00%	16	0.18%	13	0.15%	9	0.14%	38	0.43%
>5000		0	0.00%	12	0.14%	45	0.51%	28	0.29%	85	0.97%

LNP COL 2.3-2

Table 2.3.2-246 (Sheet 2 of 2)
Predicted Visible Cooling Tower Vapor Plume Heights and Lengths for LNP 1 and LNP 2

		Winter		Spring		Summer		Fall		Annual[a]	
		hours	%	hours	%	hours	%	hours	%	hours	%
Daylight Hours(Except Existing Fog/Calm)		388	4.43%	301	3.44%	201	2.29%	353	4.03%	1243	14.19
Plume Height (m)											
>0	<200	388	4.43%	292	3.33%	182	2.08%	336	3.84%	1198	13.68%
>200	<400	0	0.00%	9	0.10%	9	0.10%	13	0.15%	31	0.35%
>400	<500	0	0.00%	0	0.00%	4	0.05%	4	0.05%	8	0.09%
>500		0	0.00%	0	0.00%	6	0.07%	0	0.00%	6	0.07%
Plume Length (m)[b]											
>0	<100	388	4.40%	267	3.05%	133	1.52%	301	3.44%	1089	12.43%
>100	<300	0	0.00%	4	0.05%	14	0.14%	10	0.11%	28	0.32%
>300	<500	0	0.00%	1	0.01%	1	0.01%	4	0.05%	6	0.07%
>500	<1000	0	0.00%	4	0.05%	11	0.13%	6	0.07%	21	0.24%
>1000	<1500	0	0.00%	3	0.03%	12	0.14%	5	0.06%	20	0.23%
>1500	<5000	0	0.00%	14	0.16%	3	0.03%	6	0.07%	23	0.26%
>5000		0	0.00%	8	0.09%	27	0.31%	21	0.24%	56	0.64%

Notes:

a) Period of Record is 2003 (Gainesville, FL).

b) Distance measured relative to a location midway between the two tower banks.

m = meter

LNP COL 2.3-3

Table 2.3.3-201
LNP Meteorological Monitoring Tower
Meteorological Sensor Elevations

Sensor	Approximate Elevation Above Tower Base (m)
Wind Speed and Direction	10 and 60
Dew-Point	10
Solar Radiation	2.0
Ambient Temperature	10 and 60
Delta-Temperature[a]	10 and 60
Precipitation	2.0
Barometric Pressure	2.0

Notes:

a) Used to measure differential temperature channel between these elevations.

m = meter

LNP COL 2.3-3

Table 2.3.3-202
LNP Meteorological Monitoring Tower
Accuracy of Monitored Parameters

Monitored Parameter	Basis	Accuracy Criteria
Wind Direction (10 & 60 meters) 0 – 360 degrees	NRC Regulatory Guide 1.23, Revision 1	±5 degrees (°). Starting threshold <0.45 m/s (1 mph). Resolution to 1.0°.
Wind Speed (10 & 60 meters) 0 – 112 mph	NRC Regulatory Guide 1.23, Revision 1	±0.2 m/s (±0.45 mph) or 5% of observed wind speed. Starting threshold <0.45 m/s (1 mph). Resolution to 0.1 m/s or 0.1 mph.
Ambient Temperature (10 & 60 meters) -58°F to 122°F	NRC Regulatory Guide 1.23, Revision 1	±0.5°C (±0.9°F). Resolution to 0.1°C (0.1°F).
Differential Temperature (-180°F to +180°F calculated)	NRC Regulatory Guide 1.23, Revision 1	±0.1°C (±0.18°F). Resolution to 0.01°C (0.01°F).
Wet Bulb Temperature	NRC Regulatory Guide 1.23, Revision 1	±0.5°C (±0.9°F). Resolution to 0.1°C (0.1°F).
Relative Humidity/Dew-Point 0 – 98%	NRC Regulatory Guide 1.23, Revision 1	Relative Humidity: ±4% Resolution to 0.1%. Dew-Point: ±1.5°C (±2.7°F). Resolution to 0.1°C (0.1°F).
Total Precipitation	NRC Regulatory Guide 1.23, Revision 1	Precipitation (water equivalent). ±10% for a volume equivalent to 2.54 mm (0.1 in.) of precipitation at a rate <50 mm/h (<2 in/h). Resolution to 0.25 mm (0.01 in.)
Solar Radiation[a]	ANSI/ANS 2.5-1984	Consistent with current state-of-the-art.
Barometric Pressure[a] 800 – 1100 hPa/mb	ANSI/ANS 2.5-1984	Consistent with current state-of-the-art
Datalogger Sampling Rate	NRC Regulatory Guide 1.23, Revision 1	At least once every 5 seconds
Time	NRC Regulatory Guide 1.23, Revision 1	±5 minutes Resolution to ±1 minute

Notes:

a) There are no accuracies specified in the NRC Regulatory Guide 1.23 for these parameters. ANSI/ANS 2.5-1984 guidance reflects industry and regulator-accepted state-of-the-art specifications.

° = degrees
°F = degrees Fahrenheit
hPa/mb = hectoPascal/milliBar
in/h = inches per hour
m/s = meters per second
mm = millimeter
mm/h = millimeters per hour
mph = miles per hour

LNP COL 2.3-4

Table 2.3.4-201
Predicted LNP 1 and LNP 2 Chi/Q Values

Location and Averaging Period	AP1000 DCD Acceptance Criteria Chi/Q	LNP 1 and LNP 2 Maximum Predicted Chi/Q[(a)]
Exclusion Area Boundary		
0 – 2 hr.	≤ 5.1 x 10^{-4} sec/m^3	5.08 x 10^{-4} sec/m^3
Low Population Zone		
0 – 8 hr.	≤ 2.2 x 10^{-4} sec/m^3	9.70 x 10^{-5} sec/m^3
8 – 24 hr.	≤ 1.6 x 10^{-4} sec/m^3	7.19 x 10^{-5} sec/m^3
24 – 96 hr.	≤ 1.0 x 10^{-4} sec/m^3	3.75 x 10^{-5} sec/m^3
96 – 720 hr.	≤ 8.0 x 10^{-5} sec/m^3	1.48 x 10^{-5} sec/m^3

Notes:

a) Maximum predicted Chi/Q values occurred in the WSW sector for EAB and LPZ.

Chi/Q = atmospheric dilution factor
EAB = exclusion area boundary
LPZ = low population zone
sec/m^3 = second per cubic meter
hr. = hour

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 1 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007, to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class A																
≤0.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤1.5	0	1	0	2	1	0	1	1	0	1	0	3	2	4	1	0
≤2.0	4	2	2	4	4	0	3	2	3	1	1	5	2	5	6	0
≤3.0	8	22	16	15	18	9	3	4	2	5	20	21	30	12	11	17
≤4.0	8	11	30	34	28	11	3	8	7	6	43	106	98	11	13	19
≤5.0	3	9	11	35	42	13	0	1	3	18	38	77	53	4	6	14
≤6.0	0	0	7	18	19	1	0	0	3	6	11	19	32	2	0	0
≤8.0	0	0	0	4	6	0	0	0	0	1	1	1	10	2	0	0
≤10.0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 2 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class B																
≤0.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤1.0	0	0	2	0	1	0	0	0	0	0	0	1	0	0	0	0
≤1.5	4	4	5	5	2	0	1	2	4	1	0	2	3	1	3	4
≤2.0	3	11	9	12	8	4	2	8	5	2	3	5	5	6	5	8
≤3.0	20	21	41	25	34	16	16	16	2	9	33	39	54	15	24	23
≤4.0	18	21	34	49	59	34	14	6	7	7	34	70	72	5	9	12
≤5.0	6	8	23	25	29	6	2	0	1	10	8	19	32	4	1	5
≤6.0	0	1	10	11	4	3	0	0	2	6	2	3	4	0	0	0
≤8.0	0	0	0	2	0	1	0	0	0	5	0	0	4	2	0	0
≤10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 3 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class C																
≤0.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤1.0	3	1	0	0	0	0	1	1	0	0	0	1	1	2	3	2
≤1.5	7	7	6	7	11	3	4	4	6	10	4	6	8	7	7	7
≤2.0	9	22	10	14	18	15	12	8	10	9	11	14	12	11	6	16
≤3.0	30	37	39	53	55	24	23	14	18	16	37	53	77	22	13	22
≤4.0	8	14	43	52	49	24	11	13	10	13	19	53	74	3	3	9
≤5.0	2	8	21	27	29	11	3	2	2	8	11	14	18	1	0	4
≤6.0	0	2	6	7	6	1	0	0	3	10	4	0	3	0	0	1
≤8.0	0	0	0	2	1	0	0	0	0	3	0	1	1	0	0	0
≤10.0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 4 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class D																
≤0.4	4	3	4	3	3	2	3	3	3	3	4	4	3	2	3	2
≤0.5	0	1	0	0	3	1	0	1	2	1	3	0	1	1	2	1
≤1.0	23	10	18	16	22	14	13	12	15	17	15	17	10	9	12	8
≤1.5	40	36	39	26	24	22	31	25	30	20	37	37	27	26	34	27
≤2.0	50	54	80	60	73	31	31	21	17	28	42	52	61	35	34	35
≤3.0	102	112	197	196	142	94	51	32	48	59	73	147	198	44	32	54
≤4.0	42	73	127	118	113	46	40	18	22	68	39	95	83	11	24	25
≤5.0	19	30	50	69	52	25	10	8	27	48	27	29	24	12	8	7
≤6.0	0	1	13	20	22	9	1	1	12	27	12	11	18	2	0	3
≤8.0	0	0	1	0	4	1	0	0	4	11	8	9	10	1	0	0
≤10.0	0	0	0	0	0	0	0	0	0	1	1	0	4	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 5 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class E																
≤0.4	8	19	27	29	26	17	15	12	9	7	11	11	10	11	8	6
≤0.5	6	5	5	4	3	2	4	4	4	3	3	2	3	3	5	2
≤1.0	21	60	72	72	62	42	56	53	28	25	34	37	35	52	39	17
≤1.5	34	82	133	147	133	83	53	35	38	26	47	49	39	32	21	27
≤2.0	40	51	127	134	126	58	46	14	38	19	19	31	32	15	28	30
≤3.0	61	82	101	123	131	62	42	17	30	34	9	22	26	12	35	30
≤4.0	8	15	11	17	23	10	3	1	17	6	3	12	7	2	4	8
≤5.0	1	0	3	5	1	3	1	2	6	1	0	3	1	5	1	2
≤6.0	0	0	0	0	0	0	0	0	1	0	0	2	0	0	0	0
≤8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤10.0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 6 of 7)
Meteorological Input Data for PAVAN Model
Levy Nuclear Plant Meteorological Monitoring Station
Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class F																
≤0.4	30	38	66	136	117	50	29	25	18	17	20	13	19	23	22	21
≤0.5	5	8	6	18	8	7	8	7	8	8	3	1	4	7	7	1
≤1.0	21	34	74	109	100	40	32	28	16	16	22	14	25	29	20	19
≤1.5	29	26	39	119	103	43	12	10	8	7	11	9	6	5	13	18
≤2.0	15	10	5	31	44	14	3	2	2	0	1	2	2	1	2	10
≤3.0	1	2	0	0	7	1	0	0	1	2	3	1	3	0	1	0
≤4.0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	1	0
≤5.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

LNP COL 2.3-4

Table 2.3.4-202 (Sheet 7 of 7) Meteorological Input Data for PAVAN Model Levy Nuclear Plant Meteorological Monitoring Station Period of Record: February 1, 2007 to January 31, 2009 (Lower Elevation)

Wind Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class G																
≤0.4	105	59	180	607	535	213	102	79	62	33	20	20	36	49	118	85
≤0.5	5	1	16	36	34	6	7	7	6	0	3	1	3	5	6	2
≤1.0	19	15	32	107	97	49	22	14	11	9	2	4	5	8	27	16
≤1.5	8	2	7	42	32	10	2	3	2	1	1	1	3	2	3	8
≤2.0	3	0	0	5	3	2	0	0	0	1	0	0	0	0	1	1
≤3.0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0
≤4.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤5.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤8.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤10.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
≤15.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:

a) Data representative of hours of occurrence by direction and wind speed category.
b) Calms are distributed into lowest windspeed category by PAVAN for recorded calm hours in stability classes C to G: class C – 1 hr; class D – 49 hr; Class E – 227 hr; class F – 643 h; class G – 2303 hr.
Wind direction: E = east; N = north; S = south; W = west
m/s = meters per second

LNP COL 2.3-4

Table 2.3.4-203

0- to 2-Hour 5th Percentile Exclusion Area Boundary Chi/Q Values for LNP 1 and LNP 2

Downwind Sector [(a)]	Distance (m)	Distance (ft.)	0-2 hr. Chi/Q with Wake (sec/m^3)	0-2 hr. Chi/Q without Wake sec/m^3)
S	1340	4396	5.08E-04	5.08E-04
SSW	1340	4396	3.67E-04	3.67E-04
SW	1340	4396	5.08E-04	5.08E-04
WSW	1340	4396	5.08E-04	5.08E-04
W	1340	4396	5.08E-04	5.08E-04
WNW	1340	4396	5.08E-04	5.08E-04
NW	1340	4396	5.08E-04	5.08E-04
NNW	1340	4396	4.52E-04	4.52E-04
N	1340	4396	3.27E-04	3.27E-04
NNE	1340	4396	1.52E-04	1.52E-04
NE	1340	4396	1.32E-04	1.32E-04
ENE	1340	4396	1.11E-04	1.11E-04
E	1340	4396	1.87E-04	1.87E-04
ESE	1247	4091	2.97E-04	2.97E-04
SE	1340	4396	5.08E-04	5.08E-04
SSE	1340	4396	4.96E-04	4.96E-04
MAX Chi/Q			5.08E-04	5.08E-04

Notes:

Predictions based on PAVAN model as described in FSAR Subsection 2.3.4.2.

a) Downwind Sector: E = east; N = north; S = south; W = west

Chi/Q = atmospheric dilution factor
ft. = foot
m = meter
sec/m^3 = second per cubic meter

LNP COL 2.3-4

Table 2.3.4-204
0- to 30-Day 5th Percentile Low Population Zone Chi/Q Values for LNP 1 and LNP 2

Downwind Sector [(a)]	Distance (m)	Distance (mi.)	0-8 hr. Chi/Q without Wake (sec/m3)	8-24 hr. Chi/Q without Wake (sec/m3)	1-4 days Chi/Q without Wake (sec/m3)	4-30 days Chi/Q without Wake (sec/m3)
S	4830	3	7.41E-05	4.80E-05	1.87E-05	4.83E-06
SSW	4830	3	4.82E-05	3.19E-05	1.30E-05	3.60E-06
SW	4830	3	8.22E-05	5.61E-05	2.44E-05	7.43E-06
WSW	4830	3	9.70E-05	7.19E-05	3.75E-05	1.48E-05
W	4830	3	9.50E-05	6.97E-05	3.56E-05	1.35E-05
WNW	4830	3	8.21E-05	5.60E-05	2.44E-05	7.40E-06
NW	4830	3	7.40E-05	4.79E-05	1.87E-05	4.82E-06
NNW	4830	3	6.21E-05	4.00E-05	1.54E-05	3.89E-06
N	4830	3	4.27E-05	2.79E-05	1.11E-05	2.96E-06
NNE	4830	3	1.99E-05	1.35E-05	5.78E-06	1.71E-06
NE	4830	3	1.67E-05	1.14E-05	4.93E-06	1.49E-06
ENE	4830	3	1.34E-05	9.31E-06	4.20E-06	1.34E-06
E	4830	3	2.41E-05	1.63E-05	6.97E-06	2.06E-06
ESE	4830	3	3.44E-05	2.27E-05	9.28E-06	2.56E-06
SE	4830	3	7.43E-05	4.82E-05	1.89E-05	4.91E-06
SSE	4830	3	6.90E-05	4.39E-05	1.64E-05	3.99E-06

Notes:

Predictions based on PAVAN model as described in FSAR Subsection 2.3.4.2.

Period of Record of meteorological data is from February 1, 2007, to January 31, 2009.

Chi/Qs without wake bound (are greater than or equal to) Chi/Qs with wake allowance.

a) Downwind sector: E = east; N = north; S = south; W = west
Chi/Q = atmospheric dilution factor;
hr. = hour;
m = meter; mi. = mile;
sec/m3 = seconds per cubic meter

Table 2.3.4-205
Deleted**.**

LNP COL 2.3-4

Table 2.3.4-206 (Sheet 1 of 2)
Comparison of Control Room Atmospheric Dispersion Factors for Accident Analysis for AP1000 DCD and LNP Units 1 and 2

Chi/Q (sec/m^3) at HVAC Intake for the Identified Release Points[a]														
	Plant Vent or PCS Air Diffuser[b]	**Plant Vent**	**PCS Air Diffuser**	**Ground Level Contain-ment Release Points[c]**	**Ground Level Contain-ment Release Points**	**PORV and Safety Valve Releases[d]**	**PORV and Safety Valve Releases**	**Condenser Air Removal Stack[g]**	**Condenser Air Removal Stack**	**Steam Line Break Releases**	**Steam Vent**	**Fuel Handling Area[e]**	**Fuel Handling Area Blowout Panel**	**Radwaste Building Truck Staging Area Door**
Release Time	**DCD**	**LNP**	**LNP**	**DCD**	**LNP**	**DCD**	**LNP**	**DCD**	**LNP**	**DCD**	**LNP**	**DCD**	**LNP**	**LNP**
0 - 2 hours	**3.0E-03**	1.7E-03	1.5E-03	**6.0E-03**	4.3E-03	**2.0E-02**	1.0E-02	**6.0E-03**	1.7E-03	**2.4E-02**	1.1E-02	**6.0E-03**	1.3E-03	1.0E-03
2 - 8 hours	**2.5E-03**	1.0E-03	8.4E-04	**3.6E-03**	3.5E-03	**1.8E-02**	5.7E-03	**4.0E-03**	1.4E-03	**2.0E-02**	6.1E-03	**4.0E-03**	8.3E-04	6.4E-04
8 - 24 hours	**1.0E-03**	4.5E-04	3.7E-04	**1.4E-03**	1.2E-03	**7.0E-03**	2.7E-03	**2.0E-03**	6.4E-04	**7.5E-03**	3.0E-03	**2.0E-03**	3.7E-04	3.0E-04
1 - 4 days	**8.0E-04**	4.5E-04	3.8E-04	**1.8E-03**	1.2E-03	**5.0E-03**	2.1E-03	**1.5E-03**	5.9E-04	**5.5E-03**	2.3E-03	**1.5E-03**	3.4E-04	2.6E-04
4 - 30 days	**6.0E-04**	3.6E-04	3.0E-04	**1.5E-03**	9.9E-04	**4.5E-03**	1.3E-03	**1.0E-03**	4.7E-04	**5.0E-03**	1.5E-03	**1.0E-03**	2.7E-04	2.0E-04
Chi/Q (sec/m^3) at Annex Building Door for the Identified Release Points[f]														
	Plant Vent or PCS Air Diffuser[b]	**Plant Vent**	**PCS Air Diffuser**	**Ground Level Contain-ment Release Points[c]**	**Ground Level Contain-ment Release Points**	**PORV and Safety Valve Releases[d]**	**PORV and Safety Valve Releases**	**Condenser Air Removal Stack[g]**	**Condenser Air Removal Stack**	**Steam Line Break Releases**	**Steam Vent**	**Fuel Handling Area[e]**	**Fuel Handling Area Blowout Panel**	**Radwaste Building Truck Staging Area Door**
Release Time	**DCD**	LNP	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	**DCD**	LNP	LNP
0 - 2 hours	**1.0E-03**	3.7E-04	3.8E-04	**1.0E-03**	3.4E-04	**4.0E-03**	8.3E-04	**2.0E-02**	3.2E-03	**4.0E-03**	8.0E-04	**6.0E-03**	3.3E-04	3.2E-04
2 - 8 hours	**7.5E-04**	2.4E-04	2.5E-04	**7.5E-04**	2.8E-04	**3.2E-03**	4.8E-04	**1.8E-02**	1.8E-03	**3.2E-03**	4.7E-04	**4.0E-03**	2.2E-04	2.1E-04
8 - 24 hours	**3.5E-04**	1.1E-04	1.1E-04	**3.5E-04**	1.3E-04	**1.2E-03**	2.3E-04	**7.0E-03**	7.8E-04	**1.2E-03**	2.2E-04	**2.0E-03**	1.0E-04	1.0E-04
1 - 4 days	**2.8E-04**	1.1E-04	1.1E-04	**2.8E-04**	1.2E-04	**1.0E-03**	2.2E-04	**5.0E-03**	6.9E-04	**1.0E-03**	2.1E-04	**1.5E-03**	9.8E-05	9.4E-05
4 - 30 days	**2.5E-04**	8.9E-05	9.1E-05	**2.5E-04**	1.0E-04	**8.0E-04**	1.8E-04	**4.5E-03**	5.3E-04	**8.0E-04**	1.8E-04	**1.0E-03**	7.8E-05	7.5E-05

LNP COL 2.3-4

Table 2.3.4-206 (Sheet 2 of 2)
Comparison of Control Room Atmospheric Dispersion Factors for Accident Analysis for AP1000 DCD and LNP Units 1 and 2

Notes:

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

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

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

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

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

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 handling area relief panel opens on high temperature. These dispersion factors are used for the fuel handling accident occurring outside containment and for evaluating the impact of releases associated with spent fuel pool boiling.

g) 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.

Chi/Q = atmospheric dilution factor
HVAC = heating, ventilation, and air conditioning
sec/m^3 = second per cubic meter

LNP COL 2.3-4

Table 2.3.4-207
Control Room Release/Receptor Azimuthal Angles for Input to ARCON96

	Receptor Location	
Release Location	Control Room HVAC Intake (degrees)	Annex Building Access (degrees)
Plant Vent	238	240
PCS Air Diffuser	270	251
Fuel Building Blowout Panel	223	232
Radwaste Building Truck Staging Area Door	214	227
Steam Vent	313	255
PORV/Safety Valves	322	258
Condenser Air Removal Stack	52	290
Containment Shell (Diffuse Area Source)	261	245

LNP COL 2.3-5

Table 2.3.5-201 (Sheet 1 of 4)
Long-Term Chi/Q Calculations for Routine Releases for LNP 1 and LNP 2

	Exclusion Area Boundary		Low Population Zone (a)		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector (b)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)
N	1340	2.90E-06	4829	5.20E-07	8049	2.70E-07	8049	2.70E-07	2898	1.00E-06	4990	5.00E-07
NNE	1340	2.10E-06	4829	3.60E-07	8049	1.80E-07	8049	1.80E-07	6600	2.40E-07	8049	1.80E-07
NE	1340	1.90E-06	4829	3.10E-07	8049	1.50E-07	8049	1.50E-07	6600	2.00E-07	8049	1.50E-07
ENE	1340	1.80E-06	4829	3.00E-07	8049	1.50E-07	8049	1.50E-07	8049	1.50E-07	7244	1.70E-07
E	1340	2.40E-06	4829	4.00E-07	8049	2.00E-07	8049	2.00E-07	7727	2.10E-07	6922	2.50E-07
ESE	1340	2.80E-06	4829	5.00E-07	8049	2.50E-07	8049	2.50E-07	8049	2.50E-07	5795	3.90E-07
SE	1340	4.50E-06	4829	8.40E-07	8049	4.40E-07	8049	4.40E-07	5956	6.40E-07	6600	5.60E-07
SSE	1340	2.80E-06	4829	5.10E-07	8049	2.60E-07	8049	2.60E-07	8049	2.60E-07	4185	6.10E-07
S	1340	3.80E-06	4829	6.90E-07	8049	3.60E-07	8049	3.60E-07	6761	4.50E-07	8049	3.60E-07
SSW	1340	3.80E-06	4829	6.60E-07	8049	3.40E-07	8049	3.40E-07	4829	6.60E-07	4507	7.20E-07
SW	1340	8.20E-06	4829	1.50E-06	8049	7.70E-07	8049	7.70E-07	4024	1.90E-06	8049	7.70E-07
WSW	1340	1.90E-05	4829	3.50E-06	8049	1.90E-06	8049	1.90E-06	2737	7.30E-06	8049	1.90E-06
W	1340	1.70E-05	4829	3.20E-06	8049	1.70E-06	8049	1.70E-06	8049	1.70E-06	8049	1.70E-06
WNW	1340	7.50E-06	4829	1.40E-06	8049	7.30E-07	8049	7.30E-07	8049	7.30E-07	8049	7.30E-07
NW	1340	4.60E-06	4829	8.50E-07	8049	4.40E-07	8049	4.40E-07	2576	1.90E-06	3541	1.30E-06
NNW	1340	3.60E-06	4829	6.50E-07	8049	3.40E-07	3863	8.70E-07	3380	1.00E-06	4668	6.80E-07

LNP COL 2.3-5

Table 2.3.5-201 (Sheet 2 of 4)
Long-Term Chi/Q Calculations for Routine Releases for LNP 1 and LNP 2

Nearest Residence			Downwind Distance (mi.) (X/Q in sec/m^3)										
Downwind Sector [(b)]	Distance (m)	X/Q (sec/m^3)	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2898	1.00E-06	2.20E-05	6.68E-06	3.39E-06	2.19E-06	1.28E-06	8.79E-07	6.57E-07	5.18E-07	4.24E-07	3.57E-07	3.07E-07
NNE	6600	2.40E-07	1.52E-05	4.70E-06	2.44E-06	1.58E-06	9.12E-07	6.19E-07	4.59E-07	3.60E-07	2.93E-07	2.46E-07	2.10E-07
NE	8049	1.50E-07	1.30E-05	4.10E-06	2.17E-06	1.41E-06	8.02E-07	5.38E-07	3.96E-07	3.08E-07	2.50E-07	2.08E-07	1.78E-07
ENE	8049	1.50E-07	1.29E-05	4.12E-06	2.16E-06	1.39E-06	7.82E-07	5.21E-07	3.81E-07	2.96E-07	2.39E-07	1.99E-07	1.69E-07
E	7727	2.10E-07	1.75E-05	5.44E-06	2.84E-06	1.84E-06	1.05E-06	7.03E-07	5.17E-07	4.03E-07	3.27E-07	2.73E-07	2.33E-07
ESE	5956	3.80E-07	2.08E-05	6.35E-06	3.30E-06	2.14E-06	1.25E-06	8.49E-07	6.32E-07	4.97E-07	4.06E-07	3.40E-07	2.92E-07
SE	4185	1.00E-06	3.61E-05	1.07E-05	5.34E-06	3.43E-06	2.04E-06	1.41E-06	1.06E-06	8.39E-07	6.89E-07	5.82E-07	5.01E-07
SSE	4668	5.30E-07	2.19E-05	6.56E-06	3.29E-06	2.12E-06	1.25E-06	8.60E-07	6.44E-07	5.09E-07	4.17E-07	3.52E-07	3.03E-07
S	6761	4.50E-07	2.98E-05	8.97E-06	4.52E-06	2.91E-06	1.71E-06	1.18E-06	8.79E-07	6.94E-07	5.69E-07	4.79E-07	4.12E-07
SSW	4507	7.20E-07	2.76E-05	8.56E-06	4.46E-06	2.90E-06	1.68E-06	1.14E-06	8.43E-07	6.61E-07	5.39E-07	4.52E-07	3.87E-07
SW	3220	2.50E-06	6.33E-05	1.91E-05	9.73E-06	6.29E-06	3.69E-06	2.53E-06	1.89E-06	1.49E-06	1.22E-06	1.03E-06	8.85E-07
WSW	2737	7.30E-06	1.52E-04	4.48E-05	2.22E-05	1.43E-05	8.53E-06	5.92E-06	4.46E-06	3.54E-06	2.92E-06	2.47E-06	2.13E-06
W	8049	1.70E-06	1.39E-04	4.07E-05	2.02E-05	1.30E-05	7.75E-06	5.38E-06	4.05E-06	3.22E-06	2.65E-06	2.24E-06	1.93E-06
WNW	8049	7.30E-07	5.99E-05	1.77E-05	8.83E-06	5.68E-06	3.38E-06	2.34E-06	1.76E-06	1.40E-06	1.15E-06	9.69E-07	8.36E-07
NW	2576	1.90E-06	3.59E-05	1.08E-05	5.48E-06	3.54E-06	2.09E-06	1.43E-06	1.07E-06	8.48E-07	6.95E-07	5.85E-07	5.04E-07
NNW	3380	1.00E-06	2.74E-05	8.27E-06	4.21E-06	2.72E-06	1.60E-06	1.10E-06	8.22E-07	6.49E-07	5.31E-07	4.47E-07	3.85E-07

LNP COL 2.3-5

Table 2.3.5-201 (Sheet 3 of 4) Long-Term Chi/Q Calculations for Routine Releases for LNP 1 and LNP 2

Downwind Sector (b)	Downwind Distance (Mi.) (X/Q in sec/m^3)										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	2.68E-07	1.60E-07	1.11E-07	6.68E-08	4.68E-08	3.56E-08	2.84E-08	2.36E-08	2.00E-08	1.74E-08	1.53E-08
NNE	1.83E-07	1.08E-07	7.45E-08	4.44E-08	3.09E-08	2.33E-08	1.86E-08	1.54E-08	1.30E-08	1.13E-08	9.89E-09
NE	1.54E-07	8.98E-08	6.14E-08	3.62E-08	2.50E-08	1.88E-08	1.49E-08	1.22E-08	1.03E-08	8.91E-09	7.81E-09
ENE	1.47E-07	8.50E-08	5.79E-08	3.40E-08	2.34E-08	1.76E-08	1.39E-08	1.14E-08	9.65E-09	8.32E-09	7.29E-09
E	2.03E-07	1.18E-07	8.12E-08	4.81E-08	3.33E-08	2.51E-08	2.00E-08	1.65E-08	1.39E-08	1.20E-08	1.06E-08
ESE	2.55E-07	1.51E-07	1.04E-07	6.25E-08	4.36E-08	3.30E-08	2.63E-08	2.18E-08	1.85E-08	1.60E-08	1.41E-08
SE	4.39E-07	2.64E-07	1.85E-07	1.12E-07	7.89E-08	6.02E-08	4.83E-08	4.01E-08	3.41E-08	2.97E-08	2.62E-08
SSE	2.65E-07	1.58E-07	1.10E-07	6.68E-08	4.69E-08	3.57E-08	2.86E-08	2.37E-08	2.02E-08	1.75E-08	1.54E-08
S	3.60E-07	2.15E-07	1.50E-07	9.03E-08	6.34E-08	4.82E-08	3.86E-08	3.20E-08	2.72E-08	2.36E-08	2.08E-08
SSW	3.37E-07	1.99E-07	1.37E-07	8.18E-08	5.69E-08	4.30E-08	3.43E-08	2.83E-08	2.40E-08	2.07E-08	1.82E-08
SW	7.73E-07	4.61E-07	3.21E-07	1.93E-07	1.35E-07	1.03E-07	8.23E-08	6.82E-08	5.79E-08	5.02E-08	4.42E-08
WSW	1.86E-06	1.13E-06	7.88E-07	4.80E-07	3.38E-07	2.58E-07	2.07E-07	1.72E-07	1.47E-07	1.28E-07	1.13E-07
W	1.69E-06	1.02E-06	7.16E-07	4.36E-07	3.07E-07	2.35E-07	1.88E-07	1.57E-07	1.34E-07	1.16E-07	1.02E-07
WNW	7.32E-07	4.41E-07	3.08E-07	1.87E-07	1.32E-07	1.01E-07	8.08E-08	6.71E-08	5.72E-08	4.97E-08	4.38E-08
NW	4.40E-07	2.63E-07	1.83E-07	1.11E-07	7.76E-08	5.90E-08	4.73E-08	3.92E-08	3.33E-08	2.89E-08	2.55E-08
NNW	3.36E-07	2.01E-07	1.40E-07	8.41E-08	5.89E-08	4.48E-08	3.58E-08	2.97E-08	2.52E-08	2.19E-08	1.93E-08

Notes:

Wind Reference Level: 10 m
Stability Type: ΔT (60 – 10 m)
Release Type: Ground Level:10 m
Building Height/Cross Section: 43.9 m/2730 m^2
Period of record: February 1, 2007, to January 31, 2009

LNP COL 2.3-5

Table 2.3.5-201 (Sheet 4 of 4)
Long-Term Chi/Q Calculations for Routine Releases for LNP 1 and LNP 2

Notes (continued):

a) The reported distance of the low population zone (LPZ) is measured from the centerpoint of LNP 1 and LNP 2 to the outermost boundary of the LPZ.

b) Downwind Sector: E = east; N = north, S = south W = west

X/Q = local atmospheric dilution factor
m = meter
m^2 = square meter
mi. = mile
sec/m^3 = seconds per cubic meter

LNP COL 2.3-5

Table 2.3.5-202 (Sheet 1 of 4)
Long-Term Average D/Q Calculations for Routine Releases for LNP 1 and LNP 2

	Exclusion Area Boundary		Low Population Zone [(a)]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector [(b)]	Distance (m)	D/Q (m^{-2})	Distance (m)	D/Q (m^{-2})	Distance (m)	D/Q (m^{-2})	Distance (m)	D/Q (m^{-2})	Distance (m)	D/Q (m^{-2})	Distance (m)	D/Q (m^{-2})
N	1340	2.80E-09	4829	3.10E-10	8049	1.20E-10	8049	1.20E-10	2898	7.60E-10	4990	2.90E-10
NNE	1340	3.20E-09	4829	3.50E-10	8049	1.40E-10	8049	1.40E-10	6600	2.00E-10	8049	1.40E-10
NE	1340	3.70E-09	4829	4.00E-10	8049	1.60E-10	8049	1.60E-10	6600	2.30E-10	8049	1.60E-10
ENE	1340	5.60E-09	4829	6.10E-10	8049	2.40E-10	8049	2.40E-10	8049	2.40E-10	7244	2.90E-10
E	1340	6.30E-09	4829	6.90E-10	8049	2.80E-10	8049	2.80E-10	7727	3.00E-10	6922	3.60E-10
ESE	1340	2.60E-09	4829	2.90E-10	8049	1.10E-10	8049	1.10E-10	8049	1.10E-10	5795	2.10E-10
SE	1340	3.10E-09	4829	3.40E-10	8049	1.40E-10	8049	1.40E-10	5956	2.30E-10	6600	1.90E-10
SSE	1340	3.00E-09	4829	3.20E-10	8049	1.30E-10	8049	1.30E-10	8049	1.30E-10	4185	4.20E-10
S	1340	4.00E-09	4829	4.30E-10	8049	1.70E-10	8049	1.70E-10	6761	2.40E-10	8049	1.70E-10
SSW	1340	5.00E-09	4829	5.50E-10	8049	2.20E-10	8049	2.20E-10	4829	5.50E-10	4507	6.20E-10
SW	1340	8.60E-09	4829	9.30E-10	8049	3.70E-10	8049	3.70E-10	4024	1.30E-09	8049	3.70E-10
WSW	1340	1.30E-08	4829	1.40E-09	8049	5.50E-10	8049	5.50E-10	2737	3.70E-09	8049	5.50E-10
W	1340	1.20E-08	4829	1.30E-09	8049	5.20E-10	8049	5.20E-10	8049	5.20E-10	8049	5.20E-10
WNW	1340	5.50E-09	4829	6.00E-10	8049	2.40E-10	8049	2.40E-10	8049	2.40E-10	8049	2.40E-10
NW	1340	3.60E-09	4829	3.90E-10	8049	1.60E-10	8049	1.60E-10	2576	1.20E-09	3541	6.80E-10
NNW	1340	2.60E-09	4829	2.90E-10	8049	1.10E-10	3863	4.20E-10	3380	5.40E-10	4668	3.00E-10

LNP COL 2.3-5

Table 2.3.5-202 (Sheet 2 of 4)
Long-Term Average D/Q Calculations for Routine Releases for LNP 1 and LNP 2

	Nearest Residence		Downwind Distance (mi.) (D/Q in m^{-2})										
Downwind Sector [(b)]	Distance (m)	D/Q (m^{-2})	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2898	7.60E-10	1.95E-08	6.61E-09	3.39E-09	2.08E-09	1.04E-09	6.30E-10	4.26E-10	3.09E-10	2.35E-10	1.85E-10	1.50E-10
NNE	6600	2.00E-10	2.20E-08	7.42E-09	3.81E-09	2.34E-09	1.17E-09	7.08E-10	4.78E-10	3.47E-10	2.64E-10	2.08E-10	1.68E-10
NE	8049	1.60E-10	2.53E-08	8.56E-09	4.40E-09	2.70E-09	1.35E-09	8.16E-10	5.52E-10	4.00E-10	3.04E-10	2.40E-10	1.94E-10
ENE	8049	2.40E-10	3.85E-08	1.30E-08	6.68E-09	4.10E-09	2.05E-09	1.24E-09	8.39E-10	6.08E-10	4.62E-10	3.64E-10	2.95E-10
E	7727	3.00E-10	4.36E-08	1.48E-08	7.57E-09	4.65E-09	2.32E-09	1.41E-09	9.51E-10	6.89E-10	5.24E-10	4.13E-10	3.34E-10
ESE	5956	2.00E-10	1.81E-08	6.13E-09	3.15E-09	1.93E-09	9.63E-10	5.84E-10	3.95E-10	2.86E-10	2.18E-10	1.71E-10	1.39E-10
SE	4185	4.40E-10	2.16E-08	7.29E-09	3.74E-09	2.30E-09	1.15E-09	6.95E-10	4.70E-10	3.40E-10	2.59E-10	2.04E-10	1.65E-10
SSE	4668	3.40E-10	2.05E-08	6.93E-09	3.56E-09	2.18E-09	1.09E-09	6.60E-10	4.46E-10	3.23E-10	2.46E-10	1.94E-10	1.57E-10
S	6761	2.40E-10	2.74E-08	9.26E-09	4.76E-09	2.92E-09	1.46E-09	8.83E-10	5.97E-10	4.33E-10	3.29E-10	2.59E-10	2.10E-10
SSW	4507	6.20E-10	3.47E-08	1.17E-08	6.02E-09	3.70E-09	1.84E-09	1.12E-09	7.56E-10	5.48E-10	4.16E-10	3.28E-10	2.66E-10
SW	3220	1.90E-09	5.90E-08	2.00E-08	1.02E-08	6.29E-09	3.14E-09	1.90E-09	1.29E-09	9.32E-10	7.08E-10	5.58E-10	4.52E-10
WSW	2737	3.70E-09	8.67E-08	2.93E-08	1.51E-08	9.25E-09	4.61E-09	2.80E-09	1.89E-09	1.37E-09	1.04E-09	8.21E-10	6.64E-10
W	8049	5.20E-10	8.14E-08	2.75E-08	1.41E-08	8.68E-09	4.33E-09	2.63E-09	1.78E-09	1.29E-09	9.78E-10	7.71E-10	6.24E-10
WNW	8049	2.40E-10	3.79E-08	1.28E-08	6.58E-09	4.04E-09	2.01E-09	1.22E-09	8.26E-10	5.98E-10	4.55E-10	3.59E-10	2.90E-10
NW	2576	1.20E-09	2.48E-08	8.38E-09	4.30E-09	2.64E-09	1.32E-09	7.99E-10	5.40E-10	3.91E-10	2.98E-10	2.34E-10	1.90E-10
NNW	3380	5.40E-10	1.81E-08	6.12E-09	3.14E-09	1.93E-09	9.62E-10	5.83E-10	3.94E-10	2.86E-10	2.17E-10	1.71E-10	1.39E-10

LNP COL 2.3-5

Table 2.3.5-202 (Sheet 3 of 4)
Long-Term Average D/Q Calculations for Routine Releases for LNP 1 and LNP 2

Downwind Sector (b)	Downwind Distance (mi.) (D/Q in m^{-2})										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	1.24E-10	6.07E-11	3.81E-11	1.92E-11	1.17E-11	7.81E-12	5.60E-12	4.20E-12	3.27E-12	2.61E-12	2.13E-12
NNE	1.39E-10	6.82E-11	4.28E-11	2.16E-11	1.31E-11	8.77E-12	6.28E-12	4.72E-12	3.67E-12	2.93E-12	2.39E-12
NE	1.60E-10	7.86E-11	4.93E-11	2.49E-11	1.51E-11	1.01E-11	7.25E-12	5.44E-12	4.23E-12	3.38E-12	2.76E-12
ENE	2.44E-10	1.20E-10	7.50E-11	3.79E-11	2.29E-11	1.54E-11	1.10E-11	8.27E-12	6.43E-12	5.14E-12	4.19E-12
E	2.76E-10	1.35E-10	8.50E-11	4.29E-11	2.60E-11	1.74E-11	1.25E-11	9.38E-12	7.29E-12	5.82E-12	4.75E-12
ESE	1.15E-10	5.63E-11	3.53E-11	1.78E-11	1.08E-11	7.24E-12	5.19E-12	3.90E-12	3.03E-12	2.42E-12	1.98E-12
SE	1.37E-10	6.69E-11	4.20E-11	2.12E-11	1.28E-11	8.61E-12	6.17E-12	4.63E-12	3.60E-12	2.88E-12	2.35E-12
SSE	1.30E-10	6.36E-11	3.99E-11	2.02E-11	1.22E-11	8.18E-12	5.86E-12	4.40E-12	3.42E-12	2.74E-12	2.23E-12
S	1.74E-10	8.51E-11	5.34E-11	2.70E-11	1.63E-11	1.10E-11	7.84E-12	5.89E-12	4.58E-12	3.66E-12	2.99E-12
SSW	2.20E-10	1.08E-10	6.75E-11	3.41E-11	2.07E-11	1.39E-11	9.93E-12	7.45E-12	5.80E-12	4.63E-12	3.78E-12
SW	3.74E-10	1.83E-10	1.15E-10	5.81E-11	3.52E-11	2.36E-11	1.69E-11	1.27E-11	9.86E-12	7.88E-12	6.43E-12
WSW	5.50E-10	2.69E-10	1.69E-10	8.54E-11	5.17E-11	3.47E-11	2.48E-11	1.87E-11	1.45E-11	1.16E-11	9.45E-12
W	5.16E-10	2.53E-10	1.59E-10	8.02E-11	4.85E-11	3.25E-11	2.33E-11	1.75E-11	1.36E-11	1.09E-11	8.88E-12
WNW	2.40E-10	1.18E-10	7.38E-11	3.73E-11	2.26E-11	1.51E-11	1.09E-11	8.15E-12	6.33E-12	5.06E-12	4.13E-12
NW	1.57E-10	7.69E-11	4.83E-11	2.44E-11	1.48E-11	9.90E-12	7.09E-12	5.33E-12	4.14E-12	3.31E-12	2.70E-12
NNW	1.15E-10	5.62E-11	3.53E-11	1.78E-11	1.08E-11	7.23E-12	5.18E-12	3.89E-12	3.03E-12	2.42E-12	1.97E-12

Notes:

Wind Reference Level: 10 m
Stability Type: ΔT (60 – 10 m)
Release Type: Ground Level: 10 m

LNP COL 2.3-5

Table 2.3.5-202 (Sheet 4 of 4)
Long-Term Average D/Q Calculations for Routine Releases for LNP 1 and LNP 2

Notes (continued):

Building Height/Cross Section: 43.9 m/2730 m^2
POR: February 1, 2007 – January 31, 2009

a) The reported distance of the low population zone (LPZ) is measured from the centerpoint of LNP 1 and LNP 2 to the outermost boundary of the LPZ.

b) Downwind Sector: E = east; N = north; S = south; W = west

D/Q = relative deposition
m = meter
mi. = mile
m^{-2} = 1/m^2
m^2 = square meter

LNP COL 2.3-5

Table 2.3.5-203 (Sheet 1 of 4)
Long-Term Average Chi/Q Calculations (2.26-Day Decay) for Routine Releases for LNP 1 and LNP 2

	Exclusion Area Boundary		Low Population Zone [a]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector [b]	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)
N	1340	2.80E-06	4829	4.80E-07	8049	2.40E-07	8049	2.40E-07	2898	9.70E-07	4990	4.60E-07
NNE	1340	2.00E-06	4829	3.40E-07	8049	1.60E-07	8049	1.60E-07	6600	2.20E-07	8049	1.60E-07
NE	1340	1.80E-06	4829	2.90E-07	8049	1.40E-07	8049	1.40E-07	6600	1.90E-07	8049	1.40E-07
ENE	1340	1.80E-06	4829	2.80E-07	8049	1.30E-07	8049	1.30E-07	8049	1.30E-07	7244	1.60E-07
E	1340	2.40E-06	4829	3.80E-07	8049	1.80E-07	8049	1.80E-07	7727	1.90E-07	6922	2.30E-07
ESE	1340	2.80E-06	4829	4.60E-07	8049	2.30E-07	8049	2.30E-07	8049	2.30E-07	5795	3.60E-07
SE	1340	4.40E-06	4829	7.80E-07	8049	3.90E-07	8049	3.90E-07	5956	5.90E-07	6600	5.10E-07
SSE	1340	2.70E-06	4829	4.70E-07	8049	2.40E-07	8049	2.40E-07	8049	2.40E-07	4185	5.80E-07
S	1340	3.80E-06	4829	6.50E-07	8049	3.20E-07	8049	3.20E-07	6761	4.10E-07	8049	3.20E-07
SSW	1340	3.70E-06	4829	6.20E-07	8049	3.00E-07	8049	3.00E-07	4829	6.20E-07	4507	6.80E-07
SW	1340	8.10E-06	4829	1.40E-06	8049	6.90E-07	8049	6.90E-07	4024	1.80E-06	8049	6.90E-07
WSW	1340	1.80E-05	4829	3.30E-06	8049	1.60E-06	8049	1.60E-06	2737	7.00E-06	8049	1.60E-06
W	1340	1.70E-05	4829	3.00E-06	8049	1.50E-06	8049	1.50E-06	8049	1.50E-06	8049	1.50E-06
WNW	1340	7.30E-06	4829	1.30E-06	8049	6.50E-07	8049	6.50E-07	8049	6.50E-07	8049	6.50E-07
NW	1340	4.60E-06	4829	7.90E-07	8049	3.90E-07	8049	3.90E-07	2576	1.80E-06	3541	1.20E-06
NNW	1340	3.50E-06	4829	6.00E-07	8049	3.00E-07	3863	8.20E-07	3380	9.80E-07	4668	6.30E-07

LNP COL 2.3-5

Table 2.3.5-203 (Sheet 2 of 4)
Long-Term Average Chi/Q Calculations (2.26-Day Decay) for Routine Releases for LNP 1 and LNP 2

	Nearest Residence		Downwind Distance (mi.) (X/Q in sec/m^3)										
Downwind Sector (b)	Distance (m)	X/Q (sec/m^3)	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2898	9.70E-07	2.19E-05	6.60E-06	3.34E-06	2.14E-06	1.24E-06	8.40E-07	6.20E-07	4.84E-07	3.91E-07	3.25E-07	2.76E-07
NNE	6600	2.20E-07	1.51E-05	4.65E-06	2.40E-06	1.55E-06	8.84E-07	5.93E-07	4.35E-07	3.37E-07	2.72E-07	2.25E-07	1.91E-07
NE	8049	1.40E-07	1.29E-05	4.06E-06	2.14E-06	1.39E-06	7.80E-07	5.18E-07	3.77E-07	2.91E-07	2.33E-07	1.93E-07	1.63E-07
ENE	8049	1.30E-07	1.28E-05	4.08E-06	2.13E-06	1.37E-06	7.63E-07	5.04E-07	3.65E-07	2.81E-07	2.25E-07	1.85E-07	1.56E-07
E	7727	1.90E-07	1.74E-05	5.39E-06	2.80E-06	1.80E-06	1.02E-06	6.76E-07	4.93E-07	3.80E-07	3.06E-07	2.53E-07	2.13E-07
ESE	5956	3.50E-07	2.07E-05	6.28E-06	3.24E-06	2.09E-06	1.20E-06	8.12E-07	5.97E-07	4.64E-07	3.74E-07	3.11E-07	2.63E-07
SE	4185	9.40E-07	3.58E-05	1.06E-05	5.24E-06	3.35E-06	1.96E-06	1.34E-06	9.95E-07	7.79E-07	6.32E-07	5.27E-07	4.49E-07
SSE	4668	5.00E-07	2.18E-05	6.48E-06	3.24E-06	2.07E-06	1.21E-06	8.21E-07	6.08E-07	4.75E-07	3.85E-07	3.21E-07	2.73E-07
S	6761	4.10E-07	2.96E-05	8.87E-06	4.45E-06	2.85E-06	1.66E-06	1.12E-06	8.31E-07	6.48E-07	5.25E-07	4.37E-07	3.72E-07
SSW	4507	6.80E-07	2.75E-05	8.48E-06	4.40E-06	2.84E-06	1.63E-06	1.09E-06	8.01E-07	6.21E-07	5.01E-07	4.15E-07	3.52E-07
SW	3220	2.40E-06	6.29E-05	1.89E-05	9.57E-06	6.15E-06	3.57E-06	2.42E-06	1.79E-06	1.40E-06	1.13E-06	9.40E-07	7.98E-07
WSW	2737	7.00E-06	1.51E-04	4.42E-05	2.18E-05	1.39E-05	8.23E-06	5.64E-06	4.20E-06	3.29E-06	2.68E-06	2.23E-06	1.90E-06
W	8049	1.50E-06	1.38E-04	4.02E-05	1.98E-05	1.27E-05	7.48E-06	5.12E-06	3.81E-06	2.99E-06	2.43E-06	2.03E-06	1.73E-06
WNW	8049	6.50E-07	5.95E-05	1.75E-05	8.68E-06	5.54E-06	3.26E-06	2.23E-06	1.66E-06	1.30E-06	1.05E-06	8.80E-07	7.49E-07
NW	2576	1.80E-06	3.57E-05	1.07E-05	5.39E-06	3.46E-06	2.01E-06	1.37E-06	1.01E-06	7.90E-07	6.40E-07	5.32E-07	4.52E-07
NNW	3380	9.80E-07	2.72E-05	8.17E-06	4.14E-06	2.66E-06	1.55E-06	1.05E-06	7.74E-07	6.04E-07	4.89E-07	4.07E-07	3.45E-07

LNP COL 2.3-5

Table 2.3.5-203 (Sheet 3 of 4)
Long-Term Average Chi/Q Calculations (2.26-Day Decay) for Routine Releases for LNP 1 and LNP 2

	Downwind Distance (mi.) (X/Q in sec/m^3)										
Downwind Sector [(b)]	**5.0**	**7.5**	**10.0**	**15.0**	**20.0**	**25.0**	**30.0**	**35.0**	**40.0**	**45.0**	**50.0**
N	2.39E-07	1.34E-07	8.79E-08	4.71E-08	2.94E-08	1.99E-08	1.43E-08	1.06E-08	8.10E-09	6.33E-09	5.04E-09
NNE	1.64E-07	9.16E-08	5.98E-08	3.19E-08	2.00E-08	1.36E-08	9.75E-09	7.28E-09	5.59E-09	4.40E-09	3.52E-09
NE	1.40E-07	7.73E-08	5.03E-08	2.68E-08	1.68E-08	1.15E-08	8.31E-09	6.25E-09	4.85E-09	3.85E-09	3.11E-09
ENE	1.34E-07	7.41E-08	4.83E-08	2.58E-08	1.63E-08	1.12E-08	8.16E-09	6.19E-09	4.83E-09	3.87E-09	3.15E-09
E	1.83E-07	1.02E-07	6.64E-08	3.55E-08	2.23E-08	1.53E-08	1.11E-08	8.33E-09	6.46E-09	5.13E-09	4.15E-09
ESE	2.27E-07	1.27E-07	8.28E-08	4.42E-08	2.75E-08	1.86E-08	1.33E-08	9.90E-09	7.57E-09	5.91E-09	4.70E-09
SE	3.88E-07	2.19E-07	1.44E-07	7.73E-08	4.82E-08	3.26E-08	2.32E-08	1.72E-08	1.30E-08	1.01E-08	7.98E-09
SSE	2.36E-07	1.33E-07	8.74E-08	4.71E-08	2.95E-08	2.01E-08	1.44E-08	1.07E-08	8.21E-09	6.43E-09	5.13E-09
S	3.21E-07	1.81E-07	1.19E-07	6.39E-08	4.00E-08	2.72E-08	1.96E-08	1.46E-08	1.12E-08	8.76E-09	6.99E-09
SSW	3.03E-07	1.69E-07	1.11E-07	5.94E-08	3.72E-08	2.54E-08	1.83E-08	1.37E-08	1.06E-08	8.34E-09	6.71E-09
SW	6.89E-07	3.88E-07	2.54E-07	1.37E-07	8.53E-08	5.79E-08	4.15E-08	3.09E-08	2.36E-08	1.85E-08	1.47E-08
WSW	1.65E-06	9.34E-07	6.16E-07	3.31E-07	2.07E-07	1.40E-07	1.00E-07	7.41E-08	5.64E-08	4.38E-08	3.47E-08
W	1.50E-06	8.49E-07	5.59E-07	3.01E-07	1.88E-07	1.27E-07	9.09E-08	6.73E-08	5.12E-08	3.98E-08	3.15E-08
WNW	6.48E-07	3.67E-07	2.42E-07	1.30E-07	8.11E-08	5.50E-08	3.93E-08	2.91E-08	2.22E-08	1.73E-08	1.37E-08
NW	3.91E-07	2.20E-07	1.44E-07	7.73E-08	4.81E-08	3.26E-08	2.33E-08	1.72E-08	1.31E-08	1.02E-08	8.11E-09
NNW	2.98E-07	1.68E-07	1.10E-07	5.87E-08	3.65E-08	2.47E-08	1.76E-08	1.31E-08	9.95E-09	7.74E-09	6.14E-09

Notes:

Wind Reference Level: 10 m

LNP COL 2.3-5

Table 2.3.5-203 (Sheet 4 of 4)
Long-Term Average Chi/Q Calculations (2.26-Day Decay) for Routine Releases for LNP 1 and LNP 2

Notes (continued):

Stability Type: ΔT (60 – 10 m)
Release Type: Ground Level:10 m
Building Height/Cross Section: 43.9 m/2730 m^2

a) The reported distance of the low population zone (LPZ) is measured from the centerpoint of LNP 1 and LNP 2 to the outermost boundary of the LPZ.

b) Downwind Sector: E = east; N = north; S = south; W = west

X/Q = local atmospheric dilution factor
m = meter
m^2 = square meter
mi. = mile
sec/m^3 = seconds per cubic meter

LNP COL 2.3-5

Table 2.3.5-204 (Sheet 1 of 4)
Long-Term Average Chi/Q Calculations (Depleted and 8-Day Decayed) for Routine Releases for LNP 1 and LNP 2

	Exclusion Area Boundary		Low Population Zone [(a)]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector [(b)]	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (sec/m^3)	Distance (m)	X/Q (ec/m^3)
N	1340	2.50E-06	4829	4.00E-07	8049	1.90E-07	8049	1.90E-07	2898	8.30E-07	4990	3.80E-07
NNE	1340	1.80E-06	4829	2.80E-07	8049	1.30E-07	8049	1.30E-07	6600	1.80E-07	8049	1.30E-07
NE	1340	1.60E-06	4829	2.40E-07	8049	1.10E-07	8049	1.10E-07	6600	1.50E-07	8049	1.10E-07
ENE	1340	1.60E-06	4829	2.30E-07	8049	1.10E-07	8049	1.10E-07	8049	1.10E-07	7244	1.30E-07
E	1340	2.10E-06	4829	3.10E-07	8049	1.50E-07	8049	1.50E-07	7727	1.60E-07	6922	1.80E-07
ESE	1340	2.50E-06	4829	3.90E-07	8049	1.80E-07	8049	1.80E-07	8049	1.80E-07	5795	3.00E-07
SE	1340	4.00E-06	4829	6.50E-07	8049	3.20E-07	8049	3.20E-07	5956	4.80E-07	6600	4.20E-07
SSE	1340	2.50E-06	4829	4.00E-07	8049	1.90E-07	8049	1.90E-07	8049	1.90E-07	4185	4.90E-07
S	1340	3.40E-06	4829	5.40E-07	8049	2.60E-07	8049	2.60E-07	6761	3.30E-07	8049	2.60E-07
SSW	1340	3.30E-06	4829	5.20E-07	8049	2.40E-07	8049	2.40E-07	4829	5.20E-07	4507	5.70E-07
SW	1340	7.30E-06	4829	1.20E-06	8049	5.60E-07	8049	5.60E-07	4024	1.50E-06	8049	5.60E-07
WSW	1340	1.70E-05	4829	2.80E-06	8049	1.30E-06	8049	1.30E-06	2737	6.00E-06	8049	1.30E-06
W	1340	1.50E-05	4829	2.50E-06	8049	1.20E-06	8049	1.20E-06	8049	1.20E-06	8049	1.20E-06
WNW	1340	6.60E-06	4829	1.10E-06	8049	5.30E-07	8049	5.30E-07	8049	5.30E-07	8049	5.30E-07
NW	1340	4.10E-06	4829	6.60E-07	8049	3.20E-07	8049	3.20E-07	2576	1.60E-06	3541	1.00E-06
NNW	1340	3.10E-06	4829	5.00E-07	8049	2.40E-07	3863	6.90E-07	3380	8.40E-07	4668	5.30E-07

LNP COL 2.3-5

**Table 2.3.5-204 (Sheet 2 of 4)
Long-Term Average Chi/Q Calculations (Depleted and 8-Day Decayed) for Routine Releases for LNP 1 and LNP 2**

	Nearest Residence		Downwind Distance (mi.) (X/Q in sec/m^3)										
Downwind Sector [(b)]	Distance (m)	X/Q (sec/m^3)	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2898	8.30E-07	2.08E-05	6.08E-06	3.01E-06	1.90E-06	1.08E-06	7.19E-07	5.24E-07	4.04E-07	3.24E-07	2.67E-07	2.25E-07
NNE	6600	1.80E-07	1.44E-05	4.28E-06	2.16E-06	1.37E-06	7.68E-07	5.06E-07	3.66E-07	2.81E-07	2.24E-07	1.84E-07	1.55E-07
NE	8049	1.10E-07	1.23E-05	3.73E-06	1.93E-06	1.23E-06	6.76E-07	4.41E-07	3.16E-07	2.41E-07	1.91E-07	1.56E-07	1.31E-07
ENE	8049	1.10E-07	1.22E-05	3.75E-06	1.92E-06	1.21E-06	6.59E-07	4.27E-07	3.05E-07	2.32E-07	1.83E-07	1.50E-07	1.25E-07
E	7727	1.60E-07	1.65E-05	4.96E-06	2.52E-06	1.60E-06	8.81E-07	5.75E-07	4.13E-07	3.15E-07	2.50E-07	2.05E-07	1.72E-07
ESE	5956	2.90E-07	1.96E-05	5.78E-06	2.92E-06	1.86E-06	1.05E-06	6.94E-07	5.04E-07	3.87E-07	3.09E-07	2.55E-07	2.14E-07
SE	4185	8.00E-07	3.41E-05	9.76E-06	4.73E-06	2.98E-06	1.71E-06	1.15E-06	8.42E-07	6.52E-07	5.25E-07	4.34E-07	3.67E-07
SSE	4668	4.20E-07	2.07E-05	5.97E-06	2.92E-06	1.84E-06	1.05E-06	7.03E-07	5.13E-07	3.96E-07	3.18E-07	2.63E-07	2.22E-07
S	6761	3.30E-07	2.81E-05	8.17E-06	4.01E-06	2.53E-06	1.44E-06	9.61E-07	7.01E-07	5.41E-07	4.34E-07	3.58E-07	3.02E-07
SSW	4507	5.70E-07	2.61E-05	7.80E-06	3.96E-06	2.52E-06	1.41E-06	9.31E-07	6.73E-07	5.16E-07	4.12E-07	3.39E-07	2.85E-07
SW	3220	2.10E-06	5.98E-05	1.74E-05	8.63E-06	5.47E-06	3.11E-06	2.07E-06	1.51E-06	1.16E-06	9.33E-07	7.70E-07	6.50E-07
WSW	2737	6.00E-06	1.44E-04	4.07E-05	1.97E-05	1.24E-05	7.17E-06	4.84E-06	3.55E-06	2.76E-06	2.22E-06	1.84E-06	1.56E-06
W	8049	1.20E-06	1.31E-04	3.71E-05	1.79E-05	1.13E-05	6.52E-06	4.39E-06	3.23E-06	2.50E-06	2.02E-06	1.67E-06	1.42E-06
WNW	8049	5.30E-07	5.66E-05	1.62E-05	7.83E-06	4.94E-06	2.84E-06	1.91E-06	1.40E-06	1.09E-06	8.74E-07	7.24E-07	6.12E-07
NW	2576	1.60E-06	3.39E-05	9.85E-06	4.86E-06	3.08E-06	1.75E-06	1.17E-06	8.55E-07	6.60E-07	5.30E-07	4.37E-07	3.69E-07
NNW	3380	8.40E-07	2.59E-05	7.53E-06	3.73E-06	2.37E-06	1.35E-06	8.98E-07	6.55E-07	5.05E-07	4.05E-07	3.34E-07	2.82E-07

LNP COL 2.3-5

Table 2.3.5-204 (Sheet 3 of 4)
Long-Term Average Chi/Q Calculations (Depleted and 8-Day Decayed) for Routine Releases for LNP 1 and LNP 2

Downwind Sector (b)	Downwind Distance (mi.) (X/Q in sec/m^3) 5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	1.93E-07	1.07E-07	6.98E-08	3.76E-08	2.39E-08	1.66E-08	1.22E-08	9.38E-09	7.41E-09	5.98E-09	4.91E-09
NNE	1.32E-07	7.26E-08	4.70E-08	2.51E-08	1.59E-08	1.10E-08	8.09E-09	6.19E-09	4.88E-09	3.94E-09	3.23E-09
NE	1.12E-07	6.06E-08	3.90E-08	2.06E-08	1.30E-08	8.97E-09	6.58E-09	5.03E-09	3.97E-09	3.20E-09	2.63E-09
ENE	1.07E-07	5.76E-08	3.69E-08	1.95E-08	1.23E-08	8.49E-09	6.23E-09	4.77E-09	3.77E-09	3.04E-09	2.50E-09
E	1.47E-07	7.99E-08	5.15E-08	2.74E-08	1.73E-08	1.20E-08	8.81E-09	6.75E-09	5.32E-09	4.30E-09	3.53E-09
ESE	1.84E-07	1.01E-07	6.56E-08	3.52E-08	2.23E-08	1.55E-08	1.14E-08	8.69E-09	6.85E-09	5.53E-09	4.53E-09
SE	3.16E-07	1.77E-07	1.16E-07	6.27E-08	4.00E-08	2.79E-08	2.06E-08	1.58E-08	1.25E-08	1.01E-08	8.26E-09
SSE	1.91E-07	1.06E-07	6.94E-08	3.75E-08	2.39E-08	1.67E-08	1.23E-08	9.46E-09	7.48E-09	6.04E-09	4.97E-09
S	2.60E-07	1.44E-07	9.42E-08	5.08E-08	3.24E-08	2.26E-08	1.66E-08	1.28E-08	1.01E-08	8.16E-09	6.71E-09
SSW	2.44E-07	1.34E-07	8.67E-08	4.63E-08	2.93E-08	2.04E-08	1.50E-08	1.15E-08	9.05E-09	7.31E-09	6.00E-09
SW	5.58E-07	3.10E-07	2.02E-07	1.09E-07	6.91E-08	4.81E-08	3.55E-08	2.72E-08	2.15E-08	1.73E-08	1.42E-08
WSW	1.34E-06	7.52E-07	4.94E-07	2.68E-07	1.71E-07	1.20E-07	8.84E-08	6.79E-08	5.37E-08	4.34E-08	3.57E-08
W	1.22E-06	6.83E-07	4.48E-07	2.44E-07	1.56E-07	1.09E-07	8.03E-08	6.17E-08	4.88E-08	3.94E-08	3.24E-08
WNW	5.27E-07	2.95E-07	1.93E-07	1.05E-07	6.69E-08	4.67E-08	3.45E-08	2.65E-08	2.09E-08	1.69E-08	1.39E-08
NW	3.17E-07	1.76E-07	1.15E-07	6.21E-08	3.95E-08	2.75E-08	2.03E-08	1.55E-08	1.23E-08	9.89E-09	8.12E-09
NNW	2.42E-07	1.34E-07	8.76E-08	4.72E-08	3.00E-08	2.09E-08	1.54E-08	1.18E-08	9.28E-09	7.49E-09	6.15E-09

Notes:

Wind Reference Level: 10 m

LNP COL 2.3-5

Table 2.3.5-204 (Sheet 4 of 4)
Long-Term Average Chi/Q Calculations (Depleted and 8-Day Decayed) for Routine Releases for LNP 1 and LNP 2

Notes (continued):

Stability Type: ΔT (60 – 10 m)
Release Type: Ground Level: 10 m
Building Height/Cross Section: 43.9 m/2730 m^2

a) The reported distance of the low population zone (LPZ) is measured from the centerpoint of LNP 1 and LNP 2 to the outermost boundary of the LPZ.

b) Downwind Sector: E = east; N = north; S = south; W = west

X/Q = local atmospheric dilution factor
m = meter
m^2 = square meter
mi. = mile
sec/m^3 = seconds per cubic meter

2.4 HYDROLOGIC ENGINEERING

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

STD DEP 1.1-1 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.

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

LNP SUP 2.4-1 The Levy Nuclear Plant (LNP) site is located in southern Levy County, Florida. The site covers an area of approximately 1257 hectares (ha) (3105 acres [ac.]) in a primarily rural area southwest of Gainesville and west of Ocala (Figure 2.4.1-201). The site is located, approximately 12.8 kilometers (km) (7.9 miles [mi.]) east of the Gulf of Mexico, approximately 4.8 km (3 mi.) north of Lake Rousseau, and approximately 15.5 km (9.6 mi.) north of the Crystal River Energy Complex, an energy facility owned by Florida Power Corporation doing business as Progress Energy Florida, Inc. (PEF) (Figure 2.4.1-202). (Reference 2.4.1-201) The LNP site was purchased by PEF from Rayonier, Inc., a timber company based in Jacksonville, Florida (Reference 2.4.1-202). PEF has selected Westinghouse's AP1000 Reactor (AP1000) as the certified plant design for the LNP site. The Westinghouse AP1000 units are referred to as the Levy Nuclear Plant Unit 1 (LNP 1) and the Levy Nuclear Plant Unit 2 (LNP 2) (Figure 2.4.1-202).

The elevation of the LNP site varies between approximately 9.1 m (30 ft.) and 18.3 m (60 ft.) National Geodetic Vertical Datum of 1929 (NGVD29) (Figure 2.4.1-203). The pre-construction grade elevation of the LNP site within the limits of the site grading plan ranges from 12.5 meters (m) (41 feet [ft.]) North American Vertical Datum of 1988 (NAVD88) in the southwest and west portions of the site to 14.9 m (49 ft.) NAVD88 in the northeast portion of the site, as shown on Figure 2.4.1-204. The nominal plant grade elevation for the footprint of LNP 1 and LNP 2 is 15.2 m (50 ft.) NAVD88. As shown on Figure 2.4.1-205, the actual plant grade is lower and varies to accommodate site grading, drainage, and local site flooding requirements. The nominal plant grade floor elevation for the LNP site is 15.5 m (51 ft.) NAVD88.

Major hydrologic features near the LNP site include the Gulf of Mexico, Lake Rousseau, the Withlacoochee River, and the Cross Florida Barge Canal (CFBC) (Figures 2.4.1-206 and 2.4.1-207). The CFBC was constructed as part of the decommissioned CFBC project. The project was intended to connect the Atlantic Ocean and Gulf of Mexico across Florida for barge traffic. Approximately 5.2 km (3.2 mi.) south of the LNP site, a 13.4 km (8.3 mi.) section of the unfinished

CFBC connects Lake Rousseau to the Gulf of Mexico, bifurcating the Withlacoochee River downstream of Lake Rousseau (Figure 2.4.1-206). There are three water control structures in this area: the Inglis Lock, Inglis Bypass Channel Spillway, and Inglis Dam (Figure 2.4.1-208). These structures are operated by the South West Florida Water Management District (SWFWMD) and were constructed as part of the decommissioned CFBC project.

LNP COL 2.4-3

Brackish water from the CFBC will be used to supply approximately 320,927 liters per minute (lpm) (84,780 gallons per minute [gpm]) of cooling water to LNP 1 and LNP 2. The brackish water from the CFBC will be pumped north to the LNP site from an intake structure located approximately 11.1 km (6.9 mi.) from the Gulf of Mexico on the berm that forms the north side of the canal and within 0.8 km (0.5 mi.) of the Inglis Lock as shown on Figure 2.4.1-202. The cooling water intake structure consists of the intake structure, vertical bar screens, traveling screens, pumps, and pumphouse. The elevation of the pumphouse structure deck is 11.3 m (37 ft.) NAVD88. The elevation of the pump intakes is -3.2 m (-10.6 ft.) NAVD88. Under conditions of CFBC failure, LNP 1 and LNP 2 will use a passive core cooling system to provide emergency core cooling without the use of active equipment such as pumps and alternating current (ac) power sources.

Cooling tower blowdown from LNP 1 and LNP 2 will be returned to the Gulf of Mexico via two pipelines (one for each unit) that run south approximately 6 km (3.7 mi.) to the CFBC as shown on Figure 2.4.1-202. The blowdown pipelines will run approximately 8.7 km (5.4 mi.) along the northern edge of the CFBC and cross the canal north of the Crystal River Energy Complex. The pipelines will then run south approximately 5.7 km (3.5 mi.) and discharge into the existing Crystal River Energy Complex Discharge Canal.

There is no discharge of water from the cooling towers to subbasins associated with the LNP site. All cooling water will directly discharge to the Gulf of Mexico through the Crystal River Energy Complex Discharge Canal. The Withlacoochee River will not be influenced by the project.

Groundwater from off-site wells will be used to supply general plant operation including service water tower drift and evaporation, potable water supply, raw water to demineralizer, fire protection, and media filter backwash. An estimated average of 3336.8 lpm (881.5 gpm) and a maximum of approximately 15,374.1 lpm (4061.4 gpm) of groundwater will be used for these purposes.

Figures 2.4.1-204 and 2.4.1-205 provide topographic maps of the site with existing conditions and proposed changes to the natural drainage features. The locations of LNP 1 and LNP 2 are in the central portion of the plant site at a pre-construction grade elevation of approximately 12.8 m (42 ft.) NAVD88. The locations of LNP 1 and LNP 2 include natural, poorly drained swamp and marshland. Surface water that does runoff from the plant site generally flows southwest toward the CFBC, Lower Withlacoochee River, and the Gulf of Mexico. The pre-construction grade at the location of LNP 1 and LNP 2 will be filled to a nominal grade of 15.2 m (50 ft.) NAVD88, affecting the current drainage pattern. The LNP site will drain by a stormwater sewer system and the peripheral areas of the LNP site will drain through open ditches and culverts to stormwater retention ponds. Stormwater from the retention ponds will be pumped to the cooling water tower basins, if needed. If the drainage system becomes blocked, the LNP site can be drained by overland flow directly to the Lower Withlacoochee River or the Gulf of Mexico.

Seismic Category I structures that should be considered from the hydrologic standpoint include safety-related structures such as the following nuclear island structures: basemat, the containment interior, the shield building, the containment air baffle, and the auxiliary building. Section 3.2 of this Final Safety Analysis Report (FSAR) provides details related to these structures.

Wherever possible, elevations presented in this section are presented with a consistent vertical datum of NAVD88. Where elevation information is not available with a vertical datum of NAVD88, elevation information is presented with a vertical datum of NGVD29. For the LNP site, there is an approximate -0.3 m (-1 ft.) difference between elevations measured with these datums. At the LNP site, elevations measured with a NAVD88 datum are lower than those measured with a NGVD29 datum; therefore, to convert an elevation given with a NGVD29 datum to an elevation with a NAVD88 datum, add the conversion factor to the NGVD29 elevation. Specific conversions are sometimes given at known points.

LNP COL 2.4-1

2.4.1.2 Hydrosphere

2.4.1.2.1 Levy Nuclear Plant Site

The majority of the LNP site lies within the Waccasassa River Drainage Basin, but a small portion of the site lies in the Withlacoochee River Drainage Basin (Figures 2.4.1-209 and 2.4.1-210). The northern portion of the LNP site lies within the Spring Run Subbasin of the Waccasassa River Basin. The central portion of the LNP site, which includes LNP 1 and LNP 2, lies within the Direct Runoff to Gulf Subbasin of the Waccasassa River Basin. The southeastern corner of the LNP site lies within the Withlacoochee River Basin. In addition, Lake Rousseau and the CFBC, along with the Withlacoochee River, lie within the Withlacoochee River Basin.

There are no named streams at the LNP site. Runoff from the site is primarily overland, with storage provided by wetlands. The general direction of overland flow is to the southwest toward the Lower Withlacoochee River and the Gulf of Mexico (Reference 2.4.1-203). Major freshwater bodies in the vicinity of the LNP site include the Withlacoochee River and Lake Rousseau. Lake Rousseau is located approximately 4.8 km (3 mi.) south of the LNP site. The Withlacoochee River and the Rainbow River are the primary sources of water to Lake Rousseau. The CFBC contains mostly saline water from the Gulf of Mexico. The Gulf of Mexico is located approximately 12.8 km (7.9 mi.) west of the LNP site.

The LNP site consists primarily of wetlands (Reference 2.4.1-204). Extensive salt marsh communities are found between U.S. Highway 19, the highway to the west of the site, and the open waters of the Gulf of Mexico. While some areas have been disturbed, the overall quality of coastal ecological systems in this area is high.

2.4.1.2.2 Withlacoochee River Basin

The Withlacoochee River Drainage Basin is located in the northern part of the SWFWMD and includes approximately 2100 square kilometers (km^2) (5439 square miles [$mi.^2$]) (Reference 2.4.1-205). Several lakes and ponds are present, primarily in the central portion of the basin (Figure 2.4.1-209). The Withlacoochee River and its water control structures are discussed in detail in subsequent sections.

2.4.1.2.3 Withlacoochee River

The Withlacoochee River flows north and west through eight counties and is approximately 252.7 km (157 mi.) long. The Withlacoochee River has its headwaters in Green Swamp and discharges to the Gulf of Mexico at Withlacoochee Bay Estuary near Yankeetown, Florida. (Reference 2.4.1-205) The average gradient of the river is 0.17 meter per kilometer (m/km) (0.9 foot per mile [ft/mi]). Major tributaries of the Withlacoochee River include Little Withlacoochee River, Big Grant Canal, Jumper Creek, Shady Brook, Outlet River of Lake Panasoffkee, Leslie Heifner Canal, Orange State Canal, Tsala Apopka Outfall Canal, and Rainbow River. (Reference 2.4.1-206) The Withlacoochee River and the Rainbow River are the primary sources of water to Lake Rousseau (Reference 2.4.1-205).

The Withlacoochee River is divided into three segments: upper, middle, and lower. The upper Withlacoochee River consists of the portion of the river from its confluence with the Little Withlacoochee River to its headwaters in Green Swamp. The middle Withlacoochee River consists of the portion of the river between U.S. Highway 41, which intersects the Withlacoochee River approximately 0.9 km (0.6 mi.) east of Lake Rousseau, and its confluence with the Little Withlacoochee River. The Lower Withlacoochee River consists of the portion of the river between U.S. Highway 41 and its discharge point in the Gulf of Mexico. (Reference 2.4.1-207) The LNP site is located 5.4 km (3.4 mi.) north of the Lower Withlacoochee River.

The Lower Withlacoochee River includes a portion of the CFBC, Lake Rousseau, and several water control structures: Inglis Bypass Channel Spillway, Inglis Dam, and Inglis Lock (Reference 2.4.1-207). The construction of the CFBC and the water control structures has altered the hydrology of the Lower Withlacoochee River. For example, the CFBC is sometimes used as a flood relief channel during high flow conditions, thereby reducing long-term average flows in the Lower Withlacoochee River (Reference 2.4.1-205). The CFBC, Lake Rousseau, and the associated water control structures are discussed in further detail in the following subsections.

Six U.S. Geological Survey (USGS) gauging stations are located near the LNP site. Five of these USGS stations are located on the Lower Withlacoochee River and one is located on the Rainbow River, a tributary to the Withlacoochee River (Figure 2.4.1-211):

- **Withlacoochee River at Dunnellon, Florida** (USGS ID: 02313200, #20 on Figure 2.4.1-211): This station is located on the Withlacoochee River, 1.3 km (0.8 mi.) upstream of Lake Rousseau. Gage height data from February 6, 1963, through October 7, 2007, are available for this station. Discharge data are not available for this station. The drainage area of this station is 5076 km^2 (1960 $mi.^2$). (Reference 2.4.1-208)

- **Withlacoochee River at Inglis Dam near Dunnellon, Florida** (USGS ID: 02313230, #21 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the upstream side of the Inglis Dam. Gage height data from October 1, 1985, through October 7, 2007, and discharge data from October 1, 1969, through September 10, 2007, are available for this station. The drainage area of this station is 5232 km^2 (2020 $mi.^2$). (Reference 2.4.1-209)

- **Withlacoochee River below Inglis Dam near Dunnellon, Florida** (USGS ID: 02313231, #22 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the downstream side of the Inglis Dam. Gage height data from October 1, 1969, through October 7, 2007, are available for this station. Discharge data are not available for this station. The drainage area of this station is undetermined. (Reference 2.4.1-210)

- **Withlacoochee River Bypass Channel near Dunnellon, Florida** (USGS ID: 02313250, #23 on Figure 2.4.1-211): This station is located 2.1 km (1.3 mi.) upstream of the bypass spillway. Gage height data from July 16, 1971, through October 7, 2007, and discharge data from January 1, 1970, through October 7, 2007, are available for this station. The drainage area of this station is undetermined. (Reference 2.4.1-211)

- **Withlacoochee River at Chambers near Yankeetown, Florida** (USGS ID: 02313272, #24 on Figure 2.4.1-211): This station is located 17.7 km (11 mi.) downstream of the Inglis Dam at the mouth of Gulf of Mexico. Tidal high and tidal low daily gage height data are only available from January 28, 2005, to July 23, 2007, at this station. Discharge data are not available for this station. The drainage area of this station is undetermined. (Reference 2.4.1-212)

- **Rainbow Springs near Dunnellon, Florida** (USGS ID: 02313100, #19 on Figure 2.4.1-211): This station is located at the head of Rainbow Springs, 9.2 km (5.7 mi.) upstream of the confluence of the Rainbow and

Withlacoochee rivers. Discharge data from January 1, 1965, through August 7, 2007, are available at this station. The drainage area of this station is undetermined. Gage height data are not available at this station. (Reference 2.4.1-213)

A summary of data available at the USGS stations listed above is provided in Table 2.4.1-201.

Table 2.4.1-202 presents the average monthly discharge data for the Withlacoochee River at USGS station 02313250 (Withlacoochee River Bypass Channel near Dunnellon, Florida). The average monthly discharge of the Withlacoochee River Bypass Channel is 29.7 cubic meters per second (m^3/s) (1049 cubic feet per second [cfs]). The highest average monthly discharge of 31.4 m^3/s (1110 cfs) occurs during September and the lowest average monthly discharge of 26.9 m^3/s (949 cfs) occurs during June. (Reference 2.4.1-214) The maximum daily streamflow of 52.1 m^3/s (1840 cfs) occurred on October 1, 1987 (Table 2.4.1-203) (Reference 2.4.1-215). Discharge data at USGS station 02313231, located at Inglis Dam, are discussed in FSAR Subsection 2.4.1.2.6.

2.4.1.2.4 Rainbow River

The Rainbow River and the Withlacoochee River are the major surface water contributors to Lake Rousseau (Reference 2.4.1-205). The Rainbow River is 9.2 km (5.7 mi.) long and merges with the Withlacoochee River at Dunnellon, Florida. The primary source of water for the Rainbow River is Rainbow Spring, which is a natural spring of first order magnitude (Figures 2.4.1-206 and 2.4.1-207) (Reference 2.4.1-216). Rainbow River discharges an average of 20.6 m^3/s (727 cfs) per day of water to the Withlacoochee River (Reference 2.4.1-205).

As discussed in FSAR Subsection 2.4.1.2.3, a USGS gauging station (USGS ID: 02313100, Rainbow Springs near Dunnellon, Florida) is located at the head of the springs, 9.2 km (5.7 mi.) upstream of the confluence of the Rainbow and Withlacoochee rivers (Reference 2.4.1-213). Table 2.4.1-204 presents the average monthly discharge for this station. The average monthly discharge of the spring is 19.8 m^3/s (698 cfs). The highest average monthly discharge of 21.2 m^3/s (748 cfs) occurs during October and the lowest average monthly discharge of 18.8 m^3/s (663 cfs) occurs during June. (Reference 2.4.1-217) The maximum daily streamflow of 30 m^3/s (1060 cfs) was recorded on September 19, 1988 (Table 2.4.1-205) (Reference 2.4.1-218).

2.4.1.2.5 Cross Florida Barge Canal

The CFBC was constructed in the 1960s as part of a federal project to create a northern inland water route between the Gulf of Mexico and northeast Florida (Reference 2.4.1-219). The canal was designed to have a depth of 3.7 m (12 ft.) and minimum bottom width of 45.7 m (150 ft.), and five locks. Total length of the project was about 172.2 km (107 mi.). (Reference 2.4.1-220) Construction of the CFBC was stopped in 1971 due to adverse environmental and economic impact on Florida. It is now a protected green belt corridor known as Marjorie Harris Carr Cross Florida Greenway (Reference 2.4.1-221).

While the project was abandoned, the initial construction included a lock structure and a straight canal between Lake Rousseau and the Gulf of Mexico. This canal bisected the Withlacoochee River, severing the hydraulic connection between Inglis Dam and the downstream river. To maintain flow to the Lower Withlacoochee River, a bypass channel with a control structure was built adjacent to and just downstream of the locks. The existing flow path occurs from Lake Rousseau, through the Inglis Bypass Channel and associated gated spillway, and into the Lower Withlacoochee River (Reference 2.4.1-222). There is a large embankment that separates the CFBC from the Lower Withlacoochee River. Flow is only released from the dam into the CFBC during extreme flooding (Reference 2.4.1-223).

2.4.1.2.6 Lake Rousseau

Lake Rousseau is a 16.8-km^2 (4163-ac., 6.5-mi.^{2}) impoundment on the Withlacoochee River formed by the Inglis Dam (References 2.4.1-205 and 2.4.1-224). Lake Rousseau is located approximately 17.7 km (11 mi.) upstream of the mouth of the Withlacoochee River near the city of Inglis. Lake Rousseau was constructed in 1909 by Florida Power Corporation for electric power generation. Lake Rousseau is approximately 9.2 km (5.7 mi.) long (Reference 2.4.1-205).

The Withlacoochee and Rainbow rivers are the major surface water contributors to Lake Rousseau (Reference 2.4.1-224). During dry periods, flows into Lake Rousseau are dominated by the Rainbow River and other spring-fed tributaries to the Withlacoochee River. West of Lake Rousseau, the Withlacoochee River flows to the Gulf of Mexico where it discharges into the Withlacoochee Bay Estuary. (Reference 2.4.1-205)

The pool elevation at Lake Rousseau is controlled by three structures: the Inglis Bypass Channel and associated spillway, the Inglis Dam, and the Inglis Lock. (Figure 2.4.1-208).The majority of the normal discharge (up to 43.6 m^3/s [1540 cfs]) passes through the bypass channel and spillway to the Lower Withlacoochee River. The Inglis Dam passes flows in excess of the bypass channel and spillway to the CFBC via a short section of the natural Withlacoochee River (approximately 2743.2 m (9000 ft.). (Reference 2.4.1-205)

The operating pool elevation at Lake Rousseau is maintained between 7.3 and 8.5 m (24.0 and 28.0 ft.) NGVD29 (Reference 2.4.1-225). Prior to heavy rainfall the pool elevation may be lowered up to 0.15 m (0.5 ft.) depending upon the reservoir conditions and river flow (Reference 2.4.1-223). The pool elevation is maintained at the optimum level of 8.4 m (27.5 ft.) NGVD29. Due to lack of storage capacity within the lake, heavy rainfall can drastically affect the stage within a short period of time.

As stated in FSAR Subsection 2.4.1.2.3, a USGS gauging station (USGS ID: 02313230, Withlacoochee River at Inglis Dam near Dunnellon, Florida) is located on the upstream side of the Inglis Dam (Figures 2.4.1-208 and 2.4.1-211). Discharge data available at this location includes approximately 2 m^3/s (70 cfs) of flow from a spring downstream of the spillway of the Inglis Dam. This spring flow is considered to be leakage from Lake Rousseau (Reference 2.4.1-209). Table 2.4.1-206 presents the average monthly discharge (October 1969 through April 2005) at this station. The average monthly discharge of Lake Rousseau into the CFBC via the original run of the Lower Withlacoochee River below Inglis Dam is 12.5 m^3/s (443 cfs). The highest average monthly discharge of 23 m^3/s (816 cfs) occurs in October and the lowest average monthly discharge of 5.0 m^3/s (178 cfs) occurs in June. (Reference 2.4.1-226) The maximum daily streamflow of 170 m^3/s (6030 cfs) occurred on October 19, 2004 (Table 2.4.1-207) (Reference 2.4.1-227).

As stated in FSAR Subsection 2.4.1.2.3, a USGS gauging station (USGS ID: 02313200, Withlacoochee River at Dunnellon, Florida) is located 1.3 km (0.8 mi.) upstream of Lake Rousseau at the junction of the Withlacoochee and Rainbow rivers (Figure 2.4.1-211). Stage at this station is regulated by Lake Rousseau. (Reference 2.4.1-208) Average gauge height at this station for 2002 and 2004 through 2006 is 8.6 m (28.07 ft.) NGVD29. Discharge data is not available at this station. (Reference 2.4.1-228)

2.4.1.2.7 Lower Withlacoochee River – Water Control Structures

There are three water control structures on the Withlacoochee River within the vicinity of the LNP site: 1) Inglis Bypass Channel Spillway, 2) Inglis Lock, and 3) Inglis Dam. Figure 2.4.1-208 shows the location of these structures. These water control structures were part of the unfinished CFBC project constructed by the U.S. Army Corps of Engineers (USACE) in the 1960s (Reference 2.4.1-229).

Inglis Lock is located on the CFBC between Lake Rousseau and the Gulf of Mexico (Figure 2.4.1-208) (Reference 2.4.1-229). The lock functions as a navigational facility to raise and lower vessels traveling between Lake Rousseau and Gulf of Mexico (Reference 2.4.1-225). The lock is 182.9 m (600 ft.) long by 25.6 m (84 ft.) wide and releases 43.2 million liters (11.4 million gallons) of freshwater from Lake Rousseau into the Gulf of Mexico each time it is used. However, this lock has not been used since 1999 because the upstream gate is in need of repair. Currently, there are no plans to repair this structure. (Reference 2.4.1-229)

The Inglis Bypass Channel and associated spillway are located just north of the Inglis Lock in Levy County (Figure 2.4.1-208). These structures discharge freshwater from Lake Rousseau to the Lower Withlacoochee River to sustain the prevailing environment, prevent saltwater intrusion, maintain the optimum pool level of the lake, and to accommodate navigation interests in the river. The maximum capacity of the spillway is 43.6 m^3/s (1540 cfs). The spillway is a reinforced concrete, U-shaped, two-gate spillway with an ogee weir and a baffled stilling basin. The crest elevation of the spillway is 8.5 m (28.0 ft.) NGVD29. Two hydraulically operated vertical lift gates (0.6 m x 4.3 m x 2.1 m [2 ft. x 14 ft. x 7 ft.]) are fitted to the structure to regulate the outflows. The structure is provided with an operating platform to accommodate the gate operating equipment and a service bridge that crosses the structure at an elevation of 9.1 m (30.0 ft.) NGVD29. (Reference 2.4.1-222) To convert to NAVD88 at this location, add a conversion quantity of -0.305 m (-1.00 ft.) to NGVD29 elevations (Reference 2.4.1-202).

During high inflow conditions, when the operating capacity of the spillway is exceeded, the Inglis Dam is used to control the elevation of Lake Rousseau. Inglis Dam is located at the west end of Lake Rousseau, south of the Inglis Lock and Inglis Bypass Channel Spillway, in Citrus County (Figure 2.4.1-208). The dam has a reinforced concrete, U-shaped, two-bay, gated spillway with an ogee-type weir. The crest elevation of the spillway is 8.5 m (28.0 ft.) NGVD29. Each bay has a 12.2-m- (40-ft.-) wide by 5.1-m- (16.7-ft.-) high vertical lift gate, installed on the crest of the weir. The gate operating equipment is mounted on a reinforced concrete platform at an elevation of 15.8 m (52.0 ft.) NGVD29. The structure is configured with a reinforced concrete service bridge at an elevation of 10.1 m (33.0 ft.) NGVD29. The maximum allowable headwater elevation at the dam is 8.5 m (28 ft.) NGVD29. (Reference 2.4.1-223) To convert to NAVD88 at this location, add a conversion quantity of -0.315 m (-1.03 ft.) to NGVD29 elevations (Reference 2.4.1-202).

The Inglis Dam and Inglis Bypass Channel Spillway are the main flood control structures for the Lower Withlacoochee River (Reference 2.4.1-225).

2.4.1.2.8 Other Water Control Structures

Several other water control structures are present in the Withlacoochee River Basin including Lake Tsala Apopka Dam, Slush Pond Dam, and Gant Lake Dam (Figure 2.4.1-212). None of these structures directly affect the water elevation at Lake Rousseau or the LNP site so they are not discussed in detail in this report.

2.4.1.2.9 Waccasassa River Drainage Basin

The Waccasassa River Drainage Basin is located in the southern part of the Suwannee River Water Management District (SRWMD) and includes approximately 2334 km^2 (901 $mi.^2$) (Figure 2.4.1-209) (Reference 2.4.1-230). The Waccasassa River Drainage Basin is relatively undeveloped. The basin consists of more than 55 percent forested areas, 18 percent wetlands, and 15 percent agricultural lands. (Reference 2.4.1-230) Named drainage features in the basin include the Waccasassa River, Jakes Creek, Kelly Creek, Otter Creek, Magee Branch, Wekiva Creek, Cow Creek, Ten Mile Creek, and Spring Run (Figure 2.4.1-209). Several ponds and lakes are present in the basin, primarily north of the LNP site (Figure 2.4.1-209). The basin generally slopes and drains to the southwest, toward the Gulf of Mexico (Reference 2.4.1-203). There are no known water control structures in this basin (Figure 2.4.1-212).

2.4.1.2.10 Surface Water Users

As discussed in FSAR Subsection 2.4.1.1, groundwater from off-site wells will be used for general operational purposes, such as potable water supply and fire protection. Only cooling tower makeup water at LNP 1 and LNP 2 will be withdrawn from the CFBC. LNP 1 and LNP 2 require approximately 320,927 lpm (84,780 gpm) of surface water from the CFBC for cooling tower makeup water.

There are no known communities that draw water from the Withlacoochee River or Lake Rousseau for public water supply. The primary source of water for public supply near the LNP site is groundwater (Reference 2.4.1-231). Table 2.4.1-208 summarizes the sources of public water supply for counties surrounding the LNP site. Figure 2.4.1-213 presents the map showing the counties within 16.1 km (10 mi.), 40.2 km (25 mi.), and 80.5 km (50 mi.) of the LNP site.

Counties within a 16.1-km (10-mi.) radius of LNP site include Citrus, Levy, and Marion (Figure 2.4.1-213). There are no surface water withdrawals for public, domestic, or industrial water supply in these counties (Table 2.4.1-208). Surface water withdrawals within 16.1 km (10 mi.) of the LNP site include the following (Reference 2.4.1-231):

- Irrigation – 13.89 million liters per day (mld) (3.67 million gallons per day [mgd]) of freshwater.
- Livestock – 0.45 mld (0.12 mgd) or freshwater.
- Mining – 8.52 mld (2.25 mgd) of freshwater.
- Thermoelectric Power – 1491.1 mld (393.9 mgd) of saline water.

The only additional county encountered within an 80.5-km (25-mi.) radius of the LNP site is Sumter (Figure 2.4.1-213). There are no surface water withdrawals for public, domestic, or industrial water supply in this county (Table 2.4.1-208). Surface water withdrawals in this county include the following (Reference 2.4.1-231):

- Irrigation – 2.42 mld (0.64 mgd) of freshwater.
- Livestock – 0.26 mld (0.07 mgd) or freshwater.
- Mining – 64.28 mld (16.98 mgd) of freshwater.

Additional counties within an 80.5-km (50-mi.) radius of the LNP site include Alachua, Dixie, Gilchrist, Hernando, Lake, Pasco, and Putnam (Figure 2.4.1-213). There are no surface water withdrawals for domestic water supply in these counties (Table 2.4.1-208). Surface water withdrawals in these counties include the following (Reference 2.4.1-231):

- Public Supply – 0.04 mld (0.01 mgd) of freshwater.
- Industrial Water Use – 115.64 mld (30.55 mgd) of freshwater.
- Irrigation – 61.25 mld (16.18 mgd) of freshwater.
- Livestock – 0.91 mld (0.24 mgd) or freshwater.
- Mining – 7.76 mld (2.05 mgd) of freshwater.
- Thermoelectric Power – 52.6 mld (13.9 mgd) of freshwater and 1706.2 mld (1956.5 mgd) of saline water.

Subsection 2.4.12.2.1 of this FSAR summarizes groundwater users.

2.4.2 FLOODS

2.4.2.1 Flood History

LNP COL 2.4-2 The Inglis Dam and Inglis Bypass Channel Spillway are the two main structures near the LNP site that control the flow of water in Lake Rousseau and the Withlacoochee River. The gates of the Inglis Dam are typically closed and the Inglis Bypass Channel Spillway is used to control the pool elevation at Lake Rousseau. During periods of flow that exceed the operating capacity of the bypass spillway, the Inglis Dam gates are opened to control the pool elevation of Lake Rousseau. Maximum allowable headwater elevation at both the bypass spillway and Inglis Dam is 8.5 m (28.0 ft.) NGVD29. Operating capacity of the bypass spillway is 43.6 m^3/s (1540 cfs). (References 2.4.1-222 and 2.4.1-223)

As stated in FSAR Subsection 2.4.1.2.3, five USGS stations record stages in the Withlacoochee River and Lake Rousseau near the LNP site (Figure 2.4.1-211). Maximum stages heights for these stations are summarized below:

- **Withlacoochee River at Dunnellon, Florida** (USGS ID: 02313200, #20 on Figure 2.4.1-211): This station is located on the Withlacoochee River, 1.3 km (0.8 mi.) upstream of Lake Rousseau. Daily water stage has been recorded at this station for a period of 44 years (1963 – 2007) (Reference 2.4.1-208). Maximum stage observed at this station during that period is 9.26 m (30.37 ft.) NGVD29 (September 27, 2004) (Reference 2.4.2-201). Add the conversion factor of -0.267 m (-0.876 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River at Inglis Dam near Dunnellon, Florida** (USGS ID: 02313230, #21 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the upstream side of the Inglis Dam (Reference 2.4.1-209). Daily water stage has been recorded at this station for a period of 22 years (1985 – 2007). Maximum stage observed at this station during that period is 8.54 m (28.03 ft.) NGVD29 (March 27, 2005). (Reference 2.4.2-203) Add the conversion factor of -0.309 m (-1.01 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River below Inglis Dam near Dunnellon, Florida** (USGS ID: 02313231, #22 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the downstream side of the Inglis Dam. Daily water stage has been recorded at this station for a period of 38 years (1969 – 2007). (Reference 2.4.1-210) Maximum stage observed at this station during this period is 2.82 m (9.25 ft.) NGVD29 (March 20, 1998) (Reference 2.4.2-204). Add the conversion factor of -0.309 m (-1.01 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River Bypass Channel near Dunnellon, Florida** (USGS ID: 02313250, #23 on Figure 2.4.1-211): This station is located 2.1 km (1.3 mi.) upstream of the Inglis Bypass Channel Spillway. Daily water stage has been recorded at this station for a period of 36 years (1971 – 2007). (Reference 2.4.1-211) Maximum stage observed at this station during this period is 8.57 m (28.11 ft.) NGVD29 (January 2, 1994) (Reference 2.4.2-205). Add the conversion factor of -0.310 m (-1.02 ft.) to elevations with a datum of NGVD29 to obtain elevations with a datum of NAVD88 at this station (Reference 2.4.2-202).

- **Withlacoochee River at Chambers near Yankeetown, Florida** (USGS ID: 02313272, #24 on Figure 2.4.1-211): This station is located 17.7 km (11 mi.) downstream of the Inglis Dam on the Lower Withlacoochee River at the mouth of Gulf of Mexico. Tidal high and tidal low daily gage height data are only available from January 28, 2005, to July 23, 2007, at this station. (Reference 2.4.1-212) The maximum stage observed at this station during high tides is 1.36 m (4.47 ft.) NAVD88 (June 13, 2006). The maximum stage observed at this station during low tides is 0.14 m (0.46 ft.) NAVD88 (March 21, 2006). (Reference 2.4.2-206)

The National Weather Service (NWS) has identified flood stages at USGS Station 02313200 (#20 on Figure 2.4.1-211). NWS has identified the flood stage, moderate flood stage, and major flood stage at this station to be gauge heights of 8.8 m (29 ft.), 9.1 m (30 ft.), and 9.4 m (31 ft.) NGVD29, respectively (References 2.4.2-207 and 2.4.1-208). Water levels at this station have not exceeded the major flood stage during the 44-year period of record (1963 – 2007). The moderate flood stage at this station has only been exceeded once during the period of record, for 22 consecutive days in 2004 (September 27 – October 18). The flood stage at this station has been exceeded during 15 of the 44 years of record. (Reference 2.4.2-201)

As discussed in FSAR Subsection 2.4.1.1, nominal plant grade elevation for the LNP site is 15.2 m (50 ft.) NAVD88 and the nominal plant grade floor elevation for LNP 1 and LNP 2 is 15.5 m (51 ft.) NAVD88. The pre-construction elevation of the footprints of LNP 1 and LNP 2 and associated facilities varies between 12.5 m (41 ft.) and 14.9 m (49 ft.) NAVD88, and the elevation of the LNP site varies between approximately 9.1 m (30 ft.) and 18.3 m (60 ft.) NGVD29 (Figures 2.4.1-203 and 2.4.1-204). Based on historical water level observations, flooding of the LNP site is considered unlikely. However, areas near the LNP site — specifically lower elevation areas near Lake Rousseau, the Withlacoochee River, and the CFBC — may become flooded during high water periods. Historical flooding has not been observed in the area downstream of the Inglis Dam because of the upstream water control provided by the dam (Reference 2.4.1-207).

Historical flooding associated with surges, seiches, and tsunamis is discussed in FSAR Subsections 2.4.5 and 2.4.6.

2.4.2.2 Flood Design Considerations

Safety-related structures and facilities for the LNP site are protected against floods and flood waves caused by probable maximum events, such as the probable maximum flood (PMF) and the probable maximum hurricane (PMH). Details associated with the PMF and PMH are discussed further in FSAR Subsections 2.4.3 and 2.4.5, respectively. Subsection 3.4.1 of the DCD discusses the protection of seismic Category I structures and safety-related systems against local floods. Seismic Category I structures, systems, and components within the plant site are designed to withstand the effects of flooding due to natural phenomena. The basemat and exterior walls of seismic Category I structures are designed to resist upward and lateral pressures caused by the PMF and high groundwater levels. No dynamic water forces associated with high water levels will occur because of a higher finished plant grade. The dynamic forces associated with the probable maximum precipitation (PMP) are not factors in the analysis or design because the finished grade will be adequately sloped.

Nonsafety-related structures, systems, and components have no necessity to survive flood events; therefore, there are no requirements that they be protected from either internal or external flooding. In addition, adverse effects of flooding caused by high water or ice effects do not have to be considered for water sources outside the scope of the certified AP1000 design. For example, the flooding of water intake structures, cooling canals, reservoirs, or channel diversions will not prevent the safe operation of LNP 1 and LNP 2.

2.4.2.3 Effects of Local Intense Precipitation

The effect of the local PMP on the drainage areas adjacent to the power block safety-related facilities, including the drainage from the roofs of the facilities, was evaluated. DCD Subsection 3.4.1.1.1 discusses the protection of seismic Category I structures and safety-related systems against local floods. The roofs do not have drains or parapets, but are sloped so that rainfall is directed toward gutters located along the edges of the roofs. Therefore, water does not pond on the roofs. A drainage system designed to remove runoff from up to a 50-year precipitation event will consist of conveying water from roof gutters and/or scuppers, as well as runoff from the LNP site and adjacent areas, to catch basins, underground pipes, or directly to open ditches. During a local PMP event, the drainage system is conservatively assumed to stop functioning; the LNP site is drained by overland flow on open roads and ground surface away from the safety-related structures and off-site to the nearby Lower Withlacoochee River and the Gulf of Mexico.

The proposed nominal plant grade elevation for LNP 1 and LNP 2 and other safety-related structures is 15.2 m (50 ft.) NAVD88. The proposed nominal plant grade elevation for nonsafety-related facilities, including the switchyard and construction-related facilities, is 14.3 m (47 ft.) NAVD88. The proposed floor elevation for LNP 1 and LNP 2 and other safety-related structures is 15.5 m (51 ft.) NAVD88. All subsequent elevations discussed in this subsection are references to NAVD88.

Figure 2.4.2-201 presents the conceptual grading and drainage of the LNP site, which is subdivided into Zones A through G. Zone A, which is located on the western side of LNP 2, drains over the top of the ditch embankment on the western peripheral boundary of the zone. Zone B, which is located north of LNP 2, drains over the top of the ditch embankment on the northern peripheral boundary of the zone. Zone C, which is located on the eastern side of LNP 2, drains over the high point of the grade in the north of this zone. Zone D, which is located on the eastern side of LNP 1, drains into Zone G over the north-south plant road that forms the eastern boundary of this zone. Zone E, which is located on the southern and western sides of LNP 1, drains over the plant road along the southern peripheral boundary of this zone and through the area between Pond A and the cooling tower for Unit 1. Zone F, which is located northeast of LNP 1, drains over the high point of the grade at the northern peripheral boundary of this zone. Zone G, which is located northeast of LNP 1, drains over the north-south slope embankment between Pond B and Pond C on the eastern peripheral boundary of this zone. Runoff from the PMP event flows away from safety-related structures and eventually off-site.

The local intense PMP is defined by Hydrometeorological Report (HMR) No. 52 (Reference 2.4.2-208). The 2.6-km^2 (1-mi.^{2}) PMP values for durations from 5 minutes to 24 hours are determined using the procedures described in Section 6.4 of HMR No. 52. As indicated in HMR No. 52, the 2.6-km^2 (1-mi.^{2}) PMP can be considered as point rainfall (i.e., these values are also applicable to areas that are less than 2.6 km^2 [1 mi.^{2}]). HMR No. 52 provides ratio analysis maps for 5, 15, and 30 minute durations relative to 1-hour precipitation for a 2.6-km^2 (1–mi.^{2}) area in HMR Figures 36, 37, and 38.

PMP values for a 2.6-km^2 (1-mi.^{2}) area are shown in Table 2.4.2-201. Figures 2.4.2-202 and 2.4.2-203 present the depth-duration and intensity-duration curves for the local intense PMP, respectively.

Using the 2.6-km^2 (1-mi.^{2}) PMP values given in Table 2.4.2-201, several functions were determined by best fit. The following function was found as an appropriate relationship to represent the depth-duration for a 2.6-km^2 (1-$mi.^2$) area.

$$D = \sqrt{\frac{14.473 + 360.646}{1 + 0.094T - 1.964x10^{-5}T^2}} \quad \text{Equation 2.4.2-1}$$

where

D = depth (in.).

T = duration (hr.).

The AP1000 design is based on a 2.6-km^2 (1-$mi.^2$) PMP of 52.6 centimeters per hour (cm/hr) (20.7 inches per hour [in/hr]). The 2.6-km^2 (1-$mi.^2$) PMP value for the LNP site is 49.8 cm/hr (19.6 in/hr) (Table 2.4.2-201).

The rational method was used to determine the peak runoff from each of the zones identified earlier. The rational method is given by the equation (Reference 2.4.2-209):

Q = C I A   Equation 2.4.2-2

where

Q= peak runoff (cfs).

C = coefficient of runoff (conservatively assumed to be 1.0).

I = intensity of rainfall (inch/hour).

A = drainage area (ac.).

The time of concentration for each drainage zone is estimated using Kirpich's formula (Reference 2.4.2-209), given by the equation below. The corresponding PMP intensities are determined from the times of concentration.

$Tc = 0.0078 \times L^{0.77} / S^{0.385}$ Equation 2.4.2-2a

where

Tc = time of concentration (min).

L = length of flow (ft.).

S = slope (ft/ft).

Water levels at the downstream periphery that correspond to runoff associated with a local PMP event for each drainage zone identified in Figure 2.4.2-201 are estimated assuming the local PMP peak runoff will flow over the peripheral grade at the end of each zone as flow over a broad-crested weir. The flow over a broad-crested weir is given by (Reference 2.4.2-210):

$$Q= C_d L H^{3/2} \qquad \text{Equation 2.4.2-3}$$

where

Q= Flow over a broad-crested weir (cfs).

C_d = Coefficient of discharge (2.7 for broad-crested weir per Reference 2.4.2-210).

L= length of overflow (ft.).

H= Head of water over the weir (ft.) (water level minus grade elevation).

A backwater computation is performed using HEC-RAS computer software (Reference 2.4.2-210a) to determine the maximum water surface elevation in each zone at LNP safety-related facilities. Depth of flow over the peripheral weir, calculated using Equation 2.4.2-3, and the weir length and elevation information presented on Figure 2.4.2-201, are considered to be the downstream water level boundary condition for backwater calculations. Each zone is modeled using cross sections perpendicular to the flow direction. The locations of cross sections used for the backwater analysis are shown on Figure 2.4.2-201. The flow at each cross section is estimated as a percentage of the peak flow for the entire zone, prorated according to the ratio of the area upstream of each cross section to the total area of the zone. The surfacing in the plant site area is predominantly either concrete or asphalt pavement or compacted gravel and grass. Conservatively, Manning's n-values for the cross sections are assumed to be 0.035 and 0.025 for peripheral and power block areas, respectively.

Zone A has a drainage area of 3.8 ha (9.4 ac.) and an estimated time of concentration of 10 min. The calculated peak flow in this zone is 13.2 m^3/s (465 cfs). The estimated water level at the periphery of this zone is 15.2 m (49.8 ft.), based on a weir length of 144.8 m (475 ft.) and elevation of 15.0 m (49.25 ft.). The resulting maximum water surface elevation, including backwater effect, near safety-related plant buildings in Zone A is 15.3 m (50.26 ft.).

Zone B has a drainage area of 2.6 ha (6.5 ac.) and an estimated time of concentration of 5 min. The calculated peak flow in this zone is 14.1 m^3/s (499 cfs). The estimated water level at the periphery of this zone is 15.1 m (49.4 ft.), based on a weir length of 283.5 m (930 ft.) and elevation of 14.9 m (49.0 ft.). The resulting maximum water surface elevation, including backwater effect, near the upstream end of Zone B is 15.3 m (50.10 ft.). This maximum water level will not affect water levels near the safety-related plant buildings in adjacent Zones A and C because it is lower than the adjacent flood levels in those zones.

Zone C has a drainage area of 6.9 ha (17.0 ac.) and an estimated time of concentration of 8 min. The calculated peak flow in this zone is 27.1 m^3/s (957 cfs). The estimated water level at the periphery of this zone is 14.6 m (47.8 ft.), based on a weir length of 161.5 m (530 ft.) and elevation of 14.3 m (47.0 ft.). The resulting maximum water surface elevation, including backwater effect, near safety-related plant buildings in Zone C is 15.5 m (50.69 ft.).

Zone D has a drainage area of 5.6 ha (13.9 ac.) and an estimated time of concentration of 17 min. The calculated peak flow in this zone is 14.9 m^3/s (525 cfs). The estimated water level at the periphery of this zone is 15.0 m (49.1 ft.), based on a weir length of 152.4 m (500 ft.) and elevation of 14.8 m (48.5 ft.). The resulting maximum water surface elevation, including backwater effect, near safety-related plant buildings in Zone D is 15.4 m (50.49 ft.).

Zone E has a drainage area of 22.0 ha (54.3 ac.) and an estimated time of concentration of 16 min. The calculated peak flow in this zone is 60.0 m^3/s (2120 cfs). The estimated water level at the periphery of this zone is 14.6 m (47.8 ft.), based on a weir length of 350.5 m (1150 ft.) and elevation of 14.3 m (47.0 ft.). The resulting maximum water surface elevation, including backwater effect, near safety-related plant buildings in Zone E is 15.4 m (50.39 ft.).

Zone F has a drainage area of 3.0 ha (7.3 ac.) and an estimated time of concentration of 10 min. The calculated peak flow in this zone is 10.2 m^3/s (361 cfs). The estimated water level at the periphery of this zone is 14.5 m (47.5 ft.), based on a weir length of 121.9 m (400 ft.) and elevation of 14.3 m (47.0 ft.). The resulting maximum water surface elevation, including backwater effect, near the upstream end of Zone F is 15.4 m (50.45 ft.). This maximum water level will not affect water levels near the safety-related plant buildings in adjacent Zone D.

Zones D and G have a combined drainage area of 10.9 ha (26.9 ac.) and an estimated time of concentration of 13 min. The calculated peak flow in these zones is 32.8 m^3/s (1160 cfs). The estimated water level at the periphery of these zones is 14.6 m (47.9 ft.), based on a weir length of 152.4 m (500 ft.) and elevation of 14.3 m (47.0 ft.). The resulting maximum water surface elevation near safety-related plant buildings, including backwater effect, in Zone D is 15.4 m (50.53 ft.). This value is higher than the water level of 15.4 m (50.49 ft.) calculated for Zone D alone and therefore is considered the maximum water level in Zone D.

The maximum water levels resulting from the local PMP event are below the nominal plant floor elevation of 15.5 m (51 ft.). Therefore, flooding caused by a local PMP event will not affect safety-related facilities associated with LNP 1 and LNP 2.

During the final design of the site grading and drainage, any roads in the path of surface runoff from a local PMP event will be graded to avoid adversely affecting PMP water levels near the safety-related facilities.

A review of historical rainfall records from the NWS Cooperative Observer Station No. 086414 in Ocala, Florida, for the period from 1971 to 2000, indicates monthly mean precipitation ranges between 6.27 and 18.29 cm (2.47 and 7.20 in.), and the annual mean precipitation is 126.19 cm (49.68 in.). The highest monthly precipitation of 41.58 cm (16.37 in.) occurred during April 1982. Daily extremes were calculated from 1948 to 2001 from the station's available digital record. The highest daily precipitation of 29.77 cm (11.72 in.) occurred on April 8, 1982. (Reference 2.4.2-211) FSAR Subsection 2.4.3.1 provides information pertaining to the PMP for the Withlacoochee River Drainage Basin. Because the LNP site is not expected to experience long-term accumulations of ice and snow, ice and snowmelt are not considered for flooding effects.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

The PMF has been defined as an estimate of the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe and reasonably possible at a particular location, based on comprehensive hydrometeorological application of PMP and other hydrologic factors favorable for maximum flood runoff (Reference 2.4.3-201). The PMF represents an estimated upper bound on the maximum runoff potential for a given drainage basin. Thus, the objective of this study is to obtain a PMF hydrograph and estimation of the reservoir flood level to ensure the plant's safety.

Using the previous definition as a guide, the PMF for the LNP site was developed using the following steps:

a. The Withlacoochee River Drainage Basin above the Inglis Dam of Lake Rousseau was delineated and the size of the basin that contributes to Lake Rousseau was determined. The Withlacoochee River Drainage Basin was divided into 18 subbasins.

b. The PMP storm hyetograph for the Withlacoochee River Drainage Basin was developed using the criteria and step-by-step instructions given in HMR 51 (Reference 2.4.3-202) and HMR 52 (Reference 2.4.2-208). The 72-hr total drainage-averaged PMP was determined and distributed according to the guidelines given in American National Standards Institute/American Nuclear Society (ANSI/ANS) -2.8-1992 (Reference 2.4.3-201).

c. The PMP design storm was developed by accounting for the antecedent rainfall that precedes the PMP storm as per ANSI/ANS-2.8-1992 (Reference 2.4.3-201) guidelines. Based on the requirements of ANSI/ANS-2.8-1992, Section 9.2.1.1 (Reference 2.4.3-201), the antecedent 72-hour storm having a volume of 40 percent of the PMP is followed by a period of 72 hours of no rain and then the full 72-hour PMP storm should be assumed to follow. Using this pattern, a complete PMP storm of 216 hours was developed.

d. Unit-hydrograph theory was used as the runoff model for developing runoff hydrographs for various subbasins. Therefore, various hydrological parameters required for developing unit hydrographs for the subbasins were determined. Using these parameters, unit hydrographs were developed for each subbasin.

e. The developed PMP storm hyetograph was applied to the unit hydrographs with the appropriate loss parameters using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model (References 2.4.3-203 and 2.4.3-204) to develop the estimated flood hydrographs for each subbasin, as well as for the entire drainage basin.

f. Inflow hydrographs from various subbasins were routed using the HEC-HMS model using appropriate routing parameters for various reaches to determine the combined inflow to Lake Rousseau.

g. After obtaining the combined inflow hydrograph, the PMF hydrograph was routed through the reservoir, spillway, and outlet works to estimate the maximum PMF stillwater level in Lake Rousseau.

2.4.3.1 Probable Maximum Precipitation

The PMP is theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year (Reference 2.4.3-205). In other words, the PMP is the estimated depth of precipitation for which there is virtually no risk of exceedance (Reference 2.4.3-201). The PMP depths used in this study were calculated using the criteria and step-by-step instructions given in HMR 51 (Reference 2.4.3-202) and HMR 52 (References 2.4.2-208 and 2.4.3-205).

Generally, a three-step process is followed for determining PMP in nonorographic regions: moisture maximization, transposition, and envelopment (Reference 2.4.3-205).

a. Moisture maximization consists of increasing storm precipitation measured in a major historical event by a factor that reflects the maximum amount of moisture that could have existed in the atmosphere for the storm location and time of year.

b. Transposition refers to the process of moving a storm (that is, its isohyetal pattern) from the location where it occurred to another location of interest. Transposition is carried out only within a region that is homogeneous with respect to terrain and meteorology.

c. Envelopment involves construction of smooth curves that envelope precipitation maxima for various durations and area sizes to compensate for data gaps. In addition, geographic smoothing is performed to ensure regional consistency.

Using these principles, estimates of all-season PMPs for various-sized areas and storm durations that are available in the form of generalized plots on Figures 18 through 47 in HMR 51(Reference 2.4.3-202) were obtained.

It is desired to obtain PMP estimates for the Withlacoochee River Drainage Basin above the Inglis Dam (Figure 2.4.3-201). The drainage area is 5171 km^2 (2020 mi.^{2}) (Reference 2.4.3-206), and the location of the centroid of the basin was calculated to be approximately 28°40'48" N, 82°10'10'' W. Using HMR 52 as a guide, the PMP for the Withlacoochee River Drainage Basin was developed using the following steps (Reference 2.4.2-208):

a. Determination of 6-hour incremental PMP.

b. Determination of 6-hour incremental PMP isohyetal pattern.

c. Maximization of precipitation volume.

d. Distribution of storm-area averaged PMP over the drainage basin.

e. Development of design storm for Withlacoochee River Basin above the Inglis Dam of Lake Rousseau.

2.4.3.1.1 Determination of 6-Hour Incremental PMP

The generalized estimates of all-season PMP depths available from Figures 18 through 47 of HMR 51 (Reference 2.4.3-202) were obtained for various area sizes, both larger and smaller than the drainage area under study for the Withlacoochee River Drainage Basin. Table 2.4.3-201 provides the 6-hour incremental depth-area-duration data taken from Figures 18 through 47 of HMR 51 (Reference 2.4.3-202). From the data presented in Table 2.4.3-201, Figure 2.4.3-202 plotted the smooth depth-area-duration curves for the Withlacoochee River Drainage Basin above the Inglis Dam.

This initial plotting of the basic input data serves two functions:

- It eliminates reader errors due to basic misinterpretation of values in the figures in HMR 51 (Reference 2.4.3-202).

- It applies initial important smoothing of the basic precipitation data.

From the smooth curves of Figure 2.4.3-202, the PMP depths for various durations were read as tabulated in Table 2.4.3-202. Using the depth-area-duration graph of Figure 2.4.3-202, depth-area-duration values for a set of standard isohyet area sizes, both larger and smaller than the size of the drainage area under study, were read. The selected standard isohyet area sizes for the current study are 1165.5 km^2 (450 mi.^{2}), 1813 km^2 (700 mi.^{2}), 2590 km^2 (1000 mi.^{2}), 3885 km^2 (1500 mi.^{2}), 5568.5 km^2 (2150 mi.^{2}), 7770 km^2 (3000 mi.^{2}), 11,655 km^2 (4500 mi.^{2}), 16,834.9 km^2 (6500 mi.^{2}), and 25,899.9 km^2 (10,000 mi.^{2}).

The depth-area-duration data for the selected standard areas from Table 2.4.3-202 were plotted on a linear grid and smooth curves were fitted as shown on Figure 2.4.3-203. From Figure 2.4.3-203, the PMP values corresponding to 18-hour duration were read as tabulated in Table 2.4.3-203. Incremental differences for the first three 6-hour periods were obtained by successive subtraction of the values contained in Tables 2.4.3-202 and 2.4.3-203. Table 2.4.3-204 shows the incremental PMP values obtained for the first three periods. Each set of 6-hour values was plotted against the corresponding area values and smooth lines were fitted through these points, as shown in Figure 2.4.3-204. Using the smooth curves from Figure 2.4.3-204, the data in Table 2.4.3-205 were tabulated for the 6-hour incremental PMP differences.

2.4.3.1.2 Determination of 6-Hour Incremental PMP Isohyetal Pattern

There is a preferred orientation for storms at a given geographic location. That orientation is related to the general movement of storm systems and the direction of moisture-bearing winds. The preferred orientation for storms at the location having its latitude 28°40’48” N and longitude 82°10’10’’ W is about 205° (Reference 2.4.2-208). The orientation of the storm pattern to produce maximum precipitation volume in the drainage basin was found to be approximately 150°, as shown in Figure 2.4.3-205. The angular difference in the orientations is 55°, which is more than 40°. This indicates that the storm-area averaged PMP given in Table 2.4.3-206 must be adjusted for orientation (Reference 2.4.2-208). The adjusted storm area averaged PMP is given in Table 2.4.3-206.

2.4.3.1.3 Maximization of Precipitation Volume

The maximum precipitation volume for the three largest 6-hour incremental periods resulting from placement of the storm pattern given in Table 2.4.3-206 over the Withlacoochee River Drainage Basin above the Inglis Dam was determined. To do this, it was necessary to obtain the value to be assigned to each isohyet in the pattern that occurs over the drainage basin during each time period. Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209 present the computations based on the HMR 52 procedure (Reference 2.4.2-208) for the first, second, and third increments, respectively.

Based on the calculations presented in Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209, the pattern area size that maximizes the volume of precipitation for the three largest 6-hour incremental periods was found to be 3840 km^2 (1500 $mi.^2$).

2.4.3.1.4 Distribution of Storm-Area Averaged PMP over the Drainage Basin

It was concluded that the maximum volume of precipitation occurs for a PMP pattern near 3840 km^2 (1500 $mi.^2$) when placed over the Withlacoochee River Drainage Basin. With this information, the values for each isohyet for all 12 six-hour increments can be determined. Table 2.4.3-210 provides the incremental average depths for each 6-hour period of the 72-hour storm. With this information, the isohyet values were obtained for all 12 increments (Table 2.4.3-211).

The values in Table 2.4.3-211 represent the incremental isohyet values for the Withlacoochee River Drainage Basin with a 3840 km^2 (1500 $mi.^2$) PMP pattern. To obtain incremental average depths for this drainage, it was necessary to compute the incremental volumes as determined in Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209 and then divide each incremental volume by the drainage area. The computations were performed in the tabular format as shown in Tables 2.4.3-212 and 2.4.3-213.

Based on the previous calculations, Table 2.4.3-214 provides the 72-hour total drainage averaged PMP. After obtaining the drainage-averaged PMP storm depths, they were distributed according to ANSI/ANS-2.8-1992 guidelines, as provided in Table 2.4.3-215 (Reference 2.4.3-201). Total rainfall for the 72-hour duration was found to be 90.9 cm (35.8 in.). The resulting hourly PMP rainfall distribution has been tabulated in Table 2.4.3-216 and plotted in Figure 2.4.3-206.

2.4.3.1.5 Development of Design Storm for Withlacoochee River Basin above the Inglis Dam of Lake Rousseau

Using the PMP rainfall distribution shown on Figure 2.4.3-206, a design storm was developed. The design storm was developed by accounting for the antecedent rainfall that precedes the PMP storm based on ANSI/ANS-2.8-1992 guidelines (Reference 2.4.3-201). This design storm, which was used as the rainfall input in the hydrologic modeling, consists of the following components:

- An antecedent 72-hour storm that comprises 40 percent of the PMP volume.
- A 72-hour dry period following the antecedent 72-hour storm.
- The full 72-hour PMP following the 72-hour no-rain period.

Combining the above three components, Figure 2.4.3-207 shows the resulting design storm rainfall data that were developed for the basin above the Inglis Dam of Lake Rousseau.

2.4.3.2 Precipitation Losses

This subsection pertains to assigning precipitation loss rates in the PMF hydrologic model. The amount of rainfall loss (the portion that does not contribute to runoff) is a function of the type of soil, the ground cover (vegetated, bare or paved), and the soil moisture prior to the storm. The loss methods and their parameters need to be selected in accordance with recognizable characteristics of the drainage basin under study. The amount of rainfall loss can be characterized by various methods and the HEC-HMS model offers several options for estimating precipitation losses.

Based on Federal Energy Regulatory Commission (FERC) recommendations (Reference 2.4.3-207), the traditional initial and constant loss rate methods for the PMF computations were selected from the HEC-HMS model precipitation loss methods. The following assumptions were made:

- Saturated antecedent conditions existed in the entire drainage basin prior to the start of the PMP.
- To be conservative for PMF runoff computations, the initial loss for the subbasins was zero inches.
- To be consistent with the saturated soil conditions, infiltration was set to occur at the minimum expected rate.

Figure 2.4.3-208 shows Withlacoochee River drainage subbasin areas above the Inglis Dam. Table 2.4.3-217 provides the drainage areas of various subbasins of the Withlacoochee River Drainage Basin above the Inglis Dam. To determine the minimum infiltration rate, the soil hydrologic group covering each subbasin was determined using the soil data from SWFWMD (Reference 2.4.3-208). Figure 2.4.3-209 provides a map describing soil hydrologic groups in the study basin. Table 2.4.3-218 presents distribution of hydrologic soil groups in various subbasins of Withlacoochee River Drainage Basin. Figure 2.4.3-210 summarizes the overall distribution of soil hydrologic groups in the Withlacoochee River Drainage Basin.

Figure 2.4.3-211 (Reference 2.4.3-208) presents a land use map for the Withlacoochee River Drainage Basin. Figure 2.4.3-212 and Table 2.4.3-219 summarize the overall land use distribution in the Withlacoochee River Drainage Basin. The dominant land uses and coverages in the Withlacoochee Drainage Basin are wetlands, upland forest, rangeland, agriculture, and mining with some transitional and urban areas.

Based on the U.S. Department of Agriculture (USDA), Natural Resources Conservation Service's [NRCS] "Urban Hydrology for Small Watersheds," the minimum infiltration rates reported for the various hydrologic soil groups were used (Reference 2.4.3-209). Subbasin-area-weighted average loss parameters were used for the constant infiltration rate (Reference 2.4.3-210). Table 2.4.3-220 lists the loss parameters for various subbasins of the Withlacoochee River Drainage Basin.

2.4.3.3 Runoff and Stream Course Models

The PMF event was simulated by a runoff and stream routing model that estimates rainfall-runoff response characteristics of a given drainage area and then computes the accumulation of runoff through river channels and reservoirs to determine corresponding water level conditions. The USACE HEC-HMS was the selected model for the Withlacoochee River hydrologic analyses. HEC-HMS is well-documented, updated, supported, and widely accepted throughout the water resources industry. HEC-HMS is flexible and offers many options to input precipitation, estimate runoff hydrographs, and manipulate and route hydrographs. HEC-HMS has been used extensively throughout the U.S to predict stream flows in drainage basins with and without gauging stations (Reference 2.4.3-204).

HEC-HMS is a deterministic model of the hydrologic processes of rainfall and runoff. The HEC-HMS model simulates the surface runoff response of a stream basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component simulates an aspect of the precipitation-runoff process within a subbasin.

HEC-HMS performs four basic functions:

- Computes rainfall losses and generates subbasin hydrographs.
- Combines hydrographs from different areas with correct timing.
- Routes hydrographs through channels.
- Routes hydrographs through ponds and flood control dams.

These functions are combined in a logical manner to model a particular drainage basin. Representation of a component requires a set of input parameters that specify the particular characteristics of the component, and mathematical relationships describe the physical processes.

2.4.3.3.1 Runoff Model

A runoff model is used to transform excess precipitation into surface runoff and is generally represented in the form of a unit hydrograph. A unit hydrograph is defined as the direct runoff hydrograph produced by one unit (inch) of effective rain uniformly distributed over a subbasin. Unit hydrographs are combined with precipitation data to determine the direct runoff hydrograph for a particular basin. Thus, unit hydrographs are developed for each subbasin using their specific parameters.

Several different methods can be used to develop a unit hydrograph for a given subbasin. In this study, Snyder's synthetic hydrograph method was selected. The Snyder unit hydrograph method determines only the unit hydrograph peak discharge (Q_P) and the lag time (t_L) that are defined as (Reference 2.4.3-207):

$$t_L = CC_t\left(LL_C\right)^{0.3} \qquad \text{Equation 2.4.3-1}$$

$$Q_P = \frac{640 C_P A}{t_L}$$ Equation 2.4.3-2

where

L = flow path length from outlet to the hydraulically farthest point (basin divide).

L_C = flow path length from outlet to subbasin centroid.

C_t = Snyder basin lag coefficient.

C_P = Snyder peaking coefficient.

The parameters L and L_C of the Snyder method are determined from the geometry of each subbasin. The parameters C_t and C_P are strictly empirical in nature and their values are often recommended as applicable to a specific region. C_t accounts for storage and shape of the drainage basin, and C_P is a function of flood-wave velocity and storage. Typical values of C_t and C_P reported by Viessman (Reference 2.4.3-211) for eastern Gulf of Mexico localities are 8.0 and 0.6, respectively. To better represent PMP conditions that would have rapid concentration of runoff from various subbasins, the C_P values for all subbasins, except the surface of Lake Rousseau, were increased by 33 percent from 0.6 to 0.8.

To apply the unit hydrograph approach to the Withlacoochee River watershed, unit hydrographs were developed for each of the 18 subbasins of the Withlacoochee River drainage basin above the Inglis Dam except for the lake surface. To determine a specific unit hydrograph for each subbasin, various subbasin characteristics such as hydraulic length, gradient, drainage density, and drainage patterns were determined. Therefore, it is necessary to delineate various subbasins according to their natural drainage divides as shown on Figure 2.4.3-208.

According to Reference 2.4.3-219, reservoir inflow unit hydrographs for inflow design flow determinations should be peaked 25 to 50 percent to account for the fact that unit hydrographs are usually derived from smaller floods. Therefore, in addition to increasing the regional C_P values of the subbasins, the peak flow of each subbasin's unit hydrograph was further increased by an additional 25 percent to account for non-linearity effects. Therefore, the overall increase in the unit hydrograph peak flow rate for each subbasin was about 66 percent [(1+0.33)*(1+0.25) = 1.66]. To increase the peak of each unit hydrograph, the lag time parameter was reduced appropriately to keep the volume within each unit hydrograph equal to unity. Table 2.4.3-221 lists various subbasin characteristics, along with the derived Snyder unit hydrograph parameters. Figure 2.4.3-213 shows the unit hydrographs developed for various subbasins of the

Withlacoochee River drainage basin above the Inglis Dam using the parameters listed in Table 2.4.3-213.

For the pool area associated with Lake Rousseau, runoff was calculated using the direct runoff method assuming zero travel time neglecting the fact that Lake Rousseau is 5.7 mi. long. Further, zero loss was assumed from the lake. Thus, precipitation falling on the surface of Lake Rousseau is directly converted into equivalent runoff without considering any lag time or water loss. This calculation was conducted by multiplying the rainfall hyetograph ordinates by the area of the lake surface.

2.4.3.3.2 Base Flow

According to ANSI/ANS-2.8-1992 (Reference 2.4.3-201), the mean monthly flow should be used as the base flow rate for the PMF analysis. The base flow rate to Lake Rousseau was conservatively equal to the mean monthly average flow of 28.5 m^3/s (1008 cfs). This value was calculated based on the published USGS mean monthly flow statistics of Withlacoochee River from 1928 to 2006 near Holder (USGS Station 02313000, #18 on Figure 2.4.1-211) (Reference 2.4.3-213).

2.4.3.3.3 Basin Data

Basin data include the elements of the basin, connectivity, runoff, storage, discharge relationships of hydraulic structures, and routing parameters of stream reaches and reservoirs. Figure 2.4.3-214 presents a schematic of the Withlacoochee River Drainage Basin above the Inglis Dam and its elements along with their connectivity. The drainage system of the Withlacoochee River is very complex and some of its major features (Reference 2.4.1-205) are briefly discussed in this subsection.

The Withlacoochee River originates in the potentiometric high for the central Florida region in north central Polk County, known as the Green Swamp. Withlacoochee River's headwaters flow through several natural control points, or plateaus, in the Green Swamp. These areas include: Eva, located in the uppermost headwaters of the river; Rock Ridge, west of Eva; Stanley Fish Hole; Cumpressco,east of Rock Ridge; and Richland. The Richland control point is a natural separation between the Withlacoochee River and the headwaters of the Hillsborough River. During periods of heavy rainfall within the Green Swamp, the Withlacoochee River will reach elevations where overflow ultimately occurs across this natural control point to the Hillsborough River. (Reference 2.4.1-205) For this PMF evaluation, all runoff was kept in the Withlacoochee River watershed.

The Withlacoochee River Drainage Basin also encompasses a number of small intermittent streams, connected lakes and wetlands, sinkholes, and tributaries. An important feature of the Withlacoochee River is Lake Tsala Apopka. Lake Tsala Apopka covers an area of approximately 77.7 km^2 (30 $mi.^2$) and drains a basin encompassing approximately 238.3 km^2 (92 $mi.^2$). This lake is actually a series of three hydrologically distinct pools: Floral City, Inverness, and Hernando. During periods of high water, flow from the Withlacoochee River is diverted into the Floral City Pool through the Leslie Heifner and Orange State Canals. Both canals have control structures that regulate inflow from the river and prevent back flow from the lake system to the river. Water travels northward to the Inverness Pool through the Golf Course control structure and Moccasin Slough. Just north of Moccasin Slough, water can bypass the rest of the system through the Bryant Slough structure (when operated) to the Withlacoochee River. If not, water moves through the pool to the Brogden Bridge Structure and Culvert, into the Hernando Pool. Discharge from the Hernando Pool can occur through the Van Ness Structure to Two Mile Prairie, a series of sinks north of the lake, or through Structure S-353 to Canal 331 outfalling back to the Withlacoochee River. (Reference 2.4.1-205)

During times of flows in excess of the 10-year flood on the river and in addition to inflow through the canal, the Tsala Apopka chain of lakes system in the Floral City area also receives considerable uncontrolled inflow from the Withlacoochee River. As the river rises above the natural control elevation along the west boundary of Flying Eagle Ranch, the river spills over into the lake system. This overbank flood flow can amount to several thousand cfs, many times greater than the maximum potential inflow through the Orange State and Leslie Heifner Canals. Additionally, driveways have been constructed across several canals, creating small dams. (Reference 2.4.1-205)

Flood routing describes the movement of a flood wave as it traverses a reach of channel. Of particular interest in flood routing are: the reduction of the peak discharge as it moves downstream (attenuation); the travel time of the flood peak between points of interest; the maximum water stage at points of interest; and the change in shape of the flood hydrograph as it moves downstream. These effects are governed by factors such as the channel bedslope, the cross-sectional area and geometry of the main channel and overbank areas, the roughness of the main channel and overbank, the existence of storage of floodwaters in off-channel areas offset from active water conveyance areas, and the shape of the flood hydrograph as it enters the channel reach.

In the absence of detailed hydraulic geometry data for the Withlacoochee River drainage system, the Muskingum routing method was selected as the routing computation option for the streams. The Muskingum method is a commonly used hydrologic routing procedure for handling a variable discharge-storage relationship and is appropriate for the large flows estimated here. This method models the storage volume of flooding in a river channel by a combination of wedge and prism storage. During the advance of a flood wave, inflow exceeds outflow, producing a wedge of increasing storage. During the recession, outflow exceeds inflow, resulting in a negative wedge of decreasing storage. This method consists of two parameters, K and X. The value of X depends on the shape of the modeled wedge storage. The value of X ranges from 0 for reservoir-type storage to 0.5 for a full wedge. The parameter K is the time of travel of the flood wave through the channel reach.

For the present study, a trial and error procedure was used to determine the appropriate values of K and X by varying their values to match the simulated peak flows corresponding to several reported extreme events. The parameters of K and X were varied between 0.1 to 150 hours and 0 to 0.5, respectively (Reference 2.4.3-214). The extreme events that were considered include the 10-, 25-, 50-, 100-, 500-year floods, as well as a standard project flood. To determine peak flows corresponding to the 10-, 25-, 50-, 100-, 500-year, 24-hour rainfall events, a flood frequency analysis (FFA) was used. The discharge corresponding to the standard project flood was 509.7 m^3/s (18,000 cfs) (Reference 2.4.3-215).

The following steps were used to determine the approximate values of K and X:

- Based on the long record (1928 to 2006) of the discharge data at the USGS station near Holder (Reference 2.4.3-213), several frequency distributions were fitted as presented in Figure 2.4.3-215.

- The Log Pearson Type III distribution was selected as the appropriate distribution to represent the annual peak flow at the USGS Station near Holder as given in Figure 2.4.3-216.

- From the upper 95th percentile confidence interval of Figure 2.4.3-216, 10-, 25-, 50-, 100-year discharges were obtained. Using this information, a relationship between discharge (Q, cfs) and 24-hour rainfall with concurrent probability (P, inches) was determined as follows:

  $$Q = 541.16P^{1.17}, \ R^2 = 1.00$$

- Using the above relationship, 24-hour rainfall amounts for the 500-year and standard project event were determined.

- The initial flood frequency based flows were associated with the drainage area of 4727 km^2 (1825 $mi.^2$), while the drainage area of the Withlacoochee River is 5232 km^2 (2020 $mi.^2$). As such, the peak discharges were multiplied by the ratio of 2020/1825 = 1.11.

- Using the rainfall amounts and SCS Type-II distribution, the HEC-HMS model was run for 10-, 25-, 50-, 100-, 500-year events and a standard project flood by varying the values of K and X while matching HEC-HMS-based peak flows with those based on the upper 95th percentile flood frequency. To be conservative, the HEC-HMS-based peak flows were kept on the higher side of the targeted flow rates.

Table 2.4.3-222 presents a comparison between peak flows based on the FFA presented in Figure 2.4.3-216 and those obtained using the HEC-HMS model. It is clear from Table 2.4.3-222 that the HEC-HMS-based peak flows are consistently higher than the FFA-based flows by 50 percent for all extreme events. Table 2.4.3-223 tabulates the parameters that were used for reach routing. By applying these high-end runoff estimates, the evaluation of the LNP safety during a potential PMF is conservative.

2.4.3.3.4 Reservoir Data

Lake Rousseau is a 9.2-km (5.7-mi.) long, man-made impoundment of the river formed by the Inglis Dam. The Inglis Dam is located near the city of Inglis and approximately 17.7 km (11 mi.) upstream of the mouth of the Withlacoochee River. Structures that control flow from the reservoir include part of the USACE's CFBC facilities. Lake Rousseau's elevation is controlled by three structures: the Inglis Bypass Channel Spillway, the Inglis Dam, and the Inglis Lock. The Inglis Bypass Channel Spillway passes the majority of the normal discharge (up to 43.6 m^3/s [1540 cfs]) to the Lower Withlacoochee River which was bifurcated by the CFBC project. The Inglis Dam passes flows in excess of the bypass channel and spillway capacity to the CFBC via a short remnant section of the natural Withlacoochee River that is now separated from the current flow of the river but connects the dam to the CFBC and then to the Gulf of Mexico. At present, the Inglis Lock is not functional and it was not included in the analysis. (Reference 2.4.1-205)

2.4.3.3.4.1 Stage-Storage Analysis – Lake Rousseau

The available bathymetric image presented in Figure 2.4.3-217 (Reference 2.4.3-216) was geo-referenced to directly correspond with surrounding USGS digital terrain data downloaded from the EPA data repository (Reference 2.4.3-217). The bathymetric contours were digitized to develop the stage-storage curve. Table 2.4.3-224 and Figure 2.4.3-218 present the stage-storage relationship for Lake Rousseau.

2.4.3.3.4.2 Control Structure Description and Stage-Discharge Relationship – Lake Rousseau

2.4.3.3.4.2.1 Inglis Dam

The Inglis Dam Spillway is a reinforced concrete, U-shaped, two-bay, gated spillway with an ogee-type weir (crest elevation of 3.4 m [11.3 ft.] NGVD29, this is also the invert elevation of the structure) and reinforced concrete wingwalls. Each bay is provided with a 12.2-m (40-ft.) wide by 5.1-m (16.7-ft.) high vertical lift gate, installed on the crest of the weir. The gate operating equipment is mounted on a reinforced concrete operating platform at an elevation of 15.8 m (52 ft.) NGVD29. The structure is configured with a reinforced concrete service bridge at an elevation of 10.1 m (33 ft.) NGVD29. Riprap has been provided upstream and downstream of the spillway to protect against eroding velocities. (Reference 2.4.1-223)

The gates of the Inglis Dam are normally closed while the Inglis Bypass Channel Spillway is used to maintain normal pool levels and pass most discharge to the Lower Withlacoochee River. During periods of high inflow to Lake Rousseau that exceed the operating capacity of the Inglis Bypass Channel Spillway, the Inglis Dam is operated to control the reservoir elevation. To meet the structural and stability requirements of the Inglis Dam, the maximum allowable headwater elevation on the structure is not allowed to exceed the elevation of 8.5 m (28 ft.) NGVD29. All gates are operated at the same gate opening and are opened gradually to allow tailwater stages to rise before large releases are made. The pool may be routinely lowered up to 0.15 m (0.5 ft.) in advance of predicted heavy rainfall depending on reservoir conditions and river flow. (Reference 2.4.1-223)

2.4.3.3.4.2.2 Inglis Bypass Channel Spillway

The purpose of the Inglis Bypass Channel Spillway is to discharge freshwater into the Lower Withlacoochee River in sufficient quantities to sustain the prevailing environment, prevent saltwater intrusion, maintain the level of the lake under normal flows, and to accommodate navigation interests (Reference 2.4.1-222).

The Inglis Bypass Channel Spillway is a reinforced concrete, U-shaped, two-gate spillway with an ogee weir and a baffled stilling basin with an invert elevation of 6.4 m (21 ft.) NGVD29. The structure is fitted with two hydraulically operated vertical lift gates that measure 0.61 m by 4.27 m by 2.13 m (2 ft. by 14 ft. by 7 ft.) to regulate outflows. The structure is provided with an operating platform to accommodate the gate operating equipment and a service bridge that crosses the structure at an elevation of 9.1 m (30 ft.) NGVD29. Steel sheet pile wing walls are constructed at 45° angles from the direction of flow at the upstream and downstream ends of the spillway. Bulkhead slots are provided upstream of the vertical lift gates for temporary closure for maintenance and gate repairs. (Reference 2.4.1-222)

2.4.3.3.4.2.3 Operation of Inglis Dam and Inglis Bypass Channel Spillways

Lake Rousseau is primarily formed by the presence of the Inglis Dam. The operating pool elevation is between 7.3 m (24 ft.) to 8.5 m (28 ft.) NGVD29, with a normal pool elevation of 8.38 m (27.5 ft.) NGVD29. Normally, all flow from Lake Rousseau, except for water quality releases, are passed to the lower river through the Inglis Bypass Channel Spillway up to the maximum capacity of the bypass channel, which is 43.6 m^3/s (1540 cfs). This is accomplished by operating with partial gate openings at the Inglis Bypass Channel Spillway until inflow into the pool exceeds the capacity of the bypass facility with the gates fully open. The minimum target regulated flow is not less than 8.5 m^3/s (300 cfs). (Reference 2.4.1-222)

During storm tide events, when abnormally high tides from tropical storms, hurricanes, or strong winter storms occur, the Inglis Bypass Channel Spillway is operated in a manner that will not add to tidal flooding in the lower river. In advance of storm tides predicted to be in excess of 1.5 m (5 ft.) NGVD29, the Inglis Bypass Channel Spillway discharge is reduced to 8.53 m^3/s (300 cfs) until the threat of high tides begins to recede. The Inglis Bypass Channel Spillway is then reopened as soon as possible. The purpose of reducing the bypass spillway discharge is not only to reduce flooding in the lower river, but also to avoid filling the floodplains storage capacity in advance of high tides. (Reference 2.4.1-222)

2.4.3.3.4.2.4 Stage-Discharge Relationship

The stage-discharge relationship for both the Inglis Dam and the Inglis Bypass Channel Spillway were obtained from the State of Florida Environmental Protection Agency's *Water Control Plan for Inglis Project Works* (2001) (Reference 2.4.3-215). There is a natural low-lying area on a ridge south of the Inglis Dam where the lake will overflow once the water elevation is above 8.5 m (28 ft.) NGVD29. Figure 2.4.3-219 shows that the length of this low-lying area is at least 1.6 km (1 mi.), i.e., 1609 m (5280 ft.). It was assumed that this low-lying overflow area behaves hydraulically as an ogee spillway with a design head (H_0) to upstream dam height (P) ratio of 1.0. The discharge over an ogee crest is given by the following equation (Reference 2.4.3-218):

$$Q = CLH_e^{3/2}$$ Equation 2.4.3-3

where

Q = discharge (cfs).

L = effective length of crest (ft.).

H_e = total head on the spillway crest including velocity of approach (ft.).

C = variable discharge coefficient.

The effective length of the spillway is determined by taking contraction effects from piers and abutments into account. The effective length of spillway L is determined using the following relationship:

$$L = L' - 2(NK_P + K_a)H_e$$ Equation 2.4.3-4

where

L' is the net length of the spillway.

N is the number of piers.

K_P and K_a are pier and abutment contraction coefficients, respectively.

In the present study, $K_P = K_a = 0.01$ and N = 1 were assumed. The discharge coefficient C varies with the ratio of upstream dam height P to water depth above the spillway crest H_0 and with the ratio of total head H_e to design head H_0. Figures 9.23 and 9.24 in Section 9.12 of *Design of Small Dams* provide discharge coefficient curves (Reference 2.4.3-218).

To determine the discharge coefficients, the following relationships were developed and used in the calculations:

$$C = C_0\left[0.862 + 0.137\sqrt{\frac{H_e}{H_0}}\right]^2 \qquad \text{Equation 2.4.3-5}$$

where, C_0 is the discharge coefficient. For $P/H_0 = 1$, C_0 is 3.89. The value of C_0 is further reduced to account for friction effects. Using Equations 2.4.3-3 to 2.4.3-5, flow through the low-lying area was calculated.

The stage-discharge relationships for the Inglis Dam Spillway, the Inglis Bypass Channel Spillway, and low-lying overflow area are given in Table 2.4.3-225 and Figure 2.4.3-220. The stage-discharge relationships given in Table 2.4.3-225 and Figure 2.4.3-220 for both the Inglis Dam and Bypass Channel spillways correspond to uncontrolled flow conditions. The total flow relationship was used to route the PMF through Lake Rousseau.

2.4.3.4 Probable Maximum Flood Flow

The 1-hour incremental PMP values tabulated in Table 2.4.3-216 were applied to the unit hydrographs of the subbasins presented in Figure 2.4.3-213, along with values of initial loss and infiltration parameters given in Table 2.4.3-220. The HEC-HMS model estimated the PMF hydrographs for the subbasins and the entire Withlacoochee River Drainage Basin routed it through Lake Rousseau. It was assumed that Lake Rousseau was completely full at the start of the simulation. Figure 2.4.3-221 presents the inflow hydrograph to Lake Rousseau. The peak inflow is 1720 m^3/s (60,755 cfs), and it occurs about 4 weeks after the PMP event.

2.4.3.5 Water Level Determinations

The outflow from the lake and PMF peak water levels in Lake Rousseau were determined utilizing the HEC-HMS model with the stage-storage relationship as given in Figure 2.4.3-218, and stage-discharge relationships given in Figure 2.4.3-220 for Lake Rousseau. As mentioned previously, the stage-discharge relationships given in Table 2.4.3-225 and Figure 2.4.3-220 correspond to the uncontrolled flow conditions for both the Inglis Dam and Bypass Channel spillways. While there can potentially be many scenarios of gate operation and corresponding water level in the lake, the most conservative scenario was assumed in the present study. In this scenario, it was assumed that both the control structures at the Inglis Dam and Inglis Bypass Channel Spillway stopped working and, hence, could not be operated to release excess flow. Figure 2.4.3-222 and Table 2.4.3-226 show the water levels along with the inflow and outflow hydrographs in Lake Rousseau.

From Figure 2.4.3-222, it is clear that the maximum water elevation in Lake Rousseau is 9.05 m (29.7 ft.) NAVD88.

2.4.3.6 Coincident Wind Wave Activity

Lake Rousseau is 9.2-km (5.7-mi.) long and is located approximately 4.8 km (3 mi.) south of the LNP site. Further, as discussed in FSAR Subsection 2.4.3.5, the PMF elevation for Lake Rousseau is 9.05 m (29.7 ft.) NAVD88. The nominal plant grade floor elevation for the LNP site is 15.5 m (51 ft.) NAVD88. Thus, the elevation difference between the Lake Rousseau PMF elevation and the floor elevation of LNP safety-related facilities is 6.45 m (21.3 ft.). It should be noted that there are no safety-related structures situated adjacent to Lake Rousseau.

Because of the physical characteristics of Lake Rousseau, described in FSAR Subsections 2.4.1.2.6 and 2.4.3.3.4, no LNP safety-related structures could be affected by dynamic effects of wave action from wind generated activity that may occur concurrently with the peak PMF water level in Lake Rousseau.

2.4.4 POTENTIAL DAM FAILURES

LNP COL 2.4-2
LNP COL 2.5-15

As discussed in FSAR Subsection 2.4.1.2.7, the Inglis Dam is located at the west end of Lake Rousseau (Figure 2.4.1-208). During normal flow periods, Lake Rousseau discharges to the Lower Withlacoochee River via the Inglis Bypass Channel Spillway. During periods of high inflow, the dam may be used to pass excess flows to a remnant stretch of the Withlacoochee River to maintain the pool elevation of Lake Rousseau between 7.3 and 8.5 m (24 and 28 ft.) NGVD29. The crest elevation of the dam spillway is 8.5 m (28 ft.) NGVD29. The maximum stage observed at USGS gauge 02313250, located downstream of the Inglis Dam, during the 38 years of record is 2.82 m (9.25 ft.).

The primary purpose of this analysis is to assess the adverse impacts at the LNP site resulting from flooding downstream of the Inglis Dam. In particular, an inundation analysis was conducted to determine the water levels in the CFBC following the postulated failure of the Inglis Dam and then these levels were compared to the LNP plant site. The dam break inundation analyses included three distinct steps:

- Estimation of the dam-break outflow hydrograph.
- Routing of the dam-break hydrograph through the downstream channel.
- Estimation of inundation levels.

The flood hydrograph from a dam failure is dependent upon the primary factors of the physical characteristics of the dam, the volume of the reservoir, and the mode of failure. The parameters which control the magnitude of the peak discharge and the shape of the outflow hydrograph include: the breach dimensions; the manner and length of time for the breach to develop; the depth and volume of water stored in the reservoir; and the inflow to the reservoir at the time of failure.

The USACE HEC-RAS (River Analysis System) computer program (Reference 2.4.3-203) was the selected model for calculating water surface elevation downstream of Lake Rousseau during a dam failure. This program is specifically designed for applications in floodplain management and flood insurance studies. The HEC-RAS model is the most widely used computer program for water surface profile computations and is accepted by the Federal Emergency Management Agency (FEMA) for modeling open channel hydraulic systems. The HEC-RAS step-backwater surface profile model requires the following input data: flow regime, starting or ending water surface elevation, peak discharge rates, roughness and transition energy loss coefficients, cross section geometry, and reach lengths.

For this study, USGS digital elevation model data (Reference 2.4.3-217) were used to cut cross sections covering the entire Lower Withlacoochee River Drainage Basin in lateral directions (i.e., on both sides of the centerline of the river). Cross sections at intervals of 152.4 m (500 ft.) were created starting from downstream of the Inglis Dam and ending at the shoreline. Figure 2.4.4-201 shows the HEC-RAS schematic and approximate location of the LNP site.

The worst-case scenario of routing the peak flow was considered using a steady-state model rather than routing the complete hydrograph. This approach yields the maximum water elevation from the release of the maximum discharge from Lake Rousseau because it holds the upstream level at a constant high peak value. The dam breach peak discharge was calculated using the following relationship (References 2.4.4-201 and 2.4.4-202):

$$Q_P = 40.1 V_w^{0.295} H_w^{1.24} \qquad \text{Equation 2.4.4-1}$$

where Q_p = dam breach peak discharge (cfs), V_w = reservoir volume at the time of failure (ft.^{3}), and H_w = height of water in the reservoir at the time of failure above the base elevation of the breach (ft.). To determine the maximum value of Q_p, maximum values of V_w = 4194 hectare-meters (ha-m) (34,000 acre-feet [ac-ft]) and H_w = 9.36 m (30.7 ft.) were assumed. The corresponding value of Q_p is 1722 m^3/s (60,811 cfs). It should be noted that the maximum outflow from Lake Rousseau during a PMF is 1720 m^3/s (60,755 cfs) so the dam break flow is nearly the same and the increase in risk from Lake Rousseau's failure by a dam breach is negligible.

The downstream boundary condition at the shoreline was a water depth set equal to the 10 percent exceedance high tide of 2.01 m (6.59 ft.) NGVD29. The obtained results are presented in Figures 2.4.4-202 and 2.4.4-203. The corresponding water elevations are given in Table 2.4.4-201. The maximum water surface elevations in the Lower Withlacoochee River under these conditions do not exceed 7.53 m (24.72 ft.) NGVD29 at river station locations south of the LNP site (river stations 24288.89 through 55472.83). The pre-construction elevation of the footprints of LNP 1 and LNP 2 and associated facilities varies from 12.5 to 14.9 m (41 to 49 ft.) NAVD88 (approximately 12.8 to 15.2 m [42 to 50 ft.] NVGD29). It is clear from these results that the LNP site will not be affected by flooding due to a dam failure.

Similar to the above analysis, a failure of the Inglis Lock would not affect the safety-related structures at the LNP site. It is highly unlikely that such a failure could occur because it would require the triple failure of both lock doors and the precautionary bulwark on the lake-side of the lock. As described above for a postulated failure of the Inglis Dam, flooding due to a postulated lock failure could not exceed 7.3 to 7.6 m (24 to 25 ft.) NGVD29.

In the event of an Inglis Dam failure, the remnant portion of the Withlacoochee River located downstream of the Inglis Dam and its floodplain will be inundated. Because the CFBC is located between the remnant portion of the Withlacoochee River and the LNP site, flood flows will be diverted to the Gulf of Mexico via the CFBC before moving north. The failure of the Inglis Dam will not affect the LNP site safety-related facilities or the availability of the cooling water supply. Furthermore, LNP 1 and LNP 2 will use a passive core cooling system designed to provide emergency core cooling without the use of active equipment such as pumps and ac power sources. The passive core cooling system depends on reliable passive components and processes such as gravity injection and expansion of compressed gases.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

2.4.5.1 Probable Maximum Winds and Associated Meteorlogical Parameters

2.4.5.1.1 Historic Storm Surge Events

LNP COL 2.4-2

As stated in FSAR Subsection 2.4.1.1, the LNP is located in southern Levy County, Florida, about 12.8 km (7.9 mi.) east of the Gulf of Mexico. Between 1851 and 2006, northwest Florida was struck by 57 hurricanes. While 14 of these storms were classified as “major hurricanes” (Category 3 or higher), no storms of Category 4 or Category 5 were reported to have struck the area near the LNP site during this time (Reference 2.4.5-201). A list of hurricanes that have impacted the areas within 80 km (50 mi.) of the LNP site from 1867 to 2004 is presented in Table 2.4.5-201 (Reference 2.4.5-202). Figure 2.4.5-201 depicts the hurricane tracks of these storms. Figures 2.4.5-202, 2.4.5-203, 2.4.5-204, 2.4.5-205, 2.4.5-206, 2.4.5-207, 2.4.5-208, 2.4.5-209, 2.4.5-210, 2.4.5-211, 2.4.5-212, 2.4.5-213, 2.4.5-214, 2.4.5-215, 2.4.5-216, and 2.4.5-217 show the tracks of major hurricanes (Category 3 or higher) that impacted the region during each decade from 1850 through 2006.

The coastline of Levy County has a very shallow slope, and this allows hurricane-induced surges to inundate coastal communities

(Reference 2.4.5-203). Inglis and Yankeetown are two residential areas near the LNP site. Analyses of hurricane flood stages for both towns are presented in Table 2.4.5-202. The maximum reported flood elevation for Inglis is 4.3 m (14.2 ft.) above msl (Reference 2.4.5-203). The maximum reported flood elevation for Yankeetown is 6.6 m (21.5 ft.) above msl (Reference 2.4.5-203). The maximum storm surge reported in Levy County from Tropical Storm Frances, which struck land in September of 2004, occurred at Cedar Key and was less than 1.5 m (5 ft.) (Reference 2.4.5-204).

2.4.5.1.2 Probable Maximum Hurricane

As defined by National Oceanic and Atmospheric Administration (NOAA) NWS Report 23 (Reference 2.4.5-205), the PMH is a hypothetical steady-state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speed that can probably occur at a specified coastal location. A PMH is specified in terms of several meteorological parameters that vary with location: central pressure, peripheral pressure, radius of maximum winds, forward speed, track direction, and inflow angle. Parameters for the PMH analysis are taken from the NOAA NWS Report 23 as shown in Table 2.4.5-203 (Reference 2.4.5-205).

Based on Regulatory Guide 1.59, Revision 2 (1977), the 10 percent exceedence antecedent high spring tide is taken as 1.3 m (4.3 ft.) mean low water (MLW). The maximum astronomical tide (MAT) is 1.5 m (4.9 ft.) mean lower-low water (MLLW), as shown on Figure 2.4.5-218 (Reference 2.4.5-206).

2.4.5.2 Surge and Seiche Water Levels

2.4.5.2.1 Storm Surge Analysis

A storm surge is a temporary rise in sea level caused by water being driven over land primarily by the onshore hurricane force winds and only secondarily by the reduction in sea-level barometric pressure between the eye of the storm and the outer region. The magnitude of the surge at a specific site is also a function of the radius of the maximum hurricane winds, the forward speed of the storm's eye, and the bathymetry near the shoreline.

Three different approaches were used to estimate the storm surge at the LNP site. The first approach was based on using estimates of probable maximum surge levels at open-coast locations computed by the U.S. Nuclear Regulatory Commission (NRC) staff. NUREG-0800, Revision 3 (March 2007) recommends that the PMH surge level should be determined using the data presented in Regulatory Guide 1.59.

The second approach was based on using results obtained by the Evaluation Branch/Meteorological Development Lab, National Weather Service/NOAA/United States Department of Commerce using the Sea, Lake, and Overland Surge from Hurricanes (SLOSH) model (Reference 2.4.5-207) for various categories of hurricanes. Given a group of hypothetical hurricanes of a particular category, speed, and landfall direction, the SLOSH model outputs the maximum storm surge heights at each grid cell in a basin. The SLOSH model results were used to obtain estimates of surge elevations at various locations such as at the coastal line, Yankeetown, Inglis, and near the LNP site due to hypothetical hurricanes of Category 1 through Category 5 impacting the Cedar Key region.

The third approach was based on correlating the estimates of surges at the coastline from the SLOSH model and Hsu's empirical equation (Reference 2.4.5-208) for various categories of hurricanes.

Coastal line surge results obtained from the second and third approaches were used to determine a relationship between these two approaches. The obtained relationship was used to determine the expected PMH surge elevation at the coastal line. Further, the coastal line surge elevations for various categories of hurricanes were related to surge elevations at inland locations such as Yankeetown and Inglis, Florida. These relationships were utilized to determine the PMH surge elevation at the LNP site. Before discussing the PMH and probable maximum surge (PMS) analyses, it is necessary to discuss the datums used for the surge elevation.

There are several datums that have been mentioned in this report. Datums are of two types: tidal and fixed. For example, msl pertains to the local mean sea level (msl), which is a tidal datum as it is based on astronomical tides. A tidal datum is determined over a 19-year National Tidal Datum Epoch. NAVD88 and NGVD29 are fixed geodetic datums whose elevation relationships to local msl and other tidal datums may not be consistent from one location to another. NAVD88 replaced NGVD29 as the national standard geodetic reference for heights. Benchmark elevations relative to NAVD88 are available from National Geodetic Survey (NGS) through the World Wide Web. For the LNP site, the nearest tidal datum is located at the Cedar Key, Florida. Elevations of the Cedar Key tidal datum are defined with respect to tides as given in Table 2.4.5-204 based on 1983 – 2001 epoch (Reference 2.4.5-209).

2.4.5.2.2 PMH Surge Level Determination Using Regulatory Guide 1.59

The NUREG-0800, Revision 3 (March 2007) recommends the storm surge induced by the PMH should be estimated as recommended by Regulatory Guide 1.59 and supplemented by current best practices. Appendix C of Regulatory Guide 1.59 presents timesaving methods of estimating the maximum stillwater level of the PMS from hurricanes at open coast sites on the Atlantic Ocean and Gulf of Mexico. According to Regulatory Guide 1.59, these procedures are based on PMS values determined by the NRC staff and its consultants and by applicants for licenses that have been reviewed and accepted by the NRC staff. Estimates of open-coast stillwater levels at various selected locations are given in Regulatory Guide 1.59. In order to determine estimates of the PMS stillwater level at an open coast site other than these locations, interpolation between these known locations on either side of the site under study can be used after locating it, based on its site-specific bathymetry. The location of the site under study is determined by comparing its ocean profile down to a depth of 182.9 m (600 ft.) MLW to the profiles given in Appendix C of Regulatory Guide 1.59.

In the current analysis, the site under study is very close to Crystal River where a PMS estimate is readily available. Therefore, it can be assumed that the PMS estimate at the coastal line for the LNP site is approximately equal to the open coast PMS estimates for Crystal River. For the open coast location in Crystal River, Table C.1 of Regulatory Guide 1.59 gives the following PMS data:

Wind setup	= 8.09 m (26.55 ft.)
Pressure setup	= 0.81 m (2.65 ft.)
Initial rise	= 0.18 m (0.60 ft.)
10 percent exceedance high tide	= 1.3 m (4.30 ft.) MLW
Total surge	= 10.39 m (34.10 ft.) MLW

Using the above information along with the Cedar Key datum elevation given in Table 2.4.5-204, the PMS estimate at the coastal line for the LNP site is 10.89 m (35.72 ft.) NAVD88.

2.4.5.2.3 Storm Surge Analysis with SLOSH

The second approach for estimating the maximum storm surge at the LNP site is based on results from the SLOSH numerical model corresponding to various categories of hurricanes. SLOSH is a mathematical model that stands for Sea, Lake, and Overland Surge from Hurricanes (Reference 2.4.5-210). SLOSH was developed by the NWS to estimate storm surge heights and winds associated with historical, hypothetical, and predicted hurricanes (Reference 2.4.5-210). SLOSH is the basis for Hurricane Evacuation Studies (HES) and is the primary model used by FEMA, the NOAA, and the USACE (Reference 2.4.5-210).

The National Hurricane Center (NHC) uses the SLOSH model to determine the timing, severity, and sequence of wind and storm surge hazards that can be expected from hurricanes of various intensities, tracks, and forward speeds striking the study area, computing the potential effects of many hundreds of theoretical hurricanes (Reference 2.4.5-211). SLOSH is a mathematical model that cannot perfectly replicate nature. Verification of the SLOSH model was conducted by the NHC with real-time runs of historical storms (Reference 2.4.5-212). The computed surge heights are compared with those measured from historical storms and, if necessary, adjustments are made to the input or basin data. The SLOSH model is generally accurate within plus or minus 20 percent (Reference 2.4.5-213). Based on a statistical analysis conducted by the NHC, adding 20 percent to the computed SLOSH surge values would eliminate most of the potential negative errors. However, such an adjustment would also add additional surge height to those values that already contain positive errors, possibly overestimating the surge heights produced by the SLOSH model.

The point of a hurricane's landfall is crucial to determining which areas will be inundated by the storm surge. If the hurricane forecast track is inaccurate, SLOSH model results will be inaccurate. The SLOSH model, therefore, is best used for defining the potential maximum surge for a location (Reference 2.4.5-213).

Additional features of the SLOSH model output are given below (Reference 2.4.5-210):

- SLOSH can estimate water surface elevations due to the storm surge for both the open coast and on land.
- MEOW: It stands for "Maximum Envelope of Water." Envelope refers to the maximum height the water reaches at any point in time at every grid cell in the SLOSH basin, for a given hypothetical storm. The MEOW is formed from a composite of many individual SLOSH model runs. It is the set of the highest surge values at each grid location for a given storm category, forward speed, and direction of motion, regardless of which individual storm simulation produced the value. The NHC has generated one MEOW for each storm category, storm direction, forward speed, and tide level used in the simulation study.
- MOM: Maximum of the MEOWs (MOMs) are further combinations of MEOWs. As in the case of MEOWs, the purpose of preparing MOMs is to compensate for forecasting inaccuracies. MOMs are created by the NHC by extracting the highest peak surge values from two or more MEOWs.
- Adjustments to Astronomical Tide: The SLOSH output is a combination of the normal tide and the storm surge to generate the total increase in water level due to a given storm. The SLOSH model accounts for astronomical tides by specifying the initial tide level. In this analysis, the SLOSH model considers an initial mean tide of 2.5 ft. NGVD29. However, the 10 percent exceedance high tide for Cedar Key is 4.3 ft. MLW. The equivalent value of the 10 percent exceedance high tide for Cedar Key in NGVD29 is 2.01 ft. Therefore, the assumed initial tide elevation in the SLOSH model is 0.49 ft. higher than the 10 percent exceedance high tide for the Cedar Key NOAA gauge site. Thus, the SLOSH model output needs to be corrected by subtracting 0.49 ft.
- Adjustments to Wave Effect: The SLOSH model does not provide data concerning the additional heights of waves generated by wind-driven waves on top of the stillwater storm surge.

SLOSH is a two-dimensional finite difference code that uses an adaptive curvilinear grid for various regions along the Gulf and Atlantic coasts. SLOSH assumes uniform friction to solve the equations of motion for reference basins along the Gulf of Mexico and Atlantic Ocean coast. A geographical region with known values for topography and bathymetry is called a SLOSH basin. The individual elements of the SLOSH grid are the basis for calculating water surface elevations caused by storm surge in a specific SLOSH basin. The grid allows for barriers to flow, cuts in barriers, one-dimensional flow in rivers and streams, and increased friction for trees and mangroves in certain grid blocks to be taken into consideration in the calculations. The water depth is calculated based on the elevation of the grid cell and the amount of water that is able to flow into that cell. The water surface is found at the elevation of the water depth combined with the average ground elevation of the grid cell. The SLOSH model contains topographic information for each grid cell. These data are combined with the storm surge calculations based on the storm characteristics to determine the water surface elevations caused by storm surge.

Given a SLOSH basin and a hurricane track (identified by its pressure, radius of maximum winds, location, direction, and speed), the SLOSH model solves a complex set of equations to determine the water surface elevations caused by storm surge. Figure 2.4.5-219 shows the grid for the SLOSH model's Cedar Key Basin, which contains the LNP site location. The parameters for various categories of hurricanes considered in the SLOSH modeling are given in Table 2.4.5-205 (Reference 2.4.5-214).

To determine the surge levels at the coastal line, four different points near the LNP site were selected as shown on Figure 2.4.5-220. Tables 2.4.5-206, 2.4.5-207, 2.4.5-208, and 2.4.5-209 show the maximum surge levels at these locations for each category of hurricane. Table 2.4.5-210 lists the maximum surge levels at these points along with the average of these four surge levels.

Maximum water elevations obtained by the SLOSH model at Yankeetown (29° 1'46.99"N, 82°42'58.00"W) and Inglis (29°1'48.00"N, 82°40'8.00"W) (Reference 2.4.5-215) are given in Table 2.4.5-211.

As is clear from Table 2.4.5-211, the SLOSH MOM scenario predicts that the LNP site is dry for Category 1 through Category 5 hurricanes. The maximum water surface elevations as obtained from the SLOSH model are shown graphically on Figures 2.4.5-221, 2.4.5-222, 2.4.5-223, 2.4.5-224, and 2.4.5-225.

2.4.5.2.4 PMH Surge Level Determination Using Hsu's Empirical Method

The maximum storm surge may be estimated rapidly by modifying Jelesnianski's method (1972) as proposed by Hsu (2004) (References 2.4.5-208, 2.4.5-216, and 2.4.5-217). According to Hsu (Reference 2.4.5-208), the maximum storm surge, *Sp*, can be estimated using the equation:

$$S_P = 0.07(1010 - P_0)F_S F_M \qquad \text{Equation 2.4.5-1}$$

where *Sp* is in meters msl, P_0 is the minimum sea level pressure in millibars (mb), F_S is the shoaling factor (obtained from Figure 2.4.5-226, Reference 2.4.5-208), and F_M is the correction factor for storm motion (from Figure 2.4.5-227, Reference 2.4.5-208). In order to determine F_S, one should know the coastal distance of the shoreline site. The locator map with coastal distances given in NOAA NWS Report 23 (Reference 2.4.5-205), was used to determine the coastal distance of the Cedar Key NOAA gauge site. The coastal distance of the Cedar Key NOAA gauge site is little more than 1931.2 km (1200 mi.). Figure 2.4.5-226 also specifies the location of the Cedar Key NOAA gauge site and its corresponding value of F_S = 1.6. In order to determine F_M, one should know both the forward speed and track direction of the PMH storm. Table 2.4.5-203 lists these parameters for the PMH storm. Using PMH parameters given in Table 2.4.5-203 along with Figure 2.4.5-227, the maximum value of F_M was found to be 0.7. Table 2.4.5-212 presents the PMH parameters used for the Hsu (2004) method.

This technique is of great value, particularly when either a rapid estimation of probable storm surge is needed or areas with limited availability of input data are required for detailed modeling. Using Equation 2.4.5-1, Hsu et al. estimated storm surge heights generated by hurricanes Katrina and Rita in 2005 (Reference 2.4.5-208). Based on comparisons between these estimated and observed surge heights, the surge heights calculated using Equation 2.4.5-1 were found to be in reasonable agreement with preliminary watermark measurements.

The maximum coastal surge heights were calculated using Hsu's empirical equation for various categories of hurricanes. The surge heights estimated using Equation 2.4.5-1 were converted into NGVD29 datum. As is clear from Table 2.4.5-204, the elevation of the Cedar Key NOAA gauge site is 1.24 m (4.06 ft.) NAVD88. Using the VERTCON tool (Reference 2.4.2-202), latitude, longitude, and orthographic height in NAVD88, the corresponding elevation of the Cedar Key NOAA gauge site was found to be 1.443 m (4.733 ft.) NGVD29.

Using this information, a given elevation in msl datum can be converted to NGVD29 datum. For example, an elevation of X ft. msl can be converted into the NGVD29 datum using the following expression:

> Elev ft. NGVD29 = X ft. msl + msl datum (3.84 ft.) - NGVD29 datum (4.733 ft.)

The above expression can be written as:

> Elev ft. NGVD29 = (X ft. msl - 0.893)

After making a datum conversion from msl to NGVD29, surge elevations estimated using Hsu's model were compared with those obtained from the SLOSH model. Table 2.4.5-213 and Figure 2.4.5-228 present this comparison. The data plotted on Figure 2.4.5-228 give the following relationship between the coastline surge elevations resulted by these two different approaches:

$$\begin{aligned} &SLOSH\ Model\ Coastline\ Storm\ Surge\ Elev.\ (ft.\ NGVD29) = \\ &1.07 * Hsu's\ Model\ Coastline\ Storm\ Surge\ Elev.\ (ft.\ NGVD29) + 0.8 \end{aligned}$$

Equation 2.4.5-2

It is clear from Figure 2.4.5-228 that the maximum storm surge heights at the coastal line for hurricane Category 1 through Category 5, calculated using Hsu's method, are in good agreement with the storm surge heights obtained from the SLOSH model. Thus, Hsu's model can be used to predict the coastline storm surge due to PMH. As indicated in Table 2.4.5-213, the surge elevations estimated using Hsu's model are consistently higher than those obtained from the SLOSH model. Therefore, the surge elevations obtained from Hsu's model are conservative.

Using the results tabulated in Table 2.4.5-210 and Table 2.4.5-211, the coastal storm surge heights and their corresponding water elevations at Yankeetown and Inglis were plotted as shown on Figure 2.4.5-229. This figure suggests that there is a relationship between the open coast surge height and water elevation at a given inland location. Knowing the coastal surge height for a given storm, the water elevation at Yankeetown can be calculated using the following equation:

$$\begin{aligned}&Water\,Elev.\ at\ Yankeetown\ (ft.\ NGVD29) =\\&1.06 * Coastal\ Surge\ Height\ (ft.\ NGVD29) - 0.02\end{aligned}$$ Equation 2.4.5-3

Similarly, the water elevation at Inglis can be calculated using the following equation:

$$\begin{aligned}&Water\ Elev.\ at\ Inglis\ (ft.\ NGVD29) =\\&1.182 * Coastal\ Surge\ Height\ (ft.\ NGVD29) - 1.9\end{aligned}$$ Equation 2.4.5-4

For a given storm, the water elevation at the LNP site can be extrapolated using the following equation:

$$WE_{LNP} = \mathrm{W}E_{Yankee} + \frac{(\mathrm{W}E_{Inglis} - \mathrm{W}E_{Yankee})}{(\mathrm{CD}_{\mathrm{Inglis}} - \mathrm{CD}_{\mathrm{Yankee}})}(\mathrm{CD}_{\mathrm{LNP}} - \mathrm{CD}_{\mathrm{Yankee}})$$ Equation 2.4.5-5

where, WE_{LNP} and CD_{LNP} are the water elevation (ft. NGVD29) and distance (mi.) of the LNP site from the coastal line, WE_{Yankee} and CD_{Yankee} are the water elevation (ft. NGVD29) and distance (mi.) of Yankeetown from the coastal line, and WE_{Inglis} and CD_{Inglis} are the water elevation (ft. NGVD29) and distance (mi.) of Inglis from the coastal line. Using Equation 2.4.5-5, the water elevation at the LNP site corresponding to the various hurricane storm Category 1 through Category 5 were determined as tabulated in Table 2.4.5-214.

2.4.5.2.5 Determination of Water Elevation at the LNP Site Corresponding to PMH

As shown in Table 2.4.5-203, the PMH parameters vary in a certain range. In order to determine surge elevation, various combinations of probable maximum hurricane parameters were randomly selected using the Monte Carlo Simulation

(MCS) technique. Based on 1000 MCS simulations, the coastal surge and water elevations at Yankeetown and Inglis were determined. Using these elevations, water elevations at the LNP site were determined as given on Figure 2.4.5-230. The maximum surge stillwater elevation including 10 percent exceedance high tide was found to be 12.97 m (42.54 ft.).

2.4.5.2.6 Seiches

Seiches are standing waves of a relatively long period that occur in lakes, canals, bays, and on the open coast. According to the USACE, a seiche is defined as (Reference 2.4.5-218):

a. A standing wave oscillation of an enclosed waterbody that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric.

b. An oscillation of a fluid body in response to a disturbing force having the same frequency as the natural frequency of the fluid system. Tides are now considered to be seiches induced primarily by the periodic forces caused by the Sun and Moon.

Other than the Gulf of Mexico and Lake Rousseau, there are no large bodies of water in the study area. Further, neither of these water bodies are in the immediate vicinity of the LNP site. Additionally, seiche has not been considered as the controlling influence for these bodies of water. Therefore, the potential for flooding at the site due to seiche effects is considered insignificant.

2.4.5.3 Wave Action

2.4.5.3.1 Wave Action and Breaking Wave Setup

As mentioned in FSAR Subsection 2.4.5.2.3, the SLOSH model does not include the additional heights generated by wind-driven waves on top of the stillwater storm surge. Therefore, wind-driven wave height needs to be determined. Within the surf zone, wave breaking is the dominant hydrodynamic process. Waves approaching the coast increase in steepness as water depth decreases. When the wave steepness reaches a limiting value, the wave breaks, dissipates energy, induces nearshore currents, and results in an increase in water level. This super elevation of mean water level caused by wave action is called wave setup. The most important physical parameter that affects the magnitude of wave setup is the depth of the water on which the surface waves are traveling. In the surf zone, water depth is not a constant; instead it varies with surge stage and ground elevation. These variations in water depth influence wave breaking. The variable water depths can produce major variations in wave conditions over short distances. In order to account for these variations and select the most critical combination of ground elevation and surge elevation, an MCS technique has been used.

Assuming the deep-water depth of 10 m (32.8 ft.), the limiting wave period was determined to be approximately 10 seconds. Further, assuming variation in ground surface elevation from 6.1 m to 7.6 m (20 ft. to 25 ft.) and surge elevation from 12.2 m to 12.8 m (40 ft. to 42 ft.), the wave setup was determined using the step-by-step approach in Chapter 4 of the *Coastal Engineering Manual* (Reference 2.4.5-219). In order to account for the variation in ground surface and surge elevations, both the ground surface and surge elevations were assumed to be uniformly distributed between their ranges. The wave setup was determined by using 1000 random combinations of ground surface and surge elevation to conduct MCS simulations. The maximum wave setup was found to be 1.85 m (6.08 ft.).

2.4.5.3.2 Wave Runup

To calculate wave runup under PMH conditions for the LNP site, the step-by-step approach in Chapter 4 of *Coastal Engineering Manual* (Reference 2.4.5-219) was used. The same parameters used in the wave setup calculation were used for the wave runup calculation. Further, an MCS simulation was used to incorporate variability in wave runup parameters. Based on the MCS simulation, the maximum wave runup was found to be 0.27 m (0.90 ft.).

The combined effect of wave setup and runup from wind-driven wave action as obtained by 1000 MCS runs is given on Figure 2.4.5-231. It is important to mention that the surge boundary remains on the west side of U.S. Highway 19, which is about 3.2 km (2 mi.) away from the LNP site. Thus, this temporary increase in water level due to wave setup will quickly disperse and it is unlikely to reach to the LNP site.

2.4.5.3.3 Total Water Depth Due to PMH Surge and Wave Action

According to the NRC guidelines, the total potential water depth at the LNP site is the sum of stillwater depth including 10 percent exceedance high tide, wave setup, and wave runup. In order to combine the surge, wave setup, and wave runup, the combined surge and wave action components were incorporated into a single MCS model. The obtained result of the MCS simulation is given on Figure 2.4.5-232. The maximum total water elevation was found to be about 15.10 m (49.52 ft.) NGVD29 or 14.79 m (48.52 ft.) NAVD88. The histogram of the maximum water elevation that indicates the expected probability associated with various water elevations is shown graphically on Figure 2.4.5-233. Additionally, 0.4 ft. and 0.6 ft. were added into the total water elevation to account for potential long-term sea level rise and sea level anomaly, respectively. Thus, including potential long-term sea level rise and sea level anomaly, the PMF elevation is 15.40 m (50.52 ft.) NGVD29 or 15.10 m (49.52 ft.) NAVD88. The results of PMH surge and wave action analysis have been summarized in Table 2.4.5-215.

2.4.5.4 Confirmatory Analysis for PMH Surge Using SLOSH Computer Model

A confirmatory Sea, Lake, and Overland Surges from Hurricanes (SLOSH) computer model analysis was performed for different PMH scenarios based on PMH parameters obtained from National Oceanic and Atmospheric Administration (NOAA), National Weather Services (NWS) Report 23 (Reference 2.4.5-205). The highest surge elevation at the LNP site, including 10 percent exceedance high tide and long-term sea level rise, was computed. To this computed surge elevation, corresponding wind-wave setup and wind-wave runup were added to obtain the maximum surge level at the LNP site.

2.4.5.4.1 SLOSH Computer Model

SLOSH is a two-dimensional, finite difference code that uses an adaptive curvilinear, polar coordinate grid for regions along the Gulf and Atlantic coasts. SLOSH assumes uniform friction to solve the equations of motion for reference basins. A geographical region with known values of topography and bathymetry is called a SLOSH basin.

The SLOSH computer model is developed and maintained by the NWS and is used to generate real time forecasting of hurricane storm surge on continental shelves, along coastlines and across inland water bodies. A detailed discussion of the model is presented in FSAR Subsection 2.4.5.2.3. SLOSH computer program Version 3.95 (v3.95) was used for the estimation of PMH surge water level at the LNP site. The SLOSH computer program was obtained from NWS and is the version currently used by the NWS for hurricane prediction for the region where the LNP site is located.

2.4.5.4.2 Cedar Key Basin Grid

For purposes of modeling the coastline of Gulf of Mexico, the NWS generated multiple SLOSH basin grids. The project site falls within the Apalachicola, Tampa, and Cedar Key SLOSH basin grids (Figure 2.4.5-234). The Cedar Key basin grid best represents the conditions specific to the project site and this basin was used for the PMH surge computation.

The basin grid, topographic and bathymetric data provided with SLOSH input files by NWS was used in the computation. In the examination of the LIDAR coverage used by NWS in the Cedar Key SLOSH basin, it was observed that the state LIDAR coverage did not encompass the area around the site. Thus, the highest elevation assigned by the NWS in the Cedar Key grid is 10.7 m (35 ft.) NAVD88 and serves as a default maximum height of any grid cell.

A LIDAR topographic survey was performed for the LNP site area with the vertical datum NAVD88. This LIDAR data and processed topographic contours for site area were incorporated into the state LIDAR data used by the NWS in the original Cedar Key grid. These new contours were then averaged for specific grids where the NWS assigned elevation did not correctly represent the topography. Subsequently, at each grid point, the existing grid elevation was compared with the average elevation obtained from the LIDAR data for the site. When a difference was noted, the existing grid elevation was modified based on this additional LIDAR data set. Grid cells outside the project area were also checked against elevations from South Florida Water Management District (SFWMD) USGS topographic information (Reference 2.4.5-220) to ensure that the elevations in the NWS grid were correctly represented. This revised grid data file was used for the SLOSH PMH simulations.

The SLOSH v3.95 FORTRAN code provided by NWS also contained a limitation wherein grid cells with elevations greater than 10.7 m (35 ft.) NAVD88 were removed from the flooding computation (i.e., these cells could never be flooded). It was confirmed from NWS that the 10.7 m (35 ft.) limit for surge in the SLOSH program is historical and does not pose any particular problems when it is relaxed. Since the LNP site is located at elevation greater than 10.7 m (35 ft.), the code was modified to allow flooding for any grid with elevations less than 17.1 m (56 ft.), where surge elevations were greater than the elevation of the cell, including those near the site. Once the code was modified, a new executable file was compiled. The SLOSH program code was validated with and without the changes in the code to determine that the changes in the code were effective and accurate in allowing flooding at elevations greater than 10.7 m (35 ft.) The validation for the SLOSH program was performed by comparing the same hurricane scenario for each code. The revised code was then validated against historical High Water Mark (HWM) data points for locations within Cedar Key Data Grid from a published FEMA report (Reference 2.4.5-221) for Hurricane Frances (2004) with the surge elevations computed using the SLOSH model.

2.4.5.4.3 PMH Parameters for LNP Site

NOAA Technical Report NWS 23 (Reference 2.4.5-205) defines the PMH as a hypothetical steady state hurricane, with a specific combination of five meteorological parameters that will generate the highest sustained wind speed that can probably occur at a specified coastal location. The five meteorological parameters are central pressure, Po; peripheral pressure, Pw; radius of maximum winds, R; forward speed, T; and track direction, Θ.

It is determined that the location of the LNP site is at approximately nautical mile marker 1125 in the NWS Report 23 (Reference 2.4.5-205). Using this location the PMH parameters were extracted from Reference 2.4.5-205. The peripheral pressure Pw for a PMH for the site is fixed at 1020 mb. As the Central Pressure, Po, increases, the pressure deficit (Pw – Po) decreases and consequently, the PMH induced surge will decrease with increasing P0. For the evaluation of maximum surge at site, the PMH is assumed to be a steady state PMH and the value of Central Pressure, Po, is taken to be the minimum specified value in NWS 23. Table 2.4.5-216 is a compilation of the selected PMH values, which are the same as the PMH parameters presented in the Table 2.4.5-203, except for minor variations in central pressure (Po), forward speed (T) and lower limit of the Track Direction (Θ). However, the track directions for maximum surge producing cases are well within the range of the values presented in Table 2.4.5-216.

2.4.5.4.4 Antecedent Water Level

For the computation of PMH surge water level at the LNP site using the SLOSH model, the initial open water level at the coast was determined by adding the long term sea level rise to the 10 percent exceedance high tide estimated based on observed tide data for this region. For the period from 1983 to 2010, the monthly extreme tide values were obtained from Reference 2.4.5-222. The spring high tide values were sorted from high to low and converted from the local station datum to NAVD88. The percent exceedance was tabulated from the sorted data. The 10 percent exceedance spring high tide elevation is calculated to be 1.0 m (3.23 ft.) NAVD88.

NOAA has evaluated sea level rise trends for each tide station. Figure 2.4.5-235 provides the data for the mean sea level trend at the Cedar Key tide gauge, station 8727520. The mean sea level trend has been calculated by NOAA to be +1.80 millimeters/year with a 95 percent confidence interval of +/-0.19 mm/yr based on monthly mean sea level data from 1914 to 2006. This is equivalent to a mean sea level rise of 0.2 m (0.59 ft.) in 100 years. The sea level rise of 0.2 m (0.59 ft.) in 100 years as evaluated by NOAA at the Cedar Key tide gauge station is appropriate for use as the sea level rise rate for the LNP site. Combining the initial water level of 1.0 m (3.23 ft.) NAVD88, corresponding to the 10 percent exceedance spring high tide with long term sea level rise of 0.2 m (0.59 ft.), an initial water level of 1.2 m (3.82 ft.) NAVD88 was used for all the SLOSH model runs.

2.4.5.4.5 Preliminary SLOSH Model Runs

A set of preliminary runs (matrix of 576 cases), as presented in Table 2.4.5-217, were input into the SLOSH model. The matrix of simulations representing the lower and upper limits of the PMH as listed in the table encompassed 16 landfall locations within 27 mi. north and 6.5 mi. south of the project site, 3 radii of maximum winds, 3 forward speeds, and 4 directions for the storm track. For the preliminary simulations, the pressure deficit was fixed at 130 mb at all times during the SLOSH simulation. These preliminary runs were used to narrow the range of parameters which had the greatest effect on surge values at the site.

Figure 2.4.5-236 provides a map of the landfall locations examined. As seen in Figure 2.4.5-236, most of the landfall locations are north of the project site. This is because the northeast quadrant of a hurricane (north being the axis of the hurricane track) will produce the greatest surge due to the counter clockwise rotation of the wind field; therefore, it would be expected that landfalls north of the project site will produce the greatest surge. This was confirmed from the preliminary run results.

Table 2.4.5-218 provides a summary of the minimum and maximum surge values calculated for each of the landfall locations (Figure 2.4.5-236). For all combinations of landfall locations, and forward speed, with the radius of maximum winds set at 7.5 mi. or 17 mi., no surge is produced at the site. Only simulations that used a radius of maximum winds of 26 mi. produced surge this far inland at the site.

2.4.5.4.6 Pressure Deficit Scenarios

Data and studies demonstrate that a constant pressure deficit is not representative of the normal evolution of a large hurricane as it approaches and makes landfall. As hurricanes reach landfall, central pressure begins to rise resulting in an exponential decay of pressure deficit with time (Reference 2.4.5-223). After the preliminary simulations were completed, the pressure deficit for each storm simulation was varied in the matrix of cases for the final simulations. Reference 2.4.5-223 provides a method for calculating the pressure deficit decay for hurricanes making landfall on the peninsula of Florida. The calculated change in pressure deficit at 6, 12, 18, and 24 hours after landfall is shown in Table 2.4.5-219.

Three scenarios for the change in pressure were selected to examine the effect of a change in pressure deficit with respect to the time of landfall. Table 2.4.5-220 describes each scenario and Figure 2.4.5-237 provides a graph of the change in pressure deficit with time for the three scenarios. Figure 2.4.5-237 also provides a comparison with pressure data from significant hurricanes that made landfall along the Gulf of Mexico. It is seen that as the storms approach landfall (at time=0 hours) the pressure deficit increases with the maximum occurring before or at landfall. Figure 2.4.5-237 shows that most of the storms also show a nearly linear ramp up of pressure deficit prior to landfall with an exponential decay after landfall. The decay after landfall follows the calculation of decay provided by Reference 2.4.5-223.

2.4.5.4.7 Final SLOSH Model Runs

It was determined from the preliminary runs that storms from the 13 most northern landfall locations produced the greatest surge when combined with a radius of maximum winds of 26 mi., and forward speed of 23 mph. These parameters were used to generate the matrix of simulations for the final computations of PMH surge.

For the final runs, as shown in Table 2.4.5-221, 182 SLOSH simulations were performed for each scenario, at 26 landfall locations spaced approximately a mile apart. These 26 landfall locations were chosen within the range of the 13 landfalls short listed from the preliminary runs. The radius of maximum winds and forward speed were fixed at 26 mi. and 23 mph respectively.

At each landfall location, the storm approach direction was varied, at 5° intervals, within the track direction range of 215°N-245°N PMH outlined in Table 2.4.5-216. Consequently for each of the 26 landfall locations seven storm directions were modeled resulting in 182 SLOSH simulations for each scenario. Figure 2.4.5-238 shows the storm parameters for a single landfall location, used for the final simulations.

For each of the scenarios, 182 simulation runs were preformed. From these simulations for each of the scenarios the highest surge elevation at the LNP site was extracted. Table 2.4.5-222 provides a listing of the storm parameters and, the corresponding maximum surge for each pressure deficit scenario.

Figures 2.4.5-239, 2.4.5-240, 2.4.5-241, 2.4.5-242, 2.4.5-243, and 2.4.5-244 provide the SLOSH display screenshots and maps displaying the surge at the time of the peak surge for the site, for Scenarios 1, 2, and 3, shown in Table 2.4.5-222.

Scenario 1 (constant Δp) produces marginally higher surge value than the other two pressure deficit cases. This result was expected and represents the most conservative of the final simulations.

For all scenarios, the time variations of surge elevation, wind speed and wind direction follow the pattern shown in Figure 2.4.5-245. The maximum surge condition of 14.5 m (47.7 ft.) for Scenario 1, Run #101, is depicted in Figure 2.4.5-245. The cells that include the site have an average elevation of 12.8 m (42 ft.); the cell remains dry until the surge elevation exceeds 12.8 m (42 ft.). The peak surge elevation occurs at the site for a narrow time frame, one time step, of ten minutes. Water enters the cell at one time step (10 minutes) prior to the peak. Peak winds of 180 mph are felt for about one hour with the peak surge occurring 20 minutes after the winds have begun to decline below 180 mph. Wind direction is northwesterly (onshore) as the hurricane makes landfall, clocking around to an easterly direction (offshore) over a 5-hour period as the hurricane passes, consistent with the typical dynamics of hurricanes.

2.4.5.4.7.1 Additional SLOSH Run with Higher Initial Water Level Based on RG 1.59

In FSAR Subsection 2.4.5.4.4 the 10 percent exceedance spring high tide was computed based on observed tide data. The 10 percent exceedance spring high tide was also determined following the Regulatory Guide 1.59 (RG 1.59). The 10 percent exceedance spring high tide value of 4.3 ft. MLW for Crystal River was directly obtained from Table C1 of RG 1.59. As this value from RG 1.59 is based on MLW datum, it was converted to NAVD88 using the datum conversion chart from Reference 2.4.5-226. Using the above conversion, the 10 percent exceedance high tide value of 4.3 ft. MLW is converted to 0.82 m (2.68 ft.) NAVD88. Also using the latest tidal epoch from Reference 2.4.5-227 for the period of 1983 to 2001 for Cedar Key, and following the RG 1.59, the 10 percent exceedance, predicted high tide level is computed to be 0.80 m (2.63 ft.) NAVD88. Conservatively, the 10 percent exceedance predicted high tide value of 0.82 m (2.68 ft.) NAVD88, computed using Table C1 of RG 1.59, was combined with initial rise of 0.18 m (0.6 ft.) to obtain 10 percent exceedance high tide level of 1.00 m (3.28 ft.) NAVD88. This 10 percent exceedance high tide level of 3.28 ft. NAVD88 is 0.02 m (0.05 ft.) higher than 10 percent exceedance high tide level of 3.23 ft. NAVD88 computed using observed tide data. By adding 10 percent exceedance high tide of 3.28 ft. NAVD88 to the long-term sea level rise of 0.59 ft., an initial water level of 1.18 m (3.87 ft.) NAVD88 was obtained.

In FSAR Subsection 2.4.5.4.7 Scenario 1, which resulted in maximum PMH surge, an initial water level of 1.16 m (3.82 ft.) NAVD88 was used based on observed tide data. An additional SLOSH run was performed with a higher initial water level of 3.87 ft. NAVD88 along with other input data used for Scenario 1. The PMH surge level obtained for this run was 14.54 m (47.7 ft.) NAVD88, which is the same as obtained for Scenario 1. Figure 2.4.5-247 shows the maximum surge at the LNP site for this additional SLOSH run.

2.4.5.4.8 Wind Wave Set-up and Run-up

Waves near the LNP site generated by the PMH design storm, as they propagate from the deep water of the Gulf, are influenced by gently sloping continental shelf. The winds continue to add energy into the wave field, however, energy dissipation due to bottom friction is significant and plays a major role in controlling and reducing the height of the waves as they approach the coast and then pass over the flooded landscape.

Based on Reference 2.4.5-224 and Reference 2.4.5-225, the wave setup at LNP site is conservatively estimated as 0.2 m (0.6 ft.).

For computation of wave runup, the elevation of the top of structure was chosen as the plant grade elevation of 15.2 m (50 ft.) NAVD88 for Units 1 and 2. The elevation at the toe of structure was determined based on the grade at the toe of the fill slope for Units 1 and 2. The depth of water at the toe of structure was based on the water level (PMH surge + Wave setup) of 14.7 m (48.3 ft.). The slope of the earthen structure (embankment) for all scenarios was assumed to be 3H:1V. The nearshore slope for all scenarios was calculated from station 300 to station 1300 as shown in Figure 2.4.5-246.

The depth of water at the embankment slope is shallow and the high hurricane winds are expected to generate waves that will break. Therefore, wave runup at the plant buildings was estimated due to a breaking wave generated based on the local depth from the maximum surge elevation of 14.72 m (48.3 ft.) NAVD88. The Coastal Engineering Manual approach was used to estimate the wave runup at the LNP site. Wave runup was computed using the CEM Equation VI-5-7 (Reference 2.4.5-228) which is given by:

$$\frac{R_{u2\%}}{H_s} = 1.5\xi_{op}\gamma_r\gamma_b\gamma_h\gamma_\beta \qquad \text{Equation 2.4.5-6}$$

Where $R_{u2\%}$ is the runup level exceeded by 2 percent of the incident waves,

H_s is the significant wave height,
ξ_{op} is the surf similarity parameter,
γ_r is the reduction factor for surface roughness (γ_r = 1 for smooth slopes),
γ_b is the reduction factor for influence of a berm (γ_b = 1 for non-bermed profiles),

γ_h is the reduction factor for non-Rayleigh distributed waves (γ_h = 1 for Rayleigh distribution), γ_β is the reduction factor for angle of incidence, β, of the waves (γ_β = 1 for head-on waves).

This equation includes reduction factors (coefficients) that are selected based upon the specifics of the wave environment and the character of the structure. They are applied to account for the influence of surface roughness, γ_r, the presence of a berm γ_b, shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution, γ_h, (as is the case herein), and the angle of incidence of the waves, γ_β.

The surf similarity parameter is defined by CEM Equation VI-5-2 (Reference 2.4.5-228) as,

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{g T_p^{\ 2}}}}$$

Equation 2.4.5-7

Where: α is the slope angle, g is the acceleration due to gravity (32.2 ft/s^2), H_s is the significant wave height, and T_p is the peak wave period.

The limiting wave period from CEM Equation II-2-39 (Reference 2.4.5-229) is calculated as 1.96 seconds. Using the embankment slope of 3H:1V (tan α = 0.3333) and breaking wave height of 0.78 times depth of water (0.78 x 1.3 ft.), the surf similarity parameter, ξ_{op}, is estimated as 1.479.

Because the design wave is a regular, depth-limited wave, and not based on a Rayleigh distribution, a reduction factor γ_h is calculated using CEM Equation VI-5-10 (Reference 2.4.5-228).

$$\gamma_h = \frac{H_{2\%}}{1.4 H_s}$$

Equation 2.4.5-8

As all waves are uniform under the depth-limited condition ($H_{2\%} = H_s$), a reduction factor γ_h is estimated as 0.71. A surface roughness reduction factor, γ_r, of 0.94 is used based on the condition that the embankment slope is covered with grass. Table VI-5-3 (page VI-5-11 of Reference 2.4.5-228) provides a range for γ_r and 0.94 was chosen as the appropriate value for the LNP site. Conservatively using γ_b = 1, γ_β = 1 (for normally incident waves), and a significant wave height H_s of 1.0 ft., the wave runup, $R_{u2\%}$ from Equation 2.4.5-6 is calculated to be 0.45 m (1.48 ft.).

2.4.5.4.9 PMH Water Surface Elevation Using SLOSH Computer Model

Table 2.4.5-223 provides a summary of the total PMSS at the site with the considerations from the maximum PMH surge value at the site as well as contributions from wave setup and wave runup. This table also includes the results from PMH surge with higher initial water level of 1.18 m (3.87 ft.) NAVD88 based on RG 1.59. For all scenarios, the maximum water level remains below the plant floor elevation of 15.5 m (51 ft.) NAVD88 for Units 1 and 2.

Consistent with the purpose and scope, plausible scenarios for the PMH were input into SLOSH. The maximum PMH surge was calculated to be 14.5 m (47.7 ft.) which is the most conservative of all of the peak simulation scenarios (Δp is constant) and includes the initial open-water condition of the 10 percent exceedance spring high tide and the published sea level rise for the next 100 years taken for the nearest tide gauge. Realistic values for wind wave setup have been calculated and take into consideration realistic values for bed friction. Runup was generated based on the specific conditions at the LNP site.

The computed surge elevation of 14.5 m (47.7 ft.) NAVD88 combined with wave setup of 0.2 m (0.6 ft.) and wave runup of 0.5 m (1.48 ft.), results in a maximum water level of 15.2 m (49.78 ft.) NAVD88. This PMH surge elevation is below the plant floor elevation of 15.5 m (51 ft.) NAVD88.

As shown in Table 2.0-201, the DCD plant elevation for the LNP site is 100 ft. which is equivalent to 51 ft. NAVD88. The PMH maximum water surface elevation including wind-wave effect is 15.17 m (49.78 ft.) NAVD88. The highest PMH water surface elevation of 49.78 ft. NAVD88 provides 0.37 m (1.22 ft.) of margin to the DCD Maximum Flood Level site parameter of 51.0 ft. NAVD88.

2.4.5.5 Resonance

The LNP site is located approximately 4.8 km (3 mi.) north of Lake Rousseau and 12.8 km (7.9 mi.) east of the Gulf of Mexico. The adverse effects on the safety-related structures at the LNP site due to the possibility of resonance of oscillations of waves generated either in Lake Rousseau or in the Gulf of Mexico appears unlikely as the resonance induced water column will be quickly dissipated.

2.4.5.6 Protective Structures

All safety-related structures at the plant site are protected from high water levels up to elevation 15.5 m (51 ft.) NAVD88, which is higher than anticipated flood levels due to wave runup associated with the Gulf of Mexico or direct rainfall at the plant site.

2.4.6 PROBABLE MAXIMUM TSUNAMI HAZARDS

2.4.6.1 Probable Maximum Tsunami

According to the NRC:

> The probable maximum tsunami (PMT) is defined as that tsunami for which the impact at a site is derived from the use of best available

scientific information to arrive at a set of scenarios reasonably expected to affect the nuclear power plant site taking into account: (a) appropriate consideration of the most severe of the natural phenomena that have been historically reported or determined from geological and physical data for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, (b) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena, and (c) the importance of the safety functions to be performed (Reference 2.4.6-201).

2.4.6.2 Historical Tsunami Record

Tsunamis are ocean waves generated either by seismic events (such as large earthquakes) or nonseismic disturbances (such as volcanic eruptions or landslides) that occur near or under the ocean. Based on the United States historical tsunami record (Reference 2.4.6-201), tsunamis were induced by:

- Earthquake (71 percent).
- Landslides triggered by earthquakes (13 percent).
- Landslides (10 percent).
- Volcano Eruptions (2 percent).
- Others (4 percent).

These waves, generally characterized by an extremely long wavelength and period, travel out of the area of their origin and can be extremely dangerous and damaging when they reach the shore. As tsunamis are relatively rare, instrumentation dedicated to collecting tsunami data has been slow to be developed. Tide gauges were developed in the 1850s and the first tsunami was recorded on the U.S. West Coast in 1854, after having been generated in Japan (Reference 2.4.6-202).

As noted, the vast majority of tsunamis are produced by earthquakes. The magnitude of an earthquake is a logarithmic measure of the amount of energy released in the form of seismic waves from its epicenter (Reference 2.4.6-203). Magnitude is a useful measure for characterizing earthquakes that are likely to produce destructive tsunamis. However, there are multiple scales used to measure the magnitude of an earthquake, and the following three have been referenced in the literature reviewed for this report: local magnitude (M_L), surface-wave magnitude (M_s), and moment magnitude (M_w).

Local magnitude, also known as the “Richter magnitude,” describes the logarithmic relationship between earthquake size and observed peak ground motion (Reference 2.4.6-204). Based on either S-waves or surface-waves, the local magnitude is generally measured at distances less than 600 km (370 mi.) and depths less than 70 km (43 mi.) (Reference 2.4.6-203). Surface-wave magnitude is a scale based on ground roll waves (a.k.a., Rayleigh waves), and it is useful for more distant observations (Reference 2.4.6-204). Surface-wave magnitude can be determined for earthquakes that are located at distances between 20 and 160 geocentric degrees from a recording station, have a seismic-wave period between 18 and 22 seconds, and have a depth less than 50 km (31 mi.). The moment magnitude is usually similar in value to the surface-wave magnitude. The moment magnitude is a function of the rupture area, fault offset or "slip," and rock strength (as measured by the shear modulus, μ) associated with the earthquake source (Reference 2.4.6-203).

2.4.6.2.1 Sources of Historic Tsunami Data

To summarize the historical tsunamis that affected the gulf coasts of the United States, various sources were explored for relevant information on tsunami-generating sources and wave height and runup events. These data sources include primarily the National Geophysical Data Center (NGDC) tsunami database, Science of Tsunami Hazards journal archives, the USGS, the NOAA Center for Tsunami Research and published literature on historical Caribbean tsunamis.

The maximum height a tsunami reaches on shore is known as the "runup." This is the vertical difference between the msl surface and the maximum height reached by the water on shore. A tsunami is considered particularly dangerous if the resulting runup exceeds 1 m (3.28 ft.). The magnitude of runup is dependent on several factors including how the tsunami's energy is focused at the point of impact, the tsunami's travel path, coastal configuration in the region of impact, and offshore topography. In general, land with steep coastal slopes or barrier reefs experience very little runup, and are only at moderate risk from tsunamis (Reference 2.4.6-205).

The NGDC tsunami database (Reference 2.4.6-206) is a listing of historical tsunami source events and runup locations throughout the world that range in date from 2000 B.C. to the present. The events were gathered from scientific and scholarly sources, regional and worldwide catalogs, tide gauge reports, individual event reports, and unpublished works. In this database, there are currently over 1700 source events. The global distribution of these events is 71 percent Pacific Ocean, 11 percent Mediterranean Sea, 9 percent Atlantic Ocean and Caribbean Sea, 6 percent Indian Ocean, and 3 percent Black Sea. There are over 9600 runup locations where tsunami effects were observed. The global distribution of these locations is 82 percent Pacific Ocean, 6 percent Atlantic Ocean and Caribbean Sea, 3 percent Mediterranean, 9 percent Indian Ocean, and <1 percent in the Red Sea and Black Sea (Reference 2.4.6-206).

The USGS has published a fact sheet on improving earthquake and tsunami warnings for the Caribbean Sea, the Gulf of Mexico, and the Atlantic Coast that includes a map showing the seismology and tectonic setting of the Gulf Coast and Caribbean Region from 1530 to 1991 (Reference 2.4.6-207). The map is reproduced on Figure 2.4.6-201 and it shows the locations of historic earthquake epicenters and the tectonic plate boundaries in the region. NOAA's Center for Tsunami Research, in conjunction with the Pacific Marine Environmental Laboratory, has developed a database that includes worldwide event monitoring and numerical model simulations. NOAA's database includes recent tsunami events (Reference 2.4.6-208). Finally, the publication titled, "A Brief History of Tsunamis in the Caribbean Sea," is a compendium of data and anecdotal material on tsunamis reported in the Caribbean from 1498 to 1997 (Reference 2.4.6-209). Similar historical tsunami events during 1668 to 1998 have been documented elsewhere (Reference 2.4.6-210).

2.4.6.2.2 Observed Historic Tsunami Events Impacting the Caribbean

Reference 2.4.6-209 gives an overview of the tsunami history from 1498 to 1997 in the Caribbean Sea in terms of source events and runup elevations illustrating future expected geologic hazards. Based on this document, tsunamis are a relatively minor hazard in the Caribbean. The record for the last hundred years lists 33 possible tsunamis or 1 about every 3 years. It was observed that the typical recurrence interval for the destructive tsunamis in the Caribbean is about 21 years. The last destructive tsunami in the Caribbean occurred in August 1946, more than 60 years ago. Wave heights of 2.5 m (8.2 ft.) at Matancitas and 4 to 5 m (13.1 to 16.4 ft.) at Julia Molina were reported (Reference 2.4.6-209). This tsunami was generated by an M_s = 7.8, M_w = 8.1 earthquake that occurred about 65 km (40.4-mi.) off the northeast coast of the Dominican Republic. The waves produced by this tsunami were recorded at Daytona Beach, Florida, at Atlantic City, New Jersey, and at Bermuda. The travel time from the earthquake epicenter to Atlantic City was 4.8 hours, and 4.0 hours for Daytona Beach. An aftershock that occurred 4 days later produced a small tsunami that impacted the same areas (Reference 2.4.6-210).

In the Caribbean, there are four source mechanisms that have produced tsunamis in the past: tsunamis from remote sources (teletsunamis), tsunamis generated by mass movements (landslide tsunamis), tsunamis generated by volcanic processes (volcanic tsunamis), and tsunamis produced by earthquakes (tectonic tsunamis) (Reference 2.4.6-209). Table 2.4.6-201 lists verified historic Caribbean tsunamis from 1498 to 2000 in terms of their origin and impacted locations (Reference 2.4.6-209). Based on this data, it can be stated that historically no Caribbean tsunami has resulted in significant danger to the United States Gulf Coast. Thus, it is unlikely that any particularly dangerous tsunami generated in the Caribbean Sea will impact the Gulf Coast of northern-central Florida where the LNP site is located.

2.4.6.2.3 Observed Historic Tsunami Events Impacting the Gulf Coast

In the recorded history, tsunami waves recorded along the Gulf Coast have all been less than 1 m (3.28 ft.) (Reference 2.4.6-211). Tables 2.4.6-202 and Table 2.4.6-203 list various tsunami events that have affected the Caribbean and gulf coasts, respectively, as indicated by the NGDC tsunami database (Reference 2.4.6-206). These records include a tsunami event generated by the

1964 Gulf of Alaska earthquake. Though this event was recorded in both Louisiana and Texas, the waves that impacted these locations are technically termed a "seiche." As defined in FSAR Subsection 2.4.5.2.6, a seiche is an oscillation of a water body and is most commonly recognized in tsunamis as a standing wave. Though most often caused by atmospheric disturbances, they can also be generated by the ground motion associated with earthquakes. There are multiple early 20th-century reports of tsunami waves from Caribbean earthquakes along the Gulf Coast that are difficult to evaluate, but in each case, the maximum wave heights generated appeared to be less than 1 m (3.28 ft.) (Reference 2.4.6-211).

Three historical tsunamis have been documented for the Gulf Coast in the available tsunami databases and literature referenced above. The first of these tsunami events occurred on October 24, 1918, when a small wave was recorded at the Galveston, Texas, tide gauge. This tsunami was presumed to be the result of an aftershock of an October 11, 1918, earthquake (Reference 2.4.6-209). The M_s = 7.5, M_w = 7.3 earthquake originated in the Mona Passage, approximately 15 km (9.3 mi.) northwest of Puerto Rico (Reference 2.4.6-210). It was likely the result of subduction activity in the Brownson Deep. The initial earthquake produced a tsunami that reached a runup height of 6 m (19.7 ft.), and caused significant damage in Puerto Rico (Reference 2.4.6-209). The October 24 tsunami event has a validity rating in the NGDC database of four on a scale from zero to four, where zero and one are used for erroneous or very doubtful events, respectively, and four is used for definite events (Reference 2.4.6-209). The magnitude of the runup of this tsunami in the Gulf Coast was not reported.

The second documented tsunami event in the Gulf occurred on May 2, 1922 (Reference 2.4.6-209). The epicenter of the earthquake associated with this event was near Isla de Vieques, Puerto Rico. Four hours after the occurrence of the earthquake, a wave with an amplitude of 0.6 m (1.97 ft.) was reported on a tide gauge at Galveston, Texas. A train of three waves with a 45-minute period that were followed 8 hours later by a similar train of smaller waves was observed (Reference 2.4.6-209). However, the validity rating of this event in the NGDC database is a two (i.e., doubtful). The magnitude of the 1922 earthquake and the aftershock has not been estimated.

The third reported tsunami event that impacted the Gulf occurred on March 27, 1964, and was recorded throughout the Gulf of Mexico (Reference 2.4.6-206). This event coincided with the 1964 Alaska earthquake (M_w = 9.2) earthquake that originated in Prince William Sound, Alaska (Reference 2.4.6-212). The earthquake resulted in a vertical displacement ranging from 11.5 m (37.7 ft.) of uplift to 2.3 m (7.5 ft.) of subsidence over more than 520,000 km^2 (200,800 mi.^{2}) of land in the region of origin. While 15 people were killed in the initial earthquake, another 110 lost their lives in the tsunami that followed. The maximum wave height measured was 67 m (220 ft.) at Valdez Inlet. The majority of tsunami damage occurred in the Gulf of Alaska and along the west coast of the United States. According to the USGS, the resultant "Seiche action in rivers, lakes, bayous, and protected harbors and waterways along the Gulf Coast of Louisiana and Texas caused minor damage" (Reference 2.4.6-212). The validity of this event is a four (Reference 2.4.6-206).

2.4.6.3 Source Generator Characteristics

2.4.6.3.1 Tsunamigenic Source Mechanisms

Historically, 71 percent of tsunamis striking the United States have been induced by earthquakes (Reference 2.4.6-201). Considering all source mechanisms, the most destructive tsunamis are the result of large, shallow earthquakes with an epicenter or fault line near the ocean floor. Large earthquakes can tilt, offset, or otherwise displace large areas of ocean floor for distances ranging from a few kilometers to 1000 km (621 mi.) or more. When large vertical offsets occur, these earthquakes also displace water and produce destructive tsunami waves. Tsunami waves can travel large distances from their source. For example, in 1960, there was an earthquake off the coast of Chile with a magnitude of M_W = 9.5 (M_S = 8.6) and a rupture zone of 1000 km (621 mi.). This earthquake produced the Great 1960 Chilean tsunami, as well as destructive waves that hit Hawaii, Japan, and other locations in the Pacific (Reference 2.4.6-205).

Though less common, tsunami events can also result from rock falls, icefalls, and sudden submarine translational landslides or rotational slumps (Reference 2.4.6-205). Historically, 23 percent of tsunamis striking the United States have been the result of landslides (Reference 2.4.6-201). These events are caused by sudden failures of submarine slopes, which are often triggered by earthquakes. In the 1980s, construction work along the coast of Southern France triggered an underwater landslide that produced destructive tsunami waves in the harbor of Thebes (Reference 2.4.6-205). It is also thought that a 1998 earthquake triggered a large underwater slump of sediments, which produced a tsunami that destroyed coastal villages and killed thousand of people along the northern coast of Papua, New Guinea.

A description of historical tsunami records is presented in FSAR Subsection 2.4.6.1. Based on an extensive literature search and site-specific borings at LNP (FSAR Section 2.5), no geologic evidence of paleo-tsunami or tsunami-like deposits or geologically conducive locations for deposition were found in the vicinity of the Levy County site or in nearby coastal regions. There are no permanent slopes or hill slopes present on the LNP site (FSAR Subsection 2.5.5) nor within the coastal areas near the site that could adversely affect safety-related structures from local landslides. Potential tsunamis from offshore landslides are evaluated later in this section.

Volcano-induced tsunamis are rare, and account for only about 2 percent of tsunami events impacting the United States (Reference 2.4.6-201). However, like landslides, volcanic eruptions are impulsive disturbances, and they are capable of displacing large volumes of water and producing extremely destructive tsunami waves in the area in close proximity to their source. Volcanoes can produce tsunamis by one of three methods. According to the International Tsunami Information Center, "waves may be generated by the sudden displacement of water caused by a volcanic explosion, by a volcano's slope failure, or more likely by a phreatomagmatic explosion and collapse/engulfment of the volcanic magmatic chambers." The 1883 explosion and collapse of the Indonesian volcano Krakatoa produced one of the largest and most destructive tsunamis ever recorded. The resulting tsunami waves reached a height of 41.15 m (135 ft.), and resulted in significant damage to property and loss of human life. A similar explosion and collapse of the volcano Santorin in the Aegean Sea may have produced a tsunami that destroyed Greece's Minoan civilization in 1490 B.C. (Reference 2.4.6-205).

Most meteorites burn up within the atmosphere and no asteroid has fallen during recorded history. However, large craters are evidence that large meteorites have struck the Earth's surface in ancient history, and it is possible that a large asteroid fell on Earth sometime during the Cretaceous period, 65 million years ago. Given that water covers four-fifths of the planet's surface, falling asteroids and meteorites have a good chance of impacting oceans and seas. According to the International Tsunami Information Center, "The fall of meteorites or asteroids in the earth's oceans has the potential of generating tsunamis of cataclysmic proportions." The impact of a moderately sized asteroid, 5 to 6 km (3.1 to 3.7 mi.) in diameter, in the Atlantic Ocean could produce a tsunami that would destroy Atlantic Coast cities and travel to the Appalachian Mountains in the northern two-thirds of the United States (Reference 2.4.6-205). Meteorites and asteroids are potential tsunamigenic sources; however, the occurrence of such an event is highly unlikely.

It is believed that a large nuclear explosion could also serve as a tsunamigenic source. However, no significant tsunami has been reported as the result of nuclear testing, which is currently banned by international treaty (Reference 2.4.6-205).

2.4.6.3.2 Locations of Tsunamigenic Sources

Two historic tsunamigenic sources have been observed and documented in the Gulf of Mexico. The historic tsunamigenic sources include seismic events originating in the North Caribbean Sea and in the Aleutian Trench in Alaska (Reference 2.4.6-206). In addition, simulations indicate that an historic earthquake originating in the Azores-Gibraltar fracture zone (near Lisbon, Portugal), may have also produced a tsunami that reached the Gulf Coast (Reference 2.4.6-213). All three of these sources are far-field sources, or more than 1000 km (620 mi.) from the Levy County coastline.

According to a recent report prepared by the USGS for the NRC (Reference 2.4.6-214), far-field submarine landslides may also be potential tsunamigenic sources. However, local tsunamis are generally much more destructive than tsunamis generated from a distant source. In addition, they may occur within minutes of the earthquake or landslide that produces them, allowing little time for evacuation. Both the Caribbean region and the Gulf of Mexico provide potential local tsunamigenic sources, as discussed in detail in the following subsections.

2.4.6.3.2.1 Tsunamigenic Sources in the Caribbean

As shown on Figure 2.4.6-202, the Caribbean region, in particular, is characterized by high seismic activity and is prone to strong, shallow earthquakes capable of generating tsunamis (Reference 2.4.6-215). This may occur as a result of the movement of the Caribbean plate. The Caribbean plate underlies a region bounded by Honduras, Nicaragua, Costa Rica, Panama, Colombia, Venezuela, the Lesser Antilles, Puerto Rico, Hispaniola, and Jamaica. The plate moves semi-independently of its surrounding plates, which include the North American, South American, and Cocos plates (Figure 2.4.6-203) (Reference 2.4.6-209). As the Caribbean plate moves eastward (and slightly north of eastward), the two American plates are driven under, or subducted, beneath its eastern edge. This process has produced a chain of active volcanoes along the Lesser Antilles, which may also prove to be an additional tsunamigenic source. In addition, the Cocos plate is moving northeastward and is being subducted beneath the western edge of the Caribbean plate. The strain of the Caribbean plate against the surrounding plates results in a band of high earthquake potential that surrounds the plate (Reference 2.4.6-209). The type of vertical displacement of the ocean floor necessary for the generation of a tsunami event can readily occur in this region (Reference 2.4.6-215).

The Caribbean Sea is bordered in the east by the Lesser Antilles islands, an active volcanic island arc and potential tsunamigenic source. Large submarine landslides are also potential tsunamigenic sources in the Caribbean Sea (Figure 2.4.6-204) (Reference 2.4.6-207). Many of the Caribbean islands are characterized by steep slopes in relatively shallow waters, and are prone to submarine slides and slumps as a result (Reference 2.4.6-215). In particular, side-scan sonar has revealed unusual submarine formations north of Puerto Rico, which are likely the result of slumping. Such events are capable of producing destructive tsunamis (Reference 2.4.6-209).

2.4.6.3.2.2 Tsunamigenic Sources in the Gulf of Mexico

Though no documented tsunami has originated within the Gulf of Mexico (Reference 2.4.6-206), potential tsunamigenic sources exist. In particular, the southern portion of the Gulf of Mexico is characterized by historic seismic activity (Figure 2.4.6-201). According to the USGS National Earthquake Information Center, earthquakes originating beneath the center of the Gulf of Mexico have also been reported. In the past 30 years, more than a dozen such events have been recorded from the eastern Gulf of Mexico. The most recent of these occurred on September 10, 2006, originating 405 km (250 mi.) south-southwest of Apalachicola, Florida (26.331°N, 86.577°W). This earthquake had a magnitude of M_W = 5.8, and is the largest on record for the eastern Gulf of Mexico. Though the quake caused no damage to life or property, it was felt in parts of Florida, Georgia, Alabama, Kentucky, Louisiana, Mississippi, North Carolina, South Carolina, Tennessee and Texas, as well as in the Bahamas and at Cancun and Merida, Mexico (Figure 2.4.6-205). The earthquake also produced seiches in swimming pools in Florida. Prior to the September quake, the region's last similar quake occurred on February 10, 2006, with a magnitude of $M_W = 5.2$ (Reference 2.4.6-216).

Though the Gulf of Mexico is characterized by frequent landslide events, they have not been a source of tsunamis that have been documented instrumentally or in the geologic record for the Gulf Coast (Reference 2.4.6-206). However, in the 1960s, the petroleum industry discovered a large, potentially tsunamigenic, submarine slump site in the northwest corner of the Gulf of Mexico, which was later mapped by the USGS (Reference 2.4.6-217). Known as the East Breaks slump, the site is over 1000 km (620 mi.) from the LNP site, and is marked by a 20 km (12.4 mi.) indentation in the shelf edge (Reference 2.4.6-217). Beryhill et al. estimated that the slump occurred 5000 to 10,000 years ago (Reference 2.4.6-217); however, McGregor suggests that it may have occurred as long ago as 15,000 to 20,000 years (References 2.4.6-218 and 2.4.6-219). The total estimated volume of the slide is 50 to 60 cubic kilometers (km^3) (12 to 14.4 cubic miles [$mi.^3$]), and the slump deposits cover more than 3200 km^2 (1200 $mi.^2$) (Reference 2.4.6-217).

A recent study conducted by the USGS (Reference 2.4.6-214) summarized the threat of landslides in the Gulf of Mexico as follows (Figure 2.4.6-206):

> Landslides in the Gulf of Mexico occur in all three depositional provinces (carbonate, salt, and canyon/fan). The largest failures are found in the canyon/fan province. . . . Available information suggests they occurred during the early part of the Holocene (10,000 - 15,000 yr BP). The resumption of hemipelagic sedimentation in the head of Mississippi Canyon at 7500 yr BP indicates that at least the largest of these landslide complexes had ceased being active by mid-Holocene time.
>
> Landslides within the salt province are in general considerably smaller than those in the canyon/fan province, many of them are confined to the walls of mini-basins, but some occupy the Sigsbee escarpment. These landslides appear to be active and are driven by salt creep. Landslides in the carbonate provinces that fringe the eastern and southern Gulf of Mexico appear to have been derived from both the steep West Florida and Campeche Escarpments as well as from the gentler slope above the escarpments. The northern part of the Florida Escarpment has probably undergone little erosion since it originally formed during the Cretaceous, but the southern part of the Florida Escarpment shows sign of active erosion.

2.4.6.4 Propagation of Tsunami Waves

2.4.6.4.1 Propagation in Deep Waters

Tsunami waves travel on the surface of the ocean outward and away from the source in all directions (Reference 2.4.6-205). A tsunami is composed of a series of large amplitude, shallow-water (wavelength, $\lambda > 20\ H$, where H is water depth) gravity waves. Even when generated in deep water, tsunamis are considered shallow-water waves because typical wavelengths can exceed 200 km (120 mi.), while ocean depths are only a few kilometers. The depth of the Caribbean Sea, for example, is 2.6 km (1.6 mi.) (Reference 2.4.6-224). Tsunami waves are generally long, though the actual wavelength and period of tsunami waves are dependent on the source mechanism and its characteristics. For example, tsunamis originating from a large earthquake over a large area will have a much larger initial wavelength and period than a tsunami generated by a local landslide. Tsunami wave crests can span lengths up to 1000 km (621.4 mi.). The period of tsunami waves typically ranges from 5 to 90 minutes. The height of a tsunami wave on the deep ocean will depend on its source, but it will generally be anywhere from a few centimeters to a meter or more (Reference 2.4.6-205).

When a tsunami is produced, its energy is distributed throughout the water column, regardless of the ocean's depth. In fact, tsunami waves in the deep ocean can travel at high speeds over large distances and lengths of time without losing much energy (Reference 2.4.6-205). According to the USGS (Reference 2.4.6-224), "Tsunamis propagate at the shallow water gravity wave phase speed of $c = (gH)^{1/2}$, which can be in excess of 222 m/s (~ 800 [kilometers per hour] km/hr) [500 miles per hour {mph}], until they dissipate or encounter a shelf and shallow coastal water where they slow to 8 to 14 m/s (~ 30 to 50 km/hr) [19 to 31 mph]". This indicates that tsunamis travel fastest in deep waters. At the deepest ocean depths, a tsunami wave can travel at 800 km/h (497.1 mph), which is comparable to the speed of a jet aircraft (Reference 2.4.6-205).

2.4.6.4.2 Propagation in Shallow Waters

When tsunamis enter shallow waters typically found at coastlines, bays, or harbors, their speed decreases to 50 to 60 km/h (31 to 37 mph). At the same time, waves farther from shore and in deeper waters are traveling toward the same area at much greater speeds. The waves are compressed in the shallow water such that the wavelength is shortened. Because the wave energy is then applied over a smaller volume of water and the wave energy is directed upward, the waves grow in height. Tsunamis generally arrive on shore with a wavelength exceeding 10 km (6.2 mi.), and successive wave crests arrive anywhere from 10 to 45 minutes apart. However, the flooding from a single wave can last from 10 minutes to 30 minutes, such that the period of danger during a tsunami event might exceed several hours (Reference 2.4.6-205).

Features such as water depth and coastal configuration may cause a significant amount of wave refraction, which can also serve to converge energy and increase wave heights. In addition, a tsunami that occurs during high tide or a localized storm will produce cumulative effects resulting in an even more severe event. A 1-m (3.28-ft.) tsunami in the deep ocean may transform into a 30 to 35 m (98 to 115 ft.) wave onshore. Such waves can cause extensive damage to life and property (Reference 2.4.6-205).

2.4.6.5 Analysis of Historical Tsunami Events

Any of the world's oceans, inland seas, and large bodies of water may experience a tsunami event. The majority of the tsunamis occur in the Pacific Ocean as a result of its large size (more than one-third of the Earth's surface) and the presence of highly active seismic regions. Less common, destructive tsunamis have also originated in the Atlantic and Indian oceans, in the Mediterranean Sea, and within smaller bodies of water, such as the Caribbean Sea. According to the International Tsunami Information Center, "In the last decade alone, destructive tsunamis have occurred in Nicaragua (1992), Indonesia (1992, 1994, 1996), Japan (1993), Philippines (1994), Mexico (1995), Peru (1996, 2001), Papua-New Guinea (1998), Turkey (1999), and Vanuatu (1999)" (Reference 2.4.6-205).

However, according to NOAA's NGDC, only a small percentage of all tsunami events are destructive (Figure 2.4.6-207) (Reference 2.4.6-220). In the past 500 years, 100 tsunamis have been reported in the Caribbean region, only 30 of which caused significant damage (Reference 2.4.6-207). Many of these events were the result of large shallow earthquakes in the Caribbean (Reference 2.4.6-209). Not all earthquakes generate tsunamis, but thrust, reverse, or normal faulting earthquakes of magnitude, $M_W \geq 6.5$, which deform or rupture the seafloor are the most likely to produce tsunamis (Reference 2.4.6-215). The Richter magnitude of an earthquake must generally exceed $M_L = 7.5$ if it is to produce a destructive tsunami. Considering all source mechanisms, the most destructive tsunamis are the result of large, shallow earthquakes with an epicenter or fault line near the ocean floor. Such events are common in highly seismic regions characterized by the collision of tectonic plates, and in particular, by tectonic subduction (Reference 2.4.6-205).

2.4.6.5.1 Historical Tsunamis Generated in the Caribbean

The North Panama Deformation Belt (9-12 degrees [º]N, 83ºW-77ºW) and the Northern South America Convergence Zone (11.5°-14°N, 77°W-64°W) lie in the southern portion of the Caribbean region (Figure 2.4.6-208) (Reference 2.4.6-214). The North Panama Deformation Belt has a relatively slow rate of convergence (~7-11 millimeter per year [mm/yr]), resulting in a long recurrence interval for large earthquakes (Reference 2.4.6-214). However, in 1882 the source produced an $M_W = 8.0$ event known as the 1882 Panama Earthquake (Figure 2.4.6-209). Despite significant local tsunami damage, there is no report of a tsunami impacting the Gulf Coast as a result of this earthquake (Reference 2.4.6-214). In a thorough review of the seismic threats in the southern Caribbean (Figure 2.4.6-210), the USGS concluded that sources in this region, "…do not appear to be capable of generating very large earthquakes, and thus do not appear to pose a significant tsunami hazard to the Gulf of Mexico coastal zones" (Reference 2.4.6-214).

The Puerto Rico Trench and Hispaniola Trench both lie in the northern portion of the Caribbean region. Though there is no historical record of large earthquakes originating from the Puerto Rico Trench (Figure 2.4.6-211), it is a potentially tsunamigenic seismic source. The largest instrumentally recorded event occurred in 1943 and was an M_W = 7.3 earthquake, though McCann suggests that a 1787 earthquake north of Puerto Rico had a magnitude, M_W = 8.0 (Reference 2.4.6-214). The Hispaniola Trench (Figure 2.4.6-212) also poses a tsunamigenic threat. In fact, relatively larger earthquakes and more vertical motion are expected for the Hispaniola Trench than for the Puerto Rico Trench (Reference 2.4.6-214). Several earthquakes of magnitude M_S = 7.0 to 8.1 (M_W 6.8 to 7.6) occurred between 1946 and 1953 in northern Hispaniola. A destructive local tsunami was produced by one of the 1946 earthquakes (Reference 2.4.6-214).

McCann (Reference 2.4.6-221) examined the seismic reflection records of Western Puerto Rico to characterize tsunamigenic faults that may have a potential to generate tsunamic hazards. He found that the Mona Passage, which is a segment of island arc crust lying between the islands of Puerto Rico and Hispaniola, has many active faults as a result of rapid extension. While some of these active faults are capable of producing earthquakes as large as magnitude M_W = 7, most are relatively short in length and are probably only capable of producing events in the magnitude M_W = 6 ranges. However, of the 84 active faults, 30 should be considered potentially tsunamigenic, as they are considered capable of producing events of magnitude M_W = 6.5 or larger (Reference 2.4.6-221).

According to the USGS (Reference 2.4.6-214),

> The northern Caribbean subduction zone has the potential to cause a major tsunami similar to the 2004 Sumatra tsunami. However, detailed work in the Puerto Rico Trench indicates that slip there is highly oblique and the subducting lithosphere is very old, two indications that perhaps the subduction zone is not capable of generating very large earthquakes. The Hispaniola segment of this subduction zone, while perhaps capable of very large earthquakes, is fringed to the north by an almost continuous line of islands and shallow banks that obstruct, but not completely block, propagating tsunami waves.

It is believed that major earthquakes produce many underwater landslides. However, the energy of tsunamis generated by submarine landslides and slumps is thought to dissipate rapidly as the waves travel across the ocean or even within partially enclosed water bodies such as lakes and fjords (Reference 2.4.6-205). Due to their steep slopes and shallow waters, many islands in the Caribbean region are prone to landslides and slumps

(Reference 2.4.6-215). Such an event likely occurred just north of Puerto Rico, and may have been the result of ongoing plate-tectonic movement that tilted the seafloor 4° down to the north. This tilt may have caused massive slope failures that are continuing, as evidenced by large fissures in the seafloor (Figure 2.4.6-204) (Reference 2.4.6-207).

2.4.6.5.2 Historical Tsunamis Generated in the Gulf of Mexico

In contrast to the Caribbean, the Gulf of Mexico contains no subduction zone faults that are a primary source of large, tsunamigenic earthquakes (Reference 2.4.6-214). The Gulf of Mexico has produced some notable earthquakes in the recent past. The most recent and largest event occurred in September of 2006 and had a magnitude of M_W = 5.8. However, most of these events have been produced at locations distant from faults and plate boundaries, and are generally known as “midplate” earthquakes. These events result from the release of long-term tectonic stresses that originated from forces applied at the plate boundary, and they are generally rare. With regard to the most recent and largest seismic event in the Gulf, the USGS National Earthquake Information Center suggests that, “Earthquakes of this magnitude are unlikely to generate destructive tsunami. No significant tsunami was generated by this earthquake” (Reference 2.4.6-216). Given the infrequent occurrence and modest magnitude of “midplate” seismic events, there is little likelihood that a seismic event in the Gulf of Mexico would produce a tsunami.

Though the Gulf of Mexico is also characterized by frequent translational landslides, no tsunamis originating from this source have been documented instrumentally or in the geologic record for the Gulf Coast (Reference 2.4.6-206). Trabant et al. (Reference 2.4.6-217) performed a preliminary calculation to estimate the offshore wave-height of the tsunami associated with the historic East Breaks slump at 7.6 m (25 ft.). It should be noted that this calculation has not been supported by subsequent publication, and there is no documented geologic evidence of the impact of such a wave along the Gulf Coast.

The closest volcanoes to the LNP site are located in the southwestern region of the Gulf of Mexico (Figure 2.4.6-201) (Reference 2.4.6-207) and are also abundant in the eastern Caribbean Sea among the Lesser Antilles. However, no tsunamis as a result of recent volcanic eruptions or associated mass wasting events have been documented in the Gulf of Mexico (Reference 2.4.6-206).

2.4.6.5.3 Historical Tsunamis Generated from Sources Situated Other than the Gulf of Mexico and Caribbean

Potential far-field tsunamigenic sources include the Aleutian Trench in Alaska, the Azores - Gibraltar fracture zone (near Lisbon, Portugal), and far-field submarine landslides. As previously discussed, the impact of the tsunami generated by the 1964 Alaska earthquake was less than 1 m (3.28 ft.) in the Gulf of Mexico. The potential threat of the Azores - Gibraltar fraction zone as a tsunamigenic source is discussed in FSAR Subsection 2.4.6.6.1.

In a report to the NRC (Reference 2.4.6-214), the USGS summarizes the tsunamigenic threat of far-field submarine landslides to the Gulf and Atlantic Coasts as follows:

> Far-field submarine landslide sources have been quoted as potential sources for trans-oceanic tsunamis. The most widely known is the threat of a large-volume landslide caused by an imminent eruption of Cumbre Vieja volcano in the Canary Island. However, models of tsunami propagation, which take into account dispersion and nonlinearity of the landslide-generated waves, show rapid amplitude decay with distance and predict <1 meter of flooding in Florida. In addition, the recurrence time of a major eruption-related landslide is 10^5 yr. The giant Storegga landslide offshore Norway caused large tsunami waves within 600 km radius in the northeast Atlantic, but the waves are not known to have propagated to the U.S. East Coast. Some large landslides have been identified along the Scotian margin north off New England. Most of them are Holocene and older in age and appear to be related to the expansion and contraction of the Laurentide ice sheet. The 1929 Grand Banks landslide generated a damaging tsunami locally, but not in New England. However, larger landslides than the 1929 Grand Banks landslide have been identified in the stratigraphic record.

Impulsive events such as meteor and asteroid strikes and nuclear explosions are unlikely, but are potential near-field and far-field tsunamigenic sources.

2.4.6.6 Tsunami Water Levels

2.4.6.6.1 Water Levels Due to Simulated Historic Earthquakes

In addition to the recorded events in the Gulf of Mexico, numerical simulations of tsunamis generated by historic earthquakes provide additional insight into the potential tsunami hazards in the Gulf of Mexico. Wave generation and propagation modeling of the tsunami generated by the 1755 Lisbon (M_w = 8.7) earthquake was conducted using the nonlinear long wave equations and a 10-minute Mercator grid for the Atlantic Ocean (Figure 2.4.6-212). The modeling predicted a teletsunami (i.e., a tsunami from a source over 1000 km [621.4 mi.] away) arriving in the Caribbean and entering the Gulf of Mexico (Reference 2.4.6-213). Mader states

> …the east coast of the U.S.A. and the Caribbean [would] receive a tsunami wave offshore in deep water about two meters [6.6 ft.] high with periods of 1.25 to 1.5 hours. Such a wave would give waves along the shore about 10 feet [3.0 m] high with Saba being unique with about a 20 feet [6.1 m] high wave after run-up. After the wave travels into the Gulf of Mexico the wave amplitudes are less than one meter (Reference 2.4.6-213).

2.4.6.6.2 Water Levels Due to Simulated Potential Tsunamigenic Seismic Events Identified by NOAA

The NOAA West Coast and Alaska Tsunami Warning Center has identified four seismic tsunamigenic sources that could potentially produce “worst-case” impacts for the Gulf Coast. These locations were selected based on the results of numerical models that simulated hypothetical tsunamis originating in the Atlantic Ocean, Gulf of Mexico, and Caribbean Sea (Reference 2.4.6-225). The four sources are shown on Figure 2.4.6-213. The point of origin and magnitude of the four earthquakes are as follows: 1) the Puerto Rico Trench (66°W, 18°N, M_w = 9.0); 2) the Caribbean Sea (85°W, 21°N, M_w = 8.2) from the Swan fault to the mouth of the Gulf of Mexico near Cancun; 3) the North Panama Deformed Belt (66°W, 12°N, M_w = 9.0); and 4) the Gulf of Mexico off the coast of Veracruz, Mexico (95°W, 20°N, M_w = 8.2) (Reference 2.4.6-225).

According to Knight (Reference 2.4.6-225), a simulated tsunami generated by a seismic event (M_w = 9.0) in the Puerto Rico Trench (Figure 2.4.6-214) has unique impacts on the Gulf and Atlantic coasts, respectively. The tsunami resulting from the specified seismic event impacts the Atlantic Coast with an amplitude exceeding 150 centimeters (cm) (59 inches [in.]), and with a leading edge elevation. In contrast, the tsunami waves impacting the Gulf Coast as a result of the same event have much lower amplitudes (less than 25 cm [9.8 in.]), and are characterized by a leading edge depression (Figure 2.4.6-215) (Reference 2.4.6-225).

For a tsunami generated outside of the Gulf of Mexico to impact lands in the Gulf, such as from the Puerto Rico Trench source, it must travel through either the Caribbean Sea or the Straits of Florida. According to Knight’s analysis using Kowalick and Murty’s energy flux vector,

$$\rho d\vec{V}\left(g\zeta + \frac{1}{2}V^2\right), \qquad \text{Equation 2.4.6-1}$$

the path via the Caribbean is 1 hour faster, but more energy reaches the Gulf of Mexico via the Straits of Florida than via the Caribbean pathway (Reference 2.4.6-225). A separate analysis of energy losses due to friction suggests that significant energy losses occur in the Caribbean region due to bottom friction (Figure 2.4.6-216). A dissipation curve representing these energy losses due to bottom friction is shown on Figure 2.4.6-217.

Knight’s analysis also suggests that roughly 10 times more energy flows into the Atlantic Ocean than into the Caribbean as a result of the tsunami originating at the Puerto Rico Trench (Reference 2.4.6-225). This is supported by work presented by Maul, as shown on Figure 2.4.6-218 (Reference 2.4.6-226). A summary of the peak wave elevations generated by a tsunami originating at the Puerto Rico Trench is shown in Table 2.4.6-204. The resulting wave amplitudes at Gulf Coast locations are 25 cm (10 in.) or less (Reference 2.4.6-225).

Similar analyses were performed for the remaining three sources indicated by NOAA to be capable of generating "worse-case" events. The initial amplitudes of a tsunami generated by a seismic event (M_w = 8.2) in the Caribbean Sea near Cancun are shown on Figure 2.4.6-219. The resulting tsunami amplitude is less than 30 cm (12 in.) in the Gulf Coast, again as a result of significant energy losses due to bottom friction in the Caribbean Sea (Reference 2.4.6-225).

The initial amplitudes of a tsunami generated by a seismic event (M_w = 9.0) near Venezuela are shown on Figure 2.4.6-220. In this case, Gulf Coast impacts are mitigated due to energy losses via bottom friction, and over time, as a result of multiple wave reflections in the Caribbean Sea. The resulting maximum tsunami waves in the Gulf Coast are less than 15 cm (5.9 in.), while Atlantic Coast waves are under 50 cm (20 in.) (Reference 2.4.6-225).

The NOAA West Coast and Alaska Tsunami Warning Center suggests that sources outside of the Gulf of Mexico will not likely produce a tsunami capable of damaging the Gulf Coast. There are only two paths available to a tsunami originating outside of the Gulf of Mexico — through the narrow Straits of Florida and through the Caribbean. In both cases, the tsunami's energy losses due to bottom friction would be significant. As a result, the Gulf Coast is effectively shielded from sources outside of the Gulf of Mexico (Reference 2.4.6-225).

The scenario of an earthquake in the Gulf of Mexico off the coast of Veracruz, Mexico, is based on a hypothetical scenario. This region has been seismically active historically. The initial amplitudes of a tsunami generated by a seismic event (M_w = 8.2) in the Gulf of Mexico near Veracruz are shown on Figure 2.4.6-221. Most of the energy produced in this event is confined to the Gulf of Mexico. The resulting tsunami impacting the Gulf Coast would have amplitudes less than 35 cm (14 in.) (Reference 2.4.6-225).

In 2007, the USGS conducted a preliminary analysis (Reference 2.4.6-214) of tsunami threats to the United States Gulf and Atlantic coasts as a compliment to Knight's study (Reference 2.4.6-225). The USGS evaluated the tsunami threat of the following five seismic sources in the Caribbean: 1) the west Cayman oceanic transform fault (OTF), also known as Swan Island fault; 2) the east Cayman (OTF), also known as Oriente fault; 3) the northern Puerto Rico/Lesser Antilles subduction zone (SUB); 4) the north Panama deformation belt, classified by Bird (Reference 2.4.6-227) as an oceanic convergent boundary (OCB); and 5) the north coast of South America convergence zone classified by Bird (Reference 2.4.6-227) as a subduction zone (SUB) (termed the north Venezuela subduction zone below) (Reference 2.4.6-214). A classification scheme by Bird (Reference 2.4.6-227) was used to determine the most likely maximum rupture length and earthquake magnitude associated with each of the sources (Reference 2.4.6-214). The range of magnitude and average slip for each fault source are presented in Table 2.4.6-205.

In the USGS study, "Tsunami propagation was modeled using the linear long-wave equation, numerically implemented with a leap-frog, finite-difference algorithm." Because linear theory is most readily applied to deep-ocean tsunami propagation, the model was limited to a depth of 250 m (820.2 ft.), such that propagation across the continental shelf and wave runup were not modeled. However, the study suggests that runup can be approximated as 3 times the tsunami amplitude at a water depth of 250 m (820.2 ft.), accounting for shoaling and runup amplification. This approximation does not include energy dissipation from geometric spreading, bottom friction, and nonlinear attenuation. The model's spatial grid size is 2 arc-minutes. Tsunami propagation was modeled using an 8-second time step, for a total simulation time of 4.4 to 6.6 hours (Reference 2.4.6-214).

The simulation results for each source are presented on Figures 2.4.6-222, 2.4.6-223, 2.4.6-224, 2.4.6-225, and 2.4.6-226, which show the maximum open ocean tsunami amplitude out of 100 simulations for each source. The results suggest that the transform faults (OTF) are much less efficient at generating tsunami waves (Figures 2.4.6-222 and 2.4.6-223) than the thrust faults along subduction zones (SUB) and oceanic convergent boundaries (OCB) (Figures 2.4.6-224, 2.4.6-225, and 2.4.6-226) (Reference 2.4.6-214). In general, within the Gulf of Mexico tsunami amplitudes are highest "where the shelf edge is approximately normal to the incidence of tsunami waves propagating from the south (i.e., between ~83-85°W and ~87.5-88.5°W)". A time series analysis indicates that, with the exception of the northern Puerto Rico subduction zone scenario, tsunami onset is "emergent" in the Gulf of Mexico, such that initial tsunami waves are smaller than some of those that come after. This is due, in part, to the natural obstructions to the wave propagation (Reference 2.4.6-214).

Figure 2.4.6-227 shows the range of peak offshore tsunami amplitudes from all 100 simulations at the 250-m (820.2-ft.) isobath for a latitudinal profile in the Gulf of Mexico. The maximum tsunami wave height is roughly 0.65 m (2.1 ft.), and is generated by the Venezuela subduction zone scenario (Reference 2.4.6-214). Though impacts will vary because of nearshore propagation and runup effects, we can estimate the maximum runup height at 2.0 m (6.6 ft.) (3 x 0.65 m = 1.95 m ~ 2.0 m). The maximum tsunami wave height generated by the other four sources is less than 0.25 m (10 in.), suggesting a maximum runup height of less than 0.75 m (2.5 ft.).

The simulation results indicate that the most severe impacts for the Gulf Coast are the result of large earthquakes along the north Venezuela subduction zone. However, it should be noted that the resulting tsunami amplitudes were heavily dependent on the seismic event magnitude for each source. The USGS (Reference 2.4.6-214) summarizes its findings as follows:

> In general, these results are consistent with the findings of Knight (2006) [Reference 2.4.6-225], where the far-field tsunamis generated from earthquakes located beneath the Caribbean Sea are higher along the Gulf coast than the Atlantic coast because of dissipation through the Greater Antilles islands. Conversely, tsunamis generated from earthquakes north of the Greater Antilles are higher along the Atlantic coast than the Gulf coast.

The USGS study concludes by suggesting that more refined hydrodynamic modeling is needed in order to determine the potential tsunami impact on the Gulf Coast due to tsunami events generated by the seismic sources in the Caribbean (Reference 2.4.6-220).

2.4.6.7 Probable Maximum Tsunami Analysis

Three different tsunami sources are considered for the PMT analysis: one seismic source and two landslide sources.

- Venezuela Seismic Source
- Mississippi Canyon Landslide Source
- Florida Escarpment Landslide Source

For the seismic source, the initial condition consists of a static surface displacement and a stationary water body. The initial static displacement is derived from earthquake source parameters using the method described by Okada (Reference 2.4.6-239).

For the two landslide sources, two approaches are considered. The first approach uses a static source based on the geometry of the initial and final positions of the slide mass. The second approach employs a dynamic source, which specifies both surface displacement and depth averaged horizontal velocity fields. This source is computed from the slide geometry using the model NHWAVE (Non-Hydrostatic Wave), Version 1.0, which is described in Reference 2.4.6-231. The computation of the initial source requires a value for slide velocity. This is computed using a methodology described by Enet and Grilli (Reference 2.4.6-229). For each of the landslides, a slide geometry equivalent to that in Reference 2.4.6-229 is employed with an adjustment to slide aspect ratio (width/length) to best fit the model slide to the measured shape of the excavated source region for the measured slide. This choice allows the use of the same geometric relationships between slide volume, area, thickness, length, and width as utilized in Reference 2.4.6-229. The geometry of the slides is described below.

Parameters defining tsunami sources were obtained from ten Brink et al (Reference 2.4.6-238); a listing of parameters is provided in Table 2.4.6-206 and Table 2.4.6-207. The simulated tsunamis that are generated using source parameters for the different scenarios described herein are severe enough to be considered equivalent to the PMT for the LNP site.

The flood level near the LNP site due to PMT is estimated using the numerical wave model FUNWAVE-TVD (Fully Nonlinear Wave – Total Variation Diminishing Scheme), Version 1.0, described in References 2.4.6-230, 2.4.6-234, and 2.4.6-235. Benchmark testing of FUNWAVE-TVD is described in Reference 2.4.6-237. Inputs to FUNWAVE-TVD include depth grids, whose development is described herein, and information about the configuration of a tsunami source, used as the initial condition for the model run. Program documentation and users' manual for FUNWAVE-TVD is available in Reference 2.4.6-235.

Outputs generated by FUNWAVE-TVD include gridded surface displacement and horizontal velocities as a function of time during the model simulation. The model results are presented as snapshots of evolving surface displacement during each simulated case. The model accumulates information on the maximum runup water level occurring at each grid location during the simulation, and provides estimates of maximum inundation and runup values near the LNP site based on these accumulated values. The results are used to estimate the maximum water level due to the PMT near the LNP site, and to determine if LNP Units 1 and 2 will be affected by the PMT maximum water level.

The verification and validation of the FUNWAVE-TVD and NHWAVE computer programs are described in the Verification and Validation document (Appendix 1). Computation grids are generated using Fortran programs and MATLAB scripts, for which detailed operation procedures and verification are also described in Appendix 1.

2.4.6.7.1 FUNWAVE-TVD Model Description

The propagation, shoreline runup and inundation caused by tsunamis are calculated using the Boussinesq wave model FUNWAVE-TVD. In the present application, FUNWAVE-TVD solves the spherical-polar form of the weakly-nonlinear, weakly-dispersive Boussinesq equations described by Kirby et al (Reference 2.4.6-230). Shi et al (Reference 2.4.6-235) describes the operation of both Cartesian and spherical-polar versions of the code. The model incorporates bottom friction and turbulent mixing effects. The model is available to the public as open source software.

The Cartesian coordinate version of FUNWAVE-TVD, described in Shi et al (Reference 2.4.6-234 and Reference 2.4.6-235), has been benchmarked for tsunami application using the PMEL-135 benchmarks provided in Synolakis et al (Reference 2.4.6-236), which are the presently accepted benchmarking standards adopted by the National Tsunami Hazard Mitigation Program (NTHMP) for judging model acceptance for use in development of coastal inundation maps and evacuation plans. Benchmark tests for the Cartesian FUNWAVE-TVD are described in Tehranirad et al (Reference 2.4.6-237). The spherical-polar version of the code used here is subjected to several of these benchmarks in order to document consistency and accuracy of the model.

The equations solved by FUNWAVE-TVD consist of a depth-integrated volume conservation equation together with depth-integrated horizontal momentum equations. The equations retain effects to second order in the ratio of water depth to wavelength, accounting for deviations from depth-uniform horizontal flow up to quadratic terms in the vertical coordinate. The resulting volume conservation equation is given by (Reference 2.4.6-230):

$$\frac{1}{\delta}H_t + \frac{1}{\cos\theta}\left\{\left(H\bar{u}\right)_{\phi*} + \left(H\bar{v}\cos\theta\right)_{\theta*}\right\} = 0 \qquad \text{Equation 2.4.6-2}$$

where *H* is the local total water depth, *u* is Easterly, depth-averaged horizontal velocity and *v* is depth-averaged Northerly horizontal velocity, θ is latitude in radians, δ is a dimensionless ratio of surface displacement scale to representative water depth, and subscripts *t,* ϕ^* and θ^* represent a time derivative and spatial derivatives with respect to scaled latitude and longitude (see Reference 2.4.6-230 for further details of the derivation).

The corresponding horizontal momentum equations for the Easterly (longitudinal) direction are given by (Reference 2.4.6-230):

$$\begin{aligned}&\bar{u}_t - \mu^2 f \bar{\upsilon} + \delta\left\{\frac{\bar{u}}{\cos\theta}\bar{u}_{\phi*} + \bar{\upsilon}\bar{u}_{\theta*}\right\} + \frac{1}{\cos\theta}\eta_{\phi*} \\ &+ \frac{\mu^2}{\cos^2\theta}\left\{\frac{h^2}{6}\left[\bar{u}_{\phi*\phi*t} + \left(\bar{\upsilon}\cos\theta\right)_{\phi*\theta*t}\right] - \frac{h}{2}\left[\left(h\bar{u}_t\right)_{\phi*\phi*} + \left(h\cos\theta\bar{\upsilon}_t\right)_{\phi*\theta*}\right]\right\} \\ &+ \frac{\mu^2}{\cos\theta}\left(BFT\right)_{\phi*} - \tau_b^x = O\left(\delta^2, \delta\mu^2, \mu^4\right)\end{aligned}$$

Equation 2.4.6-3

The corresponding horizontal momentum equations for the Northerly (latitudinal) direction are given by (Reference 2.4.6-230):

$$\begin{aligned}&\bar{\upsilon}_t + \mu^2 f \bar{u} + \delta\left\{\frac{\bar{u}}{\cos\theta}\bar{\upsilon}_{\phi*} + \bar{\upsilon}\bar{\upsilon}_{\theta*}\right\} + \eta_{\theta*} \\ &+ \mu^2\left\{\frac{h^2}{6}\left[\frac{1}{\cos\theta}\left\{\bar{u}_{\phi*t} + \left(\bar{\upsilon}\cos\theta\right)_{\theta*t}\right\}\right]_{\theta*} - \frac{h}{2}\left[\frac{1}{\cos\theta}\left\{\left(h\bar{u}_t\right)_{\phi*} + \left(h\cos\theta\bar{\upsilon}_t\right)_{\theta*}\right\}\right]_{\theta*}\right\} \\ &+ \mu^2\left(BFT\right)_{\theta*} - \tau_b^y = O\left(\delta^2, \delta\mu^2, \mu^4\right)\end{aligned}$$

Equation 2.4.6-4

Here *f* represents the Coriolis parameter, η represents the water surface displacement from its initial position, *h* represents local still water depth, and μ is a dimensionless parameter characterizing the ratio of characteristic water depth to characteristic surface wave length. The term *BFT* contains the effect of continuous bottom motion in time, and is not utilized in the present study since bottom displacements are described either as static initial conditions, or are tied to initial surface displacement and velocity field in a separate model computation based on NHWAVE, described below. The term τ_b represents the effect of bottom friction and is given by (Reference 2.4.6-230):

$$\tau_b^{x,y} = \frac{1}{H}C_D\left|\mathbf{u}\right|\left(u,v\right)$$

Equation 2.4.6-5

where C_D is the drag coefficient and **u** is the horizontal velocity vector. The value of the drag coefficient can range from 0.0008 to 0.0031 on continental shelf and slope environments (Reference 2.4.6-241) and can be as high as 0.005 during breaking and runup (Reference 2.4.6-240). Increased roughness of subaerial vegetation and other surface features can lead to higher apparent values of the drag coefficient. For this study, a lower end value of the drag coefficient of C_D = 0.001 is used and therefore will result in conservative estimates of runup water levels.

For tsunami applications here, FUNWAVE-TVD is run with closed boundaries and an initial hot start condition consisting of either a surface displacement alone (in the case of static initial conditions) or a surface displacement and initial velocity field (in the case of a dynamic initial condition based on NHWAVE calculations). The choice for each source will be described separately. The model is run from the initial start until past the time when significant wave activity has decayed at the target site.

2.4.6.7.2 NHWAVE Model Description

For several of the computations described below, the model NHWAVE is used to describe the early stages of surface displacement and velocity field development associated with an underwater landslide. NHWAVE solves fully non-hydrostatic Navier-Stokes equations in a surface and terrain following (sigma) coordinate system. The model is described in Ma et al (Reference 2.4.6-231). The model assumes a single valued water surface and represents turbulent stresses in terms of an eddy viscosity closure. Turbulent stresses are not modeled in the present study, and thus the model is basically solving Euler equations for incompressible flow with a moving surface and bottom.

The governing equations for NHWAVE in Cartesian tensor form are given by (Reference 2.4.6-231):

$$\frac{\partial u_i}{\partial x_i^*} = 0 \qquad \text{Equation 2.4.6-6}$$

and

$$\frac{\partial u_i}{\partial t^*} + u_j \frac{\partial u_i}{\partial x_j^*} = -\frac{1}{\rho}\frac{\partial p}{\partial x_i^*} + g_i + \frac{\partial \tau_{ij}}{\partial x_j^*} \qquad \text{Equation 2.4.6-7}$$

Here, g_i is the gravitational vector, p is fluid pressure, ρ is the fluid density, u_i is the velocity vector, and τ_{ij} is a tensor representing viscous and turbulent stresses which is not used in the current analysis. The σ coordinate version of the model is described in Ma et al (Reference 2.4.6-231). Surface boundary conditions for the model consist of a kinematic constraint on vertical velocity ω, and constraints on tangential and normal stresses (Reference 2.4.6-231).

$$\omega|_{z=\eta} = \frac{\partial \eta}{\partial t} + u\frac{\partial \eta}{\partial x} + \upsilon\frac{\partial \eta}{\partial y}$$ Equation 2.4.6-8

In the absence of turbulent and viscous effects and with no atmospheric forcing, these reduce to the following for normal stress (Reference 2.4.6-231):

$$p|_{z=\eta} = 0$$ Equation 2.4.6-9

For tangential stress, these reduce to (Reference 2.4.6-231):

$$\frac{\partial u}{\partial \sigma}|_{z=\eta} = \frac{\partial \upsilon}{\partial \sigma}|_{z=\eta} = 0$$ Equation 2.4.6-10

where vertical stretched coordinate σ = (η-*z)/H* and H is total depth defined after Equation 2.4.6-2.

At the bottom, zero tangential stress again gives (Reference 2.4.6-231):

$$\frac{\partial u}{\partial \sigma}|_{z=-h} = \frac{\partial \upsilon}{\partial \sigma}|_{z=-h} = 0$$ Equation 2.4.6-11

The kinematic constraint on vertical velocity at the bottom is given by (Reference 2.4.6-231):

$$\omega|_{z=-h} = -\frac{\partial h}{\partial t} - u\frac{\partial h}{\partial x} - \upsilon\frac{\partial h}{\partial y}$$ Equation 2.4.6-12

The condition on normal stress on the moving bottom is derived from the vertical momentum equation and is given by (Reference 2.4.6-231):

$$\frac{\partial p}{\partial \sigma}|_{z=-h} = -\rho D\frac{d\omega}{dt}|_{z=-h}$$ Equation 2.4.6-13

where *D* represents total water depth. In the current analysis, linearized forms of Equation 2.4.6-12 and Equation 2.4.6-13 are combined to obtain:

$$\frac{\partial p}{\partial \sigma}|_{z=-h} = \rho D\frac{\partial^2 h}{\partial t^2}$$ Equation 2.4.6-14

This linearized boundary condition is the same as employed in the basic testing of the model against the laboratory data of Enet and Grilli (Reference 2.4.6-229) and described in Ma et al (Reference 2.4.6-231).

For the present cases, the modeled domain is set up so that the landslide event is centrally located and the generated motion does not reach lateral boundaries during the simulated time. Results from the model at the end of the model run are saved and used in FUNWAVE-TVD as initial conditions.

2.4.6.7.3 Model Grid Development and Description

Topographic data were obtained from high resolution LIDAR surveys around the LNP site converted into an ASCII file for use in the computer model. Bathymetric data for the model domains were obtained from the National Geophysical Data Center (NGDC) ETOPO 1 (Reference 2.4.6-228) and NGDC Coastal Relief Model (CRM) (Reference 2.4.6-232). The computational scheme is comprised of a nesting of three model grids which move the computation for a lower resolution large scale Grid *A* (A_l for landslide cases), through an intermediate resolution Grid *B*, to a high resolution Grid *C* encompassing the study site. All the grids are based on global (latitude-longitude) coordinates.

Grid *A* for the Venezuela seismic tsunami case was generated based on the ETOPO1 data set (Reference 2.4.6-228). The data were extracted using GEODAS, a standard tool described in Reference 2.4.6-228. ETOPO1 uses Mean Sea Level (MSL) as a vertical datum origin. The grid resolution for Grid *A* is 2 arc-minutes.

Grid A_l for the landslide cases was generated based on ETOPO1 in the same way as the Grid *A* was generated for the Venezuela seismic case. The grid resolution for Grid A_l is 1 arc-minute.

Grid *B*, which is nested within both Grid *A* and A_l, was generated based on Volume 3 of the CRM data set (Reference 2.4.6-232). GEODAS is used for data extraction. The CRM also uses MSL as the vertical datum origin. The grid resolution for Grid *B* is 15 arc-seconds.

Grid *C*, which is nested in Grid *B*, was developed from CRM data and the local LIDAR data at the study site. The LIDAR data was first converted into the global horizontal coordinates and the MSL vertical datum using standard tools. The data was merged into the computational grid. The grid resolution for Grid *C* is 3 arc-seconds, or about 90 m (295 ft.).

Grid *A* and A_l and nested Grids *B* and *C* are presented in Figures 2.4.6-228, 2.4.6-229, 2.4.6-230, and 2.4.6-231. One-way nesting between Grid *A* (for the Venezuela seismic case) or Grid A_l (for Gulf of Mexico landslide cases), Grid *B* and Grid *C* is performed through the one-way data transfer at nesting boundaries. The grid nesting scheme transfers surface elevation and velocity components calculated from a large domain to a nested small domain through host cells at nesting boundaries. A linear interpolation is performed between a large domain and small domain at nesting boundaries.

2.4.6.7.4 Vertical Datum and Initial Water Level for Model Runs

The vertical datum used to report maximum water level results near the LNP site is based on North American Vertical Datum 1988 (NAVD88). The vertical datum conversion between MSL and NAVD88 was obtained from National Oceanic and Atmospheric Administration (NOAA) data at Cedar Key, FL (Reference 2.4.6-242).

The initial water level used in all simulations is determined by adding the long-term sea level rise to the 10 percent exceedance high tide for this region.

The 10 percent exceedance predicted high tide is determined following the Regulatory Guide 1.59 (RG 1.59) (Reference 2.4.6-233). The 10 percent exceedance high tide value of 1.31 m (4.3 ft.) MLW for Crystal River is obtained from Table C1 of RG 1.59. This value from RG 1.59 is converted from MLW datum to NAVD88 datum using the datum conversion chart from Reference 2.4.6-242. The 10 percent exceedance high tide value is converted to 0.82 m (2.68 ft.) NAVD88. This 10 percent exceedance predicted high tide value of 0.82 m (2.68 ft.) NAVD88 is combined with initial rise of 0.18 m (0.6 ft.) from Table C1 of RG 1.59, to obtain the 10 percent exceedance high tide level of 1.00 m (3.28 ft.) NAVD88.

NOAA has evaluated sea level rise trends for each tide station. Reference 2.4.6-243 provides the data for the mean sea level trend at the Cedar Key tide gauge, station 8727520. From this reference the mean sea level rise of 0.2 m (0.59 ft.) in 100 years is obtained.

By adding the 10 percent exceedance high tide of 1.00 m (3.28 ft.) NAVD88 to the long-term sea level rise of 0.2 m (0.59 ft.), an initial water level of 1.18 m (3.87 ft.) NAVD88 was obtained.

2.4.6.7.5 Seismic Source: Venezuela

Source parameters for Venezuela seismic tsunami are based on ten Brink et al (Reference 2.4.6-238) and are presented in Table 2.4.6-206. There are two fault segments associated with the potential event. The first is the W. Southern Caribbean fault in the north Venezuela subduction zone. This fault is 550 km (342 mi.) long and 50 km (31 mi.) wide. The strike angle is N53E, the dip angle is 17S, and the rake angle is 90 degrees. The second fault is the E. Southern Caribbean in the north Venezuela subduction zone. It is 200 km (124 mi.) long and 50 km (31 mi.) wide. The strike, dip and rake angles are N95E, 17S and 90 degrees, respectively. The composite source was used with a slip of 23 m (75.5 ft.) for both of the segments, equivalent to magnitude $M_w = 9.0$. Figure 2.4.6-232 shows the initial source deformation calculated from the Okada formula. Results of the tsunami simulation for the Venezuela Seismic source are described in FSAR Subsection 2.4.6.7.9.

2.4.6.7.6 Landslide Source: Initial Conditions and Determination of Slide Velocities

Two initial conditions are considered for each landslide source in this analysis. The first is a static source configuration, which ten Brink et al (Reference 2.4.6-238) suggest is a conservative approach to conduct a simulated landslide event. For the static source configuration, the initial condition for the numerical simulation is a static surface displacement with a depression over the initial slide location and having the size and shape of the slide volume, and an elevation over the final slide position having the same size and shape. The second condition employs NHWAVE to generate a dynamic source, which specifies both surface displacement and depth averaged horizontal velocity fields. For the dynamic source configuration where initial conditions for landslide sources are determined from NHWAVE, it is necessary to determine a reasonable velocity for the sliding mass, as discussed below.

Slide velocities are estimated following the methods presented by Enet and Grilli (Reference 2.4.6-229). The slide shapes given in Reference 2.4.6-229 are utilized, with the thickness of the slide relative to a local origin (x,y) = (0,0) given by:

$$\zeta = \frac{T}{1-\varepsilon}\left[\sec h\left(k_b x\right)\sec h\left(k_w y\right)-\varepsilon\right] \qquad \text{Equation 2.4.6-15}$$

where ζ is the horizontal distribution of slide thickness, T is the maximum slide thickness at the center, $k_b = 2C/b$, $k_w = 2C/w$, b is landslide length in the forward (sliding) direction, w is landslide width in the transverse direction, $C = acosh(1/\varepsilon)$, and truncation parameter $\varepsilon = 0.7$ as in Reference 2.4.6-229*.* This geometry gives a total slide volume V_b given by (Reference 2.4.6-229):

$$V_b = bwT\left(\frac{f^2-\varepsilon}{1-\varepsilon}\right) \quad \text{with} \quad f = \frac{2}{C}a\tan\sqrt{\frac{1-\varepsilon}{1+\varepsilon}} \qquad \text{Equation 2.4.6-16}$$

The motion of the landslide is estimated from a balance of inertia, gravity force, buoyancy, Coulomb friction and drag force, leading to the balance equation (Reference 2.4.6-229):

$$\left(M_b+\Delta M_b\right)\frac{d^2 s}{dt^2} = \left(M_b-\rho_w V_b\right)\left(\sin\theta - C_n\cos\theta\right)g - \frac{1}{2}\rho_w\left(C_F A_w + C_D A_b\right)\left(\frac{ds}{dt}\right)^2$$

Equation 2.4.6-17

where g is gravitational acceleration, M_b is slide mass, s is downslope slide displacement, ρ_w is water density, θ is bed slope angle, ΔM_b, A_w and A_b are slide added mass, wetted surface area and main cross section perpendicular to the direction of motion, respectively. The remaining parameters are C_F, the skin friction coefficient, C_D, the form drag coefficient, and C_n, the basal Coulomb friction coefficient. Based on this model, a terminal velocity of the slide is calculated using the following equation with the assumption that the slide moves forward a distance equal to its initial width b in the slide direction (Reference 2.4.6-229):

$$u_t = \sqrt{gb\sin\theta\left(1-\frac{\tan\phi}{\tan\theta}\right)\left(\frac{\gamma-1}{C'_d}\right)\frac{2\left(f^2-\varepsilon\right)}{f-\varepsilon}} \qquad \text{Equation 2.4.6-18}$$

The parameter $\gamma = \rho_s / \rho_w$ is the specific gravity of the sliding material, which is taken to be 2.65. Following Reference 2.4.6-229, the global drag coefficient C'_d = 1.0.

2.4.6.7.7 Landslide Source: Mississippi Canyon

The Mississippi Canyon landslide event was chosen as the largest credible event occurring in the Gulf of Mexico basin. The description of the Mississippi Canyon landslide source is based on the discussion in ten Brink et al (Reference 2.4.6-238). The bathymetric evidence for the event is illustrated in Figure 2.4.6-233, where the source region is outlined. Ten Brink et al (Reference 2.4.6-238) estimates that the initial slide has an area A = 3687 km^2 (1424 mi^2) and volume V_b = 425.5 km^3 (102.1 mi^3). From the outlined region in Figure 2.4.6-233, the slide source's length b is estimated to be 150 km (93.2 mi.). Using the geometric description in the previous section, a slide width w is estimated to be 31.3 km (19.5 mi.), and a maximum slide thickness T is estimated to be 0.306 km (0.19 mi.), which corresponds well with the reported excavation depth of approximately 300 m (984 ft.) given in ten Brink et al (Reference 2.4.6-238). The source parameters for the Mississippi Canyon landslide event are presented in Table 2.4.6-207.

Two initial conditions are considered for the Mississippi Canyon landslide simulation. The first is a static source configuration, for which it is assumed that the initial slide volume described above translates downslope along its major axis a distance equal to its initial length, 150 km (93.2 mi.). The initial and final positions of the slide are displayed in Figure 2.4.6-233. The water depths at the slides initial and final centroids are 1000 m (3281 ft.) and 2450 m (8038 ft.), respectively, giving an effective local slope (delta) h/b = 0.01. The runout distance of 150 km (93.2 mi.) is less than the estimates based on measured bathymetry given in ten Brink et al (Reference 2.4.6-238), but the spreading and flattening of the sliding mass during the slide process is neglected in the present simulations. This gives a higher and narrower initial elevation hump at the final slide location than would occur if the slide were allowed to deform, but the initial displaced water volume is equivalent in either case. The resulting initial surface displacement for input into FUNWAVE-TVD is shown in Figure 2.4.6-234.

The second initial condition employs the model NHWAVE to generate an initial surface displacement and horizontal velocity field based on direct modeling of the sliding mass described above. Input to NHWAVE includes bathymetric grid, initial slide position and orientation, total sliding distance, and down-slope translation velocity of the slide. Equation 2.4.6-18 is used to estimate a terminal velocity u_t = 151.4 m/s (496.7 ft/s). In the simulation, the slide is translated at this velocity for the entire slide event. The duration of the event is then (delta) $t = b/u_t = 990.7$ seconds. The model is run for this duration, and the final surface displacement field and horizontal velocity fields are saved for input into FUNWAVE-TVD. The resulting surface displacement at the end of the NHWAVE run is shown in Figure 2.4.6-235.

Using either the static or dynamic source as initial condition, FUNWAVE-TVD is run using one-way nesting using Grids A_l, B and C. Results of the tsunami simulation for the Mississippi Canyon source are described in FSAR Subsection 2.4.6.7.10.

2.4.6.7.8 Landslide Source: Florida Escarpment

The Florida Escarpment landslide event is the closest large event to LNP site, chosen since landslide tsunami impact is often most severe on adjacent shorelines. The Florida Escarpment landslide event is based on a collapse of the shelf break slope of the carbonate platform which forms the continental shelf on the western side of the Florida peninsula. The description of the Florida Escarpment landslide source is based on the discussion in ten Brink et al (Reference 2.4.6-238). The bathymetric evidence for the event is illustrated in Figure 2.4.6-236, where the source region is outlined, following on information presented in Figure 3-6 in ten Brink et al (Reference 2.4.6-238). Ten Brink et al (Reference 2.4.6-238) estimate that the initial slide has an area A = 647.47 km^2 (250 mi^2), width w = 42.94 km (26.68 mi.) and volume V_b = 16.2 km^3 (6.26 mi^3). From the outlined region in Figure 2.4.6-236 the slide source's length b is estimated to be 19.2 km (11.9 mi.), and a maximum slide thickness T is estimated to be 58 m (190 ft.), which differs from the estimate of 150 m (492 ft.) given in ten Brink et al (Reference 2.4.6-238), but results from the very uneven excavation pattern in the actual slide. The source parameters for the Florida Escarpment landslide event are presented in Table 2.4.6-207.

As in the Mississippi Canyon landslide event, two initial conditions are considered for the Florida Escarpment landslide simulation. For both, it is assumed that the initial source translated a downslope distance equal to its initial downslope length. Ten Brink et al (Reference 2.4.6-238) discuss this translation and point out that it is presently impossible to determine the configuration of the slide deposit from existing sonar data as the volume is buried under later deposits. The initial and final positions of the slide in the simulations are displayed in Figure 2.4.6-236. The water depths at the initial and final centroids of the slides are 1548 m (5079 ft.) and 3305 m (10,843 ft.), respectively, giving an effective local slope (delta) h/b = 0.09. For the dynamic source determination, Equation 2.4.6-18 is used to estimate a terminal velocity u_t = 175.3 m/s (575.2 ft/s). In the simulation, the slide is translated at this velocity for the entire slide event. The duration of the event is then (delta) $t = b/u_t = 109.5$ seconds. The model is run for this duration, and the final surface displacement field and horizontal velocity fields are saved for input into FUNWAVE-TVD. Initial surface displacements for FUNWAVE-TVD for the static and dynamic sources are shown in Figure 2.4.6-237 and Figure 2.4.6-238, respectively.

Using either the static or dynamic sources as initial conditions, FUNWAVE-TVD is run using one-way nesting using Grids A_l, B and C. Results of the tsunami simulation for the Florida Escarpment source are described in FSAR Subsection 2.4.6.7.11.

2.4.6.7.9 Model Results: Venezuela Seismic Event Simulation

Figure 2.4.6-239 shows snapshots for the propagation of the tsunami in the Venezuela Grid *A* at 40 minutes and 80 minutes after the start of the model. Figure 2.4.6-240 shows a plot of the maximum water level occurring in Grid *A*, and shows that tsunami runup height on the west Florida shelf is on the order of 20 to 30 cm (7.9 to 11.8 in). Figure 2.4.6-241 shows snapshots, at 440 minutes and 520 minutes after the start of the model, of the tsunami wave crossing the shelf region (Grid *B*) and making landfall on the west coast of Florida. Maximum tsunami amplitudes are greatly reduced by frictional effects over the shelf, and are limited to values on the order of 15 cm (5.9 in) along the coast covered by Grid *B* (Figure 2.4.6-242). Figure 2.4.6-243 and Figure 2.4.6-244 show propagation and maximum water level in Grid *C*, respectively, and show that tsunami runup in the vicinity of the LNP site reaches levels on the order of 30 cm (11.8 in). Surface displacements in Figures 2.4.6-239, 2.4.6-240, 2.4.6-241, 2.4.6-242, 2.4.6-243, and 2.4.6-244 are with respect to model initial water level, which includes MSL plus 10 percent exceedence high tide plus the long-term sea level rise.

Figure 2.4.6-245 shows a plot of the initial MSL shoreline and maximum extent of tsunami inundation superimposed on a topographic contour map of the Grid C region. The extent of tsunami runup in the inundated region is illustrated in Figure 2.4.6-246, which shows a vertical section through Grid C at latitude 29.075N, representing an east-west line through the location of the LNP site, indicated on the figure. Results, including maximum water levels and inland distance of inundation, are summarized in Table 2.4.6-208. It should be noted that the elevations of the Levy site shown on Figures 2.4.6-245 and 2.4.6-246 and other topographical maps contained in following descriptions of model results represent the existing grade of approximately 12.5 to 14.9 m (41 to 49 ft.) NAVD88, as opposed to the plant grade elevation of 15.24 m (50 ft.) NAVD88.

2.4.6.7.10 Model Results: Mississippi Canyon Landslide Event Simulation

The Mississippi Canyon landslide source was run using two different source configurations: a static source following the procedure in Reference 2.4.6-238, and a dynamic source using the NHWAVE model and the determination of slide. Results of the tsunami simulation based on the static source are displayed in Figures 2.4.6-247, 2.4.6-248, 2.4.6-249, 2.4.6-250, 2.4.6-251, 2.4.6-252, 2.4.6-253, and 2.4.6-254, and results based on the dynamic source are displayed in Figures 2.4.6-255, 2.4.6-256, 2.4.6-257, 2.4.6-258, 2.4.6-259, 2.4.6-260, 2.4.6-261, and 2.4.6-262.

Although the methodology described in Reference 2.4.6-238 is aimed at giving the most conservative estimate of tsunami wave amplitude for a given source configuration, results of these simulations indicate that the geometry for the source also plays a role in determining the susceptibility of any segment of coastline to tsunami attack. For example, a comparison of Figure 2.4.6-247 (for the static source) and Figure 2.4.6-255 (for the dynamic source) show a tendency for wave energy to be directed in a more southerly direction for the dynamic source. The resulting patterns of maximum wave amplitude for Grid A_I for the two sources, shown in Figure 2.4.6-248 and Figure 2.4.6-256, show that the tsunami wave height arriving at the west Florida shelf break is significantly larger for the dynamic source than for the static source.

The evolution of the landward propagating wave for either source is controlled by strong wave breaking dissipation over the shallow and broad extent of the shelf. A comparison of the maximum wave heights for the two sources over Grid *B*, presented in Figure 2.4.6-250 and Figure 2.4.6-258, show that the dynamic source produces higher waves over the inner shelf region, but that wave height is significantly reduced by the time waves reach the inner Grid *C* region. Results for the two sources over Grid *C* are very similar, as revealed by plot plans in Figure 2.4.6-253 and Figure 2.4.6-261 and transect plots in Figure 2.4.6-254 and Figure 2.4.6-262. Inundation depth offshore reaches about 4.6 m (15 ft.) NAVD88 in both cases, and there is significant inland flooding which reaches the base of the steeper terrain surrounding the LNP site. Details of the spatial distribution of maximum runup elevation differ for the two cases, although neither is likely to be strictly accurate due to the relatively inaccurate nature of the CRM bathymetry for the subaerial regions fronting the LNP site.

The maximum water level near the LNP site due to a tsunami caused by the simulated Mississippi Canyon landslide event is 3.94 m (12.94 ft.) NAVD88, considering both the static and dynamic source configurations. The inundation due to this tsunami reaches a location approximately 15.6 km (9.7 mi.) inland and approximately 5.95 km (3.7 mi.) from the LNP site. Tsunami simulation results are summarized in Table 2.4.6-208.

2.4.6.7.11 Model Results: Florida Escarpment Landslide Event Simulation

The tsunami simulation results for both static and dynamic source configurations for the Florida Escarpment landslide event are similar. Therefore, the time-wise simulation results for the static case only are presented in Figures 2.4.6-263, 2.4.6-264, 2.4.6-265, 2.4.6-266, 2.4.6-267, 2.4.6-268, 2.4.6-269, and 2.4.6-270. For comparison with the static source case, only the inland extent of inundation is provided for the dynamic case as presented in Figure 2.4.6-271.

The maximum water level near the LNP site due to a tsunami caused by the simulated Florida Escarpment landslide event is 1.32 m (4.33 ft.) NAVD88, considering both the static and dynamic source configurations. The inundation due to this tsunami reaches a location approximately 6.8 km (4.2 mi.) inland and approximately 14.8 km (9.2 mi.) from the LNP site. Results for all of the tsunami simulations are summarized in Table 2.4.6-208.

2.4.6.7.12 PMT Maximum Water Level near LNP 1 and 2 Site

The results of maximum water level and inundation distance for each of the simulated tsunami scenarios and source configurations are presented in Table 2.4.6-208. Based on the discussion of tsunami simulation results, the most severe PMT event is associated with the large scale Mississippi Canyon landslide event in the Gulf of Mexico. This event produced the highest runup and farthest extent of onshore inundation in the vicinity of the LNP site.

The maximum PMT runup elevation due to the Mississippi Canyon landslide was computed to be 3.94 m (12.94 ft.) NAVD88, which is the most conservative of all the simulation scenarios. The PMT runup includes the initial water level due to the 10 percent exceedance high tide and the long-term sea level rise. The maximum PMT runup elevation is below the plant grade elevation of 15.24 m (50 ft.) NAVD88. The extent of inundation due to the PMT closest to the LNP site is 5.95 km (3.7 mi.) west of the LNP Units 1 and 2. Therefore, the safety-related systems and components of LNP Units 1 and 2 will not be affected by the PMT in the Gulf of Mexico.

2.4.6.8 Hydrography and Harbor or Breakwater Influences on Tsunami

Routing of the controlling tsunami, which includes breaking wave formation, bore formation, and resonance effects, is expected to be minor and limited to shorelines. As the LNP site is approximately 12.8 km (7.9 mi.) from the Gulf of Mexico, hydrography and harbor or breakwater influences are not expected to be severe enough under any circumstances to jeopardize the operation of the safety-related structures.

2.4.6.9 Effects on Safety-Related Facilities

As concluded in FSAR Subsection 2.4.6.7, the LNP site is not expected to be impacted by PMT. Thus, effects of the controlling tsunami are not expected to jeopardize the operation of the safety-related structures. Therefore, measures to protect the LNP site against the effects of a tsunami are not included in the design criteria.

2.4.7 ICE EFFECTS

LNP COL 2.4-2

A review of historical temperature records from the NWS Cooperative Observer Station No. 086414 in Ocala, Florida, for the period 1971 to 2000 indicates monthly average minimum temperatures for the months of December, January, and February as being 8.5 degrees Celsius (°C), 7.6°C, and 8.3°C (47.3 degrees Fahrenheit [°F], 45.7°F, and 47.0°F), respectively (Reference 2.4.2-211). The monthly mean temperatures for the same months are 15.3°C, 14.5°C, and 15.5°C (59.5°F, 58.1°F, and 59.9°F), respectively. Ice formation in this locality on large bodies of water is considered unlikely. It is not expected to be severe enough under any circumstances to jeopardize the operation of the safety-related structures.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

Safety systems for the AP1000 are designed to function without safety-related support systems such as component cooling water and service water. None of the safety-related equipment requires cooling water to affect a safe shutdown or mitigate the effects of design basis events. Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell supplied by a passive containment cooling water tank. Therefore, the AP1000 design does not rely on service water and component cooling water systems to provide safety-related safe shutdown. No design bases for the capacity or operating plan for a cooling water canal or reservoir are needed.

DCD Subsection 6.2.2.2 describes the system design and operation of the passive containment cooling system and components. Passive containment cooling water storage tank filling operations and normal makeup needs are discussed in DCD Subsection 9.2.4.

2.4.9 CHANNEL DIVERSIONS

The CFBC is a man-made drainage structure that is directly connected to the Gulf of Mexico and is not susceptible to migration or cutoff. Lake Rousseau and the Gulf of Mexico are the major sources of water to the CFBC. Gauge height data at USGS station 02313230, located upstream of the Inglis Dam, and USGS station 02313250, located upstream of the Inglis Bypass Channel Spillway, indicate that no channel diversions significant enough to affect surface water elevations in Lake Rousseau have occurred over the approximately 35 years of record (References 2.4.1-209 and 2.4.1-211). Due to the size of the Gulf of Mexico, complete diversions are considered unlikely.

Topographic characteristics, geological features, and the low seismic activity of the drainage basin indicate there is no possibility for the occurrence of a landslide blocking or limiting flow into the CFBC from Lake Rousseau or the Gulf of Mexico.

Because ice effects are considered unlikely, they are not expected to create flow diversion during winter months.

Although the potential for anthropogenic diversions of the CFBC exists, it is located in a relatively unpopulated area, thereby minimizing this potential. In addition, the AP1000 design does not have a safety-related cooling water system and, therefore, does not rely on the service water and component cooling water systems to provide safety-related safe shutdown. Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell.

2.4.10 FLOODING PROTECTION REQUIREMENTS

LNP COL 2.4-6 The design bases for flood protection, including wave runup, are provided in FSAR Subsections 2.4.2, 2.4.3, 2.4.4, 2.4.5, and 2.4.6. The flooding effects of a PMF on Lake Rousseau, a local PMP at the LNP site, and a PMT or PMH in the Gulf of Mexico provide the design bases for flood protection. The effects of the flood and coincident wind-wave activity are discussed in FSAR Subsections 2.4.5.2.5 and 2.4.6.

The design and flood requirements for safety-related facilities to ensure they will be capable of surviving all design bases flood conditions is provided in DCD Section 3.4. DCD Section 3.8 discusses the design of seismic Category I structures, including design loads and load combinations due to external flooding.

DCD Table 2-1 specifies the key site parameters for the design of safety-related structures, systems, and components for the AP1000. The AP1000 site parameters for flooding bound the LNP site flood levels.

2.4.11 LOW WATER CONSIDERATIONS

LNP SUP 2.4-3 Conditions such as limited flow rates and low cooling water elevations resulting from severe droughts, such as a 100-year drought, or low water levels resulting from anthropogenic water use, will not affect the ability of the safety-related facilities associated with the AP1000 design, particularly the ultimate heat sink, to perform adequately. LNP 1 and LNP 2 will use a passive core cooling system designed to provide emergency core cooling without the use of active equipment, such as pumps and ac power sources. The passive core cooling system depends on reliable passive components and processes, such as gravity injection and expansion of compressed gases. The passive safety-related systems are designed to cool the reactor coolant system from normal operating temperatures to safe shutdown conditions.

Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell. Therefore, the AP1000 design does not rely on the service water and component cooling water systems to provide safety-related safe shutdown. The CFBC is only used to provide LNP 1 and LNP 2 with cooling water for normal operations to produce electricity.

To demonstrate that LNP 1 and LNP 2 can continue to operate during low flow conditions, a review of the historical data available from the Lower Withlacoochee River near the LNP site was conducted. Historical low flow stages at following USGS stations were reviewed:

- **Withlacoochee River at Dunnellon, Florida** (USGS ID: 02313200, #20 on Figure 2.4.1-211)**:** This station is located on the Withlacoochee River, 1.3 km (0.8 mi.) upstream of Lake Rousseau. Daily water stage has been recorded at this station for a period of 44 years (1963 – 2007). (Reference 2.4.1-208) Minimum stage observed at this station during that period is 7.04 m (23.10 ft.) NGVD29 (October 11, 1972). (Reference 2.4.2-201) Add the conversion factor of -0.267 m (-0.876 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River at Inglis Dam near Dunnellon, Florida** (USGS ID: 02313230, #21 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the upstream side of the Inglis Dam (Reference 2.4.1-209). Daily water stage has been recorded at this station for a period of 22 years (1985 – 2007). Minimum stage observed at this station during that period is 7.36 m (24.14 ft.) NGVD29 (January 13, 1990). (Reference 2.4.2-203) Add the conversion factor of -0.309 m (-1.01 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River below Inglis Dam near Dunnellon, Florida** (USGS ID: 02313231, #22 on Figure 2.4.1-211): This station is located on the Withlacoochee River on the downstream side of the Inglis Dam. Daily water stage has been recorded at this station for a period of 38 years (1969 – 2007) (Reference 2.4.1-210). Minimum stage observed at this station during this period is -0.56 m (-1.85 ft.) NGVD29 (January 16, 1972) (Reference 2.4.2-204). Add the conversion factor of -0.309 m (-1.01 ft.) to elevations with a NGVD29 datum to obtain elevations with a NAVD88 datum at this station (Reference 2.4.2-202).

- **Withlacoochee River Bypass Channel near Dunnellon, Florida** (USGS ID: 02313250, #23 on Figure 2.4.1-211): This station is location 2.1 km (1.3 mi.) upstream of the bypass spillway. Daily water stage has been recorded at this station for a period of 36 years (1971 – 2007). (Reference 2.4.1-211) Minimum stage observed at this station during this period is 6.62 m (21.73 ft.) NGVD29 (October 11, 1972) (Reference 2.4.2-205). Add the conversion factor of -0.310 m (-1.02 ft.) to elevations with a datum of NGVD29 to obtain elevations with a datum of NAVD88 at this station (Reference 2.4.2-202).

- **Withlacoochee River at Chambers near Yankeetown, Florida** (USGS ID: 02313272, #24 on Figure 2.4.1-211): This station is located 17.7 km (11 mi.) downstream from the Inglis Dam on the Lower Withlacoochee River at the mouth of Gulf of Mexico. Tidal high and tidal low daily gage height data are only available from January 2005 through July 2007 at this station. (Reference 2.4.1-212) The minimum stage observed at this station during high tides was -0.19 m (-0.62 ft.) NAVD88 (April 16, 2005). The minimum stage observed at this station during low tides was -5.29 m

(-3.29 ft.) NAVD88 (January 28, 2005 and February 6, 2005). (Reference 2.4.2-206)

Hurricane occurrence periods were compared to the dates of lowest observed water levels at USGS stations to determine if a relationship existed. No relationship was noted indicating that other affects, such as drought, are more likely to cause low water levels in the CFBC, Lake Rousseau, and the Lower Withlacoochee River.

The intake elevation of the makeup water pumps in the CFBC is -3.2 m (-10.6 ft.) NAVD88. The minimum water elevation observed at this location is -0.79 m (-2.6 ft.) NAVD88 providing a minimum of approximately 2.4 m (8 ft.) of operating head at the makeup water pumps.

2.4.12 GROUNDWATER

LNP COL 2.4-4

The LNP site is located within the mid-peninsular physiographic zone of the Coastal Plain province of the Atlantic Plain division of North America. The mid-peninsular zone is characterized by discontinuous subparallel ridges lying parallel to the length of the peninsula. These ridges are separated by broad valleys of gently sloping to nearly level terrain. (Reference 2.4.12-201) As shown on Figure 2.4.12-201, the LNP site lies in the localized subdivision of the mid-peninsular zone known as the Gulf Coastal Lowlands. Karst topography is a typical component of the Gulf Coastal Lowlands where carbonate rocks are near the land surface and are subject to dissolution by downward-infiltrating rainfall (References 2.4.12-202 and 2.4.12-203).

The geologic regime underlying the LNP site is known as the Floridan platform, consisting of recently emergent Mesozoic and Cenozoic age shallow marine carbonate and evaporite sediments in a sequence approximately 5 km (3.1 mi.) thick. These sediments overlie Paleozoic igneous, sedimentary, and volcanic basement rocks (Reference 2.4.12-203). Figure 2.4.12-202 presents a generalized north-south geologic cross section of central Florida, including Levy County, within which the LNP site is located. The general relationship between these stratigraphic units and the hydrogeologic units (aquifers) in this area are shown on Figure 2.4.12-203.

The general geomorphology, stratigraphy, economic geology, and physiography of the site vicinity (16.1-km [10-mi.] radius) and region (40.2-km [25-mi.] radius) are presented in more detail in FSAR Subsection 2.5.1. Groundwater data collected at the LNP site are presented in more detail in FSAR Subsection 2.4.12.2.2.

2.4.12.1 Description and On-Site Use

This subsection presents groundwater conditions, sources, and usage of the aquifer in the region and at the LNP site.

2.4.12.1.1 Regional Groundwater Systems

In west-central Florida, the groundwater flow system is a combination of a surficial aquifer comprised of unconsolidated sediments of Quaternary age, and an underlying carbonate rock aquifer of Miocene to Paleocene age rocks known as the Floridan aquifer system (FAS). Deposits comprising the FAS extend into Georgia, Alabama, and South Carolina, receiving recharge from a broad area. In Florida, the FAS consists of an Upper and Lower Floridan aquifer, and ranges in thickness from about 152.4 m (500 ft.) to over 548.6 m (1800 ft.). The Upper Floridan aquifer is the main source of potable water and spring flow in west-central Florida. (Reference 2.4.12-204)

The aquifer systems are differentiated based upon their permeability, with the surficial aquifer being less permeable than the FAS. Where present, surficial aquifers are typically comprised of Quaternary sands and provide substantial recharge to the Floridan aquifer. The principal use of the surficial aquifer is for irrigation, domestic use on a small scale, and dewatering projects for mining or construction. (Reference 2.4.12-205)

In parts of north and central peninsular Florida, the surficial and Floridan aquifer systems are separated and hydraulically confined by the Hawthorn Group, a series of clastic Miocene age marine sediments. The Hawthorn Group is lithologically comprised of interbedded sands and clays that are locally phosphatic with carbonate interbeds. (Reference 2.4.12-206)

The general hydrostratigraphy of the Upper Floridan aquifer in west-central Florida near the LNP site consists of three principal carbonate units: the Suwannee Limestone of Oligocene age, the Ocala Limestone of upper Eocene age, and the upper part of the Avon Park Limestone of middle Eocene age. The Upper and Lower Floridan aquifers are separated by a low permeability carbonate rock sequence known as the Middle Confining Unit (MCU). The MCU is lithologically comprised of varying types of carbonate rocks, from gypsum to chalky limestone. The MCU’s occurrence has been mapped as a group of seven sub-regional to local units, which appear to be associated with the lower sequence of the Avon Park Limestone. (Reference 2.4.12-206)

The underlying Lower Floridan aquifer is less well known and understood geologically. Data indicate the aquifer contains saline water, and is therefore not typically used as a potable water source. The Lower Floridan aquifer is comprised of middle Eocene to upper Cretaceous carbonate beds of varying permeability. In northeast and southern Florida there are sub-regional permeable zones, presumed associated with Eocene paleowater table karst topography development. In some areas, the Lower Floridan aquifer is used for wastewater disposal (injection) wells. (References 2.4.12-202 and 2.4.12-203)

The Upper Floridan aquifer is very productive and serves as the main source of spring flows and potable water for private and municipal supply in the western part of Florida. The estimated transmissivity (T) of the Upper Floridan aquifer has values ranging from 537.6 centimeters squared per second (cm^2/sec)

(50,000 feet squared per day [ft^2/day]) to approximately 1075.3 cm^2/sec (100,000 ft^2/day) in Levy County in the vicinity of the LNP site, and up to 139,784.7 cm^2/sec (13,000,000 ft^2/day) at Silver Springs in Marion County to the east. (Reference 2.4.12-204) These units can be multiplied by 0.001 liters per cubic centimeter (L/cm^3) (7.48 gallons per cubic foot [gal/ft^3]) to obtain units of liters per second per centimeter (gallons per day per foot [gpd/ft]).

2.4.12.1.2 Site Groundwater Systems

Investigations conducted within the LNP site boundary reveal that geologic and hydrologic conditions are essentially the same as the regional conditions described in FSAR Subsection 2.4.12.1.1, with the following site-specific conditions.

The LNP site is a Greenfield site, formerly used as a pine plantation (silviculture), but is otherwise undeveloped except for one grade road and a perimeter unpaved road loop (Reference 2.4.1-230). No wells were known to exist on the property prior to the Combined License (COL) Application field work. The site is relatively level, with very little variation in surface topography, with no rivers, no streams, and no other major drainage features on-site (Reference 2.4.1-203).

A series of wetlands exist on-site, mainly associated with existing cypress tree growth areas. These wetlands and cypress “domes” provide preferential recharge to both the surficial and Floridan aquifers, and may be associated with increased karst feature development in the Avon Park Formation limestones underlying the Quaternary deposits (Reference 2.4.12-203).

As summarized in Table 2.4.12-201, the surface soils present at the LNP site are undifferentiated Quaternary sands of the Smyrna-Immokalee-Basinger (S1547) Series, described as a loamy fine silica sand and fine silty sand, and are poorly to very poorly drained (Reference 2.4.12-207). Distribution of surface soils within a 16.1-km (10-mi.) radius of the LNP site is shown on Figure 2.4.12-204. As discussed in detail in FSAR Subsection 2.5.4, the surficial aquifer resides within these soils, which grade into the carbonate-derived silty sediments of the Avon Park Formation unconformity zone at varying depths on-site.

Published geologic literature indicates that the local hydrostratigraphic sequence at the LNP site consists of Quaternary surficial aquifer deposits lying directly over the Floridan aquifer limestones of the Avon Park Formation. The Hawthorn Group is not present at the LNP site, nor are the Tampa, Suwannee, or Ocala Limestones. (Reference 2.4.12-202) The Upper Floridan aquifer at the LNP site contains fresh potable water, and is separated physically and hydraulically from the underlying Lower Floridan aquifer by sequences of lower permeability evaporate rock units, which act as an aquitard. (Reference 2.4.12-205)

A site investigation that included geotechnical borings was conducted at the LNP site during late 2006 and 2007 to characterize the thickness of unconsolidated Quaternary sediment deposits, to determine the depth to the Avon Park limestone bedrock, and to evaluate the engineering properties of this rock beneath the proposed improved areas. A total of 118 boreholes were advanced during the COL Application field investigations to characterize the subsurface conditions at the LNP 1 and LNP 2 locations.

Rotary drilling with standard penetration testing (SPT) was the typical method employed to advance through soil and subsurface sediments into the upper zone of Avon Park Formation rock. Rock coring was then initiated, using double-tube wireline coring methods to the borehole termination depth. Borehole depths ranged from approximately 18 to 152 m (60 to 500 ft.) bgs. Sonic drilling methods were also utilized in the initial phase for five borings at each reactor site to be used for downhole geophysical testing.

The geotechnical boring program results confirmed that the first carbonate rock units encountered below the surficial aquifer deposits are the deposits of the middle Eocene age Avon Park Limestone. To the maximum investigated depth of 152 m (500 ft.), neither the MCU nor the Lower Floridan aquifer units were encountered. The geotechnical boring program is discussed in more detail in FSAR Subsection 2.5.4.

A surficial water-table aquifer exists on-site within the Quaternary deposits, with typical water-table depths less than 1.5 m (5 ft.) bgs, varying with seasonal rainfall. The surficial aquifer at the LNP site varies in thickness from less than 3 m (10 ft.) to about 60 m (200 ft.) in isolated locations, with an average thickness of approximately 15 m (50 ft.). Water-table data collected in 2007 indicates that the water table ranges in depth at LNP 1 and LNP 2 areas from less than 0.3 m (1 ft.) below ground surface during rainy periods to approximately 1.5 m (5 ft.) bgs during drier periods.

The surficial aquifer transitions into the underlying marine carbonates of the Avon Park Formation gradually rather than with an abrupt bedding contact. The surficial aquifer typically is not used for potable supply in this area, and is not hydraulically confined from the underlying Floridan aquifer within the Avon Park Formation. The Avon Park Limestone comprises the Upper Floridan aquifer and is the main source of potable water in the area. More information on aquifer characteristics is presented in the following sections.

The Upper Floridan aquifer at the LNP site consists of fresh potable water within the Avon Park Formation, and serves as the primary potable aquifer in the area. Both productivity and water quality of the Avon Park are good for private and municipal potable supply. Based on limited downhole geophysical testing and monitoring of drilling fluid losses at the LNP site, the most productive interval of the Upper Floridan aquifer appears to be at depths of approximately 30 to 91 m (100 to 300 ft.) bgs.

The Upper Floridan aquifer is separated from the underlying Lower Floridan aquifer by sequences of lower permeability evaporite deposits and finely crystalline dense dolostones that act as an aquitard and confining unit. This layer, the MCU, can be up to 122 m (400 ft.) thick in the site vicinity and is considered to be part of the Avon Park Formation, stratigraphically.

As discussed in FSAR Subsection 2.5.4, the deep AD-series borings that were drilled to depths of approximately 152 m (500 ft.) bgs did not encounter the MCU or the underlying Oldsmar and Cedar Keys formations that comprise the Lower Floridan aquifer. However, traces of the evaporate deposits and quartz-infilled porosity typical of the MCU were observed sporadically in the AD-series borings at depths below 122 m (400 ft.), indicating that these borings may be approaching the less permeable lower portion (MCU) of the Avon Park Formation.

2.4.12.1.3 On-Site Use of Groundwater

There currently is no groundwater usage at the LNP site. The site is undeveloped with the exception of the former silviculture operations, and water for preapplication field investigations was obtained off-site from the city of Inglis.

During plant construction, the current conceptual foundation designs call for substantial dewatering of each nuclear island area to depths of approximately 30.5 m (100 ft.) below existing grade. A complete description of the site dewatering system, including its reliability in maintaining groundwater conditions within the groundwater design bases of Plant structures, systems, and components (SSC) important to safety, is provided in LNP FSAR Subsections 2.5.4.5 and 2.5.4.6.2. Necessary construction water use permits and approvals will be coordinated through the SWFWMD. Discharge and/or beneficial reuse of extracted water will be coordinated with the SWFWMD, city of Inglis, Levy County, Florida Department of Transportation, USACE, and other jurisdictional entities.

Groundwater from off-site raw water wells will be used to supply general plant operation including service water tower drift and evaporation, potable water supply, raw water to demineralizer, fire protection, and media filter backwash. An average of 3336.8 lpm (881.5 gpm) and a maximum of approximately 15,374.1 lpm (4061.4 gpm) of groundwater will be used for these purposes.

Current conceptual designs call for consumptive use of on-site groundwater extracted from the Floridan aquifer by a system of two to four raw water wells. These wells will be approximately 40.6 cm (16 in.) in diameter and about 91.4 m (300 ft.) deep, but precise well specifications and construction will be determined after installation of a water supply test well, aquifer tests, and numerical modeling, as needed. The supply wells will be spaced at least 228.6 m (750 ft.) apart.

2.4.12.2 Sources

2.4.12.2.1 Present and Future Groundwater Use

Current groundwater use near the LNP site was identified in three ways: using the SWFWMD and SRWMD well permitting databases, using the Florida Department of Environmental Protection's (FDEP's) Source Water Assessment and Protection Program (SWAPP) database, and performing a land use survey. The results of each of these data gathering activities are discussed below.

Permits are required for all wells, regardless of use, within both the SWFWMD and the SRWMD (References 2.4.12-208 and 2.4.12-209). Data was requested from SWFWMD and SRWMD for all well permits issued within 40.2 km (25 mi.) of the LNP site. Data available varied by water management district but generally included Public Land Survey (PLS) section, township, and range of the well location, general well use, well permit number, and general well construction information. Figure 2.4.12-205 presents the PLS section, township, and ranges located within 40.2 km (25 mi.) of the LNP site. Table 2.4.12-202 summarizes SWFWMD well permits and Table 2.4.12-203 summarizes SRWMD well permits; Figure 2.4.12-206 presents the distribution of well permits within 40.2 km (25 mi.) of the LNP site by PLS section, township, and range. More exact locations, such as addresses or coordinates, were not available for many of the wells permitted by SWFWMD and SRWMD.

Data provided by SWFWMD generally included all well permits issued between 1970 and November 19, 2007, and includes permits for the monitoring and production wells installed at the LNP site. Data provided by the SRWMD generally included all well permits issued between 1976 and November 29, 2007. It is important to note that not all of the wells included in the permitting records provided by SWFWMD and SRWMD may still exist.

As presented in Table 2.4.12-202, SWFWMD has issued approximately 53,670 well permits within 40.2 km (25 mi.) of the LNP site, as follows (Reference 2.4.12-210):

- Approximately 77 percent (41,484 wells) of these wells are used for domestic water supply. Figure 2.4.12-207 presents the distribution of permitted domestic wells within 40.2 km (25 mi.) of the LNP site.

- Approximately 2 percent (995 wells) of these wells are used for public water supply. Figure 2.4.12-208 presents the distribution of permitted public supply wells within 40.2 km (25 mi.) of the LNP site.

- Approximately 9 percent (4637 wells) of these wells are used for irrigation. Figure 2.4.12-209 presents the distribution of permitted irrigation wells within 40.2 km (25 mi.) of the LNP site.

- Approximately 12 percent (6554 wells) of these wells are used for other uses including industrial, mining, power, livestock, fire protection, air conditioning supply, aquaculture, geothermal, grounding rod, injection, observation or monitoring, recovery of contaminants, return air/heat, sealing water, and testing/piezometer. Figure 2.4.12-210 presents the distribution of wells permitted for other uses within 40.2 km (25 mi.) of the LNP site. The wells installed at the LNP site as part of 2007 data gathering activities are included in this category.

As presented in Table 2.4.12-203, SRWMD has issued 918 well permits within 40.2 km (25 mi.) of the LNP site, as follows (Reference 2.4.12-211):

- Approximately 88 percent of these wells (804 wells) are self-supplied residential wells. Figure 2.4.12-207 presents the distribution of permitted self-supplied residential wells within 40.2 km (25 mi.) of the LNP site.
- Approximately 3 percent of these wells (24 wells) are public water supply wells. Figure 2.4.12-208 presents the distribution of permitted public water supply wells within 40.2 km (25 mi.) of the LNP site.
- Approximately 1 percent of these wells (12 wells) are used for irrigation and landscaping (commercial and residential). Figure 2.4.12-209 presents the distribution of permitted irrigation wells within 40.2 km (25 mi.) of the LNP site.
- Approximately 8 percent of these wells (78 wells) are used for groundwater monitoring, fire protection, and other uses. Figure 2.4.12-210 presents the distribution of wells permitted for other uses within 40.2 km (25 mi.) of the LNP site.

Additional information on public water supply wells was obtained from FDEP's SWAPP database. Similar to SWFWMD (Reference 2.4.12-208), FDEP defines a public water supply system as one that provides water to 25 or more people for at least 60 days each year or serves 15 or more service connections. These systems may be publicly or privately owned and operated. Public water supply systems are divided into three categories (Reference 2.4.12-212):

- Community — Serves at least 15 service connections used by year-round residents or regularly serves at least 25 year-round residents. This group includes a range of sizes from small mobile home courts to large city utilities.
- Transient Noncommunity — Serves at least 25 people or 15 connections, to flow-through populations, such as stores, recreational vehicle (RV) parks, hotels, or churches that are open at least 60 days per year.
- Nontransient Noncommunity — Serves water to the same individuals for six months or more each year. Includes schools, factories, or large businesses with their own drinking water supplies.

Figure 2.4.12-211 and Table 2.4.12-204 summarize the 46 public water supply systems within 16.1 km (10 mi.) of the LNP site, which include 13 community, 26 transient noncommunity, and 7 nontransient noncommunity public water systems. A total of 64 public supply wells serving approximately 10,300 customers with a total design capacity of approximately 25 mld (6.6 mgd) were identified within 16.1 km (10 mi.) of the LNP site by the SWAPP. The Floridan aquifer is the source water for all of these wells. Three municipal/city public water supply systems are located within 16.1 km (10 mi.) of the LNP site for the cities of Dunellon, Inglis, and Yankeetown. These municipal/city systems account for approximately 7.2 mld (1.9 mgd) or 30 percent of the public water supply design capacity within 16.1 km (10 mi.) of the LNP site. (Reference 2.4.12-213)

Figure 2.4.12-212 and Table 2.4.12-205 summarize the 222 public water supply systems between 16.1 km (10 mi.) and 40.2 km (25 mi.) of the LNP site, which include 65 community, 140 transient noncommunity, and 17 nontransient noncommunity public water systems. A total of 305 public supply wells serving approximately 143,543 customers with a total design capacity of approximately 283.8 mld (75 mgd) were identified between 16.1 km (10 mi.) and 40.2 km (25 mi.) of the LNP site by the SWAPP. The Floridan aquifer is the source water for all of these wells. The five municipal/city public water supply systems located within between 16.1 km (10 mi.) and 40.2 km (25 mi.) of the LNP site include Cedar City Water Treatment Plant, Otter Creek Water Treatment Plant, the City of Crystal River, Inverness Water Department, and Williston Water Treatment Plant. These municipal/city systems account for approximately 29.9 mld (7.9 mgd) or 11 percent of the public water supply design capacity between 16.1 km (10 mi.) and 40.2 km (25 mi.) of the LNP site. (Reference 2.4.12-213)

In December 2007, PEF performed a land use survey within 8 km (5 mi.) of the LNP site to identify the nearest residents to the LNP site. The land area around the LNP site was divided into sixteen 22.5 degree sectors with an 8-km (5-mi.) radius. Within each sector, residents were asked to provide, or a visual inspection was performed to determine, the numbers of food animals kept on the property; whether a food garden greater than 50 square meters (m^2) (538 square feet [$ft.^2$]) was kept on the property and the number, and use, of wells on the property (e.g., potable water, irrigation, etc.). Table 2.4.12-206 presents the distances to the nearest resident to the LNP site within each sector.

All of the residents surveyed use groundwater to supply their potable water needs because no public water is available in this relatively remote portion of Levy County. Figure 2.4.12-213 shows the locations of the nearest resident in each survey sector. The closest surveyed resident is about 2.6 km (1.6 mi.) northwest of the LNP site. Private water wells ranged from 6 to 137 m (20 to 450 ft.) bgs in depth. No other water well details or usage rates were available from private residents.

SWFWMD has estimated that water demand within in Levy County would increase from approximately 49.6 mld (13.1 mgd) in 1994 to approximately 68.5 mld (18.1 mgd) in the year 2020, an increase of 18.9 mld (5.0 mgd) or 38 percent (Reference 2.4.12-214). Actual water use in Levy County in 2005 was approximately 35.942 mld (9.495 mgd), indicating a decrease in water demand in Levy County since 1994 (Reference 2.4.12-215). More current projections of water use within the SWFWMD are not available.

2.4.12.2.2 Groundwater Levels and Movement

Configuration of the potentiometric surface in the immediate vicinity of the LNP site was determined by measuring water levels in observation and monitoring wells installed during the LNP site investigation conducted from March through December of 2007. Sixteen monitoring wells and seven observation wells were installed during the LNP site investigation to accurately characterize the potentiometric surface, gradient, and flow pathways within the vicinity of LNP 1 and LNP 2. Six nested monitoring well pairs (12 out of 16 wells) were installed during the investigation to determine the connectivity between the surficial and bedrock aquifers. In addition, three observation wells were installed as a nested set and one observation well was installed nested with wells MW-13S/MW-14D. Shallow wells were screened within the silt and sand of the surficial aquifer directly above the bedrock interface. Intermediate and deep wells were screened completely within the limestone bedrock of the Upper Floridan aquifer. Groundwater gauging events were conducted quarterly (March, June, September, and December 2007) to account for seasonal and long-term variations. Table 2.4.12-207 summarizes well construction details; Figure 2.4.12-214 presents monitoring well locations; Table 2.4.12-208 summarizes quarterly groundwater elevations; Figures 2.4.12-215, 2.4.12-216, 2.4.12-217, 2.4.12-218, 2.4.12-219, 2.4.12-220, 2.4.12-221, and 2.4.12-222 show potentiometric contour maps for each of the quarterly events.

During the quarterly events in 2007, groundwater was observed to occur between 0 and 2.4 m (0 and 8 ft.) below the ground surface, with the shallowest elevations occurring during the spring event. Because groundwater is shallow and unconfined, groundwater conditions are heavily influenced by the topography of the LNP site. The direction of groundwater flow is toward the west-southwest from a topographic high of approximately 18.3 m (60 ft.) NGVD29 in the western portion of the site toward a topographic low of approximately 10.7 m (30 ft.) NGVD29 in the southwest portion of the site (Figure 2.4.1-203). In the center portion of the site, where the topography is relatively flat, the groundwater surface also becomes relatively flat. No significant differences were observed in the groundwater flow direction or gradient during the quarterly events or between the surficial and bedrock aquifer.

In addition to the groundwater elevation measurements collected during the quarterly events, pressure transducers were installed in MW-13S and MW-15S, which are screened within the surficial aquifer. MW-15S is located in the center of the footprint of LNP 1 and MW-13S is located in the center of the footprint of LNP 2. Groundwater elevation measurements (as pressure) were collected every 12 hours for more than a year at each location. Maximum groundwater elevations were observed during March 2007 and March 2008 at both locations, as shown in Figures 2.4.12-223 and 2.4.12-224. Figures 2.4.12-223 and 2.4.12-224 show that groundwater elevations were more than 2.1 m (7 ft.) below nominal plant grade elevation and more than 2.4 m (8 ft.) below nominal plant floor elevation between March 2007 and March 2008.

Six nested well sets were installed during the LNP site investigation to determine the vertical gradient between the surficial and bedrock aquifers. Shallow monitoring wells were screened within the silt and sand surficial aquifer directly above the soil/bedrock interface. Intermediate and deep monitoring wells were screened completely within the limestone bedrock of the Upper Floridan aquifer. All of the nested wells had a slightly greater hydraulic head within the surficial aquifer than the bedrock aquifer; this condition creates a slight downward vertical gradient (Table 2.4.12-209). Regionally, the USGS has identified the area where the LNP site is located as a recharge/discharge boundary of the Floridan aquifer as shown on Figure 2.4.12-226. Site-specific vertical gradients observed quarterly from early 2007 through early 2008 were all downward and low in magnitude, ranging from 0.0002 to 0.018 ft/ft (Table 2.4.12-209). The small magnitude of the downward gradient and the regional recharge/discharge information from the USGS suggest that the LNP site is located in a transitional area between upward and downward vertical gradients. Nested well pairs MW-15S/MW-16D and MW-13S/MW14D, located within the footprint of the safety-related structures for LNP 1 and LNP 2, respectively, had slight downward vertical gradients with elevation head differences as measured in the field on September 13, 2007, of 0.17 and 0.08 m (0.55 and 0.27 ft.), respectively (Table 2.4.12-209). The direction and magnitude of the vertical gradients between the surficial and bedrock aquifers remained consistent for all nested well sets during each quarterly gauging event.

"Typical" seasonal variations (higher groundwater levels in the spring, lower groundwater levels in the fall) were observed at the LNP site in both the surficial and bedrock aquifers.

An evaluation of maximum post-construction groundwater elevations in the areas between SSCs and the surrounding stormwater drainage ditches during the PMP design storm was performed. A MODFLOW groundwater flow model was developed that addressed the following site-specific post-construction conditions:

- Impervious and pervious areas,
- Groundwater recharge rate/stormwater infiltration rate,
- Subsurface properties, and
- Engineering controls, including the stormwater retention ponds, stormwater drainage system, diaphragm wall, grout curtain, and foundations of safety-related structures.

The MODFLOW model developed is based on known or designed site properties and features and conservative parameters, no calibration was performed.

To assess the maximum infiltration rate at pervious areas that would result in groundwater elevations less than 14.9 m (49 ft.) NAVD88 at safety-related structures, which is an AP1000 DCD requirement based on a nominal plant grade floor elevation of 15.5 m [51 ft.] NAVD88, an iterative approach was used as described below.

- An infiltration rate was selected and kept constant during the analysis.
- Based on the selected infiltration rate, the hourly groundwater recharge rate for pervious areas was calculated from the hourly PMP rate (Table 2.4.3-216).
- Excess precipitation, that which does not infiltrate at pervious areas and that which falls on impervious areas, was applied to the stormwater retention ponds until the ponds reached the design water level.
- Once the ponds reached the design water level, it was assumed that any additional excess precipitation was pumped to the cooling water tower basins such that the ponds remained at the design water level.
- It was assumed that no infiltration from the ponds or pump down of the ponds occurs during the 72-hour period of no precipitation, resulting in a pond condition at the beginning of the 72-hour PMP storm equivalent to that at the end of the antecedent 72-hour storm.
- The selected infiltration rate was increased until groundwater elevations of 14.9 m (49 ft.) were simulated at safety-related structures or the selected infiltration rate was equivalent to the highest hourly PMP rate, whichever occurred first.

The results of this evaluation indicate that a pervious area infiltration rate equivalent to the maximum hourly PMP rate (3.57 in./hr) will not result in groundwater elevations exceeding 14.9 m (49 ft.) NAVD88 at safety-related structures, although groundwater elevations exceeding 14.9 m (49 ft.) NAVD88 are simulated at some locations of the site. Figure 2.4.12-227 and Figure 2.4.12-228 present these results.

The results presented in Figure 2.4.12-227 and Figure 2.4.12-228 are conservative. Groundwater elevations of 50 ft. would not realistically be expected within the pervious areas of the site because groundwater would move down the topographic slope via overland flow once the groundwater surface intersected the ground surface.

The simulated infiltration rate of 3.57 in./hr was limited by the maximum hourly infiltration rate during the PMP design storm. Based on the representation of the probable maximum precipitation design storm as the maximum precipitation that will occur at the LNP site, higher infiltration rates at the pervious areas of the LNP site would result in groundwater elevations similar to those depicted on Figure 2.4.12-227 and Figure 2.4.12-228 because the rise in groundwater elevations would be limited by the precipitation rate, not the infiltration rate.

2.4.12.2.3 Site Hydrogeologic Characteristics

The slug test method was used to determine the in situ permeability or hydraulic conductivity of the surficial and Upper Floridan aquifers at the LNP site. Slug tests were performed at all 23 wells. Table 2.4.12-210 summarizes the slug test results. Average horizontal permeability (hydraulic conductivity) values range from 3.4 x 10^{-4} centimeters per second (cm/sec) (0.9 foot per day [ft/day]) to 1.0 x 10^{-2} cm/sec (28.6 ft/day) in the surficial aquifer. Values ranged from 8.3 x 10^{-4} cm/sec (2.4 ft/day) to 1.9 x 10^{-2} cm/sec (54.4 ft/day) in the Upper Floridan aquifer. These values are indicative of moderate to high permeability conditions.

In addition, an aquifer pumping test was performed at production well PW-1. Results from the aquifer pumping test were used to determine the transmissivity (related to hydraulic conductivity) and specific yield of the surficial aquifer using the Neuman (1974) approach (Reference 2.4.12-216). The Neuman analysis assumes that vertical leakage occurs within the water table during the aquifer test. However, this method does not discriminate between upward leakage and downward leakage, only drainage by gravity. Table 2.4.12-211 summarizes the results of the pumping test at monitoring and observation wells located near PW-1. Figure 2.4.12-225 shows the locations of the aquifer test pumping well and observation wells. Transmissivity values ranged from 14 cm^2/sec (1.3 x 10^3 ft^2/day) to 237 cm^2/sec (2.2 x 10^3 ft^2/day) and specific yield ranged from 1.2 x 10^{-2} to 1.7 x 10^{-1} (dimensionless). These values are indicative of moderate to high permeability conditions and reflect the results of the slug tests discussed above.

In order to more accurately estimate leakage from layers above and below the pumping layer, the surficial aquifer pumping test data and subsequently collected Upper Floridan aquifer pumping test data were analyzed using the computer model Multi-Layer Unsteady State (MLU), a commercial software for analyzing multi-layer aquifer systems. The MLU model was used to evaluate three aquifer test programs conducted at the LNP site: one within the surficial aquifer and two within the Upper Floridan aquifer. The MLU model was selected for this evaluation because it can be used for aquifer test analysis of transient well flow in layered aquifer systems and stratified aquifers. The aquifer test programs at the LNP site involved pumping and monitoring wells screened at three different depth intervals in the surficial aquifer and up to four different depth intervals in the Upper Floridan aquifer.

Table 2.4.12-213 summarizes the results of the surficial aquifer and Upper Floridan aquifer analysis using MLU. Transmissivity values ranged from 4.8 cm^2/sec (4.5 x 10^2 ft^2/day) to 6.2 cm^2/sec (5.8 x 10^2 ft^2/day) and 43 cm^2/sec (4.0 x 10^3 ft^2/day) to 570 cm^2/sec (5.3 x 10^4 ft^2/day) for the surficial and Upper Floridan aquifers, respectively. These values are indicative of moderate to high permeability conditions and are consistent with the results of the slug tests and surficial aquifer analysis using the Neuman (1974) approach discussed above.

Linear groundwater velocity and Darcy flux estimates for the surficial and Upper Floridan aquifers were calculated using site parameters for LNP.
Table 2.4.12-212 presents the results for the seepage velocity and Darcy flux for

the March, June, September, and December 2007 gauging events. Nested monitoring wells were selected both upgradient and downgradient, where possible, for each LNP unit, to more accurately compare the surficial and bedrock aquifers.

For LNP 2, the seepage velocity and Darcy flux for the surficial aquifer between monitoring wells MW-7S and MW-11S ranged from about 1.1 x 10^{-7} cm/sec (0.0003 ft/day) to 7.1 x 10^{-6} cm/sec (0.02 ft/day) and 3.3 x 10^{-4} cubic centimeters per second (cm^3/sec) (0.001 cubic feet per day [ft^3/day]) to 1.3 x 10^{-3} cm^3/sec (0.004 ft^3/day), respectively. Higher values of seepage velocity and Darcy flux were observed in March and lower values were observed during the June groundwater gauging event. For the bedrock aquifer, the seepage velocity and Darcy flux between monitoring wells MW-8D and MW-12D are about 1.8 x 10^{-5} cm/sec (0.05 ft/day) and 3.3 x 10^{-3} cm^3/sec (0.01 ft^3/day), respectively. No significant seasonal variation in these values was observed in the bedrock aquifer.

Similar estimates were calculated for LNP 1. The seepage velocity and Darcy flux for the surficial aquifer between monitoring wells MW-11S and MW-15S ranged from about 3.5 x 10^{-7} cm/sec (0.001 ft/day) to 7.1 x 10^{-6} cm/sec (0.02 ft/day) and 6.6 x 10^{-5} cm^3/sec (0.0002 ft^3/day) to 1.6 x 10^{-3} cm^3/sec (0.005 ft^3/day), respectively. As with LNP 2, higher values of seepage velocity and Darcy flux were observed in March and lower values were observed during the June groundwater gauging event. For the bedrock aquifer, the seepage velocity and Darcy flux between monitoring wells MW-12D and MW-16D are about 2.1 x 10^{-5} cm/sec (0.06 ft/day) to 2.5 x 10^{-5} cm/sec (0.07 ft/day) and 3.3 x 10^{-3} cm^3/sec (0.01 ft^3/day), respectively. No significant seasonal variation in these values was observed in the bedrock aquifer.

2.4.12.2.4 Effects of Groundwater Usage

As of 2005, the SWFWMD had permitted approximately 83.113 mld (21.956 mgd) of nondomestic groundwater use in the portion of Levy County that falls within the SWFWMD. That same year, the SWFWMD estimated that approximately only 29.061 mld (7.677 mgd) of permitted capacity was used (total water demand, which includes unpermitted domestic demands, was 35.942 [9.495 mgd]). (Reference 2.4.12-215) As stated in FSAR Subsection 2.4.1.1, an estimated average of 4.805 mld (1.269 mgd) and a maximum of approximately 22.139 mld (5.848 mgd) of groundwater will be used at the LNP site for nonsafety-related purposes. The groundwater usage at the LNP site will not result in a total groundwater use greater than that already permitted by the SWFWMD based on current water demands in Levy County.

Temporary dewatering of site excavations is anticipated during construction activities. The effects of dewatering on groundwater gradients and flow pathways within the surficial and bedrock aquifers are considered minimal and will not affect local groundwater users. With agency approval, groundwater may be used for limited activities, such as dust control and concrete mixing, at the LNP site during construction.

2.4.12.3 Subsurface Pathways

Potential pathways of contamination to nearby groundwater users and to water bodies such as lakes and streams are identified in FSAR Subsection 2.4.12.2. A conservative analysis of critical groundwater pathways for a liquid effluent release at the site is provided in FSAR Subsection 2.4.13, along with the determination of groundwater and radionuclide travel times to the nearest downgradient groundwater user or surface water body.

2.4.12.4 Monitoring or Safeguard Requirements

This subsection provides a brief summary of the monitoring programs to be used to protect the present and projected groundwater users in the vicinity of the LNP site. The objectives of the groundwater monitoring programs are to identify environmental impacts, including the hydrological and geochemical changes to groundwater, caused by the construction and operation of LNP 1 and LNP 2, and to identify alternatives or engineering measures that could be used to reduce any adverse effects that may be identified.

In general, the groundwater monitoring programs will consist of the following primary elements:

- A Preapplication Monitoring Program for groundwater will support the assessment of site acceptability and establish background conditions for groundwater prior to the construction and operation of LNP 1 and LNP 2.

- A Construction Monitoring Program for groundwater will monitor and be used to control potential effects caused by site preparation and construction activities.

- A Preoperational Monitoring Program will establish a baseline database for identifying and assessing environmental effects attributable to the operation of the LNP 1 and LNP 2.

- An Operational Monitoring Program will be implemented to document groundwater conditions and detect any unexpected effects to the groundwater system from the operation of LNP 1 and LNP 2. The Operational Monitoring Program is anticipated to extend throughout the life of the facility. Modifications to the monitoring program (e.g., changes in monitoring locations, sampling frequency, or collection procedures) will be assessed regularly over the duration of the program.

Aspects of the groundwater monitoring programs are described further in FSAR Subsection 12AA.5.4.14.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

The LNP site will not be employing a permanent dewatering system. Flood design and protection for the AP1000 design is provided in DCD Section 3.4, and the design of seismic Category I structures including the design loads and load combinations are provided in Section 3.8. Seismic Category I structures, systems, and components within the plant site are designed to withstand the effects of elevated groundwater elevations due to natural phenomena. The AP1000 is designed for a normal groundwater elevation up to 0.6 m (2 ft.) below the nominal plant grade. The nominal plant grade elevation for safety-related structures is 15.2 m (50 ft.) NAVD88. The nominal plant grade floor elevation for safety-related structures is 15.5 m (51 ft.) NAVD88.

Groundwater gauging events were conducted in March, June, September, and December 2007 to account for seasonal and long-term variations in the surficial and bedrock aquifers at the LNP site. Nested monitoring well pairs MW-15S/MW-16D and MW-13S/MW-14D were installed within the reactor locations of LNP 1 and LNP 2, respectively. Of these wells, surficial aquifer monitoring wells MW-15S and MW-13S recorded the highest groundwater elevations, which ranged from 11.55 to 12.82 m (37.88 to 42.05 ft.) NAVD88 and 11.48 to 12.78 m (37.66 to 41.94 ft.) NAVD88, respectively. The average groundwater elevation for the four monitoring events at MW-15S and MW-13S are 11.98 m (39.30 ft.) and 11.92 m (39.12 ft.) NAVD88.

Final grading of the LNP site will result in potential hydrologic alteration, including the permanent change in groundwater levels within the plant site from site grading and a series of stormwater drainage ditches. The locations of LNP 1 and LNP 2 are in the central portion of the plant site at a pre-construction grade elevation of approximately 12.8 m (42 ft.) NAVD88. The pre-construction grade at each unit location will be filled (raised) to a nominal plant grade elevation of 15.2 m (50 ft.) NAVD88, affecting the current drainage pattern.

After site grading, a series of stormwater drainage ditches will be constructed around and within the site to direct stormwater and intercepted groundwater away from the footprint of LNP 1 and LNP 2. Impervious surfaces around safety-related structures will direct stormwater runoff toward these drainage ditches and to the retention ponds. Stormwater drainage ditches installed within the LNP site will have bottom elevations ranging from approximately 12.97 m (42.55 ft.) NAVD88 or lower to approximately 14.57 m (47.80 ft.) NAVD88 (Figure 2.4.1-205).

Groundwater elevations within the footprint of LNP 1 and LNP 2 meet the requirements for the AP1000 design as provided in the DCD. No dynamic water forces associated with normal groundwater levels will occur because of a higher finished plant grade.

2.4.13 ACCIDENTAL RELEASE OF RADIOACTIVE LIQUID EFFLUENTS IN GROUND AND SURFACE WATERS

LNP COL 2.4-5
LNP COL 15.7-1

This subsection presents a conservative analysis of the effect of accidental release of liquid effluents to the groundwater and surface water environments. A release of the contents of the waste liquid system effluent holdup tank is postulated. The groundwater transport of radionuclides through the surficial and Floridan aquifers is examined. The resultant nuclide concentrations in the nearest potable water supply is evaluated and compared to 10 Code of Federal Regulations (CFR) 20 regulatory limits. Water "supplies" are defined as a well or surface water that is used for direct human consumption or indirectly through animals, crops, or food processing.

2.4.13.1 Radioactive Tank Rupture

This event is defined as an unexpected and uncontrolled release of radioactive water produced by plant operations from a tank rupture. The AP1000 tanks that normally contain radioactive liquid are listed in Table 2.4.13-201 and discussed below.

No outdoor tanks contain radioactivity. Specifically, the AP1000 does not require boron changes for load follow, and so does not recycle boric acid or water; therefore, the boric acid tank does not contain radioactivity.

The spent resin tanks are excluded from consideration in this evaluation because most of their activity is bound to the spent resins, and 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 in this evaluation because the containment building, a seismic Category I structure, is a freestanding cylindrical steel containment vessel. The steel liner is assumed to mitigate the effect of a postulated tank failure.

The liquid radwaste system (WLS) waste monitor tanks located in the radwaste building extension are considered in this evaluation because of their location in a nonseismic building. These three tanks have a maximum capacity of 56,781 liters (L) (15,000 gallons) each, and contain processed fluid ready for discharge. The radwaste building has a well-sealed, contiguous basemat with integral curbing that can hold the maximum liquid inventory of any tank. Floor drains in the area lead to the liquid radwaste system. The foundation for the entire building is a reinforced concrete mat on grade. 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 limited relative to the radiological consequences of leakage from the effluent holdup tanks, discussed below. Therefore, these tanks are excluded from further evaluation.

The remaining four tank applications were considered – the effluent holdup tanks, waste holdup tanks, waste 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 are excluded from further evaluation because they have lower isotope 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 considered a conservative selection for the purpose of calculating the effects of the failure of a radioactive liquid-containing tank. This failure is classified as a limiting fault.

There are two 105,992-liter (28,000-gallon) effluent holdup tanks per unit. For this evaluation, one tank is postulated to fail. The failed tank is assumed to be 80 percent full and contain 84,793 L (22,400 gallons) of waste effluent. The failed effluent holdup tank is assumed to have maximum radionuclide concentrations corresponding to 101 percent of the reactor coolant source term with the following:

- A tritium source term of 1.0 microCuries per gram (μCi/g).
- Corrosion product concentrations for Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58, and Co-60 taken from DCD Table 11.1-2.
- All other isotopes based on DCD Table 11.1-2 scaled to a design defective fuel fraction of 0.12 (that is multiplied by the ratio 0.12/0.25) to adjust defect rate from the design basis to a conservatively bounding value for this evaluation.

The tank inventory released to the groundwater is shown in Table 2.4.13-202.

2.4.13.2 Groundwater Scenarios

The contents of the effluent holdup tank are assumed to be immediately released 10.4 m (34 ft.) below grade at the bottom floor of the auxiliary building. No credit is taken for holdup by the building's sealed walls.

The potentiometric contours in Figures 2.4.12-215, 2.4.12-216, 2.4.12-217, 2.4.12-218, 2.4.12-219, 2.4.12-220, 2.4.12-221, and 2.4.12-222 show that the groundwater elevations in the surficial and Floridan aquifers gradually decrease from northeast to southwest across the LNP site, indicating that the groundwater in these aquifers flows toward the southwest at the LNP site. Table 2.4.12-208 summarizes the groundwater elevations in the surficial and Floridan aquifers at the LNP site, which are within 0 to 2.4 m (0 to 8 ft.) of pre-construction site grade and 1.5 to 4.6 m (5 to 15 ft.) of nominal plant floor grade, depending on seasonal recharge.

The surficial aquifer is not a well-developed aquifer system near the LNP site and no users of surface water have been identified near the LNP site. The thickness of the surficial aquifer system averages approximately 15 m (50 ft.) at the LNP site.

The Floridan aquifer is the principal source of potable water near the LNP site. The Upper Floridan aquifer is extensively developed as an aquifer system with a thickness of at least 152 m (500 ft.) to the MCU. Most wells are screened across only the more productive, initial 30 m to 91 m (100 to 300 ft.) of the aquifer, with well depths typically being no more than 76 m (250 ft.). Public supply wells in the direction of groundwater flow are at least 5 km (3.1 mi.) from LNP 1 and 2, as shown on Figures 2.4.12-208 and 2.4.12-212. The nearest resident in the direction of groundwater flow is 2.7 km (1.7 mi.) west-southwest of LNP 1 and 2, as shown on Figure 2.4.12-213.

As discussed in FSAR Subsection 2.4.12.1.3, groundwater extracted from the Upper Floridan aquifer will be used for potable and process water at LNP. Current conceptual designs call for a system of four raw water wells. These wells will be approximately 40.6 cm (16 in.) in diameter and about 91.4 m (300 ft.) deep. The wells are located south of the LNP site on PEF-owned property. Two wells are along County Road 40 and two wells are located to the north on the east side of the heavy haul road. The nearest raw water well is approximately 3.5 km from LNP 1 and LNP 2 safety-related structures. The average daily withdrawal from the Floridan aquifer is 1.58 million gallon per day (mgd).

Operation of the LNP well field will have minimal impact on the direction of radionuclide transport. Drawdown levels for the well field were analyzed as part of the process for obtaining well permits from SWFWMD (Reference 2.4.13-206). The evaluation included a MODFLOW simulation of LNP withdrawals for 60 years, the expected life of the facility. The additional drawdown in the surficial and Upper Floridan aquifers after 60 years of pumping is less than 0.06 m (0.2 ft.) southwest of LNP 1 and 2 at the postulated location of a well on the site boundary.

The direction of the radionuclide path to the well receptor on the LNP site boundary will be largely unaffected by well field pumping. The average hydraulic gradient is 5E-4 ft/ft in the Upper Floridan aquifer in the direction of the spill path (Table 2.4.12-212). The well field's drawdown will result in a smaller flow with gradient less than 3E-5 ft/ft generally perpendicular to the spill path based on the drawdown contours in Reference 2.4.13-206. The gradient is a small fraction of the hydraulic gradient along the transport path and has no significant influence on the groundwater flow to the receptor.

In the event of an accidental release, PEF is expected to monitor water pumped from the well field and restrict its use as a potable water supply, as appropriate.

2.4.13.2.1 Most Conservative of Plausible Conceptual Models

The most conservative conceptual model for well users was developed by considering a well receptor on the site boundary. The receptor is located 2 km (1.2 mi.) from the spill on the expected centerline of the transport path in the Upper Floridan aquifer.

The receptor location is closer to the spill site than the nearest private well (Figure 2.4.12-213). This approach is used since the LNP site boundary does not extend to a natural barrier or to U.S. Highway 19 to the southwest. Future wells could be constructed closer than the wells determined in the PEF site survey. The use of a well receptor at the boundary inherently makes allowances for future users and predicts more conservative concentrations.

The spill is assumed to be released to the surficial aquifer approximately 7.6 m (25 ft.). below the top of the surficial aquifer near the mid-elevation of this 15.2-m (50-ft.). thick aquifer. No credit is taken for retention of the release between the auxiliary building and the dewatering "bathtub" structure used to minimize groundwater inflow during excavation and construction (FSAR Subsection 2.5.4.6.2).

The spill is analyzed for the Upper Floridan aquifer. All wells that receive permits from the water authorities must take their potable supply from this source. Transport of some radionuclides to the Upper Floridan aquifer is plausible because (1) it is not hydraulically confined from the overlying surticial aquifer (FSAR Subsection 2.4.12.1.2) and (2) persistent downward vertical gradients exist from the surficial aquifer to the Upper Floridan aquifer (Table 2.4.12-209). Therefore, transport paths could include the following: (1) migration in the surficial aquifer; (2) migration into and through the Upper Floridan aquifer; or (3) migration in both aquifers as radionuclides are carried from the spill location. Based on the aquifer characteristics and pore velocity, a conservative analysis is performed assuming an immediate release entirely to the Upper Floridan aquifer. This assumption gives the highest radionuclide concentrations at the well receptor location.

A conservative conceptual model is also developed as an alternate scenario for users of surface water from the Withlacoochee River. There are no known users of the river for domestic or public water supplies. However, it could conceivably be used for irrigation downstream of where the discharge from the Inglis Bypass Channel enters the river. Eventually, the river becomes too brackish for use as it flows to the Gulf.

For this scenario, radionuclides are released and the contaminated groundwater resurfaces in the Withlacoochee River 7 km (4.3 mi) away at the nearest location along the expected transport path. Again, pathways could include transport through the following: (1) the surficial aquifer; (2) the Upper Floridan aquifer; or (3) both aquifers. All radionuclides are assumed to be directly released to and transported through the Upper Floridan aquifer. This assumption is conservative since it results in the maximum radionuclide flux entering the river, though it is less likely than transport through the surficial aquifer or a combination of both aquifers. The activity entering the river is then diluted by the river flow; no credit is taken for dilution by the volume of water near where the activity enters the river. River flow is assumed to be minimal, corresponding to the lowest monthly average flow from the Inglis Bypass Channel of 22.4 m^3/s (790 cfs) for the period 1990 to 2006 (Reference 2.4.13-201).

2.4.13.2.2 Radionuclide Transport

Radionuclide concentrations are evaluated from simplified models for horizontal flow through aquifers. The radionuclides are assumed to be released directly into the saturated region below the water table. No credit is taken for delays or removal of nuclides as water seeps from the building into the groundwater.

Nuclide concentrations are calculated for flow through the saturated aquifer. In this region the water flows along meandering paths through the pores and around grains of soil or rock while some is trapped in microscopic voids. The dissolved radionuclides moving through the aquifer are assumed to be in equilibrium with similar nuclides adsorbed on to pore and grain surfaces.

One-dimensional advection is assumed as a simplification with flow in the x direction; however, dispersion occurs in three dimensions. The general transport equation describing the concentration C for a nuclide in the groundwater is (NUREG/CR-3332):

$$\partial C/\partial t = 1/R_d \left(-U_x \, \partial C/\partial x + D_x \, \partial^2 C/\partial x^2 + D_y \, \partial^2 C/\partial y^2 + D_z \, \partial^2 C/\partial z^2 \right) - \lambda C$$

Equation 2.4.13-1

where

C is the concentration of the nuclide of interest.

R_d is the retardation coefficient (dimensionless).

U_x is the groundwater velocity in the x direction.

D_x, D_y, and D_z are the dispersion coefficients in the x, y, and z directions, respectively.

λ is the radionuclide decay constant.

The retardation factor is given by

$$R_d = 1 + \rho/n_e \, K_d$$

Equation 2.4.13-2

where

ρ is the bulk dry density of the material through which the groundwater moves.

n_e is the effective porosity which is the effective volume of the material where water is actually free to move.

K_d is the distribution coefficient which is dependent of the soil's physical and chemical properties.

The seepage or pore velocity is calculated as

$$U_x = v/n_e \qquad \text{Equation 2.4.13-3}$$

where v is the volumetric flow rate of groundwater per unit area perpendicular to the flow. This value is the same as the Darcy flux (which has units of velocity) determined from the hydraulic conductivity and water-table head gradient data in Table 2.4.12-212.

A general solution for the groundwater concentration C(x, y, z, t) in an aquifer is

$$C(x, y, z, t) = C_0/(n_e\,Rd)\,X(x, t)\,Y(y, t)\,Z(z, t) \qquad \text{Equation 2.4.13-4}$$

where C_0 is the initial source (Ci), and X(x, t), Y(y, t) and Z(z, t) are Green's functions in the x, y, and z directions, respectively (NUREG/CR-3332).

The radionuclides transported to the Lower Withlacoochee River are estimated from the rate that activity crosses an imaginary plane perpendicular to the x-directed flow near the Lower Withlacoochee River. After substituting in the Green's functions, appropriate to a point source configuration, and integrating out the transverse (y and z) dependencies, the rate F(x, t) that activity crosses a plane at distance x in the aquifer and flows into the surface water is

$$F(x, t) = C_0/R_d\,[U_x\,X\,(x, t) - D_x\,\partial X(x, t)/\partial x] \qquad \text{Equation 2.4.13-5}$$

The activity flowing into the Lower Withlacoochee River is diluted by the net flow Q, resulting in concentration F(x, t) / Q. After substituting the appropriate Green's function for X (x, t) and maximizing the resulting expression for C(x, t) as t approaches xR_d/U_x, the maximum nuclide concentration is

$$C_{max}(x) = \frac{C^* V_T\,U_x \exp\{-\lambda x R_d/U_x\}}{2\,R_d\,Q\,(\pi\,\alpha_L\,x)^{1/2}} \qquad \text{Equation 2.4.13-6}$$

The concentration $C_{max}(x)$ is used to bound the Lower Withlacoochee River's concentration and make comparisons to regulatory limits.

The maximum concentration at a well in the Upper Floridan aquifer is taken as the aquifer's concentration at the distance downgradient from the point of accidental release with vertical mixing across the well's depth within the aquifer. Again, a maximum concentration is determined as t approaches xR_d/U_x. The maximum concentration at a well at distance x is

$$C_{max}(x) = \frac{C^*\,V_T \exp\{-\lambda\,x\,R_d/U_x\}}{4\,\pi\,n_e\,R_d\,h\,x\,(\alpha_L\,\alpha_T)^{1/2}} \qquad \text{Equation 2.4.13-7}$$

where

C^* is the concentration in the holdup tank.

V_T is the tank's volume.

Q is the net diluting flow to the Lower Withlacoochee River.

h is the productive depth of the aquifer taken as the depth of the well within the aquifer.

α_L and α_T are the longitudinal and transverse dispersivities.

$C_{max}(x)$ is evaluated for each radionuclide and each case.

2.4.13.2.3 Distribution Coefficient and Dispersivity

Tritium, cesium, and strontium are typically the dominant contributors to the effective concentration limits in 10 CFR 20, Appendix B, Table 2. Conservative (low) values are used for cesium and strontium's distribution coefficients, K_d, based on testing of LNP site soils and limestone, as discussed below. K_d is assumed to be zero for all other nuclides.

Cesium typically exists in groundwater as an uncomplexed Cs^+ ion that adsorbs strongly to most minerals. Strontium in aqueous solution exists principally as an uncomplexed Sr^{++} ion, which increases its likelihood of adsorption. Conversely, Sr^{++} can be de-adsorbed readily from the surtaces of solids unlike Cs^+. As such, it can be expected to have a smaller K_d than cesium (Reference 2.4.13-202).

Eight limestone and eight soil samples were taken from LNP core borings. In addition, surficial and Floridan aquifer groundwater samples were collected from LNP wells MW-15S and MW-14D. The samples are identified in Table 2.4.13-206. The locations of the sample boreholes identified in Table 2.4.13-206 are shown on Figure 2.5.4.2-201A.

K_d testing was performed at Argonne National Laboratory (ANL) for cesium and strontium. Testing methods were based on ANL's Standard Operating Procedure ACL-264, "Determination of the Distribution Coefficient (K_d) in Soil Samples". The procedure included quality control measures such as blank runs, laboratory control standard runs, and reference runs for each of the K_d batches.

The test used site groundwater water as the solvent for tracer compounds used to test for the elemental K_ds. Soil and limestone samples were then saturated for 20 days in the spiked tracer solution to ensure that the adsorbed and solution concentrations were at equilibrium. K_ds were then calculated using the initial concentrations in the soil and limestone samples, as well as the initial and final equilibrated spiked groundwater tracer concentrations.

The borehole locations, sample descriptions, and measured K_ds are shown in Table 2.4.13-206. At LNP the Smyrna-Immokalee-Basinger composite above the Floridan aquifer is mainly sand and fine sand with some silt, as shown in Table 2.4.12-201. Minimum surficial aquifer K_ds are 6 milliliter per gram (ml/g) for cesium and 0 ml/g for strontium based on the site-specific tests. As discussed in FSAR Subsection 2.4.12.1.2, the Floridan aquifer consists primarily of sandy limestone. Minimum K_ds are 4 ml/g for cesium and 2 ml/g for strontium for the Floridan aquifer. All values are based on testing of site samples. The transport times for the center of the radionuclide plume to reach the Lower Withlacoochee River and the nearest resident are tabulated in Table 2.4.13-203. These effective times include the effect of sorption and retardation.

The dispersion coefficients D_x, D_y, and D_z are effective values that account for the observed dispersion through the saturated medium. The dispersion coefficients for x directed flow are $D_x = \alpha_L U_x$ and $D_y = D_z = \alpha_T U_x$ where α_L and α_T are the longitudinal and transverse dispersivities, respectively. Codell and Duguid show longitudinal dispersivity of α_L = 10 to 15 m (32.8 to 49.2 ft.) for limestone and carbonate aquifers (NUREG/CR-3332). Empirical observations also reveal that transverse dispersivity is typically 10 to 20 percent of longitudinal dispersivity (Reference 2.4.13-203). This evaluation conservatively assumes α_L = 1 m and $\alpha_L{*}\alpha_T$ = 1 m^2 (10.8 ft^2) to maximize the concentrations in the Lower Withlacoochee River and the nearest well, respectively.

2.4.13.2.4 Chelating Agents and Impact on Groundwater Transport

Chemical decontaminating agents that remove built-up radioactive activation and corrosion products often use industrial chelating agents to form complexes with transition metals such as iron, cobalt, nickel and manganese. Transition metals typically have oxidation states +2, +3, +4 and +6, which allow the complex to form. With the exception of cesium and strontium, the groundwater transport analysis does not credit retardation of radionuclides, that is, K_d = 0 ml/g, and these nuclide concentrations for the Floridan aquifer pathways are conservatively bounded for chelating effect by the existing analyses.

Unlike transition metals, aqueous complexing is not thought to influence cesium behavior greatly in most groundwater systems. Cesium and strontium are alkaline and alkaline earth metals, respectively. There is little tendency for cesium or strontium to form aqueous complexes: Cs^+ is mono-valent; studies of leachates from contaminated sites showed essentially all strontium existed as uncomplexed Sr^{+2} (References 2.4.13-202 and 2.4.13-205). Complexing of Sr^{+2} is believed to be poor because both cations must compete with naturally occurring Ca^{+2} and Mg^{+2} cations, which are at appreciably higher concentrations in groundwater.

The K_d testing described in FSAR Subsection 2.4.13.2.2 used site groundwater, soils, and limestone samples. The results of the test would implicitly include the effects of naturally occurring complexes if they be present at the LNP site.

The effluent holdup tank radionuclide inventory is from reactor coolant received from the chemical and volume control system (CVS), sampling systems, and miscellaneous leakage. The use of chelates and complexing agents in the AP1000 reactor coolant system for routine operation is not anticipated. Chemicals involved on a routine basis are boric acid buffered with 7-lithium hydroxide, gaseous hydrogen, and small amounts of soluble zinc injected as a zinc acetate solution to reduce corrosion product build-up (DCD Subsection 9.3.6.2.3.3). Hydrazine or hydrogen peroxide may be injected in small quantities at start up or during cool down, respectively (DCD Table 5.2-2).

Chelating and complexing agents will not be introduced to the liquid radwaste system. Historically, there has been some use of chelating agent ethylenediaminetetraacetic acid (EDTA) in steam generator chemical cleaning and advanced scale conditioning evolutions in the nuclear industry. However, these applications are not routine. The use of chelatIng agents at LNP would need to be reviewed by PEF prior to implementation, as would any maintenance activity that would use EDTA or other industrial chelates.

Chelating agents are unlikely to have been used at the LNP site. The site was previously used by a timber company prior to its acquisition by PEF. Southern yellow pines were grown and harvested for wood fiber using modern silviculture practices. The site is undeveloped except for a network of lime-rock roads that were constructed for logging and hunting access. Therefore, no residual chelating agents are expected to exist at the site.

2.4.13.2.5 Compliance with 10 CFR Part 20

Table 2.4.13-202 identifies the radionuclide source term in the effluent holdup tank and the public exposure effluent concentration limit (ECL) for liquid effluents given in 10 CFR 20, Appendix B,Table 2.

Table 2.4.13-204 shows the minimum dilution factors and maximum activity concentrations in the Lower Withlacoochee River. The dilution factor includes the effects of radiodecay and dispersion in the aquifer. Extremely small or zero dilution factors are indicative of essentially complete radiodecay and/or retardation prior to reaching the river. This is shown by the long transport times in Table 2.4.13-203. The long transport times are due to the low seepage velocities in the Floridan aquifer, distance between LNP 1 and 2 and the Lower Withlacoochee River, and retardation of cesium and strontium. Table 2.4.13-204 compares the relative fraction for each nuclide’s concentration to the ECL and shows that the concentrations are negligible compared to the nuclides’ ECL.

A substantial release directly to the Floridan aquifer is unlikely. However, the impact on public and private water use was examined should such a release occur. Table 2.4.13-205 shows bounding activity concentrations that could occur at the nearest private or public well 2 km (1.2 mi.) from the LNP site. With the exception of tritium, the maximum activity concentration for each radionuclide at the closest well is negligible compared to the nuclides’ ECL. The maximum activity concentration of tritium is less than 0.7 percent of its ECL. Again, the effects of dilution, radiodecay, and retardation are evident.

Table 2 in 10 CFR 20, Appendix B, has additional requirements for mixtures of radionuclides. The sum of the individual ratios of nuclide activity concentration to its ECL must be less than unity. This quantity is determined as the sum over the last column in Tables 2.4.13-204 and 2.4.13-205. Radionuclide concentrations in the Lower Withlacoochee River will result in an effective dose equivalent that is negligible when compared to allowable concentrations (Table 2.4.12-204). Similarly, Table 2.4.13-205 shows that maximum activity concentrations in well water from the Floridan aquifer will have an effective dose equivalent of less than 0.7 percent of the regulatory allowable. Tritium is responsible for essentially the entire dose for water use derived from wells.

The accidental release of effluents to groundwater results in effective dose equivalents that are very small fractions of the limits in 10 CFR 20 for water supplies derived from groundwater aquifers.

2.4.13.3 Surface Water

No outdoor tanks contain radioactivity in the AP1000 design. Therefore, no accident scenario could result in the release of liquid effluents directly to surface water.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

LNP COL 2.4-6

The LNP site, together with its safety-related facilities, will be designed to function and shut down in a safe manner despite the occurrence of any of the adverse hydrological events discussed in the preceding subsections. Seismic Category I structures, systems, and components are designed to withstand the effects of flooding due to natural phenomena as discussed in Subsection 3.4.1.1 of the DCD. The AP1000 design does not have a safety-related cooling water system and, therefore, does not rely on the service water and component cooling water systems to provide safety-related safe shutdown. Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell.

Flooding of the safety-related structures and facilities is not a concern at the LNP site. The effects of the local PMP on drainage areas adjacent to the power block safety-related facilities, including the drainage from the roofs of the facilities, are evaluated in FSAR Subsection 2.4.2.3. The effects of PMP on the Withlacoochee River Drainage Basin and the resulting PMF (including wind setup, wave height, wave period and wave runup) are described in FSAR Subsection 2.4.3. The effects of wave generating wind activity from a PMH are described in FSAR Subsection 2.4.5.

No emergency protective measures need to be designed to minimize the impact of adverse hydrology-related events on safety-related facilities.

STD DEP 1.1-1

2.4.15 COMBINED LICENSE INFORMATION

2.4.15.1 Hydrological Description

LNP COL 2.4-1 This COL item is addressed in FSAR Subsections 2.4.1.2 and 2.4.10.

2.4.15.2 Floods

LNP COL 2.4-2 This COL item is addressed in FSAR Subsections 2.4.2, 2.4.3, 2.4.4, 2.4.5, 2.4.6, 2.4.7, 2.4.8, and 2.4.9.

2.4.15.3 Cooling Water Supply

LNP COL 2.4-3 This COL item is addressed in FSAR Subsection 2.4.1.1.

2.4.15.4 Groundwater

LNP COL 2.4-4 This COL item is addressed in FSAR Subsection 2.4.12.

2.4.15.5 Accidental Release of Liquid Effluents in Ground and Surface Water

LNP COL 2.4-5 This COL item is addressed in FSAR Subsection 2.4.13.

2.4.15.6 Emergency Operation Requirement

LNP COL 2.4-6 This COL item is addressed in FSAR Subsection 2.4.14.

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LNP COL 2.4-1

Table 2.4.1-201 (Sheet 1 of 2)
USGS Station Summary, Levy County

USGS Station Name	USGS Station Identification	Latitude	Longitude	Drainage Area ($mi.^2$)	Location	Datum	Conversion to NAVD88 (ft.)	Stage Height: Monitoring Period	Stage Height: Maximum (ft.)/ Date	Stage Height: Minimum (ft.)/ Date	Monthly Discharge: Monitoring Period	Monthly Discharge: Average (cfs)	Monthly Discharge: Maximum (cfs)/ Month	Monthly Discharge: Minimum (cfs)/ Month
Withlacoochee River at Dunnellon, Florida[(a)]	02313200	29°02'45"	-82°27'53"	1960	0.8 mi. upstream of Lake Rousseau at junction with Rainbow River	NGVD 29	-0.876	February 6, 1963 – present	30.37 / September 27, 2004	23.10 / October 11, 1972	ND	ND	ND	ND
Withlacoochee River at Inglis Dam near Dunnellon, Florida[(b)]	02313230	29°00'35"	-82°37'01"	2020	Upstream of Inglis Dam	NGVD 29	-1.01	October 1, 1985 – present	28.03 / March 27, 2005	24.14 / January 13, 1990	October 1, 1969 – present	443	816 / October	178 / June
Withlacoochee River below Inglis Dam near Dunnellon, Florida[(c)]	02313231	29°00'35"	-82°37'01"	ND	Downstream of Inglis Dam	NGVD 29	-1.01	October 1, 1969 – present	9.25 / March 20, 1998	-1.85 /January 16, 1972	ND	ND	ND	ND
Withlacoochee River Bypass Channel near Inglis, Florida[(d)]	02313250	29°01'15"	-82°38'17"	ND	1.3 mi. upstream of Inglis Bypass Channel Spillway	NGVD 29	-1.02	July 16, 1971 – present	28.11 / January 2, 1994	21.73 / October 11, 1972	October 1, 1970 – present	1049	1110 / September	949 / June
Withlacoochee River at Chambers Island near Yankeetown, Florida[(e)]	02313272	29°00'03"	-82°45'44"	ND	At the Gulf of Mexico, 11 mi. downstream of Inglis Dam	NAVD 88	NA	January 28, 2005 – July 23, 2007	High Tide: 4.47 / June 13, 2006 Low Tide: 0.46 / March 21, 2006	High Tide: -0.62 / April 16, 2005 Low Tide: -3.29 / January 28 and February 6, 2005	ND	ND	ND	ND

LNP COL 2.4-1

Table 2.4.1-201 (Sheet 2 of 2)
USGS Station Summary, Levy County

USGS Station Name	USGS Station Identification	Latitude	Longitude	Drainage Area (mi.2)	Location	Datum	Conversion to NAVD88 (ft.)	Stage Height: Monitoring Period	Stage Height: Maximum (ft.)/ Date	Stage Height: Minimum (ft.)/ Date	Monthly Discharge: Monitoring Period	Monthly Discharge: Average (cfs)	Monthly Discharge: Maximum (cfs)/ Month	Monthly Discharge: Minimum (cfs)/ Month
Rainbow Springs near Dunnellon, Florida [f]	02313100	29°06'08"	-83°26'16"	ND	At the head of the Rainbow River, 5.7 mi. upstream of junction with Withlacoochee River	NGVD 29	-0.679	ND	ND	ND	January 1, 1965 – present	698	748 / October	663 / June

Notes:

ND = no data, NA = not applicable
mi.^{2} = square mile, ft. = foot, cfs = cubic foot per second

Sources:
a) References 2.4.1-208, 2.4.1-228, 2.4.2-201, 2.4.2-202
b) References 2.4.1-209, 2.4.1-226, 2.4.1-227, 2.4.2-202, 2.4.2-203
c) References 2.4.1-210, 2.4.2-202, 2.4.2-204
d) References 2.4.1-211, 2.4.1-214, 2.4.1-215, 2.4.2-202, 2.4.2-205
e) References 2.4.1-212, 2.4.2-202, 2.4.2-206
f) References 2.4.1-213, 2.4.1-217, 2.4.1-218

LNP COL 2.4-1

Table 2.4.1-202 (Sheet 1 of 2)
Monthly Average Streamflow Measurements for Withlacoochee River Bypass Channel near Dunnellon, Florida

Levy County
USGS Station Identification #: 02313250
Hydrologic Unit Code: 03100208
Latitude: 29°01'15"
Longitude: -82°38'17"

	Monthly Mean Streamflow, in cfs											
Year	January	February	March	April	May	June	July	August	September	October	November	December
1970	1131	961.8	1217	1293	1427	1372	1319	1336	1353	1366	1245	1180
1971	1184	1320	1228	1175	917.4	782	922.3	1411	1405	1393	1402	1243
1972	1123	1283	1187	1436	1098	1062	1056	1107	654.3	264.9	240.5	484
1973	966.5	1463	1396	1468	1233	1072	1221	1384	1577	1594	1243	1196
1974	1210	1030	992.2	875.5	783.6	916.3	1311	1063	1264	1513	1269	1164
1975	1100	1019	860.3	792.5	723.7	683.7	730.4	807.5	1149	1408	1278	1034
1976	971	855.6	740.1	712.2	893.4	1256	1539	1395	1282	1322	1098	1115
1977	1365	1407	1292	937.3	759.6	750	749	788.7	921	851.2	775.7	888.4
1978	1070	1394	1326	1442	1192	1158	1248	1550	1472	1040	896.3	914.2
1979	1069	1127	1255	1123	1457	1232	1035	1182	1428	918.3	1566	1551
1980	1429	1366	1258	1262	1170	1115	1388	1201	1169	962.3	1097	1080
1981	932.9	991.9	886.7	773	658	663.1	621.5	674.1	710.6	727.5	736.1	729.9
1982	844.8	1017	1414	1562	1323	1397	1122	1228	1096	997.2	1027	920
1983	684.3	1049	1459	1387	1430	1508	1482	1462	1459	1478	1462	1444
1984	1418	1467	1408	1574	1518	1551	1548	1557	1433	1168	1193	1029
1985	956.1	892	748.8	664.7	574	724.5	836.4	1289	1098	1359	1312	1171
1986	1445	1475	1480	1323	979.2	971.5	1070	896.5	1146	1043	903.5	951.1
1987	1028	1181	1339	1201	1427	1295	1287	1027	1004	861.7	492.5	1093
1988	1130	1367	835.4	1549	1291	1207	1114	1217	1124	1256	1475	1565
1989	1549	1502	1384	1046	896.3	915.6	978.5	928.2	925	827.3	831.3	779.3

LNP COL 2.4-1

Table 2.4.1-202 (Sheet 2 of 2)
Monthly Average Streamflow Measurements for Withlacoochee River Bypass Channel near Dunnellon, Florida

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1990	560.7	576.3	649.3	699.8	585.5	636	688.1	811.8	890.1	753.1	619.8	572
1991	571.9	548	688.2	807.2	853.3	1011	1075	1400	1276	920.6	754.5	692.6
1992	561.2	640.9	572.1	569	530	566.8	609.6	663.8	642.3	650	908.8	946.4
1993	839.3	894.1	1107	1210	818.1	645.2	859	770.6	818.4	734.2	715.5	619
1994	848.4	903.9	827.6	777.5	660.2	691	718.2	857.9	925.9	1135	1310	1288
1995	1225	917.5	747.1	755.6	688.5	622.1	778.2	977.5	1254	1251	1310	1217
1996	1286	1310	1289	1310	1310	1098	1255	1293	1235	1078	964.5	947.1
1997	904.3	830.9	772.7	683.7	628.6	573.3	583.3	685.2	727	983.4	1281	1574
1998	1107	1058	962.1	1108	1419	1335	1354	1362	1395	1535	1380	1329
1999	1311	1353	1076	877.9	853.3	909.5	863.8	929.9	686.2	695.1	835.2	681.9
2000	605.4	597	469.5	463.8	379.4	364.2	492.8	521.7	508.4	485.5	479.5	436
2001	434.9	442.2	504.5	463.2	382.7	407	441.5	363.7	609.2	1523	1022	737.1
2002	732.6	641.1	667.1	485.9	362.5	517.3	1068	1490	1500	1439	1175	1237
2003	1432	1426	1444	1443	1118	1279	1441	1398	1398	1451	1452	1265
2004	1161	1270	1437	1120	798.5	882.3	962	980.9	1276	1148	1373	1413
2005	1439	1396	1311	1325	1301	1378	1388	1271	1432	1445	1452	1451
2006	1381	1439	1204	955.8	734	579.6	811.5	747.4	778.2	ND	ND	ND
Mean of Monthly Streamflow (cfs)	1050	1090	1070	1040	951	949	1030	1080	1110	1100	1070	1050

Notes:

ND = no data available for the given time period
cfs = cubic foot per second

Source: Reference 2.4.1-214

LNP COL 2.4-1

Table 2.4.1-203 (Sheet 1 of 2)
Yearly Maximum Daily Streamflow Measurements for Withlacoochee River Bypass Channel near Dunnellon, Florida

Levy County
USGS Station Identification #: 02313250
Hydrologic Unit Code: 03100208
Latitude: 29°01'15"
Longitude: -82°38'17"

Year	Date	Streamflow (cfs)
1971	Sep. 09, 1971	1550
1972	Apr. 09, 1972	1690
1973	Sep. 02, 1973	1740
1974	Oct. 06, 1973	1660
1975	Oct. 01, 1974	1550
1976	Jun. 30, 1976	1620
1977	Jan. 04, 1977	1630
1978	Aug. 02, 1978	1600
1979	May 14, 1979	1660
1980	Jan. 27, 1980	1630
1981	Nov. 25, 1980	1610
1982	Jun. 19, 1982	1800
1983	Feb. 19, 1983	1610
1984	Feb. 22, 1984	1760
1985	Aug. 16, 1985	1540
1986	Jan. 22, 1986	1670
1987	Mar. 09, 1987	1580
1988	Oct. 01, 1987	1840
1989	Feb. 01, 1989	1640

LNP COL 2.4-1

Table 2.4.1-203 (Sheet 2 of 2)
Yearly Maximum Daily Streamflow Measurements for Withlacoochee River Bypass Channel near Dunnellon, Florida

Year	Date	Streamflow (cfs)
1990	Jul. 15, 1990	1540
1991	Sep. 10, 1991	1630
1992	Oct. 06, 1991	1160
1994	Jan. 31, 1994	1310
1995	Oct. 27, 1994	1400
1996	Oct. 07, 1995	1310
1997	Sep. 30, 1997	1310
1998	Dec. 25, 1997	1820
1999	Oct. 03, 1998	1580
2000	Oct. 22, 1999	1260
2001	Sep. 29, 2001	1500
2002	Oct. 08, 2001	1540
2003	Oct. 10, 2002	1470
2004	Oct. 13, 2003	1480
2005	Jul. 19, 2005	1590

Notes:

cfs = cubic foot per second

Source: Reference 2.4.1-215

LNP COL 2.4-1

Table 2.4.1-204 (Sheet 1 of 2)
Monthly Average Streamflow Measurements for Rainbow Springs near Dunnellon, Florida

Marion County
USGS Station Identification #: 02313100
Hydrologic Unit Code: 03100208
Latitude: 29°06'08"
Longitude: -82°26'16"

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1965	895.6	857.1	865.4	834.2	831.9	847	878.9	992.9	1039	1023	953.4	907
1966	895.1	824.7	846.8	845.3	840	842.7	865.2	881.5	910.4	938.1	915.1	868.7
1967	823.9	807.5	801.6	766.1	719.7	695.5	695.8	731.5	786.4	780.1	752.5	722.9
1968	692.8	667.6	649.9	628.5	611.9	618.8	692.3	777	844.6	850.5	856.6	823.8
1969	784.9	758.5	759.4	776.5	756.4	748.9	732.7	738.3	808.3	864.4	817.6	822.7
1970	842.5	914.7	945.2	940.6	924.6	914	879	911.9	986.4	933	849.2	793.9
1971	748.5	725.5	701.5	680.8	683.8	681.7	680.1	723.9	788.3	781.1	766.1	731.2
1972	705.5	689.7	672.2	677.3	665.5	652	661.4	666.4	724.9	733.1	702.8	676.4
1973	649.8	661.4	672.6	701.6	706.8	677.6	671.2	715.9	739.6	738.9	728.2	693.3
1974	650.4	624	601.7	590.1	580.5	595.4	648.8	688.2	730	733.6	697.4	666.7
1975	636.2	613.7	603.5	608.5	589	551.6	562.5	579.3	588.9	624.8	640	636.6
1976	625.8	609.2	595.4	575.2	552.4	635.9	676.3	689.1	681	675.9	661.8	645.2
1977	667.5	679.6	662.1	638.3	611.1	589.4	571.1	561.2	567.8	561.2	562	563.3
1978	579.2	642.2	801.8	814.7	753.6	715	698.7	730	746.6	719.5	689.1	662.5
1979	642.9	637.9	632.2	625.4	634.1	651	644.2	641.2	685.2	845	813.9	756.2
1980	712.2	694.8	670.1	668.3	659.6	654.8	724.8	731.5	730.1	712.4	695.2	677.6
1981	655.3	636.2	617.9	600.5	583.1	567.6	559.5	561.6	575.3	590	592	581.4
1982	573.7	571.1	589	638	685.1	714	852.2	871.8	918.8	987.1	923.5	853.6
1983	795.9	766.4	780.2	822.5	837.4	793.3	781.2	799.4	830.4	883.2	863.7	839.7
1984	837.1	835.9	829.5	840.2	842.4	825.5	838.5	863.8	854.9	821.7	775.4	732.6
1985	705.9	681.3	656.1	633	606.3	590.6	615.1	676.5	875	878.3	838.6	793.3

LNP COL 2.4-1

Table 2.4.1-204 (Sheet 2 of 2)
Monthly Average Streamflow Measurements for Rainbow Springs near Dunnellon, Florida

Year	Monthly Mean Streamflow, in cfs											
	January	February	March	April	May	June	July	August	September	October	November	December
1986	779	780	769.4	770.3	743.5	721.2	702.9	698.5	733	755.6	737.4	720.1
1987	712.1	707.2	737.3	850.7	854.6	803.9	779.8	758.1	742.1	735.3	723.6	707.5
1988	693.2	694	725.3	771.4	747.3	720.4	704.8	715.4	948.1	944.3	849.7	802
1989	756.1	718.6	694.2	666.2	641.9	628.3	643.5	639.3	633.5	640.5	639.8	626.4
1990	611.3	600.6	585.2	576.1	552.9	546.1	570.2	610.1	624.5	603.2	586.2	565.5
1991	541.9	525.1	532.5	583.5	629.6	659.5	658.1	675.8	676.8	658.3	643	618.1
1992	593.2	574	561.8	570.4	557.6	534.4	554.3	573.5	616.2	727.6	739.3	701
1993	663.5	630.7	629	643.6	644	627.2	613.4	615.7	623.5	611.7	616.5	620.2
1994	618.1	662.9	697.9	679.5	650.3	630.6	621.5	633.7	639.9	687.6	703.9	691.6
1995	677.7	672.6	650.7	636.9	613.7	607.5	610.6	642.5	678	680.5	685.2	671.2
1996	690.9	682.8	670	703.3	698	686.2	711.5	755.8	773.5	754.7	724.1	707.5
1997	686.5	665.8	641.5	633	627.5	627.6	637.5	663.1	685.1	705.6	788.3	900.5
1998	934	924	1016	956.7	884.9	844.9	790.4	777	801.5	885.2	884.5	843.9
1999	789.5	728.2	674.5	649.4	621.5	611.2	611.4	619.7	620.4	602.6	592.5	583
2000	569.2	556.8	540.9	526.6	503.6	499	524.3	536.7	549.5	525.4	536.7	532.2
2001	519.8	514.2	514.2	513.7	491	479.7	486.8	547.1	593.9	640.9	623.6	598.9
2002	578.6	560.3	541.5	519.2	493.5	485.7	529	569	581.8	589.8	568.3	564.2
2003	592.1	594.7	631.6	659.8	633.8	645.5	776.8	794.4	780.4	ND	ND	ND
2005	ND	ND	ND	ND	ND	ND	ND	ND	ND	734.2	705.3	686.2
2006	660.2	678.6	662.4	629.7	600.4	589.2	587.6	580.6	583.4	ND	ND	ND
Mean of Monthly Streamflow (cfs)	695	684	686	686	672	663	676	698	732	748	729	707

Notes:

ND = no data available for the given time period
cfs = cubic foot per second

Source: Reference 2.4.1-217

LNP COL 2.4-1

Table 2.4.1-205 (Sheet 1 of 2)
Yearly Maximum Daily Streamflow Measurements for Rainbow Springs near Dunnellon, Florida

Marion County
USGS Station Identification #: 02313100
Hydrologic Unit Code: 03100208
Latitude: 29°06'08"
Longitude: -82°26'16

Year	Date	Streamflow (cfs)
1966	Oct. 01, 1965	1040
1967	Oct. 17, 1966	945
1968	Sep. 19, 1968	856
1969	Oct. 29, 1968	867
1970	Sep. 11, 1970	993
1971	Oct. 01, 1970	978
1972	Oct. 19, 1971	785
1973	Oct. 08, 1972	744
1974	Sep. 25, 1974	755
1975	Oct. 01, 1974	750
1976	Aug. 18, 1976	692
1977	Feb. 05, 1977	684
1978	Mar. 26, 1978	852
1980	Oct. 16, 1979	859
1981	Oct. 01, 1980	724
1982	Sep. 30, 1982	987
1983	Oct. 07, 1982	1000
1984	Oct. 17, 1983	891
1985	Sep. 20, 1985	915
1986	Oct. 01, 1985	904
1987	Apr. 25, 1987	879
1988	Sep. 19, 1988	1060
1989	Oct. 01, 1988	1010

LNP COL 2.4-1

Table 2.4.1-205 (Sheet 2 of 2)
Yearly Maximum Daily Streamflow Measurements for Rainbow Springs near Dunnellon, Florida

Year	Date	Streamflow (cfs)
1990	Oct. 21, 1989	643
1991	Aug. 30, 1991	693
1992	Oct. 02, 1991	667
1993	Oct. 25, 1992	756
1994	Mar. 04, 1994	707
1995	Nov. 21, 1994	707
1996	Aug. 30, 1996	783
1997	Oct. 07, 1996	774
1998	Mar. 21, 1998	1030
1999	Nov. 05, 1998	892
2000	Oct. 22, 1999	608
2001	Sep. 27, 2001	626
2002	Oct. 23, 2001	650
2003	Aug. 23, 2003	803
2004	Sep. 30, 2004	921
2005	Oct. 20, 2004	835

Notes:

cfs = cubic foot per second

Source: Reference 2.4.1-218

LNP COL 2.4-1

Table 2.4.1-206 (Sheet 1 of 2)
Monthly Average Streamflow Measurements for Withlacoochee River at Inglis Dam near Dunnellon, Florida

Levy County
USGS Station Identification #: 02313230
Hydrologic Unit Code: 03100208
Latitude: 29°00'35"
Longitude: -82°37'01"
Drainage Area: 2020 mi.^{2}

	Monthly Mean Streamflow, in cfs											
Year	January	February	March	April	May	June	July	August	September	October	November	December
1969	ND	ND	ND	ND	ND	ND	ND	ND	ND	2655	2573	2035
1970	2445	3207	2497	1670	377.4	191.7	150	719.4	463.3	233.2	180	180
1971	173.9	238.2	182.6	125.7	80	124.3	145.5	396.5	1011	593.2	180	136.2
1972	75	75	75	99.6	75	190.2	75	91.6	882.9	951.8	897.4	695.8
1973	272.9	241.1	219.7	140.1	70	70	70	118.8	363.1	274.2	70	70
1974	70	70	70	70	70	105.6	796.3	1995	1180	354.9	81.4	70
1975	70	70	70	70	70	70	70.8	73	72.3	74.2	104.8	73.1
1976	73.9	73.3	71.9	70	119.2	103.7	204.7	369.6	330.3	168.1	70	71.2
1977	72.2	72.1	73.9	72.8	75.8	74	72.3	72.6	74.4	71.2	72.6	72.7
1978	108.2	203.1	1199	206.2	89.8	71.9	72.4	413.3	228.7	75.9	72	72.4
1979	77.4	82.3	70	72	140.2	79.3	70.2	71.3	645.1	3175	571	108.2
1980	94.5	73.1	71.6	106.9	77.2	175.1	96.3	85.4	90.1	84.4	86.2	86.5
1981	83.4	80.6	84.9	81.4	74.2	71	70.8	70.8	71	71.4	71.3	71.6
1982	71.2	71.4	190.3	191	76.3	696.5	2058	1783	2675	2908	1440	897.7
1983	922.8	1116	1647	1933	908.5	245.3	604.3	1062	980.3	912.5	491.6	561.4
1984	1147	1006	829.9	664.9	652.4	328.4	576.1	676.3	468.1	286.4	77.4	77.2
1985	84.1	82.9	84.2	106.8	84.1	91.7	105.9	239.7	2426	1225	344.8	84.6
1986	494.3	772.1	461.8	104.9	119.9	122.4	202	112.8	108.2	107.6	96	82.5
1987	107.8	70	275.9	2173	1125	214.3	95.1	119.4	129	294.4	741.6	129.8
1988	187.5	252.8	1095	287.8	119.6	104.1	112.9	133	2179	1578	311.4	524.2
1989	276.3	172.4	171	82.1	96.3	122.5	126.8	128.7	114.7	121	112.9	438.4

LNP COL 2.4-1

Table 2.4.1-206 (Sheet 2 of 2)
Monthly Average Streamflow Measurements for Withlacoochee River at Inglis Dam near Dunnellon, Florida

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1990	601.9	466.2	70	96.2	96.3	104.5	142.5	117.5	111.5	101.7	92.1	84.1
1991	84.6	85.9	70	70	70	92.3	676.6	1037	237.4	115.5	90.9	118.1
1992	194.5	100.5	101	84.7	70	83.9	93.9	146.1	266.7	759.2	167.6	70
1993	117.3	175.1	93.2	88.5	109.6	133.8	91.2	85.8	91.6	290	290	290
1994	319.5	477.3	283.9	97.6	80.5	152.2	90.7	452.1	951.2	1556	911.6	550.6
1995	606.4	669.9	652.5	556.5	458.1	522.8	456.4	596.8	1363	2359	1645	427.5
1996	907	694.4	590.3	1154	741.5	303.1	237.2	502.6	153	166.1	127	168.2
1997	90	77.3	93.8	100.7	110	94.8	131.4	196.3	118.8	107.2	197.5	848.3
1998	4417	4390	5067	3353	648.3	102.1	122.7	200.3	467.9	457.4	452.1	230.4
1999	251.4	285.3	145.3	185.2	77.1	148	163.7	162.9	252.8	208.4	100.4	142.7
2000	161.8	110.2	84.2	106.6	95.1	221.8	162.1	93.6	130.5	70	79.2	74
2001	77.1	70	70	70	70	70	196.6	352.5	458.5	359.4	75.4	70
2002	90.8	70	70	70	70	70	100.6	369.5	747.6	594.1	85.3	293.4
2003	1534	904.8	1203	1009	347.2	796.8	2030	3066	2722	1082	437.1	70
2004	70	111.9	312	70	70	70	73.4	72.3	2136	4925	2422	857.2
2005	491.5	175.9	162.7	313.1	ND	ND	ND	ND	ND	ND	ND	ND
Mean of Monthly Streamflow (cfs)	470	469	514	438	218	178	301	462	706	816	439	301

Notes:

ND = no data available for the given time period
cfs = cubic foot per second

Source: Reference 2.4.1-226

LNP COL 2.4-1

Table 2.4.1-207 (Sheet 1 of 2) Yearly Maximum Daily Streamflow Measurements for Withlacoochee River at Inglis Dam near Dunnellon, Florida

Levy County
USGS Station Identification #: 02313230
Hydrologic Unit Code: 03100208
Latitude: 29°00'35"
Longitude: -82°37'01"
Drainage Area: 2020 mi.^{2}

Year	Date	Streamflow (cfs)
1970	Jan. 21, 1970	3600
1971	Sep. 17, 1971	1450
1972	Sep. 08, 1972	2540
1973	Oct. 08, 1972	2440
1974	Aug. 07, 1974	2560
1975	Oct. 01, 1974	685
1976	May 24, 1976	890
1977	Oct. 01, 1976	370
1978	Mar. 09, 1978	2220
1979	Sep. 30, 1979	1940
1980	Oct. 12, 1979	4500
1981	Nov. 25, 1980	170
1982	Sep. 23, 1982	4280
1983	Oct. 08, 1982	3820
1984	Mar. 29, 1984	2180
1985	Sep. 01, 1985	3560
1986	Nov. 01, 1985	2540
1987	Apr. 21, 1987	3680
1988	Sep. 06, 1988	4370
1989	Oct. 01, 1988	2790
1990	Jan. 10, 1990	900
1991	Jul. 28, 1991	1620
1992	Sep. 05, 1992	1010
1993	Oct. 03, 1992	3020

LNP COL 2.4-1

Table 2.4.1-207 (Sheet 2 of 2)
Yearly Maximum Daily Streamflow Measurements for Withlacoochee River at Inglis Dam near Dunnellon, Florida

Year	Date	Streamflow (cfs)
1994	Sep. 21, 1994	1480
1995	Oct. 12, 1994	1910
1996	Oct. 05, 1995	2790
1997	Oct. 08, 1996	1240
1998	Mar. 20, 1998	6000
1999	Oct. 01, 1998	2319
2000	Sep. 18, 2000	948
2001	Sep. 16, 2001	1300
2002	Sep. 26, 2002	1690
2003	Aug. 25, 2003	3610
2004	Sep. 27, 2004	4600
2005	Oct. 19, 2004	6030

Notes:

cfs = cubic foot per second

Source: Reference 2.4.1-227

LNP COL 2.4-1

Table 2.4.1-208 (Sheet 1 of 2)
USGS County Water Use Data – Florida 2000

		All Counties within 10 Miles of LNP Site			Additional Counties within 25 Miles of LNP Site	Additional Counties within 50 Miles of LNP Site						
	Units	**Citrus**	**Levy**	**Marion**	**Sumter**	**Alachua**	**Dixie**	**Gilchrist**	**Hernando**	**Lake**	**Pasco**	**Putnam**
Federal Information Processing Standards (FIPS)		12017	12075	12083	12119	12001	12029	12041	12053	12069	12101	12107
State		FL	FL	FL	FL	FL	FL	FL	FL	FL	FL	FL
State FIPS Code		12	12	12	12	12	12	12	12	12	12	12
County FIPS Code		17	75	83	119	1	29	41	53	69	101	107
Year		2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000
Total Population of County	thousands	118.09	34.75	258.92	53.35	217.96	13.83	14.44	130.8	210.8	344.77	70.42
		Public Supply			**Public Supply**	**Public Supply**						
Total Population Served	thousands	66.23	11.07	136.84	28.24	179.12	4.62	1.85	116.03	171.14	275.8	23.31
Groundwater Withdrawals, Fresh Coded	mgd	13.97	2.16	27.99	4.44	28.26	0.67	0.27	20.26	39.92	102.67	3.2
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0.01	0	0	0
Total Withdrawals, Fresh	mgd	13.97	2.16	27.99	4.44	28.26	0.67	0.27	20.27	39.92	102.67	3.2
		Domestic Water Use			**Domestic Water Use**	**Domestic Water Use**						
Self-Supplied Population	thousands	51.86	23.38	122.08	25.11	38.84	9.21	12.59	14.77	39.39	68.97	47.11
Groundwater Withdrawals, Fresh Coded	mgd	7.2	3.95	16.42	4.7	4.12	0.98	1.33	1.41	4.27	4.5	4.99
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0	0	0	0
Total Withdrawals, Fresh	mgd	7.2	3.95	16.42	4.57	4.12	0.98	1.33	1.41	4.27	4.5	4.99
		Industrial Water Use			**Industrial Water Use**	**Industrial Water Use**						
Groundwater Withdrawals, Fresh Coded	mgd	0.14	0.01	1.1	0.26	0.45	0.02	0	6.01	3.69	3.72	16.79
Total Withdrawals, Groundwater	mgd	0.14	0.01	1.1	0.26	0.45	0.02	0	6.01	3.69	3.72	16.79
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0	0	0.27	30.28
Total Withdrawals, Surface Water	mgd	0	0	0	0	0	0	0	0	0	0.27	30.28
Total Withdrawals, Fresh	mgd	0.14	0.01	1.1	0.26	0.45	0.02	0	6.01	3.69	3.99	47.07
Total Withdrawals	mgd	0.14	0.01	1.1	0.26	0.45	0.02	0	6.01	3.69	3.99	47.07
		Irrigation			**Irrigation**	**Irrigation**						
Irrigation, Acres Irrigated, Sprinkler	thousands	2.95	14.37	13.26	3.68	15.28	0.38	6.74	3.12	9.95	9.53	3.15
Irrigation, Acres Irrigated, Micro Irrigation	thousands	0.25	0.07	1.39	0.2	0.38	0	0	1.12	17.38	9.55	0.4
Irrigation, Acres Irrigated, Surface (flood)	thousands	0	0.2	0	0	0	0	0	0	0.51	0.77	5.5
Irrigation, Acres Irrigated, Total	thousands	3.2	14.64	14.65	3.88	15.66	0.38	6.74	4.24	27.84	19.85	9.05
Irrigation, Groundwater Withdrawals, Fresh	mgd	6.31	21.16	20.74	15.29	21.48	1.55	11.99	7.41	36.21	26.76	12.33
Irrigation, Surface water Withdrawals, Fresh	mgd	0.97	0.61	2.09	0.64	0.54	0.03	0.21	0.91	9.17	1.42	3.9
Irrigation, Total Withdrawals, Fresh	mgd	7.28	21.77	22.83	15.93	22.02	1.58	12.2	8.32	45.38	28.18	16.23
		Livestock Water Use			**Livestock Water Use**	**Livestock Water Use**						
Groundwater Withdrawals, Fresh Coded	mgd	0.2	1.11	0.45	2.14	0.59	0.04	1.98	0.68	0	0.89	0
Surface Water Withdrawals, Fresh Coded	mgd	0.04	0.06	0.02	0.07	0.03	0	0.11	0	0	0.1	0
Total Withdrawals, Fresh	mgd	0.24	1.17	0.47	2.21	0.62	0.04	2.09	0.68	0	0.8	0

LNP COL 2.4-1

Table 2.4.1-208 (Sheet 2 of 2)
USGS County Water Use Data – Florida 2000

		All Counties within 10 Miles of LNP Site			Additional Counties within 25 Miles of LNP Site	Additional Counties within 50 Miles of LNP Site						
	Units	Citrus	Levy	Marion	Sumter	Alachua	Dixie	Gilchrist	Hernando	Lake	Pasco	Putnam
		Mining			Mining	Mining						
Groundwater Withdrawals, Fresh Coded	mgd	0.62	0	0	0	0	0	0	13.69	5.65	0.11	2.26
Surface Water Withdrawals, Fresh Coded	mgd	0.29	1.96	0	16.98	0	0	0	0.07	0.6	0.54	0.84
Total Withdrawals, Fresh	mgd	0.91	1.96	0	16.98	0	0	0	13.76	6.25	0.65	3.1
		Thermoelectric Power Water Use			Thermoelectric Power Water Use	Thermoelectric Power Water Use						
Groundwater Withdrawals, Fresh Coded	mgd	1.55	0	0	0	2.63	0	0	0	0	0.14	0.69
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0	0	0	13.9
Surface Water Withdrawals, Saline	mgd	393.9	0	0	0	0	0	0	0	0	1956.5	0
Total Withdrawals, Surface Water	mgd	393.9	0	0	0	0	0	0	0	0	1956.5	13.9
Total Withdrawals, Fresh	mgd	1.55	0	0	0	2.63	0	0	0	0	0.14	0.69
Total Withdrawals	mgd	395.45	0	0	0	2.63	0	0	0	0	1956.64	14.59
		Thermoelectric Power Once-Through			Thermoelectric Power Once-Through	Thermoelectric Power Once-Through						
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0	0	0	0
Surface Water Withdrawals, Saline	mgd	291.62	0	0	0	0	0	0	0	0	1956.5	0
Total Withdrawals, Surface Water	mgd	291.62	0	0	0	0	0	0	0	0	1956.5	0
		Thermoelectric Power Closed-Loop			Thermoelectric Power Closed-Loop	Thermoelectric Power Closed-Loop						
Groundwater Withdrawals, Fresh Coded	mgd	1.55	0	0	0	2.63	0	0	0	0	0.14	0.69
Surface Water Withdrawals, Fresh Coded	mgd	0	0	0	0	0	0	0	0	0	0	13.9
Surface Water Withdrawals, Saline	mgd	102.28	0	0	0	0	0	0	0	0	0	0
Total Withdrawals, Fresh	mgd	1.55	0	0	0	2.63	0	0	0	0	0	13.9
Total Withdrawals	mgd	103.83	0	0	0	2.63	0	0	0	0	0.14	14.59
		Totals			Totals	Totals						
Total Groundwater Withdrawals, Fresh Coded	mgd	29.99	28.39	66.7	26.7	57.53	3.26	15.57	49.46	89.94	138.79	48.26
Total Withdrawals, Groundwater	mgd	29.99	28.39	66.7	26.7	57.53	3.26	15.57	49.46	89.94	138.97	40.26
Total Surface Water Withdrawals, Fresh Coded	mgd	1.3	2.63	2.11	17.69	0.57	0.03	0.32	0.99	9.77	2.24	48.92
Total Surface Water Withdrawals, Saline	mgd	393.9	0	0	0	0	0	0	0	0	1956.5	0
Total Withdrawals, Surface Water	mgd	395.2	2.63	2.11	17.69	0.57	0.03	0.32	0.99	9.77	1958.74	48.92
Total Withdrawals, Fresh	mgd	31.29	31.02	68.81	44.39	58.1	3.29	15.89	50.45	99.51	141.03	89.18
Total Withdrawals	mgd	425.19	31.02	68.81	44.39	58.1	3.29	15.89	50.45	99.51	2097.53	89.18

Notes:

mgd = million gallons per day

Source: Reference 2.4.1-231

LNP COL 2.4-2

Table 2.4.2-201
Probable Maximum Precipitation Estimate for a 2.6-Km^2 (1-Mi.^{2}) Area

Duration		Precipitation (inches)
Minutes	Hours	
5	0.08	6.28
15	0.25	9.81
30	0.50	14.32
60	1	19.61
360	6	37.21
720	12	45.24
1440	24	52.42

Notes:

km^2 = square kilometer
mi.^{2} = square mile

LNP COL 2.4-2

Table 2.4.3-201
Generalized Estimates of All-Season PMP Depths

Area (mi.^{2})	6-Hr. PMP (inches)	12-Hr. PMP (inches)	24-Hr. PMP (inches)	48-Hr. PMP (inches)	72-Hr. PMP (inches)
10	32	38.7	47.1	51.8	55.7
200	24.6	31.2	39.5	44.3	48.8
1000	18.2	24.9	33.2	37.7	41.3
5000	10.1	15	21.9	26.6	30.7
10,000	7.6	11.8	17.6	22.5	26.5
20,000	5.6	9.2	13.6	18	22

Notes:

mi.^{2} = square mile
hr. = hour

Source: Reference 2.4.3-202

LNP COL 2.4-2

Table 2.4.3-202
PMP Values (Inches) for the Selected Standard Areas at 28°40’48” N, 82°10’10’’W

Duration (hr.)	450 mi.^{2}	700 mi.^{2}	1000 mi.^{2}	1500 mi.^{2}	2150 mi.^{2}	3000 mi.^{2}	4500 mi.^{2}	6500 mi.^{2}	10,000 mi.^{2}
6	21.78	19.89	18.20	16.14	14.26	12.53	10.57	9.03	7.60
12	28.55	26.68	24.90	22.60	20.36	18.19	15.62	13.58	11.80
24	36.91	35.03	33.20	30.75	28.27	25.78	22.68	20.07	17.60
48	41.58	39.60	37.69	35.19	32.71	30.29	27.34	24.89	22.50
72	45.46	43.27	41.30	38.84	36.48	34.20	31.41	29.00	26.50

Notes:

mi.^{2} = square mile
hr. = hour

Source: Reference 2.4.3-202

LNP COL 2.4-2

Table 2.4.3-203
Interpolated PMP Values for 18-Hour Duration

Area ($mi.^2$)	PMP Depth (inch)
450	32.7
700	30.9
1000	29.0
1500	26.7
2150	24.3
3000	22.0
4500	19.2
6500	16.8
10,000	14.7

Notes:

$mi.^2$ = square mile

Source: Reference 2.4.3-202

LNP COL 2.4-2

Table 2.4.3-204
Incremental Differences for the First Three 6-Hour Periods

	Incremental Difference (inch)		
Area (mi.^{2})	**1st 6-Hr. Period**	**2nd 6-Hr. Period**	**3rd 6-Hr. Period**
450	21.8	6.8	4.2
700	19.9	6.8	4.2
1000	18.2	6.7	4.1
1500	16.1	6.5	4.1
2150	14.3	6.1	4.0
3000	12.5	5.7	3.8
4500	10.6	5.1	3.5
6500	9.0	4.6	3.2
10,000	7.6	4.2	2.9

Notes:

mi.^{2} = square mile
hr. = hour

Source: Reference 2.4.3-202

LNP COL 2.4-2

Table 2.4.3-205
Incremental Differences for the First Three 6-Hour Periods Based on Smooth Curves

	Incremental Difference (inch)		
Area ($mi.^2$)	1st 6-Hr. period	2nd 6-Hr. period	3rd 6-Hr. period
450	21.8	7.8	4.9
700	19.7	7.3	4.6
1000	18.0	6.9	4.4
1500	16.1	6.4	4.2
2150	14.4	6.0	4.0
3000	12.8	5.6	3.8
4500	10.9	5.1	3.5
6500	9.1	4.7	3.3
10,000	7.1	4.2	3.1

Notes:

hr. = hour
$mi.^2$ = square mile

LNP COL 2.4-2

Table 2.4.3-206
Incremental Differences for the First Three 6-Hour Periods Adjusted for Orientation

	Incremental Difference (inch)		
Area ($mi.^2$)	1st 6-Hr. period	2nd 6-Hr. period	3rd 6-Hr. period
450	21.7	7.8	4.9
700	19.4	7.2	4.6
1000	17.6	6.7	4.3
1500	15.5	6.2	4.0
2150	13.5	5.6	3.7
3000	11.6	5.0	3.4
4500	9.9	4.6	3.2
6500	8.3	4.2	3.0
10,000	6.4	3.8	2.8

Notes:

hr. = hour
$mi.^2$ = square mile

LNP COL 2.4-2

Table 2.4.3-207 (Sheet 1 of 4)
Computation Sheet for First 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
450	A	132	28.68	28.68	10	287	2150	A	176	23.82	23.82	10	238
	B	124	26.95	27.82	15	417		B	165	22.33	23.08	15	346
	C	116	25.21	26.08	25	652		C	154	20.84	21.59	25	540
	D	108	23.47	24.34	50	1217		D	142	19.22	20.03	50	1002
	E	101	21.95	22.71	75	1703		E	131	17.73	18.48	75	1386
	F	93	20.21	21.08	125	2635		F	122	16.51	17.12	125	2140
	G	86	18.69	19.45	150	2917		G	113	15.29	15.90	150	2386
	H	63	13.69	16.19	250	4047		H	103	13.94	14.62	250	3655
	I	50	10.87	12.28	300	3683		I	95	12.86	13.40	300	4020
	J	38	8.26	9.56	400	3825		J	86	11.64	12.25	400	4900
	K	30	6.52	7.39	420	3103		K	77	10.42	11.03	420	4633
0.7*	L	23	5.00	6.06	150	909	0.7*	L	52	7.04	9.41	150	1411
0.8*	M	15	3.26	4.65	50	233	0.8*	M	33	4.47	6.52	50	326
					Sum=	25,628					Sum =	2020	26,982

LNP COL 2.4-2

Table 2.4.3-207 (Sheet 2 of 4)
Computation Sheet for First 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
700	A	140	27.22	27.22	10	272	3000	A	191	22.15	22.15	10	222
	B	132	25.66	26.44	15	397		B	179	20.76	21.45	15	322
	C	124	24.11	24.89	25	622		C	166	19.25	20.00	25	500
	D	115	22.36	23.23	50	1162		D	154	17.86	18.56	50	928
	E	107	20.80	21.58	75	1619		E	142	16.47	17.16	75	1287
	F	98	19.05	19.93	125	2491		F	132	15.31	15.89	125	1986
	G	92	17.89	18.47	150	2770		G	122	14.15	14.73	150	2209
	H	84	16.33	17.11	250	4277		H	112	12.99	13.57	250	3392
	I	63	12.25	14.29	300	4287		I	102	11.83	12.41	300	3723
	J	48	9.33	10.79	400	4316		J	92	10.67	11.25	400	4500
	K	36	7.00	8.17	420	3430		K	83	9.63	10.15	420	4262
0.7*	L	27	5.25	6.47	150	971	0.7*	L	74	8.58	9.31	150	1397
0.8*	M	18	3.50	4.90	50	245	0.8*	M	44	5.10	7.89	50	394
					Sum=	26,858						Sum=	25,121

LNP COL 2.4-2

Table 2.4.3-207 (Sheet 3 of 4)
Computation Sheet for First 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1000	A	149	26.29	26.29	10	263	4500	A	212	20.88	20.88	10	209
	B	140	24.70	25.50	15	382		B	198	19.51	20.20	15	303
	C	131	23.11	23.91	25	598		C	184	18.13	18.82	25	470
	D	122	21.53	22.32	50	1116		D	170	16.75	17.44	50	872
	E	113	19.94	20.73	75	1555		E	157	15.47	16.11	75	1208
	F	104	18.35	19.14	125	2393		F	146	14.38	14.92	125	1866
	G	97	17.11	17.73	150	2660		G	135	13.30	13.84	150	2076
	H	89	15.70	16.41	250	4102		H	124	12.22	12.76	250	3189
	I	82	14.47	15.09	300	4526		I	113	11.13	11.67	300	3502
	J	60	10.59	12.53	400	5011		J	103	10.15	10.64	400	4256
	K	44	7.76	9.17	420	3853		K	93	9.16	9.65	420	4055
0.7*	L	32	5.65	7.13	150	1069	0.7*	L	83	8.18	8.87	150	1330
0.8*	M	21	3.71	5.26	50	263	0.8*	M	71	6.99	7.94	50	397
					Sum =	27,791						Sum =	23,733

LNP COL 2.4-2

Table 2.4.3-207 (Sheet 4 of 4)
Computation Sheet for First 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	162	25.19	25.19	10	252	6500	A	233	19.26	19.26	10	193
	B	152	23.63	24.41	15	366		B	218	18.02	18.64	15	280
	C	142	22.08	22.86	25	571		C	203	16.78	17.40	25	435
	D	132	20.52	21.30	50	1065		D	187	15.46	16.12	50	806
	E	122	18.97	19.75	75	1481		E	174	14.39	14.92	75	1119
	F	112	17.41	18.19	125	2274		F	160	13.23	13.81	125	1726
	G	105	16.33	16.87	150	2530		G	148	12.24	12.73	150	1910
	H	96	14.93	15.63	250	3906		H	137	11.33	11.78	250	2946
	I	88	13.68	14.30	300	4291		I	125	10.34	10.83	300	3249
	J	80	12.44	13.06	400	5224		J	113	9.34	9.84	400	3936
	K	56	8.71	10.57	420	4441		K	103	8.52	8.93	420	3750
0.7*	L	41	6.37	8.01	150	1201	0.7*	L	93	7.69	8.27	150	1240
0.8*	M	26	4.04	5.91	50	295	0.8*	M	81	6.70	7.49	50	375
					Sum=	27,899					Sum=	2020	21,965

Notes:

mi.^{2} = square mile
* = Weighting factor (Reference 2.4.2-208)

LNP COL 2.4-2

Table 2.4.3-208 (Sheet 1 of 4)
Computation Sheet for Second 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
450	A	113	8.79	8.79	10	88	2150	A	118.5	6.65	6.65	10	67
	B	109	8.48	8.64	15	130		B	114.5	6.43	6.54	15	98
	C	105	8.17	8.33	25	208		C	111	6.23	6.33	25	158
	D	102	7.94	8.05	50	403		D	108.5	6.09	6.16	50	308
	E	99.5	7.74	7.84	75	588		E	106.5	5.98	6.03	75	452
	F	97	7.55	7.65	125	956		F	104.5	5.86	5.92	125	740
	G	95	7.39	7.47	150	1121		G	102	5.72	5.79	150	869
	H	77.5	6.03	6.71	250	1678		H	100	5.61	5.67	250	1417
	I	66	5.14	5.58	300	1675		I	99	5.56	5.58	300	1675
	J	54.5	4.24	4.69	400	1875		J	97	5.44	5.50	400	2200
	K	44.5	3.46	3.85	420	1618		K	96	5.39	5.42	420	2275
0.7*	L	36.5	2.84	3.28	150	491	0.7*	L	73	4.10	5.00	150	750
0.8*	M	25.5	1.98	2.67	50	133	0.8*	M	54	3.03	3.88	50	194
				Sum=	2020	10,963					Sum=	2020	11,204

LNP COL 2.4-2

Table 2.4.3-208 (Sheet 2 of 4)
Computation Sheet for Second 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
700	A	114.5	8.23	8.23	10	82	3000	A	119.5	6.03	6.03	10	60
	B	110	7.91	8.07	15	121		B	116	5.85	5.94	15	89
	C	107	7.69	7.80	25	195		C	112.5	5.68	5.77	25	144
	D	104	7.48	7.58	50	379		D	110	5.55	5.61	50	281
	E	101	7.26	7.37	75	553		E	108	5.45	5.50	75	413
	F	99	7.12	7.19	125	899		F	106	5.35	5.40	125	675
	G	97	6.97	7.04	150	1057		G	104	5.25	5.30	150	795
	H	95	6.83	6.90	250	1725		H	102	5.15	5.20	250	1300
	I	78	5.61	6.22	300	1865		I	100.5	5.07	5.11	300	1533
	J	65.5	4.71	5.16	400	2063		J	99	5.00	5.03	400	2014
	K	54	3.88	4.30	420	1804		K	97	4.90	4.95	420	2077
0.7*	L	44	3.16	3.67	150	550	0.7*	L	96	4.84	4.88	150	732
0.8*	M	32	2.30	2.99	50	150	0.8*	M	67	3.38	4.55	50	228
				Sum =	2020	11,442					Sum =	2020	10,340

LNP COL 2.4-2

Table 2.4.3-208 (Sheet 3 of 4)
Computation Sheet for Second 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1000	A	116	7.80	7.80	10	78	4500	A	121	5.58	5.58	10	56
	B	112	7.53	7.67	15	115		B	117	5.40	5.49	15	82
	C	108.5	7.30	7.41	25	185		C	114	5.26	5.33	25	133
	D	105	7.06	7.18	50	359		D	112	5.17	5.21	50	261
	E	103	6.93	6.99	75	525		E	109.5	5.05	5.11	75	383
	F	101	6.79	6.86	125	857		F	108	4.98	5.02	125	627
	G	99	6.66	6.72	150	1009		G	105.5	4.87	4.92	150	739
	H	97	6.52	6.59	250	1648		H	103.5	4.77	4.82	250	1205
	I	95	6.39	6.46	300	1937		I	102	4.70	4.74	300	1422
	J	76	5.11	5.75	400	2300		J	100.5	4.64	4.67	400	1868
	K	63	4.24	4.67	420	1963		K	99	4.57	4.60	420	1932
0.7*	L	51	3.43	3.99	150	599	0.7*	L	97.5	4.50	4.55	150	682
0.8*	M	38	2.56	3.25	50	163	0.8*	M	96	4.43	4.48	50	224
				Sum =	2020	11,737					Sum =	2020	9614

LNP COL 2.4-2

Table 2.4.3-208 (Sheet 4 of 4)
Computation Sheet for Second 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	117	7.22	7.22	10	72	6500	A	122	5.15	5.15	10	51
	B	113	6.97	7.10	15	106		B	119	5.02	5.08	15	76
	C	110	6.79	6.88	25	172		C	115.5	4.87	4.95	25	124
	D	107	6.60	6.70	50	335		D	113	4.77	4.82	50	241
	E	105	6.48	6.54	75	491		E	111	4.68	4.73	75	354
	F	103	6.36	6.42	125	802		F	109	4.60	4.64	125	580
	G	100.5	6.20	6.28	150	942		G	107	4.51	4.56	150	683
	H	99	6.11	6.16	250	1539		H	105	4.43	4.47	250	1118
	I	97	5.99	6.05	300	1815		I	104	4.39	4.41	300	1323
	J	95.5	5.89	5.94	400	2376		J	102	4.30	4.35	400	1738
	K	75.5	4.66	5.28	420	2217		K	100.5	4.24	4.27	420	1794
0.7*	L	60.5	3.73	4.38	150	657	0.7*	L	99	4.18	4.22	150	633
0.8*	M	45	2.78	3.54	50	177	0.8*	M	97.5	4.11	4.16	50	208
				Sum =	2020	11,702					Sum =	2020	8925

Notes:

mi.^{2} = square mile
* = Weighting factor (Reference 2.4.2-208)

LNP COL 2.4-2

Table 2.4.3-209 (Sheet 1 of 4)
Computation Sheet for Third 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
450	A	103.8	5.04	5.04	10	50	2150	A	105.3	3.93	3.93	10	39
	B	102.4	4.97	5.01	15	75		B	104.2	3.89	3.91	15	59
	C	101.2	4.91	4.94	25	124		C	103.2	3.85	3.87	25	97
	D	100.3	4.87	4.89	50	245		D	102	3.81	3.83	50	192
	E	99.8	4.85	4.86	75	364		E	101.3	3.78	3.80	75	285
	F	99.5	4.83	4.84	125	605		F	101	3.77	3.78	125	472
	G	99.2	4.82	4.82	150	724		G	100.6	3.76	3.76	150	565
	H	84	4.08	4.45	250	1112		H	100.3	3.74	3.75	250	938
	I	71	3.45	3.76	300	1129		I	100	3.73	3.74	300	1122
	J	60	2.91	3.18	400	1272		J	99.7	3.72	3.73	400	1491
	K	50	2.43	2.67	420	1122		K	99.5	3.71	3.72	420	1562
0.7*	L	39.5	1.92	2.28	150	341	0.7*	L	80.5	3.01	3.50	150	525
0.8*	M	30	1.46	1.83	50	91	0.8*	M	61	2.28	2.86	50	143
					Sum=	7254						Sum=	7488

LNP COL 2.4-2

Table 2.4.3-209 (Sheet 2 of 4) Computation Sheet for Third 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
700	A	104.2	4.75	4.75	10	47	3000	A	105.7	3.61	3.61	10	36
	B	102.9	4.69	4.72	15	71		B	104.6	3.58	3.60	15	54
	C	101.7	4.63	4.66	25	116		C	103.5	3.54	3.56	25	89
	D	100.8	4.59	4.61	50	231		D	102.5	3.50	3.52	50	176
	E	100.2	4.56	4.58	75	343		E	101.7	3.48	3.49	75	262
	F	99.9	4.55	4.56	125	570		F	101.3	3.46	3.47	125	434
	G	99.6	4.54	4.54	150	681		G	100.9	3.45	3.46	150	519
	H	99.2	4.52	4.53	250	1132		H	100.5	3.44	3.44	250	861
	I	85	3.87	4.19	300	1258		I	100.2	3.43	3.43	300	1029
	J	70.5	3.21	3.54	400	1416		J	99.9	3.42	3.42	400	1368
	K	58.5	2.66	2.94	420	1234		K	99.6	3.41	3.41	420	1433
0.7*	L	47	2.14	2.51	150	376	0.7*	L	99.3	3.40	3.40	150	510
0.8*	M	37	1.69	2.05	50	102	0.8*	M	76	2.60	3.24	50	162
					Sum=	7579						Sum=	6933

LNP COL 2.4-2

Table 2.4.3-209 (Sheet 3 of 4)
Computation Sheet for Third 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1000	A	104.6	4.52	4.52	10	45	4500	A	106	3.40	3.40	10	34
	B	103.3	4.46	4.49	15	67		B	105	3.37	3.38	15	51
	C	102.3	4.42	4.44	25	111		C	104	3.33	3.35	25	84
	D	101.3	4.38	4.40	50	220		D	103.1	3.31	3.32	50	166
	E	100.6	4.35	4.36	75	327		E	102.1	3.27	3.29	75	247
	F	100.3	4.33	4.34	125	542		F	101.7	3.26	3.27	125	408
	G	99.9	4.32	4.32	150	649		G	101.2	3.24	3.25	150	488
	H	99.6	4.30	4.31	250	1077		H	100.9	3.23	3.24	250	810
	I	99.3	4.29	4.30	300	1289		I	100.6	3.23	3.23	300	969
	J	82.5	3.56	3.93	400	1571		J	100.2	3.21	3.22	400	1288
	K	67	2.89	3.23	420	1356		K	99.9	3.20	3.21	420	1347
0.7*	L	54	2.33	2.73	150	409	0.7*	L	99.6	3.19	3.20	150	480
0.8*	M	43	1.86	2.24	50	112	0.8*	M	99.3	3.18	3.19	50	160
				Sum=	2020	7774					Sum=	2020	6531

LNP COL 2.4-2

Table 2.4.3-209 (Sheet 4 of 4)
Computation Sheet for Third 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
1500	A	105	4.24	4.24	10	42	6500	A	106.4	3.21	3.21	10	32
	B	103.8	4.19	4.21	15	63		B	105.5	3.18	3.19	15	48
	C	102.7	4.14	4.17	25	104		C	104.5	3.15	3.16	25	79
	D	101.7	4.10	4.12	50	206		D	103.5	3.12	3.13	50	157
	E	101	4.07	4.09	75	307		E	102.5	3.09	3.10	75	233
	F	100.7	4.06	4.07	125	509		F	102	3.07	3.08	125	385
	G	100.3	4.05	4.05	150	608		G	101.5	3.06	3.07	150	460
	H	100	4.03	4.04	250	1010		H	101.2	3.05	3.05	250	763
	I	99.7	4.02	4.03	300	1209		I	100.9	3.04	3.04	300	913
	J	99.4	4.01	4.02	400	1607		J	100.5	3.03	3.03	400	1214
	K	81	3.27	3.64	420	1528		K	100.2	3.02	3.02	420	1270
0.7*	L	65.5	2.64	3.08	150	462	0.7*	L	99.8	3.01	3.02	150	452
0.8*	M	51.5	2.08	2.53	50	126	0.8*	M	99.5	3.00	3.00	50	150
				Sum=	2020	7782					Sum=	2020	6156

Notes:

mi.^{2} = square mile
* = Weighting factor (Reference 2.4.2-208)

LNP COL 2.4-2

**Table 2.4.3-210
Incremental Average Depths for Each 6-Hour Period
for a 1500-Mi.^{2} Drainage Area**

Increment	Duration (hr.)	Cumulative PMP (inches)	Incremental PMP (inches)
1	6	16.01	16.01
2	12	23.03	7.02
3	18	27.05	4.03
4	24	29.78	2.73
5	30	31.79	2.02
6	36	33.37	1.58
7	42	34.65	1.28
8	48	35.72	1.07
9	54	36.62	0.91
10	60	37.41	0.79
11	66	38.10	0.69
12	72	38.71	0.61

Notes:

mi.^{2} = square mile
hr. = hour

LNP COL 2.4-2

Table 2.4.3-211
72-Hour Drainage Isohyet Values

	6-Hr. Periods											
Isohyet	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
A	25.02	7.93	4.08	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
B	23.48	7.66	4.03	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
C	21.93	7.45	3.99	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
D	20.39	7.25	3.95	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
E	18.84	7.11	3.92	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
F	17.30	6.98	3.91	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
G	16.22	6.81	3.90	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
H	14.83	6.71	3.88	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
I	13.59	6.57	3.87	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
J	12.36	6.47	3.86	2.63	1.95	1.52	1.23	1.03	0.88	0.76	0.67	0.59
K	8.65	5.11	3.15	2.13	1.58	1.23	1.00	0.83	0.71	0.61	0.54	0.48
L	6.33	4.10	2.54	1.72	1.27	1.00	0.81	0.67	0.57	0.50	0.44	0.39
M	4.02	3.05	2.00	1.34	0.99	0.78	0.63	0.53	0.45	0.39	0.34	0.30

LNP COL 2.4-2

Table 2.4.3-212 (Sheet 1 of 3)
Computation of Drainage Average Depths (Increments 1 to 6)

	Increment #1							Increment #2					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	162	25.93	25.93	10	259.3	1500	A	117	8.21	8.21	10	82.1
	B	152	24.33	25.13	15	376.9		B	113	7.93	8.07	15	121.1
	C	142	22.73	23.53	25	588.2		C	110	7.72	7.83	25	195.7
	D	132	21.13	21.93	50	1096.4		D	107	7.51	7.62	50	380.8
	E	122	19.53	20.33	75	1524.5		E	105	7.37	7.44	75	558.1
	F	112	17.93	18.73	125	2340.8		F	103	7.23	7.30	125	912.6
	G	105	16.81	17.37	150	2604.9		G	100.5	7.06	7.14	150	1071.4
	H	96	15.37	16.09	250	4021.4		H	99	6.95	7.00	250	1750.6
	I	88	14.08	14.72	300	4417.5		I	97	6.81	6.88	300	2063.9
	J	80	12.80	13.44	400	5377.8		J	95.5	6.70	6.76	400	2702.7
	K	56	8.96	10.88	420	4571.2		K	75.5	5.30	6.00	420	2520.9
0.7	L	41	6.56	8.24	150	1236.4	0.7	L	60.5	4.25	4.98	150	747.6
0.8	M	26	4.16	6.08	50	304.1	0.8	M	45	3.16	4.03	50	201.5
				Sum=		28,719.3					Sum=		13,309.2
			Average Depth =			14.2				Average Depth =			6.6

LNP COL 2.4-2

Table 2.4.3-212 (Sheet 2 of 3)
Computation of Drainage Average Depths (Increments 1 to 6)

	Increment #3							Increment #4					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size ($mi.^2$)	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	105	4.23	4.23	10	42.3	1500	A	100	2.73	2.73	10	27.3
	B	103.8	4.18	4.20	15	63.0		B	100	2.73	2.73	15	40.9
	C	102.7	4.13	4.16	25	103.9		C	100	2.73	2.73	25	68.2
	D	101.7	4.09	4.11	50	205.7		D	100	2.73	2.73	50	136.3
	E	101	4.07	4.08	75	306.0		E	100	2.73	2.73	75	204.5
	F	100.7	4.05	4.06	125	507.4		F	100	2.73	2.73	125	340.9
	G	100.3	4.04	4.05	150	606.8		G	100	2.73	2.73	150	409.0
	H	100	4.03	4.03	250	1007.8		H	100	2.73	2.73	250	681.7
	I	99.7	4.01	4.02	300	1205.8		I	100	2.73	2.73	300	818.1
	J	99.4	4.00	4.01	400	1602.9		J	100	2.73	2.73	400	1090.8
	K	81	3.26	3.63	420	1524.9		K	81	2.21	2.47	420	1036.5
0.7	L	65.5	2.64	3.07	150	461.0	0.7	L	65.5	1.79	2.08	150	312.3
0.8	M	51.5	2.07	2.52	50	126.2	0.8	M	51.1	1.39	1.71	50	85.4
				Sum=		7763.7					Sum=		5251.8
				Average Depth =		3.8					Average Depth =		2.6

LNP COL 2.4-2

Table 2.4.3-212 (Sheet 3 of 3)
Computation of Drainage Average Depths (Increments 1 to 6)

	Increment #5							Increment #6					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	100	2.02	2.02	10	20.2	1500	A	100	1.58	1.58	10	15.8
	B	100	2.02	2.02	15	30.3		B	100	1.58	1.58	15	23.6
	C	100	2.02	2.02	25	50.4		C	100	1.58	1.58	25	39.4
	D	100	2.02	2.02	50	100.9		D	100	1.58	1.58	50	78.8
	E	100	2.02	2.02	75	151.3		E	100	1.58	1.58	75	118.2
	F	100	2.02	2.02	125	252.1		F	100	1.58	1.58	125	197.0
	G	100	2.02	2.02	150	302.6		G	100	1.58	1.58	150	236.4
	H	100	2.02	2.02	250	504.3		H	100	1.58	1.58	250	394.0
	I	100	2.02	2.02	300	605.1		I	100	1.58	1.58	300	472.8
	J	100	2.02	2.02	400	806.8		J	100	1.58	1.58	400	630.4
	K	81	1.63	1.83	420	766.7		K	81	1.28	1.43	420	599.1
0.7	L	65.5	1.32	1.54	150	231.0	0.7	L	65.5	1.03	1.20	150	180.5
0.8	M	51.1	1.03	1.26	50	63.2	0.8	M	51.1	0.81	0.99	50	49.3
				Sum=		3884.8					Sum=		3035.5
			Average Depth =			1.9				Average Depth =			1.5

Notes:

mi.^{2} = square mile

LNP COL 2.4-2

Table 2.4.3-213 (Sheet 1 of 3)
Computation of Drainage Average Depths (Increments 7 to 12)

	Increment #7							Increment #8					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
1500	A	100	1.28	1.28	10	12.8	1500	A	100	1.07	1.07	10	10.7
	B	100	1.28	1.28	15	19.2		B	100	1.07	1.07	15	16.0
	C	100	1.28	1.28	25	32.0		C	100	1.07	1.07	25	26.7
	D	100	1.28	1.28	50	63.9		D	100	1.07	1.07	50	53.3
	E	100	1.28	1.28	75	95.9		E	100	1.07	1.07	75	80.0
	F	100	1.28	1.28	125	159.8		F	100	1.07	1.07	125	133.3
	G	100	1.28	1.28	150	191.8		G	100	1.07	1.07	150	159.9
	H	100	1.28	1.28	250	319.7		H	100	1.07	1.07	250	266.6
	I	100	1.28	1.28	300	383.6		I	100	1.07	1.07	300	319.9
	J	100	1.28	1.28	400	511.5		J	100	1.07	1.07	400	426.5
	K	81	1.04	1.16	420	486.0		K	81	0.86	0.97	420	405.3
0.7	L	65.5	0.84	0.98	150	146.4	0.7	L	65.5	0.70	0.81	150	122.1
0.8	M	51.1	0.65	0.80	50	40.0	0.8	M	51.1	0.54	0.67	50	33.4
				Sum =		2462.7					Sum =		2053.7
				Average Depth =		1.2					Average Depth =		1.0

LNP COL 2.4-2

Table 2.4.3-213 (Sheet 2 of 3)
Computation of Drainage Average Depths (Increments 7 to 12)

	Increment #9							Increment #10					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	Delta A	Delta V
1500	A	100	0.91	0.91	10	9.1	1500	A	100	0.79	0.79	10	7.9
	B	100	0.91	0.91	15	13.6		B	100	0.79	0.79	15	11.8
	C	100	0.91	0.91	25	22.7		C	100	0.79	0.79	25	19.7
	D	100	0.91	0.91	50	45.4		D	100	0.79	0.79	50	39.3
	E	100	0.91	0.91	75	68.1		E	100	0.79	0.79	75	59.0
	F	100	0.91	0.91	125	113.5		F	100	0.79	0.79	125	98.3
	G	100	0.91	0.91	150	136.2		G	100	0.79	0.79	150	117.9
	H	100	0.91	0.91	250	227.0		H	100	0.79	0.79	250	196.5
	I	100	0.91	0.91	300	272.4		I	100	0.79	0.79	300	235.9
	J	100	0.91	0.91	400	363.2		J	100	0.79	0.79	400	314.5
	K	81	0.74	0.82	420	345.2		K	81	0.64	0.71	420	298.8
0.7	L	65.5	0.59	0.69	150	104.0	0.7	L	65.5	0.51	0.60	150	90.0
0.8	M	51.1	0.46	0.57	50	28.4	0.8	M	51.1	0.40	0.49	50	24.6
				Sum =		1748.8					Sum =		1514.1
				Average Depth =		0.9					Average Depth =		0.7

LNP COL 2.4-2

Table 2.4.3-213 (Sheet 3 of 3)
Computation of Drainage Average Depths (Increments 7 to 12)

	Increment #11							Increment #12					
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**Delta A**	**Delta V**
1500	A	100	0.69	0.69	10	6.9	1500	A	100	0.61	0.61	10	6.1
	B	100	0.69	0.69	15	10.3		B	100	0.61	0.61	15	9.2
	C	100	0.69	0.69	25	17.2		C	100	0.61	0.61	25	15.3
	D	100	0.69	0.69	50	34.5		D	100	0.61	0.61	50	30.6
	E	100	0.69	0.69	75	51.7		E	100	0.61	0.61	75	45.9
	F	100	0.69	0.69	125	86.2		F	100	0.61	0.61	125	76.5
	G	100	0.69	0.69	150	103.5		G	100	0.61	0.61	150	91.8
	H	100	0.69	0.69	250	172.5		H	100	0.61	0.61	250	153.0
	I	100	0.69	0.69	300	207.0		I	100	0.61	0.61	300	183.6
	J	100	0.69	0.69	400	275.9		J	100	0.61	0.61	400	244.8
	K	81	0.56	0.62	420	262.2		K	81	0.50	0.55	420	232.7
0.7	L	65.5	0.45	0.53	150	79.0	0.7	L	65.5	0.40	0.47	150	70.1
0.8	M	51.1	0.35	0.43	50	21.6	0.8	M	51.1	0.31	0.38	50	19.2
				Sum =		1328.6					Sum =		1178.9
				Average Depth =		0.7					Average Depth =		0.6

Notes:

mi.^{2} = square mile

LNP COL 2.4-2

Table 2.4.3-214
72-Hour Total Drainage – Averaged PMP

Increment	Duration (hr.)	Drainage Averaged PMP (inches)	Incremental Drainage Averaged PMP (inches)
1	6	14.22	14.22
2	12	20.81	6.59
3	18	24.65	3.84
4	24	27.25	2.60
5	30	29.17	1.92
6	36	30.68	1.50
7	42	31.89	1.22
8	48	32.91	1.02
9	54	33.78	0.87
10	60	34.53	0.75
11	66	35.18	0.66
12	72	35.77	0.58

Notes:

hr. = hour

LNP COL 2.4-2

Table 2.4.3-215
Distribution of PMP According to ANSI/ANS-2.8-1992

6-Hr. Period	Time (hr.)	Incremental Average PMP	ANSI Sequence	Sequence No.	ANSI Storm Distribution	Storm Pattern	Cumulative PMP
1	6	14.22	4	1st-day	2.60	1.02	1.02
2	12	6.59	2		6.59	1.22	2.24
3	18	3.84	1		14.22	1.50	3.74
4	24	2.60	3		3.84	1.92	5.66
5	30	1.92	8	2nd-day	1.02	2.60	8.26
6	36	1.50	6		1.50	6.59	14.85
7	42	1.22	5		1.92	14.22	29.07
8	48	1.02	7		1.22	3.84	32.91
9	54	0.87	12	3rd-day	0.58	0.87	33.78
10	60	0.75	10		0.75	0.75	34.53
11	66	0.66	9		0.87	0.66	35.18
12	72	0.58	11		0.66	0.58	35.77

Notes:

PMP depths are in inches.
hr. = hour

Source: Reference 2.4.3-201

LNP COL 2.4-2

Table 2.4.3-216
Incremental and Cumulative Probable Maximum Precipitation for the Inglis Dam

Time (hr.)	Incremental PMP for the Inglis Dam (inches)	Cumulative PMP for the Inglis Dam (inches)	Time (hr.)	Incremental PMP for the Inglis Dam (inches)	Cumulative PMP for the Inglis Dam (inches)
1	0.06	0.06	37	1.42	12.33
2	0.06	0.12	38	1.90	14.24
3	0.07	0.19	39	2.89	17.12
4	0.07	0.26	40	3.57	20.70
5	0.08	0.34	41	2.32	23.02
6	0.09	0.43	42	1.63	24.64
7	0.09	0.52	43	1.26	25.91
8	0.10	0.62	44	1.03	26.94
9	0.11	0.73	45	0.87	27.81
10	0.12	0.85	46	0.76	28.57
11	0.12	0.97	47	0.67	29.23
12	0.13	1.10	48	0.59	29.83
13	0.14	1.25	49	0.53	30.36
14	0.15	1.40	50	0.48	30.85
15	0.16	1.56	51	0.44	31.29
16	0.17	1.74	52	0.40	31.69
17	0.19	1.93	53	0.37	32.06
18	0.20	2.13	54	0.34	32.41
19	0.21	2.34	55	0.32	32.73
20	0.23	2.57	56	0.29	33.02
21	0.25	2.82	57	0.27	33.29
22	0.26	3.08	58	0.25	33.55
23	0.28	3.36	59	0.24	33.79
24	0.31	3.67	60	0.22	34.01
25	0.33	4.00	61	0.21	34.22
26	0.36	4.36	62	0.19	34.41
27	0.39	4.74	63	0.18	34.59
28	0.42	5.17	64	0.17	34.76
29	0.46	5.63	65	0.16	34.92
30	0.51	6.14	66	0.15	35.07
31	0.56	6.70	67	0.14	35.21
32	0.63	7.32	68	0.13	35.33
33	0.71	8.03	69	0.12	35.45
34	0.81	8.84	70	0.11	35.57
35	0.94	9.79	71	0.10	35.67
36	1.13	10.92	72	0.10	35.77

Notes:

hr. = hour

LNP COL 2.4-2

Table 2.4.3-217
Subbasin Areas

Basin ID (Figure 2.4.3-208)	Area ($mi.^2$)	Notes
Subbasin-1	230.0	Land area
Subbasin-2	102.9	Land area
Subbasin-3	133.9	Land area
Subbasin-4	78.1	Land area
Subbasin-5	141.1	Land area
Subbasin-6	34.1	Land area
Subbasin-7	169.1	Land area
Subbasin-8	78.0	Land area
Subbasin-9	86.2	Land area
Subbasin-10	102.5	Land area
Subbasin-11	164.9	Land area
Subbasin-12	194.8	Land area
Subbasin-13	220.3	Land area
Subbasin-14	50.3	Land area
Subbasin-15	64.8	Land area
Subbasin-16	91.7	Land area
Subbasin-17	48.8	Land area
Subbasin-18	21.9	Land area
Lake Surface	6.5	Water Surface
Total	**2020**	

Notes:

$mi.^2$ = square mile

LNP COL 2.4-2

Table 2.4.3-218
Hydrologic Soil Groups in the Withlacoochee River Drainage Basin

Sub-Basin	A	B	C	B/D	D	W
Subbasin 1	10.9%	0.0%	4.1%	61.5%	21.3%	2.2%
Subbasin 2	8.0%	0.0%	3.6%	51.9%	36.1%	0.5%
Subbasin 3	4.0%	0.0%	3.1%	49.2%	43.3%	0.4%
Subbasin 4	56.0%	0.0%	6.5%	21.2%	15.1%	1.2%
Subbasin 5	6.7%	0.0%	10.9%	41.8%	40.3%	0.3%
Subbasin 6	28.9%	0.0%	9.8%	38.3%	20.7%	2.4%
Subbasin 7	63.2%	0.1%	17.2%	4.8%	12.9%	1.9%
Subbasin 8	24.0%	0.0%	36.7%	22.8%	14.7%	1.7%
Subbasin 9	9.8%	0.0%	23.7%	19.5%	37.8%	9.2%
Subbasin 10	24.2%	0.0%	11.6%	35.9%	26.6%	1.8%
Subbasin 11	52.5%	0.0%	10.3%	12.6%	16.0%	8.6%
Subbasin 12	81.7%	0.0%	2.5%	4.0%	8.5%	3.3%
Subbasin 13	62.0%	0.9%	3.1%	8.3%	15.3%	10.3%
Subbasin 14	92.5%	0.1%	1.9%	2.4%	3.0%	0.0%
Subbasin 15	85.5%	0.4%	3.4%	3.7%	3.7%	3.3%
Subbasin 16	94.8%	0.0%	1.5%	2.4%	0.7%	0.6%
Subbasin 17	91.3%	1.6%	1.6%	1.1%	1.1%	3.3%
Subbasin 18	5.5%	0.0%	9.3%	76.9%	3.4%	4.9%
Lake Surface	0.0%	0.0%	0.0%	0.0%	0.0%	100%

Notes:

A = high infiltration rate
B = moderate infiltration rate
C = slow infiltration rate
D = very slow infiltration rate
B/D = moderate to very slow infiltration rate
W = open water

Source: Reference 2.4.3-208

LNP COL 2.4-2

Table 2.4.3-219
Land Use in the Withlacoochee River Drainage Basin

Land Use	%
Cropland and Pasture	33.50%
Forested Wetland	20.70%
Evergreen Forest Land	18.84%
Herbaceous Rangeland	6.47%
Orchard, Grove, Vineyard, Nursery, Ornamental	5.12%
Nonforested Wetland	4.23%
Transitional Areas	3.99%
Residential	2.90%
Lakes	1.94%
Strip Mines	0.54%
Reservoirs	0.52%
Transportation, Communication, Utility	0.52%
Commercial and Services	0.21%
Streams and Canals	0.17%
Industrial	0.11%
Other Urban or Built-Up	0.10%
Other Agricultural Land	0.05%
Confined Feeding Ops	0.04%
Shrub & Brush Rangeland	0.01%
Dry Salt Flats	0.00%
Bays and Estuaries	0.00%

Source: Reference 2.4.3-208

LNP COL 2.4-2

Table 2.4.3-220
Subbasin Loss Parameters

Subbasin	Initial Loss I_a (inch)	Constant rate (in/hr)	% Impervious
Subbasin 1	0.0	0.08	0
Subbasin 2	0.0	0.06	0
Subbasin 3	0.0	0.05	0
Subbasin 4	0.0	0.19	0
Subbasin 5	0.0	0.06	0
Subbasin 6	0.0	0.12	0
Subbasin 7	0.0	0.20	0
Subbasin 8	0.0	0.11	0
Subbasin 9	0.0	0.06	0
Subbasin 10	0.0	0.11	0
Subbasin 11	0.0	0.17	0
Subbasin 12	0.0	0.25	0
Subbasin 13	0.0	0.20	0
Subbasin 14	0.0	0.28	0
Subbasin 15	0.0	0.26	0
Subbasin 16	0.0	0.29	0
Subbasin 17	0.0	0.28	0
Subbasin 18	0.0	0.08	0
Lake Surface	NA	NA	NA

Notes:

in. = inch
in/hr = inch per hour
NA = Not applicable

LNP COL 2.4-2

Table 2.4.3-221
Subbasin Unit Hydrograph Characteristics

Subbasin	A ($mi.^2$)	L (mi.)	L_{ca} (mi.)	C_t[(a)]	C_p[(b)]	Initial Computed[(c)] t_L (hr.)	Final t_L Used in UH (hr.)	UH Qp (cfs)
Subbasin 1	230.0	23.7	10.8	8	0.8	42.2	33.8	3488
Subbasin 2	102.9	35.2	19.5	8	0.8	56.8	45.4	1161
Subbasin 3	133.9	17.9	7.9	8	0.8	35.3	28.2	2429
Subbasin 4	78.1	12.1	5.0	8	0.8	27.5	22.0	1819
Subbasin 5	141.1	26.5	12.3	8	0.8	45.4	36.3	1990
Subbasin 6	34.1	12.0	7.3	8	0.8	30.6	24.5	714
Subbasin 7	169.1	18.5	9.2	8	0.8	37.4	29.9	2893
Subbasin 8	78.0	17.3	7.6	8	0.8	34.6	27.7	1443
Subbasin 9	86.2	22.2	8.6	8	0.8	38.7	30.9	1426
Subbasin 10	102.5	28.3	20.5	8	0.8	54.0	43.2	1216
Subbasin 11	164.9	27.6	18.5	8	0.8	51.9	41.5	2032
Subbasin 12	194.8	26.6	13.4	8	0.8	46.6	37.3	2676
Subbasin 13	220.3	24.2	12.0	8	0.8	43.8	35.1	3216
Subbasin 14	50.3	11.3	7.4	8	0.8	30.2	24.2	1065
Subbasin 15	64.8	12.7	7.8	8	0.8	31.8	25.4	1305
Subbasin 16	91.7	12.3	6.3	8	0.8	29.5	23.6	1989
Subbasin 17	48.8	8.9	3.2	8	0.8	21.8	17.4	1432
Subbasin 18	21.9	5.3	2.5	8	0.8	17.3	13.9	808
Lake Surface	NA	NA	NA	NA	NA	NA	NA	NA

Notes:

a) Data Source: Reference 2.4.3-211
b) Cp values were increased by 33% from 0.6 to 0.8 to account for reduction in travel time during PMP conditions.
c) The computed Snyder lag times, tL, for all subbasins were adjusted downward to increase the peak flow of various unit hydrographs by an additional 25% for a total increase in peak flow of about 66%.

NA = Not Applicable

hr. = hour, mi. = mile, $mi.^2$ = square mile, cfs = cubic feet per second, UH = unit hydrograph

LNP COL 2.4-2

Table 2.4.3-222
Comparison of Flood Frequency Based Peak Flows with Those Obtained from HEC-HMS Model

T (yrs)	24-Hr. Rain (in.)	Flood Frequency Based Peak Flow (cfs)	HEC-HMS Based Peak Flow (cfs)	% Error
10	7.5	6442	9574	-49%
25	9.5	8301	12,630	-52%
50	10.5	9408	14,131	-50%
100	11.5	10,709	15,748	-47%
500	14.49	13,678	20,646	-51%
18,358[a]	19.99	18,000	29,734	-65%
3.23E+08[b]	35.80[c]	39,850	60,598	-52%

Notes:

a) Standard Project Flood (Magnitude = 18,000 cfs, [Reference 2.4.3-215], estimated return period)

b) Probable Maximum Flood based on the developed relationship $T = \exp\left[\frac{P - 4.08}{1.62}\right]$

c) 72-hour PMP (inches)

yrs = years
hr. = hour
cfs = cubic foot per second

LNP COL 2.4-2

Table 2.4.3-223
Reach Routing Parameters for Muskingum Method

Reach	K	X	# of Reaches
R-1	48	0	4
R-2	12	0	2
R-3	12	0	1
R-4	96	0	4
R-5	96	0	4
R-6	96	0	4
R-7	96	0	2
R-8	96	0	1
R-9	96	0	2
R-10	96	0	1
R-11	24	0	1

LNP COL 2.4-2

Table 2.4.3-224
Stage-Storage Relationship for Lake Rousseau

Elevation (ft.)	Cumulative Vol (ac-ft)	Surface Area (ac.)
0	6.9	3.4
1	9.1	3.8
2	11.6	4.3
3	14.7	5.3
4	75.1	116.7
5	104.7	119.1
6	140	125.3
7	181.3	131.7
8	368.3	284.1
9	439.4	296.5
10	520.3	306.1
11	611.9	316.9
12	856.5	675.2
13	1014.2	689.8
14	1195.6	713.4
15	1401.3	736.7
16	2005.6	1649.7
17	2414.7	1686.5
18	2899.1	1734.8
19	3458.5	1782.2
20	5008.9	2526.2
21	6063.2	2625.3
22	7271.2	2743.2
23	8638.8	2875.5
24	12,432.3	4309.8
25	14,815.7	4373.9
26	17,321.2	4450.6
27	19,944.6	4526.5

Notes:

ft. = foot
ac. = acre

Source: Reference 2.4.3-216

LNP COL 2.4-2

Table 2.4.3-225
Stage-Discharge Relationships for Lake Rousseau

Lake Water Elevation (ft.)	Inglis Main Dam Discharge (cfs)[a]	ByPass Discharge (cfs)[b]	Flow through Surrounding Low Lying Area (cfs) [c]	Total Q (cfs)
14	1042	-	-	1042
15.3	1895	-	-	1895
16.6	3041	-	-	3041
18.4	5030	-	-	5030
20.7	8082	-	-	8082
22	9976	110	-	10,087
23.2	11,786	311	-	12,098
24	13,008	480	-	13,489
24.8	14,230	677	-	14,908
25.2	14,838	786	-	15,625
26	16,042	1025	-	17,067
26.8	17,220	1292	-	18,512
27.2	17,797	1436	-	19,233
28	18,919	1745	-	20,663
29	20,248	2170	12,997	35,416
30	21,480	2639	38,236	62,355
31	22,592	3152	72,358	98,103
32	23,566	3709	114,184	141,459
33	24,382	4309	163,041	191,731
34	25,019	4953	218,481	248,453
35	25,458	5641	280,180	311,279

Notes:

a) Inglis Dam discharge (Reference 2.4.1-223)
b) Bypass Channel discharge (Reference 2.4.1-222)
c) approximate calculation

cfs = cubic foot per second, ft. = foot

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 1 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 1	0:00	1028.6	21,561.1	27.5	4733.7
Day 1	1:00	1112.5	21,302.3	27.4	3668.4
Day 1	2:00	1112.5	21,119.8	27.4	2974.7
Day 1	3:00	1154.4	20,985.3	27.3	2546.3
Day 1	4:00	1154.4	20,883.7	27.3	2222.4
Day 1	5:00	1154.4	20,805.7	27.3	1973.9
Day 1	6:00	1154.4	20,745.8	27.3	1783.2
Day 1	7:00	1196.4	20,701.4	27.2	1641.8
Day 1	8:00	1196.4	20,668.9	27.2	1538.2
Day 1	9:00	1196.4	20,643.9	27.2	1458.6
Day 1	10:00	1238.3	20,626.3	27.2	1402.5
Day 1	11:00	1238.3	20,614.3	27.2	1364.3
Day 1	12:00	1238.3	20,605.1	27.2	1335.0
Day 1	13:00	1280.3	20,599.6	27.2	1317.4
Day 1	14:00	1280.3	20,596.9	27.2	1308.7
Day 1	15:00	1322.2	20,596.3	27.2	1307.0
Day 1	16:00	1322.2	20,597.5	27.2	1310.5
Day 1	17:00	1322.2	20,598.3	27.2	1313.3
Day 1	18:00	1364.2	20,600.5	27.2	1320.2
Day 1	19:00	1406.1	20,605.2	27.2	1335.3
Day 1	20:00	1406.1	20,610.4	27.2	1351.8
Day 1	21:00	1448.1	20,615.9	27.2	1369.3
Day 1	22:00	1490.0	20,623.2	27.2	1392.5
Day 1	23:00	1490.0	20,630.3	27.2	1415.2
Day 2	0:00	1532.0	20,637.3	27.2	1437.5
Day 2	1:00	1573.9	20,645.7	27.2	1464.4
Day 2	2:00	1615.9	20,655.3	27.2	1494.7
Day 2	3:00	1657.9	20,665.6	27.2	1527.8
Day 2	4:00	1741.9	20,678.2	27.2	1567.9
Day 2	5:00	1783.9	20,692.5	27.2	1613.2

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 2 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 2	6:00	1868.0	20,708.0	27.2	1662.7
Day 2	7:00	1952.1	20,726.1	27.2	1720.3
Day 2	8:00	2078.3	20,747.6	27.3	1788.9
Day 2	9:00	2204.5	20,773.3	27.3	1870.9
Day 2	10:00	2372.8	20,803.8	27.3	1968.1
Day 2	11:00	2625.2	20,842.6	27.3	2091.6
Day 2	12:00	2919.8	20,892.3	27.3	2250.1
Day 2	13:00	3424.7	20,959.7	27.3	2464.6
Day 2	14:00	4223.9	21,059.0	27.3	2781.0
Day 2	15:00	5863.6	21,222.8	27.4	3341.1
Day 2	16:00	7044.0	21,442.7	27.5	4246.3
Day 2	17:00	4955.9	21,566.5	27.5	4756.2
Day 2	18:00	3795.3	21,539.7	27.5	4645.6
Day 2	19:00	3185.7	21,458.1	27.5	4309.7
Day 2	20:00	2834.6	21,366.3	27.4	3931.8
Day 2	21:00	2616.7	21,281.1	27.4	3581.0
Day 2	22:00	2449.0	21,207.1	27.4	3276.2
Day 2	23:00	2373.8	21,144.8	27.4	3054.4
Day 3	0:00	2307.6	21,092.7	27.4	2888.3
Day 3	1:00	2250.8	21,048.2	27.3	2746.6
Day 3	2:00	2245.2	21,011.8	27.3	2630.6
Day 3	3:00	2290.8	20,985.3	27.3	2546.2
Day 3	4:00	2303.2	20,967.1	27.3	2488.2
Day 3	5:00	2365.8	20,955.9	27.3	2452.5
Day 3	6:00	2436.0	20,952.1	27.3	2440.5
Day 3	7:00	2512.8	20,954.6	27.3	2448.4
Day 3	8:00	2595.4	20,962.3	27.3	2473.0
Day 3	9:00	2683.0	20,974.4	27.3	2511.7
Day 3	10:00	2774.5	20,990.3	27.3	2562.2
Day 3	11:00	2910.8	21,010.8	27.3	2627.4

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 3 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 3	12:00	2910.8	21,010.8	27.3	2627.4
Day 3	13:00	3006.9	21,035.0	27.3	2704.5
Day 3	14:00	3103.6	21,060.6	27.3	2786.1
Day 3	15:00	3241.8	21,088.8	27.4	2876.1
Day 3	16:00	3337.0	21,119.0	27.4	2972.2
Day 3	17:00	3472.5	21,150.6	27.4	3072.9
Day 3	18:00	3564.6	21,183.1	27.4	3177.5
Day 3	19:00	3697.1	21,215.1	27.4	3309.3
Day 3	20:00	3828.2	21,247.1	27.4	3441.1
Day 3	21:00	3916.0	21,277.6	27.4	3566.5
Day 3	22:00	4044.5	21,306.8	27.4	3686.8
Day 3	23:00	4129.7	21,335.1	27.4	3803.2
Day 4	0:00	4255.6	21,362.6	27.4	3916.4
Day 4	1:00	4380.0	21,390.9	27.4	4033.1
Day 4	2:00	4335.0	21,413.8	27.5	4127.5
Day 4	3:00	4456.2	21,432.8	27.5	4205.4
Day 4	4:00	4575.4	21,454.7	27.5	4295.7
Day 4	5:00	4692.8	21,478.6	27.5	4394.1
Day 4	6:00	4808.1	21,503.7	27.5	4497.7
Day 4	7:00	4921.3	21,529.7	27.5	4604.4
Day 4	8:00	5032.3	21,556.0	27.5	4712.7
Day 4	9:00	5141.1	21,582.4	27.5	4821.5
Day 4	10:00	5247.6	21,608.7	27.5	4929.9
Day 4	11:00	5351.9	21,634.8	27.5	5037.4
Day 4	12:00	5454.0	21,660.6	27.5	5143.7
Day 4	13:00	5553.8	21,686.1	27.5	5248.5
Day 4	14:00	5651.4	21,711.1	27.5	5351.5
Day 4	15:00	5746.9	21,735.7	27.6	5452.6
Day 4	16:00	5840.1	21,759.7	27.6	5551.7
Day 4	17:00	5931.3	21,783.3	27.6	5648.8

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 4 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 4	18:00	6107.3	21,829.0	27.6	5836.9
Day 4	19:00	6192.3	21,851.0	27.6	5930.5
Day 4	20:00	6275.4	21,871.9	27.6	6032.0
Day 4	21:00	6356.5	21,891.4	27.6	6126.9
Day 4	22:00	6435.7	21,910.0	27.6	6217.0
Day 4	23:00	6513.1	21,927.7	27.6	6303.1
Day 5	0:00	6588.6	21,944.7	27.6	6386.0
Day 5	1:00	6662.2	21,961.2	27.6	6466.0
Day 5	2:00	6734.0	21,977.2	27.6	6543.6
Day 5	3:00	6804.1	21,992.7	27.6	6619.0
Day 5	4:00	6872.7	22,007.8	27.6	6692.4
Day 5	5:00	6939.8	22,022.5	27.6	6763.9
Day 5	6:00	7005.4	22,036.9	27.6	6833.7
Day 5	7:00	7069.7	22,050.9	27.6	6901.9
Day 5	8:00	7132.5	22,064.6	27.7	6968.5
Day 5	9:00	7194.1	22,078.0	27.7	7033.7
Day 5	10:00	7254.3	22,091.1	27.7	7097.4
Day 5	11:00	7313.3	22,104.0	27.7	7159.8
Day 5	12:00	7371.1	22,116.5	27.7	7220.8
Day 5	13:00	7427.6	22,128.8	27.7	7280.5
Day 5	14:00	7483.0	22,140.8	27.7	7339.0
Day 5	15:00	7537.2	22,152.6	27.7	7396.2
Day 5	16:00	7590.4	22,164.1	27.7	7452.3
Day 5	17:00	7642.4	22,175.4	27.7	7507.2
Day 5	18:00	7693.4	22,186.5	27.7	7560.9
Day 5	19:00	7743.3	22,197.3	27.7	7613.6
Day 5	20:00	7792.3	22,207.9	27.7	7665.2
Day 5	21:00	7840.2	22,218.3	27.7	7715.7
Day 5	22:00	7887.1	22,228.5	27.7	7765.2
Day 5	23:00	7933.1	22,238.5	27.7	7813.7

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 5 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 6	0:00	7978.1	22,248.3	27.7	7861.1
Day 6	1:00	8022.2	22,257.8	27.7	7907.6
Day 6	2:00	8065.4	22,267.2	27.7	7953.2
Day 6	3:00	8107.7	22,276.4	27.7	7997.8
Day 6	4:00	8149.1	22,285.4	27.7	8041.5
Day 6	5:00	8189.7	22,294.2	27.7	8084.3
Day 6	6:00	8229.5	22,302.8	27.7	8126.2
Day 6	7:00	8268.4	22,311.3	27.7	8167.2
Day 6	8:00	8306.6	22,319.5	27.7	8207.5
Day 6	9:00	8344.0	22,327.6	27.7	8246.9
Day 6	10:00	8380.6	22,335.6	27.7	8285.5
Day 6	11:00	8416.6	22,343.4	27.7	8323.3
Day 6	12:00	8451.8	22,351.0	27.7	8360.4
Day 6	13:00	8486.3	22,358.5	27.7	8396.7
Day 6	14:00	8520.2	22,365.8	27.7	8432.4
Day 6	15:00	8553.5	22,373.0	27.7	8467.3
Day 6	16:00	8586.0	22,380.1	27.7	8501.6
Day 6	17:00	8618.0	22,387.0	27.7	8535.2
Day 6	18:00	8649.4	22,393.7	27.7	8568.1
Day 6	19:00	8680.2	22,400.4	27.7	8600.5
Day 6	20:00	8710.3	22,406.9	27.8	8632.2
Day 6	21:00	8740.0	22,413.3	27.8	8663.3
Day 6	22:00	8769.1	22,419.6	27.8	8693.8
Day 6	23:00	8797.6	22,425.8	27.8	8723.7
Day 7	0:00	8825.6	22,431.8	27.8	8753.1
Day 7	1:00	9104.8	22,446.4	27.8	8824.0
Day 7	2:00	9173.7	22,468.1	27.8	8929.5
Day 7	3:00	9200.2	22,485.8	27.8	9015.6
Day 7	4:00	9268.1	22,500.9	27.8	9088.7
Day 7	5:00	9335.6	22,515.5	27.8	9160.0

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 6 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 7	6:00	9360.7	22,528.4	27.8	9224.7
Day 7	7:00	9427.3	22,539.8	27.8	9287.2
Day 7	8:00	9493.4	22,551.5	27.8	9351.1
Day 7	9:00	9559.1	22,563.3	27.8	9415.8
Day 7	10:00	9582.5	22,573.7	27.8	9473.0
Day 7	11:00	9647.4	22,583.3	27.8	9525.4
Day 7	12:00	9711.8	22,593.7	27.8	9582.3
Day 7	13:00	9776.0	22,604.6	27.8	9642.0
Day 7	14:00	9839.7	22,615.8	27.8	9703.2
Day 7	15:00	9903.1	22,627.1	27.8	9765.3
Day 7	16:00	10,008.2	22,639.9	27.8	9835.6
Day 7	17:00	10,071.0	22,653.7	27.8	9910.9
Day 7	18:00	10,133.5	22,666.6	27.8	9981.5
Day 7	19:00	10,237.7	22,680.3	27.8	10,056.9
Day 7	20:00	10,341.8	22,696.0	27.8	10,142.9
Day 7	21:00	10,403.6	22,711.5	27.8	10,227.7
Day 7	22:00	10,507.3	22,726.9	27.8	10,311.8
Day 7	23:00	10,652.8	22,744.9	27.9	10,410.8
Day 8	0:00	10,756.4	22,764.7	27.9	10,519.3
Day 8	1:00	10,902.0	22,785.6	27.9	10,633.7
Day 8	2:00	11,047.6	22,808.6	27.9	10,759.6
Day 8	3:00	11,193.7	22,832.9	27.9	10,892.9
Day 8	4:00	11,382.2	22,859.5	27.9	11,038.8
Day 8	5:00	11,613.5	22,890.5	27.9	11,208.2
Day 8	6:00	11,845.8	22,925.6	27.9	11,400.7
Day 8	7:00	12,163.6	22,966.3	27.9	11,623.7
Day 8	8:00	12,525.4	23,014.9	27.9	11,889.8
Day 8	9:00	12,973.7	23,072.8	27.9	12,207.2
Day 8	10:00	13,551.2	23,143.9	28.0	12,596.8
Day 8	11:00	14,384.7	23,236.0	28.0	13,111.3

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 7 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 8	12:00	15,643.1	23,362.0	28.0	13,868.6
Day 8	13:00	17,705.5	23,547.7	28.1	14,985.3
Day 8	14:00	21,916.5	23,867.1	28.2	16,906.1
Day 8	15:00	24,838.2	24,289.9	28.3	19,617.5
Day 8	16:00	19,679.8	24,462.1	28.3	20,732.6
Day 8	17:00	16,889.6	24,302.5	28.3	19,699.2
Day 8	18:00	15,464.1	24,072.9	28.2	18,212.3
Day 8	19:00	14,651.2	23,866.3	28.2	16,901.3
Day 8	20:00	14,159.8	23,701.1	28.1	15,907.9
Day 8	21:00	13,907.8	23,577.1	28.1	15,161.9
Day 8	22:00	13,771.2	23,489.5	28.1	14,635.6
Day 8	23:00	13,709.1	23,430.2	28.0	14,279.2
Day 9	0:00	13,764.0	23,394.3	28.0	14,063.2
Day 9	1:00	13,894.2	23,378.8	28.0	13,970.0
Day 9	2:00	14,099.2	23,380.6	28.0	13,980.6
Day 9	3:00	14,335.7	23,396.3	28.0	14,074.9
Day 9	4:00	14,643.6	23,423.7	28.0	14,240.0
Day 9	5:00	14,978.9	23,461.5	28.1	14,467.4
Day 9	6:00	15,380.7	23,508.7	28.1	14,750.9
Day 9	7:00	15,762.2	23,563.0	28.1	15,077.5
Day 9	8:00	16,204.4	23,623.0	28.1	15,438.1
Day 9	9:00	16,703.9	23,690.2	28.1	15,842.5
Day 9	10:00	17,173.5	23,762.8	28.1	16,278.8
Day 9	11:00	17,651.7	23,837.9	28.2	16,730.1
Day 9	12:00	18,176.7	23,916.2	28.2	17,201.4
Day 9	13:00	18,661.3	23,995.7	28.2	17,712.9
Day 9	14:00	19,186.8	24,074.7	28.2	18,224.2
Day 9	15:00	19,709.8	24,154.5	28.3	18,740.9
Day 9	16:00	20,229.5	24,234.6	28.3	19,259.6
Day 9	17:00	20,745.3	24,314.7	28.3	19,777.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 8 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 9	18:00	21,257.0	24,394.4	28.3	20,294.3
Day 9	19:00	21,764.3	24,473.8	28.3	20,807.8
Day 9	20:00	22,267.0	24,552.5	28.4	21,317.7
Day 9	21:00	22,764.6	24,630.6	28.4	21,823.4
Day 9	22:00	23,256.8	24,707.2	28.4	22,346.0
Day 9	23:00	23,785.2	24,782.7	28.4	22,867.6
Day 10	0:00	23,887.7	24,845.0	28.4	23,297.6
Day 10	1:00	24,403.0	24,899.5	28.5	23,673.9
Day 10	2:00	24,911.1	24,962.7	28.5	24,110.3
Day 10	3:00	25,411.6	25,030.3	28.5	24,576.9
Day 10	4:00	25,904.3	25,099.8	28.5	25,056.8
Day 10	5:00	26,388.9	25,169.9	28.5	25,540.5
Day 10	6:00	26,865.0	25,239.8	28.5	26,022.8
Day 10	7:00	27,332.6	25,309.0	28.6	26,500.4
Day 10	8:00	27,791.6	25,377.2	28.6	26,971.7
Day 10	9:00	28,241.7	25,443.7	28.6	27,453.9
Day 10	10:00	28,682.9	25,507.8	28.6	27,919.4
Day 10	11:00	29,115.2	25,570.0	28.6	28,371.7
Day 10	12:00	29,538.5	25,630.8	28.7	28,812.6
Day 10	13:00	29,952.9	25,690.1	28.7	29,243.4
Day 10	14:00	30,358.3	25,748.0	28.7	29,664.5
Day 10	15:00	30,755.0	25,804.8	28.7	30,076.4
Day 10	16:00	31,142.9	25,860.2	28.7	30,479.2
Day 10	17:00	31,522.2	25,914.5	28.7	30,873.2
Day 10	18:00	31,893.1	25,967.5	28.7	31,258.4
Day 10	19:00	32,255.5	26,019.4	28.8	31,635.1
Day 10	20:00	32,609.8	26,070.1	28.8	32,003.3
Day 10	21:00	32,956.0	26,119.6	28.8	32,363.2
Day 10	22:00	33,294.2	26,167.6	28.8	32,725.4
Day 10	23:00	33,624.5	26,213.8	28.8	33,076.5

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 9 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 11	0:00	33,947.0	26,258.4	28.8	33,415.7
Day 11	1:00	34,261.9	26,301.7	28.8	33,745.2
Day 11	2:00	34,569.7	26,343.8	28.8	34,066.0
Day 11	3:00	34,870.7	26,385.0	28.9	34,378.9
Day 11	4:00	35,165.0	26,425.2	28.9	34,684.5
Day 11	5:00	35,452.9	26,464.4	28.9	34,983.2
Day 11	6:00	35,734.6	26,502.8	28.9	35,275.3
Day 11	7:00	36,010.3	26,540.4	28.9	35,560.9
Day 11	8:00	36,280.1	26,577.1	28.9	35,840.4
Day 11	9:00	36,544.2	26,613.0	28.9	36,113.9
Day 11	10:00	36,802.8	26,648.2	28.9	36,381.6
Day 11	11:00	37,055.9	26,682.7	28.9	36,643.6
Day 11	12:00	37,303.8	26,716.4	28.9	36,900.1
Day 11	13:00	37,546.6	26,749.4	28.9	37,151.3
Day 11	14:00	37,784.3	26,781.7	29.0	37,397.2
Day 11	15:00	38,017.2	26,813.4	29.0	37,638.1
Day 11	16:00	38,245.4	26,844.4	29.0	37,874.0
Day 11	17:00	38,468.9	26,874.8	29.0	38,105.1
Day 11	18:00	38,687.9	26,904.4	29.0	38,334.5
Day 11	19:00	38,902.4	26,933.1	29.0	38,561.7
Day 11	20:00	39,112.7	26,960.9	29.0	38,781.6
Day 11	21:00	39,318.6	26,987.9	29.0	38,995.7
Day 11	22:00	39,520.4	27,014.3	29.0	39,204.7
Day 11	23:00	39,718.1	27,040.1	29.0	39,409.2
Day 12	0:00	39,911.8	27,065.4	29.0	39,609.3
Day 12	1:00	40,101.6	27,090.1	29.0	39,805.3
Day 12	2:00	40,287.6	27,114.3	29.0	39,997.3
Day 12	3:00	40,469.9	27,138.1	29.0	40,185.5
Day 12	4:00	40,648.5	27,161.4	29.1	40,369.8
Day 12	5:00	40,823.5	27,184.2	29.1	40,550.4

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 10 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 12	6:00	40,995.1	27,206.5	29.1	40,727.5
Day 12	7:00	41,163.4	27,228.4	29.1	40,901.0
Day 12	8:00	41,328.3	27,249.9	29.1	41,071.1
Day 12	9:00	41,490.1	27,270.9	29.1	41,237.9
Day 12	10:00	41,648.7	27,291.6	29.1	41,401.4
Day 12	11:00	41,804.3	27,311.8	29.1	41,561.7
Day 12	12:00	41,956.9	27,331.7	29.1	41,719.0
Day 12	13:00	42,106.7	27,351.2	29.1	41,873.3
Day 12	14:00	42,253.6	27,370.3	29.1	42,024.6
Day 12	15:00	42,397.8	27,389.0	29.1	42,173.1
Day 12	16:00	42,539.2	27,407.4	29.1	42,318.8
Day 12	17:00	42,678.1	27,425.5	29.1	42,461.8
Day 12	18:00	42,814.4	27,443.2	29.1	42,602.1
Day 12	19:00	42,948.1	27,460.5	29.1	42,739.8
Day 12	20:00	43,079.5	27,477.6	29.1	42,874.9
Day 12	21:00	43,208.4	27,494.4	29.1	43,007.6
Day 12	22:00	43,335.0	27,510.8	29.1	43,137.9
Day 12	23:00	43,459.2	27,526.9	29.1	43,265.7
Day 13	0:00	43,581.3	27,542.8	29.2	43,391.3
Day 13	1:00	43,701.1	27,558.3	29.2	43,514.5
Day 13	2:00	43,818.8	27,573.6	29.2	43,635.6
Day 13	3:00	43,934.3	27,588.6	29.2	43,754.4
Day 13	4:00	44,047.8	27,603.4	29.2	43,871.1
Day 13	5:00	44,159.3	27,617.8	29.2	43,985.8
Day 13	6:00	44,268.8	27,632.0	29.2	44,098.4
Day 13	7:00	44,376.3	27,646.0	29.2	44,208.9
Day 13	8:00	44,481.9	27,659.7	29.2	44,317.5
Day 13	9:00	44,585.7	27,673.2	29.2	44,424.8
Day 13	10:00	44,687.6	27,686.2	29.2	44,532.0
Day 13	11:00	44,787.7	27,698.9	29.2	44,636.0

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 11 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 13	12:00	44,886.1	27,711.3	29.2	44,737.6
Day 13	13:00	44,982.7	27,723.5	29.2	44,837.1
Day 13	14:00	45,077.7	27,735.4	29.2	44,934.8
Day 13	15:00	45,170.9	27,747.1	29.2	45,030.6
Day 13	16:00	45,262.6	27,758.6	29.2	45124.8
Day 13	17:00	45,352.6	27,769.9	29.2	45,217.2
Day 13	18:00	45,441.1	27,781.0	29.2	45,308.1
Day 13	19:00	45,528.1	27,791.9	29.2	45,397.4
Day 13	20:00	45,613.5	27,802.6	29.2	45,485.1
Day 13	21:00	45,697.5	27,813.1	29.2	45,571.3
Day 13	22:00	45,780.1	27,823.5	29.2	45,656.0
Day 13	23:00	45,861.2	27,833.6	29.2	45,739.3
Day 14	0:00	45,941.0	27,843.6	29.2	45,821.1
Day 14	1:00	46,019.4	27,853.4	29.2	45,901.6
Day 14	2:00	46,096.6	27,863.1	29.2	45,980.7
Day 14	3:00	46,172.4	27,872.6	29.2	46,058.5
Day 14	4:00	46,247.0	27,881.9	29.2	46,135.0
Day 14	5:00	46,320.4	27,891.1	29.2	46,210.2
Day 14	6:00	46,392.5	27,900.1	29.2	46,284.2
Day 14	7:00	46,463.6	27,909.0	29.2	46,356.9
Day 14	8:00	46,533.4	27,917.8	29.2	46,428.5
Day 14	9:00	46,602.2	27,926.4	29.3	46,499.0
Day 14	10:00	46,669.9	27,934.8	29.3	46,568.3
Day 14	11:00	46,736.6	27,943.2	29.3	46,636.6
Day 14	12:00	46,802.3	27,951.4	29.3	46,703.8
Day 14	13:00	46,866.9	27,959.4	29.3	46,769.9
Day 14	14:00	46,930.6	27,967.4	29.3	46,835.1
Day 14	15:00	46,993.4	27,975.2	29.3	46,899.3
Day 14	16:00	47,055.3	27,983.0	29.3	46,962.5
Day 14	17:00	47,116.2	27,990.6	29.3	47,024.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 12 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 14	18:00	47,176.4	27,998.1	29.3	47,086.3
Day 14	19:00	47,235.7	28,005.5	29.3	47,146.9
Day 14	20:00	47,294.2	28,012.7	29.3	47,206.6
Day 14	21:00	47,352.0	28,019.9	29.3	47,265.5
Day 14	22:00	47,409.0	28,027.0	29.3	47,323.6
Day 14	23:00	47,465.2	28,034.0	29.3	47,381.0
Day 15	0:00	47,520.8	28,041.0	29.3	47437.7
Day 15	1:00	47,575.7	28,047.8	29.3	47,493.6
Day 15	2:00	47,630.0	28,054.5	29.3	47,548.9
Day 15	3:00	47,683.6	28,061.2	29.3	47,603.5
Day 15	4:00	47,736.7	28,067.8	29.3	47,657.4
Day 15	5:00	47,789.1	28,074.3	29.3	47,710.8
Day 15	6:00	47,841.1	28,080.7	29.3	47,763.5
Day 15	7:00	47,892.4	28,087.1	29.3	47,815.7
Day 15	8:00	47,943.3	28,093.4	29.3	47,867.4
Day 15	9:00	47,993.7	28,099.7	29.3	47,918.5
Day 15	10:00	48,043.6	28,105.8	29.3	47,969.2
Day 15	11:00	48,093.1	28,112.0	29.3	48,019.3
Day 15	12:00	48,142.2	28,118.0	29.3	48,069.1
Day 15	13:00	48,190.8	28,124.1	29.3	48,118.3
Day 15	14:00	48,239.1	28,130.0	29.3	48,167.2
Day 15	15:00	48,287.0	28,135.9	29.3	48,215.7
Day 15	16:00	48,334.6	28,141.8	29.3	48,263.8
Day 15	17:00	48,381.8	28,147.6	29.3	48,311.5
Day 15	18:00	48,428.7	28,153.4	29.3	48,358.9
Day 15	19:00	48,475.4	28,159.2	29.3	48,406.0
Day 15	20:00	48,521.7	28,164.9	29.3	48,452.8
Day 15	21:00	48,567.9	28,170.6	29.3	48,499.3
Day 15	22:00	48,613.7	28,176.2	29.3	48,545.6
Day 15	23:00	48,659.4	28,181.8	29.3	48,591.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 13 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 16	0:00	48,704.8	28,187.4	29.3	48,637.4
Day 16	1:00	48,750.1	28,193.0	29.3	48,682.9
Day 16	2:00	48,795.2	28,198.5	29.3	48,728.3
Day 16	3:00	48,840.1	28,204.0	29.3	48,773.5
Day 16	4:00	48,884.9	28,209.5	29.3	48,818.5
Day 16	5:00	48,929.5	28,215.0	29.3	48,863.4
Day 16	6:00	48,974.0	28,220.5	29.3	48,908.1
Day 16	7:00	49,018.5	28,225.9	29.3	48,952.7
Day 16	8:00	49,062.8	28,231.3	29.3	48,997.2
Day 16	9:00	49,107.0	28,236.8	29.3	49,041.5
Day 16	10:00	49,151.2	28,242.2	29.3	49,085.8
Day 16	11:00	49,195.3	28,247.6	29.3	49,130.0
Day 16	12:00	49,239.4	28,253.0	29.3	49,174.2
Day 16	13:00	49,283.4	28,258.3	29.3	49,218.3
Day 16	14:00	49,327.4	28,263.7	29.3	49,262.4
Day 16	15:00	49,371.4	28,269.1	29.3	49,306.4
Day 16	16:00	49,415.4	28,274.5	29.3	49,350.4
Day 16	17:00	49,459.4	28,279.8	29.3	49,394.4
Day 16	18:00	49,503.4	28,285.2	29.3	49,438.4
Day 16	19:00	49,547.4	28,290.6	29.3	49,482.4
Day 16	20:00	49,591.5	28,296.0	29.3	49,526.4
Day 16	21:00	49,635.6	28,301.3	29.3	49,570.5
Day 16	22:00	49,679.7	28,306.7	29.3	49,614.6
Day 16	23:00	49,723.9	28,312.1	29.3	49,658.7
Day 17	0:00	49,768.1	28,317.5	29.3	49,702.8
Day 17	1:00	49,812.4	28,322.9	29.4	49,747.1
Day 17	2:00	49,856.8	28,328.3	29.4	49,791.3
Day 17	3:00	49,901.2	28,333.7	29.4	49,835.7
Day 17	4:00	49,945.8	28,339.1	29.4	49,880.1
Day 17	5:00	49,990.4	28,344.6	29.4	49,924.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 14 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 17	6:00	50,035.1	28,350.0	29.4	49,969.2
Day 17	7:00	50,079.9	28,355.5	29.4	50,013.8
Day 17	8:00	50,124.8	28,360.9	29.4	50,058.6
Day 17	9:00	50,169.8	28,366.4	29.4	50,103.5
Day 17	10:00	50,214.9	28,371.9	29.4	50,148.4
Day 17	11:00	50,260.1	28,377.4	29.4	50,193.5
Day 17	12:00	50,305.5	28,382.9	29.4	50,238.7
Day 17	13:00	50,350.9	28,388.4	29.4	50,283.9
Day 17	14:00	50,396.5	28,394.0	29.4	50,329.3
Day 17	15:00	50,442.2	28,399.5	29.4	50,374.9
Day 17	16:00	50,488.0	28,405.1	29.4	50,420.5
Day 17	17:00	50,533.9	28,410.7	29.4	50,466.3
Day 17	18:00	50,580.0	28,416.3	29.4	50,512.1
Day 17	19:00	50,626.2	28,421.9	29.4	50,558.2
Day 17	20:00	50,672.5	28,427.5	29.4	50,604.3
Day 17	21:00	50,719.0	28,433.2	29.4	50,650.6
Day 17	22:00	50,765.6	28,438.8	29.4	50,696.9
Day 17	23:00	50,812.3	28,444.5	29.4	50,743.5
Day 18	0:00	50,859.1	28,450.2	29.4	50,790.1
Day 18	1:00	50,906.1	28,455.9	29.4	50,836.9
Day 18	2:00	50,953.2	28,461.7	29.4	50,883.8
Day 18	3:00	51,000.4	28,467.4	29.4	50,931.5
Day 18	4:00	51,047.8	28,473.0	29.4	50,979.4
Day 18	5:00	51,095.3	28,478.7	29.4	51,027.1
Day 18	6:00	51,142.9	28,484.3	29.4	51,074.7
Day 18	7:00	51,190.6	28,490.0	29.4	51,122.3
Day 18	8:00	51,238.5	28,495.6	29.4	51,170.1
Day 18	9:00	51,286.5	28,501.3	29.4	51,217.9
Day 18	10:00	51,334.6	28,507.0	29.4	51,265.8
Day 18	11:00	51,382.8	28,512.6	29.4	51,313.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 15 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 18	12:00	51,431.2	28,518.4	29.4	51,362.1
Day 18	13:00	51,479.6	28,524.1	29.4	51,410.4
Day 18	14:00	51,528.2	28,529.8	29.4	51,458.8
Day 18	15:00	51,576.9	28,535.5	29.4	51,507.3
Day 18	16:00	51,625.7	28,541.3	29.4	51,556.0
Day 18	17:00	51,674.6	28,547.1	29.4	51,604.7
Day 18	18:00	51,723.6	28,552.9	29.4	51,653.6
Day 18	19:00	51,772.7	28,558.7	29.4	51,702.5
Day 18	20:00	51,821.9	28,564.5	29.4	51,751.6
Day 18	21:00	51,871.2	28,570.3	29.4	51,800.7
Day 18	22:00	51,920.6	28,576.1	29.4	51,850.0
Day 18	23:00	51,970.1	28,582.0	29.4	51,899.3
Day 19	0:00	52,019.7	28,587.8	29.4	51,948.8
Day 19	1:00	52,069.3	28,593.7	29.4	51,998.3
Day 19	2:00	52,119.0	28,599.6	29.4	52,047.9
Day 19	3:00	52,168.8	28,605.4	29.4	52,097.6
Day 19	4:00	52,218.7	28,611.3	29.4	52,147.4
Day 19	5:00	52,268.7	28,617.2	29.4	52,197.2
Day 19	6:00	52,318.7	28,623.1	29.4	52,247.1
Day 19	7:00	52,368.7	28,629.1	29.4	52,297.1
Day 19	8:00	52,418.9	28,635.0	29.4	52,347.1
Day 19	9:00	52,469.0	28,640.9	29.4	52,397.2
Day 19	10:00	52,519.3	28,646.8	29.4	52,447.4
Day 19	11:00	52,569.6	28,652.8	29.4	52,497.6
Day 19	12:00	52,619.9	28,658.7	29.4	52,547.8
Day 19	13:00	52,670.2	28,664.7	29.4	52,598.1
Day 19	14:00	52,720.6	28,670.7	29.4	52,648.5
Day 19	15:00	52,771.1	28,676.6	29.4	52,698.9
Day 19	16:00	52,821.5	28,682.6	29.4	52,749.3
Day 19	17:00	52,872.0	28,688.6	29.4	52,799.7

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 16 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 19	18:00	52,922.5	28,694.5	29.4	52,850.2
Day 19	19:00	52,973.0	28,700.5	29.4	52,900.6
Day 19	20:00	53,023.5	28,706.5	29.4	52,951.1
Day 19	21:00	53,074.0	28,712.5	29.4	53,001.7
Day 19	22:00	53,124.6	28,718.5	29.4	53,052.2
Day 19	23:00	53,175.1	28,724.4	29.5	53,102.7
Day 20	0:00	53,225.6	28,730.4	29.5	53,153.2
Day 20	1:00	53,276.2	28,736.4	29.5	53,203.8
Day 20	2:00	53,326.7	28,742.4	29.5	53,254.3
Day 20	3:00	53,377.2	28,748.4	29.5	53,304.8
Day 20	4:00	53,427.6	28,754.3	29.5	53,355.3
Day 20	5:00	53,478.1	28,760.3	29.5	53,405.8
Day 20	6:00	53,528.5	28,766.3	29.5	53,456.3
Day 20	7:00	53,578.9	28,772.3	29.5	53,506.7
Day 20	8:00	53,629.3	28,778.2	29.5	53,557.1
Day 20	9:00	53,679.6	28,784.2	29.5	53,607.5
Day 20	10:00	53,729.9	28,790.2	29.5	53,657.8
Day 20	11:00	53,780.2	28,796.1	29.5	53,708.1
Day 20	12:00	53,830.4	28,802.1	29.5	53,758.4
Day 20	13:00	53,880.5	28,808.0	29.5	53,808.6
Day 20	14:00	53,930.6	28,814.0	29.5	53,858.8
Day 20	15:00	53,980.6	28,819.9	29.5	53,908.9
Day 20	16:00	54,030.6	28,825.8	29.5	53,958.9
Day 20	17:00	54,080.5	28,831.7	29.5	54,008.9
Day 20	18:00	54,130.3	28,837.6	29.5	54,058.8
Day 20	19:00	54,180.1	28,843.5	29.5	54,108.7
Day 20	20:00	54,229.8	28,849.4	29.5	54,158.5
Day 20	21:00	54,279.4	28,855.3	29.5	54,208.2
Day 20	22:00	54,328.9	28,861.2	29.5	54,257.9
Day 20	23:00	54,378.4	28,867.1	29.5	54,307.4

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 17 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 21	0:00	54,427.7	28,872.9	29.5	54,356.9
Day 21	1:00	54,477.0	28,878.8	29.5	54,406.3
Day 21	2:00	54,526.2	28,884.6	29.5	54,455.6
Day 21	3:00	54,575.3	28,890.4	29.5	54,504.8
Day 21	4:00	54,624.3	28,896.3	29.5	54,554.0
Day 21	5:00	54,673.2	28,902.1	29.5	54,603.0
Day 21	6:00	54,721.9	28,907.9	29.5	54,651.9
Day 21	7:00	54,770.6	28,913.6	29.5	54,700.7
Day 21	8:00	54,819.2	28,919.4	29.5	54,749.5
Day 21	9:00	54,867.7	28,925.2	29.5	54,798.1
Day 21	10:00	54,916.0	28,930.9	29.5	54,846.6
Day 21	11:00	54,964.2	28,936.6	29.5	54,895.0
Day 21	12:00	55,012.4	28,942.4	29.5	54,943.3
Day 21	13:00	55,060.4	28,948.1	29.5	54,991.4
Day 21	14:00	55,108.2	28,953.7	29.5	55,039.5
Day 21	15:00	55,156.0	28,959.4	29.5	55,087.4
Day 21	16:00	55,203.6	28,965.1	29.5	55,135.2
Day 21	17:00	55,251.1	28,970.7	29.5	55,182.9
Day 21	18:00	55,298.5	28,976.4	29.5	55,230.5
Day 21	19:00	55,345.7	28,982.0	29.5	55,277.9
Day 21	20:00	55,392.9	28,987.6	29.5	55,325.2
Day 21	21:00	55,439.8	28,993.2	29.5	55,372.4
Day 21	22:00	55,486.7	28,998.7	29.5	55,419.4
Day 21	23:00	55,533.4	29,004.3	29.5	55,466.3
Day 22	0:00	55,579.9	29,009.8	29.5	55,513.0
Day 22	1:00	55,626.4	29,015.3	29.5	55,559.7
Day 22	2:00	55,672.6	29,020.8	29.5	55,606.1
Day 22	3:00	55,718.8	29,026.3	29.5	55,652.5
Day 22	4:00	55,764.8	29,031.8	29.5	55,698.7
Day 22	5:00	55,810.6	29,037.2	29.5	55,744.7

LNP COL 2.4-2

**Table 2.4.3-226 (Sheet 18 of 48)
Water Elevation in Lake Rousseau Resulting from PMF**

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 22	6:00	55,856.3	29,042.7	29.5	55,790.6
Day 22	7:00	55,901.9	29,048.1	29.5	55,836.4
Day 22	8:00	55,947.3	29,053.5	29.5	55,882.0
Day 22	9:00	55,992.5	29,058.9	29.5	55,927.5
Day 22	10:00	56,037.6	29,064.2	29.5	55,972.8
Day 22	11:00	56,082.6	29,069.6	29.5	56,018.0
Day 22	12:00	56,127.3	29,074.9	29.5	56,063.0
Day 22	13:00	56,172.0	29,080.2	29.5	56,107.8
Day 22	14:00	56,216.5	29,085.5	29.5	56,152.5
Day 22	15:00	56,260.8	29,090.8	29.5	56,197.1
Day 22	16:00	56,305.0	29,096.1	29.5	56,241.5
Day 22	17:00	56,349.0	29,101.3	29.5	56,285.7
Day 22	18:00	56,392.8	29,106.5	29.5	56,329.8
Day 22	19:00	56,436.5	29,111.7	29.5	56,373.7
Day 22	20:00	56,480.1	29,116.9	29.5	56,417.5
Day 22	21:00	56,523.4	29,122.1	29.5	56,461.1
Day 22	22:00	56,566.7	29,127.2	29.6	56,504.5
Day 22	23:00	56,609.7	29,132.3	29.6	56,547.8
Day 23	0:00	56,652.6	29,137.4	29.6	56,590.9
Day 23	1:00	56,695.3	29,142.5	29.6	56,633.9
Day 23	2:00	56,737.9	29,147.6	29.6	56,676.7
Day 23	3:00	56,780.3	29,152.6	29.6	56,719.3
Day 23	4:00	56,822.6	29,157.7	29.6	56,761.8
Day 23	5:00	56,864.6	29,162.7	29.6	56,804.1
Day 23	6:00	56,906.6	29,167.7	29.6	56,846.3
Day 23	7:00	56,948.3	29,172.6	29.6	56,888.3
Day 23	8:00	56,989.9	29,177.6	29.6	56,930.1
Day 23	9:00	57,031.3	29,182.5	29.6	56,971.7
Day 23	10:00	57,072.6	29,187.4	29.6	57,013.2
Day 23	11:00	57,113.7	29,192.3	29.6	57,054.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 19 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 23	12:00	57,154.6	29,197.2	29.6	57,095.7
Day 23	13:00	57,195.3	29,202.0	29.6	57,136.7
Day 23	14:00	57,235.9	29,206.9	29.6	57,177.6
Day 23	15:00	57,276.4	29,211.7	29.6	57,218.2
Day 23	16:00	57,316.6	29,216.5	29.6	57,258.7
Day 23	17:00	57,356.7	29,221.3	29.6	57,299.1
Day 23	18:00	57,396.6	29,226.0	29.6	57,339.2
Day 23	19:00	57,436.4	29,230.8	29.6	57,379.2
Day 23	20:00	57,476.0	29,235.5	29.6	57,419.0
Day 23	21:00	57,515.4	29,240.2	29.6	57,458.7
Day 23	22:00	57,554.7	29,244.8	29.6	57,498.2
Day 23	23:00	57,593.8	29,249.5	29.6	57,537.5
Day 24	0:00	57,632.7	29,254.1	29.6	57,576.7
Day 24	1:00	57,671.4	29,258.8	29.6	57,615.7
Day 24	2:00	57,710.0	29,263.4	29.6	57,654.5
Day 24	3:00	57,748.4	29,267.9	29.6	57,693.2
Day 24	4:00	57,786.7	29,272.5	29.6	57,731.7
Day 24	5:00	57,824.7	29,277.0	29.6	57,770.7
Day 24	6:00	57,862.6	29,281.4	29.6	57,809.2
Day 24	7:00	57,900.4	29,285.8	29.6	57,847.4
Day 24	8:00	57,937.9	29,290.2	29.6	57,885.3
Day 24	9:00	57,975.3	29,294.5	29.6	57,923.0
Day 24	10:00	58,012.6	29,298.9	29.6	57,960.5
Day 24	11:00	58,049.6	29,303.1	29.6	57,997.8
Day 24	12:00	58,086.5	29,307.4	29.6	58,034.9
Day 24	13:00	58,123.2	29,311.7	29.6	58,071.8
Day 24	14:00	58,159.7	29,315.9	29.6	58,108.6
Day 24	15:00	58,196.1	29,320.1	29.6	58,145.2
Day 24	16:00	58,232.3	29,324.3	29.6	58,181.6
Day 24	17:00	58,268.3	29,328.5	29.6	58,217.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 20 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 24	18:00	58,304.1	29,332.7	29.6	58,254.0
Day 24	19:00	58,339.8	29,336.8	29.6	58,289.9
Day 24	20:00	58,375.3	29,340.9	29.6	58,325.6
Day 24	21:00	58,410.6	29,345.0	29.6	58,361.2
Day 24	22:00	58,445.7	29,349.1	29.6	58,396.5
Day 24	23:00	58,480.7	29,353.1	29.6	58,431.7
Day 25	0:00	58,515.4	29,357.2	29.6	58,466.8
Day 25	1:00	58,550.0	29,361.2	29.6	58,501.6
Day 25	2:00	58,584.5	29,365.2	29.6	58,536.3
Day 25	3:00	58,618.7	29,369.1	29.6	58,570.8
Day 25	4:00	58,652.7	29,373.1	29.6	58,605.1
Day 25	5:00	58,686.6	29,377.0	29.6	58,639.2
Day 25	6:00	58,720.3	29,380.9	29.6	58,673.1
Day 25	7:00	58,753.8	29,384.8	29.6	58,706.9
Day 25	8:00	58,787.1	29,388.7	29.6	58,740.4
Day 25	9:00	58,820.2	29,392.5	29.6	58,773.8
Day 25	10:00	58,853.1	29,396.4	29.6	58,807.0
Day 25	11:00	58,885.9	29,400.2	29.6	58,840.0
Day 25	12:00	58,918.4	29,403.9	29.6	58,872.8
Day 25	13:00	58,950.8	29,407.7	29.6	58,905.5
Day 25	14:00	58,983.0	29,411.4	29.6	58,937.9
Day 25	15:00	59,014.9	29,415.1	29.6	58,970.1
Day 25	16:00	59,046.7	29,418.8	29.6	59,002.2
Day 25	17:00	59,078.3	29,422.5	29.6	59,034.0
Day 25	18:00	59,109.6	29,426.2	29.6	59,065.7
Day 25	19:00	59,140.8	29,429.8	29.6	59,097.1
Day 25	20:00	59,171.8	29,433.4	29.6	59,128.4
Day 25	21:00	59,202.5	29,436.9	29.6	59,159.4
Day 25	22:00	59,233.1	29,440.5	29.6	59,190.2
Day 25	23:00	59,263.4	29,444.0	29.6	59,220.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 21 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 26	0:00	59,293.5	29,447.5	29.6	59,251.3
Day 26	1:00	59,323.5	29,451.0	29.6	59,281.5
Day 26	2:00	59,353.2	29,454.5	29.6	59,311.5
Day 26	3:00	59,382.6	29,457.9	29.6	59,341.3
Day 26	4:00	59,411.9	29,461.3	29.6	59,370.9
Day 26	5:00	59,441.0	29,464.7	29.6	59,400.2
Day 26	6:00	59,469.8	29,468.0	29.6	59,429.3
Day 26	7:00	59,498.4	29,471.4	29.6	59,458.2
Day 26	8:00	59,526.7	29,474.7	29.6	59,486.9
Day 26	9:00	59,554.9	29,477.9	29.6	59,515.4
Day 26	10:00	59,582.8	29,481.2	29.6	59,543.6
Day 26	11:00	59,610.4	29,484.4	29.6	59,571.6
Day 26	12:00	59,637.8	29,487.6	29.6	59,599.3
Day 26	13:00	59,665.0	29,490.8	29.6	59,626.8
Day 26	14:00	59,692.0	29,493.9	29.6	59,654.1
Day 26	15:00	59,718.7	29,497.0	29.6	59,681.2
Day 26	16:00	59,745.1	29,500.1	29.6	59,707.9
Day 26	17:00	59,771.3	29,503.2	29.6	59,734.5
Day 26	18:00	59,797.2	29,506.2	29.6	59,760.8
Day 26	19:00	59,822.9	29,509.2	29.6	59,786.8
Day 26	20:00	59,848.3	29,512.2	29.6	59,812.6
Day 26	21:00	59,873.5	29,515.1	29.6	59,838.1
Day 26	22:00	59,898.3	29,518.0	29.6	59,863.3
Day 26	23:00	59,922.9	29,520.9	29.6	59,888.3
Day 27	0:00	59,947.3	29,523.8	29.6	59,913.0
Day 27	1:00	59,971.3	29,526.6	29.6	59,937.5
Day 27	2:00	59,995.1	29,529.3	29.6	59,961.6
Day 27	3:00	60,018.6	29,532.1	29.6	59,985.5
Day 27	4:00	60,041.8	29,534.8	29.7	60,009.1
Day 27	5:00	60,064.7	29,537.5	29.7	60,032.4

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 22 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 27	6:00	60,087.3	29,540.2	29.7	60,055.5
Day 27	7:00	60,109.7	29,542.8	29.7	60,078.2
Day 27	8:00	60,131.7	29,545.4	29.7	60,100.6
Day 27	9:00	60,153.4	29,547.9	29.7	60,122.8
Day 27	10:00	60,174.8	29,550.4	29.7	60,144.6
Day 27	11:00	60,195.9	29,552.9	29.7	60,166.1
Day 27	12:00	60,216.7	29,555.3	29.7	60,187.3
Day 27	13:00	60,237.1	29,557.8	29.7	60,208.2
Day 27	14:00	60,257.2	29,560.1	29.7	60,228.8
Day 27	15:00	60,277.0	29,562.5	29.7	60,249.0
Day 27	16:00	60,296.5	29,564.7	29.7	60,268.9
Day 27	17:00	60,315.6	29,567.0	29.7	60,288.5
Day 27	18:00	60,334.4	29,569.2	29.7	60,307.8
Day 27	19:00	60,352.8	29,571.4	29.7	60,326.7
Day 27	20:00	60,370.9	29,573.5	29.7	60,345.3
Day 27	21:00	60,388.6	29,575.6	29.7	60,363.5
Day 27	22:00	60,406.0	29,577.7	29.7	60,381.4
Day 27	23:00	60,423.0	29,579.7	29.7	60,398.9
Day 28	0:00	60,439.7	29,581.7	29.7	60,416.0
Day 28	1:00	60,455.9	29,583.6	29.7	60,432.8
Day 28	2:00	60,471.8	29,585.5	29.7	60,449.2
Day 28	3:00	60,487.3	29,587.4	29.7	60,465.3
Day 28	4:00	60,502.5	29,589.2	29.7	60,480.9
Day 28	5:00	60,517.2	29,590.9	29.7	60,496.2
Day 28	6:00	60,531.6	29,592.6	29.7	60,511.1
Day 28	7:00	60,545.5	29,594.3	29.7	60,525.6
Day 28	8:00	60,559.0	29,595.9	29.7	60,539.7
Day 28	9:00	60,572.2	29,597.5	29.7	60,553.4
Day 28	10:00	60,584.9	29,599.0	29.7	60,566.7
Day 28	11:00	60,597.2	29,600.5	29.7	60,579.5

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 23 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 28	12:00	60,620.5	29,603.3	29.7	60,604.1
Day 28	13:00	60,631.6	29,604.7	29.7	60,615.7
Day 28	14:00	60,642.2	29,606.0	29.7	60,626.9
Day 28	15:00	60,652.3	29,607.2	29.7	60,637.6
Day 28	16:00	60,662.0	29,608.4	29.7	60,647.9
Day 28	17:00	60,671.3	29,609.5	29.7	60,657.8
Day 28	18:00	60,680.0	29,610.6	29.7	60,667.2
Day 28	19:00	60,688.4	29,611.7	29.7	60,676.2
Day 28	20:00	60,696.2	29,612.6	29.7	60,684.7
Day 28	21:00	60,703.6	29,613.6	29.7	60,692.8
Day 28	22:00	60,710.6	29,614.4	29.7	60,700.3
Day 28	23:00	60,717.0	29,615.3	29.7	60,707.4
Day 29	0:00	60,723.0	29,616.0	29.7	60,714.1
Day 29	1:00	60,728.4	29,616.7	29.7	60,720.2
Day 29	2:00	60,733.4	29,617.4	29.7	60,725.9
Day 29	3:00	60,737.9	29,618.0	29.7	60,731.0
Day 29	4:00	60,741.8	29,618.5	29.7	60,735.7
Day 29	5:00	60,745.3	29,619.0	29.7	60,739.9
Day 29	6:00	60,748.3	29,619.4	29.7	60,743.5
Day 29	7:00	60,750.7	29,619.8	29.7	60,746.7
Day 29	8:00	60,752.6	29,620.1	29.7	60,749.3
Day 29	9:00	60,753.9	29,620.3	29.7	60,751.4
Day 29	10:00	60,754.8	29,620.5	29.7	60,753.0
Day 29	11:00	60,755.1	29,620.6	29.7	60,754.0
Day 29	12:00	60,754.8	29,620.7	29.7	60,754.5
Day 29	13:00	60,754.0	29,620.7	29.7	60,754.5
Day 29	14:00	60,752.7	29,620.6	29.7	60,753.9
Day 29	15:00	60,750.8	29,620.5	29.7	60,752.7
Day 29	16:00	60,748.3	29,620.3	29.7	60,751.1
Day 29	17:00	60,620.5	29,603.3	29.7	60,604.1

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 24 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 29	18:00	60,745.3	29,620.0	29.7	60,748.8
Day 29	19:00	60,741.7	29,619.7	29.7	60,746.0
Day 29	20:00	60,737.5	29,619.3	29.7	60,742.6
Day 29	21:00	60,732.7	29,618.8	29.7	60,738.6
Day 29	22:00	60,727.3	29,618.3	29.7	60,734.1
Day 29	23:00	60,721.4	29,617.7	29.7	60,729.0
Day 30	0:00	60,714.8	29,617.1	29.7	60,723.2
Day 30	1:00	60,707.7	29,616.3	29.7	60,716.9
Day 30	2:00	60,699.9	29,615.5	29.7	60,710.0
Day 30	3:00	60,691.6	29,614.7	29.7	60,702.5
Day 30	4:00	60,682.6	29,613.7	29.7	60,694.3
Day 30	5:00	60,673.0	29,612.7	29.7	60,685.6
Day 30	6:00	60,662.8	29,611.7	29.7	60,676.2
Day 30	7:00	60,651.9	29,610.5	29.7	60,666.3
Day 30	8:00	60,640.4	29,609.3	29.7	60,655.7
Day 30	9:00	60,628.3	29,608.0	29.7	60,644.4
Day 30	10:00	60,615.5	29,606.6	29.7	60,632.5
Day 30	11:00	60,602.1	29,605.2	29.7	60,620.0
Day 30	12:00	60,588.1	29,603.7	29.7	60,606.9
Day 30	13:00	60,573.4	29,602.1	29.7	60,593.1
Day 30	14:00	60,558.0	29,600.4	29.7	60,578.6
Day 30	15:00	60,541.9	29,598.7	29.7	60,563.5
Day 30	16:00	60,525.2	29,596.9	29.7	60,547.7
Day 30	17:00	60,507.9	29,595.0	29.7	60,531.2
Day 30	18:00	60,489.8	29,593.0	29.7	60,514.1
Day 30	19:00	60,471.1	29,590.9	29.7	60,496.3
Day 30	20:00	60,451.7	29,588.8	29.7	60,477.9
Day 30	21:00	60,431.6	29,586.6	29.7	60,458.7
Day 30	22:00	60,410.8	29,584.3	29.7	60,438.9
Day 30	23:00	60,389.3	29,582.0	29.7	60,418.4

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 25 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 31	0:00	60,367.1	29,579.5	29.7	60,397.1
Day 31	1:00	60,344.2	29,577.0	29.7	60,375.2
Day 31	2:00	60,320.6	29,574.4	29.7	60,352.6
Day 31	3:00	60,296.3	29,571.7	29.7	60,329.3
Day 31	4:00	60,271.3	29,568.9	29.7	60,305.3
Day 31	5:00	60,245.6	29,566.1	29.7	60,280.5
Day 31	6:00	60,219.1	29,563.1	29.7	60,255.1
Day 31	7:00	60,191.9	29,560.1	29.7	60,228.9
Day 31	8:00	60,164.0	29,557.0	29.7	60,202.0
Day 31	9:00	60,135.4	29,553.9	29.7	60,174.4
Day 31	10:00	60,106.0	29,550.6	29.7	60,146.0
Day 31	11:00	60,075.9	29,547.2	29.7	60,117.0
Day 31	12:00	60,045.1	29,543.8	29.7	60,087.1
Day 31	13:00	60,013.5	29,540.3	29.7	60,056.6
Day 31	14:00	59,981.2	29,536.7	29.7	60,025.3
Day 31	15:00	59,948.1	29,533.0	29.6	59,993.3
Day 31	16:00	59,914.3	29,529.2	29.6	59,960.5
Day 31	17:00	59,879.7	29,525.4	29.6	59,927.0
Day 31	18:00	59,844.4	29,521.4	29.6	59,892.7
Day 31	19:00	59,808.3	29,517.4	29.6	59,857.6
Day 31	20:00	59,771.4	29,513.2	29.6	59,821.8
Day 31	21:00	59,733.8	29,509.0	29.6	59,785.3
Day 31	22:00	59,695.4	29,504.7	29.6	59,748.0
Day 31	23:00	59,656.3	29,500.4	29.6	59,709.9
Day 32	0:00	59,616.4	29,495.9	29.6	59,671.0
Day 32	1:00	59,575.7	29,491.3	29.6	59,631.4
Day 32	2:00	59,534.2	29,486.7	29.6	59,591.0
Day 32	3:00	59,492.0	29,481.9	29.6	59,549.9
Day 32	4:00	59,449.0	29,477.1	29.6	59,507.9
Day 32	5:00	59,405.2	29,472.2	29.6	59,465.2

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 26 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 32	6:00	59,360.6	29,467.2	29.6	59,421.7
Day 32	7:00	59,315.2	29,462.1	29.6	59,377.5
Day 32	8:00	59,269.1	29,456.9	29.6	59,332.4
Day 32	9:00	59,222.1	29,451.6	29.6	59,286.6
Day 32	10:00	59,174.4	29,446.2	29.6	59,239.9
Day 32	11:00	59,125.9	29,440.8	29.6	59,192.5
Day 32	12:00	59,076.6	29,435.2	29.6	59,144.3
Day 32	13:00	59,026.5	29,429.6	29.6	59,095.3
Day 32	14:00	58,975.6	29,423.8	29.6	59,045.6
Day 32	15:00	58,924.0	29,418.0	29.6	58,995.0
Day 32	16:00	58,871.5	29,412.1	29.6	58,943.6
Day 32	17:00	58,818.2	29,406.1	29.6	58,891.5
Day 32	18:00	58,764.2	29,400.0	29.6	58,838.5
Day 32	19:00	58,709.3	29,393.8	29.6	58,784.8
Day 32	20:00	58,653.6	29,387.5	29.6	58,730.2
Day 32	21:00	58,597.2	29,381.1	29.6	58,674.9
Day 32	22:00	58,539.9	29,374.7	29.6	58,618.7
Day 32	23:00	58,481.9	29,368.1	29.6	58,561.8
Day 33	0:00	58,423.0	29,361.5	29.6	58,504.0
Day 33	1:00	58,363.4	29,354.7	29.6	58,445.5
Day 33	2:00	58,302.9	29,347.9	29.6	58,386.2
Day 33	3:00	58,241.6	29,341.0	29.6	58,326.0
Day 33	4:00	58,179.6	29,333.9	29.6	58,265.1
Day 33	5:00	58,116.7	29,326.8	29.6	58,203.3
Day 33	6:00	58,053.1	29,319.6	29.6	58,140.8
Day 33	7:00	57,988.6	29,312.3	29.6	58,077.5
Day 33	8:00	57,923.4	29,304.9	29.6	58,013.3
Day 33	9:00	57,857.3	29,297.5	29.6	57,948.4
Day 33	10:00	57,790.5	29,289.9	29.6	57,882.7
Day 33	11:00	57,722.8	29,282.2	29.6	57,816.1

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 27 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 33	12:00	57,654.4	29,274.5	29.6	57,748.8
Day 33	13:00	57,585.2	29,266.6	29.6	57,681.8
Day 33	14:00	57,515.1	29,258.5	29.6	57,613.7
Day 33	15:00	57,444.3	29,250.3	29.6	57,544.4
Day 33	16:00	57,372.7	29,242.0	29.6	57,474.1
Day 33	17:00	57,300.3	29,233.6	29.6	57,402.9
Day 33	18:00	57,227.1	29,225.0	29.6	57,330.8
Day 33	19:00	57,153.1	29,216.4	29.6	57,258.0
Day 33	20:00	57,078.3	29,207.7	29.6	57,184.4
Day 33	21:00	57,002.7	29,198.9	29.6	57,109.9
Day 33	22:00	56,926.4	29,190.0	29.6	57,034.7
Day 33	23:00	56,849.2	29,181.0	29.6	56,958.7
Day 34	0:00	56,771.3	29,171.9	29.6	56,881.9
Day 34	1:00	56,692.6	29,162.7	29.6	56,804.3
Day 34	2:00	56,613.1	29,153.4	29.6	56,725.9
Day 34	3:00	56,532.8	29,144.0	29.6	56,646.8
Day 34	4:00	56,451.8	29,134.6	29.6	56,566.9
Day 34	5:00	56,370.0	29,125.0	29.5	56,486.2
Day 34	6:00	56,287.4	29,115.4	29.5	56,404.7
Day 34	7:00	56,204.1	29,105.6	29.5	56,322.5
Day 34	8:00	56,120.0	29,095.8	29.5	56,239.4
Day 34	9:00	56,035.1	29,085.9	29.5	56,155.7
Day 34	10:00	55,949.4	29,075.9	29.5	56,071.1
Day 34	11:00	55,863.0	29,065.8	29.5	55,985.8
Day 34	12:00	55,775.9	29,055.6	29.5	55,899.7
Day 34	13:00	55,688.0	29,045.3	29.5	55,812.9
Day 34	14:00	55,599.3	29,034.9	29.5	55,725.3
Day 34	15:00	55,509.9	29,024.5	29.5	55,637.0
Day 34	16:00	55,419.7	29,013.9	29.5	55,547.9
Day 34	17:00	55,328.8	29,003.3	29.5	55,458.1

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 28 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 34	18:00	55,237.2	28,992.6	29.5	55,367.5
Day 34	19:00	55,144.8	28,981.8	29.5	55,276.2
Day 34	20:00	55,051.7	28,970.9	29.5	55,184.1
Day 34	21:00	54,957.9	28,959.9	29.5	55,091.3
Day 34	22:00	54,863.3	28,948.8	29.5	54,997.8
Day 34	23:00	54,768.0	28,937.6	29.5	54,903.5
Day 35	0:00	54,672.0	28,926.4	29.5	54,808.6
Day 35	1:00	54,575.3	28,915.1	29.5	54,712.9
Day 35	2:00	54,477.8	28,903.7	29.5	54,616.5
Day 35	3:00	54,379.7	28,892.2	29.5	54,519.3
Day 35	4:00	54,280.8	28,880.6	29.5	54,421.5
Day 35	5:00	54,181.2	28,868.9	29.5	54,322.9
Day 35	6:00	54,081.0	28,857.2	29.5	54,223.7
Day 35	7:00	53,980.0	28,845.3	29.5	54,123.7
Day 35	8:00	53,878.4	28,833.4	29.5	54,023.0
Day 35	9:00	53,776.0	28,821.4	29.5	53,921.7
Day 35	10:00	53,673.0	28,809.3	29.5	53,819.7
Day 35	11:00	53,569.3	28,797.2	29.5	53,716.9
Day 35	12:00	53,464.9	28,784.9	29.5	53,613.5
Day 35	13:00	53,359.9	28,772.6	29.5	53,509.5
Day 35	14:00	53,254.2	28,760.2	29.5	53,404.7
Day 35	15:00	53,147.8	28,747.7	29.5	53,299.3
Day 35	16:00	53,040.8	28,735.2	29.5	53,193.2
Day 35	17:00	52,933.1	28,722.5	29.5	53,086.5
Day 35	18:00	52,824.7	28,709.8	29.4	52,979.1
Day 35	19:00	52,715.8	28,697.0	29.4	52,871.0
Day 35	20:00	52,606.1	28,684.1	29.4	52,762.3
Day 35	21:00	52,495.9	28,671.2	29.4	52,653.0
Day 35	22:00	52,385.0	28,658.2	29.4	52,543.0
Day 35	23:00	52,273.5	28,645.1	29.4	52,432.4

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LNP COL 2.4-2

Table 2.4.3-226 (Sheet 29 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 36	0:00	52,161.4	28,631.9	29.4	52,321.2
Day 36	1:00	52,048.6	28,618.7	29.4	52,209.3
Day 36	2:00	51,935.3	28,605.3	29.4	52,096.8
Day 36	3:00	51,821.3	28,592.0	29.4	51,983.7
Day 36	4:00	51,706.7	28,578.5	29.4	51,870.0
Day 36	5:00	51,591.6	28,565.0	29.4	51,755.7
Day 36	6:00	51,475.8	28,551.4	29.4	51,640.8
Day 36	7:00	51,359.5	28,537.7	29.4	51,525.3
Day 36	8:00	51,242.6	28,523.9	29.4	51,409.3
Day 36	9:00	51,125.1	28,510.1	29.4	51,292.6
Day 36	10:00	51,007.0	28,496.2	29.4	51,175.4
Day 36	11:00	50,888.4	28,482.3	29.4	51,057.6
Day 36	12:00	50,769.2	28,468.3	29.4	50,939.2
Day 36	13:00	50,649.4	28,454.1	29.4	50,822.1
Day 36	14:00	50,529.1	28,439.7	29.4	50,704.3
Day 36	15:00	50,408.3	28,425.2	29.4	50,585.2
Day 36	16:00	50,286.9	28,410.5	29.4	50,465.0
Day 36	17:00	50,165.0	28,395.8	29.4	50,344.1
Day 36	18:00	50,042.6	28,380.9	29.4	50,222.6
Day 36	19:00	49,919.6	28,366.0	29.4	50,100.4
Day 36	20:00	49,796.2	28,351.0	29.4	49,977.8
Day 36	21:00	49,672.2	28,336.0	29.4	49,854.6
Day 36	22:00	49,547.7	28,320.9	29.4	49,730.8
Day 36	23:00	49,422.7	28,305.7	29.3	49,606.6
Day 37	0:00	49,297.2	28,290.5	29.3	49,481.9
Day 37	1:00	49,171.3	28,275.2	29.3	49,356.6
Day 37	2:00	49,044.8	28,259.9	29.3	49,230.9
Day 37	3:00	48,917.9	28,244.5	29.3	49,104.7
Day 37	4:00	48,790.5	28,229.0	29.3	48,978.0
Day 37	5:00	48,662.6	28,213.5	29.3	48,850.8

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 30 of 48) Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 37	6:00	48,534.3	28,197.9	29.3	48,723.2
Day 37	7:00	48,405.6	28,182.3	29.3	48,595.1
Day 37	8:00	48,276.4	28,166.6	29.3	48,466.6
Day 37	9:00	48,146.7	28,150.8	29.3	48,337.6
Day 37	10:00	48,016.6	28,135.0	29.3	48,208.1
Day 37	11:00	47,886.1	28,119.2	29.3	48,078.3
Day 37	12:00	47,755.2	28,103.3	29.3	47,948.0
Day 37	13:00	47,623.8	28,087.3	29.3	47,817.2
Day 37	14:00	47,492.0	28,071.3	29.3	47,686.1
Day 37	15:00	47,359.9	28,055.2	29.3	47,554.5
Day 37	16:00	47,227.3	28,039.1	29.3	47,422.5
Day 37	17:00	47,094.3	28,023.0	29.3	47,290.2
Day 37	18:00	46,961.0	28,006.7	29.3	47,157.4
Day 37	19:00	46,827.3	27,990.5	29.3	47,024.2
Day 37	20:00	46,693.2	27,974.2	29.3	46,890.7
Day 37	21:00	46,558.7	27,957.8	29.3	46,756.8
Day 37	22:00	46,423.9	27,941.4	29.3	46,622.5
Day 37	23:00	46,288.7	27,925.0	29.3	46,487.9
Day 38	0:00	46,153.2	27,908.5	29.2	46,352.8
Day 38	1:00	46,017.3	27,892.0	29.2	46,217.5
Day 38	2:00	45,881.1	27,875.4	29.2	46,081.8
Day 38	3:00	45,744.5	27,858.8	29.2	45,945.7
Day 38	4:00	45,607.6	27,842.2	29.2	45,809.3
Day 38	5:00	45,470.5	27,825.5	29.2	45,672.6
Day 38	6:00	45,333.0	27,808.8	29.2	45,535.6
Day 38	7:00	45,195.2	27,792.0	29.2	45,398.3
Day 38	8:00	45,057.1	27,775.2	29.2	45,260.6
Day 38	9:00	44,918.7	27,758.3	29.2	45,122.7
Day 38	10:00	44,780.0	27,741.5	29.2	44,984.4
Day 38	11:00	44,641.0	27,724.6	29.2	44,845.9

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 31 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 38	12:00	44,501.8	27,707.6	29.2	44,707.0
Day 38	13:00	44,362.3	27,690.6	29.2	44,567.9
Day 38	14:00	44,222.5	27,673.6	29.2	44,428.6
Day 38	15:00	44,082.5	27,656.4	29.2	44,291.7
Day 38	16:00	43,942.2	27,639.1	29.2	44,153.9
Day 38	17:00	43,801.7	27,621.5	29.2	44,014.8
Day 38	18:00	43,660.9	27,603.8	29.2	43,875.0
Day 38	19:00	43,519.9	27,586.1	29.2	43,734.6
Day 38	20:00	43,378.7	27,568.4	29.2	43,593.9
Day 38	21:00	43,237.3	27,550.6	29.2	43,452.9
Day 38	22:00	43,095.6	27,532.7	29.2	43,311.6
Day 38	23:00	42,953.8	27,514.9	29.1	43,170.1
Day 39	0:00	42,811.7	27,497.0	29.1	43,028.4
Day 39	1:00	42,669.5	27,479.1	29.1	42,886.4
Day 39	2:00	42,527.0	27,461.1	29.1	42,744.3
Day 39	3:00	42,384.4	27,443.1	29.1	42,601.9
Day 39	4:00	42,241.6	27,425.2	29.1	42,459.4
Day 39	5:00	42,098.6	27,407.1	29.1	42,316.7
Day 39	6:00	41,955.5	27,389.1	29.1	42,173.9
Day 39	7:00	41,812.2	27,371.0	29.1	42,030.8
Day 39	8:00	41,668.8	27,353.0	29.1	41,887.6
Day 39	9:00	41,525.2	27,334.9	29.1	41,744.3
Day 39	10:00	41,381.5	27,316.8	29.1	41,600.8
Day 39	11:00	41,237.6	27,298.6	29.1	41,457.1
Day 39	12:00	41,093.6	27,280.5	29.1	41,313.3
Day 39	13:00	40,949.5	27,262.3	29.1	41,169.4
Day 39	14:00	40,805.3	27,244.1	29.1	41,025.4
Day 39	15:00	40,660.9	27,225.9	29.1	40,881.2
Day 39	16:00	40,516.5	27,207.7	29.1	40,737.0
Day 39	17:00	40,372.0	27,189.5	29.1	40,592.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 32 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 39	18:00	40,227.3	27,171.2	29.1	40,448.1
Day 39	19:00	40,082.6	27,153.0	29.1	40,303.5
Day 39	20:00	39,937.8	27,134.7	29.0	40,158.9
Day 39	21:00	39,793.0	27,116.5	29.0	40,014.1
Day 39	22:00	39,648.0	27,098.2	29.0	39,869.3
Day 39	23:00	39,503.0	27,079.9	29.0	39,724.4
Day 40	0:00	39,357.9	27,061.6	29.0	39,579.4
Day 40	1:00	39,212.8	27,043.3	29.0	39,434.4
Day 40	2:00	39,067.7	27,025.0	29.0	39,289.3
Day 40	3:00	38,922.5	27,006.6	29.0	39,144.2
Day 40	4:00	38,777.2	26,988.3	29.0	38,999.0
Day 40	5:00	38,632.0	26,970.0	29.0	38,853.8
Day 40	6:00	38,486.7	26,951.7	29.0	38,708.6
Day 40	7:00	38,341.4	26,933.3	29.0	38,563.3
Day 40	8:00	38,196.1	26,915.0	29.0	38,418.0
Day 40	9:00	38,050.7	26,896.6	29.0	38,272.7
Day 40	10:00	37,905.4	26,878.2	29.0	38,130.7
Day 40	11:00	37,760.1	26,859.4	29.0	37,988.2
Day 40	12:00	37,614.8	26,840.5	29.0	37,844.3
Day 40	13:00	37,469.5	26,821.5	29.0	37,699.8
Day 40	14:00	37,324.2	26,802.5	29.0	37,554.9
Day 40	15:00	37,179.0	26,783.4	29.0	37,409.8
Day 40	16:00	37,033.7	26,764.3	29.0	37,264.7
Day 40	17:00	36,888.6	26,745.2	28.9	37,119.5
Day 40	18:00	36,743.4	26,726.1	28.9	36,974.3
Day 40	19:00	36,598.3	26,707.1	28.9	36,829.2
Day 40	20:00	36,453.3	26,688.0	28.9	36,684.1
Day 40	21:00	36,308.3	26,668.9	28.9	36,539.0
Day 40	22:00	36,163.4	26,649.9	28.9	36,394.0
Day 40	23:00	36,018.5	26,630.8	28.9	36,249.0

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 33 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 41	0:00	35,873.7	26,611.8	28.9	36,104.1
Day 41	1:00	35,729.0	26,592.7	28.9	35,959.3
Day 41	2:00	35,584.4	26,573.7	28.9	35,814.6
Day 41	3:00	35,439.9	26,554.7	28.9	35,669.9
Day 41	4:00	35,295.4	26,535.7	28.9	35,525.3
Day 41	5:00	35,151.1	26,516.7	28.9	35,380.8
Day 41	6:00	35,006.8	26,497.7	28.9	35,236.4
Day 41	7:00	34,862.7	26,478.7	28.9	35,092.1
Day 41	8:00	34,718.7	26,459.8	28.9	34,947.9
Day 41	9:00	34,574.7	26,440.8	28.9	34,803.8
Day 41	10:00	34,431.0	26,421.9	28.9	34,659.9
Day 41	11:00	34,287.3	26,403.0	28.9	34,516.0
Day 41	12:00	34,143.8	26,384.1	28.9	34,372.3
Day 41	13:00	34,000.4	26,365.2	28.8	34,228.7
Day 41	14:00	33,857.1	26,346.4	28.8	34,085.2
Day 41	15:00	33,714.0	26,327.5	28.8	33,941.9
Day 41	16:00	33,571.0	26,308.7	28.8	33,798.7
Day 41	17:00	33,428.2	26,289.9	28.8	33,655.6
Day 41	18:00	33,285.6	26,271.1	28.8	33,512.7
Day 41	19:00	33,143.1	26,252.4	28.8	33,370.0
Day 41	20:00	33,000.7	26,233.6	28.8	33,227.4
Day 41	21:00	32,858.6	26,214.9	28.8	33,085.0
Day 41	22:00	32,716.6	26,196.2	28.8	32,942.7
Day 41	23:00	32,574.8	26,177.5	28.8	32,800.7
Day 42	0:00	32,433.2	26,158.9	28.8	32,658.8
Day 42	1:00	32,291.8	26,140.2	28.8	32,517.0
Day 42	2:00	32,150.5	26,121.6	28.8	32,377.2
Day 42	3:00	32,009.5	26,102.7	28.8	32,240.0
Day 42	4:00	31,868.6	26,083.5	28.8	32,101.1
Day 42	5:00	31,728.0	26,064.3	28.8	31,961.3

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 34 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 42	6:00	31,587.6	26,045.0	28.8	31,821.2
Day 42	7:00	31,447.4	26,025.7	28.8	31,681.0
Day 42	8:00	31,307.4	26,006.4	28.8	31,540.8
Day 42	9:00	31,167.6	25,987.1	28.7	31,400.8
Day 42	10:00	31,028.0	25,967.9	28.7	31,260.9
Day 42	11:00	30,888.7	25,948.6	28.7	31,121.2
Day 42	12:00	30,749.6	25,929.4	28.7	30,981.8
Day 42	13:00	30,610.7	25,910.2	28.7	30,842.5
Day 42	14:00	30,472.1	25,891.1	28.7	30,703.5
Day 42	15:00	30,333.7	25,872.0	28.7	30,564.7
Day 42	16:00	30,195.6	25,852.9	28.7	30,426.2
Day 42	17:00	30,057.7	25,833.9	28.7	30,287.9
Day 42	18:00	29,920.1	25,814.9	28.7	30,149.9
Day 42	19:00	29,782.7	25,795.9	28.7	30,012.1
Day 42	20:00	29,645.6	25,777.0	28.7	29,874.5
Day 42	21:00	29,508.7	25,758.1	28.7	29,737.2
Day 42	22:00	29,372.2	25,739.2	28.7	29,600.2
Day 42	23:00	29,235.8	25,720.4	28.7	29,463.5
Day 43	0:00	29,099.8	25,701.6	28.7	29,327.0
Day 43	1:00	28,964.0	25,682.8	28.7	29,190.7
Day 43	2:00	28,828.5	25,664.1	28.7	29,054.8
Day 43	3:00	28,693.3	25,645.4	28.7	28,919.1
Day 43	4:00	28,558.4	25,626.8	28.7	28,783.7
Day 43	5:00	28,423.8	25,608.2	28.6	28,648.6
Day 43	6:00	28,289.5	25,589.6	28.6	28,513.8
Day 43	7:00	28,155.4	25,571.1	28.6	28,379.3
Day 43	8:00	28,021.7	25,552.6	28.6	28,245.1
Day 43	9:00	27,888.2	25,534.2	28.6	28,111.1
Day 43	10:00	27,755.1	25,515.8	28.6	27,977.5
Day 43	11:00	27,622.3	25,497.4	28.6	27,844.1

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 35 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 43	12:00	27,489.7	25,479.1	28.6	27,711.1
Day 43	13:00	27,357.5	25,460.8	28.6	27,578.4
Day 43	14:00	27,225.6	25,442.6	28.6	27,446.0
Day 43	15:00	27,094.1	25,424.4	28.6	27,313.9
Day 43	16:00	26,962.8	25,406.2	28.6	27,182.1
Day 43	17:00	26,831.9	25,388.1	28.6	27,050.6
Day 43	18:00	26,701.3	25,370.0	28.6	26,921.7
Day 43	19:00	26,571.0	25,351.6	28.6	26,794.9
Day 43	20:00	26,441.0	25,333.0	28.6	26,666.7
Day 43	21:00	26,311.4	25,314.4	28.6	26,537.8
Day 43	22:00	26,182.2	25,295.7	28.6	26,408.6
Day 43	23:00	26,053.2	25,277.0	28.6	26,279.5
Day 44	0:00	25,924.6	25,258.3	28.6	26,150.5
Day 44	1:00	25,796.4	25,239.6	28.5	26,021.8
Day 44	2:00	25,668.5	25,221.0	28.5	25,893.3
Day 44	3:00	25,540.9	25,202.5	28.5	25,765.2
Day 44	4:00	25,413.7	25,183.9	28.5	25,637.4
Day 44	5:00	25,286.8	25,165.5	28.5	25,509.9
Day 44	6:00	25,160.3	25,147.1	28.5	25,382.8
Day 44	7:00	25,034.2	25,128.7	28.5	25,256.1
Day 44	8:00	24,908.4	25,110.4	28.5	25,129.6
Day 44	9:00	24,783.0	25,092.1	28.5	25,003.6
Day 44	10:00	24,657.9	25,073.9	28.5	24,877.9
Day 44	11:00	24,533.2	25,055.8	28.5	24,752.6
Day 44	12:00	24,408.8	25,037.7	28.5	24,627.6
Day 44	13:00	24,284.9	25,019.6	28.5	24,503.0
Day 44	14:00	24,161.3	25,001.6	28.5	24,378.7
Day 44	15:00	24,038.0	24,983.7	28.5	24,254.9
Day 44	16:00	23,915.2	24,965.8	28.5	24,131.3
Day 44	17:00	23,792.7	24,947.9	28.5	24,008.2

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 36 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 44	18:00	23,670.6	24,930.2	28.5	23,885.5
Day 44	19:00	23,548.9	24,912.4	28.5	23,763.1
Day 44	20:00	23,427.5	24,894.8	28.5	23,641.1
Day 44	21:00	23,306.6	24,877.1	28.5	23,519.4
Day 44	22:00	23,186.0	24,859.6	28.4	23,398.2
Day 44	23:00	23,065.8	24,842.1	28.4	23,277.3
Day 45	0:00	22,946.0	24,824.6	28.4	23,156.9
Day 45	1:00	22,826.6	24,807.2	28.4	23,036.8
Day 45	2:00	22,707.6	24,789.9	28.4	22,917.1
Day 45	3:00	22,588.9	24,772.6	28.4	22,797.7
Day 45	4:00	22,470.7	24,755.4	28.4	22,678.8
Day 45	5:00	22,352.8	24,738.2	28.4	22,560.3
Day 45	6:00	22,235.4	24,721.1	28.4	22,442.1
Day 45	7:00	22,118.3	24,704.0	28.4	22,324.4
Day 45	8:00	22,001.7	24,687.0	28.4	22,207.0
Day 45	9:00	21,885.4	24,670.1	28.4	22,090.1
Day 45	10:00	21,769.5	24,653.2	28.4	21,973.5
Day 45	11:00	21,654.1	24,636.3	28.4	21,859.9
Day 45	12:00	21,539.0	24,619.1	28.4	21,748.7
Day 45	13:00	21,424.3	24,601.7	28.4	21,636.0
Day 45	14:00	21,310.1	24,584.2	28.4	21,522.5
Day 45	15:00	21,196.2	24,566.6	28.4	21,408.8
Day 45	16:00	21,082.8	24,549.0	28.4	21,295.1
Day 45	17:00	20,969.8	24,531.5	28.4	21,181.6
Day 45	18:00	20,857.1	24,514.0	28.4	21,068.4
Day 45	19:00	20,744.9	24,496.6	28.3	20,955.6
Day 45	20:00	20,633.1	24,479.2	28.3	20,843.0
Day 45	21:00	20,521.7	24,461.9	28.3	20,730.9
Day 45	22:00	20,410.7	24,444.6	28.3	20,619.2
Day 45	23:00	20,300.2	24,427.4	28.3	20,507.8

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 37 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 46	0:00	20,190.0	24,410.3	28.3	20,396.9
Day 46	1:00	20,080.2	24,393.2	28.3	20,286.4
Day 46	2:00	19,970.9	24,376.2	28.3	20,176.3
Day 46	3:00	19,862.0	24,359.3	28.3	20,066.6
Day 46	4:00	19,753.5	24,342.4	28.3	19,957.3
Day 46	5:00	19,645.4	24,325.6	28.3	19,848.5
Day 46	6:00	19,537.7	24,308.8	28.3	19,740.0
Day 46	7:00	19,430.5	24,292.1	28.3	19,632.0
Day 46	8:00	19,323.6	24,275.5	28.3	19,524.4
Day 46	9:00	19,217.2	24,259.0	28.3	19,417.2
Day 46	10:00	19,111.2	24,242.5	28.3	19,310.4
Day 46	11:00	19,005.7	24,226.0	28.3	19,204.0
Day 46	12:00	18,900.5	24,209.7	28.3	19,098.1
Day 46	13:00	18,795.8	24,193.4	28.3	18,992.6
Day 46	14:00	18,691.4	24,177.2	28.3	18,887.5
Day 46	15:00	18,587.6	24,161.0	28.3	18,782.8
Day 46	16:00	18,484.1	24,144.9	28.2	18,678.5
Day 46	17:00	18,381.0	24,128.8	28.2	18,574.7
Day 46	18:00	18,278.4	24,112.9	28.2	18,471.3
Day 46	19:00	18,176.2	24,097.0	28.2	18,368.3
Day 46	20:00	18,074.4	24,081.1	28.2	18,265.7
Day 46	21:00	17,973.0	24,065.3	28.2	18,163.6
Day 46	22:00	17,872.1	24,049.6	28.2	18,061.8
Day 46	23:00	17,771.6	24,034.0	28.2	17,960.5
Day 47	0:00	17,671.5	24,018.4	28.2	17,859.6
Day 47	1:00	17,571.8	24,002.9	28.2	17,759.2
Day 47	2:00	17,472.5	23,987.4	28.2	17,659.1
Day 47	3:00	17,373.7	23,972.0	28.2	17,559.5
Day 47	4:00	17,275.3	23,956.7	28.2	17,460.3
Day 47	5:00	17,177.3	23,941.5	28.2	17,361.5

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 38 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 47	6:00	17,079.8	23,926.3	28.2	17,263.2
Day 47	7:00	16,982.6	23,911.0	28.2	17,169.6
Day 47	8:00	16,885.9	23,895.4	28.2	17,075.9
Day 47	9:00	16,789.6	23,879.6	28.2	16,981.1
Day 47	10:00	16,693.7	23,863.8	28.2	16,885.8
Day 47	11:00	16,598.3	23,847.9	28.2	16,790.4
Day 47	12:00	16,503.3	23,832.0	28.2	16,695.0
Day 47	13:00	16,408.7	23,816.2	28.2	16,599.9
Day 47	14:00	16,314.5	23,800.4	28.2	16,505.0
Day 47	15:00	16,220.7	23,784.7	28.1	16,410.5
Day 47	16:00	16,127.4	23,769.1	28.1	16,316.4
Day 47	17:00	16,034.5	23,753.5	28.1	16,222.7
Day 47	18:00	15,942.0	23,738.0	28.1	16,129.4
Day 47	19:00	15,849.9	23,722.5	28.1	16,036.4
Day 47	20:00	15,758.2	23,707.1	28.1	15,944.0
Day 47	21:00	15,667.0	23,691.8	28.1	15,851.9
Day 47	22:00	15,576.2	23,676.6	28.1	15,760.2
Day 47	23:00	15,485.8	23,661.4	28.1	15,669.0
Day 48	0:00	15,395.8	23,646.3	28.1	15,578.2
Day 48	1:00	15,306.2	23,631.2	28.1	15,487.8
Day 48	2:00	15,217.1	23,616.3	28.1	15,397.8
Day 48	3:00	15,128.4	23,601.4	28.1	15,308.2
Day 48	4:00	15,040.1	23,586.6	28.1	15,219.1
Day 48	5:00	14,952.2	23,571.8	28.1	15,130.3
Day 48	6:00	14,864.7	23,557.1	28.1	15,042.0
Day 48	7:00	14,777.6	23,542.5	28.1	14,954.1
Day 48	8:00	14,691.0	23,527.9	28.1	14,866.6
Day 48	9:00	14,604.8	23,513.5	28.1	14,779.6
Day 48	10:00	14,518.9	23,499.1	28.1	14,692.9
Day 48	11:00	14,433.5	23,484.7	28.1	14,606.7

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 39 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 48	12:00	14,348.5	23,470.4	28.1	14,520.8
Day 48	13:00	14,264.0	23,456.2	28.1	14,435.4
Day 48	14:00	14,179.8	23,442.1	28.1	14,350.4
Day 48	15:00	14,096.0	23,428.0	28.0	14,265.8
Day 48	16:00	14,012.7	23,414.0	28.0	14,181.7
Day 48	17:00	13,929.7	23,400.1	28.0	14,097.9
Day 48	18:00	13,847.2	23,386.2	28.0	14,014.5
Day 48	19:00	13,765.1	23,372.4	28.0	13,931.6
Day 48	20:00	13,683.4	23,358.7	28.0	13,849.1
Day 48	21:00	13,602.1	23,345.1	28.0	13,766.9
Day 48	22:00	13,521.2	23,331.5	28.0	13,685.2
Day 48	23:00	13,440.7	23,317.9	28.0	13,603.9
Day 49	0:00	13,360.6	23,304.5	28.0	13,523.0
Day 49	1:00	13,280.9	23,291.1	28.0	13,442.5
Day 49	2:00	13,201.6	23,277.8	28.0	13,362.4
Day 49	3:00	13,122.7	23,264.5	28.0	13,282.7
Day 49	4:00	13,044.2	23,251.3	28.0	13,203.4
Day 49	5:00	12,966.1	23,238.2	28.0	13,124.5
Day 49	6:00	12,888.4	23,225.1	28.0	13,046.0
Day 49	7:00	12,811.2	23,212.1	28.0	12,970.0
Day 49	8:00	12,734.3	23,198.8	28.0	12,897.2
Day 49	9:00	12,657.8	23,185.2	28.0	12,822.9
Day 49	10:00	12,581.7	23,171.5	28.0	12,747.9
Day 49	11:00	12,506.0	23,157.8	28.0	12,672.6
Day 49	12:00	12,430.6	23,144.0	28.0	12,597.1
Day 49	13:00	12,355.7	23,130.3	28.0	12,521.8
Day 49	14:00	12,281.2	23,116.6	28.0	12,446.8
Day 49	15:00	12,207.1	23,102.9	28.0	12,372.0
Day 49	16:00	12,133.3	23,089.3	27.9	12,297.5
Day 49	17:00	12,060.0	23,075.8	27.9	12,223.3

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 40 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 49	18:00	11,987.0	23,062.3	27.9	12,149.5
Day 49	19:00	11,914.4	23,048.9	27.9	12,076.1
Day 49	20:00	11,842.2	23,035.6	27.9	12,003.1
Day 49	21:00	11,770.4	23,022.3	27.9	11,930.5
Day 49	22:00	11,699.0	23,009.1	27.9	11,858.2
Day 49	23:00	11,627.9	22,996.0	27.9	11,786.3
Day 50	0:00	11,557.3	22,982.9	27.9	11,714.8
Day 50	1:00	11,487.0	22,970.0	27.9	11,643.7
Day 50	2:00	11,417.1	22,957.1	27.9	11,572.9
Day 50	3:00	11,347.6	22,944.2	27.9	11,502.6
Day 50	4:00	11,278.4	22,931.4	27.9	11,432.6
Day 50	5:00	11,209.6	22,918.7	27.9	11,363.0
Day 50	6:00	11,141.2	22,906.1	27.9	11,293.7
Day 50	7:00	11,073.2	22,893.5	27.9	11,224.9
Day 50	8:00	11,005.6	22,881.0	27.9	11,156.4
Day 50	9:00	10,938.3	22,868.6	27.9	11,088.3
Day 50	10:00	10,871.4	22,856.2	27.9	11,020.6
Day 50	11:00	10,804.9	22,843.9	27.9	10,953.2
Day 50	12:00	10,738.7	22,831.7	27.9	10,886.2
Day 50	13:00	10,672.9	22,819.5	27.9	10,819.6
Day 50	14:00	10,607.5	22,807.5	27.9	10,753.4
Day 50	15:00	10,542.4	22,795.4	27.9	10,687.5
Day 50	16:00	10,477.7	22,783.5	27.9	10,622.0
Day 50	17:00	10,413.3	22,771.6	27.9	10,556.8
Day 50	18:00	10,349.4	22,759.8	27.9	10,492.1
Day 50	19:00	10,285.7	22,748.0	27.9	10,427.6
Day 50	20:00	10,222.5	22,736.3	27.8	10,363.6
Day 50	21:00	10,159.6	22,724.7	27.8	10,299.9
Day 50	22:00	10,097.0	22,713.1	27.8	10,236.5
Day 50	23:00	10,034.8	22,701.6	27.8	10,173.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 41 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 51	0:00	9973.0	22,690.2	27.8	10,110.9
Day 51	1:00	9911.5	22,678.8	27.8	10,048.7
Day 51	2:00	9850.4	22,667.5	27.8	9986.8
Day 51	3:00	9789.6	22,656.3	27.8	9925.2
Day 51	4:00	9729.2	22,645.1	27.8	9864.0
Day 51	5:00	9669.1	22,634.0	27.8	9803.1
Day 51	6:00	9609.4	22,623.0	27.8	9742.6
Day 51	7:00	9550.0	22,612.0	27.8	9682.5
Day 51	8:00	9490.9	22,601.1	27.8	9622.6
Day 51	9:00	9432.2	22,590.2	27.8	9563.2
Day 51	10:00	9373.9	22,579.4	27.8	9504.1
Day 51	11:00	9315.8	22,568.7	27.8	9445.3
Day 51	12:00	9258.2	22,558.0	27.8	9386.8
Day 51	13:00	9200.8	22,547.4	27.8	9328.7
Day 51	14:00	9143.8	22,536.9	27.8	9271.0
Day 51	15:00	9087.1	22,526.4	27.8	9213.6
Day 51	16:00	9030.8	22,515.8	27.8	9161.1
Day 51	17:00	8974.8	22,504.9	27.8	9108.2
Day 51	18:00	8919.1	22,493.8	27.8	9054.3
Day 51	19:00	8863.8	22,482.6	27.8	8999.8
Day 51	20:00	8808.8	22,471.3	27.8	8945.1
Day 51	21:00	8754.1	22,460.1	27.8	8890.3
Day 51	22:00	8699.7	22,448.8	27.8	8835.7
Day 51	23:00	8645.7	22,437.6	27.8	8781.2
Day 52	0:00	8592.0	22,426.4	27.8	8726.9
Day 52	1:00	8538.6	22,415.3	27.8	8672.8
Day 52	2:00	8485.5	22,404.2	27.8	8619.0
Day 52	3:00	8432.8	22,393.2	27.7	8565.6
Day 52	4:00	8380.3	22,382.3	27.7	8512.4
Day 52	5:00	8328.2	22,371.4	27.7	8459.5

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 42 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 52	6:00	8276.4	22,360.6	27.7	8406.9
Day 52	7:00	8224.9	22,349.8	27.7	8354.7
Day 52	8:00	8173.8	22,339.1	27.7	8302.7
Day 52	9:00	8122.9	22,328.5	27.7	8251.1
Day 52	10:00	8072.4	22,317.9	27.7	8199.8
Day 52	11:00	8022.1	22,307.5	27.7	8148.7
Day 52	12:00	7972.2	22,297.0	27.7	8098.0
Day 52	13:00	7922.6	22,286.7	27.7	8047.7
Day 52	14:00	7873.3	22,276.3	27.7	7997.6
Day 52	15:00	7824.2	22,266.1	27.7	7947.8
Day 52	16:00	7775.5	22,255.9	27.7	7898.3
Day 52	17:00	7727.1	22,245.8	27.7	7849.2
Day 52	18:00	7679.0	22,235.8	27.7	7800.3
Day 52	19:00	7631.2	22,225.8	27.7	7751.7
Day 52	20:00	7583.7	22,215.8	27.7	7703.5
Day 52	21:00	7536.5	22,206.0	27.7	7655.5
Day 52	22:00	7489.5	22,196.2	27.7	7607.8
Day 52	23:00	7442.9	22,186.4	27.7	7560.5
Day 53	0:00	7396.6	22,176.7	27.7	7513.4
Day 53	1:00	7350.5	22,167.1	27.7	7466.6
Day 53	2:00	7304.8	22,157.5	27.7	7420.1
Day 53	3:00	7259.3	22,148.0	27.7	7374.0
Day 53	4:00	7214.1	22,138.6	27.7	7328.1
Day 53	5:00	7169.2	22,129.2	27.7	7282.4
Day 53	6:00	7124.6	22,119.9	27.7	7237.1
Day 53	7:00	7080.3	22,110.6	27.7	7192.1
Day 53	8:00	7036.2	22,101.4	27.7	7147.3
Day 53	9:00	6992.5	22,092.2	27.7	7102.8
Day 53	10:00	6949.0	22,083.1	27.7	7058.6
Day 53	11:00	6905.8	22,074.1	27.7	7014.7

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 43 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 53	12:00	6862.8	22,065.1	27.7	6971.1
Day 53	13:00	6820.2	22,056.2	27.6	6927.7
Day 53	14:00	6777.8	22,047.3	27.6	6884.7
Day 53	15:00	6735.6	22,038.5	27.6	6841.9
Day 53	16:00	6693.8	22,029.8	27.6	6799.3
Day 53	17:00	6652.2	22,021.1	27.6	6757.1
Day 53	18:00	6610.9	22,012.5	27.6	6715.1
Day 53	19:00	6569.9	22,003.9	27.6	6673.4
Day 53	20:00	6529.1	21,995.3	27.6	6631.9
Day 53	21:00	6488.6	21,986.9	27.6	6590.8
Day 53	22:00	6448.3	21,978.5	27.6	6549.9
Day 53	23:00	6408.3	21,970.1	27.6	6509.2
Day 54	0:00	6368.6	21,961.8	27.6	6468.8
Day 54	1:00	6329.1	21,953.5	27.6	6428.7
Day 54	2:00	6289.9	21,945.3	27.6	6388.9
Day 54	3:00	6251.0	21,937.2	27.6	6349.3
Day 54	4:00	6212.3	21,929.1	27.6	6309.9
Day 54	5:00	6173.8	21,921.0	27.6	6270.8
Day 54	6:00	6135.6	21,913.1	27.6	6232.0
Day 54	7:00	6097.7	21,905.1	27.6	6193.4
Day 54	8:00	6060.0	21,897.2	27.6	6155.1
Day 54	9:00	6022.6	21,889.4	27.6	6117.0
Day 54	10:00	5985.4	21,881.6	27.6	6079.2
Day 54	11:00	5948.4	21,873.9	27.6	6041.6
Day 54	12:00	5911.7	21,866.2	27.6	6004.3
Day 54	13:00	5875.2	21,858.6	27.6	5967.2
Day 54	14:00	5839.0	21,851.0	27.6	5930.4
Day 54	15:00	5803.0	21,843.4	27.6	5896.1
Day 54	16:00	5767.3	21,835.5	27.6	5863.8
Day 54	17:00	5731.8	21,827.5	27.6	5830.6

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 44 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 54	18:00	5696.5	21,819.2	27.6	5796.7
Day 54	19:00	5661.5	21,810.9	27.6	5762.5
Day 54	20:00	5626.7	21,802.6	27.6	5728.1
Day 54	21:00	5592.2	21,794.2	27.6	5693.6
Day 54	22:00	5557.8	21,785.8	27.6	5659.1
Day 54	23:00	5523.7	21,777.5	27.6	5624.7
Day 55	0:00	5489.9	21,769.1	27.6	5590.4
Day 55	1:00	5456.2	21,760.8	27.6	5556.3
Day 55	2:00	5422.8	21,752.6	27.6	5522.3
Day 55	3:00	5389.6	21,744.4	27.6	5488.6
Day 55	4:00	5356.7	21,736.2	27.6	5455.0
Day 55	5:00	5323.9	21,728.1	27.6	5421.7
Day 55	6:00	5291.4	21,720.1	27.5	5388.5
Day 55	7:00	5259.1	21,712.1	27.5	5355.6
Day 55	8:00	5227.1	21,704.2	27.5	5322.9
Day 55	9:00	5195.2	21,696.3	27.5	5290.4
Day 55	10:00	5163.6	21,688.4	27.5	5258.1
Day 55	11:00	5132.1	21,680.6	27.5	5226.0
Day 55	12:00	5100.9	21,672.9	27.5	5194.2
Day 55	13:00	5069.9	21,665.2	27.5	5162.6
Day 55	14:00	5039.2	21,657.6	27.5	5131.2
Day 55	15:00	5008.6	21,650.0	27.5	5100.0
Day 55	16:00	4978.2	21,642.5	27.5	5069.0
Day 55	17:00	4948.1	21,635.0	27.5	5038.2
Day 55	18:00	4918.1	21,627.6	27.5	5007.6
Day 55	19:00	4888.4	21,620.2	27.5	4977.3
Day 55	20:00	4858.9	21,612.9	27.5	4947.1
Day 55	21:00	4829.5	21,605.6	27.5	4917.2
Day 55	22:00	4800.4	21,598.4	27.5	4887.5
Day 55	23:00	4771.5	21,591.2	27.5	4858.0

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 45 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 56	0:00	4742.8	21,584.1	27.5	4828.6
Day 56	1:00	4714.3	21,577.0	27.5	4799.5
Day 56	2:00	4685.9	21,570.0	27.5	4770.6
Day 56	3:00	4657.8	21,563.1	27.5	4741.9
Day 56	4:00	4629.9	21,556.1	27.5	4713.4
Day 56	5:00	4602.2	21,549.3	27.5	4685.1
Day 56	6:00	4574.6	21,542.4	27.5	4657.0
Day 56	7:00	4547.3	21,535.6	27.5	4629.0
Day 56	8:00	4520.1	21,528.9	27.5	4601.3
Day 56	9:00	4493.2	21,522.2	27.5	4573.8
Day 56	10:00	4466.4	21,515.6	27.5	4546.4
Day 56	11:00	4439.8	21,509.0	27.5	4519.3
Day 56	12:00	4413.4	21,502.4	27.5	4492.3
Day 56	13:00	4387.2	21,495.9	27.5	4465.6
Day 56	14:00	4361.2	21,489.5	27.5	4439.0
Day 56	15:00	4335.3	21,483.1	27.5	4412.6
Day 56	16:00	4309.6	21,476.7	27.5	4386.4
Day 56	17:00	4284.2	21,470.4	27.5	4360.4
Day 56	18:00	4258.9	21,464.1	27.5	4334.5
Day 56	19:00	4233.7	21,457.9	27.5	4308.9
Day 56	20:00	4208.8	21,451.7	27.5	4283.4
Day 56	21:00	4184.0	21,445.6	27.5	4258.1
Day 56	22:00	4159.5	21,439.5	27.5	4233.0
Day 56	23:00	4135.0	21,433.4	27.5	4208.1
Day 57	0:00	4110.8	21,427.4	27.5	4183.3
Day 57	1:00	4086.7	21,421.4	27.5	4158.7
Day 57	2:00	4062.9	21,415.5	27.5	4134.3
Day 57	3:00	4039.1	21,409.6	27.5	4110.1
Day 57	4:00	4015.6	21,403.8	27.5	4086.0
Day 57	5:00	3992.2	21,398.0	27.5	4062.1

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 46 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 57	6:00	3969.0	21,392.2	27.4	4038.4
Day 57	7:00	3945.9	21,386.5	27.4	4014.9
Day 57	8:00	3923.1	21,380.8	27.4	3991.5
Day 57	9:00	3900.3	21,375.2	27.4	3968.3
Day 57	10:00	3877.8	21,369.6	27.4	3945.3
Day 57	11:00	3855.4	21,364.0	27.4	3922.4
Day 57	12:00	3833.2	21,358.5	27.4	3899.7
Day 57	13:00	3811.1	21,353.0	27.4	3877.1
Day 57	14:00	3789.2	21,347.6	27.4	3854.7
Day 57	15:00	3767.4	21,342.2	27.4	3832.5
Day 57	16:00	3745.8	21,336.8	27.4	3810.4
Day 57	17:00	3724.4	21,331.5	27.4	3788.5
Day 57	18:00	3703.1	21,326.2	27.4	3766.8
Day 57	19:00	3682.0	21,321.0	27.4	3745.2
Day 57	20:00	3661.0	21,315.8	27.4	3723.8
Day 57	21:00	3640.2	21,310.6	27.4	3702.5
Day 57	22:00	3619.6	21,305.5	27.4	3681.4
Day 57	23:00	3599.0	21,300.4	27.4	3660.4
Day 58	0:00	3578.7	21,295.3	27.4	3639.6
Day 58	1:00	3558.5	21,290.3	27.4	3619.0
Day 58	2:00	3538.4	21,285.3	27.4	3598.4
Day 58	3:00	3518.5	21,280.4	27.4	3578.1
Day 58	4:00	3498.7	21,275.5	27.4	3557.9
Day 58	5:00	3479.1	21,270.6	27.4	3537.8
Day 58	6:00	3459.6	21,265.8	27.4	3517.9
Day 58	7:00	3440.2	21,261.0	27.4	3498.1
Day 58	8:00	3421.0	21,256.2	27.4	3478.5
Day 58	9:00	3402.0	21,251.5	27.4	3459.0
Day 58	10:00	3383.1	21,246.8	27.4	3439.7
Day 58	11:00	3364.3	21,242.1	27.4	3420.5

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 47 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 58	12:00	3345.6	21,237.5	27.4	3401.4
Day 58	13:00	3327.1	21,232.9	27.4	3382.5
Day 58	14:00	3308.8	21,228.3	27.4	3363.8
Day 58	15:00	3290.6	21,223.8	27.4	3345.1
Day 58	16:00	3272.5	21,219.3	27.4	3326.6
Day 58	17:00	3254.5	21,214.9	27.4	3308.3
Day 58	18:00	3236.7	21,210.4	27.4	3290.0
Day 58	19:00	3219.0	21,206.0	27.4	3272.0
Day 58	20:00	3201.4	21,201.7	27.4	3254.0
Day 58	21:00	3184.0	21,197.3	27.4	3236.2
Day 58	22:00	3166.7	21,193.0	27.4	3218.5
Day 58	23:00	3149.5	21,188.8	27.4	3200.9
Day 59	0:00	3132.5	21,184.6	27.4	3183.5
Day 59	1:00	3115.6	21,180.3	27.4	3167.6
Day 59	2:00	3098.8	21,175.9	27.4	3153.5
Day 59	3:00	3082.1	21,171.3	27.4	3138.9
Day 59	4:00	3065.6	21,166.5	27.4	3123.7
Day 59	5:00	3049.2	21,161.7	27.4	3108.3
Day 59	6:00	3032.9	21,156.8	27.4	3092.6
Day 59	7:00	3016.7	21,151.8	27.4	3076.9
Day 59	8:00	3000.7	21,146.9	27.4	3061.0
Day 59	9:00	2984.8	21,141.9	27.4	3045.1
Day 59	10:00	2969.0	21,136.9	27.4	3029.2
Day 59	11:00	2953.3	21,131.9	27.4	3013.4
Day 59	12:00	2937.7	21,126.9	27.4	2997.6
Day 59	13:00	2922.3	21,122.0	27.4	2981.9
Day 59	14:00	2906.9	21,117.1	27.4	2966.2
Day 59	15:00	2891.7	21,112.2	27.4	2950.6
Day 59	16:00	2876.6	21,107.4	27.4	2935.2
Day 59	17:00	2861.6	21,102.5	27.4	2919.8

LNP COL 2.4-2

Table 2.4.3-226 (Sheet 48 of 48)
Water Elevation in Lake Rousseau Resulting from PMF

Day	Time	Inflow (cfs)	Storage (ac-ft)	Lake Elevation (ft.)	Outflow (cfs)
Day 59	18:00	2846.8	21,097.7	27.4	2904.5
Day 59	19:00	2832.0	21,093.0	27.4	2889.4
Day 59	20:00	2817.3	21,088.3	27.4	2874.3
Day 59	21:00	2802.8	21,083.6	27.4	2859.4
Day 59	22:00	2788.4	21,078.9	27.4	2844.5
Day 59	23:00	2774.1	21,074.3	27.4	2829.8
Day 60	0:00	2759.9	21,069.7	27.4	2815.2
Day 60	1:00	2745.8	21,065.1	27.4	2800.7
Day 60	2:00	2731.8	21,060.6	27.3	2786.3
Day 60	3:00	2717.9	21,056.1	27.3	2772.0
Day 60	4:00	2704.1	21,051.7	27.3	2757.8
Day 60	5:00	2690.4	21,047.3	27.3	2743.7
Day 60	6:00	2676.9	21,042.9	27.3	2729.7
Day 60	7:00	2663.4	21,038.5	27.3	2715.9
Day 60	8:00	2650.1	21,034.2	27.3	2702.1
Day 60	9:00	2636.8	21,029.9	27.3	2688.5
Day 60	10:00	2623.6	21,025.7	27.3	2674.9
Day 60	11:00	2610.6	21,021.4	27.3	2661.5
Day 60	12:00	2597.6	21,017.3	27.3	2648.1
Day 60	13:00	2584.8	21,013.1	27.3	2634.9
Day 60	14:00	2572.0	21,009.0	27.3	2621.7
Day 60	15:00	2559.4	21,004.9	27.3	2608.7
Day 60	16:00	2546.8	21,000.8	27.3	2595.8
Day 60	17:00	2534.3	20,996.8	27.3	2582.9
Day 60	18:00	2522.0	20,992.8	27.3	2570.2
Day 60	19:00	2509.7	20,988.8	27.3	2557.5
Day 60	20:00	2497.5	20,984.9	27.3	2545.0
Day 60	21:00	2485.5	20,981.0	27.3	2532.5
Day 60	22:00	2473.5	20,977.1	27.3	2520.2
Day 60	23:00	2461.6	20,973.3	27.3	2507.9
Day 61	0:00	2449.8	20,969.4	27.3	2495.8

Notes:

cfs = cubic foot per second, ac-ft = acre-foot, ft. = foot

LNP COL 2.4-2

Table 2.4.4-201 (Sheet 1 of 5)
Water Elevations in the Lower Withlacoochee River Resulting from PMF

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft.)	W.S. Elev (ft.)	Crit W.S. (ft.)	E.G. Elev (ft.)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area ($ft.^2$)	Top Width (ft.)	Froude # Chl
STREAM	55472.83	PF 1	60,811	15.66	24.72	24.72	27.61	0.006004	15.28	6035.76	1424.33	0.96
STREAM	55029.23	PF 1	60,811	6.00	22.13		22.55	0.000409	6.08	17,366.25	2767.04	0.28
STREAM	54521.44	PF 1	60,811	6.00	21.90		22.35	0.000388	5.83	16,147.86	2901.89	0.27
STREAM	54040.53	PF 1	60,811	6.00	21.88		22.14	0.000238	4.38	19,414.68	3180.69	0.21
STREAM	53457.92	PF 1	60,811	6.00	21.84		22.01	0.000173	3.45	22,116.77	3468.91	0.17
STREAM	53039.9	PF 1	60,811	6.00	21.76		21.92	0.000190	3.25	21,422.07	3123.64	0.18
STREAM	52556.84	PF 1	60,811	4.58	21.53		21.79	0.000325	4.66	19,407.98	2641.77	0.24
STREAM	51990.35	PF 1	60,811	4.00	21.12		21.57	0.000464	6.17	16,321.07	2527.41	0.29
STREAM	51547.63	PF 1	60,811	4.00	20.83		21.33	0.000498	6.19	13,551.45	1673.18	0.30
STREAM	50972.46	PF 1	60,811	8.00	19.35	17.16	20.77	0.002342	9.69	6873.93	1224.67	0.60
STREAM	50534.48	PF 1	60,811	8.00	19.91		20.06	0.000303	3.28	19,749.85	2336.49	0.21
STREAM	49936.97	PF 1	60,811	7.02	19.83		19.93	0.000166	2.89	25,693.41	2958.43	0.16
STREAM	49469.38	PF 1	60,811	6.23	19.72		19.84	0.000229	3.17	24,623.66	3243.93	0.19
STREAM	49008.78	PF 1	60,811	4.00	19.65		19.74	0.000144	3.18	29,088.68	3592.79	0.16
STREAM	48558.78	PF 1	60,811	6.00	19.58		19.67	0.000143	2.85	28,586.66	3494.37	0.15
STREAM	48006.51	PF 1	60,811	4.00	19.55		19.61	0.000080	2.30	35,575.85	3729.13	0.12
STREAM	47587.45	PF 1	60,811	7.01	19.47		19.55	0.000121	2.67	28,918.69	3131.30	0.14
STREAM	46984.27	PF 1	60,811	2.00	19.46		19.51	0.000040	1.78	34,490.81	2732.20	0.09
STREAM	46544.97	PF 1	60,811	2.00	19.22		19.45	0.000252	4.91	21,253.95	2269.61	0.22
STREAM	45954.43	PF 1	60,811	2.00	19.13		19.32	0.000216	4.54	23,836.45	2711.81	0.20

LNP COL 2.4-2

Table 2.4.4-201 (Sheet 2 of 5)
Water Elevations in the Lower Withlacoochee River Resulting from PMF

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft.)	W.S. Elev (ft.)	Crit W.S. (ft.)	E.G. Elev (ft.)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (ft.^{2})	Top Width (ft.)	Froude # Chl
STREAM	45505.86	PF 1	60,811	2.00	19.09		19.21	0.000150	3.91	30,160.27	3194.89	0.17
STREAM	44957.04	PF 1	60,811	2.00	18.95		19.12	0.000216	4.63	26,327.54	3898.29	0.20
STREAM	44445.6	PF 1	60,811	2.00	18.89		19.01	0.000165	3.98	32,053.72	4442.29	0.18
STREAM	43988.74	PF 1	60,811	2.00	18.81		18.92	0.000175	4.08	34,274.95	4926.14	0.18
STREAM	43516	PF 1	60,811	2.00	18.64		18.82	0.000199	4.24	26,368.94	4066.89	0.19
STREAM	42974.48	PF 1	60,811	2.00	18.54		18.72	0.000211	4.43	27,266.31	3696.32	0.20
STREAM	42488.89	PF 1	60,811	2.00	18.26		18.56	0.000400	5.94	22,760.41	4003.07	0.27
STREAM	41955.36	PF 1	60,811	2.00	18.27		18.38	0.000174	3.45	32,366.98	4970.36	0.17
STREAM	41381.78	PF 1	60,811	2.00	18.15		18.28	0.000211	4.02	32,243.76	7746.50	0.19
STREAM	40993.37	PF 1	60,811	2.00	17.91		18.14	0.000340	5.13	22,605.65	5219.35	0.25
STREAM	40457.15	PF 1	60,811	2.00	17.71		17.98	0.000294	4.97	20,412.16	4028.36	0.23
STREAM	40022.67	PF 1	60,811	2.00	17.49		17.80	0.000416	5.74	19,511.29	3989.76	0.27
STREAM	39523.39	PF 1	60,811	2.00	17.24		17.58	0.000463	5.81	19,227.41	5036.46	0.29
STREAM	39034.41	PF 1	60,811	2.00	17.08		17.36	0.000345	5.12	19,416.70	5937.98	0.25
STREAM	38529.39	PF 1	60,811	2.00	16.76		17.14	0.000563	6.46	18,614.95	6396.56	0.32
STREAM	37945.3	PF 1	60,811	2.00	16.62		16.88	0.000352	4.92	21,319.59	6309.10	0.25
STREAM	37500.19	PF 1	60,811	2.00	16.11		16.61	0.000786	7.43	15,780.14	5602.54	0.37
STREAM	36022.63	PF 1	60,811	2.00	15.89		16.24	0.000543	6.17	19,700.53	4586.74	0.31
STREAM	35480.8	PF 1	60,811	2.00	15.37		15.86	0.001004	8.16	16,854.03	4323.54	0.42
STREAM	35039.64	PF 1	60,811	2.00	15.04		15.39	0.000736	6.62	21,574.74	6748.21	0.35

LNP COL 2.4-2

Table 2.4.4-201 (Sheet 3 of 5)
Water Elevations in the Lower Withlacoochee River Resulting from PMF

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft.)	W.S. Elev (ft.)	Crit W.S. (ft.)	E.G. Elev (ft.)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (ft.^{2})	Top Width (ft.)	Froude # Chl
STREAM	34438.63	PF 1	60,811	2.00	14.87		15.07	0.000425	4.81	29,399.97	9282.15	0.27
STREAM	34003.74	PF 1	60,811	2.00	14.66		14.86	0.000405	5.13	31,755.84	11,014.90	0.26
STREAM	33489.43	PF 1	60,811	2.00	14.35		14.60	0.000662	6.19	28,957.72	11,895.97	0.33
STREAM	32997.06	PF 1	60,811	2.00	14.05		14.28	0.000605	5.79	31,870.27	12,993.24	0.32
STREAM	32464.44	PF 1	60,811	2.00	13.80		13.98	0.000516	5.48	35,498.34	13,137.55	0.29
STREAM	32005.59	PF 1	60,811	2.00	13.51		13.71	0.000586	5.52	34,413.58	13,983.75	0.31
STREAM	31500.25	PF 1	60,811	2.00	13.13		13.37	0.000784	6.36	31,494.04	13,274.49	0.36
STREAM	31021.13	PF 1	60,811	2.00	12.99		13.08	0.000331	4.00	43,455.15	13,791.16	0.23
STREAM	30508.14	PF 1	60,811	2.00	12.85		12.92	0.000292	3.84	45,893.11	13,866.74	0.22
STREAM	29979.39	PF 1	60,811	2.00	12.69		12.76	0.000323	3.88	44,445.50	14,097.54	0.23
STREAM	29557.06	PF 1	60,811	2.00	12.53		12.61	0.000303	3.90	44,844.18	13,854.17	0.22
STREAM	28919.24	PF 1	60,811	2.00	12.41		12.46	0.000247	2.95	48,417.20	14,103.03	0.19
STREAM	28484.21	PF 1	60,811	2.00	12.30		12.35	0.000213	2.72	48,968.79	13,868.48	0.18
STREAM	27920.79	PF 1	60,811	2.00	12.18		12.24	0.000192	2.70	46,990.45	13,720.57	0.17
STREAM	27490.05	PF 1	60,811	2.00	12.06		12.12	0.000309	3.82	45,834.11	13,617.11	0.22
STREAM	26967.58	PF 1	60,811	2.00	11.90		11.97	0.000313	3.86	45,374.94	13,464.07	0.22
STREAM	26502.22	PF 1	60,811	2.00	11.74		11.81	0.000306	3.79	44,419.36	13,520.70	0.22
STREAM	25991.91	PF 1	60,811	2.00	11.58		11.66	0.000296	3.37	42,436.81	13,187.45	0.21
STREAM	25465.82	PF 1	60,811	2.00	11.40		11.47	0.000473	4.52	40,602.80	13,643.38	0.27
STREAM	25008.34	PF 1	60,811	2.00	11.15	7.97	11.23	0.000489	4.29	39,189.85	13,458.43	0.27

LNP COL 2.4-2

Table 2.4.4-201 (Sheet 4 of 5)
Water Elevations in the Lower Withlacoochee River Resulting from PMF

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft.)	W.S. Elev (ft.)	Crit W.S. (ft.)	E.G. Elev (ft.)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (ft.^{2})	Top Width (ft.)	Froude # Chl
STREAM	24487.59	PF 1	60,811	2.00	10.98		11.04	0.000297	3.40	45,137.49	13,482.15	0.21
STREAM	23968.48	PF 1	60,811	2.00	10.84		10.88	0.000306	2.83	44,719.54	13,243.63	0.21
STREAM	23428.13	PF 1	60,811	2.00	10.64		10.71	0.000407	3.65	41,346.67	12,959.63	0.24
STREAM	22988.77	PF 1	60,811	2.00	10.42		10.49	0.000434	4.08	40,247.91	12,856.22	0.26
STREAM	22499.4	PF 1	60,811	2.00	10.21		10.27	0.000434	3.87	42,431.29	13,941.25	0.25
STREAM	21978.23	PF 1	60,811	2.00	9.99		10.06	0.000421	3.78	41,983.18	13,844.38	0.25
STREAM	21491.63	PF 1	60,811	2.00	9.79		9.85	0.000412	3.69	42,749.38	13,827.13	0.25
STREAM	20963.79	PF 1	60,811	2.00	9.62		9.69	0.000258	2.98	45,021.23	13,771.05	0.20
STREAM	20461.32	PF 1	60,811	2.00	9.48		9.54	0.000338	3.13	44,853.80	13,858.85	0.22
STREAM	19986.66	PF 1	60,811	2.00	9.36		9.38	0.000250	2.60	51,196.59	13,882.01	0.19
STREAM	19503.98	PF 1	60,811	2.00	9.22		9.25	0.000269	2.80	48,939.08	13,558.14	0.19
STREAM	18971.31	PF 1	60,811	2.00	9.11		9.14	0.000195	2.34	53,302.19	13,349.43	0.17
STREAM	18483.74	PF 1	60,811	2.00	9.03		9.05	0.000143	2.02	57,793.98	12,941.01	0.14
STREAM	17970.8	PF 1	60,811	2.00	8.97		8.99	0.000110	1.73	59,659.00	12,688.68	0.12
STREAM	17476.77	PF 1	60,811	2.00	8.91		8.94	0.000112	1.74	58,95.77	12,579.85	0.12
STREAM	16965.41	PF 1	60,811	2.00	8.86		8.88	0.000089	1.54	62,090.79	12,422.51	0.11
STREAM	16474.29	PF 1	60,811	2.00	8.82		8.84	0.000082	1.48	62,158.07	12,172.37	0.11
STREAM	15996.13	PF 1	60,811	2.00	8.78		8.80	0.000078	1.41	62,705.33	11,875.39	0.10
STREAM	15481.87	PF 1	60,811	2.00	8.74		8.76	0.000079	1.45	62,143.79	11,465.09	0.10
STREAM	14979.88	PF 1	60,811	2.00	8.70		8.72	0.000077	1.47	62,925.27	11,132.02	0.10

LNP COL 2.4-2

Table 2.4.4-201 (Sheet 5 of 5)
Water Elevations in the Lower Withlacoochee River Resulting from PMF

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft.)	W.S. Elev (ft.)	Crit W.S. (ft.)	E.G. Elev (ft.)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area ($ft.^2$)	Top Width (ft.)	Froude # Chl
STREAM	14541.77	PF 1	60,811	2.00	8.67		8.69	0.000067	1.36	64,194.82	10,982.33	0.10
STREAM	13985.21	PF 1	60,811	2.00	8.64		8.65	0.000065	1.34	67,485.72	11,274.96	0.10
STREAM	13501.78	PF 1	60,811	2.00	8.60		8.62	0.000072	1.43	65,807.31	10,821.10	0.10
STREAM	13103.11	PF 1	60,811	2.00	8.56		8.58	0.000089	1.62	61,362.08	9907.78	0.11
STREAM	12505.61	PF 1	60,811	2.00	8.52		8.54	0.000086	1.59	58,539.93	9334.95	0.11
STREAM	11986.09	PF 1	60,811	2.00	8.47		8.49	0.000077	1.50	57,970.29	9177.01	0.10
STREAM	11458.13	PF 1	60,811	2.00	8.43		8.46	0.000077	1.50	56,755.61	8904.17	0.10
STREAM	11006.01	PF 1	60,811	2.00	8.39		8.42	0.000085	1.57	54,740.62	8585.15	0.11
STREAM	10318.36	PF 1	60,811	2.00	8.35		8.37	0.000091	1.62	53,223.96	8397.17	0.11
STREAM	9991.704	PF 1	60,811	2.00	8.30		8.32	0.000101	1.69	51,665.82	8209.75	0.12
STREAM	9577.629	PF 1	60,811	2.00	8.25		8.27	0.000095	1.63	49,554.99	8086.39	0.12
STREAM	8999.89	PF 1	60,811	2.00	8.19		8.22	0.000107	1.73	47,751.79	7940.44	0.12
STREAM	8485.945	PF 1	60,811	2.00	8.13		8.17	0.000122	1.83	46,886.28	7967.99	0.13
STREAM	7982.569	PF 1	60,811	2.00	8.07		8.10	0.000131	1.88	44,766.37	7768.88	0.13
STREAM	7471.1	PF 1	60,811	2.00	8.00		8.04	0.000130	1.86	42,952.61	7528.85	0.13
STREAM	6966.052	PF 1	60,811	2.00	7.91		7.96	0.000171	2.12	38,595.70	6873.37	0.15
STREAM	6498.988	PF 1	60,811	2.00	7.82		7.87	0.000193	2.22	36,921.13	6343.14	0.16
STREAM	5897.896	PF 1	60,811	2.00	7.69		7.75	0.000288	2.67	32,869.83	5776.51	0.20
STREAM	5442.304	PF 1	60,811	2.00	7.52		7.59	0.000392	3.06	31,401.10	5753.57	0.23
STREAM	4991.98	PF 1	60,811	2.00	7.29		7.37	0.000491	3.33	28,999.58	5550.48	0.26
STREAM	4490.749	PF 1	60,811	2.00	7.00		7.09	0.000626	3.62	26,486.96	5392.21	0.29
STREAM	3990.597	PF 1	60,811	2.00	6.59	4.06	6.73	0.000825	3.92	23,130.91	5374.48	0.32

Notes:

Q = flow rate, Min Ch El = minimum channel elevation, W.S. Elev = water surface elevation, Crit W.S. = critical water surface elevation, E.G. Elev = energy grade line elevation, E.G. Slope = energy grade line slope, Vel Chnl = velocity in the channel, Froude # Chl = Froude number for the channel
cfs = cubic foot per second, ft. = foot, ft/ft = foot per foot, ft/s = foot per second

LNP COL 2.4-2

Table 2.4.5-201 (Sheet 1 of 2)
Hurricanes within 80.5 Km (50 Mi.) of the LNP Site, Levy County, Florida

	Year	Month	Day	Name	Wind Speed (KTS)	Pressure (Mb)	Category
1	1867	10	6	NOT NAMED	70	NA	H1
2	1871	8	17	NOT NAMED	70	NA	H1
3	1871	8	18	NOT NAMED	70	NA	H1
4	1871	8	25	NOT NAMED	70	NA	H1
5	1871	9	6	NOT NAMED	70	NA	H1
6	1871	9	6	NOT NAMED	70	NA	H1
7	1874	9	28	NOT NAMED	70	NA	H1
8	1878	9	10	NOT NAMED	80	NA	H1
9	1878	9	10	NOT NAMED	90	NA	H2
10	1878	9	10	NOT NAMED	80	970	H1
11	1878	9	10	NOT NAMED	70	NA	H1
12	1880	8	30	NOT NAMED	70	NA	H1
13	1880	10	8	NOT NAMED	70	NA	H1
14	1880	10	8	NOT NAMED	70	NA	H1
15	1882	10	11	NOT NAMED	70	NA	H1
16	1886	7	18	NOT NAMED	70	NA	H1
17	1886	7	19	NOT NAMED	70	NA	H1
18	1888	10	10	NOT NAMED	95	NA	H2
19	1888	10	11	NOT NAMED	95	970	H2
20	1896	9	29	NOT NAMED	110	960	H3
21	1896	9	29	NOT NAMED	100	963	H3
22	1928	9	17	NOT NAMED	110	955	H3
23	1928	9	17	NOT NAMED	90	NA	H2
24	1935	9	4	NOT NAMED	85	NA	H2
25	1935	9	4	NOT NAMED	80	NA	H1
26	1944	10	19	NOT NAMED	65	968	H1
27	1945	6	24	NOT NAMED	80	NA	H1
28	1945	6	24	NOT NAMED	70	NA	H1
29	1946	10	8	NOT NAMED	65	NA	H1

LNP COL 2.4-2

Table 2.4.5-201 (Sheet 2 of 2)
Hurricanes within 80.5 Km (50 Mi.) of the LNP Site, Levy County, Florida

	Year	Month	Day	Name	Wind Speed (KTS)	Pressure (mb)	Category
30	1949	8	27	NOT NAMED	65	974	H1
31	1950	9	5	EASY	110	NA	H3
32	1950	9	5	EASY	105	958	H3
33	1950	9	5	EASY	105	NA	H3
34	1950	9	5	EASY	100	NA	H3
35	1950	9	6	EASY	85	NA	H2
36	1950	10	18	KING	65	NA	H1
37	1968	10	18	GLADYS	70	NA	H1
38	1968	10	19	GLADYS	70	977	H1
39	1968	10	19	GLADYS	70	978	H1
40	2000	9	17	GORDON	65	985	H1

Notes:

H1 = Category 1 hurricane
H2 = Category 2 hurricane
H3 = Category 3 hurricane
KTS = knots
mb = millibar
NA = Not available

Source: Reference 2.4.5-202

LNP COL 2.4-2

Table 2.4.5-202
Hurricane Flood Stage Data at Inglis and Yankeetown in Levy County, Florida

Town	Ground Elevation (ft.)	Hurricane Flood Stage Height					
		Cat 5	Cat 4	Cat 3	Cat 2	Cat 1	Tropical Storm
Yankeetown	5	29.7*	25.8*	21.2*	16.3*	10.4*	8.1*
		(21.5)	(17.7)	(13.4)	(8.7)	(2.7)	(0.5)
Inglis	15	31.0*	27.0*	22.1*	15.8*	dry*	dry*
		(14.2)	(10.2)	(5.3)	(dry)	(dry)	(dry)

Notes:

ft. = foot

*Source: Reference 2.4.5-207, Based on 2007 SLOSH Model Output
()Source: Reference 2.4.5-203, Based on Levy County Emergency Management Website

LNP COL 2.4-2

Table 2.4.5-203
Characteristics of the Probable Maximum Hurricane

Parameter	Value		Unit
	Min	Max	
Central Pressure	889	891	mb
Peripheral Pressure	1020	1020	mb
Radius of maximum winds	6.7	22.3	Nautical mi.
Forward speed	16	23	mph
Maximum wind speed	156	157	mph
Track Direction	200	245	Degree from North

Notes:

mb = millibarm mi. = mile/miles, mph = miles per hour

Source: Reference 2.4.5-205

LNP COL 2.4-2

Table 2.4.5-204
Elevations on Station Datum

Station:	8727520	**T.M.:**	0 W
Name:	Cedar Key, Gulf of Mexico, Fl	**Units:**	Feet
Status:	Accepted	**Epoch:**	1983 – 2001

Datum	Value	Description
MHHW	5.6	Mean Higher-High Water
MHW	5.27	Mean High Water
DTL	3.7	Mean Diurnal Tide Level
MTL	3.85	Mean Tide Level
MSL	3.84	Mean Sea Level
MLW	2.44	Mean Low Water
MLLW	1.8	Mean Lower-Low Water
GT	3.8	Great Diurnal Range
MN	2.83	Mean Range of Tide
DHQ	0.34	Mean Diurnal High Water Inequality
DLQ	0.63	Mean Diurnal Low Water Inequality
HWI	6.49	Greenwich High Water Interval (in Hours)
LWI	0.34	Greenwich Low Water Interval (in Hours)
NAVD	4.06	North American Vertical Datum
Maximum	10.75	Highest Water Level on Station Datum
Max Date	10/7/1996	Date Of Highest Water Level
Max Time	22:48	Time Of Highest Water Level
Minimum	-2.4	Lowest Water Level on Station Datum
Min Date	9/18/1947	Date Of Lowest Water Level
Min Time	11:30	Time Of Lowest Water Level

Notes:

T.M. = time meridian
0 W = 0 west
Source: Reference 2.4.5-209

LNP COL 2.4-2

Table 2.4.5-205
Hurricane Parameters

Hurricane Category Number	Sustained Winds (km/h)	Sustained Winds (mph)	Atmospheric Pressure in the Eye (millibars)	Storm Surge (m)	Storm Surge (ft.)	Damage Level
1	119 – 153	74 - 95	980	1.2 - 1.5	4.0 - 4.9	Low
2	154 – 177	96 - 110	965 - 979	1.8 - 2.4	5.9 - 7.9	Moderate
3	179 – 209	111 - 130	945 - 964	2.7 - 3.7	8.9 - 12.2	Extensive
4	211 – 249	131 - 155	920 - 944	4.0 - 5.5	13.0 - 18.0	Extreme
5	> 249	> 155	< 920	> 5.5	> 18.0	Catastrophic

Notes:

m = meters, ft. = feet
km/h = kilometer per hour
mph = miles per hour

Source: Reference 2.4.5-214

Table 2.4.5-206 (Sheet 1 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 1, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
ENE 25	7.9	ENE 25	10	ENE 25	15.6	ENE 25	20.3	ENE 25	24.3	ENE 25	28.9
NE 25	7.9	E 25	9.8	E 25	15.3	E 25	19.9	E 25	23.8	E 25	27.4
E 25	7.6	NE 25	9.2	E 15	14.4	ENE 15	19	ENE 15	23	NE 15	27.3
NNE 25	7.5	E 15	9.1	NE 25	14.4	NE 25	19	NE 15	22.9	ENE 15	27.2
NE 15	6.8	ENE 15	9.1	ENE 15	14.3	E 15	18.9	NE 25	22.9	NE 25	27.1
ENE 15	6.7	NE 15	8.7	NE 15	13.9	NE 15	18.6	E 15	22.6	E 15	26.3
E 15	6.6	NNE 15	7.9	NNE 15	12.7	NNE 15	17.4	NNE 15	21.7	NNE 15	26.3
NNE 15	6.6	NNE 25	7.9	NNE 25	12.5	NNE 25	16.9	NNE 25	20.7	NNE 25	24.8
N 25	6.5	ENE 05	7.7	ENE 05	12.4	ENE 05	16.4	ENE 05	20	ENE 05	23.2
N 15	6	NE 05	7.6	NE 05	12.2	NE 05	16.2	NE 05	19.9	NE 05	23.2
E 05	5.5	E 05	7.4	E 05	11.7	E 05	15.5	E 05	19	E 05	22.2
ENE 05	5.5	NNE 05	7	NNE 05	11.2	NNE 05	15	NNE 05	18.6	NNE 05	21.7
NE 05	5.5	N 05	6.8	N 05	10.8	N 05	14.7	N 15	17.8	N 15	21.4
NNE 05	5.4	N 15	6.4	N 15	10.4	N 15	14.2	N 25	17.4	N 25	21.1
N 05	5.1	N 25	6.1	N 25	9.8	N 25	13.6	N 05	17.3	N 05	19.8
NNW 15	5.1	NNW 05	5.8	NNW 05	9.1	NNW 05	12.2	NNW 05	15	NNW 05	17.8
NNW 05	4.9	NNW 15	5.3	NNW 15	8.4	NNW 15	11.5	NNW 15	14.5	NNW 15	17.6
WNW 05	4	NW 05	4.1	W 15	6	W 15	8.4	W 15	9.2	W 15	9.8
NW 05	3.9	W 05	4	WNW 15	5.2	WNW 15	7.5	WNW 15	8.5	WNW 15	9
W 05	3.9	WNW 05	4	W 05	4.2	W 05	6	W 05	6.9	W 05	7.2

Table 2.4.5-206 (Sheet 2 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 1, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
NW 15	3.8	NW 15	3.8	NW 15	4.1	NW 15	5.9	NW 15	6.8	NW 15	7.2
W 15	3.6	W 15	3.8	NW 05	4	WNW 05	5.4	WNW 05	6.3	WNW 05	6.7
WNW 15	3.5	WNW 15	3.6	WNW 05	4	NW 05	5.3	NW 05	6.2	NW 05	6.6

Notes:

E = east, W = west, N = north, S = south

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-207 (Sheet 1 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 2, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
ENE 25	8	E 25	10.2	E 25	15.9	E 25	20.6	ENE 25	24.9	ENE 25	28.9
NE 25	7.9	ENE 25	10.2	ENE 25	15.9	ENE 25	20.6	E 25	24.5	E 25	28.1
E 25	7.8	E 15	9.4	E 15	14.6	E 15	19.2	NE 25	23.1	NE 25	27.2
NNE 25	7.4	ENE 15	9.2	NE 25	14.5	ENE 15	19.1	ENE 15	23	ENE 15	27.1
ENE 15	6.8	NE 25	9.2	ENE 15	14.4	NE 25	19.1	E 15	22.8	NE 15	27
NE 15	6.8	NE 15	8.6	NE 15	13.8	NE 15	18.5	NE 15	22.7	E 15	26.3
E 15	6.6	NNE 15	7.8	NNE 25	12.6	NNE 15	17.2	NNE 15	21.4	NNE 15	25.9
N 25	6.5	NNE 25	7.8	NNE 15	12.5	NNE 25	16.8	NNE 25	20.7	NNE 25	24.8
NNE 15	6.5	ENE 05	7.6	ENE 05	12.2	ENE 05	16.2	ENE 05	19.7	ENE 05	22.8
N 15	5.9	NE 05	7.4	NE 05	11.9	NE 05	15.9	NE 05	19.5	NE 05	22.7
E 05	5.5	E 05	7.3	E 05	11.6	E 05	15.5	E 05	19.1	E 05	22.3
ENE 05	5.5	NNE 05	6.9	NNE 05	11.2	NNE 05	14.8	NNE 05	18.1	N 25	21.2
NE 05	5.5	N 05	6.7	N 05	10.7	N 05	14.5	N 15	17.6	NNE 05	21.2
NNE 05	5.3	N 15	6.3	N 15	10.2	N 15	13.9	N 25	17	N 15	21.1
N 05	5	N 25	6.1	N 25	9.8	N 25	13.7	N 05	17.2	N 05	19.8
NNW 15	5	NNW 05	5.6	NNW 05	8.7	NNW 05	11.7	NNW 05	14.5	NNW 05	17.2
NNW 05	4.8	NNW 15	5.2	NNW 15	8.3	NNW 15	11.3	NNW 15	14.1	NNW 15	17.1
NW 05	3.9	NW 05	4	W 15	5.9	W 15	8.2	W 15	9	W 15	9.4
W 05	3.9	W 05	4	WNW 15	5.1	WNW 15	7.3	WNW 15	8.3	WNW 15	8.8
WNW 05	3.9	WNW 05	3.9	W 05	4.2	W 05	6.1	W 05	7	W 05	7.3

LNP COL 2.4-2

Table 2.4.5-207 (Sheet 2 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 2, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
NW 15	3.8	W 15	3.8	NW 15	4.1	NW 15	5.8	NW 15	6.7	NW 15	7.1
W 15	3.6	NW 15	3.7	NW 05	4	NW 05	5.2	NW 05	6.2	NW 05	6.6
WNW 15	3.4	WNW 15	3.6	WNW 05	4	WNW 05	5.2	WNW 05	6.2	WNW 05	6.6

Notes:
E = east, W = west, N = north, S = south

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-208 (Sheet 1 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 3, near the LNP Site

Tropical storm		Category 1		Category 2		Category 3		Category 4		Category 5	
ENE 25	7.8	E 25	10	E 25	15.6	E 25	20.1	ENE 25	24.2	ENE 25	28
E 25	7.7	ENE 25	9.9	ENE 25	15.5	ENE 25	20	E 25	23.8	E 25	27.5
NE 25	7.7	E 15	9.1	E 15	14.1	E 15	18.6	NE 25	22.5	NE 25	26.4
NNE 25	7.3	NE 25	8.9	NE 25	14.1	NE 25	18.5	ENE 15	22.2	NE 15	26.1
ENE 15	6.6	ENE 15	8.8	ENE 15	13.8	ENE 15	18.4	NE 15	22.2	ENE 15	26
NE 15	6.6	NE 15	8.3	NE 15	13.3	NE 15	18	E 15	22	E 15	25.7
E 15	6.4	NNE 25	7.6	NNE 25	12.3	NNE 15	16.6	NNE 15	20.6	NNE 15	24.8
N 25	6.4	NNE 15	7.4	NNE 15	12.1	NNE 25	16.3	NNE 25	20.2	NNE 25	24.3
NNE 15	6.3	ENE 05	7.2	ENE 05	11.6	NE 05	15.7	E 05	19	NE 05	22.4
N 15	5.8	E 05	7.1	NE 05	11.6	ENE 05	15.5	NE 05	19	E 05	22.2
E 05	5.3	NE 05	7.1	E 05	11.4	E 05	15.4	ENE 05	18.9	ENE 05	22
ENE 05	5.3	NNE 05	6.6	NNE 05	10.8	NNE 05	14.5	NNE 05	17.4	N 25	20.9
NE 05	5.3	N 05	6.4	N 05	10.3	N 05	14	N 15	17	N 15	20.3
NNE 05	5.1	N 15	6.1	N 15	9.9	N 15	13.5	N 25	17	NNE 05	20.3
N 05	4.9	N 25	6	N 25	9.6	N 25	13.5	N 05	16.7	N 05	19.4
NNW 15	4.9	NNW 05	5.4	NNW 05	8.4	NNW 05	11.3	NNW 05	14	NNW 05	16.6
NNW 05	4.7	NNW 15	5.1	NNW 15	8	NNW 15	10.9	NNW 15	13.6	NNW 15	16.3
WNW 05	3.9	NW 05	3.9	W 15	5.7	W 15	7.9	W 15	8.6	W 15	9
NW 05	3.8	W 05	3.9	WNW 15	5	WNW 15	7	WNW 15	8	WNW 15	8.4
W 05	3.8	WNW 05	3.9	W 05	4.2	W 05	5.9	W 05	6.7	W 05	7.1

LNP COL 2.4-2

Table 2.4.5-208 (Sheet 2 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 3, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
NW 15	3.7	W 15	3.7	NW 15	4	NW 15	5.6	NW 15	6.5	NW 15	6.9
W 15	3.5	NW 15	3.6	NW 05	3.9	NW 05	5.1	NW 05	6	NW 05	6.4
WNW 15	3.4	WNW 15	3.5	WNW 05	3.9	WNW 05	5.1	WNW 05	6	WNW 05	6.3

Notes:
E = east, W = west, N = north, S = south

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-209 (Sheet 1 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 4, near the LNP Site

Tropical storm		Category 1		Category 2		Category 3		Category 4		Category 5	
ENE 25	7.6	E 25	9.6	E 25	14.5	E 25	19	E 25	23.3	E 25	27.3
NE 25	7.5	ENE 25	9.3	ENE 25	14.3	ENE 25	19	ENE 25	23.2	ENE 25	26.8
E 25	7.5	E 15	8.6	E 15	13.4	E 15	17.8	NE 15	21.9	NE 15	25.9
NNE 25	7.1	NE 25	8.6	NE 25	13.1	NE 15	17.8	NE 25	21.8	E 15	25.7
ENE 15	6.4	ENE 15	8.3	ENE 15	13	NE 25	17.6	E 15	21.7	NE 25	25.6
NE 15	6.4	NE 15	8.0	NE 15	13	ENE 15	17.4	ENE 15	21.1	ENE 15	24.8
E 15	6.3	NNE 25	7.3	NNE 25	11.6	NNE 15	15.9	NNE 15	20	NNE 15	23.8
N 25	6.3	NNE 15	7.1	NNE 15	11.6	NNE 25	15.7	NNE 25	19.8	NNE 25	23.8
NNE 15	6.2	E 05	7.0	E 05	11.3	E 05	15.4	E 05	19.1	E 05	22.2
N 15	5.6	NE 05	7.0	NE 05	11.3	NE 05	15.4	NE 05	18.8	NE 05	22.2
ENE 05	5.2	ENE 05	6.8	ENE 05	10.9	ENE 05	14.7	ENE 05	17.9	ENE 05	21
NE 05	5.2	NNE 05	6.4	NNE 05	10.4	NNE 05	14.1	NNE 05	17.1	N 25	20.6
E 05	5.1	N 05	6.1	N 05	10	N 05	13.5	N 15	16.4	NNE 05	19.9
NNE 05	5.0	N 15	5.9	N 15	9.4	N 15	12.9	N 25	16.4	N 15	19.6
NNW 15	4.8	N 25	5.9	N 25	9.3	N 25	12.8	N 05	16.3	N 05	18.9
N 05	4.7	NNW 05	5.3	NNW 05	8.1	NNW 05	10.8	NNW 05	13.5	NNW 05	15.9
NNW 05	4.6	NNW 15	5.0	NNW 15	7.7	NNW 15	10.4	NNW 15	13	NNW 15	15.6
NW 05	3.8	NW 05	3.8	W 15	5.6	W 15	7.7	W 15	8.4	W 15	9.1
W 05	3.8	W 05	3.8	WNW 15	4.8	WNW 15	6.8	WNW 15	7.8	WNW 15	8.3
WNW 05	3.8	WNW 05	3.8	W 05	4.1	W 05	5.7	W 05	6.6	W 05	6.9

LNP COL 2.4-2

Table 2.4.5-209 (Sheet 2 of 2)
MOM Surge Heights (Feet NGVD29) at Coastal Point 4, near the LNP Site

Tropical Storm		Category 1		Category 2		Category 3		Category 4		Category 5	
NW 15	3.7	W 15	3.7	NW 15	3.9	NW 15	5.4	NW 15	6.3	NW 15	6.6
W 15	3.5	NW 15	3.6	NW 05	3.8	NW 05	4.9	NW 05	5.8	WNW 05	6.2
WNW 15	3.4	WNW 15	3.5	WNW 05	3.8	WNW 05	4.9	WNW 05	5.8	NW 05	6.1

Notes:
E = east, W = west, N = north, S = south

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-210
Maximum Coastal Line Surge Levels (Feet NGVD29) for Hurricanes of Categories 1 through 5 near the LNP Site

Case	Cat-1	Cat-2	Cat-3	Cat-4	Cat-5
1	10.2	15.9	20.6	24.9	28.9
2	10	15.6	20.3	24.3	28.9
3	10	15.6	20.1	24.2	28
4	9.6	14.5	19	23.3	27.3
Average	10.0	15.4	20.0	24.2	28.3

Notes:

Assumed tide = 2.5 ft. NGVD29.

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-211
Maximum Water Levels at the LNP Site, Yankeetown, and Inglis Obtained Using the SLOSH Model for Hurricanes of Categories 1 through 5

	Water Surface Elevation (Feet NGVD29) Using the SLOSH Model		
Category of Hurricane Storm	**Yankeetown** (29° 1'46.99" N, 82°42'58.00" W)	**Inglis** (29° 1'48.00" N, 82°40'8.00" W)	**LNP Site** (29° 4'26.72" N, 82°37'14.91" W)
CAT-1	10.40	Dry	Dry
CAT-2	16.30	15.8	Dry
CAT-3	21.20	22.10	Dry
CAT-4	25.80	27.00	Dry
CAT-5	29.70	31.00	Dry

Notes:

Assumed tide = 2.5 ft. NGVD29.

Source: Reference 2.4.5-207

LNP COL 2.4-2

Table 2.4.5-212
PMH Parameters Used for Hsu (2004) Method

Parameter	Value	Unit	Remarks
Central Pressure, P_0	890.4	mb	Table 2.4.5-204
Shoaling Factor, Fs	1.6	None	Figure 2.4.5-226
Correction Factor, Fm	0.7	None	Figure 2.4.5-227
Storm Surge Height, Sp	30.76	ft. msl	Equation 2.4.5 1

Notes:

mb = millibars, ft. = feet

Source: References 2.4.5-217, 2.4.5-205, 2.4.5-208

LNP COL 2.4-2

Table 2.4.5-213
Coastline Storm Surges for Category 1 through 5 Hurricanes

	Storm Surge Height (Feet NGVD29)	
Hurricane Storm Category	Hsu Method	SLOSH Model
1	9.7	9.5
2	11.8	14.9
3	16.3	19.5
4	22.1	23.7
5	25.2	27.8

Notes:

SLOSH model results presented in FSAR Table 2.4.5-210 were corrected for 10% high tide of 2.01 ft. NGVD29.

LNP COL 2.4-2

Table 2.4.5-214
Determination of Water Elevation at the LNP Plant Site by Extrapolation for Hurricanes of Categories 1 through 5

Location	Ground Elev. (ft.)	Distance from Sea (mi.)	Cat-1	Cat-2	Cat-3	Cat-4	Cat-5
Yankeetown	5	2.4	9.9	15.8	20.7	25.3	29.2
Inglis	15	5.97		15.3	21.6	26.5	30.5
Plant Site	51	8.5		15.0	22.2	27.4	31.4

Notes:

All elevations are in feet NGVD29. The nominal plant grade elevation is 50 ft. NAVD88 (51 ft. NGVD29).
ft. = foot, mi. = mile

LNP COL 2.4-2

Table 2.4.5-215
Results Summary

Item	Elevation NGVD29 (ft.)	Elevation NAVD88 (ft.)
LNP grade elevation	51	50
10% exceedance high tide	2.01	1.01
PMH storm surge w/o wind action including 10% exceedance high tide	42.54	41.54
Additional wind driven wave heights (wave setup)	6.08	6.08
Additional wind driven wave heights (wave runup)	0.90	0.90
Adjustment to long-term sea level rise	0.40	0.40
Sea Level Anomaly	0.60	0.60
Total PMH surge including wave effects	50.52	49.52

Note:

At the LNP site, NAVD88 (ft.) = NGVD29 (ft.) - 1 ft.

LNP COL 2.4-2

Table 2.4.5-216
PMH Parameters from NWS 23 Used for SLOSH Model Simulations

Parameter	Lower	Upper	Unit
Central Pressure, P_O	890	890	mb
Peripheral Pressure, P_W	1020	1020	mb
Pressure Deficit, $\Delta_p = P_W - P_O$	130	130	mb
Radius of maximum winds, R	7.5	26	Statute mi.
Forward speed, T	16.4	23	mph
Maximum wind speed[(a)]	152	155	mph
Track Direction, Θ	215	245	Degrees from North

Note:

a) NWS 23 contains several wind speed values. Shown here are those defined in NWS23 as the maximum gradient wind speed.

mb = millibar, mi. = mile, mph = miles per hour

LNP COL 2.4-2

Table 2.4.5-217
PMH Parameters for SLOSH Preliminary Runs

Landfall Location	Radius of Maximum Winds (mi.)	Forward Speed (mph)	Direction of Storm Track with Respect to North (degrees)
16 locations	7.5 17 26	16 20 23	215 225 235 245

Note:

mi. = mile

LNP COL 2.4-2

Table 2.4.5-218 (Sheet 1 of 2)
Minimum and Maximum Surges for Each Landfall Location from Preliminary Runs

Landfall Location	Preliminary Run #	Forward Speed, V (mph)	Track Direction, Θ (degrees)	Radius of Maximum Winds (mi.)	Surge Elevation at Site (ft. NAVD88)	Comment
				7.5	DRY	
				17	DRY	
1			No Surge Calculated			Northernmost Landfall
2	513	23	215	26	42.20	
	522	23	225	26	41.1	
3	495	23	235	26	41.00	
	477	23	215	26	42.60	
	486	23	225	26	42.60	
4	468	23	245	26	41.20	
	441	23	215	26	44.40	
5	402	20	215	26	41.00	
	405	23	215	26	44.80	
6	393	20	245	26	41.00	
	378	23	225	26	46.10	
7	357	20	245	26	41.20	
	351	23	235	26	46.30	
8	321	20	245	26	41.40	
	306	23	225	26	47.00	
9	285	20	245	26	41.30	
	270	23	225	26	46.90	
10	249	20	245	26	41.20	
	225	23	215	26	46.10	
	234	23	225	26	46.10	
11	213	20	245	26	41.00	
	198	23	225	26	46.20	
12	177	20	245	26	41.30	
	162	23	225	26	47.40	
13	141	20	245	26	42.30	
	126	23	225	26	46.90	
14	105	20	245	26	42.60	
	90	23	225	26	47.50	Highest Surge in Preliminary Runs

LNP COL 2.4-2

Table 2.4.5-218 (Sheet 2 of 2)
Minimum and Maximum Surges for Each Landfall Location from Preliminary Runs

Landfall Location	Preliminary Run #	Forward Speed, V (mph)	Track Direction, Θ (degrees)	Radius of Maximum Winds (mi.)	Surge Elevation at Site (ft. NAVD88)	Comment
15	69	20	245	26	42.70	
	54	23	225	26	47.10	
16	33	20	245	26	41.50	Southernmost Landfall

Note:

ft. = foot, mi. = mile

LNP COL 2.4-2

Table 2.4.5-219
Pressure Deficit Decay after Landfall

Time After Landfall (hr.)	Pressure Deficit (mb)
0	130
6	86
12	57
18	38
24	25

Note:

hr. = hour, mb = millibar

LNP COL 2.4-2

Table 2.4.5-220
Scenarios for the Change in Pressure Deficit with Respect to Landfall

Scenario	Pressure Deficit Profile
1	Pressure Deficit ΔP = 130mb constant
2	Pressure Deficit remains at maximum until landfall then decays exponentially after landfall according to the rate calculated in Table 2.4.5-219.
3	80% of Maximum: Until 12 hr. before landfall (Start to -12 hr.) Maximum from 12 hr. before land fall to Landfall(-12 to 0 hr.) Decays exponentially as per Table 2.4.5-219 after landfall (> 0 hr.)

Note:

mb = millibar, hr. = hour

LNP COL 2.4-2

Table 2.4.5-221
PMH Parameters for SLOSH Final Runs

Landfall Location	Radius of Maximum Winds (mi.)	Forward Speed (mph)	Direction of Storm Track with Respect to North (degrees)
26 locations	26	23	215 220 225 230 235 240 245

Note:

mi. = mile, mph = miles per hour

LNP COL 2.4-2

Table 2.4.5-222
Storm Parameters Producing the Maximum Surge for Different Scenarios

Scenario	Radius of Maximum Winds (mi.)	Forward Speed (mph)	Direction of Storm Track with Respect to North (degrees)	Surge at Site (ft. NAVD88)
1	26	23	225	47.7
2	26	23	225	47.3
3	26	23	230	46.8

Note:

mi. = mile, mph = miles per hour, ft. = feet

LNP COL 2.4-2

Table 2.4.5-223
Total PMH Surge Elevation at LNP Site Including Wave Effects

Component	Units	Scenario 1	Scenario 2	Scenario 3	Scenario with Higher Initial Water Level of 3.87 ft. NAVD88
LNP Site Grade Elevation	ft. NAVD88	50.00	50.00	50.00	50.00
10 percent Exceedance Spring High Tide Elevation Including the Initial Rise.	ft. NAVD88	3.23	3.23	3.23	3.28
Long-term sea Level Rise	ft.	0.59	0.59	0.59	0.59
Initial Water Level	ft. NAVD88	3.82	3.82	3.82	3.87
SLOSH Surge Elevation With Initial Water Level Including 10 percent Exceedance High Tide, Initial Rise and Long-term Sea Level Rise	ft. NAVD88	47.70	47.30	46.80	47.70
Wave Setup	ft.	0.60	0.50	0.40	0.60
Wave Runup	ft.	1.48	0.90	0.23	1.48
TOTAL PMSS Including Wave Effects	ft. NAVD88	49.78	48.70	47.43	49.78

Note:

ft. = foot/feet

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 1 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
1530 09 01	10.7N 64.1W	MMI=X	Venezuela	Venezuela:		
[14:30 UT]				Paria	7.3	
				Cumana	6.0	
				Cubagun Island	6.0	
				Gulf of Cariaco		
1690 04 16	17.5N 61.5W	M_S 8.0	Leeward Is.	U.S. Virgin Islands:		
				St. Thomas:		
				Charlotte Amalie		
				Nevis:		
				Charleston		
1692-06-07	17.8N 76.7W	M_S 7.7	Jamaica	Jamaica:		
[11:43 LT]				Port Royal	1.8	2000
				Liganee (Kingston)		
				Saint Ann's Bay		
1751-10-18	18.5N 70.7W	M_S 7.3	Hispaniola	Hispaniola:		
[19:00 UT]				Azua de Compostela		
				Santa Domingo		
				Santa Cruz El Seybo		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 2 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
1755-11-01	36.0N 11.0 W	MMI = XI	Lisbon, Portugal	Netherlands Antilles		
[9:50 LT]				Saba	7.0	
				St. Martin	4.5	
				Antigua	3.6	
				Dominica	3.6	
				Barbados	1.5-1.8	
				Martinique		
				Cuba:		
				Santiago de Cuba		
1755-11-18	42.7N 70.3W	VIII	Cape Ann, Massachusetts	St. Martins, West Indies		
1761-03-31 [12:05 LT]	37.0N I0.0W	MMI = IX	Lisbon, Portugal	Barbados	1.2	
1767-04-24	14.4N 61.OW		Martinique and	Martinique		
[6:00 UT]			Barbados	Barbados		
1770-06-03 [19:15 LT]	18.3N 72.2W		Haiti	Golfe de la Gonave and Arcahaie		
1802-05-05	9.2N 61.5W		Venezuela	Venezuela: Orinoco River		
1823-11-30	14.4N 61.OW		Martinique	Martinique:		
[3: 10 LT]				Saint-Pierre Harbor		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 3 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
1842-05-07	19.IN 72.8W	M_S 7.7	Haiti	Haiti:		-5000
[17:30 LT]				Mole St. Nicolas		
				Cap Haitien		
				Port-de-Paix	5.0	200-300
				Forte-Liberte		
				Santiago De 10s Caballeros		
				Dominican Republic		
				Santa Domingo	2.0	
				U.S. Virgin Islands		
				St. John	3.1	
				North coast of Hispaniola	2	
1843-02-08	16.5N 62.2W	MMI=IX	Guadeloupe	Antigua	1.2	
[14:50 UT]						
1853-07-15	12.1N 63.6W	M_S 6.7	Venezuela	Venezuela:		
				Cumana		
				Puerto Sucre		
				Sabana de Caiguire		
				Sabana de Salgado		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 4 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
1856-08-09	16.0N 88.0W	M_S 7.5	Honduras	Honduras:	5.0	
				Rio Patuca		
				Omoa		
				Cortez		
				Atlantida		
				Trujillo		
1860-03-08	19.0N 72.0W		Hispaniola	Hispaniola:		
				Golfe de la Gonave		
				Les Cayes		
				Acquin		
				Anse -A-Veau		
1867-11-18 [18:45 UT]	18.0N 65.5W	M_S 7.5	St. Croix and St. Thomas, U.S. Virgin Islands	Guadeloupe:	19.8	
				Dechaies	18.3	23
				Basse-Terre	1.0	
				Sainte-Rose	10.0	
				Isles des Saintes		
				Grande Terre		
				Fond-du-Cure	1	
				Pointe-a-Pure		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 5 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
				U.S. Virgin Islands:		
				St. Thomas:		
				Charlotte Amalie	4.5-6.0	12
				Hassle Island	4.9	
				Altona		
				St. John		
				St. Croix:	7.0-9.0	
				Christiansted		
				Frederiksted	7.6	5
				Gallows Bay		
				Puerto Rico:	1.0-6.0	
				Arroyo	0.9-1.5	
				San Juan	0.9-1.5	
				Vieques Islands	6.1	
				Fajardo		
				Puerto Yabucoa	1.37	
				British Virgin Islands:		
				Peter Island	1.2-1.5	

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 6 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
				Tortola		
				Road town	1.5	
				Netherrlands Antilles:		
				Saba		
				St. Kitts and Nevis:		
				St. Christoopher (St. Kitts)		
				Netherlands and France		
				St. Martin		
				France		
				St. Barthelemy		
				Antigua and Barbuda:		
				St. Johns	2.4-3.0	
				Martinique	3.0	
				St. Vincent and the Grenadines:		
				Becquina	1.8	
				Grenada:	3.0	
				St. Georges	1.5	
				Charlotte Town (Gouyave)	3.0	
				Venezuela		
				Maiquetia Island		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 7 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
1882-09-07	1.3N 77.8W	M_S 8.0	Panama	Panama:		
[7:50 UT]				San Bias Archipelago	3.0	75-100
1883-08-27	5.8s 106.3E		Krakatoa,	U.S. Virgin Islands:		
[l0:00 LT]			Indonesia	St. Thomas		
1887-09-23	19.7N 74.4W		Haiti	Haiti:		
[12:00 UT]				Mole-Saint-Nicolas		
				Jeremie		
				Anse-d'Hainault		
				Point Tiburon		
1900-10-29	10.9N 66.8W	M_S 8.4	Venezuela	Venezuela		
				Macuto		
				Puerto Tuy	10.0	
1902-08-30	14.4N 61.0W		Martinique	Martinique	1.0	
[21:25 LT]				Fort-de-France		
1906-01-31	2.4N 19.3W	M_S 8.9	Venezuela	Venezuela:		
[15:36 UT]				Cumana		
				Campano		
				Costas Nueva Esparta		
				Rio Caribe		
				Isla de Margarita		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 8 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
1907-01-14	8.1N 76.7W	M_S 6.5	Jamaica	Jamaica:		
[21:36 UT]				Hope Bay	2.5	
				Orange Bay	2.5	
				Sheerness Bay	2.5	
				St. Ann's Bay	2.5	
				Annotto Bay	1.8-2.4	
				Port Maria	1.8-2.4	
				Ocho Rios		
				Bluff Bay		
				Port Antonia		
				Kingston	2.5	
1911-11-03	10.5N 61.2W		Trinidad	Trinidad		
1916-04-24	11.0N 85.0W	M_S 7.6	Panama	Panama:		
[8:02 UT]				Almirante		
				Bocas del Toro		
				Isla de Carenero		
				Isla Bastimento		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 9 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
1918-10-11	18.5N 61.5W	M_S 7.5	Puerto Rico	Puerto Rico:		140
[4:14 UT]				Aguadilla	2.4-3.4	32
				Punta Agujereada	5.5-6.1	8
				Punta Higuero	5.2	
				800 m SE of Punta Higuero	2.6-2.7	
				Punta Borinquen	4.5	
				Isla Mona	3.0	
				Rio Culebrinas	4.0	
				Bahia de Boqueron	0.9	
				800 m SE at bay entrance	0.4	
				Isabella	2.0	
				Cayo Cardona	0.75	
				Guanica	0.5	
				Mayaguez	1.5	
				Isla Caja de Muertos	1.5	
				Puerto Arecibo	0.6	
				Rio Grande	0.1	
				Rio Grande de Loiza	1.0	
				Playa Ponce		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 10 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	Lat. Long.	Earthquake Magnitude	Area	Location of Effect	Runup (m)	Deaths
				St. Thomas		
				Krum Bay	1.2	
				Charlotte Amalie	0.45	
				Dominican Republic		
				Santo Domingo (Rio Ozama)	0.7	
				U.S. Virgin Islands	0.3-0.6	
				Tortola		
1918-10-24	18.5N 67.5W		Puerto Rico	Mona Passage		
[3:43 UT]				Puerto Rico		
				Texas		
				Galveston		
1929-01-17	10.6N 65.6W	M_S 6.9	Venezuela	Venezuela:		
[11:52 UT]				Cumana		
				Manicuare		
				El Dique		
				El Barbudo		
				El Salado		
				Puerto Sucre		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 11 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
1939-08-15	22.5N 79.2W	M_S 8.1	Cuba	Cuba:		
[3:52 UT]				Cayo Frances		
1946-08-04	19.3N 68.9W	M_S 8.1	Dominican Republic, Haiti and Puerto Rico	Dominican Republic:		
[17:51 UT]				Matancitas	2.5	1790
				Julia Molina	4.0-5.0	
				Cabo Samana		
				Puerto Rico:		
				San Juan		
				Bermuda		
				Florida:		
				Daytona Beach		
				New Jersey:		
				Atlantic City		
1946-08-08	19.5N 69.5W	M_S 7.9	Puerto Rico	Puerto Rico:		75
[3:28 UT]				Aguadilla		
				Mayaguez		
				San Juan		
				Bermuda		
				Florida:		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 12 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
				Daytona Beach		
				New Jersey		
				Atlantic City		
1969-12-25	15.8N 59.1W	M_S 7.6	Leeward Is.	Barbados	0.46	
[21:32 UT]				Antigua	0.3	
				Dominica	0.12	
1985-03-16	17.0N 62.4W	M_S 6.8	Leeward Is.	Guadeloupe	0.1	
14:54				Basse-Terre		
1989-11-01	19.0N 68.8W	M_b 5.2	Costa Rica	Panama:		
[10:25 UT]				Bocas del Toro	0.6	
				Isla de Carenero		
				San Cristobal Island		
				Bastimento	0.1	
				Cristobal	0.1	
				Portobelo	0.6	
				Colon		
				Coca Solo	0.8	
				Costa Rica		
				Limon		

LNP COL 2.4-2

Table 2.4.6-201 (Sheet 13 of 13)
Verified and Probable Caribbean Tsunamis (1498 – 2000)

Origin Data				Effects Data		
Date	**Lat. Long.**	**Earthquake Magnitude**	**Area**	**Location of Effect**	**Runup (m)**	**Deaths**
				Punta Cahuita-Puerto Viejo	2.0	
				U.S. Virgin Islands		
				St. Croix		
				Limetree	0.07	
1997-07-09 [19:24 UT]	10.6N 63.5W	M_w 7.0	Venezuela	Venezuela:		
				Isla de Margarita		
				Tobago		
1997-12-26 [3:00 LT]	16.7N 62.2W		Montserrat	Montserrat	3.0	

Notes:

LT = local time
m = meter
MMI = Modified Mercalli Intensity
M_S = surface-wave magnitude
M_w = moment magnitude
UT = universal time

Source: Reference 2.4.6-209

LNP COL 2.4-2

Table 2.4.6-202 (Sheet 1 of 3)
Tsunami Events Affecting the Caribbean

Date						Tsunami Cause				Tsunami Source Location				Tsunami Parameters						Tsunami Effects				
																Magnitude								
Year	Month	Day	Hour	Minute	Sec.	Validity[a]	Source Code[b]	Earthquake Magnitude	Volcano	Country	Name	Latitude	Longitude	Max Water Height	Num. of Runups	Abe	Iida	Tsunami Intervals	Warn Status	Number of Deaths	Number of Injuries	Damage (Millons of Dollars)	Number of Houses Destroyed	Photos
1530	9	1	14	30		4	1	*		Venezuela	Cumana	11	-64	7.3	3		2.6							
1690	4	16				4	1	8		Antigua And Barbuda	Antigua; Saint Kitts And Nevis	18	-62		3									
1692	6	7				4	3	7.7		Jamaica	Port Royal	18	-77	1.8	4					2000				
1755	11	1				4	1	*		Portugal	Lisbon	36	-11	30	52		3.6							
1761	3	31	12	5		4	1	*		Portugal	Lisbon	37	-10	2.4	11		1.3							
1767	4	24	6			3	1	*		Martinique	Martinique & Barbados	14	-61		2									
1770	6	3				4	1	*		Haiti	Port-Au-Prince	19	-72		4									
1775	2	11				3	1	*		Cuba	Santiago De Cuba	20	-76		2									
1802	8	15				3	1	*		Venezuela	Cumana	10	-64		1									
1823	11	30				4	1	*		Martinique	Saint Pierre	14	-61		1									
1842	5	7				4	1	7.7		Haiti	Cap-Haitian	20	-72	5	8		4.2			300				
1843	2	8	14	50		4	1	8.3	Volcano	Guadeloupe	Pointe-A-Pitre	17	-62	1.2	1		0.3							
1853	7	15				3	1	6.7		Venezuela	Cumana	11	-64		4					113				
1856	8	9				4	1	7.5		Honduras	Omoa	16	-88	5	5									
1860	4	8				4	1	7.5		Haiti	Anse-A-Veau	19	-73		4									
1867	11	18	18	45		4	1	7.5		USA Territory	Virgin Islands	18	-65	10	33		2.3			30				
1868	3	17	11	37		3	1	*		USA Territory	Virgin Islands	18	-65	1.5	5		-1							
1882	9	7	7	50		4	3	7.9		Panama	San Blas Archipelago	9.5	-79	3	4			1		100				
1883	8	27	2	59		4	6		Volcano	Indonesia	Krakatau	-6	105	35	75		5.1	5		36,500				
1887	9	23	12			4	1	*		Haiti	Mole Saint-Nicolas	20	-74		4									
1900	10	29				3	1	8.4		Venezuela	Mancuto	11	-67	10	4									
1902	5	5				4	7		Volcano	Martinique	Mont Pelee	15	-61	5	1									
1902	5	7				4	6		Volcano	Saint Vincent And The Grenadines	Soufriere Volcano	13	-61		3									
1902	8	30				4	6		Volcano	Martinique	Mount Pelee	15	-61	1	1									

LNP COL 2.4-2

Table 2.4.6-202 (Sheet 2 of 3)
Tsunami Events Affecting the Caribbean

Date						Tsunami Cause				Tsunami Source Location				Tsunami Parameters						Tsunami Effects				
																Magnitude								
Year	Month	Day	Hour	Minute	Sec.	Validity[a]	Source Code[b]	Earthquake Magnitude	Volcano	Country	Name	Latitude	Longitude	Max Water Height	Num. of Runups	Abe	Iida	Tsunami Intervals	Warn Status	Number of Deaths	Number of Injuries	Damage (Millons of Dollars)	Number of Houses Destroyed	Photos
1906	1	31				4	1	*		Venezuela	Caracas	11	-67		6									
1907	1	14	21	36		4	1	6.5		Jamaica	Jamaica	18	-77	2.5	10		1.3							
1911	11	3				3	6		Volcano	Trinidad And Tobago	Trinidad	11	-61		1									
1916	4	25	8	2		4	1	7.6		Panama	Bocas Del Toro	9.3	-82	1.3	5									
1918	10	11	14	14	30	4	1	7.3		USA Territory	Puerto Rico: Mona Passage	19	-68	6.1	21		2.6			142		4		
1918	10	24	3	43		4	1	*		USA Territory	Puerto Rico	19	-68		2									
1929	1	17	11	52		4	1	6.9		Venezuela	Cumana	11	-66		5									
1929	11	18	20	32		4	3	7.4		Canada	Grand Banks, Newfoundland	45	-56	7	45		2.2			28		1		
1946	8	4	17	51	6	4	1	8.1		Dominican Republic	Northeastern Coast	19	-69	5	8		2.2			1790				
1946	8	8	13	28		4	1	7.9		Dominican Republic	Northeastern Coast	20	-70	0.6	13					75				
1967	7	29	23	59	59	4	1	6.5		Venezuela	Caracas	11	-67	0.1	1									
1969	12	25	21	32		4	1	7.6		Guadeloupe	Grand Bourg	16	-60	0.5	3		-3							
1985	3	16	14	54	0.7	4	1	6.4		Guadeloupe	Guadeloupe	17	-62	0.1	1		-3							
1989	11	1	10	25	52	3	1	4.4		USA Territory	Puerto Rico	19	-69	0.1	1									
1991	4	22	21	56	52	4	1	7.7		Costa Rica	Limon, Pandora	9.7	-83	3	21		1			2				
1997	7	9	19	24	13	3	1	7		Venezuela	Cariaco-Cumana	11	-63		1									
1997	12	26	8			4	7		Volcano	Montserrat	White River Valley	17	-62	3	1									
1999	1	20				4	6		Volcano	Montserrat	Soufriere Hills Volcano	17	-62	2	3									

LNP COL 2.4-2

Table 2.4.6-202 (Sheet 3 of 3)
Tsunami Events Affecting the Caribbean

Date						Tsunami Cause				Tsunami Source Location				Tsunami Parameters						Tsunami Effects				
																Magnitude								
Year	Month	Day	Hour	Minute	Sec.	Validity[(a)]	Source Code[(b)]	Earthquake Magnitude	Volcano	Country	Name	Latitude	Longitude	Max Water Height	Num. of Runups	Abe	Iida	Tsunami Intervals	Warn Status	Number of Deaths	Number of Injuries	Damage (Millons of Dollars)	Number of Houses Destroyed	Photos
2003	7	12				4	6		Volcano	Montserrat	Soufriere Hills Volcano	17	-62	4	5									
2004	11	21	11	41	7.7	4	1	6.3		Guadeloupe	Basse-Terre, Les Saintes	16	-62	0.7	7									
2004	12	26	0	58	53	4	1	9		Indonesia	Off W. Coast Of Sumatra	3.3	96	50	716				4	250,000		10,000		39
2006	5	20	11	20		4	6		Volcano	Montserrat	Soufriere Hills Volcano	17	-62	1	4				4					

Notes:

Abe = Abe defined two different tsunami magnitude amplitudes. His first tsunami magnitude (1979) is Mt = logH + B, where H is the maximum single crest or trough amplitude of the tsunami waves (in meters) and B a constant. The second definition (1981) is Mt = logH + alogR + D, where R is the distance in km from the earthquake epicenter to the tide station along the shortest oceanic path, and a and D are constants.
Iida = Iida and others (1967) defined tsunami magnitude (M) as M = log2h, where "h" is the maximum runup height of the wave.

a) Validity:
0 = erroneous entry
1 = very doubtful tsunami
2 = questionable tsunami
3 = probable tsunami
4 = definite tsunami

b) Source Code:
0 = Unknown Cause
1 = Earthquake
2 = Questionable Earthquake
3 = Earthquake and Landslide
4 = Volcano and Earthquake
5 = Volcano, Earthquake, and Landslide
6 = Volcano
7 = Volcano and Landslide
8 = Landslide
9 = Meteorological
10 = Explosion
11 = Astronomical Tide

Source: Reference 2.4.6-206

LNP COL 2.4-2

Table 2.4.6-203
Tsunami Events Affecting the Gulf of Mexico

														Tsunami Parameters						Tsunami Effects				Photos
Date						Tsunami Cause				Tsunami Source Location						Magnitude								
Year	Month	Day	Hour	Minute	Sec.	Validity[a]	Source Code[b]	Earthquake Magnitude	Volcano	Country	Name	Latitude	Longitude	Max Water Height	Num. of Runups	Abe	Iida	Tsunami Intervals	Warn Status	Number of Deaths	Number of Injuries	Damage (Millons of Dollars)	Number of Houses Destroyed	
1918	10	24	3	43		4	1			USA Territory	Puerto Rico	18.5	-68		2									
1964	3	28	3	36	14	4	3	9.2		USA	Prince William Sound, AK	61.1	-148	67	373		6.1	5		221		124		8

Notes:

a) Validity:
0 = erroneous entry
1 = very doubtful tsunami
2 = questionable tsunami
3 = probable tsunami
4 = definite tsunami

b) Source Code:
0 = Unknown Cause
1 = Earthquake
2 = Questionable Earthquake
3 = Earthquake and Landslide
4 = Volcano and Earthquake
5 = Volcano, Earthquake, and Landslide
6 = Volcano
7 = Volcano and Landslide
8 = Landslide
9 = Meteorological
10 = Explosion
11 = Astronomical Tide

Source: Reference 2.4.6-206

LNP COL 2.4-2

Table 2.4.6-204 (Sheet 1 of 2)
Mareogram Summary for Tsunami Source 1: Puerto Rico Trench

Location	Region	Travel Time (hr-min)	Peak Height (cm)	Initial Motion	Period (hr-min)
Brownsville, TX	Gulf	6 hr. 22 min	4	depression	2 hr. 3 min
Corpus Christi, TX	Gulf	6 hr. 45 min	4	depression	1 hr. 18 min
Galveston, TX	Gulf	8 hr. 2 min	6	depression	1 hr. 58 min
High Island, TX	Gulf	8 hr. 30 min	3	depression	1 hr. 57 min
Eugene Island, LA	Gulf	8 hr. 10 min	3	depression	1 hr. 56 min
Port Fourchon, LA	Gulf	5 hr. 52 min	10	depression	2 hr. 3 min
Grand Isle, LA	Gulf	6 hr.	12	depression	1 hr. 38 min
Waveland, MS	Gulf	10 hr. 36 min	1	depression	
Biloxi, MS	Gulf	8 hr. 28 min	5	depression	2 hr. 5 min
MS – AL Border	Gulf	9 hr. 35 min	3	depression	2 hr. 2 min
Destin, FL	Gulf	5 hr. 38 min	7	depression	1 hr. 55 min
Suwanee, FL	Gulf	8 hr. 37 min	3	depression	2 hr. 2 min
Panama Beach, FL	Gulf	5 hr. 47 min	5	depression	1 hr. 54 min
Panama City, FL	Gulf	6 hr. 20 min	11	depression	2 hr. 2 min
Clearwater Beach, FL	Gulf	6 hr. 58 min	8	depression	1 hr. 6 min
St Petersburg, FL	Gulf	7 hr. 48 min	5	depression	2 hr. 56 min
Tampa, FL	Gulf	8 hr. 28 min	5	depression	2 hr. 28 min
Port Manatee, FL	Gulf	7 hr. 28 min	5	depression	1 hr. 28 min
Bonita, FL	Gulf	7 hr. 37 min	25	depression	1 hr. 50 min
Naples, FL	Gulf	7 hr. 28 min	23	depression	1 hr.
Virginia Key, FL	Atlantic	2 hr. 57 min	15	elevation	49 min
Ocean Reef, FL	Atlantic	3 hr. 13 min	28	elevation	1 hr. 40 min
Jupiter, FL	Atlantic	2 hr. 47 min	54	elevation	1 hr. 2 min
Flagler, FL	Atlantic	4 hr. 18 min	117	elevation	1 hr. 10 min
Vaca Key, FL	Atlantic	4 hr.	13	elevation	1 hr. 11 min
St Simons, GA	Atlantic	5 hr. 30 min	40	elevation	1 hr. 13 min
Altamaha, GA	Atlantic	5 hr. 33 min	47	elevation	1 hr. 15 min
So Santee, SC	Atlantic	4 hr. 32 min	77	elevation	1 hr. 22 min

LNP COL 2.4-2

Table 2.4.6-204 (Sheet 2 of 2)
Mareogram Summary for Tsunami Source 1: Puerto Rico Trench

Location	Region	Travel Time (hr-min)	Peak Height (cm)	Initial Motion	Period (hr-min)
Springmaid, SC	Atlantic	4 hr. 57 min	129	elevation	1 hr. 8 min
Charleston, SC	Atlantic	4 hr. 57 min	49	elevation	1 hr. 15 min
Surf City, NC	Atlantic	4 hr. 23 min	112	elevation	1 hr. 8 min
Beaufort, NC	Atlantic	3 hr. 38 min	147	elevation	45 min
Oregon Inlet, NC	Atlantic	3 hr. 45 min	38	elevation	42 min
Duck, NC	Atlantic	3 hr. 57 min	140	elevation	drained
Currituck, NC	Atlantic	4 hr. 15 min	102	elevation	36 min
Chesapeake B, VA	Atlantic	7 hr. 12 min	6	elevation	46 min
Annapolis, MD	Atlantic	10 hr. 28 min	3	elevation	~2 hr.
Cape Henlopen, DE	Atlantic	4 hr. 52 min	64	elevation	42 min
Cape May, NJ	Atlantic	5 hr.	68	elevation	45 min
Atlantic City, NJ	Atlantic	4 hr. 45 min	155	elevation	45 min
Montauk, NY	Atlantic	4 hr. 48 min	68	elevation	16 min
Bar Harbor, ME	Atlantic	5 hr. 33 min	71	elevation	6 min
D41424 (32.4N, 73W)	Atlantic	1 hr. 52 min	35	elevation	
D41420 (23.3N, 67.6W)	Atlantic	32 min	131	elevation	
D41421 (23.4N, 63.9W)	Atlantic	31 min	175	elevation	
D7-2 (38.6N, 68 W)	Atlantic	2 hr. 10 min	78	elevation	
D42407 (23.4N, 63.9W)	Caribbean	10 min	-61	depression	
D8-1 (25.4N, 86.8W)	Gulf	3 hr. 27 min	-2	depression	
Bermuda	Atlantic	1 hr. 57 min	511	elevation	12 min
Limetree, St Croix	Caribbean	1 min	240	depression	15 min
Punta, Guayanilla	Caribbean	0 min	173	elevation	21 min

Notes:

hr. = hour
min = minute
cm = centimeter

Source: Reference 2.4.6-225

LNP COL 2.4-2

Table 2.4.6-205
Source Parameters and Range of Average Slip and Moment Magnitudes of Earthquakes from which Tsunami Simulations Were Computed by USGS

Fault #*	Name	Type	Length (km)	Width (km)	Strike (°)	Dip (°)	Rake (°)	Avg. Slip-low (m)	Avg. Slip-high (m)	Moment Magnitude (low)	Moment Magnitude (high)
1	W. Cayman	Oceanic Transform Fault	746	15	N73E	83N	185	10.6	12.4	8.3	8.35
2	E. Cayman	Oceanic Transform Fault	915	15	N77E	80S	175	12.1	14.2	8.4	8.45
3a	Hispaniola	Subduction Zone	525	50	N98E	20S	70	8.2	9.4	8.8	8.84
3b	Puerto Rico	Subduction Zone	385	50	N83E	20S	23				
3c	Virgin Islands	Subduction Zone	485	50	N102E	20S	42				
4a	W. Northern Panama	Oceanic Convergent Boundary	200	40	N113E	30S	90	3.7	4.3	8.24	8.28
4b	E. Northern Panama	Oceanic Convergent Boundary	350	40	N75E	35S	90				
5a	W. Southern Caribbean	Subduction Zone	550	50	N53E	17S	90	4.7	5.4	8.46	8.5
5b	E. Southern Caribbean	Subduction Zone	200	50	N95E	17S	90				

Notes:
* = Faults with same numeral are treated as one tsunami source.
km = kilometer
m = meter

Source: Reference 2.4.6-214

LNP COL 2.4-2

Table 2.4.6-206
Tsunami Source Parameters for the Venezuela Seismic Source

Seismic Source: Fault	Fault Length (km)	Fault Width (km)	Strike Angle	Dip Angle	Rake Angle (º)
Venezuela: W. Southern Caribbean	550	50	N53E	17S	90
Venezuela: E. Southern Caribbean	200	50	N95E	17W	90

Notes:

km = kilometer

Source: Reference 2.4.6-238

Seismic Source: Fault	Fault Length (km)	Fault Width (km)	Strike Angle	Dip Angle	Rake Angle (º)
Venezuela: W. Southern Caribbean	550	50	N53E	17S	90
Venezuela: E. Southern Caribbean	200	50	N95E	17W	90

LNP COL 2.4-2

Table 2.4.6-207
Tsunami Source Parameters for the Landslide Sources

Landslide Source	Total Volume V_b (km^3)	Area A (km^2)	Excavation Depth (m)	Runout Distance (km from toe)
Mississippi Canyon	425.54	3687.26	~300	297
Florida Escarpment	16.2	647.57	~150	Uncertain

Notes:

km^3= cubic kilometers, km^2 = square kilometers, km = kilometer,

Source: Reference 2.4.6-238

LNP COL 2.4-2

Table 2.4.6-208
Probable Maximum Tsunami Simulation Results Using FUNWAVE

Scenario	Maximum Wave Amplitude at Five Miles Offshore (ft.) [(a)]	Maximum Wave Amplitude at Shoreline (ft.) [(a)]	Maximum Extent of Inundation Distance from Shoreline (mi.) [(b)]	Maximum Extent of Inundation Distance from LNP Site (mi.) [(b)]	Maximum Water Level Nearest the LNP Site (ft. NAVD88)	Grade Elevation / Floor Elevation of LNP Safety-Related Facilities (ft. NAVD88)
Venezuela Seismic Source	0.22	0.30	4.2	9.2	4.27	50.0 / 51.0
Mississippi Canyon Landslide – Static Source	9.93	9.11	9.7	3.7	12.94	50.0 / 51.0
Mississippi Canyon Landslide – Dynamic Source	10.05	8.93	9.6	3.8	11.66	50.0 / 51.0
Florida Escarpment Landslide – Static Source	0.27	0.29	4.2	9.2	4.33	50.0 / 51.0
Florida Escarpment Landslide – Dynamic Source	0.25	0.27	4.2	9.2	4.30	50.0 / 51.0

Notes:
a) The tsunami wave amplitudes presented in the table are measured above the initial water level of 3.87 ft. NAVD 88.
b) The distances presented in the table are measured along latitude 29.075N. The LNP site is located approximately 13.4 mi. from the shoreline.

ft. = feet, mi. = mile

LNP COL 2.4-4

Table 2.4.12-201
U.S. Department of Agriculture (USDA) Soil Summary

Soil Name	Depth (in.)	USDA Texture	Unified Classification	Fragment >10 Inches (%)	Fragment 3-10 Inches (%)	Sieve No. 200 (%)	Organic Matter (%)	Available Water Capacity[a] (in/in)	Moist Bulk Density[b] (g/cm^3)	Porosity[c] (cm^3/cm^3)	Saturated Hydraulic Conductivity[d] (micro m/sec)	pH[e]
Smyrna	0-5	Fine sand	SP, SP-SM	0	0	2-12	1.0-5.0	0.03-0.07	1.35-1.50	0.43-0.49	42-141	3.5-7.3
	5-19	Fine sand, sand	SP, SP-SM	0	0	2-12	0.0-0.5	0.03-0.07	1.35-1.50	0.43-0.49	42-141	3.5-7.3
	19-23	Fine sand, loamy fine sand, sand	SM, SP-SM	0	0	5-20	1.5-6.0	0.10-0.20	1.30-1.45	0.45-0.51	4-42	3.5-7.3
	23-80	Fine sand, sand	SP, SP-SM	0	0	2-10	0.0-0.5	0.03-0.07	1.45-1.70	0.36-0.45	42-141	4.5-5.5
Immokalee	0-9	Fine sand	SP, SP-SM	0	0	2-10	1.0-2.0	0.05-0.10	1.20-1.50	0.43-0.55	42-141	3.5-6.0
	9-38	Fine sand, sand	SP, SP-SM	0	0	2-10	0.0-0.5	0.02-0.05	1.45-1.70	0.36-0.45	42-141	3.5-6.0
	38-43	Fine sand, sand	SM, SP-SM	0	0	5-21	2.0-5.0	0.10-0.25	1.30-1.70	0.36-0.51	4-14	3.5-6.0
	43-80	Fine sand, sand	SP, SP-SM	0	0	2-10	0.0-0.3	0.02-0.05	1.40-1.70	0.36-0.47	42-141	3.5-6.0
Basinger	0-6	Sand	SP-SM	0	0	5-12	0.5-4.0	0.03-0.07	1.40-1.55	0.42-0.47	141-353	4.5-6.0
	6-35	Sand	SP-SM	0	0	5-12	0.0-0.5	0.05-0.10	1.40-1.55	0.42-0.47	141-353	5.6-7.8
	35-64	Sand	SP-SM	0	0	5-12	0.5-2.0	0.10-0.15	1.40-1.65	0.38-0.47	141-353	5.6-7.8
	64-80	Sand	SP-SM	0	0	5-12	0.0-0.5	0.05-0.10	1.50-1.70	0.36-0.43	141-353	5.6-7.8

Notes:

in. = inch, in/in = inch per inch, g/cm^3 = gram per cubic centimeter, cm^3/cm^3 = cubic centimeter per cubic centimeter, micro m/sec = micro meter per second
a) Available water capacity refers to the quantity of water that the soil is capable of storing for use by plants. The capacity for water storage in given in inches of water per inch of soil for each soil layer.
b) Moist bulk density is the weight of soil (oven dry) per unit volume. The moist bulk density of a soil indicates the pore space available for water and roots. Depending on soil texture, a bulk density of more than 1.4 can restrict water storage and root penetration.
c) Porosity was calculated using the following equation: Porosity = 1 - (Bulk Density/ Particle Density), where particle density is assumed to equal 2.65 grams per cubic centimeter (g/cm^3)
d) Saturated hydraulic conductivity refers to the ease with which pores in a saturated soil transmit water.
e) pH refers to the soil pH range in water.
SP = poorly graded sand
SP-SM = poorly graded sand and silty sand
SM = silty sand

Source: Reference 2.4.12-207

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 1 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Citrus	16	18	31	1								
	16	18	33	15	2	1						
	16	18	34	19	2							10
	17	16	3	1								
	17	16	4	11		3						28
	17	16	5	6		3						1
	17	16	8	3								24
	17	16	9							1		38
	17	16	10	6	1		1	1				10
	17	16	11	7	7	1						4
	17	16	12	36								7
	17	16	13	43		1						1
	17	16	14	1		1						
	17	16	15	2								2
	17	16	16	1								
	17	16	17	1								6
	17	16	18	2								
	17	16	20	1								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 2 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	17	16	21	4								
	17	16	23			2						1
	17	16	24	30	1	1						
	17	16	25	2								
	17	16	27									9
	17	16	28	1	1							28
	17	16	30			1						
	17	16	31	1								
	17	16	32	1							6	17
	17	16	33	1		1	1					52
	17	16	34	5								
	17	16	35	5								
	17	16	36	1			1					6
	17	17	1	100		2						6
	17	17	2	15								1
	17	17	7	10								20
	17	17	9	74	3	2						1
	17	17	10	109		1						2

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 3 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	17	17	11	75	4	2					1	16
	17	17	12	65		2						2
	17	17	13	26		1						1
	17	17	14	57	2	1						2
	17	17	15	115	2					1		2
	17	17	16	104		1						2
	17	17	17	22								3
	17	17	18	43								3
	17	17	19	103	1	1						7
	17	17	20	87								
	17	17	21	154		4				1		4
	17	17	22	85	1	2						1
	17	17	23	113	4	5						15
	17	17	24	91		2						1
	17	17	25	67		1						1
	17	17	26	96	2	2					1	21
	17	17	27	44	1							13
	17	17	28	59		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 4 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	17	17	29	41	1							1
	17	17	30	16	2							1
	17	17	31	26	6	3	4		7		1	56
	17	17	32	6	2	1		1	3			4
	17	17	33	59		2						2
	17	17	34	52	4	2						1
	17	17	35	55								
	17	17	36	50								2
	17	18	1	27	3	3						2
	17	18	2	19	2	3						26
	17	18	3	28		5						10
	17	18	4	69	4	6					1	5
	17	18	5	73	1	4						4
	17	18	6	99	2	2		1				9
	17	18	7	60		1						
	17	18	8	63		6				1		
	17	18	9	52		3						
	17	18	10	5		3						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 5 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Citrus cont.	17	18	11	7		2						15
	17	18	12	6		7						1
	17	18	13	4		5						
	17	18	14	5		1						8
	17	18	15	3								
	17	18	16	7		3						
	17	18	17	30	1	2						4
	17	18	18	48		1						
	17	18	19	109								3
	17	18	20	3		2						
	17	18	21	4		4						6
	17	18	22	11		4						8
	17	18	23	8	1	3						
	17	18	24	15		2						
	17	18	25	11		1		1				2
	17	18	26	9								9
	17	18	27	3	1	2						13
	17	18	28	8		6						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 6 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	17	18	29	3		3						
	17	18	30	61								2
	17	18	31	80		1						
	17	18	32	20		7						
	17	18	33	18		5						
	17	18	34	15	1	7						1
	17	18	35	9		3						12
	17	18	36	16	1	1						1
	17	19	6	2								
	17	19	7	26		1						
	17	19	8	12		3						
	17	19	14	6	1							
	17	19	15	35		1						
	17	19	17	5		1						
	17	19	18	7	1	2						
	17	19	19	6								
	17	19	20	2		1						
	17	19	22	10								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 7 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	17	19	23	69		4						2
	17	19	24	42								2
	17	19	25	49	3	5						13
	17	19	26	11								
	17	19	27	4		1						8
	17	19	29	106	7	10		1				6
	17	19	30	25	2	3	1					4
	17	19	31	34	11	2						4
	17	19	32	19	7	1				1		17
	17	19	34	14	1							5
	17	19	35	81	5	3	1					4
	17	19	36	33								1
	17	20	29	13		2						
	17	20	30	59	4	3						9
	17	20	31	40		2						
	17	20	32	41		1						6
	17	20	33	3						1		
	18	16	3	2								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 8 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	16	5	1								
	18	16	13									7
	18	16	23	2								
	18	16	24	1		1						6
	18	16	25	1								
	18	16	34				1					2
	18	16	35									4
	18	17	1	26		5						1
	18	17	2	64	1							2
	18	17	3	37		1						1
	18	17	4	56								1
	18	17	5	86		4						6
	18	17	6	26	6	3						1
	18	17	7	24	1							
	18	17	8	65	1	1						8
	18	17	9	142	5	1						3
	18	17	10	114		2				1		3
	18	17	11	81		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 9 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	17	12	16		25						1
	18	17	13	41		20	1					
	18	17	14	125		2						2
	18	17	15	35	1	6						4
	18	17	16	75	3	1						7
	18	17	17	35		4						34
	18	17	18	6								4
	18	17	19	6		3						1
	18	17	20	11		10						15
	18	17	21	26	1	28					1	294
	18	17	22	74	1	17						141
	18	17	23	91	12	3						5
	18	17	24	106	4	5						31
	18	17	25	100	6	25	2				1	38
	18	17	26	72		3						
	18	17	27	66	5	8	1	1				39
	18	17	28	108		9						16
	18	17	29	16		3						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 10 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
				Number of Well Permits by Well Use Type								
Citrus cont.	18	17	30	4		4						5
	18	17	31	7		1						1
	18	17	32	24		2						2
	18	17	33	80	6	16					1	10
	18	17	34	69	11	7						47
	18	17	35	194		3						9
	18	17	36	324	1	5						13
	18	18	1	8		2	1					12
	18	18	2	11		29						1
	18	18	3	10		72						4
	18	18	4	6		39						7
	18	18	5	17		50						1
	18	18	6	15	1	35						
	18	18	7	2		33						1
	18	18	8	21		55						
	18	18	9	6	2	38						3
	18	18	10	12		38						
	18	18	11	7	4	14		2				35

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 11 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

County	Township	Range	Section	Number of Well Permits by Well Use Type: Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	18	12	5		3						26
	18	18	13	8		1						8
	18	18	14	3	1	1						5
	18	18	15	5	4	8	1	1				19
	18	18	16	6	1	72						
	18	18	17	10		30						
	18	18	18	11		28						
	18	18	19	12	2	25						2
	18	18	20	11		20				1		1
	18	18	21	10	1	14						1
	18	18	22	52	2	9						10
	18	18	23	6	4	9						40
	18	18	24	54	2	11						10
	18	18	25	227	5	7						26
	18	18	26	41	1							10
	18	18	27	281		2						6
	18	18	28	40	1	4		1				1
	18	18	29	42		1						4

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 12 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	18	30	85	12	9					1	31
	18	18	31	136	4	15						8
	18	18	32	49	3	5						7
	18	18	33	24		9				1		16
	18	18	34	5		2						1
	18	18	35	218						1		1
	18	18	36	260	1	2						9
	18	19	1	183	2	4						10
	18	19	2	232	6	6	2					9
	18	19	3	27		1						
	18	19	4	7		3						
	18	19	5	139	5	3						6
	18	19	6	73	1	4						21
	18	19	7	9		13						
	18	19	8	91		20						1
	18	19	9	9								3
	18	19	10	70								6
	18	19	11	179	1	7						5

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 13 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	19	12	18								1
	18	19	13	51	2	8						7
	18	19	14	193	4	10						9
	18	19	15	88		1						5
	18	19	16	8								
	18	19	17	20		13						
	18	19	18	146		3						1
	18	19	19	137	1	8						27
	18	19	20	19								
	18	19	21	47		7						11
	18	19	22	104	1	2						2
	18	19	23	33	10	3						38
	18	19	24	61	4	3						17
	18	19	25	164	7	3				1		13
	18	19	26	67	15	5						19
	18	19	27	131	2	1	1			1	1	7
	18	19	28	175	2	4						6
	18	19	29	2								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 14 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	19	30	291	6	11						17
	18	19	31	240		4						5
	18	19	32	9	1							7
	18	19	33	21		22						2
	18	19	34	75	3	5				1		7
	18	19	35	212		2						3
	18	19	36	231	6	79						18
	18	20	3	12		2						2
	18	20	4	7								
	18	20	5	8								1
	18	20	6	37		5						4
	18	20	7	9								
	18	20	8	7								
	18	20	9	6		2						
	18	20	10	5		1						1
	18	20	14	8	1							
	18	20	15	5								
	18	20	16	1		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 15 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	20	17	4		3						1
	18	20	18	5		2						
	18	20	19	16		3						2
	18	20	20	3		2						
	18	20	21	6		1						
	18	20	22	11		1						
	18	20	23	44	1							5
	18	20	25	18								
	18	20	26	46		1						3
	18	20	27	55		2						4
	18	20	28	48		1						
	18	20	29	13		2						
	18	20	30	11								2
	18	20	31	14		2						2
	18	20	32	18		2						1
	18	20	33	62		6		1				2
	18	20	34	44		3						1
	18	20	35	29		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 16 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	18	20	36	20		18						1
	19	16	5			1						
	19	16	7	1								
	19	16	10									1
	19	16	13	1								
	19	16	14									1
	19	16	19	2								
	19	16	20	1								
	19	16	26	1								
	19	16	30									1
	19	16	31									5
	19	16	32	1								
	19	16	36									5
	19	17	1	31		4						
	19	17	2	10	1	1						
	19	17	3	59	1							7
	19	17	4	51		1						7
	19	17	5	13								1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 17 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	17	6	10								1
	19	17	7	5								
	19	17	8	7								
	19	17	9	19								16
	19	17	10	165	11	2						30
	19	17	11	75								1
	19	17	12	172								2
	19	17	13	236	1	1					1	4
	19	17	14	91		1						4
	19	17	15	49	5	4	1					23
	19	17	16	12								
	19	17	17	1								
	19	17	18	4		1						
	19	17	19	1		2						
	19	17	20	1		1						4
	19	17	21	4								18
	19	17	22	97	7	5		1				113
	19	17	23	265	7	4	1					7

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 18 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	17	24	254	6	5						5
	19	17	25	151	3	11						1
	19	17	26	199	5	4		1				21
	19	17	27	49	11	10		1			1	118
	19	17	28	10	2	4						33
	19	17	29	12		7						6
	19	17	30	10		12						18
	19	17	31	19		11						22
	19	17	32	16		4						25
	19	17	33	7		2						8
	19	17	34	24		2						1
	19	17	35	257	15							20
	19	17	36	219	2	9						2
	19	18	1	73	4	6						77
	19	18	2	108	13	5						6
	19	18	3	131	5	5						16
	19	18	4	41	6	8						41
	19	18	5	94	8	2						21

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 19 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	18	6	25	4	2	1					14
	19	18	7	76	2	7	1					2
	19	18	8	17		1						1
	19	18	9	30	3	4				1		25
	19	18	10	15								1
	19	18	11	11								1
	19	18	12	53		1						3
	19	18	13	57	2	1						1
	19	18	14	24	2							
	19	18	15	4	1	2						
	19	18	16	11	7	5						7
	19	18	17	10		2						3
	19	18	18	64		3						9
	19	18	19	64	1		1					31
	19	18	20	20		4						14
	19	18	21	21	9	2					1	17
	19	18	22	5			1					2
	19	18	23	25		2						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 20 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	18	24	105		2						6
	19	18	25	104	1	8						1
	19	18	26	26	1							5
	19	18	27	6	2	1						9
	19	18	28	179								18
	19	18	29	159	3	5						3
	19	18	30	232	3	5						2
	19	18	31	356		2						5
	19	18	32	246		1				1		2
	19	18	33	123		5						3
	19	18	34	74						1		1
	19	18	35	100	1	2						2
	19	18	36	98		3						8
	19	19	1	160	4	3						5
	19	19	2	279	2	3						8
	19	19	3	26	1	3						16
	19	19	4	21	1	13						5
	19	19	6	45	6	3						23

LNP COL 2.4-4

**Table 2.4.12-202 (Sheet 21 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site**

County	Township	Range	Section	Number of Well Permits by Well Use Type: Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	19	7	4		1						
	19	19	9	25		4						4
	19	19	10	61	3	6	1	1				93
	19	19	11	290	16	3	3	1				55
	19	19	12	56	5	2		1				20
	19	19	13	69	15	9						33
	19	19	14	116	3	3						5
	19	19	15	24								1
	19	19	16	10			1					
	19	19	18	2		2						6
	19	19	19	15								
	19	19	21	9	1	1						
	19	19	22	7								
	19	19	23	45		1						1
	19	19	24	13	1	2						
	19	19	25	75	2	2				1		3
	19	19	26	15								4
	19	19	27	8		4						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 22 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	19	28	10								
	19	19	30	29								4
	19	19	31	124		1						4
	19	19	33	9		1						
	19	19	34	11								4
	19	19	35	21								
	19	19	36	34	1	1			1			
	19	20	2	40	3	9						7
	19	20	3	156	1	10		1				12
	19	20	4	66	1	4						3
	19	20	5	69		9	1					1
	19	20	6	119	6	8						130
	19	20	7	27	5	10					1	13
	19	20	8	32	1	20						
	19	20	9	155	1	13						10
	19	20	10	55	1	5						4
	19	20	16	81	1	12						8
	19	20	17	13		11	1					304

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 23 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	19	20	18	14		17						50
	19	20	19	48	3	4						20
	19	20	20	541	2	13	2					19
	19	20	30	158	1	2						2
	20	16	11	1								
	20	16	14	1								
	20	16	24			1						
	20	16	26	1								
	20	17	1	8	16	1						3
	20	17	2	1		2						
	20	17	4	1		1						
	20	17	5	9								3
	20	17	6	28	1	1						
	20	17	7	31	1	1						
	20	17	8	14		3						5
	20	17	9	3								
	20	17	10									8
	20	17	11			1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 24 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	20	17	12	6	7	3						38
	20	17	13	6	4	6						35
	20	17	16	1		2						
	20	17	17			16						3
	20	17	18	13		27						
	20	17	19	4	1	18						4
	20	17	20	2		4						
	20	17	22	2								
	20	17	23	3		1						
	20	17	24	11	6	4						8
	20	17	25	67	2	1						10
	20	17	26	80	2	2						4
	20	17	27									6
	20	17	29	3		8						
	20	17	30	5		1						1
	20	18	1	54								2
	20	18	2	50	3	9						3
	20	18	3	70		2					1	1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 25 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	20	18	4	66		2						
	20	18	5	87								10
	20	18	6	105								
	20	18	7	158	1	5						45
	20	18	8	157	2	15						
	20	18	9	101	1	4						
	20	18	10	69		3						3
	20	18	11	66		4				1		3
	20	18	12	78		4	1			1		2
	20	18	13	23	1	6						10
	20	18	14	1		1						1
	20	18	15	2		3						1
	20	18	16	12	2	77						4
	20	18	17	15		170						1
	20	18	18	43		125		1				22
	20	18	19	26		123						16
	20	18	20	22		175			1			12
	20	18	21	37	2	95						9

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 26 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Citrus cont.	20	18	22	1		1						3
	20	18	23	3		1						6
	20	18	24	6	2	1						10
	20	18	25	17		5						
	20	18	26	4		1						
	20	18	27	3		4				1		16
	20	18	28	13		9						21
	20	18	29	3		128						1
	20	18	30	15	6	33						8
	20	19	3	7								1
	20	19	4									2
	20	19	5	4								
	20	19	6	15		1						6
	20	19	8	4		1						
	20	19	9	4								
	20	19	18	4		1						2
Citrus Total				**22,509**	**607**	**2977**	**35**	**19**	**12**	**20**	**20**	**4158**

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 27 of 66) Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy	12	15	36									1
	12	17	16	58								24
	12	17	17	7	1	3						1
	12	17	18	2								
	12	17	19									1
	12	17	20	20						1		1
	12	17	21	98		1						3
	12	17	22	70	3	3						2
	12	17	23	152	2							5
	12	17	24	52	1							40
	12	17	25	44								1
	12	17	26	33		1						
	12	17	27	93								
	12	17	28	40		1				2		1
	12	17	29	4								
	12	17	30									1
	12	17	33	18		2				1		
	12	17	34	16		3				1		1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 28 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	12	17	35	5		3				1		
	12	17	36	38		1						
	12	18	19	54						1		
	12	18	20	17		1						
	12	18	21	26		1						
	12	18	25	24								14
	12	18	26	11	1	2				1		
	12	18	27	58	1	1				1		1
	12	18	28	54		1						
	12	18	29	62								
	12	18	30	104	2							3
	12	18	31	46								
	12	18	32	137		1				1		1
	12	18	33	83		1						
	12	18	34	26		1						
	12	18	35	33		5						
	12	18	36	19		3				2		1
	12	19	31	11		5				1		25

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 29 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Levy cont.	12	19	32	17		2					1	
	13	14	22									1
	13	17	1	15		4						
	13	17	2	6		3				4		
	13	17	3	12		1				1		
	13	17	4	4								1
	13	17	5									1
	13	17	9	1						1		
	13	17	10	14								
	13	17	11	35		1						
	13	17	12	14		3						
	13	17	13	84		1						
	13	17	14	11								
	13	17	19									2
	13	17	22	8		1						1
	13	17	23	4		2						
	13	17	24	10		1						1
	13	17	25	3								3

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 30 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	13	17	26	29						1		
	13	17	27	2								
	13	17	33									1
	13	17	34	1		1						
	13	17	35	27						1		
	13	17	36	30		2				1		
	13	18	1	30	1	3						4
	13	18	2	38		3						2
	13	18	3	24								1
	13	18	4	14						1		
	13	18	5	13		1						6
	13	18	6	8		3				1		10
	13	18	7	2		1				1		4
	13	18	8	10								
	13	18	9	3		1						
	13	18	10	43		1						16
	13	18	11	33						2		
	13	18	12	15	1	3						7

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 31 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	13	18	13			2	1			1		2
	13	18	14							1		
	13	18	15	32		1						
	13	18	16	112		1						2
	13	18	17	26		2						1
	13	18	18	10		3						1
	13	18	19	132		1						5
	13	18	20	28		1						1
	13	18	21	142	1	1						1
	13	18	22	51		1						
	13	18	23	13		2				1		
	13	18	24	9		3						
	13	18	25	11		3						
	13	18	26	3								
	13	18	27	5		1				2		
	13	18	28	22		4				2		1
	13	18	29	88								1
	13	18	30	9		3						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 32 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	13	18	31	11		6						
	13	18	32	41		1						
	13	18	33	4		5						
	13	18	34	9		1						
	13	18	35	4		1						
	13	18	36	45								
	13	19	2	1								
	13	19	4	45		3						7
	13	19	5	14			1					4
	13	19	6	27		4					1	302
	13	19	7	16	1					1		5
	13	19	8	25	1							
	13	19	9	15	2	2				2		1
	13	19	16	4								
	13	19	17	2								
	13	19	18	12		1						
	13	19	19	16		1						
	13	19	20	28						1		

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 33 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	13	19	21	10		1				1		
	13	19	28	5		3						
	13	19	29	2								
	13	19	30	23								
	13	19	31	35		1				3		15
	13	19	32	6						1		
	13	19	33	6		4						
	14	17	1	13		8						
	14	17	2	42								
	14	17	3	4								
	14	17	4									1
	14	17	6									2
	14	17	7									1
	14	17	10									1
	14	17	11	43								6
	14	17	12	6		6						5
	14	17	13	1		3						3
	14	17	14	5								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 34 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	14	17	18									1
	14	17	23	60								
	14	17	24	36								
	14	17	25	28	1					1		
	14	17	26	11								
	14	17	35	39		1						
	14	17	36	58								1
	14	18	1	15		3				1		
	14	18	2	18		2						
	14	18	3	13		1						
	14	18	4	40		2				1		
	14	18	5	148	2							1
	14	18	6	121		2						
	14	18	7	2		2						5
	14	18	8	7		7				1		
	14	18	9	3		1				1		
	14	18	10	8		3						1
	14	18	11	9		2						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 35 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

County	Township	Range	Section	Number of Well Permits by Well Use Type: Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	14	18	12	24		1						
	14	18	13	20		2						
	14	18	14	2								
	14	18	16	6		3				1		
	14	18	17	3		2						
	14	18	18	22		1						
	14	18	19	27		2				1		1
	14	18	20	7								
	14	18	21	2		1						
	14	18	22	4		3						
	14	18	23	2		1						
	14	18	24	8		7						
	14	18	25	25		2						
	14	18	26	39		9						
	14	18	27	13		14						1
	14	18	28	12		1						
	14	18	29	63		1						1
	14	18	30	15		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 36 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	14	18	31	36		2						
	14	18	32	19		2						
	14	18	33	18								
	14	18	34	8								
	14	18	35	10		7						
	14	18	36	25		2						1
	14	19	4	9		3				1		
	14	19	5	7		1						
	14	19	6	29		3						
	14	19	7	40		2						15
	14	19	8	17	1	4						
	14	19	9	16		2				2		1
	14	19	16	22	1	4				3		
	14	19	17	1		1				1		1
	14	19	18	9		3						
	14	19	19	13		2						
	14	19	20	3		2				1		1
	14	19	21	8		2						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 37 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	14	19	28	156		1						
	14	19	29	12		2						
	14	19	30	37		4				1		
	14	19	31	56	1							1
	14	19	32	5		2						
	14	19	33	37								
	15	15	12	1								
	15	15	28									1
	15	16	1									1
	15	16	2									1
	15	16	9									1
	15	16	11									1
	15	16	18									1
	15	16	19									1
	15	16	25									1
	15	16	26									1
	15	16	27									1
	15	16	33	1								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 38 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	15	16	36	9								
	15	17	1	24								
	15	17	2	2		4				1		
	15	17	3	17		1						12
	15	17	5									1
	15	17	6									1
	15	17	10	8	3	1						
	15	17	11	24		1						
	15	17	12	17								
	15	17	13	30								
	15	17	14	25								
	15	17	15	1								
	15	17	18									1
	15	17	19									1
	15	17	22	1								1
	15	17	23	24								1
	15	17	24	14		1						
	15	17	25	40								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 39 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	15	17	26	17								
	15	17	27			1						
	15	17	29									1
	15	17	34	1								3
	15	17	35		1							
	15	17	36	1								
	16	15	1	1								
	16	15	12	1								
	16	15	14	1								
	16	15	23	1								
	16	16	1	5	1	1						
	16	16	2	4								
	16	16	3	11		1						3
	16	16	4	4		1						
	16	16	5	2		1						
	16	16	6	2								
	16	16	8	5								4
	16	16	10	1	1	1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 40 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	16	16	12	6		1						
	16	16	13	12								4
	16	16	17									1
	16	16	19									3
	16	16	20	2								
	16	16	21	6								
	16	16	23	1		1						
	16	16	24	1		1						
	16	16	25	10	2							
	16	16	26	3	1							
	16	16	27	2								
	16	16	28	5								1
	16	16	29			1						
	16	16	30	1								
	16	16	31	1								1
	16	16	32	12		1					1	9
	16	16	33	13	3	2						2
	16	16	34	71	1	2		1				3

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 41 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	16	16	35	31	1	1				1		
	16	16	36	8								
	16	17	1	7								
	16	17	2	8	1							
	16	17	3	7	1	4						1
	16	17	4	5		1						
	16	17	5	7								2
	16	17	6	41	2	3						1
	16	17	7	11						1		
	16	17	8									3
	16	17	10	2								
	16	17	11	3								
	16	17	12	5								
	16	17	13	4								
	16	17	14	1								
	16	17	15	1								
	16	17	16	2								
	16	17	17	1								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 42 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Levy cont.	16	17	18	6								
	16	17	19	1								12
	16	17	21	4								7
	16	17	22	1								
	16	17	23	1								
	16	17	24	3								1
	16	17	25	3								
	16	17	26	6								
	16	17	27									3
	16	17	28	2								
	16	17	29	1								
	16	17	30	9								25
	16	17	31	5								
	16	17	32	4		1						
	16	17	33	45								
	16	17	34	101		3						2
	16	17	35	7								
	16	17	36	34	1	1						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 43 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Levy cont.	17	16	1	44	4	4		1				12
	17	16	2	41		8				1		7
	17	16	3	55	7	5		1				39
	17	16	4	59	1	3						23
	17	16	5	36		4						39
	17	16	6	5		2				2		4
	17	16	7	1								4
	17	17	2	99		3						2
	17	17	3	54	1							1
	17	17	4	38	2							1
	17	17	5	23								2
	17	17	6	33	2	2						1
	17	17	7	18	1							
	17	17	8	5								4
	17	17	9	1								
Levy Total				**6121**	**61**	**354**	**2**	**3**		**65**	**3**	**850**

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 44 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion	13	19	1	2								
	13	19	2	8						1		8
	13	19	3	18		4						1
	13	19	10	11		1						2
	13	19	11	8		1				1		
	13	19	12	5	1	2						2
	13	19	13	17								
	13	19	14	6		3				2		
	13	19	15	16		1				1		
	13	19	22	21		2				1		
	13	19	23	4								
	13	19	24	2								
	13	19	25	1								
	13	19	26	6		3						
	13	19	27	26		1				1		
	13	19	34	8								
	13	19	35	2		2						
	13	19	36	5	1	3						3

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 45 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

County	Township	Range	Section	Number of Well Permits by Well Use Type: Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	13	20	19	2								
	13	20	29							1		
	13	20	30	4		1						
	13	20	31	4		3				1		1
	13	20	32	1						1		
	13	20	33	9	2					2		
	14	19	1	15								
	14	19	2	16								
	14	19	3	6		2				2		
	14	19	10	22		5				3		
	14	19	11	10		3						1
	14	19	12	29	2	1				2		1
	14	19	13	13								
	14	19	14	8		4						
	14	19	15	16		1						1
	14	19	22	10		1						
	14	19	23	10		2						1
	14	19	24	17		2				1		

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 46 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	14	19	25	27		1				1		2
	14	19	26	26		1				1		
	14	19	27	13		1				1		
	14	19	34	1								
	14	19	36	4						1		1
	14	20	3	19								4
	14	20	4	16						3		
	14	20	5	10								
	14	20	6	8		1				1		4
	14	20	7	29	4	1				7		6
	14	20	8	35		1				3		1
	14	20	9	25	1	1				1		
	14	20	10	13		1						
	14	20	11	4								
	14	20	12	11		2						
	14	20	13	52		1				2		
	14	20	14	20						1		
	14	20	15	14		2						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 47 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	14	20	16	45		1				1		
	14	20	17	46		7				3		3
	14	20	18	47	2	3				5		13
	14	20	19	31		1				3		
	14	20	20	23	4	3				1		6
	14	20	21	25	1	2				2		1
	14	20	22	6						1		
	14	20	23	47						3		1
	14	20	24	39	1	3				2		2
	14	20	25	10	1	5				1		
	14	20	26	19		1				2		1
	14	20	27	22	7	5						15
	14	20	28	15						2		
	14	20	29	13						3		
	14	20	30	29		3		1		3		
	14	20	31	12						2		
	14	20	32	16	1	2				4		
	14	20	33	26		1				4		

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 48 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	14	20	34	20		3				5		
	14	20	35	21		2						
	14	20	36	19	2	21				2		2
	14	21	31	21	8	30						4
	15	18	1	17		2						1
	15	18	2	29		2						1
	15	18	3	7		4						1
	15	18	4	11		4				1		
	15	18	5	8		2						
	15	18	6	21		1						1
	15	18	7	16								18
	15	18	8	16								
	15	18	9	5		3						
	15	18	10	17		2				2		2
	15	18	11	7		3				2		1
	15	18	12	12	1	1						5
	15	18	13	32		2				1		
	15	18	14	11		7						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 49 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	18	15	15		1				1		
	15	18	16	3								
	15	18	17	34								
	15	18	18	23								1
	15	18	19	30								1
	15	18	20	33		1						1
	15	18	21	38								
	15	18	22	22	1	1						
	15	18	23	16		1				1		1
	15	18	24	28		3				2		
	15	18	25	68	4	2				1		
	15	18	26	34		1						
	15	18	27	25	1							
	15	18	28	53								1
	15	18	29	51								4
	15	18	30	39		1						
	15	18	31	43								
	15	18	32	79								6

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 50 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	18	33	64								1
	15	18	34	79		1						3
	15	18	35	120	2							
	15	18	36	87	2	2						1
	15	19	1	14								6
	15	19	2	17						1		1
	15	19	3	6								
	15	19	4	24		3						
	15	19	5	4		3				1		
	15	19	6	10	1	1						
	15	19	7	10		2						
	15	19	8	14		1						
	15	19	9	17		1				1		
	15	19	10	1								
	15	19	11	16		1						
	15	19	12	87		1				2		4
	15	19	13	41		8				3		1
	15	19	14	67		2						2

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 51 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	19	15	9								
	15	19	16	5								
	15	19	17	2								
	15	19	18	24		3						
	15	19	19	93		1						
	15	19	20	84		1				1		1
	15	19	21	118								1
	15	19	22	2								
	15	19	23	36								
	15	19	24	156		2				1		2
	15	19	25	140	1	1						11
	15	19	26	72								
	15	19	28	210		2						1
	15	19	29	83	1	1						1
	15	19	30	156						1		1
	15	19	31	170		1						1
	15	19	32	116	2	1						4
	15	19	33	212	1							1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 52 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	19	35	61	1							
	15	19	36	27		2				4		
	15	20	1	4	3	4						4
	15	20	2	10		1				1		
	15	20	3	10		1				2		1
	15	20	4	29		4				3		1
	15	20	5	18		3				5		
	15	20	6	22								
	15	20	7	119		1				2		
	15	20	8	76						1		
	15	20	9	577								2
	15	20	10	155	1	2						1
	15	20	11	10		1						
	15	20	12	26	1							
	15	20	13	46		4				2		12
	15	20	14	22		1						
	15	20	15	37								1
	15	20	16	104	4	7						7

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 53 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	20	17	85	1	5				1		9
	15	20	18	236		3				1		1
	15	20	19	96		2				1		2
	15	20	20	24		8				6		
	15	20	21	10		1						
	15	20	22	4	1							
	15	20	23	9		7						
	15	20	24	57		5	1			1		2
	15	20	25	61		1						1
	15	20	26	15	1							1
	15	20	27	9		3						
	15	20	28	20		2						
	15	20	29	5								
	15	20	30	13	2	1				2		1
	15	20	31	46		5				6		1
	15	20	32	97		1						1
	15	20	33	52								
	15	20	34	6		2						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 54 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	15	20	35	2								
	15	20	36	91		4				2		2
	15	21	6	17	3	2						20
	15	21	7	104		6	1					5
	15	21	17	10	20	4						93
	15	21	18	90	7	4						40
	15	21	19	282		3		1				17
	15	21	20	5	1	3						1
	15	21	29	17	4	3						8
	15	21	30	15	5	2				1		8
	15	21	31	2								
	15	21	32	8	5	1				1		13
	15	21	33	17	4	10						6
	16	18	1	11	5	1				1		2
	16	18	2	82		1						1
	16	18	3	8								1
	16	18	4	3		1						1
	16	18	5	6								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 55 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	18	6	8		2						
	16	18	7	5								
	16	18	9	4								
	16	18	10	58	2	4						3
	16	18	11	388	1	3						7
	16	18	12	33	6	30						5
	16	18	13	32	2	179						2
	16	18	14	230	1	11						7
	16	18	15	121		3						4
	16	18	16	7								1
	16	18	17	1		1						3
	16	18	18	4								
	16	18	19	3	1							
	16	18	20	2		1						1
	16	18	21	1								
	16	18	22	9								5
	16	18	23	62	2	1						17
	16	18	24	31	1	54	1					22

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 56 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	18	25	24		8						3
	16	18	26	11	3	11	1					37
	16	18	27	20		2				1		
	16	18	28	6		1						
	16	18	29	16	1	1						5
	16	18	30	7		2						
	16	18	31	33		3						4
	16	18	32	39								1
	16	18	33	149	4	4						7
	16	18	34	59	4							3
	16	18	35	23	3	12						256
	16	18	36	20	2	6	1					6
	16	19	1	6						1		4
	16	19	2	6		1						
	16	19	3	3		2						
	16	19	4	39		4						1
	16	19	5	5	2	3						8
	16	19	6	62		1						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 57 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	19	7	91		4						
	16	19	8	5	1	4						
	16	19	10	1								
	16	19	11	7		1						
	16	19	12	7		7				5		
	16	19	13	23		28						
	16	19	14	4		1				3		
	16	19	15	3								
	16	19	16			2						
	16	19	17							1		
	16	19	18	38	5	16						5
	16	19	19	37	3	8						1
	16	19	20		4	2				1		1
	16	19	21	1								
	16	19	22			1						
	16	19	23	5		1						
	16	19	24	9		12						
	16	19	25	20		2				2		1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 58 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	19	26	5								4
	16	19	27	1								
	16	19	28	9								2
	16	19	29	29		4				1		1
	16	19	30	16	1	4						1
	16	19	31	24								17
	16	19	32	10		2						1
	16	19	33	7								
	16	19	34	3		1						
	16	19	35	3			1	1				
	16	19	36	25	3	1				1		2
	16	20	1	2	2	3						3
	16	20	2			1						
	16	20	3	2								
	16	20	4	1								
	16	20	5	84								1
	16	20	6	116		1						2
	16	20	7	90		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 59 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	20	8	70								8
	16	20	9	1								
	16	20	10	3								9
	16	20	11	1		2						
	16	20	12	5	1	2						6
	16	20	13	4		2						4
	16	20	14	1	1	1						1
	16	20	15	2								
	16	20	16			1						
	16	20	17	57								
	16	20	18	85								1
	16	20	19	88		3				1		1
	16	20	20	70								
	16	20	22	4								
	16	20	23	2	5	1				1		11
	16	20	24	5	3	3						9
	16	20	25	74	22	19					1	16
	16	20	26	44	15	21				1		14

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 60 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	16	20	27	72	2	3						
	16	20	28	82	1							1
	16	20	29	50		6						1
	16	20	30	181						1		
	16	20	31	142	1	2						2
	16	20	32	36	3	2						
	16	20	33	39	2	4				1		
	16	20	34	32	2	8						7
	16	20	35	9	11	11					1	25
	16	20	36	5		8						4
	16	21	4	15	12	12				1		15
	16	21	5	39	5	4				1		2
	16	21	6	1		2						
	16	21	7	62	1					1		2
	16	21	8	36	12	7				1		12
	16	21	9	5	2	1						1
	16	21	16	25	4	5				1		1
	16	21	17	12	12	14						15

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 61 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Marion cont.	16	21	18	82	5	2				1		8
	16	21	19	86	5	3						2
	16	21	20	30		16						1
	16	21	21	26		14				1		4
	16	21	28	122		51						2
	16	21	29	150	3	64				1		6
	16	21	30	56	2	23						2
	16	21	31	5	1							
	16	21	32	30		7		1				1
	16	21	33	57		42						2
	17	19	1	7								
	17	19	2	18		1						2
	17	19	3	3						1		1
	17	19	5	3								1
	17	19	8	7								
	17	19	10	6								5
	17	19	11	1								
	17	19	12	2		1						

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 62 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	17	19	13	6								
	17	19	14	1								
	17	19	15	13								
	17	19	24	3							1	
	17	20	1	20	1							
	17	20	2			1						
	17	20	3	7	1	1						
	17	20	4	1		1						
	17	20	5			1						1
	17	20	7	1								1
	17	20	8	1	1	1						12
	17	20	9	1	3	4						7
	17	20	10	2								
	17	20	11	1								
	17	20	12	31	1	5						
	17	20	13	67		3						4
	17	20	14	1								
	17	20	15	3								

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 63 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

County	Township	Range	Section	Number of Well Permits by Well Use Type: Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Marion cont.	17	20	16	56		2						1
	17	20	17	7	2	2						6
	17	20	18	3		1						
	17	20	19	10	1	3						
	17	20	20	11		2						1
	17	20	21	47		2						
	17	20	22	180		2						3
	17	20	23	111		1					1	4
	17	20	24	85	1	41						5
	17	20	25	7								
	17	20	26	2								
	17	20	27	1								
	17	20	28	2								
	17	20	33	2								
	17	20	34	1								
	17	20	35	4								
	17	20	36	5								
	17	21	4	17		3						1

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 64 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	**Township**	**Range**	**Section**	**Domestic**	**Public Supply**	**Irrigation**	**Industrial**	**Mining**	**Power**	**Livestock**	**Essential Services (Fire Protection)**	**Other**
Marion cont.	17	21	5	34		3						1
	17	21	6	38								1
	17	21	7	86	1	3						4
	17	21	8	48		3						4
	17	21	9	4	1	1						
	17	21	16	1		5						
	17	21	17	23		10						
	17	21	18	27	1							
	17	21	19	78	1	2						1
	17	21	20	43		3						2
	17	21	21	15	1	10						1
	17	21	29	5		2						1
	17	21	30	2								
	17	21	31	1								
	17	21	32			1						
Marion Total				**12,784**	**327**	**1303**	**6**	**4**		**186**	**4**	**1162**

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 65 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Sumter	18	20	1	17								
	18	20	2	8		1						
	18	20	3	3								
	18	20	11	6		1						
	18	20	12	5								1
	18	20	13	1								
	18	20	23	7								3
	18	20	24	12		1						
	18	20	25	4								1
	18	21	5	3								
	18	21	6	2								
	18	21	7	2								
Sumter Total				**70**		**3**						**5**

LNP COL 2.4-4

Table 2.4.12-202 (Sheet 66 of 66)
Southwest Florida Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type								
County	Township	Range	Section	Domestic	Public Supply	Irrigation	Industrial	Mining	Power	Livestock	Essential Services (Fire Protection)	Other
Total SWFWMD				41,484	995	4637	43	26	12	271	27	6175

Notes:

Domestic well use category includes permits for "Domestic" and "Repair Domestic" well use types.

Public Supply well use category includes permits for "Public Supply" and "Repair Public Supply" well use types.

Irrigation well use category includes permits for "Irrigation" and "Repair Irrigation" well use types.

Other well use category includes permits for "Air Conditioning Supply - Heat Pump", "Aquaculture", "Back Plugged", "Geothermal Well", "Grounding Rod", "Injection Well", "Observation or Monitor Well", "Plugged", "Recovery of Contaminants", "Repair or Deepen (Use not Specified)", "Return Air/Heat", "Sealing Water", "Test Well/Piezometer", and unspecified well use types.

Source: Reference 2.4.12-210

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 1 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	Township	Range	Section	Self-Supplied Residential	Public Supply	Irrigation	Fire Protection	Other
Levy	12	15	26	1				
	12	15	32	1				
	12	16	21					2
	12	16	24	1				
	12	16	25	1				
	12	16	34	1				
	12	17	18	5				5
	12	17	19	12				
	12	17	20	24				
	12	17	29	3				
	12	17	30	3				
	12	17	31	1				
	12	17	32	4				
	13	14	11	23		1		
	13	14	12	18	1	2		
	13	14	13	4				
	13	14	14	6				
	13	14	22	3				
	13	14	23	2				
	13	14	27	6				

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 2 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	Township	Range	Section	Self-Supplied Residential	Public Supply	Irrigation	Fire Protection	Other
Levy, cont.	13	14	31	1				
	13	14	32	1				
	13	14	33	4				
	13	14	34	1				
	13	14	35	4				
	13	14	36	2				
	13	15	6	25				
	13	15	12	1				
	13	15	15	1				
	13	15	16	4				
	13	15	17	1				
	13	15	18	1				
	13	15	22	1				
	13	15	25					20
	13	15	26	5				
	13	15	27	1				
	13	15	28	1				
	13	15	29	2				
	13	15	30	1				
	13	15	32	1				

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 3 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	**Township**	**Range**	**Section**	**Self-Supplied Residential**	**Public Supply**	**Irrigation**	**Fire Protection**	**Other**
Levy, cont.	13	15	33	1				
	13	15	34	5				
	13	15	35	2				
	13	15	36	1				
	13	16	1	1				
	13	16	32	1				
	13	17	4	2				
	13	17	5	3		1		
	13	17	7	1				
	13	17	8	3				
	13	17	9	2				
	13	17	17		2	1	1	
	13	17	30					1
	13	17	34		1			
	14	13	12	3				
	14	13	13	1				
	14	13	14	3				
	14	13	22	1				
	14	13	24	5				
	14	13	25	19				
	14	13	26	17				

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 4 of 7) Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	**Township**	**Range**	**Selection**	**Self-Supplied Residential**	**Public Supply**	**Irrigation**	**Fire Protection**	**Other**
Levy, cont.	14	13	27	22				
	14	13	34	43		1		
	14	13	35	52		2		
	14	13	36	53	5	1	1	4
	14	14	4	2				
	14	14	5	4				
	14	14	7	3				
	14	14	8	1				
	14	14	9	1				
	14	14	10	2				
	14	14	12	1				
	14	14	14	2				
	14	14	15	2				
	14	14	16	11				
	14	14	17	6				
				4				
	14	14	19	49				
	14	14	20	6				
	14	14	22	1				
	14	14	26	1				
	14	14	27	2				

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 5 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	Township	Range	Selection	Self-Supplied Residential	Public Supply	Irrigation	Fire Protection	Other
Levy, cont.	14	14	28	4				
	14	14	29	16	2	1		1
	14	14	30	55	4			
	14	14	31	4	2			
	14	14	36	1				
	14	15	3	1				
	14	15	5	2				
	14	15	11	1				
	14	15	20	4				3
	14	15	21	2				
	14	15	33	2				
	14	15	36	5				
	14	16	11	4				
	14	16	12	8				
	14	16	13	1				
	14	16	15	2				
	14	16	16	7				
	14	16	17	12				4
	14	16	20	1	1			15

LNP COL 2.4-4

**Table 2.4.12-203 (Sheet 6 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site**

				Number of Well Permits by Well Use Type				
County	Township	Range	Selection	Self-Supplied Residential	Public Supply	Irrigation	Fire Protection	Other
Levy, cont.	14	16	21	7	1			2
	14	16	22	6				
	14	16	28	6				
	14	16	29	17				
	14	16	30	13				
	14	16	31	8				
	14	16	32	11		2		
	14	16	33	11				
	14	17	3	2				
	14	17	7	1	1			16
	14	17	8	1				
	14	17	10	8				
	14	17	15	2				
	14	17	23	9				
	14	17	30	2				
	15	13	2		1			
	15	13	3		1			
	15	13	4	33	1			
	15	13	9	3	1			1

LNP COL 2.4-4

Table 2.4.12-203 (Sheet 7 of 7)
Suwannee River Water Management District Permitted Wells within 40.2 Km (25 Mi.) of the LNP Site

				Number of Well Permits by Well Use Type				
County	**Township**	**Range**	**Selection**	**Self-Supplied Residential**	**Public Supply**	**Irrigation**	**Fire Protection**	**Other**
Levy, cont.	15	13	16	2				
	15	14	8	1				
	15	15	24	2				
	15	16	5	3				
	15	16	6	4				
	15	16	22	1				
	15	16	24	2				
	15	17	19					2
	15	17	28	1				
SRWMD Total				**804**	**24**	**12**	**2**	**76**

Notes:

Irrigation category includes "Agricultural Irrigation", "Landscape Irrigation (not including residential)", and "Home Garden or Residential Landscape" well use types.

Other category includes "Monitor" and "Other" wells use types.

Source: Reference: 2.4.12-211

LNP COL 2.4-4

Table 2.4.12-204 (Sheet 1 of 4)
Public Water Supply Users within 16.1 Km (10 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
2381438	Inglis	Levy	Noncommunity	25	14,400	Recreation Area
2381455	Inglis	Levy	Noncommunity	25	-	Camp Ground
6090099	Belleview	Citrus	Community	315	100,000	Subdivision
6090099	Belleview	Citrus	Community	315	100,000	Subdivision
6090523	Belleview	Citrus	Community	84	86,000	Subdivision
6090601	Crystal River	Citrus	Nontransient Noncommunity	750	1,000,000	Nuclear Reactor
6090601	Crystal River	Citrus	Nontransient Noncommunity	750	1,000,000	Nuclear Reactor
6090601	Crystal River	Citrus	Nontransient Noncommunity	750	1,000,000	Nuclear Reactor
6091516	Crystal River	Citrus	Nontransient Noncommunity	25	-	RV Park
6091516	Crystal River	Citrus	Nontransient Noncommunity	25	-	RV Park
6091672	Dunnellon	Citrus	Community	300	561,600	Subdivision
6092186	Crystal River	Citrus	Nontransient Noncommunity	450	100,000	Hospital
6092325	Inverness	Citrus	Noncommunity	75	-	Mobile Home Park
6092325	Inverness	Citrus	Noncommunity	75	-	Mobile Home Park
6092326	Crystal River	Citrus	Community	85	3000	Subdivision
6092331	Inglis	Citrus	Community	170	-	Mobile Home Park
6092961	Dunnellon	Citrus	Noncommunity	25	-	Restaurant
6094482	Crystal River	Citrus	Noncommunity	25	2000	Restaurant
6094523	Crystal River	Citrus	Community	25	-	Subdivision

LNP COL 2.4-4

Table 2.4.12-204 (Sheet 2 of 4)
Public Water Supply Users within 16.1 Km (10 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity	Primary Use
6094810	Crystal River	Citrus	Noncommunity	25	58,000	Nursing Home
6094869	Crystal River	Citrus	Noncommunity	25	-	Retail/General Merchant
6094917	Crystal River	Citrus	Noncommunity	25	-	RV Park
6094920	Inglis	Citrus	Noncommunity	50	-	Camp Ground
6094928	Dunnellon	Citrus	Community	94	-	Subdivision
6094956	Crystal River	Citrus	Nontransient Noncommunity	342	1,600,000	Nuclear Reactor
6094956	Crystal River	Citrus	Nontransient Noncommunity	342	1,600,000	Nuclear Reactor
6094956	Crystal River	Citrus	Nontransient Noncommunity	342	1,600,000	Nuclear Reactor
6094956	Crystal River	Citrus	Nontransient Noncommunity	342	1,600,000	Nuclear Reactor
6094972	Crystal River	Citrus	Noncommunity	25	2000	Nuclear Reactor
6094988	Crystal River	Citrus	Noncommunity	25	-	Nuclear Reactor
6094999	Dunnellon	Citrus	Nontransient Noncommunity	25	-	Church
6095026	Crystal River	Citrus	Noncommunity	25	-	Office for Business
6095050	Crystal River	Citrus	Noncommunity	30	2000	RV Park
6095059	Inglis	Citrus	Noncommunity	100	86,400	Recreation Area
6095062	Dunnellon	Citrus	Noncommunity	100	-	Church
6095083	Lecanto	Citrus	Nontransient Noncommunity	90	20,000	Office for Business
6382056	Inglis	Levy	Community	1500	500,000	Municipal/City

LNP COL 2.4-4

Table 2.4.12-204 (Sheet 3 of 4)
Public Water Supply Users within 16.1 Km (10 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity	Primary Use
6382056	Inglis	Levy	Community	1500	500,000	Municipal/City
6382056	Inglis	Levy	Community	1500	500,000	Municipal/City
6382106	Inglis	Levy	Noncommunity	56	9442	Mobile Home Park
6382108	Inglis	Levy	Community	150	74,880	Mobile Home Park
6382108	Inglis	Levy	Community	150	74,880	Mobile Home Park
6382112	Inglis	Levy	Noncommunity	70	11,495	RV Park
6382116	Yankeetown	Levy	Community	711	288,000	Municipal/City
6382116	Yankeetown	Levy	Community	711	288,000	Municipal/City
6382121	Inglis	Levy	Noncommunity	25	11,495	RV Park
6384606	Dunnellon	Levy	Noncommunity	92	7369	RV Park
6384610	Inglis	Levy	Noncommunity	64	11,780	Mobile Home Park
6384611	Inglis	Levy	Noncommunity	102	18,604	Camp Ground
6384611	Inglis	Levy	Noncommunity	102	18,604	Camp Ground
6384612	Inglis	Levy	Noncommunity	25	28,887	Mobile Home Park
6384612	Inglis	Levy	Noncommunity	25	28,887	Mobile Home Park
6384619	Inglis	Levy	Noncommunity	50	12,155	Travel Trailer Park
6384621	Inglis	Levy	Nontransient Noncommunity	200	27,000	Camp Ground
6384634	Inglis	Levy	Noncommunity	56	33,900	Mobile Home Park
6384634	Inglis	Levy	Noncommunity	56	33,900	Mobile Home Park

LNP COL 2.4-4

Table 2.4.12-204 (Sheet 4 of 4)
Public Water Supply Users within 16.1 Km (10 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity	Primary Use
6384636	Inglis	Levy	Community	75	41,000	Recreation Area
6421462	Dunnellon	Marion	Noncommunity	35	21,600	Recreation Area
6424073	Dunnellon	Marion	Community	1922	1,152,000	Municipal/City
6424073	Dunnellon	Marion	Community	1922	1,152,000	Municipal/City
6424083	Dunnellon	Marion	Community	1793	714,000	Subdivision
6424083	Dunnellon	Marion	Community	1793	714,000	Subdivision
6424083	Dunnellon	Marion	Community	1793	714,000	Subdivision
6424784	Dunnellon	Marion	Noncommunity	35	5000	Recreation Area

Notes:

\- = Data not available.
gpd = gallons per day
RV = recreational vehicle

Source: Reference 2.4.12-213

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 1 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
2380178	Cedar Key	Levy	Community	1500	360,000	Municipal/City
2380178	Cedar Key	Levy	Community	1500	360,000	Municipal/City
2380854	Otter Creek	Levy	Community	120	108,000	Municipal/City
2380854	Otter Creek	Levy	Community	120	108,000	Municipal/City
2381414	Bronson	Levy	Community	359	144,000	Labor Camp
2381415	Cedar Key	Levy	Noncommunity	120	12,114	RV Park
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090150	Beverly Hills	Citrus	Community	12,426	5,600,000	Subdivision
6090156	Lecanto	Citrus	Community	70	28,000	Mobile Home Park
6090204	Homosassa Springs	Citrus	Noncommunity	50	57,000	Restaurant
6090236	Homosassa	Citrus	Noncommunity	80	-	Mobile Home Park
6090281	Homosassa	Citrus	Noncommunity	25	50,000	Mobile Home Park

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 2 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6090307	Homosassa	Citrus	Noncommunity	50	43,000	Retail/General Merchant
6090312	Dunnellon	Marion	Community	15,675	5,158,000	Subdivision
6090312	Dunnellon	Marion	Community	15,675	5,158,000	Subdivision
6090312	Dunnellon	Marion	Community	15,675	5,158,000	Subdivision
6090317	Crystal River	Citrus	Community	4528	2,232,000	Municipal/City
6090317	Crystal River	Citrus	Community	4528	2,232,000	Municipal/City
6090411	Crystal River	Citrus	Community	75	22,000	Mobile Home Park
6090541	Homosassa	Citrus	Community	125	-	Mobile Home Park
6090624	Homosassa Springs	Citrus	Nontransient Noncommunity	50	5000	Retail/General Merchant
6090729	Inverness	Citrus	Community	63	72,000	Mobile Home Park
6090828	Homosassa	Citrus	Community	6429	1,580,000	Recreation Area
6090828	Homosassa	Citrus	Community	6429	1,580,000	Recreation Area
6090828	Homosassa	Citrus	Community	6429	1,580,000	Recreation Area
6090828	Homosassa	Citrus	Community	6429	1,580,000	Recreation Area
6090860	Inverness	Citrus	Community	383	294,000	Apartment
6090860	Inverness	Citrus	Community	383	294,000	Apartment
6090861	Inverness	Citrus	Community	7295	3,450,000	Municipal/City
6090875	Inverness	Citrus	Community	200	40,000	Mobile Home Park
6090898	Inverness	Citrus	Community	48	36,000	Mobile Home Park

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 3 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6091178	Longwood	Citrus	Community	113	160,000	Subdivision
6091178	Longwood	Citrus	Community	113	160,000	Subdivision
6091193	Hernando	Citrus	Noncommunity	50	-	Mobile Home Park
6091608	Crystal River	Citrus	Noncommunity	75	43,000	Recreation Area
6091625	Homosassa	Citrus	Community	213	93,000	Mobile Home Park
6091625	Homosassa	Citrus	Community	213	93,000	Mobile Home Park
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091735	Homosassa	Citrus	Community	10,308	4,960,000	Subdivision
6091798	Crystal River	Citrus	Nontransient Noncommunity	50	30,000	Lodge
6091798	Crystal River	Citrus	Nontransient Noncommunity	50	30,000	Lodge
6091816	Crystal River	Citrus	Community	81	33,000	Mobile Home Park
6091876	Crystal River	Citrus	Community	80	-	Mobile Home Park
6092000	Homosassa	Citrus	Noncommunity	25	10,000	Recreation Area
6092049	Inverness	Citrus	Noncommunity	58	-	Mobile Home Park

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 4 of 17) Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6092197	Inverness	Citrus	Community	464	565,000	Subdivision
6092197	Inverness	Citrus	Community	464	565,000	Subdivision
6092199	Dunnellon	Citrus	Community	290	86,000	Subdivision
6092199	Dunnellon	Citrus	Community	290	86,000	Subdivision
6092262	Brooksville	Citrus	Noncommunity	25	-	Camp Ground
6092328	Lecanto	Citrus	Community	89	-	Mobile Home Park
6092328	Lecanto	Citrus	Community	89	-	Mobile Home Park
6092329	Hernando	Citrus	Noncommunity	60	86,000	Camp Ground
6092334	Beverly Hills	Citrus	Community	90	150,000	Mobile Home Park
6092334	Beverly Hills	Citrus	Community	90	150,000	Mobile Home Park
6092338	Crystal River	Citrus	Community	93	70,000	Subdivision
6092438	Homosassa Springs	Citrus	Noncommunity	100	-	Restaurant
6092698	Inverness	Citrus	Noncommunity	25	-	Restaurant
6092760	Hernando	Citrus	Noncommunity	45	-	Bar Or Lounge
6092767	Hernando	Citrus	Noncommunity	25	-	Bar Or Lounge
6092772	Lecanto	Citrus	Noncommunity	1000	-	RV Park
6092921	Inverness	Citrus	Noncommunity	50	-	Lodge
6092922	Hernando	Citrus	Noncommunity	25	-	Other
6092959	Homosassa Springs	Citrus	Noncommunity	50	-	Bar or Lounge

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 5 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6092963	Homosassa	Citrus	Noncommunity	26	2000	Restaurant
6092965	Inverness	Citrus	Noncommunity	25	-	Bar or Lounge
6093075	Inverness	Citrus	Noncommunity	25	-	Retail/General Merchant
6093077	Crystal River	Citrus	Noncommunity	25	-	Retail/General Merchant
6093082	Inverness	Citrus	Noncommunity	25	-	Retail/General Merchant
6094480	Dunnellon	Citrus	Community	112	90,000	Subdivision
6094480	Dunnellon	Citrus	Community	112	90,000	Subdivision
6094556	Inverness	Citrus	Community	180	396,000	Subdivision
6094556	Inverness	Citrus	Community	180	396,000	Subdivision
6094656	Longwood	Citrus	Community	192	72,000	Mobile Home Park
6094713	Hudson	Citrus	Community	505	400,000	Subdivision
6094713	Hudson	Citrus	Community	505	400,000	Subdivision
6094773	Lecanto	Citrus	Community	800	360,000	Subdivision
6094773	Lecanto	Citrus	Community	800	360,000	Subdivision
6094870	Homoasassa	Citrus	Noncommunity	25	2000	Convenience Store
6094871	Hernando	Citrus	Community	220	887,000	Subdivision
6094871	Hernando	Citrus	Community	220	887,000	Subdivision
6094872	Inverness	Citrus	Noncommunity	25	-	Other
6094874	Crystal River	Citrus	Community	100	-	Subdivision

Table 2.4.12-205 (Sheet 6 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

LNP COL 2.4-4

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6094875	Dunellon	Citrus	Community	100	129,600	Subdivision
6094876	Beverly Hills	Citrus	Noncommunity	25	28,000	Retail/General Merchant
6094878	Hernando	Citrus	Noncommunity	25	-	Retail/General Merchant
6094879	Hernando	Citrus	Noncommunity	25	-	Convenience Store
6094883	Crystal River	Citrus	Nontransient Noncommunity	25	-	Office For Business
6094886	Homosassa Springs	Citrus	Community	140	41,000	Subdivision
6094886	Homosassa Springs	Citrus	Community	140	41,000	Subdivision
6094895	Hernando	Citrus	Noncommunity	25	-	Bar or Lounge
6094898	Hernando	Citrus	Noncommunity	25	-	Restaurant
6094899	Homosassa Springs	Citrus	Community	80	-	Subdivision
6094908	Homosassa Springs	Citrus	Noncommunity	25	-	Retail/General Merchant
6094913	Homosassa	Citrus	Noncommunity	25	-	Convenience Store
6094915	Inverness	Citrus	Noncommunity	74	-	Retail/General Merchant
6094916	Homosassa	Citrus	Noncommunity	25	-	Convenience Store
6094918	Lecanto	Citrus	Noncommunity	25	-	Retail/General Merchant
6094924	Hernando	Citrus	Noncommunity	25	-	Bar or Lounge
6094926	Hernando	Citrus	Noncommunity	25	-	Restaurant
6094931	Homosassa	Citrus	Noncommunity	25	-	Convenience Store
6094934	Orlando	Citrus	Community	168	43,500	Subdivision

Table 2.4.12-205 (Sheet 7 of 17)

LNP COL 2.4-4 **Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6094939	Homosassa	Citrus	Noncommunity	25	7200	Retail/General Merchant
6094941	Homosassa	Citrus	Noncommunity	25	14,400	Restaurant
6094942	Homosassa	Citrus	Noncommunity	25	-	Retail/General Merchant
6094944	Holder	Citrus	Noncommunity	25	-	Restaurant
6094947	Homosassa	Citrus	Noncommunity	25	86,400	Church
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094948	Lecanto	Citrus	Community	19,715	14,544,000	County Wide
6094953	Inverness	Citrus	Community	62	72,000	Subdivision
6094954	Holder	Citrus	Noncommunity	25	-	Restaurant
6094957	Longwood	Citrus	Community	172	-	Subdivision
6094969	Hernando	Citrus	Community	165	1,080,000	Subdivision
6094971	Hernando	Citrus	Noncommunity	25	-	Convenience Store
6094975	Homosassa	Citrus	Noncommunity	25	2000	Convenience Store
6094976	Inverness	Citrus	Noncommunity	25	-	Church

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 8 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6094980	Inverness	Citrus	Noncommunity	30	-	Convenience Store
6094987	Hernando	Citrus	Noncommunity	25	1725	Convenience Store
6094989	Lecanto	Citrus	Noncommunity	25	-	Restaurant
6094993	Homosassa Springs	Citrus	Noncommunity	50	-	Office for Business
6094994	Hernando	Citrus	Community	100	-	Apartment
6095001	Holder	Citrus	Noncommunity	25	-	Convenience Store
6095002	Homosassa	Citrus	Noncommunity	25	28,080	Bar or Lounge
6095003	Homosassa	Citrus	Noncommunity	150	1800	Restaurant
6095005	Lecanto	Citrus	Nontransient Noncommunity	25	1000	Day Care
6095006	Inverness	Citrus	Nontransient Noncommunity	100	-	Other
6095013	Inverness	Citrus	Nontransient Noncommunity	40	7500	Church
6095014	Holder	Citrus	Noncommunity	25	-	Restaurant
6095022	Inverness	Citrus	Noncommunity	100	11,000	Medical Center
6095025	Homosassa	Citrus	Noncommunity	100	-	Church
6095028	Inverness	Citrus	Noncommunity	25	-	Church
6095029	Hernando	Citrus	Noncommunity	25	-	Church
6095031	Homosassa Springs	Citrus	Noncommunity	25	2000	Other
6095032	Homosassa	Citrus	Noncommunity	25	2000	Convenience Store

Table 2.4.12-205 (Sheet 9 of 17)

LNP COL 2.4-4 **Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6095033	Inverness	Citrus	Noncommunity	25	-	Convenience Store
6095042	Hernando	Citrus	Noncommunity	25	-	Restaurant
6095044	Crystal River	Citrus	Nontransient Noncommunity	25	-	Company Town
6095045	Homosassa	Citrus	Noncommunity	25	-	Bar or Lounge
6095046	Lecanto	Citrus	Community	25	5000	Subdivision
6095046	Lecanto	Citrus	Community	25	5000	Subdivision
6095047	Hernando	Citrus	Noncommunity	25	-	Recreation Area
6095051	Crystal River	Citrus	Noncommunity	45	-	Church
6382055	Williston	Levy	Community	1250	1,760,000	Municipal/City
6384607	Williston	Levy	Noncommunity	30	57,399	Mobile Home Park
6384622	Gulf Hammock	Levy	Noncommunity	25	9550	Convenience Store
6384623	Morriston	Levy	Noncommunity	25	36,000	Convenience Store
6384626	Williston	Levy	Noncommunity	37	11,472	Convenience Store
6384627	Williston	Levy	Noncommunity	41	4629	Convenience Store
6384629	Morriston	Levy	Noncommunity	100	9319	Convenience Store
6384630	Williston	Levy	Noncommunity	41	5000	Restaurant
6384635	Williston	Levy	Noncommunity	25	10,000	Recreation Area
6421144	Ocala	Marion	Community	14,749	2,962,000	Subdivision
6421470	Dunnellon	Marion	Noncommunity	99	114,000	Camp Ground

Table 2.4.12-205 (Sheet 10 of 17)
LNP COL 2.4-4 **Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6421470	Dunnellon	Marion	Noncommunity	99	114,000	Camp Ground
6421472	Dunnellon	Marion	Noncommunity	25	216,000	Office for Business
6421472	Dunnellon	Marion	Noncommunity	25	216,000	Office for Business
6421512	Ocala	Marion	Community	400	227,000	Subdivision
6421512	Ocala	Marion	Community	400	227,000	Subdivision
6421561	Dunnellon	Marion	Noncommunity	25	21,000	Mobile Home Park
6422679	Dunnellon	Marion	Community	2300	1,500,000	Subdivision
6422679	Dunnellon	Marion	Community	2300	1,500,000	Subdivision
6424071	Ocala	Marion	Noncommunity	70	43,200	Mobile Home Park
6424076	Ocala	Marion	Community	1779	636,000	Subdivision
6424076	Ocala	Marion	Community	1779	636,000	Subdivision
6424371	Ocala	Marion	Community	86	10,000	Subdivision
6424618	Ocala	Marion	Community	385	430,000	Subdivision
6424619	Ocala	Marion	Community	6763	5,760,000	Subdivision
6424619	Ocala	Marion	Community	6763	5,760,000	Subdivision
6424619	Ocala	Marion	Community	6763	5,760,000	Subdivision
6424619	Ocala	Marion	Community	6763	5,760,000	Subdivision
6424620	Ocala	Marion	Community	650	4,80,000	Restaurant
6424620	Ocala	Marion	Community	650	4,80,000	Restaurant

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 11 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6424620	Ocala	Marion	Community	650	4,80,000	Restaurant
6424622	Dunnellon	Marion	Nontransient Noncommunity	1000	-	High School
6424622	Dunnellon	Marion	Nontransient Noncommunity	1000	-	High School
6424622	Dunnellon	Marion	Nontransient Noncommunity	1000	-	High School
6424622	Dunnellon	Marion	Nontransient Noncommunity	1000	-	High School
6424623	Ocala	Marion	Community	323	36,000	Mobile Home Park
6424623	Ocala	Marion	Community	323	36,000	Mobile Home Park
6424625	Ocala	Marion	Noncommunity	25	90,000	Office For Business
6424627	Ocala	Marion	Community	1000	1,704,000	Subdivision
6424627	Ocala	Marion	Community	1000	1,704,000	Subdivision
6424628	Ocala	Marion	Noncommunity	25	-	Office For Business
6424629	Ocala	Marion	Community	435	-	Restaurant
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424632	Ocala	Marion	Community	1722	560,000	Subdivision
6424632	Ocala	Marion	Community	1722	560,000	Subdivision

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 12 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6424633	Dunnellon	Marion	Nontransient Noncommunity	25	43,200	Church
6424635	Ocala	Marion	Nontransient Noncommunity	25	-	Restaurant
6424636	Ocala	Marion	Nontransient Noncommunity	25	-	Church
6424638	Dunnellon	Marion	Noncommunity	25	-	Restaurant
6424639	Ocala	Marion	Noncommunity	25	-	Retail/General Merchant
6424640	Dunnellon	Marion	Noncommunity	25	-	Retail/General Merchant
6424646	Ocala	Marion	Noncommunity	25	-	Retail/General Merchant
6424651	Ocala	Marion	Noncommunity	25	122,400	Subdivision
6424652	Ocala	Marion	Community	4000	1,530,000	Subdivision
6424653	Ocala	Marion	Community	165	24,200	Mobile Home Park
6424655	Ocala	Marion	Noncommunity	25	-	Service Station
6424659	Ocala	Marion	Noncommunity	25	-	Church
6424660	Ocala	Marion	Noncommunity	25	-	Retail/General Merchant
6424661	Dunnellon	Marion	Noncommunity	25	23,760	Bathing/Swimming
6424663	Ocala	Marion	Noncommunity	25	36,000	Convenience Store
6424666	Ocala	Marion	Noncommunity	25	36,000	Other
6424667	Dunnellon	Marion	Noncommunity	25	36,000	Church
6424669	Ocala	Marion	Noncommunity	25	-	Convenience Store

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 13 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6424673	Ocala	Marion	Community	35	72,000	Office For Business
6424673	Ocala	Marion	Community	35	72,000	Office For Business
6424678	Ocala	Marion	Community	490	1,950,000	Subdivision
6424678	Ocala	Marion	Community	490	1,950,000	Subdivision
6424679	Ocala	Marion	Noncommunity	25	-	Convenience Store
6424685	Ocala	Marion	Noncommunity	25	-	Restaurant
6424686	Dunnellon	Marion	Noncommunity	600	4000	Church
6424694	Ocala	Marion	Noncommunity	25	-	Church
6424695	Ocala	Marion	Noncommunity	25	-	Convenience Store
6424700	Ocala	Marion	Noncommunity	25	-	Office for Business
6424703	Ocala	Marion	Noncommunity	25	-	Church
6424704	Ocala	Marion	Community	405	255,000	Mobile Home Park
6424706	Dunnellon	Marion	Community	50	72,000	Other
6424712	Ocala	Marion	Nontransient Noncommunity	74	1500	Office for Business
6424719	Ocala	Marion	Noncommunity	25	-	Convenience Store
6424720	Ocala	Marion	Noncommunity	25	-	Convenience Store
6424722	Ocala	Marion	Noncommunity	25	-	Restaurant
6424723	Ocala	Marion	Noncommunity	25	22,000	Other
6424724	Ocala	Marion	Noncommunity	25	-	Retail/General Merchant

LNP COL 2.4-4

Table 2.4.12-205 (Sheet 14 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6424726	Ocala	Marion	Nontransient Noncommunity	200	20,000	Retail/General Merchant
6424728	Ocala	Marion	Noncommunity	25	-	Restaurant
6424729	Dunnellon	Marion	Nontransient Noncommunity	100	2000	Elementary School
6424729	Dunnellon	Marion	Nontransient Noncommunity	100	2000	Elementary School
6424730	Dunnellon	Marion	Noncommunity	25	-	Convenience Store
6424732	Dunnellon	Marion	Noncommunity	25	-	Restaurant
6424735	Ocala	Marion	Noncommunity	30	-	Office for Business
6424736	Dunnellon	Marion	Noncommunity	25	25	Office for Business
6424738	Ocala	Marion	Noncommunity	25	-	Church
6424739	Ocala	Marion	Noncommunity	50	-	Church
6424741	Dunnellon	Marion	Community	40	23,000	Nursing Home
6424746	Ocala	Marion	Nontransient Noncommunity	137	2000	Day Care
6424749	Dunnellon	Marion	Community	1456	130,2000	Subdivision
6424749	Dunnellon	Marion	Community	1456	130,2000	Subdivision
6424749	Dunnellon	Marion	Community	1456	130,2000	Subdivision
6424750	Ocala	Marion	Noncommunity	25	7560	Retail/General Merchant
6424751	Dunnellon	Marion	Noncommunity	25	-	Bar or Lounge

LNP COL 2.4-4

**Table 2.4.12-205 (Sheet 15 of 17)
Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6424752	Ocala	Marion	Noncommunity	25	2000	Office for Business
6424753	Ocala	Marion	Noncommunity	50	675	Church
6424760	Dunnellon	Marion	Noncommunity	25	2500	Church
6424763	Ocala	Marion	Noncommunity	25	-	Other
6424764	Ocala	Marion	Noncommunity	30	4500	Church
6424768	Ocala	Marion	Community	28	104,000	Subdivision
6424770	Dunnellon	Marion	Noncommunity	25	5000	Restaurant
6424771	Ocala	Marion	Noncommunity	100	-	Church
2381441	Williston	Levy	Nontransient Noncommunity	500	230,4000	Industrial
2381441	Williston	Levy	Nontransient Noncommunity	500	230,4000	Industrial
6095054	Crystal River	Citrus	Noncommunity	100	2000	Church
6424772	Ocala	Marion	Noncommunity	84	2000	Recreation Area
6095058	Crystal River	Citrus	Nontransient Noncommunity	35	2000	Day Care
6424778	Ocala	Marion	Noncommunity	35	10,000	Church
6424780	Ocala	Marion	Noncommunity	25	5000	Recreation Area
6095061	Homosassa	Citrus	Noncommunity	25	10,000	Convenience Store
6424782	Ocala	Marion	Noncommunity	25	5000	Recreation Area
6424787	Dunnellon	Marion	Noncommunity	30	5000	Church

Table 2.4.12-205 (Sheet 16 of 17)

LNP COL 2.4-4 **Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6095064	Hernando	Citrus	Noncommunity	25	5000	Retail/General Merchant
6424678	Ocala	Marion	Community	490	1,950,000	Subdivision
6424662	Dunnellon	Marion	Noncommunity	25	150,000	Airport
6424662	Dunnellon	Marion	Noncommunity	25	150,000	Airport
6424793	Dunnellon	Marion	Noncommunity	25	1400	Convenience Store
6424652	Ocala	Marion	Community	4000	1,530,000	Subdivision
6424796	Ocala	Marion	Noncommunity	350	3000	Church
6422679	Dunnellon	Marion	Community	2300	1,500,000	Subdivision
2381446	Williston	Marion	Noncommunity	100	-	Church
6424802	Ocala	Marion	Noncommunity	25	10,000	Lodge
6424807	Ocala	Marion	Noncommunity	50	5000	Retail/General Merchant
6424808	Ocala	Marion	Noncommunity	25	-	Restaurant
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision

Table 2.4.12-205 (Sheet 17 of 17)

LNP COL 2.4-4 **Public Water Supply Users between 16.1 and 40.2 Km (10 and 25 Mi.) of the LNP Site**

Public Water Supply ID	City	County	Public Water Supply Type	Population Served	Design Capacity (gpd)	Primary Use
6382055	Williston	Levy	Community	1250	1,760,000	Municipal/City
2381452	Morriston	Levy	Noncommunity	36	4704	Restaurant
6424630	Ocala	Marion	Community	11,760	6,141,600	Subdivision
6092000	Homosassa	Citrus	Noncommunity	25	10,000	Recreation Area
6384625	Williston	Levy	Noncommunity	32	11,570	Restaurant
6090312	Dunnellon	Citrus	Community	15,675	5,158,000	Subdivision
6090312	Dunnellon	Citrus	Community	15,675	5,158,000	Subdivision
6090312	Dunnellon	Citrus	Community	15,675	5,158,000	Subdivision
6424811	Ocala	Marion	Noncommunity	25	1000	Convenience Store

Notes:

- = Data not available.
gpd = gallon per day
RV = recreational vehicle

Source: Reference 2.4.12-213

LNP COL 2.4-4

Table 2.4.12-206
Nearest Residences Relative to the LNP Site

	Nearest Residence
Sector	**Distance From LNP 1/LNP 2 (mi.)**
N	3.2
NNE	4.1
NE	–
ENE	–
E	4.8
ESE	3.7
SE	2.6
SSE	2.9
S	4.2
SSW	2.8
SW	2
WSW	1.7
W	–
WNW	–
NW	1.6
NNW	2.4

Notes:

Distances measured from the center point of LNP 1 and LNP 2
"–" indicates that no receptor was identified within 8 km (5 mi.)

mi. = miles
E = east, W = west, N = north, S = south

LNP COL 2.4-4

Table 2.4.12-207
Summary of Piezometer and Monitoring Well Construction Details for the LNP Site

Well ID	Surficial or Bedrock Aquifer	Northing	Easting	Ground Elevation[b]	Top of Casing (TOC) Elevation	Flush / Stick-up	Height from TOC to Ground Surface	Depth, Top of Screen	Depth, Bottom of Screen[e]	Measured Total Depth[f]	Riser Material	Riser Diameter	Screen Length	Borehole Log / Completion Form available?	Date Installed
		(NAD83)[a]		(feet NAVD88)[c]			(feet)	(feet BTOC)[d]	(feet BTOC)	(feet BTOC)		(inch)	(feet)		
MW-1S	Surficial	1719510.77	455053.80	41.95	45.09	Stick-up	3.14	23.40	33.40	33.65	Sch 40 PVC	2	10	Y/Y	1/30/2007
MW-2S	Surficial	1729669.64	455298.09	43.34	45.84	Stick-up	2.50	23.83	33.83	34.08	Sch 40 PVC	2	10	Y/Y	1/29/2007
MW-3S	Surficial	1730335.14	460606.33	48.41	51.55	Stick-up	3.14	23.49	33.49	33.74	Sch 40 PVC	2	10	Y/Y	1/31/2007
MW-4S	Surficial	1721283.93	461369.67	46.38	48.83	Stick-up	2.45	22.80	32.80	33.05	Sch 40 PVC	2	10	Y/Y	1/30/2007
MW-5S	Surficial	1724805.79	456749.35	42.80	45.52	Stick-up	2.72	23.50	33.50	33.75	Sch 40 PVC	2	10	Y/Y	2/6/2007
MW-6D	Bedrock	1724807.03	456756.14	42.66	45.59	Stick-up	2.93	114.11	124.11	124.36	Sch 40 PVC	2	10	Y/Y	2/5/2007
MW-7S	Surficial	1724925.96	458463.69	44.22	46.91	Stick-up	2.69	23.02	33.02	33.27	Sch 40 PVC	2	10	Y/Y	2/1/2007
MW-8D	Bedrock	1724922.19	458475.09	44.00	46.83	Stick-up	2.83	143.25	153.25	153.50	Sch 40 PVC	2	10	Y/Y	1/31/2007
MW-9S	Surficial	1722583.32	458432.35	43.45	46.08	Stick-up	2.63	23.05	33.05	33.30	Sch 40 PVC	2	10	Y/Y	2/6/2007
MW-10D	Bedrock	1722591.07	458428.60	43.51	46.00	Stick-up	2.49	113.67	123.67	123.92	Sch 40 PVC	2	10	Y/Y	1/31/2007
MW-11S	Surficial	1722919.13	456631.88	42.06	44.70	Stick-up	2.64	22.19	32.19	32.44	Sch 40 PVC	2	10	Y/Y	2/13/2007
MW-12D	Bedrock	1722919.35	456622.58	41.89	44.54	Stick-up	2.65	113.39	123.39	123.64	Sch 40 PVC	2	10	Y/Y	2/13/2007
MW-13S	Surficial	1724099.32	457688.61	42.58	45.78	Stick-up	3.20	23.33	33.33	33.58	Sch 40 PVC	2	10	Y/Y	2/7/2007
MW-14D	Bedrock	1724099.32	457695.56	42.56	45.72	Stick-up	3.16	113.84	123.84	124.09	Sch 40 PVC	2	10	Y/Y	2/6/2007
MW-15S	Surficial	1723091.18	458117.36	43.35	46.24	Stick-up	2.89	23.18	33.18	33.43	Sch 40 PVC	2	10	Y/Y	2/11/2007
MW-16D	Bedrock	1723086.94	458110.46	43.34	46.01	Stick-up	2.67	112.50	122.50	122.75	Sch 40 PVC	2	10	Y/Y	2/12/2007
OW-1	Surficial	1724114.06	457688.26	43.21	45.89	Stick-up	2.68	23.31	33.31	33.56	Sch 40 PVC	2	10	Y/Y	2/7/2007
OW-2	Surficial	1724084.22	457702.12	42.56	45.62	Stick-up	3.06	22.96	32.96	33.21	Sch 40 PVC	2	10	Y/Y	2/10/2007
OW-3	Surficial	1724083.78	457718.85	42.39	45.48	Stick-up	3.09	22.97	32.97	33.22	Sch 40 PVC	2	10	Y/Y	2/6/2007
OW-4	Surficial	1724074.76	457678.12	42.41	45.48	Stick-up	3.07	23.05	33.05	33.30	Sch 40 PVC	2	10	Y/Y	2/6/2007
OW-5	Bedrock	1724076.16	457702.65	43.15	45.53	Stick-up	2.38	112.84	122.84	123.09	Sch 40 PVC	2	10	Y/Y	2/8/2007
OW-6	Intermediate	1724100.18	457680.60	42.46	45.57	Stick-up	3.11	68.86	78.86	79.11	Sch 40 PVC	2	10	Y/Y	1/31/2007
OW-7	Intermediate	1724092.30	457702.57	42.59	45.61	Stick-up	3.02	68.77	78.77	79.02	Sch 40 PVC	2	10	Y/Y	2/9/2007
PW-1	Surficial	1724085.88	457687.90	41.99	45.82	Stick-up	3.83	13.68	33.43	33.68	Sch 40 PVC	6	20	Y/Y	2/13/2007

Notes:

a) NAD83 = North American Datum 83 (1999) SPC FL W US Survey Feet.

b) Ground surface elevation is measured from the concrete well pad at the base of the well with the exception of PW-1, which was taken from ground surface because a well pad was not installed.

c) NAVD88 = North American Vertical Datum 88.

d) BTOC = below top-of-casing.

e) Well is finished with a 3-inch flat bottom PVC sump attached to the bottom of the screen.

f) Measured in the field on March 6, 2007, by CH2M HILL.

LNP COL 2.4-4

Table 2.4.12-208 (Sheet 1 of 2)
Summary of Groundwater Levels within the Plant Area

	Ground Elevation	Top of Casing (TOC) Elevation	Groundwater Surface Elevation			
Well Identification	(feet NAVD88)	(feet NAVD88)	March 6, 2007	June 14, 2007	September 13, 2007	December 4, 2007
MW-1S	41.95	45.09	40.50	37.40	36.21	36.31
MW-2S	43.34	45.84	41.93	37.98	36.87	36.21
MW-3S	48.41	51.55	45.82	41.93	41.12	40.66
MW-4S	46.38	48.83	45.09	41.78	40.77	40.93
MW-5S	42.80	45.52	41.74	39.14	37.68	37.59
MW-6D	42.66	45.59	41.40	38.59	37.18	37.27
MW-7S	44.22	46.91	42.54	39.30	38.03	37.99
MW-8D	44.00	46.83	42.21	39.28	37.95	37.97
MW-9S	43.45	46.08	41.75	39.22	37.94	38.05
MW-10D	43.51	46.00	41.72	38.95	37.47	37.71
MW-11S	42.06	44.70	41.30	39.28	37.64	37.66
MW-12D	41.89	44.54	40.73	37.83	36.49	36.58
MW-13S	42.58	45.78	41.94	39.17	37.66	37.70
MW-14D	42.56	45.72	41.83	38.91	37.39	37.56
MW-15S	43.35	46.24	42.05	39.25	38.01	37.88
MW-16D	43.34	46.01	41.73	38.93	37.46	37.68
OW-1	43.21	45.89	41.96	39.17	37.64	37.71
OW-2	42.56	45.62	42.09	39.25	37.73	37.80

LNP COL 2.4-4

Table 2.4.12-208 (Sheet 2 of 2)
Summary of Groundwater Levels within the Plant Area

	Ground Elevation	Top of Casing (TOC) Elevation	Groundwater Surface Elevation			
Well Identification	**(feet NAVD88)**	**(feet NAVD88)**	**March 6, 2007**	**June 14, 2007**	**September 13, 2007**	**December 4, 2007**
OW-3	42.39	45.48	42.12	39.20	37.68	37.75
OW-4	42.41	45.48	41.97	39.13	37.61	37.69
OW-5	43.15	45.53	41.75	38.87	37.38	37.54
OW-6	42.46	45.57	41.89	39.08	37.55	37.64
OW-7	42.59	45.61	41.98	39.14	37.63	37.73
PW-1	41.99	45.82	42.00	39.17	37.65	37.72

Notes:

Elevation units are feet NAVD88.

Ground surface elevation is measured from the concrete well pad at the base of the well with the exception of PW-1 which was taken from ground surface because a well pad was not installed.

LNP COL 2.4-4

Table 2.4.12-209 (Sheet 1 of 4)
Summary of Groundwater Vertical Gradients within the LNP Site

					March 6, 2007										
		Top of Casing (TOC) Elevation	Depth to Well Screen	Screen Length	Depth to Water	Bottom of Screen to Top of Screen (L:H)		Top of Screen to Top of Screen (H:H)		Mid-point of Screen to Mid-point of Screen (M:M)		Bottom of Screen to Bottom of Screen (L:L)		Top of Screen to Bottom of Screen (H:L)	
Well Identification	Interval	(feet NAVD88)	(feet BTOC)	(feet)	(feet BTOC)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)
MW-5S	Shallow	45.52	23.50	10.0	3.78	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
MW-6D	Deep	45.59	114.11	10.0	4.19										
MW-7S	Shallow	46.91	23.02	10.0	4.37	0.003	Down	0.003	Down	0.003	Down	0.003	Down	0.003	Down
MW-8D	Deep	46.83	143.25	10.0	4.62										
MW-9S	Shallow	46.08	23.05	10.0	4.33	0.0003	Down	0.0003	Down	0.0003	Down	0.0003	Down	0.0004	Down
MW-10D	Deep	46.00	113.67	10.0	4.28										
MW-11S	Shallow	44.70	22.19	10.0	3.40	0.006	Down	0.006	Down	0.006	Down	0.006	Down	0.007	Down
MW-12D	Deep	44.54	113.39	10.0	3.81										
MW-13S	Shallow	45.78	23.33	10.0	3.84	0.001	Down	0.001	Down	0.001	Down	0.001	Down	0.001	Down
MW-14D	Deep	45.72	113.84	10.0	3.89										
MW-15S	Shallow	46.24	23.18	10.0	4.19	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
MW-16D	Deep	46.01	112.50	10.0	4.28										
OW-2	Shallow	45.62	22.96	10.0	3.53	0.002	Down	0.002	Down	0.002	Down	0.002	Down	0.003	Down
OW-7	Intermediate	45.61	68.77	10.0	3.63										
OW-7	Intermediate	45.61	68.77	10.0	3.63	0.004	Down	0.005	Down	0.005	Down	0.005	Down	0.007	Down
OW-5	Deep	45.53	112.84	10.0	3.78										
OW-2	Shallow	45.62	22.96	10.0	3.53	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
OW-5	Deep	45.53	112.84	10.0	3.78										
OW-6	Intermediate	45.57	68.86	10.0	3.68	0.001	Down	0.001	Down	0.001	Down	0.001	Down	0.002	Down
MW-14D	Deep	45.72	113.84	10.0	3.89										

LNP COL 2.4-4

Table 2.4.12-209 (Sheet 2 of 4)
Summary of Groundwater Vertical Gradients within the LNP Site

		Top of Casing (TOC) Elevation	Depth to Well Screen	Screen Length	June 14, 2007										
					Depth to Water	Bottom of Screen to Top of Screen (L:H)		Top of Screen to Top of Screen (H:H)		Mid-point of Screen to Mid-point of Screen (M:M)		Bottom of Screen to Bottom of Screen (L:L)		Top of Screen to Bottom of Screen (H:L)	
Well Identification	Interval	(feet NAVD88)	(feet BTOC)	(feet)	(feet BTOC)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)
MW-5S	Shallow	45.52	23.50	10.0	6.38	0.005	Down	0.006	Down	0.006	Down	0.006	Down	0.007	Down
MW-6D	Deep	45.59	114.11	10.0	7.00										
MW-7S	Shallow	46.91	23.02	10.0	7.61	0.0002	Down	0.0002	Down	0.0002	Down	0.0002	Down	0.0002	Down
MW-8D	Deep	46.83	143.25	10.0	7.55										
MW-9S	Shallow	46.08	23.05	10.0	6.86	0.003	Down	0.003	Down	0.003	Down	0.003	Down	0.003	Down
MW-10D	Deep	46.00	113.67	10.0	7.05										
MW-11S	Shallow	44.70	22.19	10.0	5.42	0.014	Down	0.016	Down	0.016	Down	0.016	Down	0.018	Down
MW-12D	Deep	44.54	113.39	10.0	6.71										
MW-13S	Shallow	45.78	23.33	10.0	6.61	0.003	Down	0.003	Down	0.003	Down	0.003	Down	0.003	Down
MW-14D	Deep	45.72	113.84	10.0	6.81										
MW-15S	Shallow	46.24	23.18	10.0	6.99	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
MW-16D	Deep	46.01	112.50	10.0	7.08										
OW-2	Shallow	45.62	22.96	10.0	6.37	0.002	Down	0.002	Down	0.002	Down	0.002	Down	0.003	Down
OW-7	Intermediate	45.61	68.77	10.0	6.47										
OW-7	Intermediate	45.61	68.77	10.0	6.47	0.005	Down	0.006	Down	0.006	Down	0.006	Down	0.008	Down
OW-5	Deep	45.53	112.84	10.0	6.66										
OW-2	Shallow	45.62	22.96	10.0	6.37	0.004	Down	0.004	Down	0.004	Down	0.004	Down	0.005	Down
OW-5	Deep	45.53	112.84	10.0	6.66										
OW-6	Intermediate	45.57	68.86	10.0	6.49	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.005	Down
MW-14D	Deep	45.72	113.84	10.0	6.81										

LNP COL 2.4-4

Table 2.4.12-209 (Sheet 3 of 4)
Summary of Groundwater Vertical Gradients within the LNP Site

					September 13, 2007										
		Top of Casing (TOC) Elevation	Depth to Well Screen	Screen Length	Depth to Water	Bottom of Screen to Top of Screen (L:H)		Top of Screen to Top of Screen (H:H)		Mid-point of Screen to Mid-point of Screen (M:M)		Bottom of Screen to Bottom of Screen (L:L)		Top of Screen to Bottom of Screen (H:L)	
Well Identification	Interval	(feet NAVD88)	(feet BTOC)	(feet)	(feet BTOC)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)
MW-5S	Shallow	45.52	23.50	10.0	7.84	0.005	Down	0.006	Down	0.006	Down	0.006	Down	0.006	Down
MW-6D	Deep	45.59	114.11	10.0	8.41										
MW-7S	Shallow	46.91	23.02	10.0	8.88	0.001	Down	0.001	Down	0.001	Down	0.001	Down	0.001	Down
MW-8D	Deep	46.83	143.25	10.0	8.88										
MW-9S	Shallow	46.08	23.05	10.0	8.14	0.005	Down	0.005	Down	0.005	Down	0.005	Down	0.006	Down
MW-10D	Deep	46.00	113.67	10.0	8.53										
MW-11S	Shallow	44.70	22.19	10.0	7.06	0.011	Down	0.013	Down	0.013	Down	0.013	Down	0.014	Down
MW-12D	Deep	44.54	113.39	10.0	8.05										
MW-13S	Shallow	45.78	23.33	10.0	8.12	0.003	Down	0.003	Down	0.003	Down	0.003	Down	0.003	Down
MW-14D	Deep	45.72	113.84	10.0	8.33										
MW-15S	Shallow	46.24	23.18	10.0	8.23	0.006	Down	0.006	Down	0.006	Down	0.006	Down	0.007	Down
MW-16D	Deep	46.01	112.50	10.0	8.55										
OW-2	Shallow	45.62	22.96	10.0	7.89	0.002	Down	0.002	Down	0.002	Down	0.002	Down	0.003	Down
OW-7	Intermediate	45.61	68.77	10.0	7.98										
OW-7	Intermediate	45.61	68.77	10.0	7.98	0.005	Down	0.006	Down	0.006	Down	0.006	Down	0.007	Down
OW-5	Deep	45.53	112.84	10.0	8.15										
OW-2	Shallow	45.62	22.96	10.0	7.89	0.004	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
OW-5	Deep	45.53	112.84	10.0	8.15										
OW-6	Intermediate	45.57	68.86	10.0	8.02	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.005	Down
MW-14D	Deep	45.72	113.84	10.0	8.33										

LNP COL 2.4-4

Table 2.4.12-209 (Sheet 4 of 4)
Summary of Groundwater Vertical Gradients within the LNP Site

					December 4, 2007										
		Top of Casing (TOC) Elevation	Depth to Well Screen	Screen Length	Depth to Water	Bottom of Screen to Top of Screen (L:H)		Top of Screen to Top of Screen (H:H)		Mid-point of Screen to Mid-point of Screen (M:M)		Bottom of Screen to Bottom of Screen (L:L)		Top of Screen to Bottom of Screen (H:L)	
Well Identification	Interval	(feet NAVD88)	(feet BTOC)	(feet)	(feet BTOC)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)
MW-5S	Shallow	45.52	23.50	10.0	7.93	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
MW-6D	Deep	45.59	114.11	10.0	8.32										
MW-7S	Shallow	46.91	23.02	10.0	8.92	0.0002	Down	0.0002	Down	0.0002	Down	0.0002	Down	0.0002	Down
MW-8D	Deep	46.83	143.25	10.0	8.86										
MW-9S	Shallow	46.08	23.05	10.0	8.03	0.003	Down	0.004	Down	0.004	Down	0.004	Down	0.004	Down
MW-10D	Deep	46.00	113.67	10.0	8.29										
MW-11S	Shallow	44.70	22.19	10.0	7.04	0.011	Down	0.012	Down	0.012	Down	0.012	Down	0.013	Down
MW-12D	Deep	44.54	113.39	10.0	7.96										
MW-13S	Shallow	45.78	23.33	10.0	8.08	0.001	Down	0.002	Down	0.002	Down	0.002	Down	0.002	Down
MW-14D	Deep	45.72	113.84	10.0	8.16										
MW-15S	Shallow	46.24	23.18	10.0	8.36	0.002	Down	0.002	Down	0.002	Down	0.002	Down	0.003	Down
MW-16D	Deep	46.01	112.50	10.0	8.33										
OW-2	Shallow	45.62	22.96	10.0	7.82	0.001	Down	0.002	Down	0.002	Down	0.002	Down	0.002	Down
OW-7	Intermediate	45.61	68.77	10.0	7.88										
OW-7	Intermediate	45.61	68.77	10.0	7.88	0.004	Down	0.004	Down	0.004	Down	0.004	Down	0.006	Down
OW-5	Deep	45.53	112.84	10.0	7.99										
OW-2	Shallow	45.62	22.96	10.0	7.82	0.003	Down	0.003	Down	0.003	Down	0.003	Down	0.003	Down
OW-5	Deep	45.53	112.84	10.0	7.99										
OW-6	Intermediate	45.57	68.86	10.0	7.93	0.001	Down	0.002	Down	0.002	Down	0.002	Down	0.002	Down
MW-14D	Deep	45.72	113.84	10.0	8.16										

Notes:

BTOC = below top-of-casing

LNP COL 2.4-4

Table 2.4.12-210 (Sheet 1 of 3)
Slug Test Results Data Reduction

	Hydraulic Conductivity (ft/day)		
	Minimum	Maximum	Average
Shallow Monitoring/Observation Wells:	**0.9**	**28.6**	**9.2**
Intermediate Monitoring/Observation Wells:	**4.0**	**9.9**	**8.1**
Bedrock Monitoring/Observation Wells:	**2.4**	**54.4**	**13.9**

Well ID	Test Type	Fully or Partially Penetrating Well [(a)]	Well Screen Diameter (ft.)	Borehole Diameter (ft.)	Depth to Top of Screen (ft. BTOC)	Depth to Bottom of Screen (ft. BTOC)	Measured Total Depth[(b)] (ft. BTOC)	Depth to Static Water Level[(c)] (ft. BTOC)	Aquifer Thickness (ft.)	Is Water Level in the Well Screen?	Hydraulic Conductivity [(d,e)] (cm/sec)	Hydraulic Conductivity (ft/day)
MW-1S	In	*Partially*	0.17	0.50	23.40	33.40	33.65	4.59	*45.0*	No	8.4E-03	23.7
MW-1S	Out	*Partially*	0.17	0.50	23.40	33.40	33.65	4.59	*45.0*	No	4.4E-03	12.5
MW-2S	In	*Partially*	0.17	0.50	23.83	33.83	34.08	3.91	*45.0*	No	3.8E-03	10.9
MW-2S	Out	*Partially*	0.17	0.50	23.83	33.83	34.08	3.91	*45.0*	No	3.9E-03	10.9
MW-3S	In	*Partially*	0.17	0.50	23.49	33.49	33.74	5.73	*45.0*	No	8.7E-04	2.5
MW-3S	Out	*Partially*	0.17	0.50	23.49	33.49	33.74	5.73	*45.0*	No	6.9E-04	2.0
MW-4S	In	*Partially*	0.17	0.50	22.80	32.80	33.05	3.74	*45.0*	No	3.8E-03	10.7
MW-4S	Out	*Partially*	0.17	0.50	22.80	32.80	33.05	3.74	*45.0*	No	4.4E-03	12.4
MW-5S	In	*Partially*	0.17	0.50	23.50	33.50	33.75	3.78	*45.0*	No	3.7E-03	10.5
MW-5S	Out	*Partially*	0.17	0.50	23.50	33.50	33.75	3.78	*45.0*	No	3.8E-03	10.7
MW-6D	In	*Partially*	0.17	0.50	114.11	124.11	124.36	4.19	*250.0*	No	1.5E-03	4.1
MW-6D	Out	*Partially*	0.17	0.50	114.11	124.11	124.36	4.19	*250.0*	No	1.3E-03	3.7
MW-7S	In	*Partially*	0.17	0.50	23.02	33.02	33.27	4.37	*45.0*	No	8.8E-03	24.9
MW-7S	Out	*Partially*	0.17	0.50	23.02	33.02	33.27	4.37	*45.0*	No	1.0E-02	28.6
MW-8D	In	*Partially*	0.17	0.50	143.25	153.25	153.50	4.62	*250.0*	No	1.3E-03	3.8
MW-8D	Out	*Partially*	0.17	0.50	143.25	153.25	153.50	4.62	*250.0*	No	1.3E-03	3.7

LNP COL 2.4-4

Table 2.4.12-210 (Sheet 2 of 3)
Slug Test Results Data Reduction

	Hydraulic Conductivity (ft/day)		
	Minimum	Maximum	Average
Shallow Monitoring/Observation Wells:	0.9	28.6	9.2
Intermediate Monitoring/Observation Wells:	4.0	9.9	8.1
Bedrock Monitoring/Observation Wells:	2.4	54.4	13.9

Well ID	Test Type	Fully or Partially Penetrating Well [a]	Well Screen Diameter (ft.)	Borehole Diameter (ft.)	Depth to Top of Screen (ft. BTOC)	Depth to Bottom of Screen (ft. BTOC)	Measured Total Depth[b] (ft. BTOC)	Depth to Static Water Level[c] (ft. BTOC)	Aquifer Thickness (ft.)	Is Water Level in the Well Screen?	Hydraulic Conductivity [d,e] (cm/sec)	Hydraulic Conductivity (ft/day)
MW-9S	In	*Partially*	0.17	0.50	23.05	33.05	33.30	4.33	*45.0*	No	3.7E-04	1.0
MW-9S	Out	*Partially*	0.17	0.50	23.05	33.05	33.30	4.33	*45.0*	No	3.4E-04	0.9
MW-10D	In	*Partially*	0.17	0.50	113.67	123.67	123.92	4.28	*250.0*	No	4.1E-03	11.7
MW-10D	Out	*Partially*	0.17	0.50	113.67	123.67	123.92	4.28	*250.0*	No	3.0E-03	8.4
MW-11S	In	*Partially*	0.17	0.50	22.19	32.19	32.44	3.40	*45.0*	No	9.4E-04	2.7
MW-11S	Out	*Partially*	0.17	0.50	22.19	32.19	32.44	3.40	*45.0*	No	9.9E-04	2.8
MW-12D	In	*Partially*	0.17	0.50	113.39	123.39	123.64	3.81	*250.0*	No	3.2E-03	9.0
MW-12D	Out	*Partially*	0.17	0.50	113.39	123.39	123.64	3.81	*250.0*	No	2.7E-03	7.6
MW-13S	In	*Partially*	0.17	0.50	23.33	33.33	33.58	3.84	*45.0*	No	6.0E-04	1.7
MW-13S	Out	*Partially*	0.17	0.50	23.33	33.33	33.58	3.84	*45.0*	No	6.2E-04	1.8
MW-14D	In	*Partially*	0.17	0.50	113.84	123.84	124.09	3.89	*250.0*	No	8.7E-04	2.5
MW-14D	Out	*Partially*	0.17	0.50	113.84	123.84	124.09	3.89	*250.0*	No	8.3E-04	2.4
MW-15S	In	*Partially*	0.17	0.50	23.18	33.18	33.43	4.19	*45.0*	No	6.8E-04	1.9
MW-15S	Out	*Partially*	0.17	0.50	23.18	33.18	33.43	4.19	*45.0*	No	7.1E-04	2.0
MW-16D	In	*Partially*	0.17	0.50	112.50	122.50	122.75	4.28	*250.0*	No	1.9E-02	54.4
MW-16D	Out	*Partially*	0.17	0.50	112.50	122.50	122.75	4.28	*250.0*	No	1.7E-02	47.9
OW-1	In	*Partially*	0.17	0.50	23.31	33.31	33.56	3.93	*45.0*	No	2.1E-03	6.0
OW-1	Out	*Partially*	0.17	0.50	23.31	33.31	33.56	3.93	*45.0*	No	2.2E-03	6.3

LNP COL 2.4-4

Table 2.4.12-210 (Sheet 3 of 3)
Slug Test Results Data Reduction

	Hydraulic Conductivity (ft/day) Minimum	Maximum	Average
Shallow Monitoring/Observation Wells:	**0.9**	**28.6**	**9.2**
Intermediate Monitoring/Observation Wells:	**4.0**	**9.9**	**8.1**
Bedrock Monitoring/Observation Wells:	**2.4**	**54.4**	**13.9**

Well ID	Test Type	Fully or Partially Penetrating Well [a]	Well Screen Diameter (ft.)	Borehole Diameter (ft.)	Depth to Top of Screen (ft. BTOC)	Depth to Bottom of Screen (ft. BTOC)	Measured Total Depth[b] (ft. BTOC)	Depth to Static Water Level[c] (ft. BTOC)	Aquifer Thickness (ft.)	Is Water Level in the Well Screen?	Hydraulic Conductivity [d,e] (cm/sec)	Hydraulic Conductivity (ft/day)
OW-2	In	*Partially*	0.17	0.50	22.96	32.96	33.21	3.53	*45.0*	No	7.4E-03	20.8
OW-2	Out	*Partially*	0.17	0.50	22.96	32.96	33.21	3.53	*45.0*	No	7.5E-03	21.2
OW-3	In	*Partially*	0.17	0.50	22.97	32.97	33.22	3.36	*45.0*	No	1.7E-03	4.8
OW-3	Out	*Partially*	0.17	0.50	22.97	32.97	33.22	3.36	*45.0*	No	1.3E-03	3.7
OW-4	In	*Partially*	0.17	0.50	23.05	33.05	33.30	3.51	*45.0*	No	4.3E-03	12.1
OW-4	Out	*Partially*	0.17	0.50	23.05	33.05	33.30	3.51	*45.0*	No	3.0E-03	8.4
OW-5	In	*Partially*	0.17	0.50	112.84	122.84	123.09	3.78	*250.0*	No	6.7E-03	19.1
OW-5	Out	*Partially*	0.17	0.50	112.84	122.84	123.09	3.78	*250.0*	No	5.8E-03	16.4
OW-6	In	*Partially*	0.17	0.50	68.86	78.86	79.11	3.68	*250.0*	No	3.1E-03	8.8
OW-6	Out	*Partially*	0.17	0.50	68.86	78.86	79.11	3.68	*250.0*	No	3.5E-03	9.9
OW-7	In	*Partially*	0.17	0.50	68.77	78.77	79.02	3.63	*250.0*	No	3.5E-03	9.8
OW-7	Out	*Partially*	0.17	0.50	68.77	78.77	79.02	3.63	*250.0*	No	1.4E-03	4.0

Notes:

a) Fully penetrating means the entire saturated aquifer was screened.
b) Total well depth = length of casing + length of screen + 3-inch sump
c) Depth-to-groundwater measurements were collected on March 6, 2007.
d) Pressure heads were measured using a Level Troll 700, manufactured by In-Situ Inc.
e) AquiferWin32 software (developed by Environmental Simulations, Inc., Version 3, 1999) and the Bouwer & Rice, 1976 method were used.

BTOC = below top of casing, cm/sec = centimeter per second, ft/day = foot per day, ft. = foot

LNP COL 2.4-4

Table 2.4.12-211
Aquifer Test Results Data Reduction

Well ID	Fully or Partially Penetrating Well [(a)]	Well Screen Diameter (ft.)	Borehole Diameter (ft.)	Depth to Top of Screen (ft. BTOC)	Depth to Bottom of Screen (ft. BTOC)	Measured Total Depth[(b)] (ft. BTOC)	Depth to Static Water Level[(c)] (ft. BTOC)	Calculated Aquifer Thickness[(d)] (ft.)	Is Water Level in the Well Screen?	Transmissivity[(e,f)] (ft^2/d)	Storage Coefficient[(e,f)]	Beta (B)[(e,f)]	Specific Yield[(e,f)]
MW-13S	*Partially*	0.17	0.50	23.33	33.33	33.58	3.84	*29.5*	No	1.3E+03	1.6E-03	2.7E-03	1.7E-01
OW-1	*Partially*	0.17	0.50	23.31	33.31	33.56	3.93	*29.4*	No	2.1E+03	3.4E-04	1.7E-03	1.2E-02
OW-2	*Partially*	0.17	0.50	22.96	32.96	33.21	3.53	*29.4*	No	2.0E+03	7.1E-04	4.3E-03	2.7E-02
OW-3	*Partially*	0.17	0.50	22.97	32.97	33.22	3.36	*29.6*	No	2.2E+03	5.4E-04	2.1E-03	1.7E-02
OW-4	*Partially*	0.17	0.50	23.05	33.05	33.30	3.51	*29.5*	No	2.2E+03	5.3E-04	1.0E-03	1.6E-02

Notes:

a) Fully penetrating means the entire saturated aquifer was screened.

b) Total well depth = length of casing + length of screen + 3-inch sump.

c) Depth-to-groundwater measurements were collected on March 6, 2007.

d) Software uses the value of Aquifer Thickness = depth to bottom of screen - depth to static water level.

e) Pressure heads were measured using a Level Troll 700, manufactured by In-Situ Inc.

f) AquiferWin32 software (developed by Environmental Simulations, Inc., Version 3, 1999) and the Neuman, 1974 method were used.

ft. = foot
BTOC = Below top of casing
ft^2/d = square foot per day

LNP COL 2.4-4

Table 2.4.12-212 (Sheet 1 of 4)
Groundwater Linear Flow Velocity

March 6, 2007 [(a)] [(b)]

Monitoring Wells			Hydraulic Conductivity[(c)] [K] (feet/day)			Water Level Gauging Date	Water Level - Up Gradient Well (feet NAVD88)	Water Level - Down Gradient Well (feet NAVD88)	Water Level Change [dH] (feet)	Distance Between Wells [dL][(d)] (feet)	Hydraulic Gradient [dH/dL] (feet/feet)	Effective Porosity[(e)] [n_e]	Seepage Velocity [v_x] (feet/day)			Cross Sectional Area (ft.^{2})	Darcy Flux or Velocity (ft^3/d)		
			Minimum	Average	Maximum								Minimum	Average	Maximum		Minimum	Average	Maximum
Surficial Aquifer																			
MW-1S	to	MW-4S	0.9	9.2	28.6	6-Mar-07	45.09	40.50	4.59	6560	0.0007	0.2	0.003	0.03	0.1	1	0.001	0.006	0.02
MW-1S	to	MW-9S	0.9	9.2	28.6	6-Mar-07	41.75	40.50	1.25	4567	0.0003	0.2	0.001	0.01	0.04	1	0.0002	0.003	0.008
MW-2S	to	MW-3S	0.9	9.2	28.6	6-Mar-07	45.82	41.93	3.89	5350	0.0007	0.2	0.003	0.03	0.1	1	0.001	0.007	0.02
MW-5S	to	MW-7S	0.9	9.2	28.6	6-Mar-07	42.54	41.74	0.80	1719	0.0005	0.2	0.002	0.02	0.07	1	0.0004	0.004	0.01
MW-11S	to	MW-15S	0.9	9.2	28.6	6-Mar-07	42.05	41.30	0.75	1495	0.0005	0.2	0.002	0.02	0.07	1	0.0005	0.005	0.01
Bedrock Aquifer																			
MW-6D	to	MW-8D	2.4	13.9	54.4	6-Mar-07	42.21	41.40	0.81	1723	0.0005	0.15	0.01	0.04	0.2	1	0.001	0.01	0.03
MW-12D	to	MW-8D	2.4	13.9	54.4	6-Mar-07	42.21	40.73	1.48	2728	0.0005	0.15	0.01	0.05	0.2	1	0.001	0.01	0.03
MW-12D	to	MW-16D	2.4	13.9	54.4	6-Mar-07	41.73	40.73	1.00	1497	0.0007	0.15	0.01	0.06	0.2	1	0.002	0.01	0.04

LNP COL 2.4-4

Table 2.4.12-212 (Sheet 2 of 4)
Groundwater Linear Flow Velocity

June 14, 2007 [(a) (b)]

Monitoring Wells			Hydraulic Conductivity[(c)] [K] (feet/day)			Water Level Gauging Date	Water Level - Up Gradient Well (feet NAVD88)	Water Level - Down Gradient Well (feet NAVD88)	Water Level Change [dH] (feet)	Distance Between Wells [dL][(d)] (feet)	Hydraulic Gradient [dH/dL] (feet/feet)	Effective Porosity[(e)] [n_e]	Seepage Velocity [v_x] (feet/day)			Cross Sectional Area (ft.^{2})	Darcy Flux or Velocity (ft^3/d)		
			Minimum	Average	Maximum								Minimum	Average	Maximum		Minimum	Average	Maximum
Surficial Aquifer[(f)]																			
MW-1S	to	MW-4S	0.9	9.2	28.6	14-Jun-07	41.78	37.40	4.38	6560	0.0007	0.2	0.003	0.03	0.10	1	0.001	0.006	0.02
MW-1S	to	MW-9S	0.9	9.2	28.6	14-Jun-07	39.22	37.40	1.82	4567	0.0004	0.2	0.002	0.02	0.06	1	0.0004	0.004	0.011
MW-2S	to	MW-3S	0.9	9.2	28.6	14-Jun-07	41.93	37.98	3.95	5350	0.0007	0.2	0.003	0.03	0.11	1	0.001	0.007	0.02
MW-5S	to	MW-7S	0.9	9.2	28.6	14-Jun-07	39.30	39.14	0.16	1719	0.0001	0.2	0.000	0.00	0.01	1	0.0001	0.001	0.00
Bedrock Aquifer																			
MW-6D	to	MW-8D	2.4	13.9	54.4	14-Jun-07	39.28	38.59	0.69	1723	0.0004	0.15	0.01	0.04	0.15	1	0.001	0.01	0.02
MW-12D	to	MW-8D	2.4	13.9	54.4	14-Jun-07	39.28	37.83	1.45	2728	0.0005	0.15	0.01	0.05	0.19	1	0.001	0.01	0.03
MW-12D	to	MW-16D	2.4	13.9	54.4	14-Jun-07	38.93	37.83	1.10	1497	0.0007	0.15	0.01	0.07	0.27	1	0.002	0.01	0.04

LNP COL 2.4-4

Table 2.4.12-212 (Sheet 3 of 4)
Groundwater Linear Flow Velocity

September 13, 2007 (a) (b)

Monitoring Wells			Hydraulic Conductivity(c) [K] (feet/day) Minimum	Average	Maximum	Water Level Gauging Date	Water Level - Up Gradient Well (feet NAVD88)	Water Level - Down Gradient Well (feet NAVD88)	Water Level Change [dH] (feet)	Distance Between Wells [dL](d) (feet)	Hydraulic Gradient [dH/dL] (feet/feet)	Effective Porosity(e) [n_e]	Seepage Velocity [v_x] (feet/day) Minimum	Average	Maximum	Cross Sectional Area (ft.^{2})	Darcy Flux or Velocity (ft^3/d) Minimum	Average	Maximum
Surficial Aquifer																			
MW-1S	to	MW-4S	0.9	9.2	28.6	13-Sep-07	40.77	36.21	4.56	6560	0.0007	0.2	0.003	0.03	0.10	1	0.001	0.006	0.02
MW-1S	to	MW-9S	0.9	9.2	28.6	13-Sep-07	37.94	36.21	1.73	4567	0.0004	0.2	0.002	0.02	0.05	1	0.0003	0.003	0.011
MW-2S	to	MW-3S	0.9	9.2	28.6	13-Sep-07	41.12	36.87	4.25	5350	0.0008	0.2	0.004	0.04	0.11	1	0.001	0.007	0.02
MW-5S	to	MW-7S	0.9	9.2	28.6	13-Sep-07	38.03	37.68	0.35	1719	0.0002	0.2	0.001	0.01	0.03	1	0.0002	0.002	0.01
MW-11S	to	MW-15S	0.9	9.2	28.6	13-Sep-07	38.01	37.64	0.37	1495	0.0002	0.2	0.001	0.01	0.04	1	0.0002	0.002	0.01
Bedrock Aquifer																			
MW-6D	to	MW-8D	2.4	13.9	54.4	13-Sep-07	37.95	37.18	0.77	1723	0.0004	0.15	0.01	0.04	0.16	1	0.001	0.01	0.02
MW-12D	to	MW-8D	2.4	13.9	54.4	13-Sep-07	37.95	36.49	1.46	2728	0.0005	0.15	0.01	0.05	0.19	1	0.001	0.01	0.03
MW-12D	to	MW-16D	2.4	13.9	54.4	13-Sep-07	37.46	36.49	0.97	1497	0.0006	0.15	0.01	0.06	0.23	1	0.002	0.01	0.04

LNP COL 2.4-4

Table 2.4.12-212 (Sheet 4 of 4)
Groundwater Linear Flow Velocity

December 4, 2007 (a) (b)

Monitoring Wells			Hydraulic Conductivity(c) [K] (feet/day)			Water Level Gauging Date	Water Level - Up Gradient Well (feet NAVD88)	Water Level - Down Gradient Well (feet NAVD88)	Water Level Change [dH] (feet)	Distance Between Wells [dL](d) (feet)	Hydraulic Gradient [dH/dL] (feet/feet)	Effective Porosity(e) [n_e]	Seepage Velocity [v_x] (feet/day)			Cross Sectional Area (ft.^{2})	Darcy Flux or Velocity (ft^3/d)		
			Minimum	Average	Maximum								Minimum	Average	Maximum		Minimum	Average	Maximum
Surficial Aquifer																			
MW-1S	to	MW-4S	0.9	9.2	28.6	4-Dec-07	40.93	36.31	4.62	6560	0.0007	0.2	0.003	0.03	0.10	1	0.001	0.006	0.02
MW-1S	to	MW-9S	0.9	9.2	28.6	4-Dec-07	38.05	36.31	1.74	4567	0.0004	0.2	0.002	0.02	0.05	1	0.0003	0.004	0.011
MW-2S	to	MW-3S	0.9	9.2	28.6	4-Dec-07	40.66	36.21	4.45	5350	0.0008	0.2	0.004	0.04	0.12	1	0.001	0.008	0.02
MW-5S	to	MW-7S	0.9	9.2	28.6	4-Dec-07	37.99	37.59	0.40	1719	0.0002	0.2	0.001	0.01	0.03	1	0.0002	0.002	0.01
MW-11S	to	MW-15S	0.9	9.2	28.6	4-Dec-07	37.88	37.66	0.22	1495	0.0001	0.2	0.001	0.01	0.02	1	0.0001	0.001	0.00
Bedrock Aquifer																			
MW-6D	to	MW-8D	2.4	13.9	54.4	4-Dec-07	37.97	37.27	0.70	1723	0.0004	0.15	0.01	0.04	0.15	1	0.001	0.01	0.02
MW-12D	to	MW-8D	2.4	13.9	54.4	4-Dec-07	37.97	36.58	1.39	2728	0.0005	0.15	0.01	0.05	0.18	1	0.001	0.01	0.03
MW-12D	to	MW-16D	2.4	13.9	54.4	4-Dec-07	37.68	36.58	1.10	1497	0.0007	0.15	0.01	0.07	0.27	1	0.002	0.01	0.04

Notes:

a) Seepage Velocity [vx] = (Hydraulic Conductivity [K] * Hydraulic Gradient [dH/dL])/Effective Porosity [ne]). Reference 2.4.12-218.

b) Darcy Flux [Q] = Hydraulic Conductivity [K] * Hydraulic Gradient [dH/dL] * Cross-sectional Area [A]. Reference 2.4.12-219.

c) Hydraulic conductivity estimates are maximum values derived from Table 2.4.12-210, Slug Test Results Data Reduction.

d) Well distances were derived from well survey conducted from March 21, 2007 through March 25, 2007.

e) Effective porosity estimates from References 2.4.12-220, 2.4.12-217, 2.4.12-221, and 2.4.12-222.

f) Due to the difference in water levels between MW-11S and MW-15S, MW-11S was not considered to be down gradient of MW-15S. Therefore these wells were not used in the calculation for June 14, 2007.

ft.^{2} = square foot
ft^3/d = cubic foot per day

LNP COL 2.4-4

Table 2.4.12-213
Aquifer Properties Computed from MLU Analysis

MLU Layer	Layer Thickness (feet)	Horizontal Conductivity (Kh) (feet/day)	Vertical Conductivity (Kv) (feet/day)[1]	Transmissivity ($feet^2$/day)	Leakance (1/day) [(a)]	Storativity	Aquifer
Unit 1 Aquifer Test Results							
1	35	13	8	450	0.20	4E-08	Surficial
2	45	13	10	580	0.29	4E-08	Surficial
3	25	120	120	3000	4.8	2E-08	Upper Floridan
4	25	120	120	3000	4.8	2E-08	Upper Floridan
5	25	120	120	3000	0.51	2E-08	Upper Floridan
6	445	120	--	53,000	--	2E-08	Upper Floridan
Unit 2 Aquifer Test Results							
1	35	13	8	450	0.20	5E-08	Surficial
2	45	13	10	580	0.27	5E-08	Surficial
3	30	130	130	4000	4.4	4E-10	Upper Floridan (top interval)
4	30	130	130	4000	4.4	4E-10	Upper Floridan (middle interval)
5	30	130	130	4000	4.4	4E-10	Upper Floridan (deep interval)
6	30	130	130	4000	0.62	4E-10	Upper Floridan (PW-2 deep interval)
7	400	130	--	53,000	--	5E-09	Upper Floridan (below screened interval)

a) Vertical conductivity and leakance values apply to the interface between layers (i.e., the leakance value given for Layer 1 applies to the interface between Layer 1 and Layer 2).

LNP COL 2.4-5

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 CMTs)	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 aux. 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 Elevation 66 ft. – 6 inches	28,000 gallon	Essentially reactor coolant	Located in unlined room at lowest portion of the auxiliary building.
WLS Waste Holdup Tanks	Auxiliary Building Elevation 66 ft. – 6 inches	15,000 gallon	Less than reactor coolant	Located in unlined room at lowest portion of auxiliary building.
WLS Monitor Tanks A, B, C	Auxiliary Building Elevation 66 ft. – 6 inches and 117 ft. – 6 inches	15,000 gallon	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 gallon	Effluent prepared for environmental discharge – much less than reactor coolant	Located in unlined room at grade level in curbed, nonseismic building.

LNP COL 2.4-5

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 Chemical Waste Tank	Auxiliary Building Elevation 66 ft. – 6 inches	8,900 gallon	Less than reactor coolant	Located in unlined room at lowest portion of auxiliary building.
WSS Spent Resin Storage Tanks	Auxiliary Building Elevation 100 ft.	300 ft^3 (liquid volume will be much less)	Approx. reactor coolant	Located entirely inside auxiliary building; does not have any potential for foundation cracks to allow leakage directly to environment. Leakage would be to another room of auxiliary building.

Notes:

a) Floor elevations are based on design plant grade of 100 ft as provided in the DCD.

PXS = passive core cooling system
IRWST = in-containment water storage tank
CMT = core makeup tank
WLS = liquid radwaste system
WSS = solid radwaste system
NA = not applicable due to the rationale discussed under Considerations/Features to Mitigate Release
ft. = feet
ft^3 = cubic foot

LNP COL 2.4-5

Table 2.4.13-202 (Sheet 1 of 3)
Effluent Tank Inventory and 10 CFR 20 Effective Concentration Limits

Nuclide	RC Activity, µCi/g	Holdup Tank Activity, µCi/cm^3	Half Life, Days	Decay Constant, Days^{-1}	ECL, µCi/cm^3
H-3	1.0E+00	1.0E+00	4.51E+03	1.54E-04	1.0E-03
Br-83	3.2E-02	1.6E-02	9.96E-02	6.96E+00	9.0E-04
Br-84	1.7E-02	8.2E-03	2.21E-02	3.14E+01	4.0E-04
Br-85	2.0E-03	9.7E-04	2.01E-03	3.45E+02	1.0E+00
I-129	1.5E-08	7.3E-09	5.73E+09	1.21E-10	2.0E-07
I-130	1.1E-02	5.3E-03	5.15E-01	1.35E+00	2.0E-05
I-131	7.1E-01	3.4E-01	8.04E+00	8.62E-02	1.0E-06
I-132	9.4E-01	4.6E-01	9.58E-02	7.24E+00	1.0E-04
I-133	1.3E+00	6.3E-01	8.67E-01	7.99E-01	7.0E-06
I-134	2.2E-01	1.1E-01	3.65E-02	1.90E+01	4.0E-04
I-135	7.8E-01	3.8E-01	2.75E-01	2.52E+00	3.0E-05
Cs-134	6.9E-01	3.3E-01	7.53E+02	9.21E-04	9.0E-07
Cs-136	1.0E+00	4.8E-01	1.31E+01	5.29E-02	6.0E-06
Cs-137	5.0E-01	2.4E-01	1.10E+04	6.301E-05	1.0E-06
Cs-138	3.7E-01	1.8E-01	2.24E-02	3.09E+01	4.0E-04
Cr-51	1.3E-03	1.3E-03	2.77E+01	2.50E-02	5.0E-04
Mn-54	6.7E-04	6.8E-04	3.13E+02	2.21E-03	3.0E-05
Mn-56	1.7E-01	1.7E-01	1.07E-01	6.48E+00	7.0E-05
Fe-55	5.0E-04	5.1E-04	9.86E+02	7.03E-04	1.0E-04
Fe-59	1.3E-04	1.3E-04	4.45E+01	1.56E-02	1.0E-05
Co-58	1.9E-03	1.9E-03	7.08E+01	9.79E-03	2.0E-05
Co-60	2.2E-04	2.2E-04	1.93E+03	3.59E-04	3.0E-06
Rb-88	1.5E+00	7.3E-01	1.24E-02	5.59E+01	4.0E-04
Rb-89	6.9E-02	3.3E-02	1.06E-02	6.54E+01	9.0E-04
Sr-89	1.1E-03	5.3E-04	5.05E+01	1.37E-02	8.0E-06
Sr-90	4.9E-05	2.4E-05	1.06E+04	6.54E-05	5.0E-07
Sr-91	1.7E-03	8.2E-04	3.96E-01	1.75E+00	2.0E-05

LNP COL 2.4-5

Table 2.4.13-202 (Sheet 2 of 3)
Effluent Tank Inventory and 10 CFR 20 Effective Concentration Limits

Nuclide	RC Activity, μCi/g	Holdup Tank Activity, μCi/cm^3	Half Life, Days	Decay Constant, Days^{-1}	ECL, μCi/cm^3
Sr-92	4.1E-04	2.0E-04	1.13E-01	6.13E+00	4.0E-05
Y-90	1.3E-05	6.3E-06	2.67E+00	2.60E-01	7.0E-06
Y-91m	9.2E-04	4.5E-04	3.45E-02	2.01E+01	2.0E-03
Y-91	1.4E-04	6.8E-05	5.85E+01	1.18E-02	8.0E-06
Y-92	3.4E-04	1.6E-04	1.48E-01	4.68E+00	4.0E-05
Y-93	1.1E-04	5.3E-05	4.21E-01	1.65E+00	2.0E-05
Zr-95	1.6E-04	7.8E-05	6.40E+01	1.08E-02	2.0E-05
Nb-95	1.6E-04	7.8E-05	3.52E+01	1.97E-02	3.0E-05
Mo-99	2.1E-01	1.0E-01	2.75E+00	2.52E-01	2.0E-05
Tc-99m	2.0E-01	9.7E-02	2.51E-01	2.76E+00	1.0E-03
Ru-103	1.4E-04	6.8E-05	3.93E+01	1.76E-02	3.0E-05
Rh-103m	1.4E-04	6.8E-05	3.90E-02	1.78E+01	6.0E-03
Rh-106	4.5E-05	2.2E-05	4.63E-04	1.50E+03	NA
Ag-110m	4.0E-04	1.9E-04	2.50E+02	2.77E-03	6.0E-06
Te-127m	7.6E-04	3.7E-04	1.09E+02	6.36E-03	9.0E-06
Te-129m	2.6E-03	1.3E-03	3.36E+01	2.06E-02	7.0E-06
Te-129	3.8E-03	1.8E-03	4.83E-02	1.44E+01	4.0E-04
Te-131m	6.7E-03	3.2E-03	1.25E+00	5.55E-01	8.0E-06
Te-131	4.3E-03	2.1E-03	1.74E-02	3.98E+01	8.0E-05
Te-132	7.9E-02	3.8E-02	3.26E+00	2.13E-01	9.0E-06
Te-134	1.1E-02	5.3E-03	2.90E-02	2.39E+01	3.0E-04
Ba-137m	4.7E-01	2.3E-01	1.81E-03	3.83E+02	NA
Ba-140	1.0E-03	4.8E-04	1.27E+01	5.46E-02	8.0E-06
La-140	3.1E-04	1.5E-04	1.68E+00	4.13E-01	9.0E-06
Ce-141	1.6E-04	7.8E-05	3.25E+01	2.13E-02	3.0E-05
Ce-143	1.4E-04	6.8E-05	1.38E+00	5.02E-01	2.0E-05

LNP COL 2.4-5

Table 2.4.13-202 (Sheet 3 of 3)
Effluent Tank Inventory and 10 CFR 20 Effective Concentration Limits

Nuclide	RC Activity, μCi/g	Holdup Tank Activity, μCi/cm^3	Half Life, Days	Decay Constant, Days^{-1}	ECL, μCi/cm^3
Pr-143	1.5E-04	7.3E-05	1.36E+01	5.10E-02	2.0E-05
Ce-144	1.2E-04	5.8E-05	2.84E+02	2.44E-03	3.0E-06
Pr-144	1.2E-04	5.8E-05	1.20E-02	5.78E+01	6.0E-04

Notes:

Effluent holdup tank activities from Westinghouse.
ECLs from 10CFR 20, Appendix B, Table 2, Column 2.
Equilibrium daughters Ba-137m and Rh-106 are included in ECLS for Cs-137 and Ru-106.

ECL = effective concentration limit
μCi/g = microCurie per gram
μCi/cm^3 = microCurie per cubic centimeter
days^{-1} = 1 per day

LNP COL 2.4-5

Table 2.4.13-203
Parameters Used in Groundwater Model Transport Analysis

Parameter	Surficial Aquifer	Upper Floridan Aquifer
Hydraulic conductivity, K [(a)]	29 ft/day	54 ft/day
Effective Porosity, n_e [(b)]	0.2	0.15
Head gradient, dh/dl [(b)]	0.0008 ft/ft	0.0007 ft/ft
Seepage velocity, U [(c)]	0.04 m/day	0.08 m/day
Bulk density [(d)]	1.4 g/cm^3	2.4 g/cm^3
Dispersivity [(e)]	1 m	1 m
Cesium distribution coefficient, K_d [(f)]	6 ml/g	4 ml/g
Strontium and other nuclide distribution coefficient, K_d [(f)]	0 ml/g	2 ml/g
Transport distance to nearest well [(h)]	NA [(g)]	2 km [(g)]
Cesium transport time to nearest well [(i)]	NA	4500 yr
Strontium transport time to nearest well [(i)]	NA	2300 yr
Other nuclides transport times to nearest well [(i)]	NA	68 yr
Transport distance to Lower Withlacoochee River [(h)]	7 km	7 km
Cesium transport time to Lower Withlacoochee River [(i)]	20,600 yr	15,600 yr
Strontium transport time to Lower Withlacoochee River [(i)]	479 yr	7900 yr
Other nuclides transport times to Lower Withlacoochee River [(i)]	479 yr	240 yr
Aquifer depth [(h) (j)]	NA [(g)]	76 m [(g)]

Notes:
a) Hydraulic conductivities are maximum values from FSAR Table 2.4.12-210.
b) Effective porosity and head gradient maximum values are from FSAR Table 2.4.12-212.
c) Seepage or pore velocity is calculated for this table using FSAR Equation 2.4.13-3.
d) Bulk densities are from FSAR Table 2.4.12-201 and Reference 2.4.13-203, Appendix B.
e) Dispersivity is from Reference 2.4.13-203 and FSAR Subsection 2.4.13.2.2.
f) K_d values are from FSAR Subsection 2.4.13.2.2.
g) Wells take their supply from the Upper Floridan aquifer.
h) Transport distances are down-gradient, southwest of LNP 1 & 2. See FSAR Subsection 2.4.13.2.
i) Transport times shown are based on transport distance X, seepage velocity U, and retardation R_d. Transport time = X R_d/U where R_d is given by FSAR Equation 2.4.13-2.
j) The analytical aquifer depth is conservatively taken as the well depth from FSAR Subsection 2.4.13.2.4.

ft/day = foot per day
ft/ft = foot per foot
g/cm^3 = gram per cubic centimeter
yr = year
km = kilometer
m = meter
m/day = meter per day
ml/g = milliliter per gram
NA = not applicable/not available

LNP COL 2.4-5

Table 2.4.13-204 (Sheet 1 of 2)
Radionuclide Concentrations in the Lower Withlacoochee River with Comparisons to 10 CFR 20 Effective Concentration Limits

Nuclide	Dilution Factor	Conc. µCi/cm^3	Conc./ ECL	Nuclide	Dilution Factor	Conc. µCi/cm^3	Conc./ ECL
H-3	1.7E-14	1.8E-14	1.8E-11	Y-90	0	0	0
Br-83	0	0	0	Y-91m	0	0	0
Br-84	0	0	0	Y-91	0	0	0
Br-85	0	0	0	Y-92	0	0	0
I-129	1.2E-08	8.5E-17	4.4E-10	Y-93	0	0	0
I-130	0	0	0	Zr-95	0	0	0
I-131	0	0	0	Nb-95	0	0	0
I-132	0	0	0	Mo-99	0	0	0
I-133	0	0	0	Tc-99m	0	0	0
I-134	0	0	0	Ru-103	0	0	0
I-135	0	0	0	Rh-103m	0	0	0
Cs-134	0	0	0	Rh-106	0	0	0
Cs-136	0	0	0	Ag-110m	0	0	0
Cs-137	0	0	0	Te-127m	0	0	0
Cs-138	0	0	0	Te-129m	0	0	0
Cr-51	0	0	0	Te-129	0	0	0
Mn-54	8.5E-93	5.7E-96	1.9E-91	Te-131m	0	0	0
Mn-56	0	0	0	Te-131	0	0	0
Fe-55	2.3E-35	1.2E-38	1.2E-34	Te-132	0	0	0
Fe-59	0	0	0	Te-134	0	0	0
Co-58	0	0	0	Ba-137m	0	0	0
Co-60	2.7E-22	6.0E-26	2.0E-20	Ba-140	0	0	0
Rb-88	0	0	0	La-140	0	0	0
Rb-89	0	0	0	Ce-141	0	0	0
Sr-89	0	0	0	Ce-143	0	0	0
Sr-90	3.6E-92	8.6E-97	1.7E-90	Pr-143	0	0	0

LNP COL 2.4-5

**Table 2.4.13-204 (Sheet 2 of 2)
Radionuclide Concentrations in the Lower Withlacoochee River with Comparisons to 10 CFR 20 Effective Concentration Limits**

Nuclide	Dilution Factor	Conc. μCi/cm^3	Conc./ ECL	Nuclide	Dilution Factor	Conc. μCi/cm^3	Conc./ ECL
Sr-91	0	0	0	Ce-144	0	0	0
Sr-92	0	0	0	Pr-144	0	0	0
∑(Max conc./ ECL) at Lower Withlacoochee River ~ 0.0%							

Notes:

Radionuclides with concentrations less than 1E-99 μCi/cm^3 are assigned a concentration of zero.
Conc. = concentration
μCi/cm^3 = microCurie per cubic centimeter
ECL = effective concentration limit

LNP COL 2.4-5

Table 2.4.13-205
Groundwater Radionuclide Concentrations at Nearest Floridan Aquifer Well with Comparisons to 10 CFR 20 Effective Concentration Limits

Nuclide	Dilution Factor	Conc./ µCi/cm^3	Conc./ ECL	Nuclide	Dilution Factor	Conc./ µCi/cm^3	Conc./ ECL
H-3	6.4E-06	6.4E-06	6.4E-03	Y-90	0	0	0
Br-83	0	0	0	Y-91m	0	0	0
Br-84	0	0	0	Y-91	0	0	0
Br-85	0	0	0	Y-92	0	0	0
I-129	3.0E-04	2.2E-12	1.1E-05	Y-93	0	0	0
I-130	0	0	0	Zr-95	0	0	0
I-131	0	0	0	Nb-95	0	0	0
I-132	0	0	0	Mo-99	0	0	0
I-133	0	0	0	Tc-99m	0	0	0
I-134	0	0	0	Ru-103	0	0	0
I-135	0	0	0	Rh-103m	0	0	0
Cs-134	0	0	0	Rh-106	0	0	0
Cs-136	0	0	0	Ag-110m	2.3E-34	4.5E-38	7.6E-33
Cs-137	1.5E-50	3.7E-51	3.7E-45	Te-127m	2.7E-73	9.9E-77	1.1E-71
Cs-138	0	0	0	Te-129m	0	0	0
Cr-51	0	0	0	Te-129	0	0	0
Mn-54	2.7E-28	1.8E-31	6.0E-27	Te-131m	0	0	0
Mn-56	0	0	0	Te-131	0	0	0
Fe-55	6.9E-12	3.5E-15	3.5E-11	Te-132	0	0	0
Fe-59	0	0	0	Te-134	0	0	0
Co-58	0	0	0	Ba-137m	0	0	0
Co-60	3.7E-08	8.3E-12	2.8E-06	Ba-140	0	0	0
Rb-88	0	0	0	La-140	0	0	0
Rb-89	0	0	0	Ce-141	0	0	0
Sr-89	0	0	0	Ce-143	0	0	0
Sr-90	3.3E-29	7.9E-34	1.6E-27	Pr-143	0	0	0
Sr-91	0	0	0	Ce-144	9.4E-31	5.5E-35	1.8E-29
Sr-92	0	0	0	Pr-144	0	0	0
∑(Max conc./ ECL) at well < 0.7%							

Notes:
Radionuclides with concentrations less than 1E-99 µCi/cm^3 are assigned a concentration zero.
Conc. = concentration
ECL = effective concentration limit
µCi/cm^3 = microCurie per cubic centimeter

LNP COL 2.4-5

Table 2.4.13-206
K_d Values from Testing of LNP Samples

Borehole/ Sample	Description	Depth (ft bgs)	K_d for Strontium ml/g	K_d for Cesium ml/g
A-1 SS-5	Soil (calcareous)	23.5	4	43
A-5 SS-3	Soil (calcareous)	13.5	2	20
A-13 SS-6	Soil (calcareous)	25	4	44
A-14 SS-6	Soil (calcareous)	28.5	3	32
B-1 SS-11	Soil (calcareous)	50	0.1	14
B-1 SS-11 QC	Soil (calcareous)	50	0.1	13
GSC-10 SS-4	Soil (calcareous)	16.5	2	6
GSC-10 SS-5 QC	Soil (calcareous)	21.5	3	6
A-1	Limestone	89 - 90	7	30
A-1 QC	Limestone	90 - 90.9	7	27
A-5	Limestone	73.8 - 74.8	59	4
A-13	Limestone	74.1 - 75	2	29
A-16	Limestone	69 – 70.1	2	22
B-1	Limestone	75.9 – 76.8	4	8
GSC-10	Limestone	91.8 – 92.6	4	37
GSC-10 QC	Limestone	92.6 – 93.5	3	42

Notes:

K_d testing results are from Reference 2.4.13-204.
Groundwater from wells MW-14D and MW-15S was used to prepare limestone and soil batch runs, respectively.

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

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

STD DEP 1.1-1

This section is numbered to follow Regulatory Guide 1.206. The COL information items in DCD Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4, and 2.5.5 are addressed in FSAR Subsection 2.5.6.

LNP SUP 2.5-1

This section of the Final Safety Analysis Report (FSAR) presents information on the geology, seismology, and geotechnical engineering characteristics of the region, vicinity, and area of the Levy Nuclear Plant Units 1 and 2 (LNP) site. This section was developed in accordance with requirements outlined in Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)." Additional regulatory and technical guidance considered during preparation of FSAR Section 2.5 are discussed within each subsection.

FSAR Subsection 2.5.0 provides a summary of information presented in detail in FSAR Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4, and 2.5.5. Combined License Information items are summarized in FSAR Subsection 2.5.6, and references are in FSAR Subsection 2.5.7.

The vertical datum used for the COLA subsurface investigation and for the LNP construction site is the North American Vertical Datum 1988 (NAVD88). The vertical datum for references cited in this FSAR is per the cited reference, which include, above mean sea level (amsl), mean sea level (msl), NAVD88, or National Geodetic Vertical Datum 1929 (NGVD29).

2.5.0 SUMMARY

2.5.0.1 Basic Geologic and Seismic Information

2.5.0.1.1 Regional Geology

The LNP site is located on the west coast of the Floridian plateau or platform, a recently emergent part of the south-central North American Plate that separates the Gulf of Mexico from the Atlantic Ocean. Basement rocks underlying the Florida platform include Precambrian – Cambrian igneous rocks, Ordovician – Devonian sedimentary rocks, and Triassic – Jurassic volcanic rocks. Paleozoic basement rocks had a Gondwana origin, and were joined to the North American Plate in the final stages of development of the Appalachian Mountains in the Late Carboniferous to Early Permian. The igneous and sedimentary basement rocks originated from the African Plate but remained attached to the North American Plate when rifting that resulted in opening of the present Atlantic Ocean occurred in the Middle Triassic to Early Jurassic.

A thick sequence of mid-Jurassic to Holocene sediments unconformably overlies the basement rocks. From Middle Jurassic to Middle Oligocene, carbonate sedimentation was widespread along the Florida platform. This depositional regime changed in response to sea-level fluctuations and uplift and erosion in the Appalachian highlands. Starting in the mid-Oligocene and continuing into the Holocene, deposition of siliciclastic-bearing carbonates and siliciclastic sediments dominated the Florida platform. This thick sequence of unconsolidated to semiconsolidated sedimentary rocks comprises the Coastal Plain physiographic province.

The south-central United States is a passive continental margin with no relative differential motion (i.e., angular velocity) between the Gulf of Mexico and the North American continental plate. The LNP site lies within a stable continental region that is characterized by low earthquake activity and low stress. The site lies within a compressive midplate stress province characterized by reverse and strike-slip faulting. Reverse focal mechanisms for earthquakes that appear to have originated in the basement in the abyssal plain region of the Gulf of Mexico west of the site region are consistent with an east-northeastward-directed compressive stress environment.

The LNP site is located near the northeastern margin of the Gulf of Mexico basin (also referred to as the Gulf Coast basin or Gulf basin) that includes the present Gulf of Mexico and adjacent rift basins. Tectonic features in the site region reflect the cumulative deformation of tectonic events throughout the Late Proterozoic to Early Paleozoic, Paleozoic, Mesozoic, and Cenozoic eras. Cenozoic faults have been postulated by numerous authors in various parts of the study region, including the site area, based on apparent displacements inferred from limited outcrops and subsurface data from widely spaced boreholes and wells. The existence of many of these structures is controversial and not well supported by available data. None of these structures is judged to be a capable tectonic source.

The Electric Power Research Institute and Seismic Owners Group (EPRI-SOG) seismic hazard analysis for the nearby Crystal River Unit No. 3 Nuclear Generating Plant (CR3) site identified the Charleston seismic zone, the source of a large, geologically recent earthquake, as a significant seismic source at a distance of approximately 500 kilometers (km) (300 miles [mi.]). Updated information regarding the location, magnitude, and recurrence of this more distant, but significant, seismic source was incorporated into the updated seismic hazard analysis for the LNP site.

2.5.0.1.2 Site Geology

The LNP site, located within southern Levy County, lies approximately 16 km (10 mi.) west of the Gulf of Mexico and approximately 12.8 km (8 mi.) north of the Withlacoochee River. The site area, located within the Gulf Coastal Lowlands geomorphic province, is characterized by both depositional and erosional features. Broad plains underlain by a series of late Tertiary and Quaternary surfaces and shorelines are pitted with karstic depressions within the limestone at or near the present land surface in the site area. The LNP site is located within the Limestone Shelf and Hammocks subzone, a zone that is characterized as a karstic, erosional limestone plain overlain by sand dunes, ridges, and coast-parallel paleoshore sand belts associated with the Pleistocene-age marine terraces.

The oldest rocks penetrated within the site area are Paleozoic shales and quartzite pebble sands that are intruded by Triassic diabase. Overlying these sediments is a thick section of Cretaceous and Cenozoic carbonates (limestone and dolomite) that are overlain by undifferentiated Pleistocene- to Holocene-age surficial sands, clayey sands, and alluvium. Stratigraphy of the LNP site location is known from the Robinson No. 1 well located approximately 500 meters (m) (1640 feet [ft.]) north of the LNP site and from over 118 geotechnical borings that were drilled as part of the Combined License Application (COLA) study.

Hydrostratigraphic units of the Floridan aquifer system carbonate depositional sequence in west-central Florida include an Upper Floridan aquifer, which typically contains fresh potable water, and a Lower Floridan aquifer. The Upper Floridan aquifer commonly is separated physically and hydraulically from the underlying Lower Floridan aquifer by sequences of lower permeability evaporite rock units known as the Middle Confining Unit (MCU), which act as an aquitard. The geotechnical boring program at the LNP site results showed that the first carbonate rock units encountered below the surficial aquifer deposits are deposits of the middle Eocene age Avon Park Formation. To the maximum investigated depth of 152 m (500 ft.), neither the MCU nor the Lower Floridan aquifer units were encountered.

The Quaternary deposits (designated unit S1) encountered in the LNP site borings generally consist of gray silty sands. The subrounded to rounded sand grains and sorting indicate that the sands likely were deposited in a nearshore beach or dune environment, possibly during the transgression and regression of the high sea level stand that formed the underlying marine terrace platform, which is interpreted to be middle to early Pleistocene in age (>340,000 years). There may be a component of younger eolian sand deposited during subsequent sea level fluctuations and locally derived fluvial deposits. In some boreholes, thicker section of the S1 deposits consist of gray sand intermixed or interbedded with medium brown sand and grayish black clay and sandy clay layers. These deposits are interpreted to represent infills of sand and marsh deposits into paleosinks. Some of the infill material in the deeper paleosinks may be Tertiary as well as Quaternary in age.

The Avon Park Formation is a carbonate mud-dominated peritidal sequence, pervasively dolomitized in places and not dolomitized in others, and contains some intergranular and interbedded evaporites in its lower part. Fossils are mostly benthic forms showing limited faunal diversity. Seagrass beds are well preserved at certain horizons. The lower portion of the Avon Park Formation consists of lower permeability evaporite deposits, which act as an aquitard separating the Upper Floridan aquifer within the Avon Park Formation from the Lower Floridan aquifer within the Oldsmar Limestone.

The LNP site stratigraphy and surface morphology are consistent with expected characteristics of a developed, older (paleo) karst landscape mantled by several meters of sand (i.e., a mantled epikarst subsurface). Although there are no recognized sinkholes in the State of Florida sinkhole database or the SDII Global Corporation's much larger, private database (Reference 2.5.1-328) within 2 km (1.28 mi.) of the LNP site and no sinkholes at the land surface were observed during site investigations and reconnaissance within the LNP site, the presence of a few voids at depths identified in some borings suggests that paleo sinks such as those developed on the barren mature epikarst surface are locally present at the site.

Based on the review and updating of the geological, seismological, geophysical, and geotechnical data for the LNP site, nothing was identified that would preclude the safe operation of the facilities. The only geologic hazard identified in the LNP site area is potential surface deformation related to carbonate dissolution and slow cover subsidence related to the occurrence of karst. Karst features encountered below the nuclear islands at the LNP site are determined to be associated with near-vertical to vertical fractures and subhorizontal bedding planes, and vary in size from a few centimeters to approximately 1.5 m (5 ft.). Karst-related solution zones and/or infilled zones that exist in the subsurface beneath the LNP foundation will be addressed through appropriate design considerations in the LNP foundation conceptual design, as described in FSAR Subsection 2.5.4.

2.5.0.2 Vibratory Ground Motion

The selected starting point for developing the site-specific ground motion assessments for the LNP site was the Probabilistic Seismic Hazard Analysis (PSHA) conducted by the EPRI-SOG in the 1980s. Following guidance in the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.208, the adequacy of the EPRI-SOG hazard results was evaluated in light of new data and interpretations and evolving knowledge pertaining to seismic hazard evaluation in the central and eastern United States (CEUS). PSHA sensitivity analyses were conducted to test the effect of the new information on the seismic hazard. Using these results, an updated PSHA analysis was performed; the results of that analysis have been used to develop uniform hazard response spectra (UHRS) and the identification of the controlling earthquakes.

2.5.0.2.1 Seismicity

For this study, an updated earthquake catalog was created that includes additional historical and instrumental events through December 2006. Only 15 earthquakes larger than body-wave magnitude (m_b) 3.0 have occurred within the LNP site region. The largest event, an m_b 4.3 earthquake, occurred at a distance of 76.6 km (47.6 mi.) from the LNP site and is the only event within 80 km (50 mi.) of the site.

Seismicity that is occurring beyond the site region also was considered. The occurrence of two moderate earthquakes in the Gulf of Mexico in 2006 has implications to the evaluation of seismicity parameters for the Gulf Coast basin source zones that include the LNP site.

2.5.0.2.2 Geologic Structures and Seismic Source Models

In the review of seismic source characterization models developed for post-EPRI-SOG seismic hazard analyses, and comparison of the updated earthquake catalog to the EPRI-SOG evaluation, one additional specific seismic source was identified and evaluated: repeated large-magnitude earthquakes in the vicinity of Charleston, South Carolina.

The EPRI-SOG seismic source models in the vicinity of Charleston, South Carolina, were updated in 2006 by the Southern Nuclear Company (SNC), in support of the Vogtle Early Site Permit Application, to incorporate new information on the possible source of future large earthquakes similar to the 1886 Charleston earthquake; new assessments of the size of the 1886 earthquake; and new information on the occurrence rate for large earthquakes in the vicinity of Charleston, South Carolina. The result was the development of an updated Charleston seismic source (UCSS).

2.5.0.2.3 Correlation of Earthquake Activity with Seismic Sources

Comparison of the updated earthquake catalog to the EPRI-SOG earthquake catalog and EPRI-SOG sources yields the following conclusions:

- In addition to those included in the EPRI-SOG characterizations, the updated earthquake catalog does not show a pattern of seismicity within the site region different from that exhibited by earthquakes in the EPRI-SOG catalog that would suggest a new seismic source.

- 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 based on seismicity patterns.

- The updated catalog does not show any earthquakes within the site region that can be associated with a known geologic structure.

- The largest earthquake known to have occurred in southeastern United States, the 1886 Charleston earthquake, likely reactivated a structure within the basement rock, but cannot be definitely associated with any of the major identified basement structures. Alternative source locations, maximum magnitudes, and recurrence for repeated large-magnitude, Charleston-type earthquakes are incorporated into the PSHA.

- The updated catalog includes two earthquakes that are larger in magnitude than some of the upper- and/or lower-bound values used by EPRI-SOG teams to characterize the maximum magnitude (M_{max}) distribution of source zones within which these earthquakes occurred. These earthquakes are the February 10, 2006, Emb[a] 4.9 earthquake, and the September 10, 2006, Emb 6.0 earthquake. Revisions to some of the EPRI earth science teams (EST) M_{max} distributions for background source zones to account for these events were incorporated into the updated PSHA.

- The February 10, 2006, Emb 4.9 earthquake, which does not exhibit typical source characteristics of a tectonic earthquake, has been potentially associated with specific geologic structures near the edge of the continental shelf. The September 10, 2006, Emb 6.0 earthquake, which has a tectonic signature, has not been tied to any unique geologic structure. This event occurred near the transition between oceanic and thin transitional crust, in extended basement crust having northwest-trending normal faults that are favorably oriented for reactivation in the present tectonic regime.

- The updated earthquake catalog adds a few 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 was assessed.

2.5.0.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

The review of new geological, geophysical, and seismological information, the review of seismic source characterization models developed for post-EPRI-SOG seismic hazard analyses, and a review of the updated earthquake catalog to the EPRI-SOG evaluation have been used to develop an updated seismic hazard model for the LNP site. The EPRI-SOG source models have been modified as follows:

- The UCSS developed by SNC has been included to account for new information regarding the location, size, and occurrence of repeated large-magnitude earthquakes in the vicinity of Charleston, South Carolina.

- Two moderate earthquakes have occurred within the Gulf of Mexico since the EPRI-SOG 1986 study. The magnitudes of these events exceed the upper and/or lower bound of the M_{max} distributions originally proposed by some of the EPRI ESTs for large areal source zones that encompass large portions of the Gulf Coastal Plain and the Gulf of Mexico. The M_{max} distributions have been revised for five of the six EPRI EST source zones to account for these earthquakes in the hazard calculations.

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[a] Emb — Expected estimate of body wave magnitude. Emb values assigned to the 2006 earthquakes in the STP 3 & 4 COLA differ slightly from the LNP catalog due to different versions of magnitude conversion relationships used in the two studies.

- An additional earthquake catalog completeness zone in the Gulf of Mexico has been added to incorporate the contribution of offshore seismicity into the hazard analysis for the LNP site.

The following PSHA model adjustments were studied as part of PSHA sensitivity tests for the LNP site based on this new information:

- Selection of the appropriate set of seismic sources for each EPRI-SOG EST using the updated EPRI ground motion models that will be used to compute the PSHA for the LNP site.

- Sensitivity to new data relative to the occurrence of large earthquakes in the Charleston, South Carolina, region.

- Sensitivity to the updated maximum magnitude distributions for seismic sources extending into the Gulf of Mexico.

- Sensitivity to the updated seismicity parameters for seismic sources extending into the Gulf of Mexico.

The PSHA for the LNP site was conducted using the updated EPRI-SOG seismic sources combined with the UCSS source. Earthquake ground motions were modeled using the median ground motion models developed by EPRI in 2004 and the ground motion aleatory variability models developed by EPRI in 2006.

PSHA calculations were performed for response spectral accelerations at the seven structural frequencies provided in the EPRI 2004 ground motion model: 0.5, 1.0, 2.5, 5, 10, 25, and 100 hertz (Hz) (peak ground acceleration [PGA]). The UCSS produces comparable hazard or somewhat larger hazard than that obtained from the updated EPRI-SOG sources for 10-Hz motions, and dominates the hazard for 1-Hz motions

The mean hazard results were interpolated to obtain UHRS for generic CEUS hard rock conditions for mean annual frequencies of exceedance of 10^{-4}, 10^{-5}, and 10^{-6}.

Deaggregation was conducted to identify the controlling earthquakes for two frequency bands: (1) the average of the 5-Hz and 10-Hz hazard results representing the high-frequency (HF) range, and (2) the average of the 1-Hz and 2.5-Hz hazard results representing the low-frequency (LF) range. The HF deaggregation shows a progression from domination of the hazard by large, distant earthquakes at a mean exceedance frequency of 10^{-3} to dominance by nearby small-magnitude earthquakes at a mean exceedance frequency of 10^{-6}. The LF deaggregation indicates that the distant large-magnitude earthquakes dominate the hazard at all four levels of exceedance frequency.

Site response Approach 2B, defined in NUREG/CR-6728, was used to assess site amplification. In this method, the response spectra of the controlling earthquakes (termed reference earthquakes [RE] in NUREG/CR-6728) are multiplied by mean site amplification function to develop hazard consistent response spectra at the reference location. The mean site amplification functions are computed for a range of earthquake magnitude-distance pairs that represent distribution of earthquakes contributing to the hazard. These are termed deaggregation earthquakes (DEs), and three are defined for both the high-frequency and low-frequency ranges. Smooth response spectra were developed for each DE.

2.5.0.2.5 Seismic Wave Transmission Characteristics of the Site

Site response analyses were conducted to evaluate the effect of the sedimentary rocks on the generic CEUS hard rock ground motions. The intent of these analyses is to develop ground motions at the surface that are consistent with the hazard levels defined for the generic rock conditions.

Shear (V_S) and compression (V_P) wave velocity data were obtained at the LNP site. A combination of suspension logging and downhole velocity surveys were used to measure shear-wave velocities to a depth of approximately 152 m (500 ft.). Measurements were conducted in or near 18 borings, 9 at the site of each LNP unit. Interpreted shear-wave velocity models for each boring was based on interpretations of the velocity data and comparisons to boring log lithology and a suite of other geophysical logging survey data. The shear-wave velocity data show a generally consistent pattern at the two units. Velocity information that was available for other wells in the site vicinity was used to estimate shear-wave velocity for the deeper part of the section down to and including the Paleozoic units underlying the site location.

The ground motion response spectra (GMRS) were calculated at an elevation of 11 m (36 ft.) in the North American Vertical Datum of 1988 (NAVD88) at the top of the calcareous silt (unit S2) (undifferentiated Tertiary unit, interpreted as weathered rock). The materials that are included in the site response analysis to develop the GMRS consist of approximately 18.3 m (60 ft.) of partly to moderately calcareous silts (units S2 and S3) above unweathered sedimentary rocks. To account for the potential of nonlinear behavior in the calcareous silt units (weathered rock), two alternative sets of modulus reduction and damping relationships were used.

Site response calculations were performed for four initial profiles. Analyses were preformed using two sets of modulus reduction and damping relationships and the best estimate value for κ. For each analysis, 60 randomized profiles were generated and the mean site amplification (response spectrum for surface motion divided by response spectrum for input motion) was computed.

Based on sensitivity analyses, two profiles (one for each unit) were selected for calculation of the site amplification. The envelope of the site amplification computed from the two profiles was used to develop surface motion.

The site response analyses profiles for the design grade case were developed by adding a layer of engineered fill to the GMRS profiles to bring the top elevation up to 15.5 m (51 ft.) NAVD88. The analyses were performed for a wide range of engineered fill properties.

2.5.0.2.6 Ground Motion Response Spectra

The final assessment of the surface UHRS was based on PSHA calculations that use cumulative absolute velocity (CAV) filtering in place of a fixed minimum magnitude. These UHRS were used to develop the GMRS.

The horizontal GMRS for the LNP site were developed using the performance-based approach defined in NRC Regulatory Guide 1.208 (based on UHRS developed using CAV filtering). The computed GMRS corresponds to the minimum of 0.45 times the 10^{-5} USRS. The vertical GMRS were developed by multiplying the horizontal GMRS by vertical/horizontal spectral ratios derived from the ratios recommended for western United States (WUS) rock and CEUS hard rock in NUREG/CR-6728. The horizontal and vertical site GMRS are enveloped by the Westinghouse Certified Seismic Design Response Spectra (CSDRS).

Performance-based surface response spectra (PBSRS) and associated soil column outcropping response (SCOR) foundation input response spectra (FIRS) were developed using the site response analysis of profiles that extended to the design grade elevation. These spectra were scaled upward to meet the requirement of a minimum peak horizontal acceleration of 0.1 gravity acceleration (g) at the reactor foundation level. These spectra are used to develop inputs for soil structure interaction (SSI) analyses. The scaled PBSRS are also enveloped by the Westinghouse CSDRS. Design grade (elevation 15.5 m [51 ft.]) SSI input response spectra were also developed. Three SSI input soil profiles were developed from the randomized soil profiles used to compute the PBSRS. These profiles accommodate the variability in the in-situ materials and the anticipated range in fill properties.

2.5.0.3 Surface Faulting

The evaluation of the potential for surface deformation at Levy Nuclear Plant Unit 1 and 2 (LNP 1 and LNP 2) considered both tectonic and nontectonic origins.

Investigations performed to evaluate the potential for surface fault rupture at LNP 1 and LNP 2, as well as the surrounding LNP site area, included compilation and review of existing data and literature, lineament analyses, discussions with current researchers in the area, field reconnaissance, geomorphic analyses, and review of seismicity data. Results of the surface faulting study indicate that there is no evidence for Quaternary tectonic surface faulting or fold deformation at the LNP site, and no capable tectonic sources have been identified within 40 km (25 mi.) of the site.

The LNP site is located on a marine terrace that is estimated to be older than 340,000 years, possibly of early Pleistocene to late Pliocene age. There is no geomorphic evidence to suggest that the bedrock surface (marine plantation surface) underlying the Quaternary terrace cover deposits in the site location has been displaced or deformed by tectonic faulting. The nearly horizontal terrace surface generally exhibits only minor relief related to karst development. There are no pronounced lineaments across the site location that suggest the presence of a through going fault or major fracture system.

The potential for nontectonic deformation at the site from phenomenon other than karst-related collapse or subsidence is negligible.

The LNP site area is situated in an area that could potentially have karst feature development (see FSAR Subsection 2.5.1). An assessment of aerial photos and site investigation was conducted to identify key features associated with solution subsidence activity. Although evidence of solution activity was encountered in some of the boreholes advanced as part of this COLA, findings from the soils and rock borings, along with geophysical testing, did not indicate the presence of major solution features that would have a significant impact on the safety of a nuclear plant with a properly designed foundation.

2.5.0.4 Stability and Uniformity of Subsurface Materials and Foundations

Surface geologic deposits observed at LNP 1 and LNP 2 consist of undifferentiated Quaternary age fluvial and marine terrace sediments, primarily silty fine sands. The sands overlie the Avon Park Formation, a shallow marine carbonate rock unit of mid-Eocene age, characterized as cream to brown or tan, poorly indurated to well-indurated, variably fossiliferous limestone, interbedded in places with tan to brown, very poorly to well-indurated, fossiliferous, vuggy dolostones. Carbonized plant remains are common in the rock sequence in the form of thin, poorly indurated laminae and cyclic interbeds.

The depth of undifferentiated Quaternary (unit S1) and Tertiary (units S2 and S3) sediments varies. The top of rock (unweathered Avon Park Formation) occurs at an approximate elevation of -7.3 m (-24 ft.) NAVD88 at the LNP site, with undulations due to the erosional nature of the surface. The reactor islands of LNP 1 and LNP 2 will be founded at basemat elevation +3.5 m (+11.5 ft.) NAVD88. Therefore, the Avon Park Formation rock is below the bottom of the basemat of each nuclear island. The Avon Park Formation rock has a weighted mean dip of 2 degrees at both LNP 1 and LNP 2 within the subsurface investigation depth, i.e. 152 m (500 ft.).

A subsurface investigation program, consisting of geotechnical boreholes, geophysical surveys, in situ testing, and laboratory testing, was performed from January 2007 through January 2008 in accordance with Regulatory Guide 1.132 and Regulatory Guide 1.138. A total of 118 boreholes were advanced, including 10 initial phase boreholes, 90 main phase boreholes, and 18 supplemental boreholes. The depth of these boreholes ranged from less than 30 m (100 ft.) to nearly 152 m (500 ft.) below the ground surface (bgs) with at least 19 at each nuclear island with depths of more than 61 m (200 ft.). Geophysical survey methods were conducted in representative boreholes. These survey methods included suspension P-S velocity logging, downhole shear-wave logging, acoustic televiewer surveys, and non-seismic borehole geophysical surveys, including natural gamma logging, gamma-gamma logging, neutron-neutron logging, and induction logging. In addition, pressuremeter testing (PMT) was performed at various depths in one borehole at each LNP site. A total of 213 special-care rock core samples were laboratory tested for unconfined compressive strength (UCS) and other index tests. Forty two special-care rock core samples were laboratory tested for split tensile strength and other index tests. Nine special-care rock core samples were used for triaxial compressive strength tests in laboratory. Numerous soil samples were tested for index properties.

Engineering properties of subsurface materials were characterized from the site investigation activities. Two of the key properties are summarized as follows:

- The average shear-wave velocity (V_S) from all suspension P-S velocity logging at LNP 1 varied from 760 to 1680 meters per second (m/sec) (2500 to 5500 ft/sec) below the top of rock. At LNP 2, the average V_S varied from 760 to 1520 m/sec (2500 to 5000 ft/sec) below the top of rock. Three and four rock layers were defined for engineering analysis based on shear-wave velocity at LNP 1 and LNP 2, respectively.

- The average UCS from laboratory tests on intact rock core samples of the rock layers varied from 4.8 to 25.5 megaPascals (MPa) (700 to 3700 pounds per square inch [psi]) at LNP 1 and varied from 4.8 to 20 MPa (700 to 2900 psi) at LNP 2. UCS results range from 0.9 to 127.3 MPa (131 to 18458 psi) among all samples tested from the LNP site.

The nuclear island building floor elevation for LNP 1 and LNP 2 is elevation +15.5 m (+51 ft.) NAVD88. The ground surface elevation immediately outside of the reactor islands will be at elevation +15.5 m (+51 ft.) NAVD88, except where required to be lower due to water control. The surrounding grade will be lower to accommodate site grading, drainage, and local site flooding requirements. The current ground surface varies approximately from +12.3 to +13.2 m (+40.3 to +43.2 ft.) NAVD88 at LNP 1, and from +12.1 to +13.4 m (+39.8 to +43.9 ft.) NAVD88 at LNP 2. Therefore, site fill of approximately +1.8 to +3.0 m (+6 to +10 ft.) will be required to raise the grade.

The nuclear island basemat will be founded at elevation +3.5 m (+11.5 ft.) NAVD88 on an 11 m (35 ft.) roller compacted concrete (RCC) bridging mat. A waterproof geomembrane will be placed on the RCC and topped with a 15-centimeter (cm) (6-inch [in.]) mudmat, as described in the DCD (Subsection 3.4.1.1.1), prior to placement of the nuclear island basemat. Excavation for construction of the RCC bridging mat and nuclear island is facilitated by permeation grouting and a perimeter diaphragm wall. Grouting from the ground surface will provide a barrier over a 23 m (75 ft.) zone of the Avon Park Formation below the planned RCC. The diaphragm wall will be keyed into the grouted limestone formation and provide a side barrier for excavation dewatering. Grouting reduces gross porosity and permeability to facilitate dewatering but also reduces long-term groundwater flow to minimize potential solution impact.

Groundwater dewatering flow rates were calculated as summarized in FSAR Subsection 2.5.4.6.2. It is anticipated that groundwater inflow during construction can be managed by six submersible pumps (each with 378 liters per minute [lpm] [100 gallons per minute [gpm]] capacity) installed in wells located around the inside perimeter of the diaphragm wall and pumps placed in sumps within the excavation. Although highly unlikely, a second round of drilling and pressure grouting in localized zones could be implemented at specific locations to help seal areas where groundwater is seeping through the engineered barriers.

The factors of safety (FS) for static and dynamic bearing capacity were analyzed for safety-related structures. Conservative methodology was used to estimate bearing capacity as summarized in FSAR Subsection 2.5.4.10.1. Static and dynamic FS were greater than 3.0 and 2.0 for both LNP 1 and LNP 2. The nuclear island foundations have no potential for liquefaction because these foundations consist of RCC, dental concrete, grouted rock, and rock. Some material adjacent to the nuclear island will be replaced or improved due to potential liquefaction, or detailed analysis for nuclear island sliding will demonstrate an adequate margin of safety without credit for passive wedge resistance. The LNP 1 and LNP 2 Annex Buildings (seismic Category II structures) will be founded on deep foundations (4000-psi concrete drilled shafts) that are socketed into the Avon Park Formation. The downdrag load on the deep foundation due to the potential liquefaction of soils will be also resisted by the rock socket in the Avon Park Formation.

Total and differential settlements of safety-related structures were estimated based on elastic compression rock mass from average elastic moduli established by suspension P-S velocity logging surveys at LNP 1 and LNP 2. Total settlements at each LNP site are estimated to be within acceptable settlement criteria for the Westinghouse AP1000 Reactor (AP1000) nuclear islands. The differential settlement (distortion) slopes are estimated to be less than 0.00083 (or 1/1200), which is within the acceptable range for the AP1000 under both LNP 1 and LNP 2.

- Adjacent nonsafety-related structures will be founded on deep foundations (4000-psi concrete drilled shafts) that are socketed into the Avon Park Formation. Preliminary settlement analyses indicate that these structures will exhibit very little total settlement (less than 25 millimeters (mm) [1 in.]), and therefore any potential for differential settlement is negligible.

2.5.0.5 Stability of Slopes

The site grade at the LNP site will be constructed at approximately 15.5 m (51 ft.) NAVD88, with minor variations to allow drainage for an area of about 370 m by 390 m (1210 ft. by 1280 ft.) around the nuclear island. No permanent slopes will be present at the site that could adversely affect safety-related structures.

The AP1000 does not utilize safety-related dams or embankments, and there are no existing upstream or downstream dams that could affect the LNP site safety-related facilities.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

LNP COL 2.5-1 This subsection presents information on the geologic and seismologic setting of the LNP site. Appendix C, "Investigations to Characterize Site Geology, Seismology, and Geophysics," of the NRC Regulatory Guide 1.208 "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion" provides additional guidance on geological, seismological, and geophysical investigations that should be conducted to develop an up-to-date, site-specific earth science database that supports site characterization and a site-specific probabilistic seismic hazard analysis (PSHA). FSAR Subsection 2.5.1 presents geologic and seismologic information, as outlined in the NRC Regulatory Guide 1.208, about the site region (within a 320 km [200 mi. radius) including site vicinity (within a 40 km [25 mi.] radius), site area (within an 8 km [5 mi.] radius), and site location (within a 1 km [0.6 mi.] radius).

Several sources of information were used to develop the information summarized in this subsection. The Final Safety Analysis Report for the Crystal River Unit No. 3 Nuclear Generating Plant (CR3) (Reference 2.5.1-201), which is located approximately 18 km (11 mi.) southwest of the LNP site, provided a limited amount of information applicable to the LNP analysis. A more comprehensive database was developed for the LNP site that incorporates reports, maps, and articles published by state and federal agencies and professional/academic journals, remote sensing imagery, aerial photographs, and digital elevation model data. Additional unpublished information and data also were obtained through communications with individual researchers and personnel at universities, the Florida Geological Survey (FGS), and Southwest Florida Water Management District (SWFWMD).

The emphasis was placed on identifying new information that would suggest significant differences from the information used to develop the Electric Power Research Institute's (EPRI) seismic source characterization model (Reference 2.5.1-202), which forms the starting point for the assessment of seismic hazard at sites in the CEUS (see discussion in FSAR Subsection 2.5.2). Regional compilations of information on the origin and development of the Appalachian-Ouchita orogen and Gulf of Mexico basin provide more recent assessments of the tectonic evolution, structural framework of the region, and geophysical characteristics of the crust that were used to evaluate the seismic source characterization parameters.

The information in this subsection is organized in accordance with Regulatory Guide 1.208. FSAR Subsection 2.5.1.1 describes the regional geologic and tectonic setting, focusing primarily on the region within a 320 km (200 mi.) radius of the LNP site. The EPRI (Reference 2.5.1-203) seismic hazard analysis for the nearby CR3 identified the Charleston seismic zone, which was the source of a large, geologically recent earthquake, as a more distant (greater than 320 km [200 mi.]), but significant, seismic source. This subsection, therefore, also presents updated information regarding the location, magnitude, and recurrence of this seismic source. In addition, recent earthquakes of moderate size that occurred in the Gulf of Mexico during 2006 have implications for the characterization of seismic source zones that include the LNP site. The tectonic settings of these events are described in this subsection.

FSAR Subsection 2.5.1.2 describes the geology and structural setting of the site vicinity (40 km [25 mi.] radius) and site area (8 km [5 mi.] radius) and also addresses geologic conditions at the site location (1 km [0.6 mi.] radius).

2.5.1.1 Regional Geology

This subsection describes the physiography, geologic history, and tectonic setting of the area within a 320 km (200 mi.) radius of the LNP site. Also presented is relevant new information on potential seismic sources for the large-magnitude 1886 earthquake in the Charleston, South Carolina, area that lies beyond the 320 km (200 mi.) radius of the site.

The LNP site is located on the west coast of the Floridian plateau or platform, a recently emergent part of the south-central North American Plate that separates the Gulf of Mexico from the Atlantic Ocean (Reference 2.5.1-204). The western boundary of the modern Florida platform is defined by the West Florida escarpment, a high-relief (as much as 2 km [1.2 mi.]), north-south-trending, largely erosional slope that is morphologically complex and vertical to overhanging in places (Reference 2.5.1-205). The Straits of Florida mark the southwestern and southern boundary of the platform (Figure 2.5.1-201). During the Late Jurassic to Early Cretaceous, the Florida – Bahamas platform complex was part of a great Mesozoic carbonate bank that stretched from Mexico and the Bahamas to Canada along the east coast of North America (Reference 2.5.1-205). This carbonate bank developed on basement structures associated with Early Mesozoic rifting and seafloor spreading between North America and Africa, involving the early formation of the North Atlantic Ocean, as well as the Late Triassic to Early or Middle Jurassic rifting and seafloor spreading that formed the Gulf of Mexico (Reference 2.5.1-205).

Basement rocks underlying the Florida platform include Precambrian – Cambrian igneous rocks, Ordovician – Devonian sedimentary rocks, and Triassic – Jurassic volcanic rocks (Reference 2.5.1-204). Paleozoic basement rocks had a Gondwana origin and were joined to the North American Plate in the final stages of development of the Appalachian Mountains (Reference 2.5.1-206) in the Late Carboniferous to Early Permian (Reference 2.5.1-207). The igneous and sedimentary basement rocks originated from the African Plate but remained attached to the North American Plate (Reference 2.5.1-204) when rifting that resulted in opening of the present Atlantic Ocean occurred in the Middle Triassic to Early Jurassic (Reference 2.5.1-208).

Over the basement rocks, lies an unconformably thick sequence of mid-Jurassic to Holocene sediments. From Middle Jurassic to Middle Oligocene, carbonate sedimentation was widespread along the Florida platform. This depositional regime changed in response to sea-level fluctuations and uplift and erosion in the Appalachian highlands. Starting in the mid-Oligocene and continuing into the Holocene, deposition of siliciclastic-bearing carbonates and siliciclastic sediments dominated the Florida platform (Reference 2.5.1-204). This thick sequence of unconsolidated to semiconsolidated sedimentary rocks comprises the Coastal Plain physiographic province.

2.5.1.1.1 Regional Physiography and Topography

The LNP site is located in the Coastal Plain province of the Atlantic Plain division of North America as shown on the regional physiographic map (Reference 2.5.1-209) (Figure 2.5.1-201). The following descriptions of the Coastal Plain province in the site region are taken from Thornbury (References 2.5.1-210, 2.5.1-211, and 2.5.1-212).

2.5.1.1.1.1 Physiography and Topography of the Coastal Plain Province

The Coastal Plain province, located along the eastern and southeastern margin of the United States, is an extensive seaward-sloping plain that extends from Cape Cod past the Mexican border. The part of the plain below sea level is called the continental shelf (Reference 2.5.1-210).

The Coastal Plain province is composed of relatively unconsolidated sediments of both marine and terrestrial origin that range in age from Early Cretaceous to Holocene. In general, younger sediments adjoin the continental shelf on the east, and older sediments lie to the west and northwest. Sediments range in thickness from a few hundred feet at the inner margin of the Coastal Plain to many thousands of feet on the continental shelf. (Reference 2.5.1-210)

Because there are significant differences in both geology and topography within the Coastal Plain province, it has been divided into six sections. Three of the six sections lie within a 320 km (200 mi.) radius of the LNP site: the Sea Island section, the East Gulf Coastal Plain section, and the Floridian section (Figure 2.5.1-201). (Reference 2.5.1-210) The LNP site is located within the Floridian section of the Coastal Plain.

2.5.1.1.1.1.1 Sea Island Coastal Plain Section

The Sea Island section includes the northeastern part of Florida and southern Georgia and continues up to North Carolina (Figure 2.5.1-201). Although its north and south boundaries are arbitrarily drawn, there are a few distinguishable characteristics for this section. In general, it is a youthful to mature terraced coastal plain having a slightly submerged margin. Although the rivers are drowned in their lower parts, they are not associated with large estuaries. In place of offshore bars (prominent in the embayment section to the north), a chain of coastal islands known as the Sea Islands have developed. Inland, there is a large nonterraced zone that has been dissected sufficiently such that the inner Coastal Plain is considered “hilly.” In addition, a series of depressions known as the “Carolina Bays” are extensively developed in this section (Reference 2.5.1-210).

The Sea Island section is divided into five distinct topographic belts. From the inner margin to the coast, the topographic belts are: the Fall Line Hills, a strip of maturely dissected hills cut out of the oldest coastal plain deposits; the Tifton Upland, an area of submaturely dissected coastal plain; a belt of older coastal terraces, which show a moderate degree of erosion; a belt of younger terraces, which are largely unmodified by stream erosion and have developed extensive swampy areas; and lastly, an offshore line of sea islands (Reference 2.5.1-210).

2.5.1.1.1.1.2 East Gulf Coastal Plain Section

The East Gulf Coastal Plain section begins near the Georgia – South Carolina boundary and continues west to Louisiana and as far north as Illinois. Although no sharp topographic boundary exists between the Sea Island section and the East Gulf Coastal Plain section, changes in the topographic characteristics between the two sections are sufficient to warrant differentiation. A westward increase in the number and thickness of the Cretaceous and Eocene formations, which results in a widening of the Coastal Plain in the East Gulf Coastal Plain section, and greater variability in the lithological characteristics and resulting erosibility of the underlying bedrock units are the primary factors responsible for the change in topography. The variation in erosibility of rocks within the East Gulf Coastal Plain have produced a youthful to maturely dissected, belted coastal plain that consists of a series of alternating cuestas, and lowlands. (Reference 2.5.1-210)

The East Gulf Coastal Plain section can be divided into two distinct types of topography. The majority of the section consists of a series of the belted coastal plain that includes a series of lowlands developed on more easily eroded rock (limestones and shales) bounded by cuesta scarps and dip slopes on the stronger, less easily eroded rocks (commonly sandstones) (Reference 2.5.1-210). The only topographic belt of the East Gulf Coastal Plain that is within the 320 km (200 mi.) radius of the LNP site is the Flatwoods belt, and it is described as a lowland that was developed on the Eocene Midway Formation (Reference 2.5.1-210). The other type of topography within the East Gulf Coastal Plain consists of a sequence of coastwise terraces that lie adjacent to the coast (Reference 2.5.1-210).

2.5.1.1.1.1.3 Floridian Coastal Plain Section

The Floridian section, which encompasses the entire peninsula of Florida, is a recent emergent platform characterized by widespread distribution of carbonate rocks with associated karst features. It is also characterized by the Florida Keys island chain along the southern tip of the peninsula, swamp areas near the southwest and extensive marine terraces on the east, south and west sides of the peninsula, and central areas where terraces are either lacking or obscure (Reference 2.5.1-210).

The peninsula of Florida is a recently emergent part of the Floridian Plateau, of which the submerged portion still greatly exceeds the emergent part (Figure 2.5.1-201). The emergent part of the plateau is asymmetrically distributed with respect to the entire plateau. The edge of the continental shelf is only a few miles off the east coast, whereas on the west coast, the continental shelf lies many miles offshore. (Reference 2.5.1-210) The east coast is bounded by a narrow and steeply sloping shelf to the north that narrows greatly to the south. In contrast, the west coast shelf is about 200 km (61 mi.) wide with shoreface gradients that range from 1:1300 in the central area to less than 1:3000 in both the north and south. Such conditions coupled with the limited fetch of the Gulf of Mexico cause wave energy reaching the coast to be very low. (Reference 2.5.1-213) The shelf is very broad (greater than 150 km [46 mi.]) and exhibits a very low, seaward-dipping gradient (1:5000) inherited from the ancient underlying carbonate platform along the northwestern part of the Florida peninsula in a region referred to as the Big Bend (Reference 2.5.1-213).

The coastal lowlands bordering the coastline of Florida are low in elevation and their topographic features include estuaries and lagoons, barrier islands, coastal ridges, relict spits and bars, and intervening coast-parallel valleys (Reference 2.5.1-214). Mapping of these shoreline landforms and related deposits in association with general elevation information provided the basis for the identification on a state-wide level of eight marine terrace intervals (Figure 2.5.1-202). (Reference 2.5.1-215) These terraces, which range in elevation from less than 3 m (10 ft.) to 65 – 97 m (215 – 320 ft.) above mean sea level (amsl), provide a record of sea-level highstands dating from Late Tertiary to possibly Late Holocene time (Reference 2.5.1-214). Further discussion of the ages of these terraces is provided in FSAR Subsection 2.5.1.2.1.2.

Another prominent feature of Florida’s landscape is karst topography, which developed in response to dissolution of carbonate rocks that are widely distributed throughout the Florida peninsula. Typical landforms include small sinkholes on nearly planar karst platforms in the Coastal Lowlands; rolling hills and sinkholes on the ridges, such as the Brooksville Ridge and Lake Wales Ridge in north-central Florida; and isolated collapse sinkholes in buried karst and newly developing karstic uplands. (Reference 2.5.1-214)

The Florida peninsula is divided into three generalized physiographic zones separated by roughly east-west boundaries. From north to south, these are the northern (or proximal) zone, the central (or midpeninsular) zone, and the southern (or distal) zone. Superimposed onto these subdivisions is a Central Highlands geomorphic providence trending north-northwest to south-southeast through the center of the peninsula border. It is surrounded to the east, south, and west by plains and coastal lowlands (Figure 2.5.1-201). (Reference 2.5.1-212) The following descriptions of the three physiographic zones are from Bryant et al. (Reference 2.5.1-216)

The northern zone is a broad upland that includes northern Florida and southern Georgia and extends westward to the Florida panhandle and southeastern Alabama. It is bound on the south and east by a scarp that is interrupted by valleys of major streams. It reaches elevations 60 to 90 m (200 to 300 ft.) amsl and generally lies above the piezometric surface. Because it is above the piezometric surface, the northern zone is composed of dry karst sinks, abandoned spring heads, dry stream courses, and dry beds of former shallow lakes that are now prairies.

The midpeninsular zone is composed of discontinuous subparallel ridges that rise to about 61 m (200 ft.) amsl, lie parallel to the length of the peninsula, and are separated by broad valleys that commonly contain numerous shallow lakes. The LNP site is located within the central or midpeninsular zone.

The southern zone is characterized by a broad, gently sloping plain that lies less than 10 m (35 ft.) amsl. This poorly drained plain predominantly lies below the piezometric surface; it is covered by extensive swamps in which the carbonate rocks may be overlain by as much as 3.5 m (10 ft.) of peat.

Both the Northern and Central highlands are thought to be the remnants of a once much larger highland that has been dissected by erosion and differential dissolution of the underlying carbonate rocks. The major ridges of the Central highlands are believed to represent relict coastal features that reflect the many advances and retreats of the shoreline resulting from changes of sea level during the Quaternary.

2.5.1.1.2 Regional Geologic History

The LNP site is located on the Florida platform, a region that has experienced a complex tectonic history. The Paleozoic features record a long period of plate convergence that resulted in the North American Plate suturing to the Gondwanan to form the supercontinent of Pangaea. The Mesozoic extensional features record the Triassic to Jurassic rifting of Pangaea and the openings of the central North American and Gulf of Mexico. Cretaceous and Cenozoic features reflect a history of regional tectonic quiescence accompanied by extensive carbonate deposition and rapid subsidence. The plate tectonic history of the Florida platform has been pieced together using borehole data, geophysical interpretations, and extensive correlations with geologic data collected from West Africa. Details of these data sets and interpretations of the geologic history of the region are presented in a series of recent publications, as noted, and together they provide the basis for the following summary of regional geologic history.

2.5.1.1.2.1 Late Proterozoic and Paleozoic Geologic History

In the Middle Proterozoic, several continental masses, including proto – North America (Laurentia) and continental terranes to the southeast (relative to the present direction), were assembled into a supercontinent during the Grenville orogeny, which occurred approximately 1.1 to 1.0 billion years ago (Ga).

(Reference 2.5.1-217) Long after the Grenville orogeny, the Proterozoic supercontinent underwent a prolonged period of extension resulting in the formation of continental rifts, intrusion of magmas and accompanying volcanism, and finally, rupture to form an ocean basin. Two pulses of continental rifting are recorded in the central and southern Appalachians separated by about 200 million years. The first pulse of rifting, about 760 to 700 million years before present (Ma), did not lead to continental separation; the second one, about 570 to 560 Ma (earliest Cambrian), resulted in continental separation. (Reference 2.5.1-217) The resulting ocean, the proto – Atlantic Ocean (the Iapetus, Theic, and Rheic oceans, as they are variously differentiated), opened somewhere east of the Grenvillian massifs along an irregular margin that researchers speculate is reflected in the present salients and recesses of the Appalachian orogen. The jagged nature of the rifted margin has been interpreted as originating from the offset of an oceanic spreading ridge by transform faults or the intersection of rifts at triple junctions. The Iapetan crustal fracture system probably influenced the location and geometry of the Mesozoic rift system of eastern United States. (Reference 2.5.1-217)

The time of continental breakup (i.e., the rift to drift transition) is near the Late Proterozoic – Cambrian boundary. With the opening of the Iapetus Ocean and thermal subsidence of the continental margin, the sea transgressed onto Laurentia beginning in the Early Cambrian along the Appalachian Valley. Platform sediments, mostly carbonates, were deposited across the Interior Lowlands and a passive margin carbonate bank developed along the Laurentian margin in the southern Appalachians. The carbonate sedimentation was interrupted at the end of the Early Cambrian by an influx of clastic sediments from the west. (Reference 2.5.1-217)

Stratigraphic and sedimentologic analyses indicate that the Appalachian region subsequently experienced several compressional events (Reference 2.5.1-218, Reference 2.5.1-217) that began in the Middle Orodovician (Reference 2.5.1-219) and culminated in the amalgamation of Pangaea by collision of Laurentia with northwestern Africa (Reference 2.5.1-220), to form the Appalachian orogen in the Late Paleozoic (Permian) (Reference 2.5.1-217). As the ocean was closing, numerous microcontinents and ocean terranes formed and were accreted as exotic or suspect terranes onto the eastern margin of North America. Differences in lithostratigraphy and deformational history, as well as fossils such as Cambrian trilobite faunas, have been used to identify some terranes as probably having originated in Europe or Africa before being accreted to North America. (Reference 2.5.1-221)

The Suwannee terrane, which underlies the coastal plain of Florida and southern Georgia and Alabama, is recognized to be a piece of proto – Gondwana, a detached fragment of the African Plate that was sutured to the North American during the Alleghanian orogeny (Figure 2.5.1-203) (Reference 2.5.1-217). During the Late Proterozoic, the Suwannee terrane was part of a felsic volcanic province, perhaps an island arc or backarc basin on the margin of Africa – South America. The southeastern part of the terrane, including the St. Lucie Metamorphic Complex and Osceola Granite, was involved in the Pan-African deformation, which formed the Rokelides of western Africa. The Suwannee terrane remained stable until at least Middle Devonian time, but probably during the Hercynian orogeny, which closed part of the Iapetus, the strata were gently folded. (Reference 2.5.1-222)

In contrast to sedimentary rocks in the Valley and Ridge province of the western Appalachians that were deformed during the Late Carboniferous and earliest Permian Alleghanian orogeny, similar age rocks of the Suwannee terrane are relatively undeformed. The Suwannee basin was on the edge of the South American – African Plate during the Early to Middle Paleozoic, away from the orogenic activity along the eastern North American Plate. The Late Paleozoic collision of the South American – African Plate with North America brought the basin against the North American Plate (Reference 2.5.1-223). Although the precise location of the Late Paleozoic plate suture (Suwannee – Wiggins suture) is not known, it must lie at some position north of the northern extent of the Suwannee basin (Reference 2.5.1-206). The zone of deformation, evidenced by the folding and thrusting in the Appalachian, Mauritanides, and Ouachitas, did not include the Suwannee basin. A transform zone or very oblique subduction zone must have existed just north and west of present-day Florida, isolating northern Florida from the orogenic activity. (Reference 2.5.1-223)

Smith et al. (Reference 2.5.1-224), as reported in Smith and Lord (Reference 2.5.1-225), postulated that the basement terranes, Suwannee basin and Osceola complex, were moved laterally into relative positions along strike-slip faults, such as the Jay fault, that formed in response to the continental collision between Laurentia and Gondwana (Figure 2.5.1-204). The Jay fault of Pindell (Reference 2.5.1-226) generally coincides with Klitgord et al.'s Bahamas Fracture Zone (Reference 2.5.1-223) and Christenson's Florida Lineament (Reference 2.5.1-227). The postulated development of right-lateral faults, as shown on Figure 2.5.1-204, is thought to be a response to the Gondwanan landmass closing with, and then rotating clockwise around, the Alabama promontory. Basement fragments southwest of the Jay fault and associated faults could then have been displaced northwest toward the Ouchita orogen, which may have reduced the stress on the Florida platform, as evidenced by the limited deformation in the Suwannee basin and the isolated thrust folds at the western edge of the platform, in addition to suturing these Gondwanan fragments to North America during the creation of Pangaea. (Reference 2.5.1-225)

2.5.1.1.2.2 Mesozoic Geologic History

During the Middle Triassic to Early Jurassic, rifting began that initiated the breakup of the supercontinent Pangaea and created the present-day Atlantic Ocean. A series of interior rifts, including many of the present-day Triassic basins of the Atlantic margin, may have developed simultaneously, facilitating rifting of North America from Gondwana along the traces of the Paleozoic margins (Figure 2.5.1-205). (Reference 2.5.1-225) Many of the normal faults bounding the rift basins were reactivated Paleozoic structures. (Reference 2.5.1-208) The rift zone stretching from North Africa to the Gulf of Mexico was initiated from several hot spots, each with radially propagating rift arms that likely contributed to the overall form of the continents. One such hot spot was located near the southern tip of the Florida platform. A plume-induced diapiric uplift, with associated volcanic activity in the South Florida – Bahamas platform, created a ridge-ridge-ridge triple junction that produced extensional rifting in the area, thereby preventing reactivation of the Paleozoic suture located north of the Suwannee basin. Thus the Florida – Bahamas Plateau remained appended to the North American plate (see Figure 2.5.1-206). (Reference 2.5.1-206)

A large graben, the South Georgia rift, formed across southern Georgia, nearly along the trace of the Alleghenian suture between Florida and North America that is marked by the Brunswick magnetic anomaly (Reference 2.5.1-228). Shifting of the rift geometry caused the South Georgia rift to become an aulacogen. As the rifting stopped, the basements of the Florida platform, the Yucatan, and the Bahamas platform were left appended to southeastern North America. (Reference 2.5.1-225)

The actual breakup of Pangaea, with the separation of North America from the South America – African Plate, produced a set of marginal basins and established a spreading center system where voluminous amounts of new igneous rock formed oceanic crust. Seafloor spreading was well established by the late Middle Jurassic. As the continents continued to rift apart in the early Middle Jurassic, block faulting and intrusion of new igneous rock produced a sediment trap along the edge of the continents that developed into the set of marginal sedimentary basins. These marginal basins are underlain by transitional crust and are bound in the landward direction by a basement hinge zone. (Reference 2.5.1-223)

Around the southeast corner of the North American Plate, the Blake Plateau, Bahamas, and South Florida basin developed as the heated, thinned crust cooled and subsided, serving as depocenters for sediment from the adjacent continents (Figure 2.5.1-207). These marginal basins developed along the eastern branch of the Atlantic twin rift zone, which succeeded in breaking apart the supercontinent of Pangaea. By contrast, the western branch is one that failed with its last tectonic activity being the Middle Jurassic diabase intrusive phase. During this same period, the Gulf of Mexico basin was cut off from the Atlantic and Pacific, and continental conditions prevailed until late Middle Jurassic. (Reference 2.5.1-223)

The Early to Middle Jurassic transform zone between the Atlantic and Gulf of Mexico was localized between the Bahamas and Capeche fracture zones and was thought to be centered over southern Florida. Based on apparent offsets of magnetic and gravity lineations on the west Florida shelf, Klitgord et al. (Reference 2.5.1-223) concluded that most of the transform motion must have taken place on the Bahamas, Cuba, and Campeche fracture zones (Figure 2.5.1-207). Jurassic igneous rock in Florida located just north of the Bahamas fracture zone is cited as evidence that the Bahamas fracture zone was a major continental edge shear zone with associated block faulting and intrusive activity. (Reference 2.5.1-223)

The tectonic outline of the Mesozoic continental margin of southern North America appears to be similar in shape to that of the latest Precambrian and Early Paleozoic; the trace of the inferred Bahamas fracture zone, which appears to extend northwest to link up with the Alabama – Arkansas fault system, essentially coincides in location with an inferred older transform fault related to the opening of the Iapetus Ocean. Similar-shaped promontories formed north of the transform (the Paleozoic Alabama promontory, and the Mesozoic Florida promontory that included Late Paleozoic accreted terranes). (Reference 2.5.1-229) Peel et al. (Reference 2.5.1-230) state that except for the evidence of Late Jurassic basement faulting on the periphery of the Gulf of Mexico basin noted by Thomas, there is no documented geologic evidence of subsequent active basement faulting within the Gulf of Mexico.

In Late Jurassic, as the Atlantic and Gulf of Mexico continued to open, the Florida – Bahamas region remained a transform plate boundary. The Bahamas basin became a subsiding marginal basin with a spreading center actively generating oceanic crust to the southeast between the Jacksonville and Bahamas fracture zones. The Gulf of Mexico was completely formed by the end of the Jurassic. (Reference 2.5.1-223)

Since the extensive Triassic-Jurassic rifting, the Florida platform has been tectonically quiescent, which is evidenced by the undisturbed Upper Cretaceous and Tertiary strata on the platform. Carbonate, evaporite, and siliciclastic sediments began to accumulate on the Florida platform with the development of the Gulf of Mexico basin of deposition during the Middle Jurassic. Within the eastern Gulf, the Apalachicola basin and Tampa embayment were two main depocenters for thick evaporite sedimentation. Thick, massive deposits of gypsum and anhydrite are believed to be the result of a silled basin with restricted water circulation. Great thicknesses of salt accumulated during the Jurassic and Cretaceous. Evaporite deposition was usually followed by clastic fluvial, eolian, and marine sediments in the northern parts of the platform, and, with a rise in sea level, carbonate sedimentation dominated. Alternating clastic fluvial and deltaic deposition and carbonate sedimentation reflect sea-level variation throughout the Middle to Late Jurassic and Early Cretaceous. (Reference 2.5.1-231)

2.5.1.1.2.3 Cenozoic Geologic History

The time interval between the Late Cretaceous and Early Cenozoic represents the greatest accumulation of sedimentary rocks in Florida. During this period, the depositional sequences formed in a relatively stable tectonic setting that produced a broad, shallow-water marine platform that experienced little tilting or disturbance. Steep submarine escarpments now bound the gently dipping Florida platform (Figure 2.5.1-201). The sedimentary sequences consist of nearly flat-lying marine rocks, terminating at these boundary escarpments. (Reference 2.5.1-231)

The continental margin experienced rapid cooling and subsidence approximately 200 to 150 Ma, and is now toward the end of its cooling and subsidence curve.

(Reference 2.5.1-232) The age and elevation of the Florida Keys that lie just beyond the 320 km (200 mi.) boundary of the site region suggests that less than a few meters of subsidence has occurred in the last 100,000 years. (Reference 2.5.1-233) Additionally, the Florida Keys are slightly influenced by isostatic motions associated with the shift in load back and forth from continental ice to the global ocean. (Reference 2.5.1-233) Peltier calculated that deglaciation-induced downward vertical motions of the Atlantic margin should be expected for the entire east coast of the United States, and that this subsidence may amount to about 0.1 centimeter per year (cm/y) in Florida. (Reference 2.5.1-234)

During the first 35 million years of the Cenozoic, sea levels were high and carbonate sediments predominated on the Florida platform, forming the thick sequence of limestones, dolostones, and evaporites. Sea levels dropped in the Late Oligocene, exposing the carbonate platform to erosion and karstification. Near the beginning of the Miocene, approximately 25 million years ago, siliciclastic sediments began to be transported onto the platform in sufficient quantities to effectively suppress carbonate deposition. By mid-Pliocene, siliciclastic sediments almost covered the entire platform, leaving only isolated areas where significant carbonate deposition occurred. By Late Pliocene, the siliciclastic sediment supply began to diminish, and carbonate sedimentation was reestablished in southernmost Florida. (Reference 2.5.1-235)

During the Late Oligocene, there was a sea-level regression resulting in major changes in the depositional regime for the Florida platform. Prior to the Late Oligocene, the Georgia Channel system acted as a barrier to siliciclastic transport onto the Florida platform. A combination of low sediment supply and occupation of the Georgia Channel system with the Suwannee current resulted in minimal transport of siliciclastics onto the platform. When the sea level dropped in the Late Oligocene, the Suwannee current abandoned the Georgia Channel system. Simultaneously, it was postulated that the Appalachians were broadly uplifted, providing a large sediment supply and drastically increasing the input of siliciclastic sediment to the marine environment. With the sea levels at a low stand, the increased sediment supply filled the channel and allowed siliciclastics to begin to encroach onto the Florida platform. (Reference 2.5.1-235)

The Georgia Channel system represents two distinct but related sedimentological systems. The older system is known as the Suwannee strait, or channel, which existed from the Late Cretaceous to the Middle Eocene and was associated with two embayments — the Tallahassee embayment and the Southeast Georgia embayment. The younger system, the Gulf trough, existed from the Middle Eocene to the Middle Miocene and is thought to have only a westerly embayment — the Chatahoochee embayment. The Tallahassee and the Chatahoochee embayments are collectively referred to as the Apalachicola embayment (Figure 2.5.1-208). (Reference 2.5.1-231)

Encroachment of the siliciclastic sediments onto the carbonate platform occurred slowly, and it was not until the Late Miocene to Pliocene that siliciclastic sediments dominated the Florida platform. Siliciclastics replaced the carbonates on the northern portion of the platform first and by Middle Miocene, these sediments were deposited over the panhandle and the northern half of the peninsula. Carbonate deposition continued on the southern portion of the peninsula until late into the Middle Miocene or earliest Pliocene. (Reference 2.5.1-235)

Neogene – Quaternary sediments vary from thick deposits in the central and western panhandle and the northeastern and southern peninsula to thin sequences on the Ocala platform and Chattahoochee “anticline” (Figure 2.5.1-209). Miocene sediments at one time likely covered the Ocala platform and the Chattahoochee “anticline,” but erosion has removed them, exposing the underlying Paleogene carbonates. Periodic regressions of the sea during the Neogene and Quaternary also helped expose vast areas of the carbonate platform. During these episodes of sub-aerial exposure, dissolution of the underlying carbonates continued, allowing karst features to develop. Weathering and reworking of marine sediments filled the karst features and the fluvial features with terrestrial sediments. These terrestrial deposits include numerous sand dune fields scattered across the platform. (Reference 2.5.1-235)

Although the distribution of Miocene and younger marine sedimentary rocks at the surface demonstrates emergence of the Florida platform from a submarine environment during the Neogene, no evidence exists to suggest tectonic deformation or orogenic activity of any nature throughout the Cenozoic (Reference 2.5.1-225). Epeirogenic uplift and regional elevation of Plio-Pleistocene beach ridges in northern Florida have been attributed to density changes within limestone formations experiencing Pleistocene and Holocene karstification (Reference 2.5.1-329). Estimates of dissolved limestone based on current erosion rates suggest an isostatic uplift of at least 9 m (30 ft.) and as much as 58 m (190 ft.) since the beginning of the Quaternary (~1.6 Ma). (Reference 2.5.1-330)

Pleistocene high sea-level stands are recorded in southern Florida and in the reef record of the tectonically stable Florida Keys island chain that lies just beyond the site region boundary (Reference 2.5.1-236). Stratigraphic studies suggest that two high sea-level stands of Middle Pleistocene age are recorded there; preliminary ages are on the order of approximately 300 to 340 thousand years before present (ka) and 220 to 230 ka. Corals in reefs of two high sea-level stands of the last interglacial complex, the approximately 80 ka and 120 ka stands, also are present on the Florida Keys, and the approximately 120 ka-highstand terrace is mapped around the perimeter of the Florida peninsula at an elevation of approximately 6 m (21 ft.) amsl. (Reference 2.5.1-237) Because the Florida platform represents long-term carbonate sedimentation on a passive margin, late Quaternary deposits on the Florida Keys have not experienced significant uplift, subsidence, or tectonic deformation. (Reference 2.5.1-237)

2.5.1.1.3 Regional Stratigraphy

The Florida peninsula is relatively low, generally less than 100 m (330 ft.) above sea level, and shows little relief, reflecting the flat attitude of the predominantly carbonate Cretaceous and Cenozoic section that underlies Florida. Beneath this carbonate platform is a basement of variable age and composition. The geologic units within a 320 km (200 mi.) radius of the LNP site are shown on Figure 2.5.1-210. Because the LNP lies entirely within the Florida peninsula, the regional stratigraphy discussion is focused on the stratigraphy of the Florida platform. Southern Georgia and the Florida panhandle have similar rock types and geologic histories, so these areas are discussed together.

2.5.1.1.3.1 Pre-Cretaceous Stratigraphy

Florida's crustal basement rocks, those rocks that predate the Cretaceous section, originated from the African Plate, a portion of which remained attached to the North American Plate when the continents separated in mid-Mesozoic. This fragment of the African Plate provided the base for the development of the carbonate platform. Figure 2.5.1-211 is a geologic map of pre-Cretaceous rocks in the basement of Florida and the surrounding areas. (Reference 2.5.1-238)

The main types of pre-Cretaceous rocks of Florida are Jurassic igneous and volcaniclastic rocks in the South Florida basin; Paleozoic granites, diorites, and rhyolites (age unknown) in central Florida (central Florida basement complex); relatively undeformed Paleozoic sedimentary rocks in northern Florida (East Suwannee basin); and block-faulted, Paleozoic sedimentary units in the Florida panhandle (West Suwannee basin). Overlying the sediments of the West Suwannee basin are Triassic age (?) arkosic sandstones of the South Georgia rift. Igneous rocks in southern Florida (South Florida basin) include diabase, basalt, and rhyolite, the ages of which range throughout the Jurassic. Just north of the Bahamas fracture zone, Cambrian age granites were observed in one drillhole, overlain by Jurassic-aged igneous rocks. (Reference 2.5.1-223) Additional details on the Osceola Granite, the St. Lucie Metamorphic Complex, the Late Proterozoic – Cambrian felsic volcanic rocks, and the Paleozoic sedimentary rocks are presented below and have been summarized from Thomas et al. (Reference 2.5.1-222)

Osceola Granite. The Osceola Granite occupies a rectangular area in central Florida (Figure 2.5.1-211). The rocks are classified as granite, alaskite, and quartz monzonite and are referred to as true igneous rocks rather than granite gneisses. Rubidium-strontium (Rb-Sr) dating indicates an age of at least 530 Ma. Biotite concentrates yield internally concordant argon isotope ratio ($^{40}Ar/^{39}Ar$) incremental-release spectra that have plateau ages of 525 to 535 Ma. The lack of either deformation or retrograde alteration suggests that these are postmagmatic cooling ages.

St. Lucie Metamorphic Complex. Southeast of the Osceola Granite subcrop, high-grade metamorphic rocks that make up the St. Lucie Metamorphic Complex have been identified in a few wells. These rocks have been classified as quartz diorite gneiss, amphibolite, and chlorite schist. An Rb-Sr age of approximately 530 Ma indicates that these metamorphic rocks are likely contemporaneous with the Osceola Granite. Hornblende concentrates yield internally concordant $^{40}Ar/^{39}Ar$ plateau ages of 495 to 515 Ma, which is interpreted as the time of postmetamorphic cooling.

Late Proterozoic – Cambrian Felsic Volcanic Rocks. Felsic volcanic rocks form a large part of the Suwannee terrane; however, determination of the age of these rocks is complicated by a range of radiometric dates. Felsic volcanic rocks that are Late Proterozoic – Cambrian ages are observed north of the projected extension of the Bahamas fracture zone, and a predominantly mafic suite that is Early Mesozoic age is located south of the Bahamas fracture zone. The felsic volcanic rocks include porphyritic rhyolite, vitric crystal tuff, and volcanic agglomerate. Most of these appear to be pyroclastic, although some intermediate and mafic lava is present.

Paleozoic Sedimentary Rocks. Overlying the felsic volcanic rocks, presumably unconformably, is a sequence of relatively flat-lying undeformed sedimentary rocks within a large subcrop area in north-central Florida and limited areas within western Florida and adjacent Alabama and Georgia. The lower part of the sequence consists of white reddish quartz sandstone that is interbedded with micaceous shales. Arenig graptolites and inarticulate brachiopods define the age of the sandstone as Early Ordovician, possibly as old as Late Cambrian. Above the Ordovician sandstones are dark gray to black shales interbedded with gray fine-grained micaceous sandstones and locally medium- to coarse-grained quartz sandstones. This sequence is divided into three sections based on paleontological data. The lowest section contains Middle to Upper Ordovician fauna, including a trilobite, inarticulate brachiopods, conularids, conodonts, and chitinozoans. Shales of Late Silurian to Early Devonian age host a wide range of fauna, including bivalves, gastropods, orthocone cephalopods, crinoids, chitinozoans, and other microfossils. At the top of the sequence is a group of shales and sandstones containing Middle Devonian land plants, bivalves, ostracodes, and marine microfossils. The total thickness of the sedimentary sequence is not known, but based on gravity modeling, it is estimated to be approximately 2500 m (8200 ft.) in parts of north-central Florida and based on seismic profiles, as much as 10 km (6 mi.) beneath the Florida panhandle, which includes approximately 3600 m (11,811 ft.) of Silurian and Devonian strata.

The pre-Cretaceous surface and postrift unconformity are shown on Figure 2.5.1-212, as are different types of rocks sampled in drillholes that penetrated this surface. (Reference 2.5.1-223) In addition to the plan view map, one geologic cross section through northern and peninsular Florida (A-A′) illustrates the generalized basement structures and overlying sedimentary units. (Figure 2.5.1-213) The profiles are based on drillhole data, Bouguer gravity anomalies, magnetic anomalies, and crystalline basement surfaces determined from gravity modeling. (Reference 2.5.1-223)

The northeast-trending Paleozoic Suwannee basin across northern Florida contains Ordovician quartzitic sandstone and Silurian – Devonian black shale. Geophysical data (Reference 2.5.1-239) indicate that a crystalline basement complex similar to that of central Florida underlies the Paleozoic basin, but only one borehole, at the edge of the basin, has encountered igneous or metamorphic basement. Elsewhere, up to 700 m (2296 ft.) of Paleozoic sedimentary rocks were penetrated by other drillholes in the Suwannee basin without reaching basement. Paleozoic rocks also underlie the Middle Ground Arch, in contrast to granite and rhyolite porphyry intruded with diabase sampled farther south on the west Florida platform. In the panhandle of Florida, Paleozoic marine sedimentary rocks in the West Suwannee basin have been block-faulted and covered with Triassic-aged sedimentary rocks. Based on a seismic profile across the panhandle, Klitgord et al. (Reference 2.5.1-223) interpreted the granitic plutons observed near the coastline in the central panhandle are part of the basement complex for the West Suwannee basin. (Reference 2.5.1-223)

2.5.1.1.3.2 Cretaceous and Post-Cretaceous Stratigraphy

Sedimentary rocks of the Coastal Plain of Florida and adjacent areas of Alabama and Georgia rest unconformably on pre-Cretaceous basement. Depositional sequences of the Cretaceous and Cenozoic formed in a relatively stable tectonic setting and consist of nearly flat-lying marine rocks stacked approximately 7 km (4.3 mi.) thick and terminating at the boundary escarpments of the platform. The stratigraphy is characterized by a general west-to-east and north-to-south gradation of clastic to carbonate sedimentation. Evaporite formation became less prevalent, gradually decreasing during the Late Cretaceous and Early Cenozoic, the time interval that saw the greatest accumulation of sedimentary rocks in Florida. (Reference 2.5.1-231) A regional correlation chart illustrating the sedimentary rock record of Florida from the Middle Jurassic to the Holocene is provided on Figure 2.5.1-214. (Reference 2.5.1-211) Regional cross sections that extend north to south along the length of the Florida peninsula and west to east along the Florida panhandle are shown on Figures 2.5.1-215, 2.5.1-216, and 2.5.1-217, respectively. (Reference 2.5.1-240)

Descriptions of the regional geologic units are summarized from Randazzo. (Reference 2.5.1-231) Because of the striking contrast between the clastic rocks of the panhandle and carbonate rocks of the peninsula, the discussion of the regional geologic units have been subdivided into peninsular Florida and panhandle Florida.

2.5.1.1.3.2.1 Peninsular Florida

In the southern part of the peninsula, the Lower Cretaceous consists of approximately 3000 m (10,000 ft.) of carbonate rocks, which include at least two thick evaporite units: the Punta Gorda Anhydrite of the Glades Group, and the Panther Camp Formation of the Naples Bay Group. Because of the limited stratigraphic control of these Lower Cretaceous units, the stratigraphic sequence has been lumped together as the Marquesas Supergroup. The most notable carbonate unit within this supergroup is the Sunniland Limestone (Aptian of the Ocean Reef Group), the dominant oil-producing formation in peninsular Florida.

The carbonate formations are generally dolomitic limestones, with varying amounts of interbedded evaporites. These units represent peritidal and subtidal shelf environments of deposition, reflecting small- and large-scale sea-level fluctuations. The carbonate-dominated Marquesas Supergroup is thinner

(600 – 2000 m [2000 – 6560 ft.]) in northern peninsular Florida, where its lower part has more interbedded clastics and fewer evaporites. (Reference 2.5.1-231)

The "Mid-Cretaceous Sequence Boundary" (MCSB) is a stratigraphic indicator representing a global fall of sea level, followed by a dramatic rise in sea level, significant drowning of the platform, and deposition of deeper-water carbonate sediments during the Late Cretaceous. (Reference 2.5.1-231) The MCSB in the Florida peninsula is marked by the Atkinson Formation (Cenomanian and possibly Turonian), a mixed carbonate-clastic unit. During the rest of the Late Cretaceous, carbonate deposition dominated the platform and is represented by the Pine Key Formation, which contains at least two extensive dolomite sequences in southern Florida (Card Sound Dolomite and Rebecca Shoal Dolomite).

In the Paleocene and Lower Eocene, the Cedar Keys Formation, which is largely dolomitized and consists of peritidal carbonate rocks with interbedded and intergranular evaporites (anhydrite and gypsum), was deposited. The overlying Oldsmar Formation (Middle Eocene) is less dolomitized and is represented by shallow-water shelf deposits and peritidal carbonates and evaporites. The depositional environment for these units likely was characterized by increasingly higher sea levels from the Paleocene to the Eocene. Peritidal evaporite and carbonate sedimentation was gradually replaced by more normal marine carbonate deposition.

The Avon Park Formation of Middle Eocene age is the oldest stratigraphic unit exposed in Florida. It lies unconformably over the underlying Oldsmar Formation, suggesting an intervening episode of sub-aerial exposure and erosion (sea-level fall). The contact, observed only in drill cores, typically is recognized by the contrast between older, porous, foraminiferal grainstones and packstones and younger dolomitic wackestones-mudstones. The Avon Park Formation is a carbonate mud-dominated peritidal sequence, pervasively dolomitized in places and not dolomitized in others, and contains some intergranular and interbedded evaporites in its lower part. Fossils are mostly benthic forms showing limited faunal diversity. Seagrass beds are well preserved at certain horizons.

The Upper Eocene Ocala Limestone is composed of two units. The lower unit is partially dolomitized and consists of both restricted- and open-marine carbonate lithologies. The boundary between the lower Ocala Limestone and the Avon Park Formation is unconformable in some places and conformable elsewhere. The Avon Park Formation commonly is thin-bedded, with algal laminations, and finer-grained in its upper part, contrasting with the interbedded skeletal-rich packstones-wackestones, and mudstones of the lower Ocala.

The upper unit of the Ocala Limestone is typically composed of white to gray foraminiferal and molluscan packstones and grainstones, with lesser amounts of wackestones and mudstones. Faunal diversity is high. During the Late Eocene, the depositional environment consisted of open-marine, shallow-water, and middle-shelf deposition.

In the Oligocene, the Suwannee Limestone is the main carbonate unit that was deposited in an open-marine environment and is characterized by packstones and grainstones. The boundary between the Ocala and Suwannee limestones is commonly difficult to identify because of their similar lithologic appearances. One difference is the sand content of the Suwannee Limestone; towards the top of the unit, the sand content increases and several dolomitized sections are present. This gradual change in lithology is the result of lowering of the sea levels and the influx of clastic sediments from the north. (Reference 2.5.1-231)

The Hawthorn Group is the predominant stratigraphic unit for the entire Miocene section (entire peninsula) and part of the lower Pliocene section (central and southern peninsula only), as shown on Figure 2.5.1-214. In the northern peninsula, the Hawthorn Group includes most of the Miocene section and comprises (in ascending order) the Penney Farms, Marks Head, and the Coosawhatchie formations, and in limited areas, the Statenville Formation and the St. Marks Formation. Carbonate sediments interbedded with siliciclastics are abundant in the Penney Farms and Marks Head formations and siliciclastics are predominant in the Coosawhatchie Formation and younger units. (Reference 2.5.1-235) In the central and southern peninsula, the Hawthorn Group includes the entire Miocene section and part of the lower Pliocene section of central and southern Florida. The units included within the Hawthorn Group include (in ascending order) the Arcadia Formation (with the Tampa and Nocatee members) and the Peace River Formation (with the Bone Valley Member and the Wabassco beds). Siliciclastic-bearing carbonates predominate in the Arcadia Formation, except in the Nocatee Member, where siliciclastics predominate. In the Peace River Formation, siliciclastics predominate. (Reference 2.5.1-235)

Overlying the Hawthorn Group in the northern peninsula is the siliciclastic Cypresshead Formation and the variably fossiliferous and siliciclastic Nashua Formation. These units are overlain by undifferentiated, variably fossiliferous, siliciclastic Pleistocene-Holocene deposits. (Reference 2.5.1-235) In the central and southern peninsula, the section overlying the Hawthorn Group is a complex of often highly fossiliferous siliciclastic sediments and siliciclastic-bearing carbonates. Pliocene sediments are included in the Tamiami Formation, Ochopee Limestone, and the lower part of the carbonate and siliciclastic Caloosahatchee Formation. The Cypresshead Formation originates from the north and extends into central and southern Florida. The Caloosahatchee Formation in central and southern Florida overlies the Tamiami Formation and in turn is overlain by the Pleistocene “Bermont Formation” (an informal name, but widely used). The Bermont is overlain by the Fort Thompson Formation and both formations are composed of fossiliferous siliciclastics interbedded with carbonates. (Reference 2.5.1-235)

In southeastern Florida, the Key Largo Limestone, a coral-dominated Quaternary carbonate rock that underlies the upper northeastern Florida Keys, is a Pleistocene reefal unit that makes up a facies of the Miami Limestone. The lower (southwestern) keys are composed of the Miami Limestone, a dominantly oolitic marine carbonate rock. (Reference 2.5.1-237) The Key Largo Limestone is believed to have formed as a complex of shallow-water shelf-margin reefs and associated deposits along a topographic break. There was debate about the origin of the limestone because of biotic differences in modern reefs; however these are likely due to environmental stresses at the time caused by lack of insular shields and distance from the shoreline. (Reference 2.5.1-233)

Perkins (Reference 2.5.1-241) mapped five Quaternary units in southern Florida based on regional sub-aerial discontinuities. Muhs et al. (Reference 2.5.1-236) also distinguish five Quaternary units in southern Florida, and based on uranium-series age dates suggest that much of the marine sedimentation took place between 400 and 144 ka. The elevation of reef tracks recorded on the Florida Keys that are approximately 300 – 340 ka (marine oxygen isotope stage [MIS] 9) and 220 – 230 ka (MIS 7) suggest that sea level may have been close to present during these two periods of high sea level. (Reference 2.5.1-237) Coral in reefs of two high sea-level stands of the last interglacial complex, the approximately 80-ka and 120-ka stands, also are present in the Florida Keys. Sea level was on the order of 6 m (21 ft.) higher than present during MIS 5e (approximately 120 ka); this shoreline, and related platform and overlying deposits, are present along the margins of almost the entire Florida peninsula (Figure 2.5.1-218). (Reference 2.5.1-237)

Surficial geologic units mapped by Fullerton et al. (Reference 2.5.1-242) in the site region include Pleistocene and Holocene beach deposits; Pliocene, Pleistocene, and Holocene coastal plain and coastal deposits; Holocene and late Wisconsin channel and floodplain alluvium; Holocene and late Wisconsin inland deposits; and solution residuum of Quaternary and Tertiary age.

2.5.1.1.3.2.2 Panhandle Florida

The Lower Cretaceous rocks of the panhandle consists of a series of undifferentiated sandstones, shales, limestones, and evaporites (more than 2000 m [6500 ft.] thick), which unconformably overlie the Jurassic to Lower Cretaceous Cotton Valley Group. The MCSB separates these rocks from the Upper Cretaceous Tuscaloosa Group, a quartz sand-rich clastic unit that is equivalent in time to the Atkinson Formation of the peninsula. The upper part of the Tuscaloosa Group is finer grained in the eastern panhandle and is unconformable with the sandy and conglomeratic Eutaw Formation of the western panhandle and unnamed carbonate and mixed carbonate clastic sequences of the central panhandle. Carbonate deposition progressed westward through the panhandle during the latest Cretaceous, as represented by the Selma Group. This carbonate unit is characterized by its chalk lithology, but it is dolomitic and interbedded with calcareous clay. (Reference 2.5.1-231)

The Paleocene rocks of the panhandle are unconformable with the Cretaceous units. The Wilcox and Midway groups, approximately 500 m (1640 ft.) thick, are mixed carbonate-clastic units that are overlain by more carbonate-rich sediments of the Claiborne Group (Lower and Middle Eocene). Late Eocene and Oligocene sediments are characterized by an extension of carbonate systems from the peninsula to the panhandle, represented by the Ocala Limestone and Suwannee Limestone in both areas. The Early Oligocene is marked by another event of clastic sedimentation in the western panhandle (Vicksburg Group), whereas in the central and eastern panhandle, carbonate deposition (Suwannee Limestone) continued until the Late Oligocene. The panhandle stratigraphy units reflect sea-level variations during the Cretaceous and Paleogene, and the responding change in position of carbonate and clastic deposition. (Reference 2.5.1-231)

In the western panhandle, the basal Miocene sediments were comprised of carbonates containing varying percentages of siliciclastics. These carbonate units are the St. Marks Formation, Tampa Limestone, and the Chattahoochee Formation. Overlying this section is an unnamed, Chipola-equivalent siliciclastic limestone and the fossiliferous Bruce Creek Limestone. During the Middle Miocene, sedimentation was dominated by siliciclastic deposition with subordinate carbonates. The Pensacola Clay was deposited during Middle to Late Miocene and was overlain by Pliocene coarse clastics and then the Plio-Pleistocene siliciclastic Citronelle Formation. The final unit in the western panhandle is the Pleistocene – Holocene alluvium. All these units may be sporadically fossiliferous. (Reference 2.5.1-235)

In the central panhandle, the siliciclastic-bearing carbonates occur in the Lower Miocene, represented here by the Chattahoochee and the St. Marks formations. Carbonate deposition was replaced as the dominant sediment early in the Middle Miocene. The Middle and Upper Miocene Alum Bluff Group, consisting of variably fossiliferous siliciclastics and carbonates, includes the Chipola, Oak Grove Stand, Shoal River, and Choctawhatchee formations. The Alum Bluff Group, Bruce Creek Limestone, and lower parts of the mixed siliciclastic-carbonate Intracoastal Formation overlie the Lower Miocene carbonates. The Pliocene Jackson Bluff Formation and the upper Intracoastal Formation overlie the Alum Bluff Group. Overlying the Jackson Bluff Formation, and interfingering with the Intracoastal Formation, is the Citronelle Formation, which is overlain by siliciclastic Pleistocene – Holocene alluvium. (Reference 2.5.1-235)

In the eastern panhandle, the siliciclastic-bearing carbonates of the Chattahoochee and St. Marks formations are the oldest Neogene units in the eastern panhandle. They are overlain by the Torreya Formation of the Hawthorn Group. The basal Torreya is carbonate that grades into a siliciclastic upper part. The Hawthorn Group is overlain by the Jackson Bluff Formation, which underlies the siliciclastic Miccosukee Formation. Undifferentiated Pleistocene – Holocene siliciclastic deposits cap the Neogene Sequence. (Reference 2.5.1-235)

2.5.1.1.4 Regional Tectonic Setting

The seismotectonic framework of a region, which includes the basic understanding of existing tectonic features and their relationship to the contemporary stress regime and seismicity, forms the foundation for assessments of seismic sources. In the probabilistic seismic hazard study performed by the Electric Power Research Institute and Seismic Owners Group (EPRI-SOG) from 1986 to 1988 (Reference 2.5.1-202), seismic source models were developed for the CEUS based on 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 geosciences community in the mid- to late 1980s.

Since the EPRI-SOG study, additional geological, seismological, and geophysical research has been performed in the site region. This subsection presents a summary of the current state of knowledge of the regional tectonic setting and highlights the more recent information that is relevant to the identification of seismic sources for the LNP site. The following subsections describe the region in terms of:

- The contemporary tectonic stress environment (FSAR Subsection 2.5.1.1.4.1).
- Regional structural setting and geophysical framework (FSAR Subsection 2.5.1.1.4.2).
- Tectonic features within a 320 km (200 mi.) radius of the site (FSAR Subsection 2.5.1.1.4.3).
- Significant seismic sources at distances greater than 320 km (200 mi.) (FSAR Subsection 2.5.1.1.4.4).
- The regional seismicity (FSAR Subsection 2.5.1.1.4.5).

2.5.1.1.4.1 Contemporary Tectonic Stress

The south-central United States is a passive continental margin with no relative differential motion (i.e. angular velocity) between the Gulf of Mexico and the North American continental plate (Reference 2.5.1-243) as indicated by tectonic plate motion models developed from space geodesy. (Reference 2.5.1-244) The region is one of low earthquake activity and low stress (Reference 2.5.1-245), and is cited as an example of a stable continental region. (Reference 2.5.1-246)

The LNP site lies within a compressive midplate stress province characterized by reverse and strike-slip faulting. (Reference 2.5.1-247) A relatively uniform east-northeast compressive stress field that extends from the midcontinent east toward the Atlantic continental margin and possibly into the western Atlantic basin was identified by Zoback and Zoback. (Reference 2.5.1-247) Their analysis was based on various indicators, including earthquake focal mechanisms, stress-induced elliptical borehole enlargements (or borehole “breakouts”), measurements of hydraulic fracturing stress, and offsets of young faults and alignments of volcanic vents. (Reference 2.5.1-247) Zoback and Zoback note that although localized stresses may be important in places, the overall uniformity in the midplate stress pattern suggests a far-field source, and the range in orientations coincides with both absolute plate motion and ridge push directions for North America. (Reference 2.5.1-247) Modeling of various tectonic processes using an elastic finite-element analysis has indicated that distributed ridge forces are capable of accounting for the dominant east-northeast trend of maximum compression throughout much of the North American Plate east of the Rocky Mountains. (Reference 2.5.1-248)

Zoback and Zoback (Reference 2.5.1-247) also conclude that the available data do not support a distinct Atlantic Coastal Plain stress province. Previously, such a stress province, characterized by northwest compression, inferred from the orientations of post-Cretaceous reverse faults in the Coastal Plain region and focal mechanisms in the northeastern United States, had been interpreted and published by Zoback and Zoback. (Reference 2.5.1-249) The 1980 reference, which was used by the EPRI-SOG teams, has been superseded by Zoback and Zoback (Reference 2.5.1-247), the findings of which are supported by higher-quality stress data.

Based on analysis of well-constrained focal mechanisms of North American midplate earthquakes, Zoback (Reference 2.5.1-250) concludes that earthquakes in the central and eastern United States occur primarily on strike-slip faults that dip between 40 and 80 degrees, primarily in the range of 60 to 75 degrees. The analysis demonstrates that central and eastern U.S. earthquakes occur primarily in response to a strike-slip stress regime.

The southward-oriented extension along the northern Gulf of Mexico region probably reflects crustal loading and deformation, within the Mississippi River deltaic complex in the Gulf of Mexico (Reference 2.5.1-249), and may be distinct from the regional east-northeastward-directed regional compressive stress in the underlying basement. Reverse focal mechanisms for earthquakes that appear to have originated in the basement in the abyssal plain region of the Gulf of Mexico are consistent with the east-northeastward-directed compressive stress environment.

2.5.1.1.4.2 Regional Structural Setting and Geophysical Framework

The LNP site is located near the northeastern margin of the Gulf of Mexico basin (also referred to as the Gulf Coast basin or Gulf basin) that includes the present Gulf of Mexico and adjacent rift basins. (Reference 2.5.1-251) The Gulf of Mexico basin is roughly a circular structural basin that has filled with 0 – 15 km (0 – 9.3 mi.) of sedimentary rocks ranging in age from Late Triassic to Holocene (Figure 2.5.1-208). (Reference 2.5.1-252)

Gravity and magnetic maps that are available for the conterminous United States provide information that has been used to evaluate crustal properties and basement structures in the site region and adjacent parts of the northern Gulf of Mexico basin (e.g., References 2.5.1-253, 2.5.1-254, and 2.5.1-255). Figures 2.5.1-219 and 2.5.1-220 show gravity and magnetic anomaly maps based on gravity and magnetic anomaly data that have been more recently compiled and integrated into digital databases by the U.S. Geological Survey. (References 2.5.1-253 and 2.5.1-256)

Sawyer et al. (Reference 2.5.1-255) provide a description of the reflection seismic, refraction seismic, gravity, magnetic, and subsidence techniques that have been used to resolve the gross characteristics and boundaries between the various types of crust under the Gulf of Mexico basin. Sawyer et al. (Reference 2.5.1-255) divide the crust under the Gulf of Mexico basin into four major types: oceanic, thin transitional, thick transitional, and continental (Figure 2.5.1-221). The crust beneath the central part of the basin is oceanic in character; it is surrounded by continental crust, which underneath much of the basin has been greatly attenuated by rift-related extension. Continental crust refers to crust that predated the formation of the Gulf of Mexico and was not significantly modified (i.e., extended, thinned, or intruded) by the Middle Jurassic or later rifting. Transitional crust is crust that was originally continental, but was significantly extended and thinned and probably intruded with magma during Middle and Late Jurassic rifting. Thick, transitional crust was only somewhat thinned during rifting, and there are many blocks that appear relatively unthinned but are surrounded by regions of greater thinning. Thin transitional crust was dramatically and fairly uniformly thinned without lateral variation during rifting. Oceanic crust was formed deep under the Gulf of Mexico basin during the Late Jurassic. Seafloor spreading probably continued for only about 5 to 10 million years. Since the cessation of seafloor spreading, transitional crust and oceanic crust cooled and subsided, allowing deposition of thick sedimentary sequences in the Gulf of Mexico basin.

Interpretation of anomaly patterns in gravity and magnetic data has been used to infer the compositional or structural variations within basement underlying the Florida platform. Smith and Lord (Reference 2.5.1-225) note that because the contributions of overlying sedimentary rocks to either the gravity or the magnetic field are considered minor, the anomaly patterns are indicative of these variations within the basement in the Florida platform region. Smith and Lord also make the following observations. The magnetic anomalies exhibit two dominant trends: a well-defined northeasterly trend in the northern half of the peninsula; and a northwesterly trend in the southern half. The anomaly pattern in northern Florida extends northward only to southernmost Georgia, where individual, prominent, positive anomalies and a sweeping east-west negative anomaly, the Brunswick magnetic anomaly, disrupt the pattern (Figure 2.5.1-220). Magnetic anomalies in the southern part of the Florida peninsula appear to be continuous with those of the southeast Bahamas platform.

Regional Bouguer anomaly maps yield patterns very similar to those described above. The Bouguer anomaly values for the Florida platform region generally range from +42 to –40 milligal (mGal) (Figure 2.5.1-219). The major positive anomalies are in southern and central peninsular Florida (coincident with positive magnetic anomalies). Most interpreters of local anomalies have identified varying basement depths or compositions as the causes of the anomalies. The marked contrast in orientations of both gravity and magnetic anomalies between southern and northern Florida is attributed to a major compositional change in the underlying crust from oceanic crust in southern Florida to continental crust in northern Florida. (Reference 2.5.1-225) This disparity of both magnetic and gravity anomalies between northern and southern Florida has been cited as evidence for a transform plate boundary through Florida in the Jurassic (Reference 2.5.1-223). The postulated feature is an extension of the Bahamas fracture zone, also referred to as the Jay fault or Florida lineament (see discussion below). (Reference 2.5.1-225) Klitgord et al. note that this fault zone marks the southern edge of the Paleozoic and older crust of North America Plate not affected by Jurassic rifting and seafloor spreading. (Reference 2.5.1-223)

2.5.1.1.4.3 Regional Tectonic Structures within a 320 km [200 mi.] Radius

Ewing subdivides the structural framework of the Gulf of Mexico basin into three major structural provinces, which correspond to the three major lithofacies provinces that persist from the Late Jurassic to the Holocene. These include (1) the northwestern progradational margin (from northeastern Mexico to Alabama, (2) the eastern carbonate margins (the Florida and Yucatan platforms), and (3) the western compressional margin. (Reference 2.5.1-252) The LNP site lies within the eastern carbonate platform province (Figure 2.5.1-208).

Principal tectonic features in the site region (320 km [200 mi.] radius) and surrounding portions of southeastern United States are shown on Figure 2.5.1-208 and Figure 2.5.1-209. Major regional structures that are shown on Figure 2.5.1-208 range in age from Paleozoic to Cenozoic. Structures beneath the Coastal Plain sediments underlying the Florida platform are largely inferred from sparse deep well data, geophysical anomalies, and limited seismic data. Various alternative locations for the older regional structures, particularly major strike slip faults and transforms thought to have been involved in the closing of the Iapetus Ocean in the late Paleozoic and the opening of the Gulf of Mexico and Atlantic Ocean in the early Mesozoic, have been inferred from synthesis of geological, geophysical, and tectonic information in the study region (Figure 2.5.1-222). These and other structures inferred to be of Paleozoic and Mesozoic age are shown on Figure 2.5.1-209. Younger structures of Cenozoic age (including likely nontectonic structures such as the Ocala platform) are shown on Figure 2.5.1-258.

2.5.1.1.4.3.1 Postulated Basement Faults

An erosional surface that formed on rocks ranging in age from Precambrian to Middle Jurassic (referred to as the sub-Zuni surface) defines the top of basement rock in Florida (Reference 2.5.1-239). Postulated ‘basement faults’ identified by Applin (Reference 2.5.1-260) and Barnett (Reference 2.5.1-239) as shown on Figure 2.5.1-222 are described below.

Postulated Northeast-trending Basement Fault Identified by Applin. Based on limited subsurface information from deep boreholes that penetrated into pre-Mesozoic rocks in the northern part of the Florida peninsula, Applin (Reference 2.5.1-260) identified a northeast-trending boundary between Ordovician and Silurian rocks and Paleozoic (?) and Precambrian (?) rhyolite, tuffs, and agglomerate that crosses the north-central part of the Florida peninsula. Applin (Reference 2.5.1-260) postulated the existence of a possible basin or graben north of this boundary based on the presence of a thickness of more than 600 m

(2000 ft.) of Ordovician and Silurian strata adjacent to and north of the regionally high area in Precambrian (?) crystalline rocks south of the boundary. Applin states that subsurface data are not sufficient to classify the downwarped feature definitively and the feature, which is depicted as 'pre-Mesozoic basin or graben,' is not shown as a fault on maps or in a diagrammatic cross section in the 1951 publication. In a 1965 publication by Applin and Applin (Reference 2.5.1-259) that refers to the earlier Applin study, the northeast-trending feature is shown as a fault, but no additional information on the postulated structure is provided (Figure 2.5.1-222). The feature, basin, or graben is below the "peneplaned (?) surface of pre-Mesozoic rocks." Applin indicated that this feature did not affect Mesozoic or Cenozoic sediments (Reference 2.5.1-260). Barnett (Reference 2.5.1-239) does not show the northeast-trending structure inferred by Applin (Reference 2.5.1-260) and Applin and Applin (Reference 2.5.1-259). See the discussion of the East Suwannee basin in FSAR Subsection 2.5.1.1.4.3.2.

Postulated Basement Faults Identified by Barnett. From a review of previous studies, examination of geologic data from nearly 80 wells in Florida and Georgia, and interpretation of geophysical data, Barnett provided a more comprehensive summary of basement structures in Florida that permitted a reconstruction of its tectonic history (Reference 2.5.1-239). Barnett concluded that the Peninsular arch was a tilted and faulted block of Precambrian continental crust covered by Paleozoic sediments. The Paleozoic rocks were subjected to Late Paleozoic uplift with some volcanic activity, followed by uplift with tilting, block faulting, and post-orogeneitic igneous intrusion during the Triassic period. Barnett describes a deep Triassic graben underlying the Apalachicola embayment in the northern part of the LNP site region. Barnett notes that evidence is interpretative for major northwest-trending shear zones in the basement of the Florida-Bahama platform close to and south of the 28th parallel. These structures are described further in FSAR Subsections 2.5.1.1.4.3.2 and 2.5.1.1.4.3.3. Barnett does not provide detailed descriptions or justification for the location of many of the faults shown on his interpretative subcrop map of the sub-Zuni surface in Florida and Georgia. The rationale is not provided for identifying many of the faults, including a northwest-trending normal fault that is inferred to occur in basement close to LNP site. Further discussion of evidence for basement faults in the LNP site area is provided in FSAR Subsection 2.5.1.2.4.

2.5.1.1.4.3.2 Paleozoic Tectonic Structures

Peninsular Arch. The term "Peninsular arch" is applied to two different structural features: (1) a basement high that is defined by well data, and (2) a subparallel high in the Upper Cretaceous strata. The exact nature of the structure is unknown, but it may have developed during the closing of the Iapetus Ocean in the Upper Paleozoic. The Upper Cretaceous structure has been interpreted by Applin (Reference 2.5.1-260) to be the result of regional movements during the Mesozoic and Cenozoic. (Reference 2.5.1-227) Miller (Reference 2.5.1-240) describes the Peninsular arch as a northwest-trending feature that was continuously positive from Early Mesozoic until Late Cretaceous time and was intermittently positive during Cenozoic time. Miller concludes that the shape of the Peninsular arch and its effect on sedimentation in north-central Florida are consistent with an upwarp produced by compressional tectonics.

Suwanee – Wiggins Suture. An Alleghanian suture, referred to as the Suwannee – Wiggins suture, extends between the North American crust on the north and the Suwannee and Wiggins terranes on the south (Reference 2.5.1-222). The precise location and tectonic style of the suture are somewhat uncertain (Reference 2.5.1-222). Nelson et al. (Reference 2.5.1-228) interpret a Consortium for Continental Reflection Profiling (COCORP) profile on the coastal plain of Georgia and northern Florida to support the hypothesis that a prominent magnetic low bordered to the south by discontinuous magnetic highs, which is referred to as the Brunswick anomaly, coincides with this Paleozoic suture. In Georgia, essentially undeformed rocks of the Suwannee terrane extend north to the suture. (Reference 2.5.1-222) Thomas et al. discuss varying interpretations of the tectonic style of the Suwannee – Wiggins suture and conclude that a possible transform or highly oblique transpressional boundary is consistent with the lack of evidence for compressional deformation along the eastern part of the suture, postulated strike slip faulting implied by paleomagnetic data, structural trends observed near the intersection of the Appalachian trends in Alabama, and Late Paleozoic metamorphism and possible arc volcanism farther west (Reference 2.5.1-222).

East Suwanee Basin (North Florida Basin). Beneath the coastal plain of southern Georgia, southeastern Alabama, and northern Florida, which encompasses much of the LNP site region, a distinct assemblage of basement and sedimentary cover rocks constitutes the Suwannee terrane of African affinities. (Reference 2.5.1-222) The subsurface distribution of the biostratigraphic units in this region suggests that the Paleozoic sequence occupies a large regional syncline. The structure has come to be known as the Suwannee basin. (Reference 2.5.1-222) The name Suwannee basin has been applied to both the Paleozoic basin of northern Florida and the basement low of the Florida panhandle and southern Georgia. Klitgord et al. have applied the name East Suwannee basin to the Paleozoic basin. (Reference 2.5.1-223) Thomas et al. recommend using the name North Florida basin for the Paleozoic structure and note that although the basin probably is complicated by faults and perhaps gentle folds, the overall structure is relatively simple (Reference 2.5.1-222). Available information indicates a low angle dip to the strata. Northeast-trending magnetic and gravity lows within the basin appear to be related to post Paleozoic grabens in which Silurian – Devonian rocks are preserved. (Reference 2.5.1-222)

Jay Fault. Smith (Reference 2.5.1-257) identified the Jay fault as a northwest-trending fault zone across the Florida panhandle. The Jay fault, which aligns with the Pickens-Gilberton fault and Bahamas fracture zone as defined by Klitgord et al. (Reference 2.5.1-233), defines a steep, down-to-the-south drop-off of basement. The fault as shown by Smith and Lord (Reference 2.5.1-225) roughly coincides with a basement fault mapped by Barnett (Reference 2.5.1-239) across the Florida peninsula near the 28th parallel. The basement structure identified by Barnett (Reference 2.5.1-239) follows a west-northwest trend of reentrants on the aeromagnetic map and an apparent truncation of northwest-trending magnetic anomalies across the Florida peninsula (Figure 2.5.1-220, Sheet 2).

Smith and Lord (Reference 2.5.1-225) and Smith et al. (Reference 2.5.1-224) speculate that the different basement terranes were moved laterally into their relative positions along strike-slip faults, such as the Jay fault, during the Late Paleozoic closure of the Iapetus Ocean. Smith and Lord (Reference 2.5.1-225) suggest that the fault, which likely experienced right-slip displacement during the Late Paleozoic, appears to have been a major transform structure that accommodated left-lateral strike-slip movement during the opening of the Gulf of Mexico in the Early Jurassic. The fault as shown by Smith and Lord (Reference 2.5.1-225) coincides with the Bahamas fracture zone as described by Klitgord et al. (Reference 2.5.1-223).

2.5.1.1.4.3.3 Mesozoic Tectonic Structures

Bahamas Fracture Zone (Florida Lineament). The Bahamas fracture zone, which trends northwest across southern Florida and the West Florida shelf, forms the northeast margin of the Gulf basin and is a Jurassic transform boundary that joined the Gulf of Mexico spreading center to the Atlantic spreading center (Reference 2.5.1-223). This fracture zone probably represents a major crustal boundary between continental and thick transitional rocks (Reference 2.5.1-205). To the northwest, it joins into the Gulf Rim fault zone (also referred to as the Alabama-Arkansas fault zone) (Reference 2.5.1-229), which is a composite fault and graben system that trends northwest across southern Alabama, central Mississippi, northern Louisiana, and central Texas (Reference 2.5.1-251). As noted in the preceding section, the Bahamas fracture zone coincides in part with the Jay fault (Reference 2.5.1-225).

Barnett (Reference 2.5.1-239) interprets two additional left-lateral strike-slip faults across the southern part of the Florida peninsula south of the Bahamas fault zone. The northern of these, which is referred to as the Northwest Providence Channel fault, is interpreted based on a linear trend of aeromagnetic lows that extends west-northwest across Florida at the 27th parallel and appears to join a left-lateral fault having 100 km (60 mi.) of offset recognized in the Gulf of Mexico by Gough (Reference 2.5.1-258). Barnett notes that these faults have been inactive since the Late Jurassic period, except for more localized younger depositional flexures (Reference 2.5.1-239). Christenson (Reference 2.5.1-227), who refers to the structure as the Florida lineament, presents data to suggest that Paleozoic basement terranes that were contiguous before Mesozoic tectonics show minimal lateral offset across this zone. This boundary is interpreted to be primarily a Triassic – Jurassic extensional rift margin.

Sunniland Fracture Zone. The magnetic and gravity fields over the West Florida shelf, south of the Bahamas fracture zone and northwest of the South Florida basin, are characterized by northeast-trending lineations that are truncated or have discontinuities at the Bahamas, Sunniland, and Cuba fracture zones (Reference 2.5.1-223). The Sunniland fracture zone coincides with the termination of magnetic anomalies between 85 degrees west and 86 degrees west. Southeast of Florida, the Sunniland fracture zone marks the termination of several magnetic and gravity anomalies. (Reference 2.5.1-223) Klitgord et al. (Reference 2.5.1-223) speculate that after the closure of the Atlantic Ocean, the Paleozoic or older Guinea Plateau was against the Paleozoic-age central Florida basement complex. The southern edge of the West Africa Plate, the Guinea fracture zone, coincides with the Sunniland fracture zone. (Reference 2.5.1-223)

Florida Elbow Fault. The Florida Elbow fault is thought to be the location in which the Florida Straits block migrated approximately 300 km (187 mi.) east-southeast out of the eastern Gulf to its present position beneath the South Florida shelf and western half of the Bahamas. The Florida Elbow fault runs from the southern escarpment of the Paleozoic Florida Middle Ground arch, continues through the Florida Elbow basin, and crosses Florida near Lake Okeechobee; it underlies the northwest Providence Channel and defines the northern margin of Great Bahama Bank. Such a trend is readily seen on a magnetic anomaly contour map. It is further suggested that sinistral motion along the fault system produced the Florida Middle Ground escarpment by translating the Tampa arch away from the Middle Ground arch. (Reference 2.5.1-226)

Apalachicola Basin (Apalachicola Embayment). The Apalachicola basin is a Jurassic rift basin located southwest of the Bahamas fault zone. The basin is associated with a magnetic high anomaly. (Reference 2.5.1-223) It is a northeast-southwest-trending basin that overlies the southwestern extension of the South Georgia basin, a Triassic – Jurassic rift-related basin. The Apalachicola embayment contains Jurassic and Cretaceous clastic sediments, with a Cretaceous thickness of more than 2000 m (6600 ft.), overlain by 1600 m (5200 ft.) of Tertiary strata. (Reference 2.5.1-252)

Middle Ground Arch. The Middle Ground arch is a poorly defined positive basement feature (Reference 2.5.1-227). The arch is underlain by Paleozoic rocks and is coincident with a narrow northeast-trending gravity low (Reference 2.5.1-223). The Middle Ground arch, like the Sarasota arch, formed over blocks of stranded continental crust broken off from the main North American Plate by Late Triassic – Early Jurassic rifting (Reference 2.5.1-223). The arch formed a north-south clastic-to-carbonate transition zone in the Tertiary (Reference 2.5.1-227).

Sarasota Arch. The Sarasota arch is a positive basement feature that parallels a gravity low. Basement, which has been penetrated by several wells, consists of Cambrian granites and Ordovician rhyolites that are overlain by a Jurassic “granitic wash.” (Reference 2.5.1-227)

South Florida Basin. The South Florida basin extends from the Florida Straits on the east and south, west to the Florida escarpment, and north to the Peninsular and Sarasota arches. (Reference 2.5.1-227) Limited well control indicates more than 8000 m (26,000 ft.) of uppermost Jurassic through Quaternary carbonate and evaporite strata in the basin. Although the offshore area is not well known, the basin probably partially overlies highly extended continental basement. (Reference 2.5.1-252)

South Georgia Rift (Northeast-trending Triassic Georgia Embayment System). A large graben, the South Georgia rift, formed across southern Georgia during Triassic rifting associated with the opening of the Atlantic Ocean. The rift formed nearly along the trace of the Alleghenian suture between Florida and North America. (Reference 2.5.1-225) The southwestern portion of this rift is included in the LNP site region. Shifting of the rift geometry caused the South Georgia rift to become an aulacogen, and as rifting ceased, the basements of the Florida platform, as well as the Yucatan and Bahamas platforms, were left appended to North America. (Reference 2.5.1-225)

Pindell (Reference 2.5.1-226) notes the absence of a well-developed Jurassic sedimentary section related to basin subsidence that would be expected if the northeast-trending Triassic Georgia Embayment system were strictly due to crustal extension. Pindell suggests that the system of faults probably was formed by right-lateral strike-slip shear, which caused local uplift and erosion, as well as deposition of Upper Triassic red beds into associated strike-slip basins.

Tampa Basin. The Tampa basin is flanked on the north by the DeSoto Canyon arch and on the south by the Sarasota arch. The basin coincides with a prominent gravity high. (Reference 2.5.1-227)

2.5.1.1.4.3.4 Cenozoic Tectonic Structures

Brevard Platform. The Brevard platform, which extends south from the Sanford high, is a low, broad ridge or platform expressed on the erosional surface of the Ocala Group. This platform plunges gently to the south-southeast and southeast. (Reference 2.5.1-261)

Chattahoochee Anticline. Eocene and Oligocene sediments are exposed near the hinge of the Chattahoochee anticline in the panhandle region of Florida. The deposition of the Citronelle formation (Miocene to Pliocene) does not appear to have been influenced by this structural feature.

Gulf Trough or Channel. The Gulf trough or channel extends from the Southeast Georgia embayment to the Apalachicola embayment. It is the Miocene expression of the older Suwannee Strait that separated the siliciclastic facies to the north from the carbonate facies to the south during the Early Cretaceous. The Gulf trough was nearly full of sediments by the Late Oligocene and Early Miocene time, allowing increasing amounts of siliciclastic sediments to invade the carbonate environments of the peninsula. (Reference 2.5.1-261) A northeast-trending series of small faults are boundary faults for a series of small grabens that form the Gulf trough (Reference 2.5.1-240).

Jacksonville Basin. The Jacksonville basin, located in northwest Florida, is the most prominent low in the northern half of the peninsula. In the deepest part of the basin, the Hawthorn Group sediments exceed 150 m (500 ft.) in thickness. The Jacksonville basin may be considered a subbasin of the larger southeast

Georgia embayment, but it is separated from it by the Nassau nose. (Reference 2.5.1-261)

Nassau Nose. The Nassau nose is an eastward-dipping positive feature that separates the southeast Georgia embayment from the Jacksonville basin (Reference 2.5.1-261).

Ocala Platform. The Ocala platform (commonly referred to as the Ocala arch or uplift), is the most prominent of the structures in peninsular Florida. Scott (Reference 2.5.1-261) prefers the term "platform" since it does not have a structural connotation. Vernon (Reference 2.5.1-262) described the approximately 370–km- [230-mi.-] long and about 112–km- [70-mi.-] wide) feature as a gentle flexure developed in Tertiary sediments with a northwest-southeast-trending crest that had been flattened by faulting. Vernon dated the formation of the uplift as being Early Miocene based on the involvement of basal Miocene sediments in the faulting and the wedging out of younger Miocene sediments against the flanks of the platform, which was interpreted to be an island area throughout much of the Miocene. Other researchers conclude that the Ocala platform does not warp or otherwise affect sediments older than Middle Eocene, and is not a true uplift. It appears to have been produced by sedimentational processes — either an anomalous buildup of Middle Eocene carbonate sediments and regional eastward tilting, or — more likely — differential compaction of Middle Eocene carbonate material shortly after deposition. Drilling on the crest of the Ocala platform shows that the feature is not of deltaic or reefal origin. (References 2.5.1-240 and 2.5.1-374) In the Ocala platform area, Lower Cretaceous sediments lie on the Paleozoic sediments and on crystalline basement (Reference 2.5.1-252). Eocene limestone presently is exposed at the surface, but various researchers believe that the Miocene-age Hawthorn Group once extended across the platform. (Reference 2.5.1-261)

The general concensus of researchers familiar with the stratigraphy and structure of the Florida peninsula is that differential subsidence, sedimentation, and erosion have created the dip patterns associated with the Ocala platform (References 2.5.1-375, 2.5.1-376, and 2.5.1-377). Fracturing of the Floridian rocks is likely the result of tensional stresses caused by the differential subsidence of the Florida platform. Another factor that may be affecting the development of the Ocala platform and occurrence of fractures is the concept of isostatic readjustment of the crust related to dissolution of carbonate and associated reduction of the weight of the crust. This effect also could lead to broad uplift. (Reference 2.5.1-375)

Okeechobee Basin. The Okeechobee basin encompasses most of southern Florida. It is an area where the strata generally gently dip to the south and southeast. Within the basin there have been postulated episodes of faulting and folding. (Reference 2.5.1-261)

Osceola Low. The Osceola low, which was originally identified as a fault-bounded low with as much as 106 m (350 ft.) of Miocene sediments, does not appear to be associated with a discrete fault, but rather is more likely a possible flexure or zone of displacement (east side up). The Osceola low trends north-south to northeast-southeast and appears to be the site of increased frequency of karst features developed in the Ocala Group limestones. (Reference 2.5.1-261)

Sanford High. The Sanford high is another positive feature in the northern half of peninsular Florida that was originally considered by Vernon (Reference 2.5.1-262) to be a pre-Miocene structure. The Hawthorn Group and the Ocala Group are missing from the crest of the Sanford high, presumably due to erosion. The Avon Park Formation lies directly below post-Hawthorn sediments. A structural high offshore that was identified using high-resolution seismic reflection profiling may be an offshore extension of the Sanford high. Deformation associated with the offshore high ended prior to Pliocene time. (Reference 2.5.1-261)

St. Johns Platform. The St. Johns platform, which extends north from the Sanford high, is a low, broad ridge or platform expressed on the erosional surface of the Ocala Group. This platform plunges gently to the north-northwest toward the Jacksonville basin. (Reference 2.5.1-261)

Suwannee Strait. A negative feature in southeastern Georgia, just north of the Peninsular arch, has been variously named the Suwannee Strait, channel, or saddle. The feature is expressed as a closed depression on the top of Paleocene rocks, but the absence of such a depression in the top of rocks of lower Eocene age or younger shows that the Suwannee Strait ceased to be an actively subsiding basin during the Early Eocene. (Reference 2.5.1-240)

Faults and Fault Systems. Cenozoic faults have been postulated by numerous authors in various parts of the study region as shown on Figure 2.5.1-223. These faults generally have been inferred from apparent displacements based on limited outcrop and subsurface data from widely spaced boreholes and wells. Information on the specific faults is presented below.

- **Vernon (1951).** Located approximately 2.15 km (1.3 mi.) southeast of the LNP site at closest distance are seven faults along the Levy-Citrus County line and into Marion County that were identified by Vernon (Reference 2.5.1-262). Vernon claimed to be able to identify all of the faults on aerial photographs as continuous trough-like depressions and ridges marked by significant vegetation changes and soil colorations. The faults are depicted in plan view as straight and continuous lines. Vertical displacements on individual faults range from 6 m (20 ft.) to 49 m (160 ft.) as illustrated on a cross section across the zone of faults. The displacements are inferred from stratigraphic correlations based on widely separated borehole and bedrock outcrops. The faults were not directly observed in outcrop due to the limited bedrock exposure. Based on the straight course of the faults across a very uneven topography, Vernon suggested that the fault planes were very steeply inclined. Vernon inferred the faults to be normal dip-slip shear faults bounding graben depressions and horst ridges that flattened the crest of the Ocala Uplift). (Reference 2.5.1-262) However, from the limited core hole data available, Vernon stated that the dip of the fault planes could not be determined, nor was it possible to estimate the extent of faulting at depth. (Reference 2.5.1-262) FSAR Subsection 2.5.1.2.4 provides additional discussion of the structures mapped by Vernon within the site vicinity.

- **Carr and Alverson (1959).** Located approximately 63 km (39 mi.) southeast from the LNP site in northern Polk County is a structural break that was mapped by Carr and Alverson (Reference 2.5.1-263) and interpreted to be a fault. The strike of the displacement is shown as N40 W, with the upthrown side to the northeast. The structure is inferred to affect rocks up through the Suwannee (Lower Oligocene) and, in some areas, the Tampa (Lower Miocene) and the Hawthorne (Miocene) also may be slightly displaced. To the east of the fault line in Polk County, the Suwannee Limestone, according to well logs, is missing and Miocene rocks rest upon the Ocala Limestone. (Reference 2.5.1-263)

- **Pride et al (1966).** Located approximately 89 km (55 mi.) southeast of the LNP site at closest distance is an area of faulting along the Lake Whales Ridge area identified by Pride et al. (Reference 2.5.1-264). The faults trend approximately northwest. The faults likely originated in post-Oligocene time and subsequent movement along the fault zones may have occurred over a long period of time. The later movements probably are associated primarily with subsidence and sinkhole collapse along the solution-widened zones. (Reference 2.5.1-264)

- **Sproul et al (1972).** Located approximately 285 km (177 mi.) south of the LNP site at closest distance is a series of vertical offsets from gamma ray logs that were interpreted by Sproul et al. (Reference 2.5.1-265) as being caused by a series of faults. It was observed that the vertical displacement of comparable beds ranged from about 15 m (50 ft.) to 33 m (110 ft.). The depth to which the faults extended was not determined, but it was speculated that the faults extend at least through the Ocala Group, and probably deeper. It was inferred from the data set used that the displacement along the faults occurred prior to deposition of the upper part of the Hawthorn Formation. (Reference 2.5.1-265)

- **Miller (1986).** Several faults and fault systems have been published by Miller. (Reference 2.5.1-240) The locations of the faults are shown on Figure 2.5.1-223, and the ages of the rocks affected by these faults are shown on Figure 2.5.1-224. The closest fault cited by Miller is located approximately 99 km (61 mi.) southeast of the LNP site. In western Alabama, north-trending faults bound the Mobile graben — a negative feature that shows much vertical displacement. The faults to the north of the Mobile graben are part of the Gilbertown-Pickens-Pollard fault zone, which is characterized by a series of both isolated and connected grabens. These faults affect rocks from the Paleocene up through the Miocene. In central Georgia, the northeast-trending faults are the boundary faults for a series of small grabens that collectively are known as the Gulf Trough (refer to the beginning of this subsection, FSAR Subsection 2.5.1.1.4.3.4, for additional details on the Gulf Trough). These faults affect rocks from the Middle Eocene up through the Oligocene. Miller shows several faults shown along Florida's eastern coast that are of limited extent and generally show little vertical displacement. In the Middle Eocene section, several of these faults exhibit only small to intermediate throw, and appear to die out downward, also within the Middle Eocene. (There are two apparently younger faults along Florida's east coast. One fault is in Palm Beach County, southern Florida, which cuts rocks at least as old as Paleocene and continues up through the Miocene. The other fault is slightly north of the Palm Beach fault; it starts in Indian River County, continues southeast through Martin County, cuts rocks as old as Middle Eocene, and continues up through the Miocene. Finally, there are a number of small- to medium-sized faults in central and northern peninsular Florida that first occur in the Late Eocene and appear to be shallow features that die out with depth. (Reference 2.5.1-240)

- **Hutchinson (1991).** Located approximately 222 km (137 mi.) south of the LNP site is a fault that Hutchinson (Reference 2.5.1-266) identified, based on a geophysical log correlation of the dolomitic limestone interval near the base of the Suwannee Limestone (Lower Oligocene). Hutchinson interpreted a 30 m (100 ft.) offset between two wells approximately 1220 m (4000 ft.) apart and mapped the approximate location of the fault by tracing this offset in several wells throughout the area. The fault strikes approximately east-west. (Reference 2.5.1-266)

- **Winston (1996).** Located approximately 200 km (124 mi.) south of the LNP site are 12 faults identified by Winston (Reference 2.5.1-267) based on well log information. The North Port fault was cited as cutting the basal Delray Dolomite and the upper Cedar Keys (mid-Paleocene) and trends northwest. The Venice fault cuts units that are Eocene in age. The Northeast McGregor fault is thought to have moved in the Late Eocene and the McGregor fault system displays structural offsets in Neogene beds. The Estero fault system was based on correlation of marker beds in the lower Tampa-Hawthorn, with apparent dips exceeding 4 degrees in less than 152 m (500 ft.), strongly suggesting faulting. In two other locations, excessively thin Delray dolomite suggests the possibility of faulting and in two different locations, thick wall collapse suggest the presence of nearby faulting. The Pepper Hammock fault was based on the absence of 76 m (250 ft.) of missing section in the Lower Eocene. Overall, the 12 suspected faults published by Winston (Reference 2.5.1-267) are based on sparse well control data. Winston also concluded that due to the sparse well control data, there may be many more faults present in the larger areas having no control. (Reference 2.5.1-267)

The existence of many of these structures is controversial and not well supported by available data. Many of the postulated faults have been identified as offsets in the top of the Ocala Limestone (Late Eocene), a karstified, unconformable surface that may have 50 m (164 ft.) or more of relief. (Reference 2.5.1-235) Scott (Reference 2.5.1-268) further summarized that many of the faults in Florida are based on limited evidence from a small selection of wells and that very few of the proposed faults have actually been tested sufficiently to prove or disprove their existence. The main problems inherent to fault identification in Florida include (1) the existence of a buried, highly irregular karst erosional surfaces, (2) other buried topography, and (3) extremely variable facies and rapid facies change. (Reference 2.5.1-268) According to Miller (Reference 2.5.1-240), there are no faults in Florida having post-Miocene movement. Postulated faults in the site vicinity are described in more detail in FSAR Subsection 2.5.3.

2.5.1.1.4.3.5 Quaternary Tectonic Structures

Because the Florida platform represents long-term carbonate sedimentation on a passive margin, late Quaternary deposits on the Florida Keys have not experienced significant uplift, subsidence, or tectonic deformation. (Reference 2.5.1-237)

The U.S. Geological Survey maintains a nationwide database on features that have known or suggested Quaternary tectonic faulting. Geologic information on Quaternary faults, folds, and earthquake-induced liquefaction in the eastern United States was compiled by Crone and Wheeler. (Reference 2.5.1-269) An update containing new assessments was published by Wheeler. (Reference 2.5.1-270) The features within the site region are categorized into three classes (A, B, or C)[b] based on information about the features. No Class A, B, or C features are identified within the LNP site region

Postulated tectonic faults within 40 km (25 mi.) radius (site vicinity) of the LNP are mapped by Vernon (Reference 2.5.1-262). As noted in FSAR Subsection 2.5.3, there is no geologic or geomorphic evidence to indicate that the postulated faults mapped within the site vicinity and area exist or have been active in the Quaternary.

2.5.1.1.4.4 Significant Seismic Sources at a Distance Greater Than 320 Km (200 Mi.)

The 1886 Charleston, South Carolina, earthquake was the largest earthquake occurring in historical time in the eastern United States. The event produced Modified Mercalli Intensity (MMI) X shaking in the epicentral area (Reference 2.5.1-297). 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

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b. Crone and Wheeler (Reference 2.5.1-269) define Class A features as those for which geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin. Class B features are those for which the fault may not extend deeply enough to be a potential source of significant earthquakes, or for which the currently available geologic evidence is not definitive enough to assign the feature to Class C or to Class A. Class C features are those for which geologic evidence is insufficient to demonstrate the existence of a tectonic fault, Quaternary slip, or deformation associated with the feature.

Charleston, but there remains uncertainty in the causative structure on which the earthquake occurred.

The EPRI-SOG evaluation indicated that the seismic sources in the Charleston, South Carolina, region are significant contributors to the hazard at the LNP site. (Reference 2.5.1-202) Several investigations that post-date the EPRI-SOG evaluation indicate that parameters related to location, maximum magnitude, and recurrence of possible seismic sources in the Charleston region should be updated. (Reference 2.5.1-202) In support of the Southern Nuclear Company Vogtle Early Site Permit (ESP) Application, a thorough review and analysis of the new data were completed (Reference 2.5.1-272). The UCSS model for the Vogtle ESP Application, which was adopted for this study, is further discussed in FSAR Subsection 2.5.2.

Recent published and unpublished studies for information on the potential location and extent of the Charleston source and the maximum earthquake and recurrence of repeated large-magnitude events expected to occur on it are described in the following subsections.

2.5.1.1.4.4.1 Location and Geometry

The source of the earthquake is inferred based on the geology, geomorphology, and instrumental seismicity of the region. Local and regional tectonic features that may be related to the Charleston seismic source are shown on Figures 2.5.1-225 and 2.5.1-226. Features are differentiated to show both pre- and post-1986 EPRI information. The description of tectonic features and zones of concentrated seismicity within the Charleston meizoseismal region is followed by discussion of more indirect evidence (association with Mesozoic rift basins, paleoliquefaction, and intensity data) that have been used to evaluate the location of the source of repeated large magnitude earthquakes that have occurred in the Charleston region.

<u>Tectonic Features</u>

Local Charleston area tectonic features and seismicity zones are listed in Table 2.5.1-201 and described below. Recent post-EPRI studies that have identified tectonic features and postulated tectonic features in the 1886 Charleston meizoseismal area include those by Marple and Talwani (References 2.5.1-273, 2.5.1-274, and 2.5.1-275); Weems et al. (Reference 2.5.1-276); Weems and Lewis (Reference 2.5.1-277); Talwani and Katuna (Reference 2.5.1-278); and Talwani and Durá-Gómez (References 2.5.1-337, 2.5.1-338, and 2.5.1-339).

Adams Run Fault. Weems and Lewis (Reference 2.5.1-277) postulated the existence of the Adams Run fault based on interpretation of microseismicity and borehole data (Figure 2.5.1-225 and Figure 2.5.1-228). Weems and Lewis's interpretation of borehole data suggests the presence of areas of Cenozoic uplift and subsidence separated by the inferred Adams Run fault (Figure 2.5.1-228). Based on review and analysis of these data the SNC Vogtle ESP Application

(Reference 2.5.1-264) presented the following conclusions: (1) the pattern of uplift and subsidence does not appear to persist through time (i.e., successive stratigraphic layers) in the same locations, and (2) the intervening structural lows between the proposed uplifts are highly suggestive of erosion along ancient river channels. Independent review of these data for the LNP study supports the conclusions presented in the SNC Vogtle ESP application.

The SNC Vogtle ESP application also stated that there is no geomorphic evidence for the existence of the Adams Run fault, and 3-D analysis of microseismicity in the vicinity of the proposed Adams Run fault does not clearly define a discrete structure (Figure 2.5.1-229). Talwani (Reference 2.5.1-295) associates seismicity observed in the Adams Run seismic zone with the Woodstock fault. The Adams Run fault is not shown as a structure on the current seismotectonic framework map of the Charleston area by Durá-Gómez and Talwani (Reference 2.5.1-339; Figure 2.5.1-259).

Ashley River Fault. Talwani (Reference 2.5.1-295), identifies the Ashley River fault in the meizoseismal area of the 1886 Charleston earthquake on the basis of a northwest-oriented linear zone of seismicity located about 9.6 km (6 mi.) west of Woodstock, South Carolina. The postulated fault was judged to be a southwest-side-up reverse fault that appeared to offset the north-northeast-striking Woodstock fault about 4.8 to 6.4 km (3 to 4 mi.) to the northwest near Summerville (References 2.5.1-295, 2.5.1-277, and 2.5.1-340). The Ashley River fault subsequently was subdivided into two structures: (1) the seismogenic Sawmill Branch fault striking N30°W with a strong reverse component and dip of about 70 degrees to the southwest, and (2) the approximately 50- to 60-degree west-striking, essentially aseismic Ashley River fault between Middleton Place and Magnolia Plantation, for which the name Ashley River fault was retained (Figure 2.5.1-259). (Reference 2.5.1-337)

Charleston Fault. Lennon (Reference 2.5.1-341) proposes the Charleston fault on the basis of geologic map relations and subsurface borehole data. Weems and Lewis (Reference 2.5.1-277) suggest that the Charleston fault is a major high-angle reverse fault that has been active at least intermittently in Holocene to modern times. It was noted in the SNC Vogtle ESP that the Charleston fault as mapped by Weems and Lewis has no clear geomorphic expression, nor is it clearly defined by microseismicity (Reference 2.5.1-272; Figure 2.5.1-229). Talwani and Durá-Gómez (Reference 2.5.1-337) state that the interpretation of Weems and Lewis is inconsistent with fault kinematics and that a more plausible explanation is that the Charleston fault is not a steep dipping (and northeast-dipping) fault, but rather, its surface projection is approximately 7 km (4.4 mi.) to the northeast along the northwest axis of the Mt. Holly dome with the southwest side upthrown (Figure 2.5.1-259). Based on interpretation of relocated earthquake hypocenters, Durá-Gómez and Talwani (Reference 2.5.1-339) associate some of the current seismicity occurring in the antidilational compressional left step at Middleton Place with the Charleston fault and infer a southwest dip of approximately 40 degrees for the fault.

Cooke Fault. Behrendt et al. (Reference 2.5.1-342) and Hamilton et al. (Reference 2.5.1-343) identify the Cooke fault based on seismic reflection profiles in the meizoseismal area of the 1886 Charleston earthquake. This east-northeast-striking, steeply northwest-dipping fault has a total length of about 9.6 km (6 mi.) (References 2.5.1-273 and 2.5.1-274). Marple and Talwani (References 2.5.1-273 and 2.5.1-274) reinterpret these data to suggest that the Cooke fault may be part of a longer, more northerly striking fault (i.e., the zone of river anomalies [ZRA] of Marple and Talwani [Reference 2.5.1-273] and the East Coast fault system [ECFS] of Marple and Talwani [Reference 2.5.1-274]). Crone and Wheeler (Reference 2.5.1-269) classify the Cooke fault as a Class C feature based on lack of evidence for faulting younger than Eocene.

Drayton Fault. The Drayton fault is imaged on onshore seismic reflection lines and was known to the six EPRI ESTs at the time of EPRI (Reference 2.5.1-345). The Drayton fault is mapped as an 8.8-km (5.5-mi.) long, apparently northeast-trending, high-angle, reverse fault in the meizoseismal area of the 1886 Charleston earthquake (Reference 2.5.1-343) (Figures 2.5.1-225 and 2.5.1-229). The Drayton fault terminates upward at approximately 762 m (2500 ft.) bgs within a Jurassic-age basalt layer (Reference 2.5.1-343), precluding significant Cenozoic slip on this fault. (Reference 2.5.1-272)

East Coast Fault System. A postulated north-northeast/south-southwest-trending buried fault system in the Coastal Plain of the Carolinas and Virginia, named the ECFS, was identified by Marple and Talwani (Reference 2.5.1-274). Based on Marple and Talwani's geomorphic analyses of Coastal Plain rivers, three nearly collinear, approximately 200-km- (125-mi.-) long segments (ECFS-S, ECFS-C, and ECFS-N) were differentiated that were initially referred to as the southern, central, and northern zones of river anomalies (ZRA-S, ZRA-C, and ZRA-N) (Figure 2.5.1-261). The southern segment, which is located primarily in South Carolina, was suggested as a possible source of the 1886 Charleston earthquake; the central segment is located primarily in North Carolina; and the northern segment extends from northeastern North Carolina through Virginia (Figure 2.5.1-226 and Figure 2.5.1-227). Identification of the postulated fault system is based on the alignment of geomorphic changes along streams, areas of uplift, and local evidence of faulting. Marple and Talwani concluded that (1) the ZRAs were produced by gentle late Quaternary uplift along an approximately 600-km- (370-mi.-) long-buried fault system and (2) because most of the river anomalies occur in unconsolidated floodplain sediments of upper Pleistocene (<130 ka) or younger age, deformation occurred during this period and may be ongoing (Reference 2.5.1-274).

The conclusions reached by Marple and Talwani suggest that the ECFS should be considered as a potential capable tectonic source (Reference 2.5.1-274). Seismic hazard studies conducted in support of the National Seismic Hazard Mapping Program as well as other ESP and COL applications have assessed various segments of the ECFS as a potential seismic source. The general conclusions reached by these studies are as follows:

- National Seismic Hazard Mapping Program (2002 and 2008) — One of the two alternative, equally weighted areal source zones that are used to account for the uncertainty in the location of the source of future earthquakes in the Charleston, South Carolina, region is a geographically narrow zone that follows the Woodstock lineament and an area of river anomalies (References 2.5.1-350 and 2.5.1-346). This zone correlates to the southern 100 km (62 mi.) of the ECFS-S segment. There is no discussion or documentation of the reasons for limiting the source zone to the southern half of the ECFS-S segment. The northern part of the ECFS-S, as well as the ECFS-C and ECFS-N, were not specifically modeled as fault sources.

- Updated Assessment of Quaternary Tectonic Faulting (2005) — In an update to the U.S. Geological Survey (USGS) compilation of known or suggested Quaternary tectonic faulting in the CEUS, Wheeler (Reference 2.5.1-270) noted that evidence for the southern section is strongest, with evidence becoming successively weaker northward (Reference 2.5.1-270). Wheeler evaluated the evidence for the East Coast fault system, noting that the southern segment is surrounded by sites at which prehistoric paleoliquefaction features document the occurrence of large earthquakes (Reference 2.5.1-270). Wheeler states that the evidence for recent uplift and possible buried faulting along the southern segment of the fault system is good; however, there is no demonstration of sudden uplift anywhere along the fault system. The 1886 and prehistoric liquefying earthquakes in South Carolina demonstrate the occurrence of repeated Quaternary tectonic faulting, but the link between those events and the postulated East Coast fault system remains speculative. Accordingly, the postulated East Coast fault system is assigned to Class C. (Reference 2.5.1-270)

- Dominion North Anna ESP Application — A comprehensive review of the reported evidence for the ECFS-N was completed for the Dominion North Anna ESP Application (Reference 2.5.1-378). From this study, which included geomorphic analyses and aerial reconnaissance in addition to the critical evaluation of the evidence cited by Marple and Talwani (Reference 2.5.1-274), it was concluded that the ECFS-N probably does not exist, or has a very low probability of activity if it does exist. The probabilities of existence and activity were assigned low weights because the existence of the fault is not well documented and was judged to be highly uncertain, and because there is no direct geologic, geomorphic, or seismologic evidence that the fault exists as a tectonic feature or is active, if it does exist. In a sensitivity analysis performed to evaluate the fault's potential contribution to hazard at the North Anna ESP site, the ECFS-N fault was assumed to have a probability of existence of 0.1 and a probability of activity (given existence) of 0.1.

- Dominion North Anna Final Safety Evaluation Report — The NRC staff in its review of the North Anna ESP Application concluded that the geologic, seismologic, and geomorphic evidence for the ECFS-N presented by Marple and Talwani (2000) is questionable and that the majority of data presented apply only to the southern and central segments of the ECFS. NRC staff concluded that although the evidence for the ECFS-N is low, it should be included as a possible contributor to the seismic hazard for the North Anna ESP site, and that a 10 percent probability of existence is an acceptable value. (Reference 2.5.1-351)

- SNC Vogtle ESP Application — The southern segment of the ECFS was evaluated and included as a possible source of repeated large-magnitude earthquakes in the updated Charleston seismic source model developed for this application. The evaluation for the Vogtle ESP (Reference 2.5.1-272) judged the ECFS-S to have a relatively low likelihood of producing Charleston-type earthquakes because there is not sufficient geologic evidence to demonstrate tectonic faulting or Quaternary slip associated with the ECFS-S, and many of the river anomalies may be due to nontectonic processes. The possibility that the ECFS-S in its entirety is the source of repeated large-magnitude Charleston earthquakes, therefore, was given a low weight (0.1) in the UCSS model. The southernmost portion of the ECFS-S, however, which lies in the meizoseismal region of the Charleston earthquake, also is included in the Charleston-area faults source zone (Zone A) (given a weight of 0.7) and the larger onshore portion of the Coastal Zone (Zone B) (given a weight of 0.2) (See FSAR Subsection 2.5.2.4.1.1 for further description of these source zones).

- Duke Energy Carolinas, LLC William States Lee III Nuclear Station Units 1 and 2 COL Application — Following the characterization of the updated Charleston seismic source developed by SNC Vogtle ESP Application, the Duke Energy William States Lee COL Application also considers the ECFS-S as a possible source of repeated large-magnitude earthquakes in the Charleston region. Based on examination of gravity and aeromagnetic maps along the northern part of the ECFS-S segment, it also was concluded that if the ECFS exists as mapped, then it has not accumulated sufficient displacement to juxtapose rocks of varying magnetic susceptibility or density, and thus does not produce an observable magnetic or gravity anomaly at the scale of maps used for the evaluation. (Reference 2.5.1-352)

- Progress Energy, Shearon Harris Nuclear Power Plant (HAR) Units 2 and 3 COL Application — In the updated PSHA for the HAR COLA, an evaluation of the evidence for the postulated ECFS was undertaken to assess whether this postulated fault system qualifies as a capable tectonic source or a new seismic source that should be included in an updated PSHA. The criteria used in the HAR study to assess fault capability were those that were judged to be the most important in the EPRI-SOG evaluations, including spatial association with instrumental and historical seismicity or paleoseismic events, geometry and sense of slip relative to the present stress regime, deep crustal expression, and evidence for brittle slip on the feature. These criteria were applied to the postulated ECFS-C segment, which lies closest to the HAR site. Based on this assessment, the ECFS-C segment, like the ECFS-N segment, was assigned a low probability (0.1) that the source exists and a probability of 0.1 that it is active if it exists. Both the ECFS-C and the ECFS-N were included as possible fault sources in the updated PSHA for the HAR site. The postulated ECFS-S was included as a possible alternative fault source for the UCSS following the characterization that was developed by SNC (Subection 2.5.2.4.1.1).

The general consensus from these studies is that only the southern segment of the ECFS warrants consideration as a source for repeated large magnitude Charleston earthquakes. Marple and Talwani initially identified and proposed the southern segment, located in South Carolina, as a possible source of the 1886 Charleston earthquake (Reference 2.5.1-273). Marple and Talwani (Reference 2.5.1-274) provided evidence for the existence of the southern section of the ECFS, including seismic reflection data, linear aeromagnetic anomalies, exposed Pliocene-Pleistocene faults, local breccias, and upwarped strata. Since most of the geomorphic anomalies associated with the southern section of the ECFS are in Late Pleistocene sediments, Marple and Talwani (Reference 2.5.1-274) speculated that the fault was active 130 – 10 ka, and perhaps remains active. Wildermuth and Talwani (Reference 2.5.1-279) subsequently used gravity and topographic data to postulate the existence of a right-stepping pull-apart basin between the southern and central sections of the ECFS. Existence of the pull-apart basin suggests a component of right-lateral slip on the northeast-trending ECFS, which is consistent with the inferred sense of slip based on the orientation of the fault in the regional stress field.

The approximately 30–km- (19-mi.-) long Woodstock fault, which is inferred to be a reactivated fault associated with the eastern margin of a Mesozoic basin, lies within the southern part of the ECFS-S. The apparent coincidence of the Woodstock fault with higher topography and the Summerville scarp has been cited as evidence of Quaternary reactivation of a fault associated with the ECFS-S.

Farther to the north, Marple and Talwani present evidence for faulting or folding of Upper Cretaceous units, including structure contours on a Black Creek clay horizon (interpreted from resistivity well logs), and interpretations of possible offset reflectors in seismic reflection profiles at two locations between the Santee and Lynches Rivers. In the northernmost seismic line located between Black Creek and the Lynches River, the interpreted faulting is shown to extend upward into the lower Coastal Plain units. North of the Lynches River, Marple and Talwani (Reference 2.5.1-274) show no mapped faults that coincide with ECFS-S (Figure 2.5.1-261). A strand of the Eastern Piedmont fault system crosses the northern end of the postulated ECFS-S, but there are no nearby faults that parallel or coincide with the postulated ECFS-S.

The Duke Energy William States III Lee FSAR (Reference 2.5.1-352) states that the northern part of the mapped trace of the southern segment of the ECFS is not expressed in the gravity field and cuts across anomalies with wavelengths on the order of tens of miles without noticeable perturbation. This implies that the southern segment of the ECFS, if present, has not accumulated sufficient displacement to systematically juxtapose rocks of differing density, and thus produce an observable gravity anomaly. The magnetic data do not show evidence for any Cenozoic structures in the Duke Energy Lee site region and generally are not of sufficient resolution to identify or map discrete faults such as border faults along the Triassic basins. In particular, the southern segment of the ECFS has no expression in the magnetic field and cuts across anomalies with wavelengths on the order of tens of miles without noticeably perturbing or affecting them. If the ECFS exists as mapped, then it has not accumulated sufficient displacement to juxtapose rocks of varying magnetic susceptibility, and thus does not produce an observable magnetic anomaly.

Based on independent evaluation of the geomorphic, seismic reflection, and seismicity data, the SNC Vogtle application judged the existence and activity of the ECFS to be low to moderate for the following reasons:

- The geomorphic "anomalies" have credible nontectonic (i.e., fluvial geomorphic) explanations.

- A 3-D (three-dimensional) analysis of microseismicity in the vicinity of the ECFS completed for that study did not clearly define a discrete structure

- Available seismic reflection data do not unambiguously delineate a through-going structure in the vicinity of the ECFS.

Gants Fault. The Gants fault is imaged on onshore seismic reflection lines and was known to the six EPRI ESTs at the time of the 1986 EPRI study (Reference 2.5.1-345) as a possible Cenozoic-active fault. The Gants fault is mapped as an 8.8-km (5.5-mi.) long, apparently northeast-trending, high-angle, reverse fault in the meizoseismal area of the 1886 Charleston earthquake (References 2.5.1-342 and 2.5.1-343) (Figures 2.5.1-225 and 2.5.1-229). The Gants fault displaces vertically a Jurassic-age basalt layer by about 45.7 m (150 ft.) at approximately 762 m (2500 ft.) bgs (Reference 2.5.1-343). Overlying Cretaceous and Cenozoic beds show apparent decreasing displacement with decreasing depth (Reference 2.5.1-343), indicating likely Cenozoic activity, but with decreasing displacement on the Gants fault during the Cenozoic. (Reference 2.5.1-272)

Helena Banks Fault Zone. The Helena Banks fault zone is clearly imaged on seismic reflection lines offshore of South Carolina (Reference 2.5.1-280, Reference 2.5.1-281) and was known to the six EPRI ESTs at the time of the 1986 EPRI study as a possible Cenozoic-active fault zone. Some ESTs recognized the offshore fault zone as a candidate tectonic feature for producing the 1886 event and included it in their Charleston seismic source zones. However, since 1986 three additional sources of information have become available, as outlined in the SNC Vogtle ESP study (Reference 2.5.1-272):

- In 2002, two magnitude $m_b \geq 3.5$ earthquakes (m_b 3.5 and 4.4) occurred offshore of South Carolina in the vicinity of the Helena Banks fault zone in an area previously devoid of seismicity (Figure 2.5.1-225 and Figure 2.5.1-226).

- Bakun and Hopper (Reference 2.5.1-282) reinterpreted intensity data from the 1886 Charleston earthquake and show that the calculated intensity center is located over 150 km (93 mi.) offshore from Charleston, suggesting that the source of the 1886 earthquake may lie offshore of South Carolina. Bakun and Hopper (Reference 2.5.1-282) ultimately conclude, however, that the epicentral location most likely lies onshore in the Middleton Place – Summerville area (Figure 2.5.1-226 and Figure 2.5.1-230) based on the concentrated seismicity in this area.

- Crone and Wheeler (Reference 2.5.1-269) described the Helena Banks fault zone as a potential Quaternary tectonic feature, but classified the fault zone as a Class C feature that lacks sufficient evidence to demonstrate Quaternary activity. There is no reported evidence for slip younger than Miocene on the Helena Banks fault zone. The youngest deformation could be as old as Miocene, depending on whether the deformed Miocene clay dates from the early or late Miocene. Accordingly, the Helena Banks fault zone is assigned to Class C for lack of evidence of faulting younger than Miocene.

In the Vogtle ESP Application (Reference 2.5.1-272), a high confidence was assigned to the existence of this fault zone, and a low to moderate confidence was assigned to the possibility that the fault may be active and the source of the 1886 earthquake. Seismic reflection data clearly show the existence of the Helena Banks fault zone (as opposed to a deep-seated landslide) extending to a depth of greater than 1 km (0.6 mi.). Furthermore, the occurrence of 2002 earthquakes and the location of the Bakun and Hopper (Reference 2.5.1-282) intensity center offshore suggest, at a low probability, that the fault zone could be considered a potentially active fault. If the Helena Banks fault zone is an active source, its length and orientation could possibly explain the distribution of paleoliquefaction features along the South Carolina coast. Therefore, the Helena Banks fault zone was included as a possible source for the 1886 Charleston earthquake in the SNC Vogtle ESP update of the Charleston seismic source geometry in order to capture the uncertainty associated with this fault.

The current USGS seismic source characterization model for the National Seismic Hazard Mapping Project also includes a revised geometry for the large Charleston zone, extending it farther offshore to include the Helena Banks fault (Reference 2.5.1-346).

Sawmill Branch Fault. The Sawmill Branch fault, which was speculated to have experienced surface rupture in the 1886 earthquake, was initially differentiated from the southeastern part of the Ashley River fault based on analysis microseismicity (Reference 2.5.1-290). According to Talwani and Katuna (Reference 2.5.1-278), this approximately 5-km- (3-mi.-) long, northwest-trending fault, which is a segment of the larger Ashley River fault, offsets the Woodstock fault in a left-lateral sense (Figure 2.5.1-225). Earthquake damage at three localities (along the banks of the Ashley River, small, discontinuous cracks in a tomb that dates to 1671 A.D., and displacements (less than 10 cm [4 in.] in the walls of colonial Fort Dorchester) was used to infer that surface rupture occurred in 1886.

Field investigations and a review of the postulated evidence for surface rupture in 1886 and seismicity were conducted in support of the SNC Vogtle ESP Application (Reference 2.5.1-272). The general conclusion from the SNC study was that the features are almost certainly the product of shaking effects as opposed to fault rupture (Reference 2.5.1-272).

Durá-Gómez and Talwani (Reference 2.5.1-339) present an updated seismogenic framework for the Charleston area based on relocated earthquake hypocenters. Their analysis of the recorded seismicity between 1974 and 2004 suggests that most of the seismicity within an approximately 6-km- (3.6-mi.-) long antidilational compressional left step in the right-lateral-oblique Woodstock fault is occurring on the approximately 3-km- (1.8-mi.-) wide Sawmill Branch fault zone. The inferred dip direction of the Sawmill Branch fault zone to the northeast is opposite to the previously interpreted southwest dip or the inferred dip of the essentially aseismic Ashley River fault. Fault plane solutions suggest that the Sawmill Branch fault behaves as a left-lateral fracture but displays a significant reverse component (Reference 2.5.1-339).

Woodstock Fault. Talwani (Reference 2.5.1-295) identifies the Woodstock fault, a postulated north-northeast-trending dextral strike-slip fault, on the basis of a linear zone of seismicity located approximately 9.6 km (0.6 mi.) west of Woodstock, South Carolina, in the meizoseismal area of the 1886 Charleston earthquake.

In a recent revised seismotectonic framework for the Charleston earthquakes, the Woodstock fault is defined by Talwani and Durá-Gómez (Reference 2.5.1-338) as an approximately 50-km- (31-mi.-) long, approximately N30°E striking, northwest-dipping fault characterized by right-lateral-oblique strike-slip motion, with an associated approximately 6-km- (3.7-mi.-) long antidilational compressional left step near Middleton Place that divides the fault zone into Woodstock North and Woodstock South faults (Figure 2.5.1-259). The Woodstock North fault lies along the southeast boundary of a buried Triassic basin, and the current seismicity is inferred to be due to its reactivation (Reference 2.5.1-337). Based on a review of available geomorphological, geodetic, shallow stratigraphic, seismic reflection, refraction, and potential field data, Talwani and Durá-Gómez (Reference 2.5.1-337) conclude that the ongoing tectonic activity has resulted in breaking the overlying basalt along the Woodstock fault and warping the overlying sediments. The current seismicity in the Charleston area is reportedly due to the reactivation of the Woodstock North fault (Reference 2.5.1-337). Talwani and Durá-Gómez (Reference 2.5.1-337) integrated their revised seismogenic framework with the observed effects of the 1886 earthquake and conclude that the most intense shaking occurred on the Woodstock fault (North and South) and the northwest-southeast-trending Charleston and Lincolnville faults, and comparatively less on the Sawmill Branch fault. That comparison suggests that most of the built-up stress along the Woodstock, Charleston, and Lincolnville faults was released in 1886, leaving only the Sawmill Branch fault currently seismic. Durá-Gómez and Talwani (Reference 2.5.1-339) infer that the main shock of the 1886 Charleston earthquake was probably associated with the N30°E oriented oblique right-lateral strike-slip Woodstock fault.

Chapman and Beale (Reference 2.5.1-344) present additional evidence for reactivation of faulting near the intersection of the inferred Woodstock and Sawmill Branch faults. Reprocessing of seismic reflection profile VT-3b, originally collected in 1981, provides an improved image of the shallow crust in the epicentral area of the 1886 Charleston earthquake. There is clear evidence in the reprocessed line of a down-to-the-east steeply dipping fault with approximately 200 m (61 m) of vertical offset that displaces lower Mesozoic sedimentary and volcanic rocks. The overlying Cretaceous and Tertiary sedimentary section shows approximately 10 m (33 ft.) of reverse up-to-the-east displacement that can be resolved to within 100 m (30 m) of the ground surface. Two other near-vertical faults with down-to-the-east offset of lower Mesozoic units are inferred to be located to the northwest of the major fault. The faulting is considered to be a likely candidate structure for the 1886 Charleston earthquake. The existing reflection data (which includes several other seismic lines), although suggestive of extensions of the faulting to the north and south, do not have sufficient resolution to constrain the strike of the faulting imaged on VT-3b.

Summerville Fault. Weems et al. (Reference 2.5.1-276) postulated the existence of the Summerville fault on the basis of microseismicity (Figure 2.5.1-225). Based on a review and analysis of these data for the SNC Vogtle ESP Application (Reference 2.5.1-272). it was concluded that there is no geomorphic or borehole evidence for the existence of the Summerville fault, and the 3-D analysis of microseismicity in the vicinity of the proposed Summerville fault does not clearly define a discrete structure (Figure 2.5.1-229).

<u>Association with Seismicity</u>

Middleton Place – Summerville Seismic Zone. The Middleton Place – Summerville seismic zone (MPSSZ) is an area of elevated microseismic activity located approximately 20 km (13 mi.) northwest of Charleston (References 2.5.1-292, 2.5.1-293, 2.5.1-294, and 2.5.1-278). Between 1980 and 1991, 58 events with duration magnitude (M_d) 0.8 – 3.3 were recorded in an 11- by 14-km^2 area, with hypocentral depths ranging from 2 to 11 km (1 to 7 mi.) (Reference 2.5.1-294). The elevated seismic activity of the MPSSZ has been attributed to stress concentrations associated with the intersection of the Ashley River and Woodstock faults (References 2.5.1-295, 2.5.1-294, 2.5.1-278, and 2.5.1-296). Persistent foreshock activity was reported in the MPSSZ area (Reference 2.5.1-283) and it has been speculated that the 1886 Charleston earthquake occurred within the MPSSZ (e.g., References 2.5.1-295, 2.5.1-292, and 2.5.1-282).

<u>Association with Mesozoic Basins</u>

Johnston et al. (Reference 2.5.1-245) evaluated the correlation of large-magnitude intraplate earthquakes to specific tectonic environments throughout the world. Johnston et al. concluded that large-magnitude earthquakes generally occur in tectonic environments characterized by Mesozoic and younger rifted crust. Schulte and Mooney (Reference 2.5.1-353) presented an updated global earthquake catalogue (**M** 4.5 or larger) for stable continental regions and reassessed the correlation of intraplate seismicity with ancient rifts on a global scale. The results of this study showed that over half (52 percent) of all events are associated with rifted crust. The largest events (**M** 7.0 or larger) have occurred predominantly within rifts (50 percent) and continental margins (43 percent).

The Charleston meizoseismal region occurs in a region of Mesozoic extended crust along the southeastern margin of the North American craton. (Reference 2.5.1-245) Several Mesozoic basins are defined in the region. The location, structural orientation (i.e., northeast-southwest), and spatial correlation of possible Mesozoic basins and structures was used by SNC in the Vogtle ESP assessment of the updated Charleston seismic source to characterize alternative models of the source zone geometry (Reference 2.5.1-272). The spatial correlation of the northern segment of the Woodstock fault to the southeast margin fault of the Mesozoic Jedberg basin shows reactivation as an oblique right-lateral-slip fault with up to the northwest displacement (Reference 2.5.1-337).

<u>Paleoliquefaction Features</u>

Based on the geographic and temporal distribution of paleoliquefaction features in coastal South Carolina, Talwani and Schaeffer (Reference 2.5.1-289) proposed two scenarios for the occurrence in time and space of Charleston-area earthquakes. In their first scenario, three seismic sources are inferred to occur within the Coastal Plain of South Carolina: a Charleston source that has produced earthquakes with magnitudes ≥ approximately 7, and a source in each of the Georgetown and Bluffton areas that have produced more moderate earthquakes with magnitudes approximately 6. In Talwani and Schaeffer's (Reference 2.5.1-289) second scenario, all events recorded in the paleoliquefaction record were centered at Charleston with magnitudes ≥ approximately 7.

<u>Intensity Data</u>

Intensity data for the 1886 Charleston earthquake reported by Dutton (Reference 2.5.1-283) and reinterpreted by Bollinger (Reference 2.5.1-291) indicate a meizoseismal area centered on Charleston (Figures 2.5.1-225 and 2.5.1-226). Bakun and Hopper (Reference 2.5.1-282) calculated an intensity center for the 1886 Charleston earthquake that is located offshore about 200 km (125 mi.) east of Charleston (Figure 2.5.1-230). The offshore location for the intensity center may be a function of the spatial distribution of the input data, all of which lie onshore (Reference 2.5.1-282). Bakun and Hopper's (Reference 2.5.1-282) preferred intensity center for the 1886 Charleston earthquake is onshore within the Middleton Place – Summerville seismic zone.

2.5.1.1.4.4.2 Maximum Magnitude

As outlined in the SNC Vogtle ESP Application (Reference 2.5.1-272), multiple methods and types of data have been used to characterize the maximum magnitude (M_{max}) of the Charleston seismic source. These approaches include using the worldwide data set to constrain the minimum and maximum range of maximum magnitude for regions of Mesozoic and younger extensional crust (Reference 2.5.1-245) and evaluating the size of the 1886 Charleston earthquake as a proxy for the maximum earthquake that may be produced by the Charleston seismic source (Table 2.5.1-202). The latter approach has used both intensity data (References 2.5.1-297 and 2.5.1-347) and the size and geographic distribution of the liquefaction fields (References 2.5.1-297, 2.5.1-348, 2.5.1-290, and 2.5.1-297) to estimate the magnitude of the 1886 event.

Because the causative fault for the 1886 event is unknown, the SNC Vogtle ESP Application (Reference 2.5.1-272) update of the Charleston seismic source model considered the 1886 earthquake magnitude and worldwide database more reliable than postulated fault dimensions to estimate maximum magnitude for the Charleston seismic source. Johnston et al. (Reference 2.5.1-245) compiled a worldwide database of earthquakes in stable continental regions (SCRs) to evaluate the correlation of large-magnitude SCR earthquakes to specific tectonic environments, if any. The database showed that the largest SCR earthquakes (>**M** 7) are confined to regions of Mesozoic and younger extended crust. The maximum observed magnitude for Mesozoic extended crust along passive cratonic margins similar to the southeastern United States is **M** 7.7 ± 0.2 (Reference 2.5.1-245). Based on an analysis of intensity data, Johnston et al. (Reference 2.5.1-295) estimated the 1886 Charleston earthquake to be **M** 7.56 ± 0.35. Using Bayesian statistics, Johnston et al. (Reference 2.5.1-245) indicated that the maximum magnitude for the Charleston seismic source should not be much larger than the 1886 event. This conclusion supports the idea that a maximum magnitude developed for the Charleston seismic source should be primarily based on the estimate of the size of the 1886 Charleston event. Martin and Clough (Reference 2.5.1-298) used a geotechnical approach to backcalculate ground motions for the 1886 Charleston earthquake based on soil properties of 1886 paleoliquefaction features. The threshold peak ground acceleration required to cause ground deformation was estimated based on the intersection of the layer curve effect of Ishihara (Reference 2.5.1-299) and the cyclic stress method (e.g., Reference 2.5.1-300). Martin and Clough (Reference 2.5.1-298) concluded that the liquefaction evidence was consistent with an earthquake no larger than **M** 7.5, and possibly as small as **M** 7.0 (Table 2.5.1-202).

Johnston (Reference 2.5.1-297) developed specific eastern North America regressions of seismic moment based on isoseismal area and averaged these with global SCR relations to estimate the magnitude of the 1886 Charleston earthquake. After considering multiple regressions, options for best-weighted values, and a correction for wedge effects of Coastal Plain sediments on isoseismals, a preferred best estimate of **M** 7.3 ± 0.26 (**M** 7.04 to 7.56) was obtained (Table 2.5.1-202). The Johnston study (Reference 2.5.1-297) also estimated a magnitude of **M** 7.4 ± 0.35 (**M** 7.05 to 7.77) using the extent and severity of liquefaction and the Liquefaction Severity Index. These estimates of maximum magnitude reflect a slight downward revision from the estimate from the estimate of maximum magnitude provided in Johnston et al. (Reference 2.5.1-245) of **M** 7.56 ± 0.35. Johnston (Reference 2.5.1-297) concluded that while uncertainties in magnitude are reported, "the final results of this study are best stated in general terms." For the 1886 Charleston earthquake, Johnston (Reference 2.5.1-297) concluded that the best estimate of magnitude is "in the low to mid-**M** 7 range." The SNC Vogtle ESP analysis considered this estimate to be a credible magnitude and incorporated it into the assessment of maximum magnitude for the UCSS.

In comparing intensity attenuation with epicentral distance for different stable continental regions, Bakun and McGarr (Reference 2.5.1-301) showed that eastern North America exhibits lower attenuation of seismic energy than other worldwide stable continental regions. Bakun and McGarr (Reference 2.5.1-301) noted that magnitude estimates based on averaging intensity attenuation relations from eastern North America and other stable continental regions may be overestimated. This suggests that Johnston (Reference 2.5.1-297) may have overestimated the magnitude of the 1886 Charleston earthquake.
Bakun and Hopper (Reference 2.5.1-282) estimated the magnitude and location of the 1886 Charleston earthquake using eastern North America intensity models that relate intensity and epicentral distance (Reference 2.5.1-302). Assuming that the 1886 event was centered in the Middleton Place – Summerville cluster of seismicity (and not offshore at their estimated intensity center), Bakun and Hopper (Reference 2.5.1-282) estimated a magnitude range of **M** 6.4 to 7.2 at the 95 percent confidence interval. Bakun and Hopper's (Reference 2.5.1-282) preferred magnitude estimate for the Charleston earthquake is M_I 6.9 (M_I is considered equivalent to **M**). The Bakun and Hopper (Reference 2.5.1-282) magnitude estimate suggests that the 1886 event may have been smaller than the Johnston (Reference 2.5.1-297) estimate. Both estimates are considered credible and are included in the UCSS model (Reference 2.5.1-272).

Obermeier et al. (References 2.5.1-284, 2.5.1-285, and 2.5.1-290) investigated the spatial distribution, size, and abundance of paleoliquefaction features in the Charleston coastal region and beyond. Based on the widespread distribution of sand blow craters in coastal South Carolina, Obermeier et al. (Reference 2.5.1-285) stated that these features were likely the result of earthquakes with magnitudes of at least m_b 5.5, and probably much stronger. Based on the observation that the limits of prehistoric liquefaction extend at least as far from Charleston as those formed during the 1886 earthquake (and the liquefaction susceptibility of deposits subjected to prehistoric earthquakes was likely as high as the liquefaction susceptibility of those subjected to the 1886 earthquake), Obermeier et al. (Reference 2.5.1-290) suggested that prehistoric Charleston-area earthquakes were probably at least as strong as the 1886 Charleston earthquake.

For paleoearthquakes, Talwani and Schaeffer (Reference 2.5.1-289) estimated the magnitudes of past Charleston-area events based on the spatial distribution and areal extent of paleoliquefaction sites (Figure 2.5.1-231). Talwani and Schaeffer (Reference 2.5.1-289) did not use a rigorous empirical method in their estimation of the magnitudes of past events. Instead they used a simple approach by which all past liquefaction episodes interpreted as having spanned a region comparable in size to the 1886 liquefaction field were assigned **M** 7+, and all past liquefaction episodes interpreted as having spanned a smaller areal extent were assigned **M** 6+.

Hu et al. (References 2.5.1-303 and 2.5.1-304) used the event chronology as interpreted by Talwani and Schaeffer (Reference 2.5.1-289) and the energy-stress method to estimate magnitudes of past Charleston-area earthquakes. For earthquakes that produced liquefaction features over extended areas centered near Charleston, Hu et al. (Reference 2.5.1-304) estimated magnitudes of **M** 6.8 to 7.8, and they estimated magnitudes of **M** 5.5 to 7.0 for earthquakes that produced liquefaction over more limited areas.

Leon (Reference 2.5.1-305) and Leon et al. (Reference 2.5.1-306) also estimated the magnitudes of past Charleston-area earthquakes using the event chronology as interpreted by Talwani and Schaeffer (Reference 2.5.1-289), but the Leon (Reference 2.5.1-305) and Leon et al. (Reference 2.5.1-306) method takes into account the effects of sediment age on the liquefaction potential of those sediments. Using the magnitude-bound method, Leon et al. (Reference 2.5.1-306) estimated magnitudes of **M** 6.9 to 7.1 for earthquakes that produced liquefaction features over extended areas, and **M** 5.7 to 6.3 for earthquakes that produced liquefaction over more limited areas. Using the energy-stress method, Leon et al. (Reference 2.5.1-306) estimated magnitudes of **M** 5.6 to 7.2 for earthquakes that produced liquefaction features over extended areas, and **M** 4.3 to 6.4 for earthquakes that produced liquefaction over more limited areas.

Based on a review of these published observations and analyses, the SNC Vogtle ESP Application (Reference 2.5.1-271 and 2.5.1-272) concluded the following:

- The magnitude ranges estimated for earthquakes that produced liquefaction over extended areas (References 2.5.1-303, 2.5.1-304, and 2.5.1-306) have significant overlap with magnitude estimates of the 1886 earthquake by Johnston (Reference 2.5.1-297) and Bakun and Hopper (Reference 2.5.1-282). However, given the large uncertainties in working with the paleoliquefaction record and methods for estimating magnitudes from these data, the best representation of the maximum magnitude for the Charleston seismic source should be based on estimates of the size of the 1886 earthquake (Table 2.5.1-202).

- The magnitudes estimated from the paleoliquefaction record for earthquakes that produced liquefaction over limited areas may have been less than **M** 6.3 (Reference 2.5.1-306). This implies that some events in the paleoliquefaction record may not represent large, 1886-type characteristic earthquakes. Therefore, the inclusion of any smaller paleoearthquakes in the recurrence model may bias the recurrence toward moderate-size earthquakes and may overestimate the frequency of large events.

Taken together, these new data suggest that the maximum magnitude for the 1886 Charleston earthquake is on the order of **M** 6.75 to 7.5 (References 2.5.1-298, 2.5.1-297, and 2.5.1-282; Table 2.5.1-202). The 95 percent confidence interval of Bakun and Hopper (Reference 2.5.1-282) implies the magnitude could have been as low as **M** 6.4; however, the preponderance of the data and evaluations indicate that the low end of this estimate likely underestimates the size of the 1886 earthquake.

2.5.1.1.4.4.3 Recurrence

Post-1986 EPRI studies of paleoliquefaction features (e.g., References 2.5.1-286, 2.5.1-287, 2.5.1-288, and 2.5.1-289) suggest that recurrence of large earthquakes on the Charleston seismic source is on the order of hundreds of years. This is significantly less than the EPRI model recurrence of several thousand years predicted by historical seismicity.

Earthquakes recorded in the paleoliquefaction record may include events significantly less than the maximum magnitude because the minimum threshold magnitude for earthquakes to cause liquefaction is estimated as $m_b > 5.5$ (Reference 2.5.1-285) or **M** 4.3 – 6.4 (Reference 2.5.1-306). It was noted in the SNC Vogtle ESP Application (Reference 2.5.1-272) that estimates of maximum-magnitude recurrence intervals based on the paleoliquefaction record may include events smaller than the maximum magnitude and may overestimate the frequency of maximum magnitude recurrence. Simply because the age determinations for paleoliquefaction features at widely distributed sites overlap does not necessitate that the features were the result of a single, large earthquake. The possibility that paleoliquefaction features of similar age (i.e., within the uncertainty in age determination) resulted from smaller earthquakes that occurred over a wide area, closely spaced in time, is an inherent uncertainty in the paleoliquefaction record. Recent (post-1986) EPRI studies that characterized the recurrence of prehistoric earthquakes from the paleoseismic record are described below.

- Amick (Reference 2.5.1-286) and Amick et al. (Reference 2.5.1-287, Reference 2.5.1-288) used liquefaction data collected from South Carolina and their more regional reconnaissance investigations along the North Carolina and Virginia coastlines to suggest that large earthquakes occur every 500 to 600 years in Coastal South Carolina, and that paleoliquefaction evidence for earthquakes located outside of South Carolina is lacking.

- Talwani and Schaeffer (Reference 2.5.1-289) combined previously published data with their own studies of liquefaction features in the South Carolina coastal region (Figure 2.5.1-231). Talwani and Schaeffer (Reference 2.5.1-289) used the spatial distribution of paleoliquefaction features in combination with estimates on the timing of the formation of the liquefaction features to derive possible earthquake recurrence histories for the region. Talwani and Schaeffer's first scenario (Reference 2.5.1-289) allows for the possibility that some events in the paleoliquefaction record are smaller in magnitude (approximately **M** 6+), and these more moderate events occurred to the northeast (Georgetown) and southwest (Bluffton) of Charleston. In Talwani and Schaeffer's second scenario (Reference 2.5.1-289), all earthquakes in the record are large shocks (approximately **M** 7+) located near Charleston. Talwani and Schaeffer's (Reference 2.5.1-289) preferred estimate for the recurrence of large earthquakes in coastal South Carolina is 500 to 600 years.

- For the SNC Vogtle ESP study, the radiocarbon ages used by Talwani and Schaeffer (Reference 2.5.1-289) were analyzed and recalibrated to report the ages with 2-sigma error bands that give broader age ranges for paleoliquefaction events in the Charleston area (Reference 2.5.1-349). (The 1-sigma error bands used by Talwani and Schaeffer [Reference 2.5.1-289] are considered by many to be too narrow and thus leading to potential over-interpretation such that more episodes of paleoliquefaction are interpreted than actually occurred.) The 2-sigma analysis identified six earthquakes (including the 1886 event) in which event ages were defined and considered to represent the 95 percent confidence interval based on grouping paleoliquefaction features that have overlapping calibrated radiocarbon ages. This analysis indicated that each of the six earthquakes represent large maximum-magnitude events, in contrast to the Talwani and Schaeffer (Reference 2.5.1-289) scenario in which some smaller moderate-magnitude events are recognized.

2.5.1.1.4.5 Regional Seismicity

The LNP site is located in an area of infrequent and low seismicity within the Gulf Coast basin tectonic province (Figure 2.5.1-232). Only 15 earthquakes larger than m_b 3.0 have occurred within the LNP site region (320 km [200 mi.]). The largest event, an m_b 4.3 earthquake, occurred at a distance of 76.6 km (47.6 mi.) from the LNP site, and is the only event within 80 km (50 mi.) of the site.

Seismicity that is occurring beyond the site region also was considered. The occurrence of two moderate earthquakes in the Gulf of Mexico in 2006 has implications to the evaluation of seismicity for the Gulf Coast basin source zones that include the LNP site. A description of these events is provided in FSAR Subsection 2.5.2.1.2.2. Focal mechanisms that are available for only two Gulf of Mexico events, the July 24, 1978, Emb 4.9 Gulf of Mexico earthquake and the September 10, 2006, Emb 6.0 earthquake, both show compressive mechanisms (Figure 2.5.1-232).

2.5.1.2 Site Geology

The following subsections provide a summary of geologic conditions in the LNP site vicinity (40 km [25 mi.] radius) and site area (8 km [5 mi.] radius). These subsections present information concerning the physiography, geologic history, stratigraphy, structural geology, hydrology, and engineering geology related to the LNP site. The information presented is based on a review geologic literature, communications with geologists and other researchers who are familiar with previous studies in the site area, and geotechnical and geologic field investigations conducted at and in the vicinity of the LNP site.

2.5.1.2.1 Site Physiography and Topography

The LNP site, located within southwestern Levy County, lies approximately 16 km (10 mi.) east of the Gulf of Mexico and approximately 12.8 km (8 mi.) north of the Withlacoochee River. Within the 40 km (25 mi.) radius of the site, two other counties border the LNP site: Citrus County located south of the site and Marion County located east of the site. A geomorphic map of the site vicinity (40 km [25 mi.] radius) and site area (8 km [5 mi.] radius) is shown on Figure 2.5.1-233. The western portion of Levy County is a poorly drained, low relief region that is characterized by extensive swamps, marshes, and terraces formed by ancient sea-level highstands. (Reference 2.5.1-307) As shown on Figure 2.5.1-201, the site lies within the central (midpeninsular) physiographic zone. (Reference 2.5.1-212) Levy County lies near the northern edge of the midpeninsular zone. This zone spans the Florida peninsula from the lower edge of the Northern Highlands southward to approximately the Caloosahatchee River, and is characterized by ridges, valleys, and terraced coastal plains. The midpeninsular zone is subdivided into a series of elevationally differentiated geomorphic provinces. Two of these geomorphic provinces occur within the 40 km (25 mi.) radius of the site — the Central Highlands, and the Gulf Coastal Lowlands. (Reference 2.5.1-308) The site lies within the Gulf Coastal Lowlands. These geomorphic provinces and their respective subzones that fall within the 40 km (25 mi.) radius of the site are shown on Figure 2.5.1-233.

2.5.1.2.1.1 Geomorphic Provinces

2.5.1.2.1.1.1 Central Highlands Geomorphic Province

The Central Highlands geomorphic province comprises the eastern third of Levy County, two thirds of Citrus County, and all of Marion County (Figure 2.5.1-233). The Central Highlands include a series of localized highlands and ridges punctuated by topographically lower valleys, all of which trend generally coast-parallel down the central Florida peninsula. The Central Highlands province is further subdivided into the Western Valley and the Brooksville Ridge. (Reference 2.5.1-308) The Tsala Apopka Plain is part of the Western Valley subzone in Citrus County. (Reference 2.5.1-307) In Marion County, there are 12 different subzones within the Central Highlands. Only seven of these subzones lie within the 40 km (25 mi.) radius of the LNP site; these are the Brooksville Ridge, Western Valley, Cotton Plant Ridge, Martel Hill, Sumter Upland, Fairfield Hills, and the Central Valley (Figure 2.5.1-233). Sumter Upland, Fairfield Hills, Martel Hill, and Cotton Plant Ridge together form a north south series of topographic highs that separate the Western Valley from the Central Valley. These ridges are thought to be relict coastal features and are largely composed of thick sand and clayey sand deposits. Surface elevations within these four subdivisions are as follows: Sumter Upland elevation is 23 to 30 m (75 to 100 ft.)

amsl; Fairfield Hills elevation is 24 to 64 m (80 to 210 ft.) amsl; Martel Hill is considered an outlier of the Fairfield Hills; and Cotton Plant Ridge has a maximum elevation of 51 m (168 ft.) amsl. Immediately to the west of Cotton Plant Ridge and the Sumter Upland lies the Western Valley and west of the Western Valley lies the Brooksville Ridge, a large, linear high that is described below. (Reference 2.5.1-309)

Western Valley. In Levy County, the Western Valley extends both east and south into Marion County and is bounded on the west by the Brooksville Ridge (see Figure 2.5.1-233). Along the eastern edge of Levy County, the Western Valley encompasses the Williston Limestone Plain, a well developed, gently rolling limestone plain with surface elevations ranging from 18 to 30 m (60 to 100 ft.) amsl. Covering the limestone is a thin layer of Pleistocene sand and clayey sands that contain localized pockets of phosphatic Alachua formation sediments. (Reference 2.5.1-308) Vernon (Reference 2.5.1-262) hypothesized that the Williston Limestone Plain was a relict erosional limestone shelf of Eocene sediments that represented the seaward extension of the ancient Wicomico Sea. (Reference 2.5.1-310)

In Citrus County, the Western Valley extends the length of the county and is bounded on the west by the Brooksville Ridge and on the east by the Withlacoochee River. The Western Valley encompasses the Tsala Apopka Plain, which consists of a number of interconnected lakes partially separated by peninsulas and islands. Sands and clayey sands of variable thickness cover the limestone surface. Land surface elevations range from 18 to 24 m (60 to 80 ft.) amsl, whereas water surface elevations vary from 11 to 14 m (35 to 45 ft.) amsl. (Reference 2.5.1-307)

Brooksville Ridge. The Brooksville Ridge is a topographic highland extending from northeastern Gilchrist County southward through eastern Levy County, terminating 177 km (110 mi.) to the south in Pasco County. (Reference 2.5.1-308) It is bounded to the east by the Western Valley subzone and to the west by the Gulf coastal lowlands. In Levy County, it is present as a thin unit on the eastern third of the county, and is only present in an isolated region in the northwest corner of Marion County. In Citrus County, it occupies the central part of the county. From north to south, the width of the ridge increases as well as surface elevations.

The Brooksville Ridge has an irregular surface due to karst activity, and elevations may vary over 30 m (100 ft.) in short distances. (Reference 2.5.1-307) The core of the ridge, overlying Eocene limestone, is composed of varying thicknesses of Pleistocene siliciclastics and is capped by a depression-pocked rolling plain of Pleistocene marine terrace sands. (Reference 2.5.1-308) These clastic sediments restrict the downward percolation of water, thereby reducing dissolution of the underlying limestone. (Reference 2.5.1-311) In Levy County, the Brooksville Ridge is more than 46 m (150 ft.) amsl, significantly higher than the surrounding plains. Vernon (Reference 2.5.1-262) named this feature the Coharie – Okefenokee Sand Ridge. (Reference 2.5.1-310)

2.5.1.2.1.1.2 Gulf Coastal Lowlands Geomorphic Province

The Gulf coastal lowlands geomorphic province occupies more than two thirds of Levy County and approximately one third of Citrus County; it is not present in Marion County (Figure 2.5.1-233). It parallels the present Gulf coast of Florida from Ft. Myers northward, then westward to the Alabama line. In Levy and Citrus counties, the Gulf coastal lowlands extend inland from the modern Gulf of Mexico shoreline, terminating at the western edge of the Brooksville Ridge. The Gulf coastal lowlands are characterized by broad, flat marine erosional plains, underlain by Eocene limestones, and covered by thin Pleistocene sands deposited by the regressing Gulf of Mexico. (Reference 2.5.1-308) The geomorphic setting is a low energy, salt or freshwater environment with insufficient sand to build beaches. The marine terraces located within this geomorphic province are gently sloping features with seaward facing escarpments. These features formed when sedimentary materials were alternatively deposited and eroded as sea levels rose and fell. (Reference 2.5.1-307)

In Levy County, the Gulf coastal lowlands are subdivided into several subzones that are differentiated based on topography (Figure 2.5.1-234). These subzones include the Waccasassa Flats, the Limestone Shelf and Hammocks, the Chiefland Limestone Plain, the Suwannee River Valley Lowlands, and the Coastal Marsh Belt. (Reference 2.5.1-308)

Waccasassa Flats. The Waccasassa Flats, located in central Levy County, is a low swampy area that extends from the Santa Fe River in Gilchrist County southeast into Levy County. Land surface elevations average 17 m (55 ft.) amsl, although isolated sand hills, possibly associated with the Wicomico marine terrace deposits and the Brooksville Ridge, reach elevations as high as 22 m (70 ft.) amsl. At the southern edge of the Waccasassa Flats, the zone broadens to approximately 22.5 km (14 mi.) wide and elevations decrease to 9 m (30 ft.) amsl as the flats merge into the hammocks of southwestern Levy County. The Waccasassa River, which originates as a poorly defined channel in the swamps, lakes, and ponds of northern Levy County, drains the lower reaches of the Waccasassa Flats. The river flows to the southwest and empties into the Gulf of Mexico. A narrow Holocene floodplain of muds and sand occurs near the coast where the river merges with the coastal swamps. (Reference 2.5.1-308)

Limestone Shelf and Hammocks. The LNP site is located within the Limestone Shelf and Hammocks subzone, a zone that is characterized as a highly karstic, erosional limestone plain overlain by sand dunes, ridges, and coast-parallel paleoshore sand belts associated with the Pleistocene age Pamlico marine terrace. Much of this subzone is underlain by the Upper Eocene Ocala Limestone, a very soluble unit. In the immediate area of the LNP site, the Ocala Limestone is absent due to erosion, and the dolomitized carbonates of the Avon Park Formation, a much less soluble unit, comprise the first carbonates encountered. The irregular Eocene surface underlying this subzone (primarily Ocala group, excepting limited areas including the LNP site where Avon Park Formation occurs) is covered by a blanket of Pleistocene sands of varying thickness. (Reference 2.5.1-308) The erosional plain developed on the Avon Park Formation, which is present at the LNP site and underlies portions of the eastern part of the subzone, is also karstified, but to a lesser degree given the more dolomitic nature of this unit. Cover collapse sinkholes are uncommon in the area of the LNP site due to the dolomitized nature of the Avon Park Formation carbonates. In this area, cover subsidence sinkholes occur where dissolution of the top of the carbonate section occurs. This slowly lowers the land surface over long periods of time. (References 2.5.1-354 and 2.5.1-355) Near the modern coast, the limestone shelf is drowned by the coastal marshes. Inland, the limestone is heavily forested. Locally, artesian springs flow from the near surface limestone, and during heavy rain events much of the zone is prone to flooding, producing shallow swamps. In the area where Ocala Group limestone is absent and Avon Park Formation underlies the sands there are very few springs (Figure 2.5.1-244). Drainage from the coastal hammocks occurs through numerous small creeks and sloughs, which empty into the coastal marshes. (Reference 2.5.1-308)

Chiefland Limestone Plain. The Chiefland Limestone Plain, located in northwestern Levy County, is a flat, karstic limestone shelf that extends from Gilchrist County southward into Levy County. The Eocene limestone plain is generally flat to rolling, covered by a veneer of well drained Pleistocene sands, generally less than 9 m (30 ft.) thick. Elevations range from 8 m (25 ft.) amsl at the southern edge of the plain to nearly 15 m (50 ft.) amsl at the Levy – Gilchrist county line. (Reference 2.5.1-308)

Suwannee River Valley Lowlands. The Suwannee River Valley Lowlands house the Suwannee River, which flows southwest and empties into the Gulf of Mexico. The river, which forms the northwestern boundary of Levy County, flows in a solution valley, formed in the near surface Eocene limestones. The lowlands adjacent to the river are made up of a thin veneer of Holocene alluvium and exposed limestone. In the lower reaches, the river valley is drowned and obscured by marshes of the Coastal Marsh Belt subzone. The broadly meandering valley is less than 1.6 km (1 mi.) wide over most of its course, broadening to about 4 km (2.5 mi.) wide just northwest of the Chiefland Limestone Plain. Elevations of the valley floor average 1.5 m (5 ft.) amsl. (Reference 2.5.1-308)

Coastal Marsh Belt. The Coastal Marsh Belt subzone is located on the drowned, seaward edge of the Eocene limestone shelf underlying Levy County. Elevations are less than 1.5 m (5 ft.) amsl. The gentle slope of the limestone plain results in a very broad, shallow continental shelf off the coast of Florida. Sediments are predominantly muds, and alluvial sand beaches are virtually absent due to the zero energy nature of the shoreline and lack of adequate sand supply. Instead, marshes of Juncus and Spartina grasses fringe the modern coastline, and a series of small islets or keys, comprised of limestone pinnacles or alluvial sand, are common offshore of the modern coast. (Reference 2.5.1-308)

Davis et al. (Reference 2.5.1-213) refer to the low wave energy coastline from the Appalachicola River delta to Anclote Key (just north of Tampa Bay) as the Big Bend Coast (or area). This part of the coastline, which includes the LNP site vicinity, is morphologically complex due to variations in underlying limestone bedrock topography, and the presence of actively discharging freshwater springs, large oyster bioherms, a modern river delta, and possible paleoshorelines. The Suwannee River, which lies approximately 138 km (86 mi.) northwest of the LNP site, is the only coastal plain river that discharges along this coast. Other rivers emanate from springs fed by the Floridan aquifer and travel only a few kilometers (miles) across the coastal plain before reaching the Gulf of Mexico.

2.5.1.2.1.2 Marine Terraces

Elevated terraces and related shorelines, beach-ridge plains, and inner-shelf sediments record the long-term effects of late Tertiary to Quaternary sea-level changes on the stable Florida platform. The general lowering of sea level from Pliocene time to the present is evident in the decrease in elevation of terrace features with age. Episodic sea-level swings of the late Quaternary and Holocene are identified in the Panhandle Florida region and in southern Florida and the Florida Keys regions, based on both submerged and raised shoreline features preserved in coastal and nearshore areas. (Reference 2.5.1-312)

Based on review of literature and examination of all data on terraces in Florida, Healy (Reference 2.5.1-215) identified eight terrace intervals that were mapped on a statewide basis (Figure 2.5.1-202). They are, in ascending order, the Silver Bluff Terrace, less than 1 to 3 m (0 to 10 ft.) amsl; the Pamlico Terrace, 2.5 to 7.6 m (8 to 25 ft.) amsl; the Talbot Terrace, 7.6 to 12.8 m (25 to 42 ft.) amsl; the Penholoway Terrace, 12.8 to 21.3 m (42 to 70 ft.) amsl; the Wicomico Terrace, 21.3 to 30.4 m (70 to 100 ft.) amsl; the Sunderland (or Okefenokee) Terrace, 30.4 to 51.8 m (100 to 170 ft.) amsl; the Coharie Terrace, 51.3 to 65.5 m (170 to 215 ft.) amsl; and the Hazlehurst Terrace, 65.6 to 97.5 m (215 to 320 ft.) amsl (Figure 2.5.1-202). (Reference 2.5.1-215)

Criteria used by Healy and in prior investigations to map and correlate terraces relied extensively on topographic position, elevation, morphology, and limited stratigraphic and dating information. There was little agreement amongst the early workers, however, regarding the number or ages of the older, higher terraces exposed in northern peninsular Florida and the Florida panhandle. Some workers thought the highest sand ridge feature related to a sea level highstand (referred to as Trail Ridge) was Miocene in age (Reference 2.5.1-314), whereas others believed the ridge to be no older than Pliocene (Reference 2.5.1-313). Several researchers suggested that terraces at and above approximately 30 m (100 ft.) amsl are likely Pliocene in age. (References 2.5.1-313, 2.5.1-215, 2.5.1-314, and 2.5.1-312). Winkler and Howard (Reference 2.5.1-359) using geomorphic evidence concluded that the Trail Ridge sequence and two younger sequences along the Atlantic coastal plain region from North Carolina to south-central Florida were deformed by warping and that correlation of terraces strictly by elevation was not meaningful.

Paleo-shoreline features adjacent to Trail Ridge near the border of northern Florida and southern Georgia, subsequently were found to contain marine fossils believed to be late Pliocene or Pleistocene age at elevations of between 42 and 49 m (130 to 161 ft.) amsl (Reference 2.5.1-362). Opdyke et al. (Reference 2.5.1-360), citing this evidence and previous work by Winkler and Howard (Reference 2.5.1-359) and others, noted that Pleistocene shoreline features do not maintain a constant elevation when traced from Georgia into southern Florida. From a low point along the Georgia coast, the shoreline features rise to maximum elevation of about 58 m (190 ft.) in northern peninsular Florida and then gradually drops to elevations of approximately 40 m (130 ft.) in south-central Florida. Opdyke et al. (Reference 2.5.1-360) and Willett (Reference 2.5.1-361) postulate that isostatic readjustment driven by carbonate dissolution has caused broad epeirogenic uplift in Florida. Readjustment rates calculated by Opdyke et al. (Reference 2.5.1-361) were 36 m (118 ft.) during the Pleistocene and Holocene. This equates to one meter of limestone every approximately 38,000 years. Willett (Reference 2.5.1-361), using a more robust data set, concluded that karst areas in Florida are losing approximately 1 m (3.3 ft.) of limestone every 160,000 years and that the impact of long-term carbonate dissolution and mass loss from the Florida platform has led to isostatic uplift of at least 9 m (30 ft.) and as much as 58 m (190 ft.) since the beginning of the Quaternary (~1.6 Ma). This process helps to explain the observed elevations of Pliocene-Pleistocene marine sediments that exceed paleosea level projections and apparent warping of the older marine shoreline features.

Means (Reference 2.5.1-363) suggests that lithospheric flexture due to sediment loading is another nontectonic mechanism in addition to isostatic adjustment due to carbonate dissolution that could explain the presence of marine-influenced sediments observed at elevations of over 66 m (216 ft.) in the panhandle region of Florida, higher than any maximum sea-level stand of the Miocene, Pliocene, or Pleistocene.

Hoenstein et al. (Reference 2.5.1-309), Lane et al. (Reference 2.5.1-310), and Yon et al. (Reference 2.5.1-311) delineated marine terraces in the site vicinity based on the terrace designations of Healy (Reference 2.5.1-215) and Vernon (Reference 2.5.1-262). Seven terraces are mapped within the site vicinity, as illustrated on Figure 2.5.1-235. From highest (and presumably oldest) to lowest (youngest) the marine terraces are: Coharie, Sunderland/Okefenokee, Wicomico, Penholoway, Talbot, Pamlico, and Silver Bluff. (Reference 2.5.1-310) The ages and correlations of the higher terraces as shown, which are generally based on elevation, are very speculative. Detailed mapping of marine terraces and geochronologic investigations to provide good age control is not available for the marine terraces in the site vicinity or throughout most of the site region. This is due in part to the limited exposures and the paucity of fossils or dateable material in the terrace cover.

Aminostratigraphic studies in central and south Florida provide information that can be used to assess the age of the marine terrace surfaces in the LNP site vicinity (Reference 2.5.1-356). Samples of fossiliferous sediments collected at two sites along the Wacasassa River in the site vicinity (Figure 2.5.1-235) were evaluated using aminostratigraphy as part of a regional study to compare the relative ages of stratigraphic units based on aminostratigraphic analysis and numerical ages determined by calibration with U-series age estimates (Reference 2.5.1-356). Site 1 samples were collected below water level from a fossiliferous layer within a sand-filled karst feature at the top of exposed Eocene limestone. Due to the presence of *Konis adversa*, this site was initially assumed to be part of the Caloosahatchee formation (late Pliocene to early Pleistocene in age). Site 2 was sampled from a fossiliferous sand layer overlying the karstified limestone bedrock approximately 1.6 km (1 mi.) south of Site 1. Shell abundance was less than at Site 1 with a different depositional environment indicated by brackish water-fauna. Site 2 was initially proposed to be a different facies of the same stratigraphic unit sampled at Site 1. The results of the aminostratigraphic analysis demonstrated that Sites 1 and 2 are of two different ages. The Wacasassa River Site 2 is correlated to a younger aminozone and possibly the Ft. Thompson formation that represents MIS 5e in central Florida. Site 1 correlates to an older amniozone that includes the Pinecrest Beds, Caloosahatchee, and Bermont formations. Aminoacid racemization analysis is not able to resolve for age differences among these units.

Uranium-series analysis of fossil coral samples obtained from Units C, D, and E (correlated to the Caloosahatchee [late Pliocene-early Pleistocene], Bermont [middle Pleistocene], and Ft. Thompson [late Pleistocene] formations) in south Florida indicated ages of greater than 400 ka for Unit C, approximately 230-360 ka for Unit D, and approximately 144 ka for part of Unit E. Since none of the samples of corals from this study experienced ideal closed-system conditions with respect to uranium and its daughter products, only approximate ages could be inferred. The results, however, indicate that marine deposition was extensive over much of southern Florida during the middle Pleistocene. (Reference 2.5.1-236). Figure 2.5.1-260 shows the distribution of Pliocene to Pleistocene marine sediments in southern Florida. The two terraces and associated shorelines that parallel the coastline in both Levy and Citrus Counties are the Silver Bluff and the Pamlico (see Figure 2.5.1-235). The Pamlico terrace and shoreline are well developed features that are recognized and mapped along the entire Gulf coast of Florida. The Pamlico terrace covers most of west-central Levy County and the eastern third of Citrus County. It is characterized by a number of karst features, including numerous sinkholes and caverns, and there are a number of streams and creeks that flow through this terrace. (Reference 2.5.1-310) Based on the elevation of the inland part of the marine terrace platform underlying the Pamlico terrace (less than 7.6 m [25 ft.] amsl) (Reference 2.5.1-310), this terrace is estimated to have formed during MIS 5e (approximately 120 ka) when sea level stood approximately 6 m (20 ft.) above present sea level. Muhs et al. (Reference 2.5.1-237) summarizes recent studies in southern Florida that suggest that sea level during the last interglacial (MIS 5e) must have been at least 5 to 8 m (16 to 26 ft.) higher than present. The estimated correlation of the deposits at Site 2 along the Wacasassa River to MIS 5e suggests they are associated with the Pamlico terrace.Tiling (Reference 2.5.1-356) does not provide information on the exact locations or elevations of the two sites sampled on the Wacasassa River. The general elevation of the river at the Wacasassa River locality is approximately 4.5 m (15 ft.), which is consistent with possible inundation during MIS 5e.

The Silver Bluff terrace in the study area may represent a continuation of the seaward dipping platform that has a thinner cover of silisticlastic cover. Alternatively, parts of the Silver Bluff terrace may have been reoccupied during a subsequent sea-level highstand during MIS 5a (approximately 80 ka), when sea level may have been close to present levels. Uranium series ages of approximately 80,000 years for corals from the tectonically stable Atlantic Coastal Plain suggest that sea level at that time was near present, whereas the oxygen isotope record suggests that sea level was then well below present (Reference 2.5.1-237). The elevation of the MIS 5a shoreline in Florida is not well constrained at this time.

The Levy County map shows the LNP site to be located at the inner edge of the Pamlico terrace below the Penholoway terrace (Figure 2.5.1-235); however, the general elevation of the LNP site (12 to 13.4 m [40 to 44 ft.] NAVD88) suggests that the site is located on the outer edge of the Penholoway terrace or possibly on an unmapped remnant of the Talbot terrace. The elevation of the unconformity between the thin mantle of Quaternary sediments and Eocene limestone at the site (approximately 11 ± 1 m [36 ± 3 ft.]) indicates that the site is well above the expected elevation of the Pamlico wave cut terrace platform (approximately 6 m [20 ft.] amsl), and therefore is on a terrace surface older than MIS 5e (approximately 120 - 125 ka). As noted previously, dating of marine terraces in southern Florida indicates that the MIS 7 (220 - 230 ka) and MIS 9 (300 - 340 ka) shorelines may have been close to present (Reference 2.5.1-237) indicating that it is not likely the terrace surface at the site was formed during either of those sea level highstands.

The marine terrace that underlies the LNP site may correlate to MIS 11 (approximately 400-440 ka), which is recognized as one of the longer and warmer interglacials of the past 1 Ma (Reference 2.5.1-357). Recent and ongoing research regarding MIS 11 suggests that the paleosea level during this highstand may have been higher than during the MIS 5e (approximately120-125 ka) highstand. Data that constrain the paleosea level during MIS 11 are available from studies in Bermuda and South Africa, both stable intraplate like Florida where marine terrace shorelines are expected to reflect glacio-eustatic sea levels (References 2.5.1-357 and 2.5.1-358). These studies suggest that sea level during MIS 11 was approximately +20 m (66 ft.) and +15 m (49 ft.), respectively, as recorded in Bermuda and South Africa. The marine terrace platform at the LNP site lies at a similar or slightly lower elevation. A correlation to the MIS 11 highstand cannot be precluded at this time, but additional work is needed to confirm the preliminary age estimates and paleosea level elevation of this highstand in South Africa and in other regions.

Based on these observations, the marine terrace underlying the LNP site is estimated to be Middle Pleistocene (approximately 400-440 ka) or older (early Pleistocene to possibly late Pliocene) in age.

2.5.1.2.1.3 Karst Terrain

Karst terrain refers to a topographic configuration of subsidence features and drainage arising mainly from dissolution of limestone and other soluble rocks. (Reference 2.5.1-315) The Florida karst displays a gently rolling topography with shallow, broad depressions. (Reference 2.5.1-316)

The LNP site, located within the Gulf Coastal Lowlands geomorphic province, is characterized by both depositional and erosional features. Broad plains underlain by a series of late Tertiary and Quaternary surfaces and shorelines are pitted with karstic depressions within the limestone and dolostone at or near the present land surface in the site area. The Gulf Coastal Lowlands represent a typical mature karst terrain overlain by a thin mantle of permeable terrace deposits (i.e., a mantled epikarstic subsurface as described below in FSAR Subsection 2.5.1.2.1.3.1). (Reference 2.5.1-316) It is important to note that dolomite/dolostone is less soluble than calcite/limestone. The LNP site is underlain by dolomitized limestones and dolostones. See the discussion of site geology (FSAR Subsection 2.5.1.2.3).

Karst topography created by dissolution processes influences local coastal morphology, sedimentation, and resulting stratigraphy. Two basic karst processes operating in the Big Bend coastal area that have produced three easily recognizable horizontal scales of surficial topography are: (1) surface dissolution due to downwelling of acid pore waters from overlying marsh sediments; and (2) regional dissolution, primarily subterranean dissolution, and subsequent collapse due to mixing-zone undersaturation. The smallest scale (centimeters to a few meters) is less important than the medium (tens to hundreds of meters) and large-scale (kilometers) karst-induced coastal features. Medium-scale features are rectilinear tidal creeks occupying enlarged joint patterns as well as rock-cored hammocks forming marsh islands. Large-scale features are broad shallow depressions in the bedrock forming shelf embayments, elevated rocky areas between embayments forming marsh-island archipelagoes, and linear channel structures etched in the bedrock by laterally moving spring-discharge events (Figure 2.5.1-236). (Reference 2.5.1-213) The morphology of the modern coastline is a likely analog for the types of features that would have been present along the older, higher paleoshorelines in the LNP site area.

Based on a regional classification of karst potential throughout the State of Florida, Sinclair and Stewart (Reference 2.5.1-317) show the LNP site to be located in a region where the limestone is bare or thinly covered and sinkholes are few, generally shallow and broad and developed gradually (Figure 2.5.1-237). Site characterization activities were conducted to evaluate the development of karst at the site (see FSAR Subsection 2.5.1.2.5.3) and potential for surface deformation related to karst (see FSAR Subsection 2.5.3.8). The following sections provide an overview of karst evolution and description of karst features that are observed in the site region.

2.5.1.2.1.3.1 Conceptual Model of Karst and Epikarst Evolution

Schematic cross sections showing development of karst features beneath a marshy coastline like that in the site area are shown on Figure 2.5.1-238. In a marsh archipelago, numerous marsh islands form on a flooded, elevated, topographically irregular bedrock surface. The marsh creeks occupy dissolved rectilinear bedrock fractures, or connect a series of sinkholes. The marsh islands or hammocks are underlain by localized bedrock highs or nubs. The stratigraphic veneer on these highs is thin and discontinuous, but may be locally thick in deep sinkholes. The holes are generally filled with basal clean, light-colored Pleistocene quartz sands followed by rooted, organic-rich, dark-colored, fine-grained quartz sands from the modern marsh. As the sea level rises, the marsh hammocks, originally surrounded by marsh grasses, become encircled by enlarging tidal creeks. Eventually, the hammocks become marsh islands. (Reference 2.5.1-213)

A schematic diagram representing the evolution of epikarst and changes in its characteristics is presented in Figure 2.5.1-239. Klimchouk (Reference 2.5.1-318) defines epikarst as follows.

> The uppermost weathered zone of carbonate rocks with substantially enhanced and more homogeneously distributed porosity and permeability, as compared to the bulk rock mass below; a regulative subsystem that functions to store, split into several components and temporally distribute authogenic infiltration recharge to the vadose zone. Permeability organization in the epikarst dynamically develops to facilitate convergence of infiltrating water towards deeply penetrating collector structures such as prominent fissures that drain the epikarstic zone. This is manifested by epikarstic morphogenesis that tends to transform disperse appearance of surface karst landforms into focused appearance adapted to the permeability structure at the base of epikarst. (Reference 2.5.1-318)

One of the early episodes of karstification in the site vicinity began with the subareal exposure of the thick carbonate platform in response to a major sea level drop during the Late Oligocene that caused carbonate sediment suppression and extensive siliciclastic deposition. Karstification continued as Miocene sea level fluctuations periodically exposed portions of the Florida Platform. During the Miocene, significant siliciclastic deposition occurred. (Reference 2.5.1-316)

Sea level reached a Miocene maximum during the middle part of this epoch and covered the Florida platform. Clay-dominated sediments of the Hawthorn Group extended across the carbonates of the Ocala platform, and this mantle of clay-rich sediment slowed the infiltration of surface runoff and thus limited the development of karst. Further dissolution of karst conduits did not commence until the overburden was eroded from the higher parts of the Ocala platform following a sea-level drop during the Miocene. (Reference 2.5.1-316) Earlier karst phases and the resultant template of karst development induced by the

Oligocene sea-level drop may have played an important role in the development of preferential dissolution. (Reference 2.5.1-316)

Formation of the large rivers in the site region, such as the Suwannee River, had a significant effect on the development of karst in that area. Creating a local base level, the rivers increased the hydraulic gradient and controlled groundwater flow patterns. Extensive dissolution of the carbonate rocks caused well-developed karst landforms that have been repeatedly covered and exhumed by the post-Miocene sea-level fluctuations. Surficial erosion of the impermeable siliciclastic cover started in response to the increasing fluvial activity in the Pleistocene. Diffuse autogenic recharge through soil layers caused extensive dissolution within the epikarst mainly because of the increased chemical aggressiveness induced by soil gases. (Reference 2.5.1-316)

The final sequence of karst development in the area involved the gradual subaerial exposure of the carbonate platform in response to periodic transgressions and regressions of interglacial seas. New areas of carbonate platform were exposed to karst activity as the Pleistocene seas retreated. Older depressions at higher elevations coalesced and expanded, resulting in more circular and larger depressions (Reference 2.5.1-316) than the younger, lower terraces, such as the Pamlico and Silver Bluff terraces along the coast.

2.5.1.2.1.3.2 Sinkholes

Karst features form when the flow of water is concentrated along well-defined conduits. Such conduits include joints or fractures, faults, and bedding planes in the rock, enlarged by rock dissolution. Dissolution of limestone in Florida appears to occur preferentially in recharge areas and near the saltwater/freshwater coastal mixing zones; recharge areas are the more important of these two environments of sinkhole development. (Reference 2.5.1-319) Factors influencing the development of karst terrain include age of the limestone; its depth below the ground surface; structural lineaments in the limestone that provide preferred areas for dissolution; permeability of the overlying material (Reference 2.5.1-315); phreatic-vadose zone fluctuations induced by sea-level changes; paleokarst templates; deposition and erosion of siliciclastic sediments; and the formation of a fluvial system (Reference 2.5.1-316). Anthropogenic factors include over-pumping of groundwater that reduces the shear-strength of the near-surface materials, and causes higher intergranular stress and a resulting reduction in the load carrying capacity of the soils, as well as the placement of structures over geologic features that have the potential for sinkhole activity. These factors often trigger sinkhole activity and ground subsidence. (Reference 2.5.1-315)

Sinkholes are a primary feature of karst terrains. The characteristic surface depression — commonly circular — can be identified on aerial photographs, maps, and in the field. Sinkhole activity involves the development of a sinkhole, including its early stages (raveling) where there is no visible manifestations of ground surface subsidence or collapse. Raveling is the lateral or downward migration of unconsolidated material into more deeply buried cavity in limestone. (Reference 2.5.1-315)

Sinkholes can occur in a variety of shapes ranging from steep-walled to funnel-shaped to bowl-shaped depressions. Three major types of sinkholes common to Florida are solution sinkholes, cover-collapse sinkholes, and cover-subsidence sinkholes. These sinkhole types are distinguished by their mode of formation, which is largely controlled by the geology and hydrology of the area. Descriptions of the types of sinkholes are presented below.

2.5.1.2.1.3.2.1 Solution Sinkholes

Solution sinkholes occur in areas where limestone is exposed at the land surface or is mantled by only a thin layer of cover (Figure 2.5.1-240). Solution is most active at the limestone surface and along joints or fractures or other openings in the rock that permit water to move easily into the subsurface. Large voids commonly do not form because subsidence of the soil layer occurs as the surface of the limestone dissolves, resulting in a gradual downward movement of the land surface and in development of a depression that collects increasing amounts of surface runoff as its perimeter expands. This type of sinkhole generally develops as a bowl-shaped depression with the slope of its sides determined by the rate of subsidence relative to the rate of erosion of the walls of the depression from surface runoff. Surface runoff may carry sand and clay particles into the depression, resulting in an impermeable seal in the bottom of the sinkhole. Due to these impermeable seals, marshes and lakes form covering these sinkholes. This process produces an undulating topography characterized by shallow depressions and is common over large parts of Florida. The LNP site lies completely within the area dominated by solution sinkholes (Figure 2.5.1-237). (Reference 2.5.1-317) This type of sinkhole is recognized and is likely to develop on the LNP site over a long timeframe as slow dissolution of the carbonate (dolostone) surface occurs.

2.5.1.2.1.3.2.2 Cover-Subsidence Sinkholes

Cover subsidence sinkholes commonly develop where the cover material is relatively cohesion less, permeable, and individual grains of sand move downward to replace other sand grains that have moved to occupy space formerly held by the dissolved limestone (raveling process) (Figure 2.5.1-241). Where the limestone is buried beneath a sufficient thickness of unconsolidated material, few sinkholes develop. Areas where the sand cover is as much as 15 to 30 m (50 to 100 ft.) thick may develop cover subsidence sinkholes that are only a few feet or meters in diameter and depth. (Reference 2.5.1-317) The limited size is due to the fact that the solution cavities in the limestone cannot develop to appreciable size before they are filled with sand. Thousands of cypress heads in west-central Florida occupy depressions formed by cover subsidence sinkholes. (Reference 2.5.1-320) This type of sinkhole occurs east of the LNP site on the Brooksville Ridge (Figure 2.5.1-237).

2.5.1.2.1.3.2.3 Cover-Collapse Sinkholes

Cover-collapse sinkholes occur where a solution cavity develops in the limestone to a size such that the overlying cover material can no longer support its own weight (Figure 2.5.1-242). Cover-collapse sinkholes provide dramatic local changes in topography. They may occur in any area of soluble rocks; however, they are less likely to occur in areas of deeply buried rocks. Cover-collapse sinkholes generally occur in areas where the limestone is near the land surface and the limestone aquifer is not covered by a low permeability confining unit such as the Hawthorn Group. Limestone is commonly exposed in the vertical or overhanging walls of cover-collapse sinkholes shortly after they form. The sinkholes generally are circular in shape and the walls may be round and smooth, but mostly they are irregular in shape because of the influence of joints and fractures in the rock. Surface drainage, erosion, and deposition of sediment into cover-collapse sinkholes will eventually smooth the sides and reduce their slopes until they may become indistinguishable from other types of sinkholes. (Reference 2.5.1-317) Cover-collapse sinkholes are unlikely to develop at the site (see FSAR Subsection 2.5.1.2.5.3)

2.5.1.2.2 Geologic History of Site Vicinity

A detailed discussion of the geologic history of the Florida platform that includes the site vicinity is presented in FSAR Subsection 2.5.1.1.2 and is summarized briefly below. In the site vicinity (40 km [25 mi.] radius), basement rocks have a Gondwanan provenance and were joined to the North American Plate, originally part of the West African continental margin, during the final stages of development of the Appalachian Mountains. (Reference 2.5.1-206) The basement rocks include felsic volcanic rocks, granite, and local higher-grade metamorphic rocks that underlie the Suwannee terrane, an Ordovician to Devonian sedimentary sequence that contains fossil faunas of African affinity. (Reference 2.5.1-222) Thomas et al. (Reference 2.5.1-222) proposed the following geotectonic history of the Suwannee terrane. During the Late Proterozoic, the Suwannee terrane was part of a felsic volcanic province that may have been part of an island arc or backarc basin on the margin of Africa - South America. Following the Pan-African deformation, the region evolved into a shallow shelf in the Cambrian or Early Ordovician. The Suwannee terrane remained stable until at least Middle Devonian, but during the Hercynian orogeny that closed parts of Iapetus, the strata were gently folded. The exact timing at which the Suwannee part of Gondwana docked with Laurasia is still uncertain, and position of the suture (the Suwannee – Wiggins suture) (Figure 2.5.1-211) has not been positively identified.

During the Early Mesozoic, rifting in the Gulf of Mexico and Atlantic Ocean led to normal faulting throughout the Suwannee terrane, especially in the area of the South Georgia basin (Figure 2.5.1-208 and Figure 2.5.1-209), which may have connected spreading centers in the Gulf of Mexico and Atlantic Ocean. At approximately 175 Ma, the spreading center jumped to the Blake Spur anomaly, leaving the Suwannee terrane appended to the North American Plate. (Reference 2.5.1-222)

Beginning in the Cretaceous through to the Holocene, the Florida platform has been tectonically quiescent, allowing a thick sequence of shallow-water marine carbonate deposition to occur within the site vicinity. (Reference 2.5.1-225) In the site vicinity as shown on Figure 2.5.1-243, this sequence of sedimentary rocks in the Humble Oil and Refining Company, Robinson No. 1 well is approximately 1320 m (4331 ft.) thick and lies unconformably upon the eroded surface of the basement rocks.

Periodically, pulses of siliciclastic sediments would migrate onto the platform from the north, temporarily interrupting the carbonate deposition. In the Late Paleogene, a significant siliciclastic depositional event occurred as a result of increased erosion from the Appalachians, Piedmont, and inner Coastal Plain. The influx of the siliciclastics suppressed the carbonate deposition in northern Florida, and by the middle to late Pliocene there was no more carbonate deposition on the platform. (Reference 2.5.1-235)

Sea-level fluctuations throughout the Neogene and Quaternary influenced the sediment deposition and distribution on the Florida platform in the site vicinity. In the Late Oligocene, there was a major sea-level regression that limited deposition to southern Florida and the platform was sub-aerially exposed in the site vicinity. In the Early and Middle Miocene, the sea level rose, covering the entire Florida platform. Sediments deposited on the crest of the Ocala platform during the Miocene were subsequently eroded away. Sea levels fell during the Late Miocene, again exposing most of the platform including the site vicinity. During the Early Pliocene, the sea levels rose, and again most of the platform was submerged. By the Late Pliocene, the sea levels had significantly dropped, exposing much of the platform to sub-aerial exposure. (Reference 2.5.1-235)

The timing and magnitude of the sea-level highstands during the Pleistocene that would have affected the site vicinity are best constrained by the geologic record in southern Florida (Reference 2.5.1-237). The record in southern Florida, which is consistent with well-dated reefs in Barbados, indicates that sea level during the last interglacial period (MIS 5e, approximately 120 ka) was approximately 6 m (20 ft.) above present (Reference 2.5.1-237) and would have inundated much of the western half of the site vicinity (Figure 2.5.1-218). Subsequent deposition of thick siliciclastic sediments (beach and eolian deposits) on the platform has obscured the exact location of the MIS 5e shoreline. During the last glacial maximum (approximately 21 ka), sea level dropped to approximately 120 m (390 ft.) below present. The paleogeography of Florida and the site vicinity would have been very different from present, with the Gulf Coast shoreline considerably farther to the west near the edge of the present shelf break. (Reference 2.5.1-237) Fluvial incision and erosion of the MIS 5e and MIS 5a platforms and overlying sediments would have occurred in response to the sea level lowering.

Following the latest Pleistocene regression, sea level has risen to its present position in the Holocene. Holocene sediments form the present coastline and represent today’s beaches, dunes, marshes, and lagoonal environments. (Reference 2.5.1-235)

2.5.1.2.3 Stratigraphy of Site Vicinity and Site Area

The oldest rocks penetrated within the site vicinity (40 km [25 mi.] radius) and site area (8 km [5 mi.] radius) are Paleozoic shales and quartzite pebble sands that are overlain by Triassic diabase. Overlying these sediments is a thick section of Cretaceous and Cenozoic carbonates (limestone and dolomite) that are overlain by undifferentiated Pleistocene- to Holocene-age surficial sands, clayey sands, and alluvium.

The Eocene to Quaternary geologic history of the Ocala platform is marked by significant erosional episodes. At the LNP site, the Middle Eocene Avon Park Formation lies immediately below Quaternary sediments with the Ocala Limestone, Suwannee Limestone and the Hawthorn Group missing. It has been postulated that these units were deposited over the Ocala platform and subsequently removed by dissolution and mechanical erosion (References 2.5.1-364, 2.5.1-231, 2.5.1-235, and 2.5.1-308). Timing of the erosional episodes is not precisely known.

The contact between the Avon Park Formation and the Ocala Limestone may be unconformable to conformable (Reference 2.5.1-231) and may locally have several meters of relief. The Ocala/Suwannee contact appears conformable with no significant erosion (Reference 2.5.1-231). Both the Ocala Limestone contact and the Suwannee Limestone contact with the Hawthorn Group are unconformable and may exhibit many meters of relief (References 2.5.1-235 and 2.5.1-261). These relationships suggest that some of the sediment removal occurred during the Late Oligocene prior to Hawthorn Group deposition. However, the Hawthorn Group sediments covered the Ocala platform and were removed by post-Middle Miocene erosion (Reference 2.5.1-235). As Hawthorn Group sediments were removed, the Suwannee Limestone and the Ocala Limestone were removed from the crest of the Ocala platform. Subsequently, Quaternary sediments were deposited covering the Avon Park Formation surface and the remnants of the Ocala Limestone and Suwannee Limestone. The weathering of the Ocala and Suwannee Limestones resulted in significant karst development. The Avon Park Formation, which was pervasively dolomitized in the Oligocene (Reference 2.5.1-231), was not heavily karstified.

The following descriptions of the geologic units from Paleozoic through to Early Eocene were summarized from Vernon (Reference 2.5.1-262), and geologic units from Middle Eocene through to the Holocene were taken from Scott (Reference 2.5.1-204) and Arthur et al. (Reference 2.5.1-321) Geologic units included on the State of Florida geological map range from Middle Eocene to Holocene and are shown on Figure 2.5.1-244. A geologic cross section illustrating the Cretaceous geologic units beneath Levy County is shown on Figure 2.5.1-243, and a geologic cross section illustrating the Middle Eocene to Holocene units through the site is shown on Figure 2.5.1-245. A geologic column of the lithostratigraphic units from the Lower Cretaceous through to the Holocene is shown on Figure 2.5.1-214.

2.5.1.2.3.1 Paleozoic and Triassic Rocks

Within the site vicinity, there are three deep wells (Robinson No. 1, J.T. Goethe No. 1, and JB & JT Ragland No. 1) that provide direct information on the deeper stratigraphy near the LNP site (Figure 2.5.1-243). Data on these wells are summarized from Table 7 of Vernon. (Reference 2.5.1-262) Based on these three wells, the top of the Paleozoic rocks in southern Levy County lies at depths between 1207 m (3960 ft.) below sea level (Goethe well) and 1820 to 1771 m (5792 or 5810 ft.) below sea level (Ragland well). Using these three wells and other deep wells from neighboring counties, Vernon (Reference 2.5.1-262) estimated that the peneplane surface on the Paleozoic rocks slopes gently southwest from an elevation of about 1064 m (3500 ft.) below sea level in the northeastern part of Levy County to around 2128 m (7000 ft.) below sea level in southwestern Citrus County — an approximate slope of 1 degree. (Reference 2.5.1-262)

The well closest to the LNP site is the Robinson No. 1 well, drilled in 1949 by Humble Oil and Refining Company and located approximately 500 m (1640 ft.) north of the LNP site (Figure 2.5.1-243). This well was drilled to a depth of 1405 m (4609 ft.) and the top of the Paleozoic surface was at a depth of 1330 m (4377 ft.) The well encountered approximately 71 m (232 ft.) of Paleozoic sediment that was described as light gray, hard, quartzitic sandstone, locally interbedded with thin beds of hard, dark gray to black, somewhat silty, micaceous shale. Based on the fossil assemblage, the Paleozoic rocks are interpreted to be Lower Ordovician. (Reference 2.5.1-262) A Mesozoic diabase was encountered in this well in the depth interval of 1320 to 1334 m (4331 to 4377 ft.). The upper portion of the diabase (1320 to1324 m [4331 to 4344 ft.]) was altered and was described as being quite calcareous. Serpentine and alteration products were common, and portions of the weathered material were thought to be reworked and included in the overlying light greenish, soft sandstone of Mesozoic age. In thin section, the diabase was greenish black in color and was largely composed of thin laths of plagioclase feldspar, augite, and a rhombic pyroxene, enstatite, with interstitial quartz and needles of apatite. The lower portion of the diabase had many inclusions of a black, waxy, clay-like substance and calcite occurring as veins and as alteration of silicates along the veins. (Reference 2.5.1-262)

The J.T. Goethe No. 1 well, approximately 16 km (10 mi.) north of the Robinson well, was drilled in 1946 by Sun Oil Company and is located approximately 24 km (15 mi.) north of the LNP site (Figure 2.5.1-243). This well was drilled to a depth of 1215 m (3997 ft.) and the top of the Paleozoic surface was 1204 m (3960 ft.) below sea level. (Reference 2.5.1-262) The well encountered approximately 11 m (37 ft.) of gray quartzitic sandstone and micaceous black shales, similar to material that was cored in the Robinson well. Most of the deep wells in the adjoining counties also penetrated a similar rock. One well, Sun Oil Company's Perpetual Forest No. 1 well located in Dixie County (Section 5, Township 11 South, Range 11 East), encountered approximately 684 m (2250 ft.) of the gray quartzitic sandstone without identifying the bottom of the unit. Based on this information, the Paleozoic section underlying Levy and Citrus counties is of considerable thickness. (Reference 2.5.1-262)

The third well is the JB & JT Ragland No. 1 well, drilled in 1947 by Coastal Petroleum Company and located approximately 48 km (30 mi.) west of the LNP site (Figure 2.5.1-243). This well was drilled to a total depth of 1778 m (5850 ft.) and the top of the Paleozoic surface was either 1766 m or 1761 m (5810 or 5792 ft.) below sea level. (Reference 2.5.1-262) The well encountered approximately 12 m (40 ft.) of soft, laminated, somewhat silty, finely micaceous and pyritic, very dark gray to black shale, of which only 3 m (10 ft.) were cored. This same shale was identified in another oil well (Humble Oil and Refining Company's J.P. Cone No. 1 well) sited in northern Columbia County, which encountered 292 m (962 ft.) of lithologically similar black shale without passing through it. Based on the fossil assemblage from the shale, it was estimated to be either Upper Silurian or Lower Devonian. Overlying the black shale at approximately 1765 to 1770 m (5792 to 5810 ft.) is a yellow, gray, lavender, and pink variegated, well-stratified shale that is in turn overlain by a conglomerate of quartzite pebbles that is thought to be Early Mesozoic in age. (Reference 2.5.1-262)

2.5.1.2.3.2 Lower Cretaceous Rocks

The distribution and relationship of the Cretaceous geologic units beneath Levy County is illustrated on a cross section shown on Figure 2.5.1-243. This cross section illustrates the Lower Cretaceous sand and shale units overlain by extensive Upper Cretaceous carbonate sediments that are present in the site area.

Only four wells in Levy County — and none in Citrus County — penetrate the Lower Cretaceous. Based on cuttings and cores collected from these wells, the sediments are variegated red, green, and brown clastics, largely sands and shales. The thicknesses of the Lower Cretaceous in the wells range from 426 m (1435 ft.) in the Ragland No. 1 well (from 1321 to 1758 m [4347 to 5782 ft.]), to 9 m (31 ft.) in Robinson No. 1 well (from 1307 to 1317 m [4300 to 4331 ft.]), to 20 m (67 ft.) in the Goethe No. 1 well (from 1186 to 1207 m [3893 to 3960 ft.]). (Reference 2.5.1-262)

The sediments of the Lower Cretaceous are interbedded shales and sandstones. The shales are variegated (red, purple, green, brown, and gray), mottled, waxy and fissile, and contain pink calcareous nodules, yellow and pink argillaceous sand lenses and sand grains of rose quartz. The sandstone is variegated (light greenish to yellowish gray, brown, and white), calcareous and siliceous, and contains black siliceous nodules, rose quartz sand grains, seams of sandy shale, and lenses of quartz conglomerate. The basal part of the sediments is coarse-grained and sands contain pebbles of Paleozoic sediments. (Reference 2.5.1-262)

2.5.1.2.3.3 Upper Cretaceous Rocks

Beds of the Upper Cretaceous lie unconformably upon older beds, and in Levy and Citrus counties, individual beds of the series appear to be conformable with each other. (Reference 2.5.1-262)

2.5.1.2.3.3.1 Atkinson Formation

The oldest Upper Cretaceous unit, the Atkinson Formation, was penetrated in four wells in Levy County. The formation is fairly uniform in thickness, ranging from 73 m (240 ft.) in the Goethe No. 1 well and 74 m (244 ft.) in the Sholtz (Cedar Keys) No. 2 well, to as much as 112 m (368 ft.) in the Robinson No. 1 well close to the LNP site. (Reference 2.5.1-262)

The Atkinson Formation is subdivided into an “A” zone and a “B” zone, based largely on fossil assemblages. In western Levy County, Vernon (Reference 2.5.1-262) describes the A zone as being “composed of medium gray to greenish gray, calcareous, micaceous shale containing seams of argillaceous limestone and variegated, micaceous, glauconitic, pyrite and carbon-flecked sandstone. Eastward, the A zone thickens from 58 to 76 m (190 to 250 ft.) and is composed of interbedded, light brownish to medium gray, sandy, dense, hard, shaley limestone with thin seams of sandstone and flecks of lignite; greenish gray, poorly sorted, slightly calcareous sandstone; and purple, blocky, micaceous shale.” Within southern Levy County, the A zone is encountered at the following depths: Robinson No. 1 well, 1196 – 1270 m (3932 – 4178 ft.); Goethe No. 1 well, 1111 – 1169 m (3653 – 3845 ft.); and Ragland No. 1 well, 1253 – 1290 m (4121 – 4243 ft.). (Reference 2.5.1-262)

The B zone in Levy County is described by Vernon (Reference 2.5.1-262) as a “gray, micaceous, calcareous sand that overlies a dark gray, fissile, calcareous shale and shaley limestone, containing thin seams of gray to cream, shaley limestone, flecks of lignite and traces of gypsum. Medium gray, calcareous, quartz sandstone with thin shale partings and a coarse sand and gravel conglomerate are irregularly interbedded with the shale.” Within southern Levy County, the B zone is encountered at the following depths: Robinson No. 1 well, 1270 – 1307 m (4178 – 4300 ft.); Goethe No. 1 well, 1169 – 1183 m (3845 – 3893 ft.); and Ragland No. 1 well, 1290 – 1321 m (4243 – 4347 ft.). (Reference 2.5.1-262)

2.5.1.2.3.3.2 Beds of Austin Age

Five wells in Levy County penetrated Austin aged beds with the thickness of these beds being 146 m (480 ft.) in the Robinson No. 1 well, 149 m (490 ft.) in the Goethe No. 1 well, and 160 m (527 ft.) in the Ragland No. 1 well. The Austin equivalent section in Levy County is divided into three subunits. Vernon (Reference 2.5.1-262) describes the upper 61 – 91 m (200 – 300 ft.) as being light green, cream, tan and gray, rather tight, shaley chalk that contains medium gray marl and shale seams. The middle 30 – 61 m (100 – 200 ft.) is light gray, speckled, fairly dense chalk. The basal 30 – 46 m (100 – 150 ft.) is a gray to cream, dense chalk with a flaky fracture and thick beds and seams of calcareous shale; gray, calcareous sand; and lignitic shale. Within southern Levy County, the beds of Austin age are encountered at the following depths: 1049 – 1195 m (3452 – 3932 ft.) in Robinson No. 1 well; 962 – 1111 m (3163 – 3653 ft.) in Goethe No. 1 well; and 1093 – 1253 m (3594 – 4121 ft.) in Ragland No. 1 well. (Reference 2.5.1-262)

2.5.1.2.3.3.3 Beds of Taylor Age

The Taylor beds section in Levy County is thick and consistent, composed of white to cream chalk separated by thin beds and seams of tan crystalline dolomite. Interspersed through the chalk beds are thin, 0.3 to 0.61 m (1 to 2 ft.) clay or ash beds. These thin beds are widely developed throughout the peninsula and are used as correlation beds for the Upper Cretaceous. The Taylor equivalent is present at depths of between 845 to 1049 m (2780 to 3452 ft.) in the Robinson No. 1 well, 771 to 962 m (2535 to 3163 ft.) in the Goethe No. 1 well, and 879 to 1092 m (2892 to 3594 ft.) in the Ragland No. 1 well. (Reference 2.5.1-262)

2.5.1.2.3.3.4 Lawson Limestone (Navarro Equivalent)

In Levy County, the Lawson Limestone is separated into an upper and lower member, each having a distinctive fauna. The upper member is a cream-colored, fragmental marine limestone with gypsum lenses and porosity impregnation. The lower member is commonly cream-to-white, pasty, marine chalk and fragmental limestone.

The top of the upper Lawson is marked by a very definite and characteristic cream-colored, porous, granular, sub-oolitic, marine, fragmental dolomite. The upper few feet are extremely fossiliferous, loosely cemented together, and greatly altered by dolomitization. The lower Lawson is a pasty to fragmental marine, chalky limestone that is commonly dolomitized. Gypsum, which is rare in the lower Lawson, is rare to common throughout the upper Lawson, occurring as impregnations of the porosity. Some carbonaceous partings are present in the lower Lawson, but these are absent in the upper part of the unit. In general, the formation thins from west to east in Levy County, being 186 m (612 ft.) thick in the Ragland No. 1 well and only 105 m (345 ft.) thick in the Goethe No. 1 well. (Reference 2.5.1-262)

The lower Lawson Limestone is present from depths of 736 to 845 m (2420 to 2780 ft.) in the Robinson No. 1 well, 693 to 771 m (2280 to 2535 ft.) in the Goethe No. 1 well, and 746 to 879 m (2453 to 2892 ft.) in the Ragland No. 1 well. The upper Lawson Limestone is present from depths of 696 to 736 m (2290 to 2420 ft.) in the Robinson No. 1 well, 666 to 693 m (2190 to 2280 ft.) in the Goethe No. 1 well, and 693 to 746 m (2280 to 2453 ft.) in the Ragland No. 1 well. (Reference 2.5.1-262)

2.5.1.2.3.4 Paleocene Rocks — Cedar Keys Formation

The Cedar Keys Formation of Levy County is composed of interbedded tan to gray, finely granular, fragmental, commonly very fossiliferous limestone and tan to brown, finely crystalline to chalky textured dolomite. Gypsum has completely filled the porosity of large sections and occurs irregularly as thin lenses. The limestone is rarely fossiliferous near the top and is dolomitized. The lower portion is more fossiliferous and some beds are completely composed of various microfaunas in a pasty limestone matrix. (Reference 2.5.1-262)

The thickness of the Cedar Keys Formation in Levy County ranges from 182 m (600 ft.) near the coast to 164 m (540 ft.) near the LNP site. The Cedar Keys Formation is present from depths of 532 to 696 m (1750 to 2290 ft.) in the Robinson No. 1 well, 504 to 666 m (1658 to 2190 ft.) in the Goethe No. 1 well, and 511 to 693 m (1680 to 2280 ft.) in the Ragland No. 1 well. (Reference 2.5.1-262)

2.5.1.2.3.5 Lower Eocene Rocks — Oldsmar Limestone

The Oldsmar Limestone is not lithologically unlike the overlying and underlying formations. It was originally differentiated based on a specific faunal zone. It is generally composed of fragmental marine limestones, partially to completely dolomitized and containing irregular and rare lenses of chert, impregnations of gypsum, and thin shale beds. Fossils are common but are sometimes dolomitized and poorly preserved. The thickness of the Oldsmar Limestone is variable, depending on the extent of the dolomitization. The formation or faunal zone is 116 m (380 ft.) thick near the coast and 152 m (500 ft.) thick at the LNP site. The Oldsmar Limestone is present in southern Levy County from depths of 380 to 532 m (1250 to 1750 ft.) in the Robinson No. 1 well, 287 to 504 m (945 to 1658 ft.) in the Goethe No. 1 well, and 395 to 511 m (1300 to 1680 ft.) in the Ragland No. 1 well. (Reference 2.5.1-262)

2.5.1.2.3.6 Middle Eocene Rocks — Avon Park Formation

The Middle Eocene Avon Park Formation (Tap) represents the oldest rocks exposed in Florida and underlies all of peninsular Florida (Reference 2.5.1-204) (Boring W-15075 on Figure 2.5.1-245). The top of the Avon Park Formation varies in depth from surface outcrop in southern Levy County, northern Citrus County, and along the crest of the Ocala platform, to nearly 46 m (150 ft.) deep in northern and Eastern Levy County. (Reference 2.5.1-308) Rupert (Reference 2.5.1-308) states that there are oil test wells that have penetrated the entire Avon Park Formation under Levy County. These test wells reveal a total thickness for this unit of approximately 243 to 304 m (800 to 1000 ft.), although no specific test well is referenced. (Reference 2.5.1-308)

Scott (Reference 2.5.1-204) described the Avon Park Formation as cream to light brown or tan, poorly indurated to well-indurated, variably fossiliferous, limestone (grainstone, packstone, and wackestone, with rare mudstone). These limestones are interbedded with tan to brown, very poorly indurated to well-indurated, very fine to medium crystalline, fossiliferous (molds and casts), vuggy dolostones. The fossils present include mollusks, foraminifers, echinoids, algae, and carbonized plant remains. (Reference 2.5.1-204) Accessory minerals include chert, pyrite, and gypsum, with gypsum becoming more abundant with depth. (Reference 2.5.1-321)

According to Arthur et al. (Reference 2.5.1-321), the uppermost part of the Avon Park Formation within Levy, Citrus, and Marion counties varies between limestone and dolostone; dolostone predominates deeper within the unit, especially southward from these counties. Porosity in this formation is generally intergranular in the limestone. Fracture porosity occurs in the more densely recrystallized dolostone, and intercrystalline porosity is characteristic of sucrosic texture. Pinpoint vugs and fossil molds are present to a lesser extent. (Reference 2.5.1-321)

The Avon Park Formation unconformably overlies the Lower Eocene Oldsmar Limestone. Miller (Reference 2.5.1-240) states that the top of Lower Eocene rocks (the approximate base of the Avon Park Formation) is at approximate elevations ranging from -334 to -562 m (-1100 to -1850 ft.) NGVD29. The Avon Park Formation varies in thickness across the study area (Levy, Citrus, Marion, Sumter, Hernando, Pasco, and Polk) from 304 m (1000 ft.) in Levy County to 456 m (1500 ft.) in Pasco County (Reference 2.5.1-240). In the Arthur et al. (Reference 2.5.1-321) study area, the top of the Avon Park Formation ranges from approximately 3 m (10 ft.) above NGVD29 to an elevation of -129 m (-425 ft.) NGVD29. Several units in that study area unconformably overlie the Avon Park Formation, including the Ocala Limestone, the Hawthorn Group (undifferentiated), and undifferentiated sands and clays. (Reference 2.5.1-321)

2.5.1.2.3.7 Upper Eocene Rocks — Ocala Limestone

The Upper Eocene Ocala Limestone (To) consists of nearly pure limestone and occasional dolostone. It is commonly subdivided into lower and upper facies based on lithology. The lower member is composed of a white to cream-colored, fine- to medium-grained, poorly to moderately indurated very fossiliferous limestone (grainstone and packstone). The lower member may not be present throughout the areal extent of the Ocala Limestone and may be partially to completely dolomitized in some regions. The upper member is a white, poorly to well-indurated, poorly sorted, very fossiliferous limestone (grainstone, packstone, and wackestone). Chert is common in the upper member. Fossils present in the Ocala Limestone include abundant large and smaller foraminifers, echinoids, bryozoans, and mollusks. The presence of large foraminifers in the upper member is a characteristic feature of the upper unit. (Reference 2.5.1-204)

The Ocala Limestone typically is bound by unconformities. Elevations to the top of the formation range from land surface to –87 m (–285 ft.) NGVD29. Subcrop extent of the Ocala Limestone is widespread except for southeast Levy and southwest Marion counties. Arthur et al. (Reference 2.5.1-321) observed that the Ocala Limestone obtains a maximum thickness of 70 m (230 ft.) and is typically identified on the flanks of the Ocala platform, which trends south-southeast across their study area.

2.5.1.2.3.8 Lower Oligocene Rocks — Suwannee Limestone

The Lower Oligocene Suwannee Limestone (Ts) crops out on the northwestern, northeastern, and southwestern flanks of the Ocala platform, and due to erosion and/or nondeposition it is absent from the eastern side of the Ocala platform. (Reference 2.5.1-204) Scott (Reference 2.5.1-204) describes the Suwannee Limestone as a white to cream, poorly to well-indurated, fossiliferous, vuggy-to-moldic limestone (grainstone and packstone). The dolomitized parts of the Suwannee Limestone are gray, tan, light brown to moderate brown, moderately to well indurated, finely to coarsely crystalline dolostone having limited occurrences of fossiliferous (molds and casts) beds. Chert is common in the Suwannee Limestone. Fossils present in this unit include mollusks, foraminifers, corals, and echinoids. (Reference 2.5.1-204)

The Suwannee Limestone unconformably overlies the Ocala Limestone and is unconformably overlain either by Hawthorn Group units or undifferentiated sediments. In southern counties of the Arthur et al. (Reference 2.5.1-321) study area (Sumter, Hernando, Pasco, and Polk counties), the Suwannee Limestone is less than 6 m (20 ft.) bgs. Their study also documented that the northern extent of the Suwannee Limestone occurs in southern Citrus County, and thus is not expected to be present within the LNP site area. Elevations of the Suwannee Limestone within their study area range from –24 m (–80 ft.) NGVD29 to 40 m (132 ft.) NGVD29, and the unit thickens to the south and west, ranging up to 68 m (225 ft.) thick. (Reference 2.5.1-321)

2.5.1.2.3.9 Lower Oligocene to Pliocene Rocks — Hawthorn Group

The Hawthorn Group sediments range in age from Lower Oligocene to Lower Pliocene and generally consist of siliciclastics (sands, silts, and clays) and carbonates. The Hawthorn Group within the site area consists of the Tampa Member (That) of the Arcadia Formation (Tha). The Hawthorn Group sediments lie unconformably above the Suwannee Limestone, Ocala Limestone, or the Avon Park Formation within the site vicinity. (Reference 2.5.1-321)

In the site vicinity, the top of the Hawthorn Group ranges from approximately sea level to 61 m (200 ft.) NGVD29, and is up to 44 m (145 ft.) thick. These sediments are observed within the Brooksville Ridge that trends north-northwest and lies just east of the LNP site, suggesting that this upland is underlain by an erosional remnant of the Hawthorn Group. (Reference 2.5.1-321)

In the site vicinity, all observed occurrences of the Arcadia Formation (Tha) are part of the Tampa member (That). The Tampa Member of the Arcadia Formation is described by Scott (Reference 2.5.1-204) as consisting predominantly of limestone with subordinate dolostone, sand, and clay. The lithology of the Tampa member is very similar to that of the subsurface limestone portion of the Arcadia Formation except that the Tampa member contains less phosphate. Scott

(Reference 2.5.1-204) describes the Arcadia Formation carbonates as having a variable siliciclastic component, including thin beds of siliciclastics. Where this unit outcrops, it is composed of yellowish gray to light olive gray to light brown, micro- to finely crystalline, variably sandy, clayey, and phosphatic fossiliferous limestone and dolostones. The limestone in the Tampa member is white to yellowish gray, fossiliferous and includes variably sandy and clayey mudstone, wackestone and packstone with minor to no phosphate grains. Sand and clay beds are like those in the undifferentiated Arcadia Formation. The sands are yellowish gray, very fine- to medium-grained, poorly to moderately indurated, clayey, dolomitic, and phosphatic. The clays are yellowish gray to light olive gray, poorly to moderately indurated, sandy, silty, phosphatic, and dolomitic. Mollusks and corals are common in the Tampa member as molds and casts, silicified pseudomorphs, and original shell material. (Reference 2.5.1-204)

The top of the Tampa member ranges in elevation from zero to 26 m (85 ft.) NGVD29, and is up to 24 m (80 ft.) thick The Tampa member lies beneath the undifferentiated sands and clays and the Hawthorne Group (undifferentiated), and is underlain by the Suwannee Limestone. (Reference 2.5.1-321)

2.5.1.2.3.10 Pleistocene to Holocene Rocks

The Post-Hawthorn Group sediments within the site vicinity consist of undifferentiated sands and clays. According to Arthur et al. (Reference 2.5.1-321), these undifferentiated sediments primarily comprise varying proportions of sand and clay and appear thickest in areas where sediments accumulated from infilling of karst features. These karst features, which are probably sinkholes, are most commonly observed in the northern part of their study area (northern Levy County). Variable amounts of chert, organics, and reworked phosphate also occur in the undifferentiated sediments. (Reference 2.5.1-321)

A white-to-gray, fossiliferous freshwater marl commonly occurs along the banks and in the valleys of the Withlacoochee and Suwannee Rivers. This marl typically contains an abundant Holocene freshwater mollusk fauna and may reach thicknesses of 0.9 to 1.2 m (3 to 4 ft.). Holocene quartz sand and mud alluvium form bars and line the valley floors of most major streams in Levy County. (Reference 2.5.1-308)

2.5.1.2.3.11 Surficial Geology of the Site Vicinity

The surficial geologic map of the central and eastern United States (Reference 2.5.1-242) depicts surficial materials that accumulated in the last two million years. Five surficial units have been mapped in the site vicinity (Figure 2.5.1-246). The following are descriptions of the units taken from regional map descriptions of Fullerton et al. (Reference 2.5.1-242)

The unit mapped at the site consists of sand, silt, and smectitic-clay decomposition residuum (zp), which is generally 1 – 2 m (3.2 – 6.5 ft.) thick. The map unit includes areas of eolian sand and locally derived colluvium and alluvium. Sinkholes and other karst phenomena associated with the underlying limestone bedrock are common.

Beach and nearshore deposits (mb) are mapped approximately 1 km (0.6 mi.) east of the site and are composed primarily of sand in narrow or broad linear ridges or sand underlying linear tracts or flats. The thickness of the beach and nearshore deposits generally is 1 – 10 m (3 – 33 ft.), and locally is 12 – 25 m (39 – 82 ft.). The ridges represent relict beaches, spits, offshore bars, and barrier islands 2 – 36 m (6.5 – 118 ft.) above the present sea level. The deposits in some areas are pebbly sand, gravelly sand, silty fine sand, or clayey silt. Deposits that are Late Pleistocene in age typically are intensely oxidized and are leached to a depth of approximately 4 m (13 ft.); calcareous shell debris is present only at greater depths. Deposits that are Middle Pleistocene in age and older in some places are leached throughout (to depths >10 m [33 ft.]), are deeply weathered, and locally are cemented by secondary iron oxides. In some areas in Florida, the deposits are phosphatic sand, clayey sand, and sandy clay, which in some places fills karst depressions along limestone ridges; the phosphatic deposits commonly are intensely stained and cemented by iron oxides. In other areas in Florida, leached and intensely oxidized quartz sand is underlain at depth by consolidated sand that contains abundant whole, but rotted, shells. Leaching of shell material from the upper part of the deposits resulted in reduction in thickness of as much as 1.5 m (5 ft.); the dissolved carbonate in some places was redeposited, with compacted insoluble clay mixed with quartz sand, as a subsurface hardpan.

Approximately 8 km (5 mi.) west of the site along the coast are mangrove-swamp deposits (ha), that consist of organic muck, calcitic mud, and woody debris that are intermittently covered by as much as 1 m (3 ft.) of seawater. The swamp deposits overlie mud- and wave-beveled surfaces cut on soft, porous limestone. The thickness of the swamp deposits generally is 0.5 – 1 m (1.6 – 3.2 ft.). The deposits are mapped only in areas that support dense stands of both red and black mangrove.

Also mapped within the site vicinity is a unit of sandy solution residuum (re) that is generally 1 – 3 m (3.2 – 10 ft.) thick. It is composed chiefly of quartz sand or calcareous sand on soft sandy limestone and shell-hash limestone. It is limonitic in some areas and includes some eolian sand and colluvium. Clay-filled sinkholes and other karst phenomena are common.

Decomposition residuum on sand or mixed-composition sand and gravel on upland surfaces (zc) is mapped to the southeast of the site and is generally 1 – 3 m (3.2 – 10 ft.) thick. It occurs on fluvial or marine deposits of different ages at several topographic positions in the landscape, generally on broad drainage divides and upland surfaces. The residuum tends to be thicker and more intensely weathered on older (topographically higher) deposits, and thinner and less intensely weathered on younger (topographically lower) deposits. In all regions, the map unit includes some colluvium, sheetwash alluvium, and other residual materials. It is chiefly quartz sand or micaceous sand that locally grades downward to clay and in some places contains lenses of silt and gravel.

More detailed description of the stratigraphy and geology of the site location is provided in FSAR Subsection 2.5.1.2.5.

2.5.1.2.4 Structural Geology of Site Vicinity and Site Area

Recent geologic and structure contour maps on Tertiary horizons (References 2.5.1-366, 2.5.1-367, 2.5.1-368, 2.5.1-369, and 2.5.1-370) show only one structural feature, the Ocala platform, and no Quaternary faults or even Cenozoic faults within the site vicinity (40 km [25 mi.]). No known structures were observed within the site location (1 km [0.6 mi.]).

The Ocala platform (previously referred to as an uplift) was described by Vernon (Reference 2.5.1-262) as a plunging anticline, approximately 370 km (230 mi.) long and about 113 km (70 mi.) wide, where visible at the ground surface in central peninsular Florida. The regional dips of the Tertiary beds that make up the Ocala platform are southwest and northeast along the flanks and northwest and southeast along the plunge. The regional dips are approximately 1.7 m per km (9 ft. per mi.) on the flanks and 0.57 m per km (3 ft. per mi.) along the plunge. (Reference 2.5.1-262) The location of the general central axis of the Ocala platform is shown on Figure 2.5.1-244.

The general consensus regarding the origin of the Ocala platform based on more recent mapping, as described in FSAR Subsection 2.5.1.1.4.3.4, is that differential subsidence, sedimentation, and erosion have created the dip patterns associated with the Ocala platform (References 2.5.1-371, 2.5.1-372, and 2.5.1-373). Another factor that may be affecting the development of the Ocala platform and occurrence of fractures is the concept of isostatic readjustment of the crust related to dissolution of carbonate and associated reduction of the weight of the crust. This effect also could lead to broad uplift. (Reference 2.5.1-371)

As noted above in FSAR Subsection 2.5.1.2.3.1, the top of the basement surface appears to slope gently (1 degree) to the southeast across the site area. (Reference 2.5.1-262).

2.5.1.2.4.1 Fractures (Joints)

A number of fracture sets and orthogonal fracture systems have been described for the LNP site vicinity (40 km [25 mi.]). Features inferred to be faults are described in FSAR Subsection 2.5.1.2.4.2. On a regional basis the orientations of fractures are inferred from analysis of lineaments using various types of remote sensing data as outlined in FSAR Subsection 2.5.3.2.1.1. Direct observations of fractures (joints) in bedrock exposures provide the most reliable information on the orientation of fractures (joints) and fracture systems that are likely to be present at the LNP site.

Vernon (Reference 2.5.1-262) mapped lineaments on a regional basis from physiographic expressions as shown on mosaics and contact prints of aerial photographs and interpreted them as faults and fractures. Vernon identified two sets of fractures: a primary set with a generally northwest trend, and a secondary set trending northeast. The two sets of fractures intersect at broad or nearly right angles and are spaced approximately 30 to 50 km (20 to 30 mi.) apart, forming a roughly rectangular pattern. Within the LNP site vicinity fractures mapped by Vernon are spaced 1.5 to 3 km (1 to 2 mi.) apart. The primary set parallels the axis of the Ocala platform, whereas the secondary set trends in the approximate direction of dip of the flanks of the Ocala platform. (Reference 2.5.1-322) Vernon (Reference 2.5.1-262) observed that the regional fracture pattern is consistent with some stream patterns and sinkhole alignments. Particularly well developed joints or faults (as interpreted by Vernon [Reference 2.5.1-262]) are shown along portions of the Ocklawaha, Withlacoochee, and Kissimmee rivers, all of which show strongly developed rectangular patterns trending northeast southwest and northwest southeast with large angle turns. (Reference 2.5.1-262)

Vernon concluded that the fractures formed under a combination of tensional stresses related to anticlinal flexure during the development of the postulated Ocala Uplift. Mapping of lineaments statewide suggest that the fracture sets described by Vernon (Reference 2.5.1-262) based on photogeologic interpretation are not unique to the Ocala platform and may reflect more regional stresses.

Two major (primary) and two minor (secondary) orthogonal fracture systems were mapped in the Crystal River plant excavation approximately 13.7 km (8.5 mi.) southwest of the LNP site. Construction observations showed that the entire foundation system of the Crystal River site contains near vertically oriented fracture zones. One primary fracture system consists of fractures oriented N 45ºW with cross fractures perpendicular to this regional trend. The second primary fracture system consists of a north south trend with cross fractures trending east west. Two secondary fracture systems that were observed during excavation are oriented N60ºW – N30ºE and N30ºW – N60ºE.

Direct observations of fracture orientations and spacing in Avon Park formation bedrock exposed at the Gulf Hammock quarry and along the Waccasassa River at distances of 19 km (11.8 mi.) and 25 km (15.7 mi.) north-northwest of the LNP site, respectively, provide information on fracture patterns and amounts of dissolution in dolomitized bedrock units comparable to the bedrock at the LNP site.Orthogonal fracture sets observed at the Wacasassa River and Gulf Hammock Quarry exposures exhibited general orientations of N39W and N51E. Less prominent fractures with orientations of N-S and E-W were also noted. Fracture spacing of approximately 6 to 8 m (20 to 25 ft.) was observed at the Wacasassa River and Gulf Hammock Quarry exposures respectively.

2.5.1.2.4.2 Postulated Faults

Postulated faults in the site vicinity and site area include basement faults identified by Barnett (Reference 2.5.1-239) and Cenozoic faults identified by Vernon (Reference 2.5.1-262).

Postulated Basement Faults. Figure 2.5.1-254 shows the locations of postulated basement faults relative to wells that penetrate basement rock in the site vicinity (40-km [25-mi.] radius). Table 2.5.1-204 summarizes available data obtained for these wells. The location of the postulated faults based on the sparse well control is speculative. The geologic data presented by Barnett does not provide strong evidence to support the interpretation of the postulated west-northwest–trending basement fault that intersects the site area (8-km [5-mi.] radius) and possibly the site location (1-km [0.6-mi.] radius). As shown on Figure 2.5.1-253, the inferred geologic contact between Ordovician sediments and Devonian and Silurian sediments extends across the fault with no apparent displacement.

The dip of the surface defined by the top of pre-Middle Jurassic basement rocks encountered in wells W-2012 (north of the postulated fault), W-7538 and W-7534 is 0.7 degrees to the south-southwest. The dip of the surface defined by the top of pre-Middle Jurassic basement rocks in wells W-2012, and two wells to the north of the postulated fault (W-1007 and W-8035) is 0.5 degree to the southwest. These data and observations do not require a significant step in the top of basement as would be expected if a normal basement fault with appreciable displacement were present. Although the available data do not preclude the presence of a fault in the sub-Zuni erosional surface basement rocks as inferred by Barnett, the rationale for the location as shown is not readily apparent from the data provided by Barnett (Reference 2.5.1-239).

The postulated basement faults are not apparent in either gravity or magnetic anomaly maps of the site vicinity (Figures 2.5.1-255 and 2.5.1-256). The magnetic anomaly maps show a strong northeast- to east-northeast-trending pattern that is at a high angle to Barnett’s postulated faults.

Results from an analysis of Bouguer gravity data by Coleman (Reference 2.5.1-336) for a study region extending from northern Levy County to Taylor County (Figure 2.5.1-256) provide additional data that can be used to evaluate the location and orientation of Barnett’s postulated basement structures. Figure 2.5.1-257 shows the locations of proposed fault traces from Coleman’s analysis superimposed on the Bouguer anomaly map. The strongly linear zones of high gravity gradients are inferred to be subcrop fault traces. Two linear, northeast-trending basins are inferred from the gravity data. The more prominent zones trend northeast and parallel the proposed trend of major normal faults in the Southwest Georgia embayment. Coleman concludes that these basins and the Southwest Georgia embayment may have developed in response to the same regional stress field in early Mesozoic time. The results of Coleman’s study do not support the interpretation of a northwest-trending fault truncating the northeast-trending structures as shown by Barnett (Reference 2.5.1-239).

Arthur (Reference 2.5.1-335) states that the influence of “basement” structures on Cenozoic and younger stratigraphic units is poorly understood and cites inconsistencies in the mapping of postulated structures. Minimum age constraints for pre-Middle Jurassic basement structures can be inferred from maps recently developed for the FGS by Arthur et al. based on the most current lithologic information available. These studies incorporated information from mapping of surface geology with interpretation of subsurface information, primarily water well and petroleum exploration well data, to develop structure contour maps on various datums. The surfaces shown on these maps include the Avon Park Formation (Eocene), Ocala Limestone (Eocene), and Suwannee Limestone (Oligocene). Discontinuities or anomalies that would suggest displacements of these surfaces by faulting were not identified in the LNP site vicinity (40-km [25-mi.] radius).

In summary, observations and conclusions from the review of data pertaining to the postulated basement faults proposed to be present in the site vicinity (40-km [25-mi.] radius) are as follows:

- There is no discussion of the rationale or criteria used by Barnett (Reference 2.5.1-239) to identify the locations of specific pre-Middle Jurassic basement (sub-Zuni erosional surface) faults in the LNP site vicinity. No discussion or description of the postulated basement fault mapped within the LNP site area is presented by Barnett.

- The sub-crop map of sub-Zuni surface provided by Barnett (Figure 2.5.1-253) shows no apparent offset of contacts between Paleozoic bedrock units across the postulated fault that lies within the LNP site area (8-km [5-mi.] radius).

- The general dip (slope) of the erosional unconformity in the site location based on the top of basement in wells ranges from 0.5 to 0.7 degree in a south-southwest direction.

- The available well data do not clearly indicate faults as mapped by Barnett, but neither do the data preclude the possibility of basement faults in the LNP site area.

- The postulated basement faults as mapped by Barnett (Reference 2.5.1-239) are not expressed in gravity or magnetic anomaly maps. The major trend shown by these data, particularly the magnetic data, is northeast to east-northeast.

- Interpretation of detailed gravity data by Coleman (Reference 2.5.1-336) for a study area that crosses the more northwest-trending basement faults inferred to be present within the LNP site vicinity, did not confirm the locations of these faults as mapped by Barnett (Reference 2.5.1-239).

- Based on the above observations, the postulated basement faults in the site vicinity as shown by Barnett (Reference 2.5.1-239) are highly speculative. The faults, if they exist, do not show evidence for post- Middle Jurassic (sub-Zuni surface) activity. Structure contour maps for overlying Cenozoic units do not show evidence for faults (Reference 2.5.1-335).

Postulated Cenozoic Faults. Vernon (Reference 2.5.1-262) interpreted seven "well-developed" northwest-trending faults in a geologic section along the Levy – Citrus county line into Marion County. Vernon claimed that each of the faults could be readily traced on aerial photographs, on which they appeared as "continuous trough-like depressions and ridges marked by significant vegetation changes and soil colorations." Vernon cited field evidence including apparent displacements inferred from outcrop and subsurface data from boreholes and wells, and limited field observations due to the minimal exposures along the fault traces. (Reference 2.5.1-262) Four of the seven postulated faults and two small domal structures are located within the LNP site vicinity (Figure 2.5.1-244). These postulated features and the evidence cited by Vernon (Reference 2.5.1-262) for their existence are briefly summarized below.

Bronson Graben. The Bronson Graben is located approximately 24 km (15 mi.) northeast of the LNP site. Vernon's (Reference 2.5.1-262) field evidence for the Bronson Graben was based on studies of outcrops in which the Ocala Limestone was identified within the graben and older Eocene beds that border the graben over much of its extent, apparently end abruptly along its borders. Vernon noted that the Moodys Branch Formation is not present in a borehole that reached a depth of –26.5 m (–87 ft.) near the town of Bronson, but is present in nearby areas at 12 to 15 m (40 to 50 ft.) amsl.

Inverness Fault. The northern end of the Inverness fault is located within the 8 km (5 mi.) radius of the LNP site. Vernon's (Reference 2.5.1-262) field evidence for the Inverness fault is based in part on outcrops of the Inglis member of the Moodys Branch Formation located east of the fault. These exposures lie at elevations of 8 m to 15 m (28 to 50 ft.) amsl, whereas five wells located 3 km (2 mi.) to the southwest of the fault, indicate that the Inglis member lies at elevations ranging from –0.3 m in the south to 11 m in the north (–1 to 37 ft.). Vernon (Reference 2.5.1-262) also stated that numerous exposures of the Williston member of the Moodys Branch Formation, the Ocala Limestone, and the Suwannee Limestone on the hills southwest of the fault also indicate comparable displacements. Based on this field evidence, Vernon (Reference 2.5.1-262) concluded that the northeast block had been tilted by faulting and the southeast side had been upthrown with a displacement of as much as 15 m (50 ft.), whereas the northwest portion was downthrown with displacements of as much as 6 m (20 ft.).

Long Pond Fault. The southern end of the Long Pond fault is located approximately 10 km (6.2 mi.) northeast of the LNP site. Vernon (Reference 2.5.1-262) described the Long Pond fault as the major fault for the area and suggested as much as 48.7 m (160 ft.) of displacement along the fault. Claiming that the fault is well exposed at Long Pond, Vernon (Reference 2.5.1-262) stated that southwest of the lake on the upthrown block, the Inglis member of the Moodys Branch Formation crops out on hills as high as 10.6 m (35 ft.) amsl. Both members of the Moodys Branch Formation extend northwest along this fault on the upthrown block. Approximately 2.4 km (1.5 mi.) east of Long Pond, on the downthrown block of the fault, the Williston member was encountered in a well at approximately sea level, suggesting a minimum displacement of 15.2 m (50 ft.).

Homosassa Springs Dome. The Homosassa Springs Dome is located approximately 25 km (15.5 mi.) south of the LNP site. This small-diameter dome is located in western Citrus County, where it exposes the Inglis member of the Moodys Branch Formation and is encircled by a band of the Williston member of the Moodys Branch Formation. The dome is approximately 4.8 by 8 km (3 by 5 mi.) and adjoins a small basin in which the Ocala Limestone is exposed.

West Levy Dome. The West Levy Dome is located approximately 45 km (28 mi.) northwest of the LNP site. This small dome is composed of an outcrop of the Inglis member and is surrounded by a broad outcrop of the thin Williston member of the Moodys Branch Formation. The broadness of the Williston outcrop implies very gentle dips westward. Vernon (Reference 2.5.1-262) concluded that the dome is of little significance.

Unnamed Faults. Unnamed postulated faults (features a and b) are located approximately 4 km (2.5 mi.) and 7 km (4.3 mi.) southwest and northeast of the LNP site, respectively. These postulated faults were not specifically described or discussed by Vernon. (Reference 2.5.1-261) The only information regarding the sense of displacement is that shown on the Vernon geologic map of the Citrus and Levy counties area. (Reference 2.5.1-262)

Subsequent mapping and geologic investigations have provided no evidence to support the existence of the faults proposed by Vernon (Reference 2.5.1-262) (References 2.5.1-331, 2.5.1-332, 2.5.1-333, 2.5.1-334, and 2.5.1-335). The following statement is from Thomas M. Scott, Ph.D., P.G., Assistant State Geologist from the Florida Geological Survey (personal communication, e-mail dated August 31, 2007).

> Vernon (Reference 2.5.1-262) postulated several faults in the Citrus-Levy County area of west-central Florida. One of these, the Inverness Fault, is of interest due its proximity to the proposed nuclear power plant site in Levy County. Vernon mapped the area using the limited number of wells that were available and the few accessible outcrops. The limestone surface is karstified and very irregular, making it extremely difficult to map in detail. I know of no investigations subsequent to Vernon that provided support for the existence of the faults. My own investigations of faults (proposed by Vernon) exposed in quarries in Citrus County did not validate the proposed faults. Instead, I found karst-related features that included slickensides and tilted bedding.

Scott (Reference 2.5.1-235) also stated that many of the postulated faults in the state have been identified as offsets in the top of the Ocala Limestone, a karstified, unconformable surface that may have 50 m (164 ft.) or more of relief. Based on this lithology, Scott surmised that it is very difficult to identify faulting in the extremely heterogeneous Neogene sediments, especially with incomplete cores, rock cuttings, and surface outcrops.

The orientations of the postulated faults mapped by Vernon (Reference 2.5.1-262) parallel regional joint and fracture trends (see discussion in FSAR

Subsection 2.5.3). The postulated faults identified by Vernon (Reference 2.5.1-262) in the site vicinity, however, are not apparent in regional scale Landsat imagery or in more detailed aerial photograph mosaics (1949 black and white, 1:20,000 scale) that cover the site vicinity and area, respectively (see discussion in FSAR Subsection 2.5.3.2.1.1).

The postulated faults identified by Vernon (Reference 2.5.1-262) likely do not exist, and there is no evidence to suggest that they have been active in the Quaternary. Therefore, none of these postulated faults are considered to be capable tectonic sources, as defined in Regulatory Guide 1.208, Appendix C (see discussion in FSAR Subsection 2.5.3.6).

2.5.1.2.5 Geology of the Site Location

This subsection presents a more detailed discussion of the geologic conditions at the LNP site based upon field reconnaissance and subsurface exploration.

2.5.1.2.5.1 Site Location Geomorphology

The LNP site is situated in an area of pine plantation and cypress domes with wetlands (Figure 2.5.1-247). The original drainage and topography of the site have been modified by logging and silviculture activities over approximately the past 30 – 40 years. Aside from the logging operations and hunting trails, the site is undeveloped.

The general morphology of the LNP site location (1-km [0.6-mi.] radius) is illustrated by the detailed geomorphologic map derived from light detection and ranging (LIDAR) data as shown on Figure 2.5.1-248. The ground surface, which represents a broad, relatively flat marine terrace mantled by thin terrace cover sediments, increases from elevations of approximately 12 m (40 ft.) NAVD88 west of LNP 1 and LNP 2 to 14.6 m (48 ft.) NAVD88 at the eastern margin of the site location. The elevation of the ground surface at LNP 1 and LNP 2 is approximately 12.8± 1 m (42 ± 2 ft.) NAVD88.

The surface morphology is characterized by dolines (shallow depressions above sinks or paleosinks) varying in size from relatively small, (less than 50 m [164 ft.] in diameter) well-defined circular depressions to large (600 m [2000 ft.] wide) irregular, broad, shallow depressions that are more widespread in the western half of the site location. Many of the circular depressions, which are generally less than 1 to 2 m (2 to 6 ft.) deep, are coincident with cypress domes that are visible in both 1949 black and white (pre-extensive logging) and 2007 aerial photographs. The rectilinear pattern and linear margins of higher areas observed in the topography are consistent with regional joint trends. (See lineament analysis described in FSAR Subsection 2.5.3.2.1.1.)

The morphology is very similar to that of the present coastline in the northern part of Citrus County, which consists of rock-cored marsh islands, broad embayments, and joint-controlled tidal creeks that locally connect a series of circular sinkholes (Figure 2.5.1-236). This supports the conclusions of previous researchers that the site area is underlain by older, karstified marine terrace surfaces mantled by a thin veneer of Quaternary sediment (i.e., References 2.5.1-308, 2.5.1-213, 2.5.1-201, and 2.5.1-235). The rectilinear pattern is more apparent in the terrain above an elevation of about 12.8 m (42 ft.) NAVD88. One possible explanation of the variable surface morphology is that the terrain to the west of the LNP units, which may be underlain by a younger (lower elevation) marine terrace, may have experienced a different erosional history. Alternatively, the variation in geomorphic expression across the site location may reflect shallower groundwater and a generally eastward increase in the thickness of Quaternary cover sand that was deposited against the Brooksville Ridge to the east of the site. The lack of well data for the undeveloped areas beyond the LNP site precludes identification of paleoshorelines and detailed mapping of the thickness of Quaternary cover that could be used to differentiate between these two hypotheses.

2.5.1.2.5.2 Site Location Stratigraphy

Stratigraphy of the LNP site location (1 km [0.5 mi.] radius) is known to a depth below ground surface of approximately -1387 m (-4551 ft.) from SWFWMD records of a deep oil and gas exploration well drilled as part of oil and gas exploration by Humble Oil in 1949. The well, known as Robinson No. 1, is located approximately 500 m (1640 ft.) north of the LNP north reactor (LNP 2) site (Figure 2.5.1-247). Site stratigraphy to a depth of 150 m (500 ft.) is known from geotechnical borings that were drilled as part of the COLA study. Details of the geotechnical boring program are provided in FSAR Subsection 2.5.4.

Well logs from Robinson No. 1 well indicate that Paleozoic age basement rock below the carbonate sequence at the LNP site is at a depth of approximately -1315 m (-4317 ft.), and that the total thickness of the Floridan aquifer carbonates is at least 610 m (2000 ft.) thick. The general stratigraphic sequence encountered in the Robinson No. 1 well (Table 2.5.1-203) consists of 9 m (30 ft.) of Quaternary sediments over approximately 870 m (2860 ft.) of interbedded dolostone and limestone, overlying 475 m (1560 ft.) of chalky limestone, shale and sand, which in turn lies above Paleozoic rock which has been intruded by a Mesozoic dike. The Paleozoic rock (quartzitic sandstone) was encountered at -1315 m (-4317 ft.) depth, and extended to the total drilled depth of -1387 m (-4551 ft.). The depths below ground surface from the Robinson Well No.1 log have been corrected in the FSAR for elevation of the drilling floor indicated on the driller’s log, and so do not exactly match the well depths reported in Vernon (1951), which were published uncorrected.

The generalized hydrostratigraphic column of Floridan aquifer system carbonate depositional sequence in west central Florida is illustrated in Figure 2.5.1-249. As shown in the figure, the Upper Floridan aquifer in the LNP site vicinity typically contains fresh potable water, and is separated physically and hydraulically from the underlying Lower Floridan aquifer by sequences of lower permeability evaporite rock units known as the Middle Confining Unit (MCU), which act as an aquitard. (Reference 2.5.1-323)

The results of the geotechnical boring program at the LNP site showed that the first carbonate rock units encountered below the surficial aquifer deposits (Unit S1) are limestone deposits (calcareous silts of Units S2 and S3; also referred to as undifferentiated Tertiary deposits) that are interpreted to be part of the middle Eocene age Avon Park Formation. Results of the subsurface investigation also indicate that the Miocene age Hawthorn Group is not present, and the Suwannee and Ocala Formations are also absent (Reference 2.5.1-324). This represents a geologic unconformity whereby all Miocene, Oligocene, and Late-Eocene age geologic deposits were eroded from the LNP site location. The contact between the S1 and S2 units is a time-stratigraphic unconformity (an erosional surface hereafter referred to as the "Quaternary/Tertiary (Q/T) unconformity") that represents a gap in the depositional record of approximately 45 million years between the Quaternary surficial deposits and the underlying Avon Park Formation in this location (Figure 2.5.1-214).

Regional hydrostratigraphic studies show that the Avon Park Formation is underlain by the Oldsmar and Cedar Keys formations of Early Eocene and late Paleocene age, respectively. The Oldsmar and Cedar Keys formations are each expected to be at least 152 m (500 ft.) thick in the vicinity of the LNP site. The Avon Park Formation comprises the upper unit and the Oldsmar Formation comprises the lower unit of the Floridan aquifer system in this area; the Cedar Keys Formation acts as the lower confining unit for the Floridan aquifer system. To the maximum investigated depth of 152 m (500 ft.), neither the MCU nor the Lower Floridan aquifer units were encountered.

2.5.1.2.5.2.1 Site Stratigraphic Unit Descriptions

A generalized diagram showing the stratigraphic units encountered at the LNP site is shown on Figure 2.5.1-250. The generalized units shown correlate to layers used in the site response analysis, which were defined based on variation in shear-wave velocity, reviews of the lithology from the boring logs, and reviews of additional downhole geophysical measurements performed by Technos, Inc. (Reference 2.5.1-324) Correlation of the site response layers to stratigraphic layers defined primarily on engineering properties from the LNP 1 and LNP 2 boring program results (FSAR Subsection 2.5.4.2) are provided in the legend for Figure 2.5.1-250.

Surficial geologic deposits at the site consist of undifferentiated Quaternary age fluvial and terrace sediments, primarily silty fine quartz sands. The sands overlie the Avon Park Formation, a shallow marine carbonate rock unit of middle Eocene age.

The Quaternary deposits (designated unit S1) encountered in the LNP site borings generally consist of gray silty quartz sands. The subrounded to rounded sand grains and sorting indicate that the sands likely were deposited in a nearshore beach or dune environment, possibly during the transgression and regression of the high sea level stand that formed the underlying marine terrace platform, which is interpreted to be middle to early Pleistocene in age (>340,000 years ago as discussed in FSAR Subsection 2.5.1.2.1.2). There may be a component of younger eolian quartz sand deposited during subsequent sea level fluctuations and locally derived fluvial deposits. In some boreholes, thicker sections of the S1 deposits consist of gray quartz sand intermixed or interbedded with medium brown sand and grayish black clay and sandy clay layers. These deposits are interpreted to represent infills of sand and marsh deposits into paleosinks. Some of the infill material in the deeper paleosinks may be Tertiary as well as Quaternary in age.

The Quaternary sediments (unit S1) at the LNP site are differentiated from the top of the underlying calcareous silts (dolosilts, designated unit S2) at the top of the Avon Park Formation by their observed siliciclastic (quartz sand or other silicates) lithology, their typically gray to brown color, an absence of fossils, their subrounded to rounded grain shapes, and their lack of reaction to hydrochloric acid (HCL). The thickness of these Quaternary sediments varies across the LNP site from less than 3 m (10 ft.) to approximately 30 m (100 ft.), with a thickness of approximately 2 m (6 ft.) under the nuclear island. At a few boring locations, the thickness of the Quaternary sediments was higher, and the maximum thickness on site was measured at 73.5 m (241 ft.) in one boring completed as part of the pre-COLA siting investigations (Borehole NB 5), located beyond the perimeter of the LNP 2 site (Figure 2.5.1-251 and Figure 2.5.1-252). Based upon available boring data at the LNP 2 site, this local thickening of Quaternary sediments may represent infilling of localized paleokarst features or paleochannels.

The Avon Park Formation is of middle Eocene age, and is characterized as cream to brown or tan, poorly indurated to well-indurated, variably fossiliferous limestone, interbedded in places with tan to brown, very poorly to well-indurated, fossiliferous, vuggy dolostones. Carbonized plant remains are common in the rock sequence in the form of thin, poorly indurated laminae and cyclic interbeds. (Reference 2.5.1-204)

The Avon Park Formation is a carbonate mud-dominated peritidal depositional sequence, pervasively dolomitized in places and not dolomitized in others, that contains some intergranular and interbedded evaporites in its lower part. Fossils are mostly benthic forms showing limited faunal diversity. Seagrass beds are well preserved as lignite lenses at certain horizons. The lower portion of the Avon Park Formation consists of lower permeability evaporite deposits, which act as an aquitard separating the Upper Floridan aquifer within the Avon Park Formation from the Lower Floridan aquifer within the Oldsmar Limestone. (Reference 2.5.1-326)

Borings at the LNP site characterized the upper 150+ m (500 ft.) of the Avon Park Formation as consisting primarily of dolomitized limestone and dolostone. Limestone was encountered but was not widespread over the site. Much of the carbonate in the Avon Park Formation in west-central Florida was deposited as limestone (Reference 2.5.1-231). Dolomitization of the limestone occurred in the Oligocene (Reference 2.5.1-231).

At the LNP site, the Avon Park Formation occurs as a fossiliferous soft limestone near the top of the sequence, with evidence of increasing dolomitization and recrystallization with depth, especially in a more dense rock zone at depths around 40 – 60 m (140 – 190 ft.) The evidence of marine fossils in site rock cores is primarily from casts and molds, which enhance rock porosity; whereas most original calcium carbonate fossil material has been dissolved away by groundwater dissolution. Lignite laminae and interbeds are common near the top of the sequence, and again at depths of approximately 120 m (400 ft.), where two lignite beds, each less than 0.3 m (1 ft.) thick, were observed site-wide. In many borings, the rock sequence becomes softer and less well-indurated limestone with poor core recovery at depths below approximately 61 m (200 ft.)

2.5.1.2.5.3 Site Location Karst Development

To evaluate the LNP site for karst potential, a variety of data sources were employed, including historical and recent aerial photos, published lineament analyses, topographic maps derived from LIDAR data, boring logs, core photos, surface geophysical testing, downhole geophysical logging, and downhole seismic testing.

The low-relief, relatively flat surface topography at the LNP site and surrounding area is characterized by circular to irregularly shaped shallow depressions of varying size. The irregular-shaped depressions are typical of karstic depressions observed elsewhere in the study region that are interpreted to result from coalescing smaller and shallow depressions (Reference 2.5.1-235). Rectilinear margins that define the edges of some of the topographic lows, orientations of the major axis of the depressions and associated wetlands, and alignments of many of the deeper circular features suggest that the location of the features is influenced by joint systems in the underlying rock (see FSAR Subsection 2.5.3.2.1.3).

Observations from the CR3 excavation indicated that solutioning occurred along fractures (joints), and in particular at bedding plane-fracture intersections, forming a network of essentially vertical solution channels that have been secondarily infilled with very fine quartz sands, organic silts and clays, and shells. The irregular surface of the top of the Avon Park Formation at the CR3 plant, which exhibits relief of approximately 10 m (30 ft.), was attributed to formation of karrenfield topography during a period of surface exposure, represented by the Jackson-Claiborne Unconformity, which marks the boundary between the Avon Park Formation and overlying Ocala Limestone. Thus, it is likely that some of the karst development observed in the top of the Avon Park Formation occurred during the late Eocene (approximately 40 million years ago).

The stratigraphy encountered at the CR3 site, which consists of Ocala Limestone over Avon Park Formation, is different from the LNP site where the Ocala Limestone is not present. The Avon Park Formation limestones typically exhibit higher degrees of dolomitization than the Upper Eocene Ocala Limestone. This is significant because the more dolomitized Avon Park Formation limestones have a higher percentage of recrystallized magnesium carbonate, and would therefore typically be less susceptible to the types of karst activity known to occur within the pure calcium carbonate limestone zones present at the top of the Ocala Formation.

Subsurface data from the LNP site investigation indicate that there is variability in the elevation of the Quaternary/Tertiary unconformity and the contact between the S3 and AV1 units at depth. This topography likely reflects a variety of processes, including: 1) weathering and dissolution related to heterogeneities within the underlying carbonate rocks that are due to variable degrees of dolomitization or initial depositional properties; 2) planation and erosion related to Neogene or Quaternary marine transgressions/regressions; and 3) location and degree of development of the paleo-epikarstic surface that likely formed in the upper strata of the Avon Park Formation over a period of as much as several million years.

The LNP site stratigraphy and surface morphology are consistent with expected characteristics of a developed paleokarst landscape mantled by several meters of sand (i.e., a mantled epikarst subsurface) (Figure 2.5.1-244). There are no recognized sinkholes in the State of Florida sinkhole database within 2 km (1.28 mi.) of the LNP site (Figure 2.5.1-244), and no sinkholes at the land surface were observed during site investigations and reconnaissance within the LNP site. Site borings revealed very few voids in the upper 150+ m (500 ft.).

Although subsurface data from exploration boreholes at the LNP site indicate that there is variability in the elevation of the Q/T unconformity at the LNP site, the Q/T unconformity is generally at an elevation of 10.7 ± 0.6 m (35 ± 2 ft.) NAVD88 under the nuclear islands. It is assumed that this represents the general elevation of the marine planation surface, which is estimated to be older than MIS 9 (340 ka) and most likely middle to early Pleistocene, or possibly late Pliocene in age (FSAR Subsection 2.5.1.2.1.2). The nature, frequency, thickness, and lateral extent of subsurface karst features identified in borings under the safety-related structures are described in FSAR Subsection 2.5.4.1.2.1. These features generally vary in lateral extent from a few centimeters to approximately 1.5 m (5 ft.) when associated with vertical fracturing, and from a few centimeters to approximately 3.0 m (10 ft.) when associated with horizontal bedding planes.

2.5.1.2.6 Site Area Geologic Hazard Evaluation

Evaluation of geologic hazards at the LNP site was based on the compilation and review of published maps and reports, reconnaissance investigations in the site area, discussions with FGS and SWFWMD personnel and karst experts, and results of the site characterization program.

- The LNP site is located in an area of infrequent and low seismicity. Earthquake activity with resulting ground motion effects are considered in the seismic design ground motions for the site (see FSAR Subsection 2.5.2). There are no capable tectonic sources in the site area; thus, there is negligible potential for surface tectonic deformation at the site (see FSAR Subsection 2.5.3).

- No natural processes that might cause uplift are active at the site.

- Unrelieved residual stresses are judged to not be a hazard to the site.

- Ground failure and differential settlement due to liquefaction are judged to not be a hazard to the site (see FSAR Subsection 2.5.4.8.5).

The only geologic hazard identified in the LNP site area is potential sulface deformation related to carbonate dissolution and subsidence related to the occurrence of karst. A discussion of the potential for surface deformation related to karst subsidence or collapse is provided in FSAR Subsection 2.5.3.8. The characterization of deeper subsurface karst features for foundation design is summarized in the following subsection and is discussed in more detail in FSAR Subsection 2.5.4.

2.5.1.2.7 Site Engineering Geology Evaluation

The engineering significance of geologic and geotechnical characteristics of features and materials, including foundation materials, are addressed in the following subsections.

2.5.1.2.7.1 Engineering Behavior of Soil and Rock

Engineering soil properties, including index properties, static and dynamic strength, and compressibility, are discussed in FSAR Subsection 2.5.4.

2.5.1.2.7.2 Zones of Alteration, Weathering, and Structural Weakness

The bedrock, which underlies the undifferentiated Quaternary sediments, is the middle Eocene-aged Avon Park Formation (FSAR Subsection 2.5.4.1). The upper portion of this formation, which consists of calcareous silts (units S2 and S3, also referred to as undifferentiated Tertiary sediment) appears to have been altered by weathering and greater degrees of dissolutioning (FSAR Subsection 2.5.1.2). No zones of structural weaknesses, such as extensive fracture zones or faults, have been identified at the LNP site. Postulated faults in the site area and vicinity have been suggested by others in literature, but more recent studies do not provide evidence of the postulated faults (FSAR Subsection 2.5.3.1). Also, regional fracture zones that are mapped in the site region do not cut across the site (FSAR Subsection 2.5.3.2.1).

Smaller subsets of these regional fractures observed in bedrock outcrops in the site area are consistent with these regional trends (FSAR Subsection 2.5.4.1.2.1.1). Bedrock outcropping was not observed in the site location, but televiewer records provide some information on fractures observed in boreholes at the site (FSAR Subsection 2.5.4.4.2.2). Additionally, as with nearly all rock formations, fractures, joints, and bedding planes exist in the Avon Park Formation. These discontinuities (vertical fractures, joints, and bedding planes) are key elements in the localization and development of karst.

2.5.1.2.7.3 Karst Features

The term “karst feature” generally includes surface sinkholes, mantled epikarst subsurface, buried ancient sinkholes (paleosinks), voids, in filled cavities, deep soil infill, and buried raveled zones. Karst features encountered within the LNP site location are expected to be associated with vertical fractures and bedding planes, and vary in lateral extent from a few centimeters to approximately 1.5 m (5 ft.), as discussed FSAR Subsection 2.5.4.1.2.1.3. Karst related features that exist in the subsurface beneath the LNP foundation will be addressed through appropriate design considerations in the LNP foundation concept as discussed in FSAR Subsection 2.5.4.

2.5.1.2.7.4 Deformational Zones

With the exception of possible paleosink features, no deformation zones have been encountered in the site characterization explorations for LNP 1 or LNP 2. Excavation mapping will be undertaken during construction to further evaluate the possibility of deformational zones (FSAR Subsection 2.5.3.8.1).

LNP COL 2.5-1

Table 2.5.1-201
Local Charleston-Area Tectonic Features

Name of Feature	Evidence
Adams Run fault	Subsurface stratigraphy
Ashley River fault	Microseismicity
Appalachian detachment (decollement)	Gravity data Magnetic data Seismic reflection & refraction data
Blake Spur fracture zone	Oceanic transform postulated to extend westward to Charleston area
Bowman seismic zone	Microseismicity
Charleston fault	Subsurface stratigraphy
Cooke fault	Seismic reflection data
Drayton fault	Seismic reflection data
East Coast fault system/ Zone of river anomalies (ZRA)	**Geomorphology Seismic reflection data Microseismicity**
Gants fault	Seismic reflection data
Garner-Edisto fault	Subsurface stratigraphy
Helena Banks fault zone	Seismic reflection data
Middleton Place-Summerville seismic zone	Microseismicity
Sawmill Branch Fault	**Microseismicity**
Summerville fault	**Microseismicity**
Woodstock fault	Geomorphology Microseismicity

Note:

Those tectonic features identified following publication of the EPRI teams' reports (post-1986) are highlighted in **boldface** type.

Source: Reference 2.5.1-270, Table 1

Table 2.5.1-202

LNP COL 2.5-1 **Comparison of Post-EPRI Magnitude Estimates for the 1886 Charleston Earthquake**

Magnitude Estimation Method	Reported Magnitude Estimate	Assigned Weights	Mean Magnitude (M)
Worldwide survey of passive-margin, extended-crust earthquakes	**M** 7.56 ± 0.35[(a)]	—	7.56
Geotechnical assessment of 1886 liquefaction data	**M** 7 – 7.5	—	7.25
Isoseismal area regression, accounting for eastern North America anelastic attenuation	**M** 7.3 ± 0.26	—	7.3
Consideration of available magnitude estimates	**M** 7.1	0.2	7.3
	M 7.3	0.6	
	M 7.5	0.2	
Consideration of available magnitude estimates	**M** 6.8	0.20	7.2
	M 7.1	0.20	
	M 7.3	0.45	
	M 7.5	0.15	
Isoseismal area regression, including empirical site corrections	M_I 6.4 – 7.2[(b)]	—	6.9[(c)]

Notes:

a) Estimate from Johnston et al. (Reference 2.5.1-244, Chapter 3).
b) Ninety-five percent confidence interval estimate; M_I (intensity magnitude) is considered equivalent to **M** (Reference 2.5.1-281).
c) Bakun and Hopper's *preferred* estimate (Reference 2.5.1-281).

Source: Reference 2.5.1-270

LNP COL 2.5-1

Table 2.5.1-203
Stratigraphy of LNP Site
(Based on Well Logs from SWFWMD for Humble Oil Robinson No. 1 Boring, W-2012, and Vernon, 1951)

Thickness		Elevation (msl)[(a)]		Depth (below drilling floor)		Geologic Age	Geologic Units
Meters	Feet	Meters	Feet	Meters	Feet		
9	30	+14 to +8.5	+46 to +28	0 to 9	0 to 30[(b)]	Quaternary	Undifferentiated
354	1160	+8.5 to –345	+28 to –1132	9 to 363	30 to 1190[(b)]	Eocene	Avon Park Formation (includes Lake City)[(c)]
152	500	–363 to –516	–1192 to –1692	381 to 533	1250 to 1750[(d)]	Eocene	Oldsmar
155	510	–345 to –501	–1132 to –1642	363 to 518	1190 to 1700[(b)]		
165	540	–516 to –680	–1692 to –2232	533 to 698	1750 to 2290[(d)]	Paleocene	Cedar Keys
204	670	–500 to –705	–1642 to –2312	518 to 722	1700 to 2370[(b)]		
613	2010	–680 to 1293	–2232 to –4242	698 to 1311	2290 to 4300[(d)]	Cretaceous	Lawson/Taylor/Austin/Atkinson
		–705+	–2312+	722+	2370+[(b)]		
9	31	–1293 to–1303	–4242 to –4273	1311 to 1320	4300 to 4331[(d)]	Cretaceous	Sandstone
14	46	–1303 to –1315	–4273 to –4317	1320 to 1334	4331 to 4377[(d)]	Triassic?	Diabase
71	232	–1317 to –1387	–4319 to –4551 (TD)	1334 to 1405	4377 to 4609 (TD)[(d)]	Ordovician	Quartzite

Notes:

a) Elevation based on subtracting drilling floor elevation (+58 ft. msl), as stated on driller's log from reported depth.
b) Depth according to Chen (Reference 2.5.1-327).
c) Vernon (Reference 2.5.1-262) reports the top of Avon Park Formation at –67 ft. msl (with ground elevation reported at +46 ft. msl). (See Table 11, pp. 153–155 of Vernon.)
d) Depth according to Vernon (Reference 2.5.1-262), measured from drilling floor elevation (+58 ft. msl).

msl = mean sea level
TD = total depth

LNP COL 2.5-1

Table 2.5.1-204
Description of Deep Wells

W number[a]	P number[b]	Company	Well Name	Date Drilled	Geologist	Elevation of Well (ft. msl)	Elevation of Well (m msl)	Depth of Well (ft.)	Depth of Well (m)	Basement Elevation (ft. msl)	Type of Basement Bedrock	Available Data: Geology Log	Available Data: Drillers Log	Available Data: Downhole Data
W-1007	P13	Sun Oil Co.	No. 1 J.T. Goethe	June 8, 1946	Lansdell	33.9	10.33	3997	1218.29	Basement El. -3926 (Coleman, 1979)	Quartzite	Yes (Applin)	Yes	Resistivity
W-1537	P66	Coastal Petroleum	Ragland No. 1	October 18, 1947	Kellough and Huston	NA	NA	5850	1783.08	Basement El. -5796 (Coleman, 1979)	Quartzite	Yes (Applin)	Yes	Resistivity
W-2012	P105	Humble Oil	Robinson No. 1	August 20, 1949	Huston	46 (GL)	14.02	4609	1404.82	Basement El. -4273 (Vernon, 1951)	Diabase (-4773 ft.) Quartzite (-4319 ft.)	No	Yes	Resistivity
W-7538	P353	Mobil Oil. Company.	Harbond No. 1 (Well No. 9 of Barnett [1975])	October 15, 1965	NA	13.45	4.10	4794	461.21	Basement El. -4714 (Barnett, 1975)	Quartzitic Sandstone	No	No	Velocity, Gamma, Induction
W-7543	P350	Mobil Oil. Company	Garby Well No. 1 (Well No. 8 of Barnett [1975])	October 23, 1965	NA	5 (GL)	1.52	5556	1693.47	Basement El. -5505 (Barnett, 1975)	Quartzitic Sandstone	Yes (Chen)	Yes	Velocity
W-7534	P358	Mobil Oil. Company	Camp Phosphate Company (Well No. 7 of Barnett [1975])	March, 1966	NA	106	32.31	4490	1368.55	Basement El. -4314 (Barnett, 1975)	Quartzitic Sandstone	No	No	Velocity
W-8035	P383	Mobil Oil. Company	Florida State Lease 224-A Well Number 1B (Well No. 35 of Barnett [1975])	December 1, 1967	G.N. Hoff	-1195	-364.24	4375	1333.5	Basement El. -4570 (Barnett, 1975)	Quartzitic Sandstone	No	Yes	None
W-8304	P382	Mobil Oil. Company	Florida State Lease 224-A Well Number A1 (Well No. 10 of Barnett [1975])	December 7, 1970	C.S. Chen	NA	NA	6035	1839.47	Basement El. -5864 (Barnett, 1975)	Quartzitic Sandstone	Yes (Chen)	Yes	None

Notes:

a) W number indicates Florida geological well number.
b) P number indicates the well permit number.

References 2.5.1-262, 2.5.1-239, and 2.5.1-336.

ft. = feet
GL = ground level
m = meters
msl = mean sea level

2.5.2 VIBRATORY GROUND MOTION

LNP COL 2.5-2

This subsection provides a detailed description of vibratory ground motion assessments that were carried out for LNP 1 and LNP 2. The subsection begins with a review of the approaches outlined in NRC Regulatory Guide 1.208 for conducting the vibratory ground motion studies. Following this review of the regulatory framework used for the project, results of the seismic hazard evaluation are documented and the site-specific GMRS for horizontal and vertical motions are developed.

The NRC Regulatory Guide 1.208 provides guidance on methods acceptable to the NRC to satisfy the requirements of the seismic and geologic regulation, 10 Code of Federal Regulations (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 the PSHA conducted by the EPRI-SOG in the 1980s (References 2.5.2-201 and 2.5.2-202) has been used for studies in the past. The EPRI-SOG study 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 multidisciplinary teams of experts in geology, seismology, and geophysics; and separately, development of state-of-knowledge earthquake ground motion modeling, including epistemic and aleatory uncertainties.[c] The uncertainty in characterizing the frequency and maximum magnitude of potential future earthquakes associated with these sources and the ground motion that may be produced 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 new data and interpretations and evolving knowledge pertaining to seismic hazard evaluation in the CEUS. The following steps describe a procedure acceptable to the NRC staff for performing a PSHA.

c. Epistemic uncertainty is uncertainty attributable to incomplete knowledge about a phenomenon that affects the ability to model it. Epistemic uncertainty is reflected in a range of viable models, model parameters, multiple expert interpretations, and statistical confidence. In principle, epistemic uncertainty can be reduced by the accumulation of additional information. Aleatory uncertainty (often called aleatory variability or randomness) is uncertainty inherent in a nondeterministic (stochastic, random) phenomenon. Aleatory uncertainty is accounted for by modeling the phenomenon in terms of a probability model. In principle, aleatory uncertainty cannot be reduced by the accumulation of more data or additional information.

1. Perform regional and site geological, seismological, and geophysical investigation in accordance with Regulatory Position 1 and Appendix C to RG 1.208.

2. Perform an evaluation of seismic sources, in accordance with Appendix C to RG 1.208, to determine whether they are consistent with the site-specific data gathered in Regulatory Positon 3.1 or if they require updating. If potentially significant differences are identified, perform sensitivity analyses to assess whether those differences have a significant effect on site hazard.

3. If Step 2 indicates that there are significant differences in site hazard, then the PSHA for the site is revised by either updating the previous calculations or, if necessary, performing a new PSHA. If not, the previous EPRI-SOG results may be used to assess the appropriate SSE ground motions.

Regulatory Guide 1.208 provides guidance on performance goal-based methods acceptable to the NRC to satisfy the requirements of the seismic and geologic regulation, 10 CFR 100.23, for assessing the appropriate site-specific performance goal-based ground motions for new nuclear power plants. Specifically, the performance-based approach described in American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities” may be used to define site-specific performance goal-based GMRS at the ground surface based on mean hazard results (Reference 2.5.2-203). The development of mean seismic hazard results is to be based on a site-specific PSHA combined with site-specific site amplification analyses. The procedures to be used to perform the PSHA and site amplification studies are in Regulatory Guide 1.208. Regulatory Guide 1.208 also provides guidance on an alternative approach for addressing the lower-bound magnitude used in the PSHA based on the likelihood that earthquakes of various sizes can produce potentially damaging ground motions. The ground motion measure used to correlate with the threshold of potential damage is cumulative absolute velocity (CAV). The alternative approach using the CAV filter is used to develop the final GMRS for LNP 1 and 2.

This subsection discusses the following aspects of vibratory ground motion:

- Seismicity (FSAR Subsection 2.5.2.1)
- Geologic and Tectonic Characteristics of the Site and Region (FSAR Subsection 2.5.2.2)
- Correlation of Earthquake Activity with Seismic Sources (FSAR Subsection 2.5.2.3)
- Probabilistic Seismic Hazard Analysis and Controlling Earthquake (FSAR Subsection 2.5.2.4)

- Seismic Wave Transmission Characteristics of the Site (FSAR Subsection 2.5.2.5)

- Ground Motion Response Spectra (FSAR Subsection 2.5.2.6)

2.5.2.1 Seismicity

An important component in developing a seismic hazard model for the LNP site is the seismic history of the region. The selected starting point for developing the site-specific PSHA for the LNP site is the EPRI-SOG (Reference 2.5.2-201) seismic hazard model for the CEUS. The data used to assess earthquake occurrence rates for the seismic sources in the EPRI-SOG model were those in the earthquake catalog.

The first step in the three-step process for evaluating the adequacy of this model for the assessment of seismic hazards at the LNP site involved an assessment of the effect of recent information on the characterization of the seismicity of the southeastern United States. The development of an updated earthquake catalog for the project region is described in FSAR Subsection 2.5.2.1.1. Information on significant earthquakes is provided in FSAR Subsection 2.5.2.1.2. In addition to the discussion of significant earthquakes within the site region, this subsection also discusses recent earthquakes in Gulf of Mexico that postdate the EPRI-SOG catalog. Although these events fall outside the 320-km (200-mi.) radius site region, they occurred within some of the EPRI-SOG background seismic source zones that include the LNP site and thus have implications for assessment of maximum magnitudes in these source zones as discussed in FSAR Subsection 2.5.2.4.1.2. In addition, further assessment of catalog completeness and earthquake recurrence parameters for the offshore region were required as discussed in FSAR Subsections 2.5.2.4.1.3 and 2.5.2.4.1.4.

2.5.2.1.1 Earthquake Catalog

Earthquake occurrence rates for the seismic sources developed in the EPRI-SOG study were based on the EPRI-SOG CEUS earthquake catalog that was developed for the time period of 1627 through February 1985. The EPRI-SOG catalog has gone through two significant revisions. Seeber and Armbruster (Reference 2.5.2-204) conducted a thorough review of the catalog, revising the magnitude estimates and locations of many events, removing some events as non-earthquakes and adding others. The revised earthquake catalog is denoted as the National Center for Earthquake Engineering Research (NCEER)-91 catalog (Reference 2.5.2-205). Subsequently, Mueller et al. reviewed the NCEER-91 catalog along with additional information and developed a catalog of independent[d] earthquakes for use in the U.S. Geological Survey's National Seismic Hazard Mapping Program (Reference 2.5.2-206). The most recent version of this catalog, which is referred to as the USGS 2002 CEUS catalog, is obtainable from the USGS National Seismic Hazard Mapping Project website (Reference 2.5.2-207).

The USGS 2002 CEUS catalog was further updated as part of studies for the Tennessee Valley Authority (TVA) Bellefonte site (Reference 2.5.2-208). The updated catalog incorporated new information on location and magnitude of historical earthquakes and included 174 newly identified historical earthquakes, principally from studies by Metzger, Metzger et al., and Munsey (References 2.5.2-209, 2.5.2-210, and 2.5.2-211). Details of the development of the Bellefonte Geotechnical, Geological, and Seismological (GG&S) earthquake catalog are provided in TVA (Reference 2.5.2-208).

The catalog for the LNP site consists of the Bellefonte GG&S earthquake catalog extended to 23°N and to 107°W, and through December 2006 using the listing of additional earthquakes from the EPRI-SOG catalog and recent earthquakes obtained from the following sources:

- Advanced National Seismic System (ANSS) website (References 2.5.2-208 and 2.5.2-212).
- USGS National Earthquake Information Center website (Reference 2.5.2-213).
- Southeastern U.S. Seismic Network website operated by Virginia Tech Seismological Observatory (Reference 2.5.2-214).
- International Seismological Center Bulletin (Reference 2.5.2-215).

Figure 2.5.2-201 shows the spatial distribution of earthquakes in the project earthquake catalog. Figure 2.5.2-202 shows the locations of earthquakes within 320 km (200 mi.) of the LNP site. Note that only one earthquake in the project catalog has occurred within 80 km (50 mi.) of the LNP site. The earthquakes are color coded on Figures 2.5.2-201 and 2.5.2-202 to indicate those events included in the EPRI-SOG earthquake catalog for the time period of 1758 to 1985, historical events added to the EPRI-SOG catalog, and those events that occurred after the EPRI-SOG catalog (1985 to 2006). The added historical earthquakes and the earthquakes occurring since the EPRI-SOG study have similar spatial distributions as the earthquakes contained in the EPRI-SOG catalog, and no new concentrations of seismicity are apparent in the updated catalog.

d. 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.

Appendix 2AA lists the earthquakes in the updated catalog that have occurred within 320 km (200 mi.) of the LNP site. The list consists of 15 events of $m_b \geq 3$ that occurred between 1826 and January 1, 2007. The size distribution of these earthquakes consists of 13 events of magnitude $3 \leq m_b < 4$; and 2 events of magnitude $4 \leq m_b < 4.5$. Estimates of the Modified Mercalli Intensity (MMI) and strong motion records are not available for these earthquakes.

In addition to these events, earthquakes that occurred in the Gulf of Mexico at greater distance from the LNP site were considered. In all, there are 17 additional earthquakes of $m_b \geq 3$ recorded from 1963 to January 1, 2007; 11 events of magnitude $3 \leq m_b < 4$; 5 events of magnitude $4 \leq m_b < 5$; and 1 event of magnitude $5 \leq m_b \leq 6$, which occurred at nearly 500 km (310 mi.) from the LNP site. Estimates of the MMI and strong motion records are not available for these earthquakes.

Focal depths are either not determined (set equal to 0) or fixed (set to 5, 10, 15, or 33 km) for most of the earthquakes. Only five events have listed depths greater than 10 km. The earthquakes do not show any correlation between depth and magnitude.

The body-wave magnitude scale, m_b, was used as the uniform magnitude scale in the original EPRI-SOG earthquake catalog and is the magnitude scale used in the catalog developed for the LNP study. Estimated seismic moments are provided for the catalog in Appendix 2AA. The values listed were estimated by first estimating moment magnitude using the three relationships described in FSAR Subsection 2.5.2.4, then computing seismic moment from each moment magnitude estimate using the Hanks and Kanamori relationship, and finally, averaging the results (Reference 2.5.2-216).

2.5.2.1.2 Significant Earthquakes

2.5.2.1.2.1 Significant Earthquakes in the Site Region (320 km [200 mi.] Radius)

Seismicity within 320 km (200 mi.) of the LNP site is sparse and minor; earthquake magnitudes do not exceed m_b 4.3. The locations of the earthquakes listed in the catalog developed for the LNP study are shown on Figure 2.5.2-202. The largest earthquake (m_b 4.3) is described as follows:

This earthquake occurred on January 13, 1879, near St. Augustine in the northeast part of Florida. Shaking caused by this event knocked plaster from walls and articles from shelves in St. Augustine and Daytona Beach. The shock was felt throughout northern and central Florida and at Savannah, Georgia (Reference 2.5.2-217).

Although located outside the site region, the 1886 Charleston, South Carolina, earthquake is the largest event known in the southeastern United States. The Charleston earthquake occurred on September 1, 1886 (August 31 local time), about 500 km (about 300 mi.) north of the LNP site. This earthquake was felt throughout northern Florida, particularly on the eastern coast. Several aftershocks were felt in Jacksonville, Florida (Reference 2.5.2-217).

This earthquake was one of the largest historic shocks in eastern North America and the most damaging earthquake to occur in the Southeast United States (Reference 2.5.2-218). The maximum intensity has been estimated at MMI X. The first shock was followed by a strong aftershock 8 minutes later, followed by six additional shocks during the next 24 hours. In Charleston an estimated 60 persons were killed and many more were injured. Damage was extensive; many buildings were totally destroyed and only a few escaped serious damage. Cities within a radius of 160 km also experienced damage, including Columbia, South Carolina, and Augusta and Savannah, Georgia. The total area affected by the earthquake included distant points in the United States such as New York City, Boston, and Milwaukee, plus Havana, Cuba, Bermuda, and Ontario, Canada (Reference 2.5.2-219).

2.5.2.1.2.2 Recent Gulf of Mexico Earthquakes

One earthquake having a body-wave magnitude of approximately 6 (Emb 6.0) and two smaller events occurred in the northern Gulf of Mexico during 2006 (Figure 2.5.2-203). A summary of the reported magnitudes for these and earlier events and distances from the LNP site is provided in Table 2.5.2-201. An unusual m_b 4.2, M_s 5.3 earthquake occurred off the coast of Louisiana, approximately 240 km (384 mi.) south of New Orleans on February 10, 2006 (References 2.5.2-220 and 2.5.2-221). This earthquake was the largest to occur in the Gulf of Mexico since the **M** 5 (Emb 4.9) earthquake of July 24, 1978 (Reference 2.5.2-222), which represents the best-recorded earthquake in the region prior to the February 10, 2006, event (Reference 2.5.2-221). Two previous earthquakes in 1994 (m_b 4.2, according to the National Earthquake Information Center [NEIC]) and 2000 (m_b 4.2, M_S 4.3; according to NEIC) also occurred in the same area (within an error of ~50 km) of the February 10, 2006, event (Figure 2.5.2-203). Following the February 2006 event, another unusual event with source characteristics similar to the February event occurred on April 18, 2006, less than 100 km (30 mi.) offshore of the tip of Louisiana's Birdfoot Delta. This earthquake, which was not detected or located by the USGS (NEIC) using traditional P-wave[e] arrivals, generated surface waves of an amplitude typical for a shallow event of approximately **M** 4.6 (Reference 2.5.2-220). A larger **M** 5.8-5.9, m_b 5.9, earthquake occurred on September 10, 2006, approximately 419 km (260 mi.) west-southwest from Clearwater, Florida, (Reference 2.5.2-223) in an abyssal plain environment. This earthquake, which was felt in parts of Florida, Georgia, Alabama, Kentucky, Louisiana, Mississippi, North Carolina, South Carolina, Tennessee, and Texas, as well as in the Bahamas and the Yucatan Peninsula, Mexico, did not generate a significant tsunami (References 2.5.2-223

e. P-wave —a body wave that can pass through all the layers of the earth, the fastest of all seismic waves; also known as a compressional wave; longitudinal wave; primary wave.

and 2.5.2-224). Felt reports at Crystal River, Florida, were intensity IV (Reference 2.5.2-223).

The earthquake size measure used in the EPRI-SOG seismic source characterization is expected body wave magnitude, Emb. The EPRI-SOG methodology for obtaining Emb was to use the reported m_b if available. If no m_b value was reported, then Emb was assessed using conversions from other size measures. As indicated in Table 2.5.2-201, the estimate of m_b based on conversion from M_s is 5.6. For the LNP site evaluation a conservative value of Emb 4.9 is used, which is obtained by averaging the converted M_s value with the reported m_b of 4.2 for the February 10 earthquake. The September 10, 2006, earthquake had a reported m_b of 5.9 and a reported moment magnitude of **M** 5.8 to 5.9. Conversion of the moment magnitude to body wave magnitude using the relationships presented in FSAR Section 2.5.2.4.2.3 yields an average estimated m_b of 6.1. For the LNP site evaluation a slightly conservative value of Emb 6.0 is used, which is obtained by averaging the reported m_b and the value converted from **M** 5.8-5.9. The STP 3 & 4 FSAR reports an Emb of 6.1 for the September 10 earthquake, which is based solely on conversions from **M** 5.8.

The source characteristics of the largest events recorded in the Gulf of Mexico, the 1978 and recent 2006 events, are quite different suggesting that different types of triggering mechanisms may give rise to earthquakes in this region. In contrast to the unusual February and April 2006 earthquakes, which did not provide good teleseismic waveforms, the faulting geometry and size of both the 1978 and September 2006 earthquakes were well constrained by standard centroid-moment-tensor (CMT) analysis (Reference 2.5.2-221). As described below, the source characteristics of the February and April 2006 events are best explained as being gravity-driven displacements on a shallow, low-angle detachment surface within or at the base of a thick sedimentary wedge; the 1978 and September 2006 earthquakes, which occurred within basement rock at depths of greater than 15 km (9.3 mi.), have source characteristics more typical of tectonic events.

Frohlich (Reference 2.5.2-222) concluded based on the focal depth (15 km [9.3 mi.]) and reverse-faulting focal mechanism that the 1978 earthquake occurred within the basement and that typical of other intraplate events the event probably occurred along relatively inactive structural trends that may represent zones of weakness in the crust. Frohlich postulated that the event may have been related to stresses associated with the downwarping of the lithosphere caused by accumulation of sediments from the Mississippi River (Reference 2.5.2-222). Different focal mechanisms are reported for this event. Frohlich (Reference 2.5.2-222) shows a reverse faulting mechanism on an east-northeast trend, whereas the global CMT catalog solution shows a reverse faulting mechanism on a northwest trend (Reference 2.5.2-225).

The September 10 earthquake, which had a deep hypocenter (22 km [13.6 mi.] per USGS solution and 31.7 km [19.6 mi.] per Harvard solution [Reference 2.5.2-223]), is recognized as a typical tectonic event (Reference 2.5.2-226). The U.S. Geological Survey did not associate this earthquake with a specific causative fault. The September earthquake occurred near the transition between oceanic crust and thin transitional crust as shown by Sawyer et al. (Reference 2.5.2-227) in an area where there are a number of northwest-trending basement faults and structures (Reference 2.5.2-228), as well as an interpreted northwest-trending regional basement structure that is inferred to have been related to rifting and opening of the Gulf of Mexico in the Mesozoic (Reference 2.5.2-229).

In contrast to the 1978 and September 2006 earthquakes, the February 10 earthquake is notable for the unusual characteristics of the teleseismic waveforms it generated (Reference 2.5.2-221). In particular, the teleseismic seismograms are depleted in high-frequency energy, and are not fit well by traditional double-couple source models typical of tectonic faulting mechanisms. A moment-tensor source can be used to model the surface waves generated by the February 10, 2006, earthquake if the earthquake centroid is placed within a few miles of the earth's surface in a medium with a very low shear modulus. The seismograms are fit well by a single-force source (that is, a model of sliding on a shallow, sub-horizontal surface). The depth of the source for the February 10 event was likely less than 6 to 8 km (3.7 to 5 mi.) The best explanation for the mechanism for the February 10, 2006, earthquake and the similar event on April 18, 2006, is that of a gravity-driven displacement occurring on a low-angle detachment surface within the sedimentary wedge. (Reference 2.5.2-221)

Peel (Reference 2.5.2-230) describes the structural context of the February earthquake and reviews possible seismogenic processes that could operate within the region. He refers to the February event, which is located within the Green Canyon Block 344, as the GC344 event. The location of this event is close to a major down-to-the-northwest basement step, corresponding to a downdip change in basement character. Peel (Reference 2.5.2-230) notes that this boundary also corresponds to a change in character of the regional magnetic pattern; and it is probably the boundary between stretched continental crust (updip) and stretched basinal crust, possibly oceanic in character (downdip). The location of the GC344 event also overlies the boundary between autochthonous and allochthonous deep salt, which appears to correspond to the basement boundary. The autochthonous deep salt is overlain in turn by a thick section of Jurassic to Upper Miocene cover sediment that has moved a distance of about 5 to 10 km (3 to 6 mi.) towards the south-southeast, as a result of gravity spreading of the whole margin. Southwards movement and folding of this sediment package occurred during the Paleogene and Miocene, and there appears to have been no further movement since the early Pliocene. Since that time, southwards movement appears to be concentrated at a higher level within the Sigsbee Salt Nappe, a major allochthonous salt canopy spread out over the folded unit. Spreading of this salt unit began during the middle Miocene, reached a peak during the late Miocene and Early Pliocene, and continues to the present day.

As reported by Peel (Reference 2.5.2-230), seismic imaging shows that the Sigsbee Salt Nappe contains large recumbent folds and a major basal shear zone. The salt nappe was dominantly emplaced by large-scale glacier-like flow from the north-northwest, with a minor component of local vertical feeding through diaper throats. On top of the Sigsbee salt is a sediment package (carapace) of Upper Miocene, Pliocene, and Pleistocene sediments that is dominated by salt withdrawal basins and salt walls. Some of the salt withdrawal basins have subsided all the way to the base of the Sigsbee Nappe, forming significant sediment-on-sediment contact areas known as welds. Peel (Reference 2.5.2-230) observes that a likely welded area can be mapped close to GC344, and concludes that the most likely mechanism for the earthquake was movement on the base of the Sigsbee Salt, with faulting occurring where a suprasalt basin is grinding against the base of the salt weld. The probable seismic expression of this mechanism would be low-angle faulting at a depth of about 8 to 10 km (5 to 6 mi.) subsea. The predicted movement is likely to be generally southwards, but a wide range of movement direction (± 90 degrees) is possible due to partitioning of movement of the Sigsbee Nappe.

Angell and Hitchcock (Reference 2.5.2-231) invoke a possible model of fault characteristics that could contribute to seismic rupture of a growth fault in which areas of both stick-slip and creep modes of displacement coexist on a single fault surface. They note that these conditions might occur along a fault plane where salt has been evacuated and the result is a sediment-sediment contact at the base of the growth fault.

Gangopadhyay and Sen (Reference 2.5.2-226) suggest a mechanism for earthquakes in the Gulf of Mexico that involves stress concentration resulting from the contrast in mechanical properties between salt and surrounding sediments driven by tectonic loading. The results of modeling suggest that some locations of relatively high shear stress correlate well with the spatial distribution of seismicity in the northern Gulf of Mexico, thereby suggesting a possible causal association.

2.5.2.2 Geologic and Tectonic Characteristics of the Site and Region

As outlined previously, Regulatory Guide 1.208, specifies that recent information should be reviewed to evaluate if this information indicates significant differences from the previous seismic hazard. FSAR Subsection 2.5.1 presents a summary of available geological, seismological, and geophysical data for the site region (320 km [200 mi.] radius), site vicinity (40 km [25 mi.] radius), and site area (8 km [5 mi.] radius) that provides the basis for evaluating seismic sources that contribute to the seismic hazard to the LNP site. This subsection presents a description of the seismic source characterizations from the EPRI-SOG evaluation (Reference 2.5.2-201) (FSAR Subsection 2.5.2.2.1), followed by a summary of general approaches and interpretations of seismic sources used in more recent seismic hazard studies (FSAR Subsection 2.5.2.2.2). FSAR Subsections 2.5.2.3 and 2.5.2.4 present evaluations of the new information relative to the EPRI-SOG seismic source evaluations (Reference 2.5.2-201).

2.5.2.2.1 EPRI-SOG Source Evaluations

During the 1980s, the Seismic Owners Group (SOG) conducted a comprehensive seismic hazard methodology development program at EPRI. The

SOG program emphasized earth science assessments of alternative explanations of earthquakes in the CEUS, with a particular emphasis on a systematic understanding and expression of uncertainties. Seismic sources and associated interpretations necessary for hazard calculations at any nuclear power plant site in the CEUS were developed. Six earth science teams (EST) provided input interpretations: Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical, and Woodward-Clyde Consultants. Each team produced a report (Volumes 5 through 10 of EPRI-SOG) that provided descriptions of how the seismic sources were identified and defined. (Reference 2.5.2-201)

The seismic source characterizations developed by the EPRI-SOG ESTs were used to conduct PSHAs for nuclear power plant sites in the CEUS that were reported in EPRI (Reference 2.5.2-202). Included in that set of plant sites was the Crystal River Unit 3 (CR3) located within the Crystal River Energy Complex (CREC) located about 15.5 km (9.6 mi.) from the LNP site. The EPRI-SOG PSHA seismic source characterization for the CR3 (Reference 2.5.2-202), thus was judged to be an appropriate initial starting point in the assessment of the seismic hazard for the LNP site.

The calculations performed by EPRI (Reference 2.5.2-202) for each site excluded the seismic sources defined by each EPRI-SOG EST that, in combination, contributed less than one percent to the total hazard computed from all sources defined by that EST. The EPRI selection of the seismic sources that are significant to assessing the seismic hazard at a site was based on calculations made with the ground motion models presented in EPRI-SOG (References 2.5.2-202 and 2.5.2-201). Since that time, there have been advances in the characterization of earthquake ground motions for CEUS earthquakes. These advances are described in FSAR Subsection 2.5.2.4.2. Because the potential contribution of a seismic source to the hazard at a site is dependent in part on the ground motion model used to compute the hazard, the identification of the significant EPRI-SOG seismic sources for the LNP site was assessed using updated ground motion models. Tables 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, and 2.5.2-207 list the seismic sources for each of the six EPRI-SOG teams that were found to contribute in aggregate 99 percent of the hazard at the LNP site. FSAR Subsection 2.5.2.4.3.1 presents the hazard contribution of the individual EPRI-SOG seismic sources. These seismic sources are shown on Figures 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, 2.5.2-208, and 2.5.2-209 and are described in FSAR Subsections 2.5.2.2.1.1, 2.5.2.2.1.2, 2.5.2.2.1.3, 2.5.2.2.1.4, 2.5.2.2.1.5, and 2.5.2.2.1.6. Many of the seismic sources described by the EPRI-SOG teams are so described in FSAR Subsection 2.5.1.1.4.4, including the zones associated with the 1886 Charleston earthquake.

2.5.2.2.1.1 Bechtel Team Seismic Sources

Five seismic sources defined by the Bechtel team (Reference 2.5.2-232) are included in the PSHA calculations for the LNP site (Figure 2.5.2-204). These sources are listed in Table 2.5.2-202 and are described below.

- Charleston Area (Source H) and Charleston Faults (Source N3): These two seismic source zones (Reference 2.5.2-232) are located in the vicinity of Charleston, South Carolina. No specific information on these sources is provided by the Bechtel team other than that they represent possible source locations for the 1886 Charleston earthquake and may contain an active fault (Reference 2.5.2-232).

- Atlantic Coastal Region (Source BZ4): This background source zone is a large area that encompasses the Eastern Mesozoic basins (Source 13), Charleston (Source H), and a source derived from three separate model sources in the Charleston, South Carolina, area (Source N3) (Reference 2.5.2-232).

- Gulf Coast Region (Source BZ1): The LNP site lies within the Gulf Coast region (Source BZ1). This zone is a large background source that extends from the continental shelf off eastern Florida to the western coastal plain of Texas and encompasses the majority of the site region. This background source zone was defined based on geopotential (gravity and magnetic anomaly data) and seismic data. (Reference 2.5.2-232)

- Southern Appalachians Region (Source BZ5): This background source zone encompasses a large area of the southern Appalachians to the north of the site region. The zone includes the Eastern Mesozoic basins (Source 13); Rosman fault (Source 15); Belair fault (Source 16); Stafford fault (Source 17); Giles County feature (Source 19); Lebanon geopolitical trend (Source 23); Bristol block trends (Source 24); a segment of the New York – Alabama lineament (Source 25); central Virginia (Source E); southeast Appalachians (Source F); and northwest South Carolina (Source G). Some of these sources are associated with moderate earthquakes (Reference 2.5.2-232).

2.5.2.2.1.2 Dames & Moore Team Seismic Sources

Six seismic sources defined by the Dames & Moore team (Reference 2.5.2-233) are included in the PSHA calculation for the LNP site (Figure 2.5.2-205). These sources are listed in Table 2.5.2-203 and are described below.

- Southern Cratonic Margin (default) (Source 41): This source zone contains deformed Grenvillian basement overlain by late Precambrian metamorphosed clastic sediment and associated mafic intrusive and extrusive rocks. The Southern Cratonic Margin default zone encompasses a large region of continental margin deformed during Mesozoic and Cenozoic rifting and includes many Triassic basins and border faults. This source is a default zone for the Newark-Gettysburg basin, Ramapo fault zone, and other Mesozoic/Cenozoic basins (Sources 42, 43, 46, respectively, which are considered mutually exclusive with the default zone, Source 41). This source zone contains seismicity in a diffuse pattern throughout the zone. (Reference 2.5.2-233)

- Southern Appalachian Mobile Belt (default) (Source 53): This default source contains crustal rocks that are younger than Grenvillian or Avalonian that have undergone multiple periods of crustal divergence and convergence. It contains several Triassic basins. Much of the seismicity associated with the zone is diffuse throughout the zone. Included within this default zone is the Charleston Mesozoic Rift (Source 52), which may have some association with seismicity in the Charleston, South Carolina, area. (Reference 2.5.2-233)

- Charleston (Source 54): This source is a zone around the Charleston region that contains recurring seismic activity. The zone includes tectonic features described in the literature as possible sources of the 1886 Charleston, South Carolina, earthquake. These features are the Woodstock fault, Ashley River fault, Cooke fault, and the Helena Banks fault. (Reference 2.5.2-233)

- Charleston Mesozoic Rift (Source 52): This source zone is in the northern part of the site region and encompasses a large area around the Charleston, South Carolina, area. This Mesozoic rift source may have some association with seismicity in the area of Charleston (Reference 2.5.2-233).

- Southern Coastal Margin (Source 20): The LNP site lies within the Southern Coastal Margin, which extends from the continental shelf off eastern Florida, along the Texas coastal plain, and into Mexico. This source zone encompasses the majority of the site region. This source zone was defined based on its fairly low, diffuse seismicity. The zone represents the down warping miogeosynclinal wedge of sediment that accumulated within the Gulf Coast Basin since the Cretaceous. (Reference 2.5.2-233)

- Paleozoic (Appalachian) Fold Belt (Source 4): This zone is located to the north of the site region and consists of a major segment of a folded mountain belt, the Appalachians from New York to Alabama. Two configurations of this zone are considered: the fold belt and the fold belt with kinks (Sources 4A, 4B, 4C, and 4D) (Reference 2.5.2-233).

2.5.2.2.1.3 Law Engineering Team

Eight seismic sources defined by the Law Engineering team (Reference 2.5.2-234) are included in the PSHA calculations for the LNP site (Figure 2.5.2-206). These sources are listed in Table 2.5.2-204 and described briefly below.

- Eastern Basement (Source 17): This zone encompasses a large area of buried (sub-decollement) Precambrian-Cambrian normal faults (Reference 2.5.2-234). The zone includes the Giles County, Virginia – East Tennessee Seismic Zone, the Pennsylvania Aulacogen, and the Scranton Gravity High tectonic features, all of which are related to the same deformational phase (Reference 2.5.2-234).

- Eastern Basement Background (Source 217): This background seismic zone is characterized by a negative Bouguer gravity field and a pattern of magnetic anomalies. The zone is inferred to overlie the Precambrian continental margin of North America (Reference 2.5.2-234).

- Reactivated Normal Faults (Source 22): This zone is known as the Wentworth Hypothesis – Reactivated Eastern Seaboard Normal Faults seismic source zone (Reference 2.5.2-234). The zone was defined based on the hypothesis that seismicity throughout the Eastern Seaboard region may be associated with faults reactivated in the current compressive stress regime. Those faults interpreted to have the highest potential for reactivation originally formed as normal faults during the Mesozoic era (Reference 2.5.2-234).

- Charleston (Source 35): This source zone was defined by the Law Engineering team based on the pattern of seismicity and "because the various tectonic features related by hypothesis to the zone cannot explain why Charleston might continue to exhibit seismicity higher than its surrounding area" (Reference 2.5.2-234). The Law Engineering team also defined three seismic source zones based on tectonic features that could allow a large earthquake to recur at Charleston; however, such an event would not be restricted to the Charleston area. Accordingly, the Charleston seismic zone (Source 35) was defined. (Reference 2.5.2-234)

- Mesozoic Basins (Source 8): Buried East Coast Mesozoic basins are recognized as potential seismic sources by the Law Engineering team (Reference 2.5.2-234). The Mesozoic Basins are northeast-trending elongated troughs of late Triassic to early Jurassic age that are bounded on one or both sides by high-angle faults. These faults are favorably oriented to be reactivated, similar to the faults described above in the Reactivated Normal Fault zone (Source 22) (Reference 2.5.2-234).

- Southern Coastal Block (Source 126): The LNP site lies within the Southern Coastal Block (Source 126). This background seismic source zone is assumed to represent an area of similar crustal structure at seismogenic depths. The southern boundary of this zone was defined based on broad wavelength magnetic anomalies that extend from the southeast Texas-Mexico border to the continental shelf offshore Florida; the northern boundary was defined by the Paleozoic edge of the North American craton. (Reference 2.5.2-234)

- Eastern Piedmont (Source 107): This background seismic source is located to the north of the site region. It is characterized by a positive Bouguer gravity field and a pattern of magnetic anomalies. The region is believed to be a crustal block that lies to the east of the relict North American continental margin (Reference 2.5.2-234).

- Brunswick (Source 108): This background seismic source has a basement terrane with a distinctive geophysical pattern that contrasts with the patterns of the Eastern Piedmont province to the northwest and the Southern Coastal Block (Source 126) to the south. The basement rock may represent a zone of Triassic and/or Jurassic crustal extension formed during the early stages of the opening of the Atlantic Ocean (Reference 2.5.2-234).

2.5.2.2.1.4 Rondout Associates Team

Five seismic sources defined by the Rondout Associates team (Reference 2.5.2-235) are included in the PSHA calculation for the LNP site (Figure 2.5.2-207). These sources are listed in Table 2.5.2-205 and described briefly below.

- Charleston (Source 24): The Charleston seismic zone includes the Ashley River fault and Woodstock fault (Reference 2.5.2-235).

- Southern Appalachians (Source 25): This seismic source zone is defined based on deep-seated seismicity in basement rocks below the regional decollement. The seismicity may be associated with faults inferred from the aeromagnetic anomalies associated with the New York – Alabama lineament (Source 13) (Reference 2.5.2-235).

- South Carolina Zone (Source 26): This seismic zone parallels and encompasses northwest, cross-cutting fracture zones mapped on the detailed aeromagnetic map of South Carolina. Seismicity is associated with this zone. (Reference 2.5.2-235)

- Appalachian Crust (Source 49): The LNP site lies within the Appalachian Crust seismic zone. The crust of this background zone was formed after the Precambrian and the basement is a complex accretionary terrane. The zone may not have a uniform seismic potential (Reference 2.5.2-235).

- Gulf Coast to Bahamas Fracture Zone (Source 51): This source zone was defined separately because of differences in the orientation of the stress regime between the Paleozoic crust within the zone and the Appalachian crust of roughly the same age to the east and northeast (Reference 2.5.2-235).

2.5.2.2.1.5 Weston Geophysical Team

Six seismic sources defined by the Weston team (Reference 2.5.2-236) are included in the PSHA for the LNP site (Figure 2.5.2-208). These sources are listed in Table 2.5.2-206 and described briefly below.

- New York – Alabama – Clingman Block (Source 24): This seismic source is a linear block of seismicity within the Southern Appalachian zone (Source 103). Relatively accurate hypocenters for seismicity in the block suggest that these earthquakes originate well below a detachment structure previously assumed to be seismogenic (Reference 2.5.2-236).

- Charleston (Source 25): Several tectonic features are included in this seismic domain, and it is assumed by the Weston team that future seismicity in the east will be localized along one or more of these features identified for the Charleston, South Carolina, region. These tectonic features are: the Woodstock, Ashley River, Helena Banks, and Cooke faults; the northwest extension of an offshore fracture zone; and a zone of decollement (Reference 2.5.2-236). This zone is part of the Southern Coastal Plain background (Source 104).

- South Carolina Zone (Source 26): This zone is also part of the Southern Coastal Plain background (Source 104) (Reference 2.5.2-236).

- Southern Appalachian (Source 103): This background zone is located to the north of the site region and includes the Inner Piedmont, Blue Ridge, and Valley and Ridge physiographic belts of the southern Appalachians. The New York – Alabama – Clingman lineaments block seismic zone (Source 24) is within this background zone (Reference 2.5.2-236).

- Southern Coastal Plain Background (Source 104): This south coastal plain background seismicity zone adjoins the Southern Appalachian background (Source 103). The zone incorporates several additional seismic sources, including the Charleston, South Carolina, seismic zone (Source 25) and the South Carolina seismic zone (Source 26) (Reference 2.5.2-236).

- Gulf Coast Background (Source 107): The majority of the site region is within this background source zone. This source zone was defined as an independent background source that does not contain any other seismic source regions. This zone extends from Texas to Florida (Reference 2.5.2-236).

2.5.2.2.1.6 Woodward-Clyde Consultants Team

Four seismic sources defined by the Woodward-Clyde Consultants team (Reference 2.5.2-237) are included in the PSHA for the LNP site (Figure 2.5.2-209). These sources are listed in Table 2.5.2-207 and described briefly below.

- Greater South Carolina (Sources 29, 29A, and 29B): This source zone pertains to seismicity located in South Carolina, Georgia, and western North Carolina (Reference 2.5.2-237). An isostatic gravity high trends northeast along the Appalachians, but in the area of central South Carolina a saddle or gap is observed in the gravity high. A northwest-trending zone of seismicity extends from the coast into western North Carolina; this zone includes the area of the 1886 Charleston earthquake and many smaller magnitude events. The expression within the isostatic gravity data of the saddle suggests that it is crustal in scale; varying crustal thickness in the area may be a potential stress concentrator (Reference 2.5.2-237). Alternative source zone designations that are mutually exclusive (Sources 29, 29A, and 29B) are used for the hazard calculations.

- Charleston (Source 30): The only tectonic features assessed to have significant seismic potential in the local Charleston area are the Ashley River and Woodstock faults (Reference 2.5.2-237). The existence of these faults is based primarily on seismological evidence, as recent microseismic activity is located along and in large part defines these faults. The correlation of these faults with the 1886 Charleston earthquake is based primarily on isoseismal patterns (Reference 2.5.2-237).

- Blue Ridge Zone and Alternative (Sources 31 and 31A): This source zone extends from the southern to the central Appalachians. Two alternative configurations are defined (Reference 2.5.2-237). The basis for the source zone is an inferred block of distinctive crust associated with an isostatic gravity low. The alternative interpretation of this zone is based on three sub-zones of seismicity (Reference 2.5.2-237).

- Crystal River Background (Source B36): The LNP site lies within a large background zone that encompasses most of the state of Florida. This source is a background zone defined as a rectangular area (2 degrees by 2 degrees) surrounding the CR3 site and is not based on any geological, geophysical, or seismological features. Because the CR3 site is located only 18 km (11 mi.) from the LNP site, the Crystal River Background Source was used for the LNP site without modification of its geometry.

2.5.2.2.2 Post-EPRI Seismic Source Characterizations

Seismic hazard studies conducted in the LNP site region since completion of the 1986 - 1988 EPRI-SOG study are described in the following subsections (Reference 2.5.2-201).

2.5.2.2.2.1 Lawrence Livermore National Laboratory Trial Implementation Program Source Evaluations

A decade after the completion of the EPRI-SOG (Reference 2.5.2-201) evaluation, Lawrence Livermore National Laboratory (LLNL) (Reference 2.5.2-238) conducted a Trial Implementation Program (TIP) of the Senior Seismic Hazard Analysis Committee (SSHAC) guidance for a Level IV analysis (Reference 2.5.2-239). 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 team, a panel of five expert evaluators, and expert proponents and presenters. Preliminary implementations for two sites in the southeastern United States (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 East Tennessee seismic zone [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 LNP site region, in particular the source for repeated large-magnitude, Charleston-type events and source zones for background events.

Seismic source models were developed for each of the five experts. Through discussions at workshops, one-on-one interviews, and white papers, a set of common sources was identified as the basic building block for all the sources and alternative sources. The general boundaries of these common sources are shown on Figure 2.5.2-210. This minimum set of zones was then used to create the composite model of seismic sources that represented the range of feasible sources. 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.

Table 2.5.2-208 provides a description of the minimum set zones. A complete description of the logic tree representation of the experts' interpretations for the Charleston and ETSZ and maximum magnitude distributions for alternative source zones is presented in Savy et al. (Reference 2.5.2-238).

2.5.2.2.2.2 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 United States were produced in 2002 (Reference 2.5.2-240). Input for revising the source characterization used in the 1996 hazard maps was provided by researchers through a series of regional workshops (Reference 2.5.2-241). Key issues that were addressed in the updated source characterization included new information regarding the location, size, and recurrence of repeated 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 seismic source model developed by the USGS are shown on Figure 2.5.2-211 (Reference 2.5.2-206).The general approach used by the USGS for modeling distributed seismicity in the CEUS is based on gridded, spatially smoothed seismicity in large background zones.

Two broad regions are defined with different maximum magnitudes in the USGS model: an extended margin zone (maximum magnitude [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 repeated large-magnitude earthquakes in the New Madrid Seismic Zone, and alternative zones for a Charleston seismic source zone (Figure 2.5.2-211). The maximum magnitude probability distribution assigned to the New Madrid fault sources is **M** 7.3 (0.15), **M** 7.5 (0.2), **M** 7.7 (0.5), and **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), and **M** 7.5 (0.15). The USGS model uses a mean recurrence time of 500 years and 550 years for repeated large-magnitude earthquakes in the New Madrid and Charleston regions, respectively, and assumes a time-independent model.

2.5.2.2.2.3 South Carolina Department of Transportation Seismic Hazard Map Study for Bridges and Highways

A probabilistic seismic hazard mapping project was completed in 2002 for the South Carolina Department of Transportation (SCDOT) as part of a program to develop seismic design specifications for highway bridges (Reference 2.5.2-242). The approach used in the SCDOT study is similar to that used by the USGS to develop the 2002 national seismic hazard maps. The SCDOT study uses a logic tree approach. It includes alternative source configurations as well as a smoothed seismicity approach for earthquakes in the magnitude range (5.0 < **M** < 7.0); alternative source models and maximum magnitudes for larger, repeated Charleston-type earthquakes (7.0 < **M** < 7.5) in the coastal areas of South Carolina; and alternative ground motion prediction models adopted by the USGS for the 2002 hazard maps. Alternative source areas defined for noncharacteristic earthquakes are shown on Figure 2.5.2-212.

The SCDOT source characterization for characteristic (i.e., repeated large-magnitude) Charleston-type earthquakes employs a combination of line and area sources and uses a slightly different M_{max} range (**M** 7.1 – 7.5) than the USGS 2002 characterization (Figure 2.5.2-211). Three equally weighted source zones defined for this study include (1) a fault zone consisting of three parallel faults that model a combined Woodstock and Ashley River fault scenario; (2) a larger Coastal South Carolina zone that includes most of the paleoliquefaction sites; and (3) a southern zone of river anomalies (postulated East Coast fault system) source zone. The magnitude distribution and weights used for M_{max} are **M** 7.1 (0.2), **M** 7.3 (0.6), and **M** 7.5 (0.2). The paleoliquefaction-based recurrence interval used in the SCDOT study is a mean recurrence interval of 550 years.

The LNP site lies within Source Area 9 as defined in the SCDOT study. This zone has not experienced sufficient seismicity to permit calculation of a recurrence model. The SCDOT study defined the recurrence model for this zone based on the seismicity rate per unit area defined for the adjacent Source Area 6. The geographic area of this Source Area 9 is defined to include transitional crust. The SCDOT study defines two alternative source zone configurations for the Piedmont and Coastal Plain region (Source Area 6), which also lie within the LNP site region. One configuration includes a zone of more concentrated seismicity in the South Carolina Coastal Plain (Source Area 7) and a localized zone in Charleston (Source Area 8). Recognizing that the borders between these zones are not well defined, an alternative configuration (Source Area 19) that includes South Carolina and adjacent parts of surrounding states was modeled using smoothed seismicity (Figure 2.5.2-212). A maximum magnitude of **M** 7.0 was used for all of the noncharacteristic source zones.

2.5.2.2.2.4 Updated PSHA for the Vogtle Plant Site

Southern Nuclear Company (SNC) updated the EPRI-SOG seismic source models in the vicinity of Charleston, South Carolina, to incorporate new information on the possible source of future large earthquakes similar to the 1886 Charleston earthquake, new assessments of the size of the 1886 earthquake, and new information on the occurrence rate for large earthquakes in the vicinity of Charleston, South Carolina (Reference 2.5.2-243). The result was the development of an updated Charleston seismic source (UCSS). The UCSS consists of the four alternative geometries for the seismic source, (shown on Figure 2.5.2-213) and the seismic source logic tree (Figure 2.5.2-214) that defines the weights assigned to the alternative geometries and the characterization of the size and frequency of large earthquakes associated with the source.

The UCSS model was used to define the location, size, and frequency of earthquakes similar to the 1886 earthquake. The occurrence of smaller earthquakes was modeled following the approaches developed in EPRI-SOG (Reference 2.5.2-201). The spatial distribution of earthquakes was modeled using the spatial smoothing approach developed in EPRI-SOG. SNC (Reference 2.5.2-243) integrated the UCSS into the EPRI-SOG seismic source characterization by replacing each EST's Charleston-specific source with the UCSS and modifying the remaining source geometries to accommodate the UCSS geometries. The frequency of earthquakes in these modified sources was modeled using the truncated exponential distribution and was based on analysis of the earthquake catalog.

2.5.2.2.2.5 FSAR South Texas Units 3 and 4 COLA

The South Texas Project (STP) Nuclear Operating Company (STPNOC) updated the EPRI-SOG seismic source parameters for Gulf of Mexico source zones as part of a recent COLA for the proposed STP Units 3 & 4 site near Bay City, Texas. The STP 3 & 4 FSAR incorporated contributions from seismic sources in the Gulf of Mexico that had not been included in the original EPRI methodology and updated the maximum magnitude probability distributions of Gulf of Mexico source zones based on the occurrence of two moderate earthquakes in the Gulf of Mexico (Reference 2.5.2-244)

2.5.2.3 Correlation of Earthquake Activity with Seismic Sources

Regulatory Guide 1.208 indicates that the earthquake activity should be correlated with seismic sources. The principal database for assessing earthquake recurrence is the historical and instrumental earthquake record. An updated catalog of independent historical and instrumental earthquakes covering the LNP site region was developed (see discussion in FSAR Subsection 2.5.2.1.1).

The distribution of earthquake epicenters from the EPRI (pre-1985) catalog, the more recent (post-1985) instrumental events, and updated historical earthquakes for the site region with respect to the EPRI-SOG sources are shown on Figures 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, 2.5.2-208, and 2.5.2-209. Comparison of the updated earthquake catalog to the EPRI-SOG earthquake catalog and EPRI-SOG sources yields the following conclusions:

- The updated earthquake catalog does not show a pattern of seismicity within the site region 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 based on seismicity patterns.

- The updated catalog does not show any earthquakes within the site region that can be associated with a known geologic structure.

- The closest principal source of seismic activity is the Charleston, South Carolina, area, which lies at a distance of greater than 430 km (267 mi.). Concentrations of seismicity in the vicinity of Charleston were recognized and considered by the EPRI-SOG teams, as discussed in FSAR Subsection 2.5.2.2.1.

- The largest historical earthquake in the southeastern United States, the 1886 Charleston earthquake, likely reactivated a structure within the basement rock, but cannot be definitely associated with any of the major identified basement structures (FSAR Subsection 2.5.1.1.4.4). Paleoliquefaction studies indicate that repeated large-magnitude earthquakes have occurred in the epicentral region of the 1886 Charleston earthquake (see discussion in FSAR Subsection 2.5.1.1.4.4). Alternative source locations, maximum magnitudes, and recurrence for repeated large-magnitude, Charleston-type earthquakes are discussed in FSAR Subsection 2.5.2.4.1.

- The updated catalog includes two earthquakes that are larger in magnitude than some of the upper- and/or lower-bound values used by EPRI-SOG teams to characterize the M_{max} distribution of source zones within which these earthquakes occurred. These earthquakes are the February 10, 2006, surface-wave magnitude (M_s) 5.3, body-wave magnitude (m_b) 4.2 earthquake (Emb 4.9), and the September 10, 2006, moment magnitude (**M**) 5.8-5.9, m_b 5.9, earthquake (Emb 6.0). These events require revisions to some of the ESTs' M_{max} distributions for background source zones, as described in FSAR Subsection 2.5.2.4.1.2.

- As discussed above in FSAR Subsection 2.5.2.1.2.2, the February 10, 2006, m_b 4.2, M_s 5.3 earthquake, which does not exhibit typical source characteristics of a tectonic earthquake, has been potentially associated with specific geologic structures near the edge of the continental shelf.

- The September 10, 2006, m_b 5.9, **M** 5.8-5.9 earthquake, which has a tectonic signature, has not been tied to any unique geologic structure. This event occurred near the transition between oceanic and thin transitional crust, in extended basement crust having northwest-trending normal faults that are favorably oriented for reactivation in the present tectonic regime (see discussion in FSAR Subsection 2.5.1.1.4.5).

- The February 10, 2006, m_b 4.2, M_s 5.3 earthquake has been proposed to be the result of a gravity-driven displacement on a shallow, low-angle detachment surface within or at the base of a thick sedimentary wedge (Reference 2.5.2-221), possibly related to a sediment-sediment contact (weld) at the base of a growth fault at the edge of the continental shelf (References 2.5.2-230 and 2.5.2-231). The smaller-magnitude April 18, 2006, earthquake that exhibits similar source characteristics is also attributed to similar gravity-driven processes. This event was neither detected nor located by the USGS (NEIC) and thus is not included in the updated earthquake catalog. This hypothesis suggests a potential association between seismicity in the Gulf of Mexico and normal growth faults at the edge of the continental shelf; however, no other events within the updated catalog have been attributed to such mechanisms. The edge of the continental shelf generally is encompassed by the various EST areal source zones for the Gulf of Mexico and environs, and as such, increases in M_{max} to account for the February 10, 2006 earthquake, as well as the September 10, 2006, m_b 5.9, **M** 5.8-5.9 earthquake adequately account for any potential association between earthquakes within the Gulf of Mexico and normal faults along the edge of the continental shelf. (Reference 2.5.2-244).

- The updated earthquake catalog adds a few 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 FSAR Subsection 2.5.2.4.1.3.

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

This subsection describes the PSHA conducted for the LNP site. Following the procedures outlined in Appendix E, Section E.3, of Regulatory Guide 1.208, FSAR Subsections 2.5.2.4.1 and 2.5.2.4.2 discuss new information on seismic source characterization and ground motion characterization, respectively, that is potentially significant relative to the EPRI-SOG (Reference 2.5.2-201) seismic hazard model. FSAR Subsection 2.5.2.4.3 presents the results of PSHA sensitivity analyses used to test the effect of the new information on the seismic hazard. Using these results, an updated PSHA analysis was performed, as described in FSAR Subsection 2.5.2.4.4. The results of that analysis are used for the development of uniform hazard response and identification of the controlling earthquakes (FSAR Subsection 2.5.2.4.4.2). The initial PSHA presented in this subsection utilizes a minimum magnitude for hazard integration of m_b 5.0, consistent with the original PSHA calculations performed by EPRI (Reference 2.5.2-202). The purpose of these calculations is to develop the controlling earthquakes for use in the site response analyses. The final PSHA to develop the GMRS is conducted using the CAV approach presented in Regulatory Guide 1.208. This calculation is described in FSAR Subsection 2.5.2.6.2.

2.5.2.4.1 New Information Relative to Seismic Sources

This subsection describes potential updates to the EPRI-SOG seismic source model. Seismic source characterization data and information that could affect the predicted level of seismic hazard include the following:

- Identification of possible additional seismic sources in the site region.
- 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.

Based on the review of new geological, geophysical, and seismological information that is summarized in FSAR Subsection 2.5.1, the review of seismic source characterization models developed for post-EPRI-SOG seismic hazard analyses (FSAR Subsection 2.5.2.2.2), and a comparison of the updated earthquake catalog to the EPRI-SOG evaluation (FSAR Subsection 2.5.2.3), the EPRI-SOG source models have been modified for the LNP 1 and LNP 2 COLA as follows:

- A UCSS developed by SNC (Reference 2.5.2-243) has been included to account for new information regarding the location, size, and occurrence of repeated large-magnitude earthquakes in the vicinity of Charleston, South Carolina.
- Two moderate earthquakes have occurred within the Gulf of Mexico since the EPRI-SOG 1986 - 1988 study. The magnitudes of these events exceed the upper and/or lower bound of the maximum magnitude (M_{max}) distributions originally proposed by some of the EPRI ESTs for large areal source zones that encompass large portions of the Gulf Coastal Plain and the Gulf of Mexico. Following the updated characterization initially developed by NuStart for the Grand Gulf Nuclear Station Unit 3 COLA (Reference 2.5.2-245) and implemented by STPNOC for the STP 3 & 4 COLA (Reference 2.5.2-244) M_{max} distributions have been revised for five of the six EPRI EST source zones to account for these earthquakes in the hazard calculations.
- An additional earthquake catalog completeness zone in the Gulf of Mexico has been added to incorporate the contribution of offshore seismicity into the hazard analysis for the LNP site.

2.5.2.4.1.1 Updated Charleston Seismic Source

The seismic source model for repeated large-magnitude, Charleston-type earthquakes is taken directly from the UCSS presented in SNC (Reference 2.5.2-243). The source for repeated large earthquakes at Charleston is modeled by the four alternatives shown on Figure 2.5.2-213. Earthquakes are modeled to occur as extended ruptures on closely spaced vertical faults oriented parallel to the long dimension of each source (Reference 2.5.2-243). The fault width was set at a depth of 20 km and rupture dimensions are modeled using an empirical relationship between rupture size and magnitude developed by Wells and Coppersmith (Reference 2.5.2-246).

SNC characterizes the occurrence of the repeated large earthquakes at Charleston by a characteristic earthquake model with the size and frequency of the characteristic earthquake defined by the parameters in the logic tree shown on Figure 2.5.2-214 (Reference 2.5.2-243).

The concept of a characteristic earthquake occurrence model was implemented in this study using the model developed by Youngs and Coppersmith, as modified by Youngs et al. (References 2.5.2-247 and 2.5.2-248). The magnitudes listed on Figure 2.5.2-214 are considered to represent the size of the expected maximum earthquake rupture for a repeated Charleston-type event. The size of the next characteristic earthquake is assumed to vary randomly about the expected value following a uniform distribution over a range of ±¼ magnitude units. This range represents the aleatory variability in the size of individual repeated large-magnitude, Charleston-type earthquakes. The alternative magnitude values listed in the logic tree represent epistemic uncertainty in the expected size of that earthquake.

SNC fully integrated the UCSS into the EPRI-SOG seismic source characterizations by modifying the geometry of the Charleston seismic sources defined by the EPRI-SOG ESTs (Reference 2.5.2-243). However, these sources are typically over 400 km (250 mi.) from the LNP site (Tables 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, and 2.5.2-207). Thus the details of their geometry, as it relates to the occurrence of smaller earthquakes, are not important to the hazard assessment for the LNP site. Accordingly, a simpler approach was adopted for incorporating the updated Charleston seismic source into the PSHA for the LNP site. The seismic source geometries shown on Figure 2.5.2-213 were used to model only the occurrence of repeated large-magnitude earthquakes in the vicinity of Charleston. The occurrence of all other earthquakes in the Charleston region was modeled using the EPRI-SOG seismic source interpretations (Reference 2.5.2-201). To eliminate double counting of the occurrence of large earthquakes near Charleston, the maximum magnitude distributions for the EPRI-SOG seismic sources related specifically to Charleston were limited to a maximum value of m_b 6.6, which is at the lower edge of the range of magnitudes for the repeated large earthquakes associated with the UCSS model. The EPRI-SOG Charleston seismic sources are indicated in Tables 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, and 2.5.2-207, and the modified maximum magnitude distributions are listed in the right-hand column of the tables. FSAR Subsection 2.5.2.4.3.3 provides further details.

2.5.2.4.1.2 New Maximum Magnitude Information

Geological and seismological data published since the 1986 EPRI seismic source model are summarized and discussed in FSAR Subsections 2.5.1 and 2.5.2.1, respectively. Based on a review of these data and the updated source characterizations implemented in the STP 3 & 4 COLA (Reference 2.5.2-244), the maximum magnitude distributions for some of the EPRI ESTs source zones that extend into the Gulf of Mexico and contain the LNP site (referred to as the Gulf Coastal Source Zones [GCSZ]) are revised. A comparison of the maximum magnitude distributions of EPRI EST characterizations of GCSZs and modifications for the STP 3 & 4 COLA is provided in Table 2.5.2-209.

The maximum magnitude distributions for some of the GCSZs were updated in the STP 3 & 4 COLA (Reference 2.5.2-244) based on the occurrence of two earthquakes that occurred after the development of the EPRI 1986 source model. The STP 3 & 4 COLA updated the maximum magnitude distribution for a particular GCSZ only when two conditions are met: (1) one or both of the 2006 moderate-magnitude earthquakes cannot be determined to have occurred outside the source zone with reasonable certainty, and (2) the observed Emb magnitude for the largest earthquake in the zone is greater than the minimum m_b magnitude of the EPRI 1986 source model maximum magnitude distribution. These criteria resulted in updates to five of the six EST GCSZs maximum magnitude distributions (Table 2.5.2-209).

The updated maximum magnitude distributions were developed by applying the maximum magnitude methodology developed by each EST to that EST's sources that contained one or both of the 2006 earthquakes. The STP 3 & 4 COLA used an Emb of 5.5 for the February 10, 2006 earthquake based on conversion from the reported M_s magnitude and an Emb of 6.1 for the September 10, 2006 earthquake based on conversions from the reported moment of magnitude. As discussed in FSAR Subsection 2.5.2.1.2.2 and indicated in Table 2.5.2-201, the reported m_b values for these two earthquakes are 4.2 and 5.9, respectively. Inclusion of the reported m_b values in the assessment of Emb results in values of 4.9 and 6.0 for the February 10 and September 10 earthquakes, respectively (Table 2.5.2-201). Use of these values to update the maximum magnitude distributions for the GCSZ would lead to small differences in the updated maximum magnitude distributions. Overall, the changes in the maximum magnitude distributions would be small and the values listed in Table 2.5.2-209 are slightly conservative compared to those that would be developed using Emb values of 4.9 and 6.0 for the February 10, 2006, and September 10, 2006, earthquakes, respectively. Therefore, the values listed in Table 2.5.2-209 were used in the updated PSHA calculation for the LNP site.

The maximum magnitude distributions for several other source zones were modified to account for the occurrence of another Emb 5.0 earthquake not associated with the GOM within their boundaries. These sources are Dames &

Moore source 52 (Table 2.5.2-203), Law Engineering sources 107 and 108 (Table 2.5.2-204), Rondout source 49 (Table 2.5.2-205), and Woodward-Clyde Consultants source B38 (Table 2.5.2-207).

2.5.2.4.1.3 Earthquake Occurrence Rates within EPRI-SOG Completeness Regions

FSAR Subsection 2.5.2.1.1 describes the development of an updated earthquake catalog for the LNP site region. This updated catalog includes modifications to the EPRI-SOG catalog by subsequent researchers, the addition of earthquakes that have occurred after completion of the EPRI-SOG seismic source characterization studies (post-March 1985), and the identification of additional earthquakes in the time period covered by the EPRI-SOG evaluation for the project region (1758 to March 1985). The effect 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 320 km (200 mi.) region around the LNP site.

The earthquake recurrence rates computed in the EPRI-SOG evaluation included a correction to remove bias introduced by uncertainty in the magnitude estimates for individual earthquakes (Reference 2.5.2-201). The bias adjustment was implemented by defining an adjusted magnitude estimate for each earthquake, m_b*, 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

$$m_b* = m_b - \beta\sigma^2_{m_b|m_b\,instrumental}/2$$ Equation 2.5.2-201

when m_b is based on instrumentally recorded m_b magnitudes and by the relationship

$$m_b* = m_b + \beta\sigma^2_{m_b|X}/2$$ Equation 2.5.2-202

when m_b is based on other size measures X, such as maximum intensity, I_0, or felt area (Reference 2.5.2-201). The change in sign in the correction term from negative in Equation 2.5.2-201 to positive in Equation 2.5.2-202 reflects the effects of the uncertainty in the conversion from size measure X to m_b. Parameter β 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 β equal to 0.95 × ln(10) based on the global *b*-value of 0.95 assigned to the CEUS by Frankel et al. (References 2.5.2-240 and 2.5.2-241). Values of $\sigma_{m_b|X}$ range from 0.55 for m_b estimated from maximum intensity, to 0.3 to 0.5 for m_b estimated from various other magnitude scales or felt area (Reference 2.5.2-201). The value of $\sigma_{m_b|m_b instrumental}$ is typically set at 0.1

The EPRI-SOG 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 (Reference 2.5.2-201). 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. The CEUS was subdivided into 13 "completeness" regions that represented different histories of earthquake recording. (Reference 2.5.2-201) Figure 2.5.2-215 shows the three completeness regions (2, 3, and 13) that cover the area within 320 km (200 mi.) of the LNP site. Note that the EPRI-SOG catalog contained only a few events in the Gulf of Mexico and no completeness region was defined for this area. The assessment of catalog completeness for the Gulf of Mexico and the incorporation of recent seismicity in that area is discussed in FSAR Subsection 2.5.2.4.1.4.

The total time span of the EPRI-SOG catalog was then divided into six time intervals. Then using the observed seismicity and information on population density and the history of earthquake reporting across the CEUS, the probability of detection was estimated for each time interval within each completeness region for six magnitude intervals. Earthquake recurrence estimates were then made using the "equivalent period of completeness," TE, for each completeness region and all of the recorded earthquakes within the usable portion of the catalog. The equivalent period of completeness is computed by the expression

$$T_{ij}^{E} = \sum_{k} T_k \times P_{ijk}^{D}$$

Equation 2.5.2-203

where P_{ijk}^{D} is the probability of detection for completeness region i, magnitude interval j, and time period k of length T_k (Reference 2.5.2-201). The estimated values of the probability of detection for all of the completeness regions are given in EPRI-SOG (Reference 2.5.2-201).

The updated earthquake catalog includes newly identified earthquakes for the time period covered by the EPRI-SOG catalog, reassessment of the sizes of previously identified events, and earthquakes that have occurred after completion of the EPRI-SOG evaluation. The event counts for the EPRI-SOG and updated catalogs are given in Table 2.5.2-210, where "Update-EPRI AS" indicates the updated catalog without the events flagged as aftershocks in the EPRI-SOG catalog. For the region within 320 km (200 mi.) of the LNP site, the difference in the number of earthquakes in the EPRI-SOG and updated catalog for the time up to 1985 is very small. The impact of the change in the number of events in particular time interval on the probability of detection within the EPRI-SOG completeness zones was approximately estimated by multiplying the value of P^D reported in EPRI-SOG (1986 - 1988) by the ratio of the earthquake count from the updated earthquake catalog to the earthquake count from the EPRI-SOG catalog, with a maximum value of 1.0 for the updated value of P^D. These assessments are presented in Table 2.5.2-210 for completeness regions 2 and

13. Completeness region 3 does not contain any earthquakes with 320 km (200 mi.) of the LNP site.

The effect of the updated earthquake catalog on earthquake occurrence rates was assessed by computing earthquake recurrence parameters for the portions of the EPRI-SOG completeness regions that lie within 320 km (200 mi.) of the LNP site. The truncated exponential recurrence model was fit to the seismicity data using maximum likelihood. Earthquake recurrence parameters were computed using the EPRI-SOG 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 January 1, 2007. The resulting earthquake recurrence rates for the portion of completeness region 13 with 320 km (200 mi.) of the LNP site are compared on Figure 2.5.2-216. The data labeled "Updated (all events)" includes earthquakes that were flagged as aftershocks in the EPRI-SOG catalog, and the data labeled "Update (no EPRI-SOG aftershocks)" have these events removed before calculating the recurrence parameters. Two sets of calculations were performed, one using unconstrained likelihood and one in which a prior of 1.0 was imposed on the *b*-value. EPRI-SOG (Reference 2.5.2-201) used the approach of applying a prior distribution for the *b*-value in the maximum likelihood estimation (MLE) of seismicity parameters. The use of the prior on *b*-value stabilized the estimate of seismicity parameters in areas with only a few earthquakes. For both sets of analyses (with and without a prior on *b*-value), the rates computed with the updated catalogs are lower than those obtained using the original EPRI-SOG catalog. Calculations were not performed for completeness in region 2 because the event count did not change, and region 3 does not have any events within 320 km (200 mi.) of the LNP site.

Based on comparisons shown on Figure 2.5.2-216, the earthquake occurrence rate parameters developed in the EPRI-SOG evaluation adequately represent the seismicity rates within 320 km (200 mi.) of the Levy site within the EPRI-SOG completeness regions. The impact of the seismicity in the Gulf of Mexico is assessed in FSAR Subsection 2.5.2.4.1.4.

2.5.2.4.1.4 Evaluation of Catalog Completeness within the Gulf of Mexico

The original EPRI completeness regions do not cover the Gulf of Mexico region. As a consequence, the earthquake recurrence parameters in that area had not been computed in the EPRI-SOG (Reference 2.5.2-201) study (Figure 2.5.2-215). Improved seismic networks have increased the detection of events in the Gulf of Mexico and the occurrence of the two moderate events discussed above in FSAR Subsection 2.5.2.1.1 indicates that seismicity in this area needs to be considered a potential contributor to the seismic hazard at the LNP site.

A new catalog completeness region covering the Gulf of Mexico has been created for this purpose. The region is bounded to the north by EPRI-SOG (Reference 2.5.2-201) completeness regions 2 and 3; to the east by region 13; and it extends south to latitude 24°N. The extent of this region is shown on

Figure 2.5.2-201 and its relationship to the EPRI-SOG EST seismic sources is shown on Figures 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, 2.5.2-208, and 2.5.2-209.

The probabilities of detection for the new Gulf of Mexico completeness region were estimated adopting the same procedure used in the EPRI-SOG study (Reference 2.5.2-201). The methodology employs a matrix of probability of detection of earthquakes for selected time and magnitude intervals. A time interval ranging from 1984 through 2006 was added to the intervals used in the EPRI-SOG (Reference 2.5.2-201) assessment to include the most recent earthquakes in the analysis.

The probabilities of detection for the Gulf of Mexico region were evaluated using the EPRI-SOG software package EQPARAM under the same conditions applied in the EPRI-SOG study (Reference 2.5.2-201), namely

- No spatial smoothing of parameter a.
- Medium smoothing of parameter b.
- Moderate smoothing of the probability of detection.
- Monotonicity in m_b and time interval.
- Probability of detection fixed to 1 for certain m_b and time intervals.

In addition, the probability of detection is not computed for the time intervals prior to 1950 because no events are reported prior to that date. Table 2.5.2-211 shows the assessed probabilities of detection for this region. These values were used to compute the earthquake occurrence parameters for the EPRI-SOG EST source zones that include portions of the Gulf of Mexico.

2.5.2.4.2 New Information Relative to Earthquake Ground Motions

2.5.2.4.2.1 Models for Median Ground Motions

The EPRI (Reference 2.5.2-202) calculation of seismic hazard characterized epistemic uncertainty in median (mean log) earthquake ground motions by using three strong-motion attenuation relationships: McGuire et al. (Reference 2.5.2-249), Boore and Atkinson (Reference 2.5.2-250), and Nuttli (Reference 2.5.2-251) combined with the response spectral relationships of Newmark and Hall (Reference 2.5.2-252). These relationships were based to a large extent on modeling earthquake ground motions using simplified physical models of earthquake sources and wave propagation.

Estimating earthquake ground motions in the CEUS has been the focus of considerable research since completion of the EPRI-SOG studies. The research has produced a number of ground motion attenuation relationships. EPRI completed a study in 2004 to update methods used to characterize the estimation of strong ground motion in the CEUS for application in PSHA for nuclear facilities (Reference 2.5.2-253). This study was conducted following the SSHAC guidelines for a Level III analysis (Reference 2.5.2-239). SSHAC provided guidance on the appropriate methods to use for quantifying uncertainty in evaluations of seismic hazard (Reference 2.5.2-239). In a SSHAC Level III analysis, the responsibility for developing the quantitative description of the uncertainty distribution for the quantity of interest lies with an individual or team designated the Technical Integrator. The Technical Integrator is guided by a panel of experts whose role is to provide information, advice, and review. In the EPRI study, a panel of six ground motion experts was assembled (Reference 2.5.2-253). During a series of workshops, the experts provided advice on the available CEUS ground motion attenuation relationships that were considered appropriate for estimating strong ground motion in the CEUS. The experts also provided information on the appropriate criteria for evaluating the available ground motion models. The Technical Integrator then used this information to develop a composite representation of the current scientific understanding of ground motion attenuation in the CEUS.

The EPRI study recommended four alternative sets of median ground motion models (termed model clusters) to represent alternative modeling approaches for defining the median ground motions as a function of earthquake magnitude and source-to-site distance (Reference 2.5.2-253). Three of these ground motion clusters are appropriate for use in assessing the hazard from moderate-sized local earthquakes occurring randomly in source zones, and all four are to be used for assessing the hazard from sources whose hazard contribution is from large-magnitude earthquakes.

EPRI (Reference 2.5.2-253) proposed the logic tree structure to be used with these models that is shown on the left-hand side of Figure 2.5.2-217. The first (leftmost) level of the logic tree shown in the figure provides the weights assigned to the three median cluster models appropriate for local sources. The second level addresses the appropriate ground motion cluster median model to use for large-magnitude distant earthquake sources. For the LNP site, these sources are Charleston-related sources (those defined in both the EPRI-SOG model, listed in Tables 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, and 2.5.2-207, and the UCSS model, for repeated large-magnitude earthquakes). Two alternatives are provided: to use the cluster model used for the local sources or to use the cluster 4 model. The effect of this logic structure on the PSHA is that by following the branch for cluster 1 at the first node, two options are available: (1) use the cluster 1 model for the large-magnitude sources, and (2) use cluster 4 for the large-magnitude sources and cluster 1 for all other sources. This same logic is repeated for the branches for clusters 2 and 3. The rift version of the cluster 4 model was used for the Charleston sources.

EPRI provided estimates of the epistemic uncertainty in the median ground motion model for each cluster (Reference 2.5.2-253). As shown by the third level of the logic tree (Figure 2.5.2-217), the uncertainty in each cluster median model is modeled by a three-point discrete distribution with ground motion relationships for the 5th, 50th, and 95th percentiles of the epistemic uncertainty in the median attenuation relationship for each ground motion cluster.

The EPRI (Reference 2.5.2-253) ground motion median models for clusters 1 and 2 were based in large part on the CEUS ground motion models developed by Silva et al. (Reference 2.5.2-254) and Atkinson and Boore (Reference 2.5.2-255), respectively. Silva et al. (Reference 2.5.2-256) and Atkinson and Boore (Reference 2.5.2-257) have since developed updated versions of their models. These newer models are compared to the EPRI (Reference 2.5.2-253) models on Figure 2.5.2-218.

- The two plots on the left compare the EPRI (Reference 2.5.2-253) 5th percentile, 50th percentile, and 95th percentile 10-Hz and 1-Hz median models for ground motion cluster 1 with the three single-corner stochastic models developed by Silva et al. (Reference 2.5.2-256). The updated models all fall well within the range of the EPRI (Reference 2.5.2-253) models.

- The two plots on the right compare the EPRI (Reference 2.5.2-253) 5th percentile, 50th percentile, and 95th percentile 10-Hz and 1-Hz median models for ground motion cluster 2 with the model developed by Atkinson and Boore (Reference 2.5.2-257). The Atkinson and Boore (Reference 2.5.2-257) model uses rupture distance as the distance measure, while the EPRI (Reference 2.5.2-253) cluster 2 models use Joyner-Boore distance. The comparisons shown on Figure 2.5.2-218 were made assuming that the top of rupture for the **M** 5 earthquake is at a depth of 4 km (2.5 mi.), based on a mean point-source depth of 6 km (3.7 mi.) (Reference 2.5.2-254). The median ground motions produced by the updated Atkinson and Boore (Reference 2.5.2-257) model fall within the range of the EPRI (Reference 2.5.2-253) cluster 2 medians except for distances less than about 7 km (4.3 mi.) for large-magnitude earthquakes.

As presented in FSAR Subsection 2.5.2.4.4, large-magnitude earthquakes at very small distances are not a significant contributor to the hazard. On the basis of the comparisons shown on Figure 2.5.2-218, it is concluded that the EPRI (Reference 2.5.2-253) median ground motion models are appropriate for use in computing the hazard for the LNP site.

2.5.2.4.2.2 Models for Ground Motion Aleatory Variability

The EPRI (Reference 2.5.2-253) study also provided a characterization of the aleatory variability in CEUS ground motions based on an assessment of information available at the time. More recently, EPRI conducted a study focused in part on evaluating the appropriate aleatory variability for CEUS ground motions (Reference 2.5.2-258). The thrust of the study was to identify reasons why the aleatory variability for CEUS motions may be different than that observed for the large empirical database of strong ground motion in the western United States and other tectonically active regions, and then evaluate the extent to which these reasons are supported by empirical data. The result of the EPRI study was a recommended model for aleatory variability for CEUS ground motions (Reference 2.5.2-258).

The EPRI (Reference 2.5.2-258) model for aleatory variability in CEUS ground motions is represented by the fourth and fifth levels of the ground motion logic tree shown on Figure 2.5.2-217. The fourth level of the logic tree addresses the overall aleatory model. Two alternatives were defined: (1) model 1A is based on WUS aleatory variability with an additional component of intra-event variability for CEUS earthquakes and (2) Model 1B is unmodified WUS aleatory variability. Model 1A was favored based on the available data.

The EPRI included an additional component of aleatory variability to account for variability in source depth at small source-to-site distances when the Joyner-Boore distance measure is used for ground motion models based on point-source numerical simulations (Reference 2.5.2-253). EPRI (Reference 2.5.2-258) evaluated the empirical evidence for additional aleatory variability at small Joyner-Boore distances and concluded that the adjustments proposed by EPRI (Reference 2.5.2-253) were not supported by empirical data. Instead, three alternatives were recommended:

1. Model 2A — no adjustment.

2. Model 2B — an additional 0.12 standard error in the natural log of ground motion amplitude.

3. Model 2C — an additional 0.23 standard error.

The additional standard error is to be combined with model 1A or 1B as the sum of variances to produce the final standard error for Joyner-Boore distances less than or equal to 10 km. A log-linear decrease in the additional standard error is to be applied over the distance range of 10 to 20 km, with no additional adjustment for distances greater than 20 km. These alternative models define the fifth level of the logic tree shown on Figure 2.5.2-217. These additional standard error models are applied to the EPRI median models that use the Joyner-Boore distance measure (clusters 1, 2, and 4) (Reference 2.5.2-253).

2.5.2.4.2.3 Conversion from Body Wave to Moment Magnitude

The last level of the ground motion logic tree shown on Figure 2.5.2-217 addresses the relationship between body-wave magnitude, m_b, and moment magnitude, **M**. This conversion is required because the EPRI (Reference 2.5.2-253, Reference 2.5.2-258) ground motion models are defined in terms of **M**, whereas the EPRI-SOG recurrence rates are defined in terms of m_b. The epistemic uncertainty in the conversion between m_b and **M** was addressed by using the three m_b-**M** relationships.

(1) By Atkinson and Boore (Reference 2.5.2-255):

$$\mathbf{M} = -0.39 + 0.98 m_b \qquad \text{for } m_b \le 5.5$$

Equation 2.5.2-204

$$\mathbf{M} = 2.715 - 0.277 m_b + 0.127 m_b^2 \qquad \text{for } m_b > 5.5$$

(2) By Johnston (Reference 2.5.2-259):

$$\mathbf{M} = 1.14 + 0.24 m_b + 0.0933 m_b^2$$

Equation 2.5.2-205

(3) By EPRI (Reference 2.5.2-260):

$$m_b = -10.23 + 6.105\mathbf{M} - 0.7632\mathbf{M}^2 + 0.03436\mathbf{M}^3$$

Equation 2.5.2-206

These three models are assigned equal weight, as the models are all credible.

2.5.2.4.3 PSHA Sensitivity Analysis

This subsection describes the sensitivity studies that were carried out to address any need for changes in the EPRI-SOG PSHA model used in EPRI (Reference 2.5.2-202). Based on the assessments in FSAR Subsection 2.5.2.4, and consistent with the requirements of Regulatory Guide 1.208, the following PSHA model adjustments were studied as part of PSHA sensitivity tests for the LNP site:

- Selection of appropriate set of seismic sources for each EPRI-SOG EST.
- Sensitivity to new data relative to the occurrence of large earthquakes in the Charleston, South Carolina, region.
- Sensitivity to the updated maximum magnitude distributions for seismic sources extending into the Gulf of Mexico.
- Sensitivity to the updated seismicity parameters for seismic sources extending into the Gulf of Mexico.

Sensitivity analyses were not conducted to address the effect of the updated ground motions models developed by EPRI (References 2.5.2-253 and 2.5.2-258) because these have become the standard set of models for the assessment of seismic hazards for proposed new power plants.

As discussed above in FSAR Subsection 2.5.2.2.1, the specific subset of EPRI-SOG seismic sources to include for each EPRI-SOG EST was assessed using updated ground motion models. The selection of the appropriate set is based on the contribution of individual sources to the total hazard at the site. The assessment of the contribution of more distant sources will be affected by the level of hazard contributed by the local sources. FSAR Subsection 2.5.2.4.1.2 presents revised maximum magnitude distributions for sources that extend into the Gulf of Mexico and FSAR Subsection 2.5.2.4.1.4 discusses updated calculation of seismicity parameters for an extension of the EPRI-SOG catalog completeness regions into the Gulf of Mexico. Both of these updates to the EPRI-SOG seismic source characterization are expected to be implemented in the PSHA for the LNP site. Therefore, these modifications were made prior to the assessment of the appropriate set of EPRI-SOG seismic sources.

2.5.2.4.3.1 Selection of EPRI-SOG Seismic Sources

The specific subset of EPRI-SOG seismic sources to include for each EST was assessed using the updated EPRI ground motion models that will be used to compute the PSHA for the LNP site (References 2.5.2-253 and 2.5.2-258). The sources examined included those within 320 km (200 mi.) of the site and those at larger distances with somewhat higher rates of seismicity, such as sources in the vicinity of Charleston, South Carolina, and eastern Tennessee. These calculations were performed for each individual team. Seismic sources were added until additional sources produced less than a 1 percent increase in the frequency of exceedance in the 10^{-4} to 10^{-5} range. The source contributions were tested for 10-Hz and 1-Hz ground motions. The calculations were performed using the preferred set of ground motion models for each ground motion cluster (i.e., the highest weighted path through the logic tree for each ground motion cluster). This corresponds to use of the 50th percentile cluster median model and aleatory variability models 1A and 2A. A single m_b-**M** conversion relationship was used (Reference 2.5.2-255). The modification to the maximum magnitude distributions and seismic parameters for Gulf of Mexico seismic sources discussed in FSAR Subsection 2.5.2.4.1.2 and FSAR Subsection 2.5.2.4.1.4 were applied to the Gulf of Mexico seismic sources as part of this assessment.

EPRI (Reference 2.5.2-253) provided ground motion models for two regions of the CEUS, the mid-continent region that covered most of CEUS, and the Gulf Coast Region. The Gulf Coast Region was originally defined by EPRI (Reference 2.5.2-260) as an area with a higher rate of ground motion attenuation than the remaining portion of the CEUS. This region is shown in relationship to the EPRI-SOG EST sources on Figures 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, 2.5.2-208, and 2.5.2-209. The Gulf Coast ground motion models were used for those sources where the travel path is primarily thought the Gulf Coast region and the Mid-continent model was used for those sources where a substantial portion of the travel path is through the Mid-continent region. The use of these two models for specific sources is indicated in Tables 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, and 2.5.2-207. Note that various crustal regions defined in EPRI (Reference 2.5.2-260) did not include the southern half of the Florida peninsula, apparently due to lack of data for this portion of the CEUS. It is assumed in this analysis that the southern half of Florida should be included along with the northern half in the Gulf Coast Region for the purpose of selection of the appropriate EPRI (2004) (Reference 2.5.2-260) ground motion models.

2.5.2.4.3.1.1 Bechtel Team's Seismic Sources

Figure 2.5.2-219 shows the mean hazard curves computed for the Bechtel team's sources listed in Table 2.5.2-202. The Gulf Coast model ground motion was applied only to Source BZ1. This source is the largest contributor to the hazard at the LNP site from the Bechtel source model.

2.5.2.4.3.1.2 Dames & Moore Team's Seismic Sources

Figure 2.5.2-220 shows the mean hazard curves computed for the Dames & Moore team's sources listed in Table 2.5.2-203. The implementation of the seismic source model for the Dames & Moore team followed the approach used in the HAR COLA (Reference 2.5.2-261) in that Sources 41 and 53 were considered to be active in some form with probability 1.0. Source 53 interacts with Sources 52 and 54. As originally defined, Source 53 overlays both of these sources. For this analysis, modified versions of Source 53 were developed that excluded the region occupied by Source 54, which has a P* = 1, and when Source 52 is active, only occupied the region north of Sources 52 and 54.

The Gulf Coast ground motion model was applied only to Source 20. This source is the largest contributor to the hazard at the LNP site from the Dames & Moore source model.

2.5.2.4.3.1.3 Law Engineering Team's Seismic Sources

Figure 2.5.2-221 shows the mean hazard curves computed for the Law Engineering team's sources listed in Table 2.5.2-204. The Gulf Coast ground motion model was applies to Source 126 and the southern part of Source 8. Source 126 is the largest contributor to the hazard at the LNP site from the Law Engineering source model.

2.5.2.4.3.1.4 Rondout Associates Team's Seismic Sources

Figure 2.5.2-222 shows the mean hazard curves computed for the Rondout Associates team sources listed in Table 2.5.2-205. The Gulf Coast ground motion model was applies to Sources 51, 13, and the southern part of Source 49. Source 49 is the largest contributor to the hazard at the LNP site from the Rondout Associates source model.

2.5.2.4.3.1.5 Weston Geophysical Team's Seismic Sources

Figure 2.5.2-223 shows the mean hazard curves computed for the Weston Geophysical team's sources listed in Table 2.5.2-206. The Gulf Coast ground motion model was applies to Source 107. Source 107 is the largest contributor to the hazard at the LNP site from the Weston Geophysical source model.

The Weston Geophysical Teams Source 103 includes a number of alternative geometries depending on whether or not sources within the boundary are considered active. These alternatives were tested and it was found that the combination of Source 24 active within the boundary of Source 103 produced slightly higher hazard than the other cases. This combination was used in the updated hazard analysis. The hazard curve for Source 103 shown on Figure 2.5.2-224 includes the contribution of Source 24.

2.5.2.4.3.1.6 Woodward-Clyde Consultants Team's Seismic Sources

Figure 2.5.2-224 shows the mean hazard curves computed for the Woodward-Clyde Consultants team's sources listed in Table 2.5.2-207. The Gulf Coast ground motion model was applies to Source B36. This source is the largest contributor to the hazard at the LNP site from the Woodward-Clyde Consultants source model.

The two alternative geometries for Source 31 were tested. It was found that they produced very similar hazard, with Alternative 31 producing slightly higher hazard than Alternative 31A. In order to simplify the model, only the Alternative 31 is shown on Figure 2.5.2-224 and this alternative was used in the updated seismic hazard analysis.

2.5.2.4.3.2 PSHA Sensitivity to Revisions of the EPRI-SOG Sources

FSAR Subsection 2.5.2.4.1.2 discusses modifications to the maximum magnitude distributions for EPRI-SOG seismic sources that extend into the Gulf of Mexico and encompass the location of one or both of the moderate magnitude earthquakes that occurred in 2006. The effect of these modified maximum magnitude distributions on the hazard from the EPRI-SOG seismic sources is shown on Figure 2.5.2-225. The modified maximum magnitude distributions primarily affect the source zones in which the LNP site is located and the result is an appreciable increase in the hazard.

FSAR Subsection 2.5.2.4.1.4 presents an updated assessment of catalog completeness and seismicity parameters for the region in the Gulf of Mexico that was not included in the original EPRI-SOG calculation of seismicity parameters. The affect of including the updated seismicity rates for these sources is also shown on Figure 2.5.2-225. The result is a small increase in the hazard from sources that extend into the Gulf of Mexico in the vicinity of the site.

In summary, the modifications to the maximum magnitude distributions to account for the occurrence of the 2006 Gulf of Mexico earthquakes and to incorporate Gulf of Mexico seismicity lead to a combined appreciable increase in the hazard at the LNP site from the EPRI-SOG seismic sources, and these modifications are incorporated into the updated PSHA for the LNP site. The limitation on the maximum magnitude distribution for EPRI-SOG Charleston-specific seismic sources is considered to be appropriate to prevent double counting of the occurrence of large-magnitude earthquakes near Charleston and is also incorporated into the updated PSHA for the LNP site.

2.5.2.4.3.3 Additional Seismic Sources

The second set of sensitivity analyses test the effect of incorporating sources of repeated large-magnitude earthquakes at Charleston. That portion of the UCSS model that defines a source for repeated large earthquakes near Charleston was implemented (Reference 2.5.2-230). The main features of this model are described in FSAR Subsection 2.5.2.4.1.1. The four alternative source geometries shown on Figure 2.5.2-213 were modeled by a series of closely spaced vertical faults parallel to the long axis of the source. Earthquakes were modeled as extended ruptures on these faults using the relationship between magnitude and rupture area defined by Wells and Coppersmith for all slip types (Reference 2.5.2-246). The epistemic uncertainty in the expected magnitude of the repeated large earthquakes occurring on this source was modeled by the weighted alternatives in the UCSS logic tree shown on Figure 2.5.2-214. The aleatory variability in the magnitude of individual earthquakes is assumed to vary randomly about the expected value following a uniform distribution over a range of ±¼ magnitude units.

The lognormal distributions for the uncertainty in the recurrence interval of the repeated earthquakes defined on Figure 2.5.2-214 were modeled by the 5-point discrete approximation to a continuous distribution developed by Miller and Rice (Reference 2.5.2-262). The discrete recurrence interval values, the associated weights, and the resulting equivalent annual frequencies are listed in Table 2.5.2-212.

Figure 2.5.2-225 compares the hazard computed from the UCSS model with that obtained from the updated EPRI-SOG model described in FSAR Subsection 2.5.2.4.3.1. The UCSS source produces exceedance frequencies for 10-Hz motions that are larger than those produced by the updated EPRI-SOG sources for exceedance frequencies in the range of 10^{-3} to 10^{-5}. For 1-Hz motions, the hazard produced by the UCSS model exceeds by a large margin the hazard produced by the updated EPRI-SOG sources for exceedance frequencies less than 10^{-3}. These results indicate that the UCSS in a major contributor to the hazard and it was incorporated into the updated PSHA for the LNP site.

As discussed in FSAR Subsection 2.5.2.4.1.1, the UCSS is used to model the occurrence of repeated large magnitude earthquakes near Charleston. To prevent double counting of the occurrence of these events, the maximum magnitude distributions of Charleston-specific sources defined by the EPRI-SOG ESTs were limited to a maximum of m_b 6.6. Figure 2.5.2-225 shows the effect of this modification on the computed hazard from the EPRI-SOG seismic sources. The modification results in a slight decrease in the 10-Hz spectral acceleration hazard and a small decrease in the 1-Hz spectral acceleration hazard. These small decreases are more that made up for by the addition of the contribution from the UCSS.

2.5.2.4.4 PSHA for the LNP Site

The PSHA for the LNP site was conducted using the updated EPRI-SOG seismic sources described in FSAR Subsection 2.5.2.4.2.1 combined with the UCSS source described in FSAR Subsection 2.5.2.4.2.2. Earthquake ground motions were modeled using the median ground motion models and the ground motion aleatory variability models developed by EPRI (References 2.5.2-253 and 2.5.2-258). The logic tree defining the epistemic uncertainty in the ground motion characterization is shown on Figure 2.5.2-217.

The hazard analysis was conducted using the m_b magnitude scale because the earthquake occurrence rates for the EPRI-SOG seismic sources are defined in terms of m_b magnitudes. Epistemic uncertainty in the conversion from m_b magnitudes to moment magnitudes for ground motion estimation was modeled by using the three equally weighted conversion relationships listed on Figure 2.5.2-217. Conversion of the moment magnitude estimates for the size of the repeated earthquakes associated with the UCSS into m_b magnitudes for summation of the hazard was done in a consistent manner such that the original value of **M** was recovered for ground motion estimation. For example, when the Atkinson and Boore (Reference 2.5.2-255) relationship was used to convert m_b to **M** for ground motion estimation, its inverse was used to convert the **M** values for the UCSS earthquakes into m_b values.

Earthquakes occurring in the EPRI-SOG seismic sources were modeled as point sources, and the EPRI models for distance adjustment and additional aleatory variability resulting from the use of point sources (epicenter) to model earthquakes were applied (Reference 2.5.2-253). The models based on the assumption of a random rupture location with respect to the epicenter were used. Earthquakes occurring on the UCSS source of repeated large earthquakes and postulated ECFS sources were modeled as extended ruptures, and the distance adjustment and additional aleatory variability models were not applied to these sources.

EPRI concluded that there was no basis for truncation of the lognormal distribution for ground motion amplitude other than the strength of the subsurface materials (Reference 2.5.2-258). Accordingly, untruncated lognormal distributions for earthquake ground motions were used in the PSHA.

The EPRI ground motion models represent the ground motions for a generic hard rock condition in the CEUS (Reference 2.5.2-253). Thus the site-specific PSHA results presented in this subsection represent the motions on outcropping rock with a shear-wave velocity in excess of about 2743 m/sec (9000 ft/sec). The effect of the sediments overlying this generic rock condition on defining the hazard at other locations is addressed in FSAR Subsections 2.5.2.5 and 2.5.2.6.

The initial generic CEUS hard rock hazard was computed using a fixed lower-bound magnitude of m_b 5.0. These results were used to develop the appropriate response spectra and time histories for the site response analyses. Once the site amplification functions were developed, a second hazard assessment was performed incorporating the CAV approach to define the minimum magnitude truncation for the PSHA.

2.5.2.4.4.1 PSHA Results for Generic Hard Rock Conditions

PSHA calculations were performed for response spectral accelerations at the seven structural frequencies provided in the EPRI ground motion model: 0.5, 1.0, 2.5, 5, 10, 25, and 100 Hz (peak ground acceleration [PGA]) (Reference 2.5.2-253). Figures 2.5.2-226, 2.5.2-227, 2.5.2-228, 2.5.2-229, 2.5.2-230, 2.5.2-231, and 2.5.2-232 show the resulting mean hazard curves and the 5th, 16th, 50th (median), 84th, and 95th fractile hazard curves for each ground motion measure. These values are listed in Tables 2.5.2-213, 2.5.2-214, 2.5.2-215, 2.5.2-216, 2.5.2-217, 2.5.2-218, and 2.5.2-219. At low spectral frequencies (≤1 Hz) the mean hazard approaches or exceeds the 84th percentile hazard due to the relatively large epistemic uncertainty in the ground motion models at these frequencies as compared to that for higher-frequency ground motions (e.g., see Figure 2.5.2-218).

Figure 2.5.2-233 shows the contribution of the two source types to the mean hazard for 10-Hz and 1-Hz spectral acceleration. As was found in the sensitivity test described in FSAR Subsection 2.5.2.4.2.2, the UCSS produces comparable hazard or somewhat larger hazard than that obtained from the updated EPRI-SOG sources for 10-Hz motions, and dominates the hazard for 1-Hz motions.

Figure 2.5.2-234 shows the effect of the alternative ground motion cluster models on the mean hazard. As described in FSAR Subsection 2.5.2.4.2.1, the cluster 4 model is only used for seismic sources where the hazard is dominated by large-magnitude earthquakes. Thus, the results labeled cluster 4 represent the mean hazard computed assigning a weight of one to the use of cluster 4 for the large-magnitude sources (e.g., the repeated large earthquake source at Charleston) combined with the weighted average of the hazard obtained from the other three cluster models for all other sources. In general, use of the cluster 3 ground motion model produces the highest hazard.

Figure 2.5.2-235 shows the effect of the epistemic uncertainty in the median ground motion models for each cluster on the mean hazard, respectively. The uncertainty in the hazard is somewhat greater for low-frequency motions than for high-frequency motions, reflecting greater uncertainty in the median low-frequency ground motion models. Examination of the hazard results concluded that the alternative aleatory variability models developed by EPRI (Reference 2.5.2-258) produced similar hazard.

Figure 2.5.2-236 shows the effect of using the alternative m_b-**M** conversion relationships on the computed mean hazard. Similar estimates of seismic hazard are obtained using each of the relationships. The effect of the alternative models on the hazard disappears at ground motion levels where the hazard is dominated by the UCSS. As discussed previously, the alternative models were used in such a way that the moment magnitudes for the repeated large earthquakes specified on Figure 2.5.2-214 are always used for ground motion estimation.

Figure 2.5.2-237 shows the range in the computed hazard from just the updated EPRI-SOG sources and the mean hazard obtained from the seismic source models for the individual teams. The difference between the individual teams' results is somewhat greater for 10-Hz motion than for 1-Hz motions.

The other model uncertainties that were found to have a significant contribution to the uncertainty in the hazard were the uncertainty in the seismicity parameters for the 10-Hz motions and the uncertainty in the expected magnitude of the repeated large earthquakes occurring on the UCSS.

2.5.2.4.4.2 Uniform Hazard Response Spectra for Generic CEUS Rock and Identification of Controlling Earthquakes

The mean hazard results listed in Tables 2.5.2-213, 2.5.2-214, 2.5.2-215, 2.5.2-216, 2.5.2-217, 2.5.2-218, and 2.5.2-219 were interpolated to obtain uniform hazard response spectra (UHRS) for generic CEUS hard rock conditions. The spectra were computed for mean annual frequencies of exceedance of 10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6}. These spectra are shown on Figure 2.5.2-238 and listed in Table 2.5.2-220.

Figures 2.5.2-239, 2.5.2-240, 2.5.2-241, and 2.5.2-242 show the deaggregation of the mean hazard for the four values of exceedance frequency. Following the procedure outlined in Appendix D of Regulatory Guide 1.208, the deaggregation is conducted for two frequency bands: (1) the average of the 5-Hz and 10-Hz hazard results representing the high-frequency (HF) range and (2) the average of the 1-Hz and 2.5-Hz hazard results representing the low-frequency (LF) range. The results shown on the figures were obtained by first computing the percentage contribution of events in each magnitude-distance bin individually for the four spectral frequencies (1, 2.5, 5, and 10 Hz). The HF deaggregation was then obtained by averaging these values for 5 and 10 Hz and the LF deaggregation obtained by averaging the results for 1 and 2.5 Hz. The HF deaggregation shows a progression from domination of the hazard by large, distant earthquakes at a mean exceedance frequency of 10^{-3} to dominance by nearby small-magnitude earthquakes at a mean exceedance frequency of 10^{-6}. This effect can be seen in the change in shapes of the UHRS, which become more sharply peaked at 25 Hz as the contributions from nearby smaller-magnitude earthquakes increase. The LF deaggregation indicates that the distant large-magnitude earthquakes dominate the hazard at all four levels of exceedance frequency.

Appendix D of Regulatory Guide 1.208 specifies how the deaggregation results are used to define what are called controlling earthquakes for the HF and LF motions. These earthquakes represent the weighted mean magnitude and weighted geometric mean distance, where the weights are defined by the relative contributions to the total hazard for each magnitude and distance interval. Table 2.5.2-221 lists the mean magnitudes and geometric mean distances computed for the HF and LF spectral frequency ranges for the four mean annual frequency of exceedance levels. The values for the LF hazard are listed considering all earthquakes and considering only those earthquakes occurring at distances greater than 100 km, consistent with the procedure outlined in Appendix D of Regulatory Guide 1.208.

The approach to be used to compute the effects of the LNP site sediments on the generic hard rock motions is Approach 2B for site response analyses described in NUREG/CR-6728 (Reference 2.5.2-263). This approach defines what are called reference earthquakes (RE). The REs are defined in the same manner as the controlling earthquakes defined in Appendix D of Regulatory Guide 1.208.

Comparison of the computed controlling or reference earthquake magnitudes and distances with the deaggregation results indicates that in many cases the mean magnitude and mean distance correspond to a magnitude-distance bin that has a relatively small contribution to the hazard, particularly for the HF hazard results. Site response Approach 2B addresses this problem by using a range of magnitude-distance pairs to reflect the distribution of earthquakes contributing to the HF and LF hazard. Typically, three deaggregation earthquakes (DEs) at high frequency and three at low frequency are adequate to represent the distribution of earthquakes contributing to the hazard. These are designated DEL, DEM, and DEH for the low-magnitude, middle-magnitude, and high-magnitude deaggregation earthquakes, respectively. The site response uses ground motions representative of the DEL, DEM, and DEH as input ground motions.

For the LNP site, the DEL, DEM, and DEH magnitude-distance values were defined to represent the modes in the magnitude-distance deaggregation. As shown by the red-blue-green color coding on Figures 2.5.2-239, 2.5.2-240, 2.5.2-241, and 2.5.2-242, three magnitude-distance domains were identified that represent peaks in the deaggregated hazard and that, in combination, account for greater than 99 percent of the hazard. The deaggregation earthquake magnitude and distances are computed as the weighted mean values over the defined domains. The resulting DEs are listed in Table 2.5.2-221. The weight assigned to each DE is defined by the relative contribution of the earthquakes in the magnitude-distance domain to the total hazard. The resulting weights are listed in the right-hand column of Table 2.5.2-221. The weighted combination of the DEs also produces a magnitude-distance pair that is very close to the RE.

2.5.2.4.4.3 Response Spectra for Reference and Deaggregation Earthquakes

Smooth response spectra were developed to represent each of the reference and deaggregation earthquakes listed in Table 2.5.2-221. These spectra were developed using the EPRI (Reference 2.5.2-253) median ground motions models, the EPRI (Reference 2.5.2-258) aleatory variability models, and the spectral shape functions for CEUS ground motions presented in McGuire et al (Reference 2.5.2-263).

The DEs are intended to represent the motions from earthquakes that are contributing to the hazard in a specific frequency range, either 1 to 2.5 Hz (LF) or 5 to 10 Hz (HF) for the purpose of computing site amplification functions. The development of the appropriate spectral shapes for the DEs uses the concept of the conditional mean spectrum developed by Baker and Cornell (Reference 2.5.2-264). The conditional mean spectrum is defined as the expected earthquake spectrum given that the spectral acceleration matches a specific value at a specific frequency. This spectrum is constructed taking into account the correlation between response spectral amplitudes at two different frequencies observed in strong ground motion. For example, the 10^{-4} UHRS amplitude at a frequency of 10 Hz may represent the 84th percentile ground 10-Hz spectral acceleration based on the DEL magnitude and distance and one of the EPRI ground motion models (References 2.5.2-253 and 2.5.2-258). Given that the spectral acceleration at 10 Hz represents a 1-epsilon ground motion, the expected value of epsilon at other frequencies is equal to the epsilon at 10 Hz multiplied by the correlation coefficient between the motions at 10 Hz and other frequencies. The resulting conditional mean spectrum represents the expected frequency content of earthquake motions that produce ground motions equal to the UHRS at the target frequency of 10 Hz.

Baker and Cornell developed a model for the correlation coefficient between spectral accelerations at any two frequencies (Reference 2.5.2-265). Their model covered the frequency range of 0.2 to 20 Hz. Baker and Jayaram (Reference 2.5.2-266) have extended the Baker and Cornell (Reference 2.5.2-265) model to cover the frequency range of 0.1 to 100 Hz.

This extended model was used to compute conditional mean spectra for the DEs. As an example, the 10^{-4} DEH for LF is listed in Table 2.5.2-221 as an m_b 7.1 earthquake occurring at a distance of 459 km from the site. A combination of a median ground motion model, aleatory variability model, and m_b-**M** conversion defined in the ground motion model logic tree (Figure 2.5.2-217) is used to compute number of standard deviations (denoted by ε) that the 1 Hz and 2.5 Hz 10^{-4} UHRS accelerations lies away from the median ground motion defined by the selected model. These two values of ε are averaged and assigned to a frequency equal to the geometric mean of 1 and 2.5 Hz. The expected value of ε at other frequencies is then computed using the Baker and Jayaram model (Reference 2.5.2-266). The conditional mean spectral shape is then computed using the selected median and aleatory variability models. The spectral shape is smoothed between the seven frequencies defined in the EPRI ground motion model using the average of the single-corner and double corner spectral shape models developed in McGuire et al. (Reference 2.5.2-263).

These spectral shape models are also used to extrapolate the EPRI median ground motion model from a frequency of 0.5 Hz down to a frequency of 0.1 Hz (spectral period of 10 seconds). This extrapolation requires an assessment of the aleatory variability in spectral acceleration at frequencies less than 0.5 Hz. The EPRI models are based on empirical ground motion models developed as part of the Pacific Earthquake Engineering Research (PEER) Center's Next Generation Attenuation Project (NGA) (Reference 2.5.2-267). The NGA ground motion models available from PEER include estimates of aleatory variability for spectral frequencies between 0.1 and 100 Hz. These models indicate that the standard deviation of the natural log of spectral acceleration is, on average, 14 percent higher at a frequency of 0.1 Hz than it is at a frequency of 0.5 Hz. A linear increase in aleatory variability with decreasing log frequency from 0 percent at 0.5 Hz to 14 percent at 0.1 Hz was used to extend the EPRI (Reference 2.5.2-258) aleatory variability models down to a frequency of 0.1 Hz. The calculation is then repeated for each combination of median, aleatory variability, and m_b-**M** conversion defined in the ground motion model logic tree (Figure 2.5.2-217). A weighed average of these spectra is then computed using the weights defined on Figure 2.5.2-217. The resulting spectral shape is then smoothed and rescaled to match on average the UHRS at 1 and 2.5 Hz. The resulting DE response spectra are shown on Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246.

The RE or controlling earthquake spectra are used to define a smooth spectral shape representative of the rock UHRS. Their primary use in Approach 2B is to produce a smooth surface spectrum consistent with the rock UHRS when multiplied by the site amplification function. As such, they represent the composite effects of a range of earthquake magnitude and distances, and it is desirable that their spectra lie close to the UHRS over a broad frequency range. Accordingly, the spectral shapes for the REs were developed using the above process with the modification that the correlation in ε between spectral frequencies was set to 1.0. The resulting RE spectral shapes are also shown on Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246.

As can be seen on Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246, the rock UHRS at 0.5 Hz typically lies above the LF RE spectra. Thus scaling the LF RE spectrum by the LF amplification function will underestimate the appropriate surface motions that are hazard consistent with the rock UHRS. To address this issue, the rock UHRS was extended from 0.5 Hz down to 0.1 Hz by computing a second LF RE spectrum that matches the UHRS at 0.5 Hz. This additional spectrum is denoted by the “LF Extended” spectral shape shown on Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The uniform hazard response spectra shown on Figure 2.5.2-238 represent ground motions occurring on generic CEUS hard rock conditions. As described in FSAR Subsection 2.5.2.5.1.2, the materials underlying the LNP site consist of approximately 1300 m (4300 ft.) of Cretaceous and Cenozoic limestone and dolomite. The upper approximately 18 m (60 ft.) of the limestone is variably weathered, and there is on the average approximately 1.8 m (6 ft.) of Quaternary sands at the surface. The velocities of these materials are lower than generic CEUS hard rock velocity, thus necessitating an assessment of site amplification to develop the site surface motions.

Site response analyses were conducted to evaluate the effect of the sedimentary rocks on the generic CEUS hard rock ground motions. The intent of these analyses is to develop ground motions at the surface that are hazard-consistent with the hazard levels defined for the generic rock conditions. This hazard consistency is achieved through the use of the site response Approach 2B outlined in NUREG/CR-6728 (Reference 2.5.2-263). The following steps are involved in this approach:

1. Characterize the dynamic properties of the subsurface materials.

2. Randomize these properties to represent their uncertainty and variability across the site.

3. Based on the deaggregation of the rock hazard, define the distribution of magnitudes contributing to the controlling earthquakes for HF and LF ground motions (these are termed deaggregation earthquakes in McGuire et al. (Reference 2.5.2-263), and define the response spectra appropriate for each of the deaggregation earthquakes.

4. Obtain appropriate rock site time histories to match the response spectra for the deaggregation earthquakes.

5. Compute the mean site amplification function for the HF and LF controlling earthquakes based on the weighted average of the amplification functions for the deaggregation earthquakes.

6. Scale the response spectra for the controlling earthquakes by the mean amplification function to obtain surface motions.

7. Envelop these scaled spectra to obtain surface motions hazard-consistent with the generic CEUS hard rock hazard levels.

Step 3 of this process is described in FSAR Subsection 2.5.2.4.3.3. Steps 6 and 7 are described in FSAR Subsection 2.5.2.6. Steps 1, 2, 4, and 5 are presented in this subsection.

Two sets of site amplification functions were developed. The first set was used to develop the site GMRS following guidance given in Regulatory Guide 1.208. The second set was used to develop the site PBSRS and associated FIRS for use in potential SSI analyses. This second set of amplification functions was developed following Subsection 5.2.1 of the Interim Staff Guidance DC/COL-ISG-017. The process used to develop the PBSRS and FIRS follows that given above for the GMRS. The primary difference is that the PBSRS is developed for the plant design grade and includes the effects of engineered fill that raises nominal site grade from approximately +12.8 m (+42 ft.) to +15.5 m (+51 ft.) NAVD88.

2.5.2.5.1 Dynamic Properties of the LNP Site

2.5.2.5.1.1 Shallow Shear-Wave Velocities

The V_S and V_P data obtained at the LNP site are described in FSAR Subsections 2.5.4.4.2.1 and 2.5.4.4.2.3. A combination of suspension logging and downhole velocity surveys were used to measure shear-wave velocities to a depth of approximately 152 m (500 ft.). Measurements were conducted in 18 borings, 9 at the site of each LNP unit. GeoVision, Inc. (References 2.5.2-268 and 2.5.2-269), provided initial interpretations of the P-S (seismic waves P and S) suspension and downhole data in the form of layered velocity models for each boring. These interpretations were reviewed and the layer boundaries refined based on boring log lithology and geophysical logging data obtained by Technos, Inc. (Reference 2.5.2-270) The resulting interpreted V_S models for each boring are presented on Figures 2.5.2-247 and 2.5.2-248 for the LNP 1 and LNP 2 units, respectively. The velocities assigned to individual layers in each boring were computed by dividing the layer thickness by the total wave travel time in the layer

The shear-wave velocity data show a generally consistent pattern at the two units, with the data at LNP 1 being a little more erratic than at LNP 2. The interpreted downhole and suspension layered velocity models show good agreement at the LNP 2 site. Larger differences were found between the two measurement techniques at LNP 1, owing perhaps to the more difficult drilling conditions at this location. It should be noted that at depth in better rock conditions the two techniques produced very consistent values (Figures 2.5.2-247 and 2.5.2-248).

The shear-wave velocities in the primary geologic units are as follows:

- The first site layer, S1, consists of approximately 1.8 m (6 ft.) of Quaternary sands. This layer is relatively loose, and as discussed below, is not considered part of the site GMRS profile. Very limited velocity data were obtained in this layer.

- The next two layers are designated S2 and S3 and consist of weathered limestone (calcareous silt). The shear-wave velocity data consistently show higher velocity in the deeper layer S3, consistent with a decrease in the degree of weathering with depth.

- The first layer of competent limestone of the Avon Park Formation is encountered at an average elevation of approximately -7 m (-24 ft.) NAVD88. Velocities in this material range from 670 m to 1280 m/sec (2200 to 4200 ft/sec). There is a tendency for the upper portion of this layer to have somewhat higher velocity than the lower portion at LNP 2. At an elevation of approximately -30 m (-100 ft.) NAVD88, the velocity increases to the range of 914 to 1524 m/sec (3000 to 5000 ft/sec).

- Below elevation approximately -46 m (-150 ft.) NAVD88, the density and shear-wave velocity of the rock decreases. Between elevation -48 m (-160 ft.) and -70 m (-230 ft.) NAVD88, a low-velocity zone is encountered consistently across both LNP 1 and LNP 2, with shear-wave velocities in the range of 640 to 1067 m/sec (2100 to 3500 ft/sec). Although the density remains low, the velocity in this layer increases below elevation -70 m (-230 ft.) NAVD88 to a value consistently near 1067 m/sec (3500 ft/sec).

- Below elevation -95 m (-310 ft.) NAVD88, the rock density increases and the shear-wave velocity begins to increase. A consistent small-velocity reversal is observed in the four deep borings at elevation approximately -110 m (-350 ft.) NAVD88.

The general shallow stratigraphy outlined in FSAR Subsection 2.5.4.1 was subdivided into layers showing consistent patterns in the velocity, lithology, and geophysical data. Geometric mean velocities were computed for each sublayer. Separate values were computed for the data at each unit. Separate values were also computed for the downhole and P-S suspension data. The resulting velocity profiles are shown on Figures 2.5.2-247 and 2.5.2-248. These are labeled as median profiles using the assumption that V_S is lognormally distributed.

GeoVision, Inc. (References 2.5.2-268 and 2.5.2-269), provided assessments of data quality. As a sensitivity analysis, these qualitative assessments were used to assign weights to the data and weighted geometric mean velocities were computed. The resulting weighted median velocities are within about 5 percent or less of the unweighted medians (Figures 2.5.2-247 and 2.5.2-248) and were not used further.

Figure 2.5.2-249 shows the resulting four median profiles, two for the site of each unit. The profiles labeled “P1” are based on the suspension logging data and those labeled “P2” are based on the downhole data. All four profiles are generally similar. The larger differences typically occur in regions of poorer data quality. The largest difference is between the suspension and downhole date for LNP 1 (profiles LNP 1 P1 and LNP 1 P2) in the elevation range of -30 m (-100 ft.) to -67 m (-220 ft.) NAVD88. The effect of the differences in the velocity profiles shown on Figure 2.5.2-249 was examined in initial site response sensitivity analyses.

Regulatory Guide 1.208 indicates that the site-specific GMRS should be defined at the ground surface or at the top of the first competent layer. Given that the upper Quaternary sands have low velocity and are to be removed during construction (FSAR Subsection 2.5.4.5), the reference point for the GMRS is taken to be the top of the calcareous silt (unit S2, weathered limestone) at an average elevation of 11 m (36 ft.) NAVD88. The planned construction approach for the units (FSAR Subsection 2.5.4.5) calls for excavation of the undifferentiated Quaternary/Tertiary sediments (units S1, S2, and S3) to an average elevation of –7 m (–24 ft.) NAVD88, which is the top of the Avon Park Formation. Evaluation of the seismic response of the proposed safety-related structures would require FIRS at this location. FIRS are also developed at the reactor foundation elevation of +3.3 m (+11 ft.) NAVD88 for the purpose of checking the requirement of the minimum level of ground motion specified in Subsection 3.7.1 of the Standard Review Plan.

As described in FSAR Subsection 2.5.4.5, engineered fill will be placed to raise the site elevation to +15.5 m (+51 ft.) NAVD88. The anticipated average shear-wave velocity of the engineered fill is in the range of 152 to 305 m/sec (500 to 1000 ft/sec) with a best estimate value of 259 m/sec (850 ft/sec) as shown in Table 2.5.4.5-201.

2.5.2.5.1.2 Deep Shear-Wave Velocities

The Robinson No. 1 well, which is approximately 500 m (1640 ft.) north of the LNP site, provides nearby information on the thickness of post-Paleozoic strata and depth to Paleozoic basement rock in the site location. Approximately 1311 m (4300 ft.) of interbedded limestone and dolostone overlie Paleozoic basement rocks, which were encountered at an elevation of -1317 m (-4319 ft.) mean sea level (msl) in the Robinson No. 1 well (Reference 2.5.2-271). Only stratigraphic information is available for this well. Therefore, velocity information that was available for other wells in the site vicinity was used to estimate shear-wave velocity for the entire section down to and including the Paleozoic units underlying the site location.

Three deep wells with velocity logs are located in the site vicinity: well W7543(P350) at a distance of 25 km (15 mi.); well W7534(P358) at a distance of 18 km (11 mi.); and well W7538(P353) at a distance of 11 km (6.8 mi.) (Figure 2.5.1-244). Figure 2.5.2-250 shows measured V_p for each of these wells. The velocities assigned to the individual layers in each boring were computed by dividing the layer thickness by the compression wave travel time for the layer.

The compression-wave velocity data show a consistent pattern with depth, indicating similar stratigraphy. The geometric mean compression-wave velocities were computed for each layer to produce the median V_P profile shown on Figure 2.5.2-250. Well W7543(P350) contains velocity data to within approximately 30.5 m (100 ft.) of the surface. The average V_P for the shallowest layer in this well is approximately 2744 m/sec (9000 ft/sec). The average V_P for the deepest layer of the four LNP site borings (i.e., AD1, AD2, AD3, and AD4) ranges from 2560 to 3078 m/sec (8400 to 10,100 ft/sec). As these two sets of measurements are consistent, the deep well compression-wave data from the three industry wells were used to extend the profiles shown on Figure 2.5.2-249 to a depth of approximately 1300 m (4300 ft.).

The average value of V_P/V_S for the deepest layer in the LNP P-S suspension surveys in the LNP site borings is 2.42. This ratio was used to compute a median V_S profile by multiplying the average V_P/V_S, by the median compression-wave velocities. The resulting shear-wave velocity profile is shown on Figure 2.5.2-250. This median profile was added to the base of the four shallow profiles shown on Figure 2.5.2-249 to create the initial velocity profiles for site response analysis.

The Paleozoic basement rocks encountered in the Robinson No. 1 well consist of Ordovician quartzitic sandstones of the Suwannee terrane (FSAR Subsections 2.5.1.1.2.1 and 2.5.1.1.3.1). The log from well W7543(P350) indicates that it penetrated to a limited extent into the Suwannee terrane and the velocity data show a sharp increase in this material. The velocity data from the other two wells, W7534(P358) and W7538(P353), also show a sharp increase in velocity at similar elevations. Given the age of this rock unit and the indication of a large increase in V_P over the limited penetration by the deep wells, the Suwannee terrane is taken to represent the start of generic hard rock and the base of the site response profile.

2.5.2.5.1.3 Rock Density

The data for the density of the weathered and unweathered limestone at the site are presented in FSAR Subsection 2.5.4.2.3.2. These data indicate bulk unit weights increasing from 120 pounds per cubic foot (pcf) near the surface to 140 pcf below elevation –91.5 m (–300 ft.) msl. The velocity of the rocks below elevation -305 m (-1000 ft.) msl is significantly higher than that at elevation -91.5 m (-300 ft.) msl (Figure 2.5.1-250). Increases in seismic velocity are typically correlated with increases in rock density. Therefore, a small increase in bulk unit weight to 150 pcf was applied to the rock layers below elevation -305 m (-1000 ft.) msl. The expected unit weight of the engineered fill is 110 pcf as shown in Table 2.5.4.5-201.

2.5.2.5.1.4 Shear Modulus and Damping

The materials that are included in the site response analysis to develop the GMRS consist of approximately 18.3 m (60 ft.) of partly to moderately weathered limestone (units S2 and S3 calcareous silts) above unweathered sedimentary rocks. To account for the potential of nonlinear behavior in the calcareous silts (weathered rock), two alternative sets of modulus reduction and damping relationships are used.

- One set consists of soft rock modulus reduction and damping relationships developed by Silva et al. (Reference 2.5.2-272), as modified by Silva (Reference 2.5.2-273) to account for depth effects.

- An alternative set consists of the “Peninsula Range” set, also developed by Silva et al. (Reference 2.5.2-272). The Peninsula Range set was used for shallow soft rock at various sites in site response analyses conducted by EPRI (Reference 2.5.2-274).

These two sets of relationships, designated as SR and PR for soft rock and Peninsula Range, respectively, are shown on Figure 2.5.2-251. The relationships provide alternative behaviors. The Peninsula Range set represents very linear behavior that might be expected for a cemented material. The soft rock set represents the more nonlinear behavior of weathered shallow rock exhibited in California. Site response calculations are performed using the two sets to examine the sensitivity of the results to the selection of the modulus reduction and damping relationships. These models are also applied to the low-velocity sublayer in the elevation range of -48.8 to -67.1 m (-160 to -220 ft.) NAVD88 because the velocity of this material is similar to that of the shallow weathered limestone layer S3 and the material also exhibits a friable nature.

The remaining rock layers are assumed to remain linear during seismic shaking. The damping within these materials was established using the following procedure.

- The site response analyses were conducted using an updated version of program SHAKE originally developed by Schnabel et al. (Reference 2.5.2-275). The energy lost in shear-wave propagation was measured by the parameter Q_S, which can be equated to two other representations of energy loss in wave-propagation analysis. For the linear viscoelastic wave-propagation modeling used in program SHAKE, the material damping, ξ, is obtained by the relationship:

  $$\xi = \frac{1}{2Q_S} \qquad \text{Equation 2.5.2-207}$$

  Parameter Q_S is also related to the high-frequency attenuation parameter κ developed by Anderson and Hough (Reference 2.5.2-276) by the relationship:

  $$\kappa = \frac{H}{Q_S V_S} \qquad \text{Equation 2.5.2-208}$$

  where H is the thickness of the crust over which the energy loss occurs, typically taken to be 1 to 2 km (Reference 2.5.2-277). Silva and Darragh (Reference 2.5.2-277) find that Q_S is proportional to shear-wave velocity:

  $$Q_S = \gamma V_S \qquad \text{Equation 2.5.2-209}$$

- Using this assumption, the amount of high-frequency attenuation in the i^{th} layer of a velocity profile, κ_i, is given by the relationship:

  $$\kappa_i = \frac{H_i}{\gamma V_{Si}^2} \qquad \text{Equation 2.5.2-210}$$

  where H_i is the layer thickness and V_{Si}, is the layer shear-wave velocity. Given the total value of κ appropriate for the site, one can solve for the corresponding value of γ. Using the resulting value of γ and Equations 2.5.2-209 and 2.5.2-210, the appropriate damping values for each layer are then obtained.

EPRI (Reference 2.5.2-274) gives the following relationship between κ and site shear-wave velocity:

$$\log(\kappa) = 2.2189 - 1.0930\log(V_S) \qquad \text{Equation 2.5.2-211}$$

where V_S is shear-wave velocity in ft/sec and κ is in seconds. The shear-wave velocity in the unweathered rock is ~3500 ft/sec. Using this value in Equation 2.5.2-211 yields a κ value of 0.0221 second. Uncertainty in κ for CEUS site is typically modeled by a range of 1/1.5 to 1.5 times the best estimate to represent the 5th to 95th range (Reference 2.5.2-274). The three-point distribution developed by Keefer and Bodily (Reference 2.5.2-278) is used to represent the uncertainty distribution, leading to a three-point distribution of 0.0148 (weight 0.185), 0.0221 (weight 0.63), and 0.0332 (weight 0.185). The attenuation models for CEUS hard rock are developed assuming a shallow crustal κ of 0.006 second (Reference 2.5.2-253). The difference between the generic CEUS hard rock κ and the sedimentary rock κ is attributed to material damping in the sedimentary rocks.

The value of κ assigned to a site profile is a measure of the total damping due to both material damping and scattering effects. To account for this in a one-dimensional (1-D) site response model, the conversion from κ to material damping should account for the scattering (reflection) of waves off layer boundaries, particularly velocity reversals. In addition to those present in the initial velocity model, the process of profile randomization to account for site variability, discussed in FSAR Subsection 2.5.2.5.1.6, will introduce additional velocity reversals. The amount of κ that is attributed to scattering in the site velocity profiles was assessed by comparing the median response of the randomized velocity profiles to a simple model with uniform velocity layers. The process used is shown on Figure 2.5.2-252.

Site response analyses were performed for the initial profiles with the largest velocity contrasts, profiles LNP 1 P1 and LNP 2 P1 (labeled L1P1 and L2P1 in Figure 2.5.2-252). Sixty randomized profiles were generated with the low κ value of 0.0088 seconds used to obtain material damping in the linear rock layers. The calculations were computed using 30 time histories with very low amplitude motions so that the materials remained linear. The curves labeled “Randomized, kappa = 0.0088” show the median surface response spectra. A second set of analyses were performed using simple four-layer models with the velocity in each large layer equal to the average velocity over the same depth range in the LNP 1 P1 and LNP 2 P1 profiles and a κ of 0.0088 second used to obtain the material damping. The median response spectra computed for these models are labeled “Uniform, kappa = 0.0088” on the figure. The response of the uniform model is higher than that of the randomized model and the difference is attributed to additional damping in the randomized model due to scattering off of the layer boundaries.

The final step is to gradually reduce the value of κ used to obtain the material damping for the randomized model until the median response spectra for the randomized model is at the same level as that for the uniform model at high frequencies. As shown on Figure 2.5.2-252, the appropriate levels of κ are 0.0059 seconds for profile LNP 1 P1 and 0.0063 seconds for profile LNP 2 P1. The largest difference between these values and the initial 0.0088 seconds is 0.0029 seconds. This value is then subtracted from the values of κ assigned to the sedimentary rocks. The resulting distribution for κ used to obtain material damping in the linear rock layers is of 0.0059 (weight 0.185), 0.0132 (weight 0.63), and 0.0234 (weight 0.185).

Shear modulus reduction and damping relationships for the engineered fill material to be placed at the site were estimated based on the work of Darendeli (Reference 2.5.2-281) and Menq (Reference 2.5.2-282) and the anticipated range of engineered fill materials that may be used (Table 2.5.4.5-201). Figure 2.5.2-299 shows the range of shear modulus relationships developed from the published literature using the range of engineered fill properties. The relationships developed from Menq (Reference 2.5.2-282) for granular fill are the most conservative in that they contain less modulus reduction and lower damping. These relationships were selected to model the nonlinear properties of the engineered fill.

2.5.2.5.1.5 Selection of GMRS and FIRS Analysis Profiles

Site response calculations were performed for the four initial profiles shown on Figure 2.5.2-249 extended to depth using the median profile shown on Figure 2.5.2-250. Analyses were preformed using both sets of modulus reduction and damping relationships and the best estimate value for κ. For each analysis 60 randomized profiles were generated, and the mean site amplification (response spectrum for surface motion divided by response spectrum for input motion) was computed. Figure 2.5.2-253 shows the resulting mean amplification functions. The curves with “PM” at the end of the label were obtained using the Peninsula Range set of modulus reduction and damping and those with “SM” at the end of the label were obtained using the soft rock set.

These results indicate that at each unit the amplification for one profile essentially envelops the amplification from the other. The highest amplification for LNP 1 is obtained for profile LNP 1 P2 and the highest amplification for LNP 2 is obtained for profile LNP 2 P1. These two profiles were selected for calculation of the site amplification for the GMRS motions. The profile for LNP 1 is designated GMRS profile LNP 1 and the profile for LNP 2 is designated GMRS profile LNP 2. These velocity profiles are plotted on Figure 2.5.2-254 and are listed in Tables 2.5.2-222 and 2.5.2-223.

The site response analyses profiles for the PBSRS were developed by adding a layer of engineered fill to the GMRS profiles to bring the surface elevation to +15.5 m (+51 ft.) NAVD88. Analyses were performed using engineered fill velocities of 152 m/sec (500 ft/sec), 259 m/sec (850 ft/sec), and 305 m/sec (1000 ft/sec) (Table 2.5.4.5-201).

2.5.2.5.1.6 Randomization of Dynamic Properties

Site response analyses were conducted using randomized shear-wave velocity profiles to account for variations in shear-wave velocity. The randomized profiles were generated using the shear-wave velocity correlation model developed in Silva et al. (Reference 2.5.2-272). In this model, the shear-wave velocity in the sediment layers are modeled as correlated, lognormally distributed variables. The expression for the correlation coefficient between the velocities in two adjacent layers, *p* is given by:

$$\rho(h,t) = (1-\rho_d(h))\rho_t(t) + \rho_d(h) \qquad \text{Equation 2.5.2-212}$$

where p_d represents the depth-dependent correlation (generally increasing with increasing depth), and p_t is the thickness-dependent correlation (generally decreasing with increasing layer thickness). The factors p_d and p_t are obtained from the expressions:

$$\rho_d(h) = \begin{cases} \rho_{200}\left[\dfrac{h+h_0}{200+h_0}\right]^b & for \quad h \le 200\ m \\ \rho_{200} & for \quad h > 200\ m \end{cases} \qquad \text{Equation 2.5.2-213}$$

and

$$\rho_t(t) = \rho_0 \exp\left[-\left(\frac{t}{\Delta}\right)^{\alpha}\right] \qquad \text{Equation 2.5.2-214}$$

where *h* is the average of the midpoint depths of layers *i* and *i-1*, and *t* is the difference between those midpoint depths. The correlation model parameters developed in Silva et al. for stiff soil sites were used in the simulations (Reference 2.5.2-272). Stiff soil site parameters were chosen because the site is underlain by a relatively flat-lying sedimentary rock sequence that is assumed to have characteristics similar to a layered soil site.

The data from the LNP site display moderate variability in velocity at shallow depth with a $\sigma_{\ln(Vs)}$ of approximately 0.2. This value decreases to 0.15 for layer S3 and the upper layers of the Avon Park Formation, then to 0.1 or less below elevation -67.1 m (-220 ft.) NAVD88. These values are similar to those obtained from analyses of individual firm sites (Reference 2.5.2-272), and these values were used to develop randomized velocity profiles. The locations of velocity layer boundaries were randomized to vary uniformly within the range of layer thickness observed in the site borings.

Sixty randomized velocity profiles were generated for the two GMRS profiles. Figures 2.5.2-255, 2.5.2-256, 2.5.2-257, and 2.5.2-258 show the randomized velocity profiles. The statistics of the randomized profiles are compared to the input target values for median velocity and standard deviation of $\ln(V_S)$ on Figures 2.5.2-259 and 2.5.2-260.

The modulus reduction and damping relationships were also randomized, as shown on Figures 2.5.2-261, 2.5.2-262, 2.5.2-263, 2.5.2-264, 2.5.2-265, 2.5.2-266, and 2.5.2-300. The standard deviation in the modulus reduction and damping were set so that the randomized relationships fell within recommended bounds provided by Silva (Reference 2.5.2-273). The damping ratio curves were limited to a maximum of 15 percent damping as recommended in Appendix E of Regulatory Guide 1.208.

The damping in the sedimentary rocks beneath the soil profile was also randomized in the analysis. The standard deviation of $\ln(\kappa)$ was set equal to 0.3, consistent with the variability in κ used in McGuire et al. and EPRI (References 2.5.2-263 and 2.5.2-274). The appropriate damping ratio in the sedimentary rock layers was then computed using the randomized sedimentary rock layer velocities and thicknesses and the randomly selected value of κ. Statistics of the resulting values of material damping assigned to the linear rock layers are given in Table 2.5.2-224.

Similar sets of randomized profiles were developed for the six PBSRS analysis cases (two site profiles times three engineered fill velocities).

2.5.2.5.2 Acceleration Time Histories for Input Rock Motions

Response spectra were developed for each deaggregation earthquake as described in FSAR Subsection 2.5.2.4.3.3. Thirty time histories were developed for each deaggregation earthquake from the time history sets given in McGuire et al. (Reference 2.5.2-263). Table 2.5.2-225 lists the time history sets used. The selected time histories were scaled to approximately match the target DE spectrum using a limited number of iterations of the program RASCALS (Reference 2.5.2-279). Figure 2.5.2-267 shows the response spectra for the 30 time histories scaled to match the HF and LF DEL and DEH spectra for mean 10^{-4} ground motions.

The purpose of randomization of the site properties is to account for natural variability in defining the site response. Part of the natural variability is variability in the ground motions of an individual earthquake. That is why only weak scaling of the time histories was performed. The weak scaling produces recordings that have, in general, the desired relative frequency content of the DE spectra while maintaining a degree of natural variability. The use of three DE earthquakes for both HF and LF motions along with a large number of recordings provides adequate coverage of the frequency band of interest. The acceleration time histories represent free field outcropping motions for generic CEUS hard rock.

2.5.2.5.3 Development of Surface Hazard-Consistent Spectra

2.5.2.5.3.1 Site Amplification Functions for GMRS Profiles

Site amplification functions were developed for each deaggregation earthquake. The 60 randomized velocity profiles were paired with the 60 sets of randomized modulus reduction and damping curves (one profile with one set of modulus reduction and damping curves). Each of the 30 scaled time histories was used to compute the response of two profile-soil property curves sets. For each analysis, the response spectrum for the computed surface motion was divided by the response spectrum for the input motion to obtain a site amplification function. The arithmetic mean of the 60 individual response spectral ratios is then computed to define the amplification function.

Figure 2.5.2-268 shows an example of the statistics of the 60 individual site amplification functions for one analysis case. Shown are the median (mean log), 16^{th} percentile (mean log – sigma log), 84^{th} percentile (mean log + sigma log), and arithmetic mean amplification. The mean amplification function is used in Approach 2B.

For each DE, mean amplification functions were computed for the three values of κ and for the two sets of modulus reduction and damping relationships. The results from the three DEs were then combined to produce a weighted mean amplification function for the RE. Figure 2.5.2-269 shows the site response model logic tree used to compute the RE mean amplification function. The weights assigned to the DEs are given in Table 2.5.2-225.

The sensitivity of the mean amplification function at the 10^{-4} loading level to the value of site κ is shown on Figure 2.5.2-270. The range in κ leads to approximately a 30 percent difference in mean amplification at 100 Hz, 40 to 50 percent differences near 40 Hz, decreasing to about 20 percent at 10 Hz. The effect of κ on site motions decreases for frequencies below 10 Hz.

Figure 2.5.2-271 shows the effect of the alternative dynamic property curves used for weathered limestone on the mean amplification for the LNP 1 GMRS profile. The difference in the amplification computed using the two sets of modulus reduction and damping is generally in the range of 10 to 15 percent.

Figures 2.5.2-272 and 2.5.2-273 show the DEL, DEM, and DEH amplification functions for 10^{-4} ground motions for the two GMRS profiles and the weighted mean amplification. The site amplification functions are insensitive to the differences in the deaggregation earthquake motions for frequencies less that about 15 Hz. At higher frequencies the site amplification is based on results obtained using the HF deaggregation earthquakes (the top plots in the figures) and the differences from the mean amplification function are generally less than 15 percent.

Figures 2.5.2-274, 2.5.2-275, 2.5.2-276, and 2.5.2-277 compare the mean site amplification functions for the two GMRS profiles for the four levels of input motion. Because both profiles have the same deep velocity structure, they produce the same level of amplification at low frequencies (less than ~ 3 Hz). The difference in amplification at frequencies greater than about 3 Hz reflects differences in the shallow velocity structure (above elevation ~ -67 m [-220 ft.] NAVD88). At shallow depths the two profiles are again very similar and produce similar amplification. Because of the similarity in site amplification, a single envelope amplification function is used for the LNP site.

The envelope amplifications are plotted on Figure 2.5.2-278. The potential for non-linear behavior is limited to the shallowest portions of the site profile, above elevation -7 m (-24 ft.) NAVD88, and a relatively thin layer between elevation -48 m (-160 ft.) and -70 m (-230 ft.) NAVD88. This portion of the profile primarily influences the response at higher frequencies. Thus, the effect of the level of motion on the level of response occurs primarily at frequencies above about 3 Hz. Figure 2.5.2-279 shows the statistics of effective strain computed in the calculations for the 10^{-5} HF ground motion inputs. The strains are generally below 10^{-2} percent, indicating the motions are inducing only limited nonlinearity in the all of the site materials.

The envelope amplification functions for the four ground motion levels were smoothed by eye. The large dip in the amplification function between 0.3 and 1 Hz was conservatively smoothed through. The resulting smoothed mean amplification functions are shown on Figure 2.5.2-280. The smoothed LF amplification functions are not extended above 10 Hz because they are not used to determine the ground motions at high frequencies. Similarly, the smoothed HF amplification functions are not extended below 1 Hz.

2.5.2.5.3.2 Site Amplification Functions for PBSRS Profiles

The process described above for developing the GMRS profile amplification functions was repeated for the design grade PBSRS profiles.

Figures 2.5.2-281 and 2.5.2-282 compare the mean site amplification functions for the two PBSRS profiles for an engineered fill velocity of 259 m/sec (850 ft/sec) and 10^{-4} and 10^{-5} input ground motions, respectively. The difference in response is similar to that found for the GMRS profiles. The amplification functions for engineered fill velocities of 152 m/sec (500 ft/sec) and 305 m/sec (1000 ft/sec) also show similar results for the two LNP profiles. Consistent with the GMRS analysis, a single envelope amplification function is developed for each ground motion level. The results are plotted on Figure 2.5.2-283A, Figure 2.5.2-283B, and Figure 2.5.2-283C for engineered fill velocities of 152 m/sec (500 ft/sec), 259 m/sec (850 ft/sec), and 305 m/sec (1000 ft/sec), respectively.

The PBSRS amplifications are more sensitive to ground motion levels because of the nonlinear behavior of the engineered fill materials, especially the lower velocity case. The envelope amplification functions are again smoothed by eye and the resulting smoothed envelope amplification functions are plotted on Figure 2.5.2-284A, Figure 2.5.2-284B, and Figure 2.5.2-284C. Amplification functions are created for each engineered fill velocity because the difference in response over the range of engineered fill velocities produces different behavior when assessing the probabilistic surface response spectra.

2.5.2.6 Ground Motion Response Spectra

LNP COL 2.5-3

2.5.2.6.1 Surface Spectra

Surface hazard spectra for the LNP GMRS profiles are obtained by scaling the rock RE and UHRS by the site amplification functions. The process used is illustrated on Figure 2.5.2-285 for the 10^{-4} level ground motions.

- The reference (controlling) spectra for LF and HF motions developed for each annual exceedance level were scaled by the appropriate smoothed amplification function to produce ground surface spectra.
- The generic hard rock UHRS was also scaled using the appropriate LF and HF amplification values.
- A smooth envelope of the scaled spectra is constructed to define the surface 10^{-4} UHRS.

The rock UHRS exhibit a sharp peak at 25 Hz. This peak is an artifact of the fact that the PSHA is computed for frequencies of 10, 25, and 100 Hz and that the RE spectra are defined for frequencies in the range of 5 to 10 Hz. The spectral shapes for CEUS earthquakes developed in McGuire et al. (Reference 2.5.2-256) show a broader peak in the spectrum in the frequency range of 10 to 100 Hz. Therefore, the approach described in FSAR Subsection 2.5.2.4.4.3 was used to smoothly interpolate the rock UHRS between 10 and 100 Hz. An additional HF RE spectral shape was constructed to match the rock UHRS at 25 Hz. This shape was then adjusted to match the UHRS at 10 and 100 Hz by applying adjustment factors that varied linearly with log frequency from 0 at 25 Hz to the appropriate value at 10 or 100 Hz. This smoothed rock UHRS was then multiplied by the HF amplification function.

Similar operations were performed to develop GMRS surface spectra for the 10^{-5} and 10^{-6} exceedance level motions. This smooth envelope spectrum represents the surface UHRS for the site defined as free field surface motions for a soil column truncated at elevation 11 m (36 ft.) NAVD88, the top of the first competent layer.

The above process was followed to develop design grade surface response spectra at elevation 15.5 m (51 ft.) NAVD88. For this case a weighted combination of the amplification functions for the three engineered fill velocities was used to construct a single surface spectrum for each level of input ground motion. The range of engineered fill velocity provided in Table 2.5.4.5-201 was treated as representing the 90 percent confidence interval for the epistemic uncertainty on the average engineered fill velocity. The three-point distribution developed by Keefer and Bodily (Reference 2.5.2-278) is used to represent the uncertainty distribution, leading to a three-point distribution of 500 ft/sec (weight 0.185), 850 ft/sec (weight 0.63), and 1000 ft/sec (weight 0.185).

2.5.2.6.2 Incorporation of CAV

The PSHA results used above for developing the RE and DE spectra were computed using a fixed lower-bound magnitude of m_b 5. Regulatory Guide 1.208 indicates that an alternative method that is based on the probability that earthquakes of a given magnitude can produce damaging ground motions, defined as ground motions with a CAV greater than 0.16 g-second, may be used. EPRI developed an approach for conducting a PSHA incorporating the probability that ground motions produced by an earthquake with magnitude value m will have a value of CAV greater than 0.16 g-second (Reference 2.5.2-280).

The EPRI CAV model was implemented in a second set of PSHA calculations for the LNP site. These calculations include the contributions from all earthquakes above m_b 4.0 weighted by the probability that they can produce a CAV greater than 0.16 g-second. The EPRI CAV model uses moment magnitude (**M**) as the magnitude scale. The model results indicate that earthquakes of magnitude less than **M** 4 have very little probability of producing a CAV greater than 0.16 g-second (Reference 2.5.2-280). The magnitude conversions used in the PSHA convert an m_b magnitude of 4.0 into **M** magnitudes that are less than 4.0.

The EPRI CAV model is based on ground motions recorded at the surface (Reference 2.5.2-280). Therefore, computation of PSHA using this model requires incorporation of site amplification into the PSHA calculation. The site amplification incorporated in the CAV PSHA is based on Approach 2B – the use of a mean amplification function that may be amplitude dependent. The dependence of the site amplification functions on the amplitude of the input rock motion, exhibited in the results presented in FSAR Subsection 2.5.2.5.3.1, was incorporated into the computation of the surface hazard spectra incorporating CAV.

Two sets of PSHA calculations with site amplification were performed. The first set incorporated the CAV filter and site amplification, producing surface hazard curves. The second set was performed using site amplification and a fixed lower-bound magnitude of m_b 5.0, producing surface hazard curves that are comparable to amplification of the rock hazard results by the site transfer functions. The purpose of performing these two sets of calculations is to provide ratios of CAV/non-CAV spectral values at the seven spectral frequencies used in the PSHA calculations. These spectral ratios are then used to adjust the smooth surface spectra discussed in FSAR Subsection 2.5.2.6.1 to produce the final hazard-consistent surface spectra.

Figures 2.5.2-286, 2.5.2-287, 2.5.2-288, 2.5.2-289, 2.5.2-290, 2.5.2-291, and 2.5.2-292 compare the surface mean hazard curves computed with and without CAV for the seven spectra frequencies of 0.5, 1, 2.5, 5, 10, 25, and 100 Hz, respectively. Also shown on these figures is the corresponding generic CEUS mean rock hazard curve from FSAR Subsection 2.5.2.4.4.

The surface mean hazard results shown on Figures 2.5.2-286, 2.5.2-287, 2.5.2-288, 2.5.2-289, 2.5.2-290, 2.5.2-291, and 2.5.2-292 are interpolated to obtain the spectral accelerations corresponding to mean annual frequencies of exceedance of 10^{-4}, 10^{-5}, and 10^{-6}. The hazard curves computed using CAV all level off at an exceedance frequency of approximately 7×10^{-5}, the frequency of earthquakes that produce sufficient peak ground acceleration to induce a CAV of 0.16 g-second or greater. As a result, the 10^{-4} surface UHRS with CAV is zero.

The ratio of the surface spectra accelerations computed with CAV to those computed without CAV for the seven spectral frequencies are then used to scale the smooth surface spectra described in FSAR Subsection 2.5.2.6.1 to produce hazard-consistent mean surface UHRS that are based on the use of the CAV filter. The CAV/no-CAV spectral ratios at intermediate periods are obtained by log-log interpolation. Figure 2.5.2-293 shows the resulting mean 10^{-5} and 10^{-6} surface UHRS for the LNP site.

CAV hazard calculations were also performed for the design grade PBSRS profiles. Because of the differences in response among the three engineered fill velocity cases, surface hazard curves were computed individually for the three engineered fill velocities. Figures 2.5.2-301, 2.5.2-302, 2.5.2-303, 2.5.2-304, 2.5.2-305, 2.5.2-306, and 2.5.2-307 show the PBSRS surface hazard curves computed with and without CAV. The difference in behavior among the three engineered fill velocities can be seen. The results for the 139 m/sec (500 ft/sec) engineered fill show greater amplification than the results for the other engineered fill velocities at low ground motion levels due to the larger velocity impedance contrast between the engineered fill and the underlying native materials. At high ground motions, the lower engineered fill velocity produces lower amplification due to greater strain levels induced in this material.

Composite mean hazard curves were computed as a weighted combination of the hazard curves for each engineered fill velocity using the weights assigned above of 0.185 for 500 ft/sec, 0.63 for 850 ft/sec, and 0.185 for 1000 ft/sec. The resulting composite mean hazard curves are shown on Figures 2.5.2-301, 2.5.2-302, 2.5.2-303, 2.5.2-304, 2.5.2-305, 2.5.2-306, and 2.5.2-307. These mean hazard curves were used to obtain CAV/no-CAV spectral ratios at the seven spectral frequencies. These CAV/no-CAV ratios are then used to scale the smooth design grade surface spectra described in FSAR Subsection 2.5.2.6.1 to produce hazard-consistent mean design grade surface UHRS that are based on the use of the CAV filter. The CAV/no-CAV spectral ratios at intermediate periods are obtained by log-log interpolation. Figure 2.5.2-308 shows the resulting mean 10^{-4} and 10^{-5} surface UHRS for the LNP site. Unlike the GMRS profile analysis, the greater amplification of the PBSRS profile results in CAV hazard results for an annual exceedance frequency of 10^{-4}.

2.5.2.6.3 Horizontal GMRS

Regulatory Guide 1.208 defines the GMRS as a risk-consistent design response spectrum computed from the site-specific UHRS at a mean annual frequency of exceedance of 10^{-4} by the relationship:

$$GMRS = DF \times UHRS\ (10^{-4}) \qquad \text{Equation 2.5.2-215}$$

Parameter *DF* is the design factor specified by the expression:

$$DF = \text{Maximum}\ (1.0,\ 0.6(A_R)^{0.8}) \qquad \text{Equation 2.5.2-216}$$

In which A_R is the ratio of the UHRS ground motions for annual exceedance frequencies of 10^{-4} and 10^{-5}, specifically:

$$A_R = \frac{UHRS(10^{-5})}{UHRS(10^{-4})} \qquad \text{Equation 2.5.2-217}$$

Regulatory Guide 1.208 also specifies that when the value of A_R exceeds 4.2, value of the GMRS is to be no less than $0.45 \times SA(0.1H_D)$ that is, 45 percent of the 10^{-5} UHRS. As the 10^{-4} UHRS with CAV is 0, this second criteria is used to define the horizontal GMRS. Figure 2.5.2-294 shows the horizontal GMRS calculated as $0.45 \times SA(0.1H_D)$. These values are listed in Table 2.5.2-226 along with the horizontal mean 10^{-5} UHRS.

2.5.2.6.4 Vertical GMRS

The vertical GMRS were developed from the horizontal GMRS using vertical to horizontal (V/H) spectral ratios recommended by McGuire et al. (Reference 2.5.2-263). These are given as a function of frequency for three levels of horizontal peak acceleration. Given the low amplitude of the horizontal GMRS of the LNP site, the V/H ratios for peak acceleration less than 0.2g are used. These ratios are plotted on Figure 2.5.2-295 for the WUS and CEUS.

McGuire et al. (Reference 2.5.2-263) indicate that V/H for intermediate sites can be obtained as a weighted combination of the V/H for WUS rock and CEUS rock with the weights determined as a function of the site κ relative to the CEUS κ of 0.006 seconds and the WUS κ of 0.04 seconds. However, computing a weighted combination would flatten the WUS and CEUS peaks in V/H at spectral frequencies of 17 and 63 Hz, respectively, without producing a peak at an intermediate frequency. It is likely that sites with intermediate values of κ would have peaks in the V/H ratios of comparable amplitude but at an intermediate frequency.

Accordingly, an intermediate V/H ratio was developed for the LNP site by first shifting the WUS and CEUS V/H amplitudes to an intermediate frequency and then averaging their amplitudes. The best estimate value of κ for the LNP site is intermediate between the WUS and CEUS values. The WUS and CEUS V/H shapes were thus shifted to a frequency midway in log space between the two. The resulting V/H ratio is shown on Figure 2.5.2-295. In computing the intermediate V/H, a minimum value of 0.5 was used for the WUS V/H ratios to make them consistent in shape to the CEUS V/H ratios. A vertical GMRS was then computed by multiplying the horizontal GMRS by this V/H ratio. The resulting vertical GMRS is listed in Table 2.5.2-226 along with the values of V/H.

2.5.2.6.5 Comparison of GMRS with CSDRS

The horizontal and vertical GMRS are compared to the Westinghouse Certified Seismic Design Response Spectra (CSDRS) (Reference 2.5.2-273) on Figure 2.5.2-296. The site GMRS are enveloped by the CSDRS.

2.5.2.6.6 PBSRS and FIRS

Following the guidance given in Section 5.2.1 of the Interim Staff Guidance DC/COL-ISG-017, a horizontal PBSRS is developed from the design grade UHRS shown on Figure 2.5.2-308 by applying the relationships described in FSAR Subsection 2.5.2.6.3. Figure 2.5.2-309 shows the resulting PBSRS spectra. At frequencies above about 1 Hz the PBSRS is controlled by the 10^{-4} UHRS multiplied by the design factor (DF). At lower frequencies the PBSRS is controlled by 0.45 times the 10^{-5} UHRS.

Section 5.2.1 of the Interim Staff Guidance DC/COL-ISG-017 procedure was then followed to develop SSI input time histories and soil profiles. The first step was to construct a FIRS at the appropriate foundation elevation by extracting ground motions as outcropping motions from the full column site response analyses used to develop the PBSRS. These outcropping motions are used to construct amplification functions that are in turn used to construct a SCOR FIRS for use in developing input time histories. As the site is to be excavated to an elevation of -7 m (-24 ft.) NAVD88 and the reactor placed on approximately 10.7 m (35 ft.) of concrete backfill, the appropriate point for placing the input motion is at the base of the excavation. Therefore, a SCOR FIRS was developed for elevation -7 m (-24 ft.) NAVD88. In addition, a SCOR FIRS was developed at the reactor foundation elevation of +3.3m (+11 ft.) NAVD88 for the purpose of checking the requirement of the minimum level of ground motion specified in 10 CFR Part 50 Appendix S. The peak ground acceleration of the reactor foundation elevation SCOR FIRS was computed to be 0.0825g. To meet the minimum ground motion requirement of 0.1g peak horizontal acceleration at the foundation elevation specified in 10 CFR Part 50 Appendix S, the site PBSRS and SCOR FIRS were scaled by the factor of 0.1/0.0825. Table 2.5.2-227 lists the resulting scaled horizontal PBSRS. The scaled PBSRS is compared to the Westinghouse CSDRS (Reference 2.5.2-273) on Figure 2.5.2-297. The scaled horizontal PBSRS is enveloped by the CSDRS.

The vertical PBSRS was constructed using V/H spectral ratios. The PBSRS profile consists of a thin layer of soil over rock. This condition is somewhat different than the generic rock conditions for which the V/H ratios shown on Figure 2.5.2-295 were developed. The PEER NGA project (Reference 2.5.2-283) developed a large database of strong motion recordings with documentation of the site conditions. A set of 108 recordings was selected from the PEER NGA database for the following conditions:

- Peak ground acceleration < 0.2g
- Depth to V_S of 1 km/sec < 100 m to obtain records on rock and shallow soil sites
- Lowpass filter used in record processing ≥ 20 Hz to obtain V/H values at moderately high frequencies.
- Vertical component available from the PEER NGA database.

These data were used to compute average V/H ratios for rock sites and shallow soil sites. Figure 2.5.2-310 shows the updated WUS V/H ratios for rock and shallow soil sites suggested by these data. The updated WUS rock V/H ratios are somewhat lower that those given in McGuire et al. (Reference 2.5.2-263). This may be due in part to a larger data set of small shaking level recordings and in part due to separation of the sites into rock and shallow soil. The empirical ground motion models used by McGuire et al. (Reference 2.5.2-263) to develop their recommended WUS rock V/H ratios contained a mixture of rock and shallow soil sites. As indicated on Figure 2.5.2-310, the V/H ratios for shallow soil sites are somewhat higher than those for rock sites in the region of the peak in the spectral ratios.

The updated WUS spectral ratios shown on Figure 2.5.2-310 were used to construct V/H ratios to develop the vertical PBSRS. First, rock V/H ratios for an intermediate κ value were constructed in the same manner as described in FSAR Subsection 2.5.2.6.4. These updated intermediate rock site V/H ratios are shown on Figure 2.5.2-311. Then the ratio of the shallow soil to rock V/H values shown on Figure 2.5.2-310 was used to scale the intermediate rock V/H ratios to account for the effects of the shallow soil (fill). The resulting intermediate shallow soil V/H ratios are shown on Figure 2.5.2-311. The intermediate shallow soil V/H ratios were then multiplied by the scaled horizontal PBSRS to produce the scaled vertical PBSRS. The vertical scaled PBSRS is shown on Figure 2.5.2-297. The vertical scaled PBSRS is enveloped by the Westinghouse CSDRS (Reference 2.5.2-273).

The SCOR FIRS developed for elevation -7 m (-24 ft.) NAVD88 is shown on Figure 3.7-201. These FIRS have been modified (enhanced) at intermediate frequencies to ensure that the surface response spectra computed using the three site velocity profiles developed for SSI analyses envelop the PBSRS.

2.5.2.6.7 Site Profiles for SSI Analysis

Soil profiles for use in SSI analyses were developed from the PBSRS site response analyses following the requirements of the Standard Review Plan and guidance given in Section 5.2.1 of the Interim Staff Guidance DC/COL-ISG-017. These profiles were based on the statistics of the iterated soil properties for the randomized site profiles used to develop the PBSRS. The best estimate profile was set equal to values interpolated between the median iterated soil properties for the 10^{-4} and 10^{-5} ground motion level input cases using the ratio of the GMRS peak ground acceleration to the peak acceleration for the 10^{-4} and 10^{-5} UHRS. The resulting site profile is listed in Table 2.5.2-228. The lower bound profile was set equal to the 16th percentile of the distribution of randomized soil properties for the 500 ft/sec engineered fill velocity case, and the upper bound profile was set equal to the 84th percentile of the distribution of randomized soil properties for the 1000 ft/sec engineered fill case. The range in the upper bound and lower bound shear wave velocities was increased where necessary to maintain the minimum coefficient of variation in shear modulus of 1.5. Tables 2.5.2-229 and 2.5.2-230 list the lower bound and upper bound profile properties, respectively. Figure 2.5.2-298 shows the top 500 feet of the three V_s profiles. The corresponding damping ratios were obtained from the statistics of the iterated profiles assuming negative correlation between V_s and damping: that is the 16th percentile damping was used for the upper bound profile and the 84th percentile damping was used for the lower bound profile. The compression wave velocities were based on the measured values for the in-situ materials and the recommend Poisson's ratio of 0.3 for the engineered fill (Table 2.5.4.5-201). The compression wave velocity of water, set to a nominal value of 5000 ft/sec, was used as a minimum value for the compression wave velocities of materials below the water table.

LNP COL 2.5-3
LNP SUP 3.7-3

A fourth profile called the Lower Lower Bound (LLB) was developed as described in FSAR Subsection 3.7.1.1.1. LLB soil profile is used to account for the degradation of soil shear modulus due foundation installation activities for use in the SSI analysis. The LLB soil profile and properties are shown in Table 2.5.2-231. The degradation of soil shear modulus for the LLB soil profile only applies to in-situ soil layers (layers 7 to 19 in Table 2.5.2-231 which corresponds to depths of 15 ft. to 75 ft.). The material properties for the engineered fill (depths 0 to 15 ft.) and rock (depths greater than 75 ft.) are the same as in the LB soil profile. The low strain shear modulus of the in-situ soil is reduced by 10 percent and the new reduced shear wave velocity was calculated from the shear modulus. The compression wave velocity (V_P) for the in-situ soil was calculated as follows: For in-situ soil below the water table, if the V_P is less than that of water (i.e., 5000 ft/sec), the V_P of the soil is set to 5000 ft/sec (layers 5 to 14 in Table 2.5.2-231). If the V_P is greater than 5000 ft/sec (layer 15 to 19 in Table 2.5.2-231), the V_P is then reduced in the same ratio that the shear wave velocity is being reduced (approximately 5 percent).

LNP COL 2.5-2

Table 2.5.2-201
Parameters of Recent Gulf of Mexico Earthquakes

Date	Source	Reported Magnitude	Type	Converted m_b	Emb	Distance to LNP Site	Comment
1994/6/30	NEIC	4.2	m_b	--	4.2	752 km (467 mi.)	
2000/12/9	NEIC	4.3	M_S	5.0	4.6*	748 km (465 mi.)	
		4.2	m_b	--			
2006/2/10	NEIC	5.3	M_S	5.6	4.9*	757 km (470 mi.)	
		4.2	m_b	--			
2006/4/18		~4.6	**M**	NA	NA	558 km (347 mi.)	Reported by Nettles (References 2.5.2-220 and 2.5.2-221). Not detected or located by USGS (NEIC). Therefore not included in the updated earthquake catalog. Moment magnitude estimated from surface wave recordings.
2006/9/10	NEIC	5.9	m_b	--		498 km (310 mi.)	
		5.9	**M**	6.1	6.0*		
		5.8	**M**				

Notes:

km = kilometer
mi. = mile
M = moment magnitude
m_b = body wave magnitude
M_S = surface wave magnitude
* Represents average reported m_b and converted m_b magnitudes.

LNP COL 2.5-2

Table 2.5.2-202
Bechtel Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
BZ1 Gulf Coast Region	1.0	0	5.4 [0.1], 5.7 [0.4], 6.0 [0.4], 6.6 [0.1]	6.11 [0.1], 6.4 [0.4], 6.6 [0.5]
BZ4 Atlantic Coastal Region	1.0	250	6.8 [0.1], 7.1 [0.4], 7.4 [0.4], 6.6 [0.1]	6.8 [0.1], 7.1 [0.4], 7.4 [0.4], 6.6 [0.1]
BZ5 Southern Appalachians Region	1.0	414.3	5.7 [0.1], 6.0 [0.4], 6.3 [0.4], 6.6 [0.1]	5.7 [0.1], 6.0 [0.4], 6.3 [0.4], 6.6 [0.1]
H Charleston Area	0.42	450.9	6.8 [0.2], 7.1 [0.4], 7.4 [0.4]	6.6 [1.0]
N3 Charleston Faults	0.53	470	6.8 [0.2], 7.1 [0.4], 7.4 [0.4]	6.6 [1.0]

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-203
Dames and Moore Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
20 Southern Coastal Margin	1.0	0	5.3 [0.8], 7.2 [0.2]	5.52 [0.8], 7.2 [0.2]
52 Charleston Mesozoic Rift	0.46	165.9	4.7 [0.75], 7.2 [0.25]	5.01 [0.75], 7.2 [0.25]
53 Southern Appalachian Mobil Belt	1.0*	165.9	5.6 [0.8], 7.2 [0.2]	5.6 [0.8], 7.2 [0.2]
54 Charleston	1.0	440.7	6.6 [0.75], 7.2 [0.25]	6.6 [1.0]
41 Southern Cratonic Margin	1.0*	470.1	6.1 [0.8], 7.2 [0.20]	6.1 [0.8], 7.2 [0.20]
4 Paleozoic (Appalachian) Fold Belt	0.35	579.8	6.0 [0.8], 7.2 [0.2]	6.0 [0.8], 7.2 [0.2]
4A+4B+4C+4D Kinks in Appalachian Fold Belt + remaining fold belt	0.65	639.3 for 4A	6.8 [0.75], 7.2 [0.25] for 4A	6.8 [0.75], 7.2 [0.25] for 4A
		853.7 for 4B	6.2 [0.75], 7.2 [0.25] for 4B	6.2 [0.75], 7.2 [0.25] for 4B
		1267.7 for 4C	5.0 [0.75], 7.2 [0.25] for 4C	5.0 [0.75], 7.2 [0.25] for 4C
		1468.9 for 4D	5.6 [0.75], 7.2 [0.25] for 4D	5.6 [0.75], 7.2 [0.25] for 4D
		579.8 for remainder of 4	6.0 [0.8], 7.2 [0.2] remainder of 4	6.0 [0.8], 7.2 [0.2] remainder of 4

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-204
Law Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
126 Southern Coastal Block	1.0, P^B = 0.49	0	4.6 [0.9], 4.9 [0.1]	5.52 [0.9], 5.7 [0.1]
8 (C09 & C10) Mesozoic Basins	0.27	104.2	6.8 [1.0]	6.8 [1.0]
108 Brunswick	0.73, P^B = 0.42	259.4	4.9 [0.5], 5.5 [0.3], 6.8 [0.2]	5.01 [0.5], 5.5 [0.3], 6.8 [0.2]
107 Eastern Piedmont	0.73	414.1	4.9 [0.3], 5.5 [0.4], 5.7 [0.3]	5.01 [0.3], 5.5 [0.4], 5.7 [0.3]
35 Charleston	0.45	442.2	6.8 [1.0]	6.6 [1.0]
22 Reactivated Normal Faults	0.27	259.4	6.8 [1.0]	6.8 [1.0]
17 Eastern Basement	0.62	445.7	5.7 [0.2], 6.8 [0.8]	5.7 [0.2], 6.8 [0.8]
217 Eastern Basement Background	0.38	445.7	5.2 [0.5], 5.7 [0.5]	5.2 [0.5], 5.7 [0.5]

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-205
Rondout Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
49 (C01) Appalachian Crust	1.0	0	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]	5.01 [0.2], 5.5 [0.6], 5.8 [0.2]
51 Gulf Coast to Bahamas Fracture Zone	1.0	84.5	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]	6.11 [0.3], 6.3 [0.55], 6.5 [0.15]
24 Charleston	1.0	410.1	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]	6.6 [1.0]
26 South Carolina Zone	1.0	314.4	5.8 [0.15], 6.5 [0.6], 6.8 [0.25]	5.8 [0.15], 6.5 [0.6], 6.8 [0.25]
25 Southern Appalachians	0.985	626.6	6.6 [0.3], 6.8 [0.6], 7.0 [0.1]	6.6 [0.3], 6.8 [0.6], 7.0 [0.1]

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-206
Weston Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
107 Gulf Coast	1.0	0	5.4 [0.71], 6.0 [0.29]	6.6 [0.89], 7.2 [0.11]
104 Southern Coastal Plain	1.0	194.1	5.4 [0.24], 6.0 [0.61], 6.6 [0.15]	5.4 [0.24], 6.0 [0.61], 6.6 [0.15]
25 Charleston	0.99	436.4	6.6 [0.9], 7.2 [0.1]	6.6 [1.0]
26 South Carolina Zone	0.86	338	6.0 [0.67], 6.6 [0.27], 7.2 [0.06]	6.0 [0.67], 6.6 [0.27], 7.2 [0.06]
103 Southern Appalachian	1.0	461.6	5.7 [0.26], 6.0 [0.58], 6.6 [0.16]	5.7 [0.26], 6.0 [0.58], 6.6 [0.16]
24 NY-Alabama-Clingman Block	0.9	592.3	5.4 [0.26], 6.0[0.58], 6.6 [0.16]	5.4 [0.26], 6.0[0.58], 6.6 [0.16]

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-207
Woodward-Clyde Team Seismic Sources

Source	P*	Closest Distance to LNP Site (km)	EPRI (1989) Maximum Magnitude Distribution (m_b)	Maximum Magnitude Distribution Used in PSHA for LNP Site (m_b)
B36 Crystal River Background	1.0	0	4.9 [0.17], 5.4 [0.28], 5.8 [0.27], 6.5 [0.28]	5.01 [0.17], 5.4 [0.28], 5.8 [0.27], 6.5 [0.28]
29 Greater South Carolina	0.122	253.4	6.7 [0.33], 7.0 [0.34], 7.4 [0.33]	6.6 [1.0]
29A South Carolina Gravity Saddle Extended	0.305	396.3	6.7 [0.33], 7.0 [0.34], 7.4 [0.33]	6.6 [1.0]
29B South Carolina Gravity Saddle Alternative	0.105	371.5	5.4 [0.33], 6.0 [0.34], 6.6 [0.33]	5.4 [0.33], 6.0 [0.34], 6.6 [0.33]
30 Charleston Zone	0.573	440.7	6.8 [0.33], 7.3 [0.34], 7.5 [0.33]	6.6 [1.0]
31 and 31A Blue Ridge Zone and Alternative	0.235	632	5.9 [0.33], 6.3 [0.34], 7.0 [0.33]	5.9 [0.33], 6.3 [0.34], 7.0 [0.33]

Notes:

km = kilometer
m_b = body-wave magnitude
P* = probability an EPRI-SOG seismic source is active

LNP COL 2.5-2

Table 2.5.2-208 (Sheet 1 of 3)
Description of the Minimum Set Zones for the LLNL TIP Study

Earthquake Source Zone	Description
1. General	Savy et al. present six maps showing the source zones significant to Vogtle and eight showing the source zones for Watts Bar (Reference 2.5.2-238). 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 Savy et al. (Reference 2.5.2-238). A summary map showing the major source zone alternative boundaries is presented on Figure 2.5.2-210.
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 boundaries 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.

LNP COL 2.5-2

Table 2.5.2-208 (Sheet 2 of 3)
Description of the Minimum Set Zones for the LLNL TIP Study

Earthquake Source Zone	Description
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 fuzzy boundaries. Possible combinations are: (4A-l) + (4A-2) + (4A-3) (4A-1) + (4A-2) (4A-1) Zone 4B is made up of two areas: 4B-1 and 4B-2. 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-l) + (4A-2) + (4A-3), but has a graded boundary defined by three cylindrical sources.

LNP COL 2.5-2

Table 2.5.2-208 (Sheet 3 of 3) Description of the Minimum Set Zones for the LLNL TIP Study

Earthquake Source Zone	Description
5. Appalachian/Central United States	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-l) + (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).

Notes:

ETSZ = East Tennessee seismic zone

Source: Reference 2.5.2-238

LNP COL 2.5-2

Table 2.5.2-209
Comparison of EPRI EST Characterizations of Gulf of Mexico Coastal Source Zones and Modifications for STP 3 & 4

EPRI EST	Source	Description	EPRI Model M_{max} (m_b) and Wts.	Updated Model for STP 3 & 4: M_{max} (m_b) and Wts.	Updated Model for STP 3 & 4: Smoothing Options and Wts.
Bechtel Group	BZ1	Gulf Coast	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	6.1 [0.10] 6.4 [0.40] 6.6 [0.50]	No Update
Dames & Moore	20	South Coastal Margin	5.3 [0.8] 7.2 [0.2]	5.5 [0.80] 7.2 [0.20]	I (0.2) II (0.4) III (0.4)
Law Engineering	126	South Coastal Block	4.6 [0.9] 4.9 [0.1]	5.5 [0.90] 5.7 [0.10]	No update
Rondout Associates	51	Gulf Coast to Bahamas Fracture Zone	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	6.1 [0.30] 6.3 [0.55] 6.5 [0.15]	No update
Weston Geophysical Corporation	107	Gulf Coast	5.4 [0.71] 6.0 [0.29]	6.6 [0.89] 7.2 [0.11]	No update
Woodward-Clyde Consultants	B43	Central US Backgrounds	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	No update	No update

Notes:

I: Constant a, constant b, strong prior on b of 1.04

II: Medium smoothing on a, medium smoothing on b, strong prior on b of 1.04

III: High smoothing on a, high smoothing on b, strong prior on b of 1.04

m_b = body-wave magnitude
wts = weight

Source: Reference 2.5.2-244

LNP COL 2.5-2

Table 2.5.2-210
Earthquake Counts and Assessed Catalog Completeness within 320 km (200 mi.) of LNP Site within EPRI-SOG Completeness Regions

	Assessed Probability of Detection for Time Period(a)								Earthquake Counts for Time Period								
	1625 to 1780	1780 to 1860	1860 to 1910	1910 to 1950	1950 to 1975	1975 to 3/1985	3/1985 to 1/2007										
	Corresponding Time Length (years)							TE	1625 to	1780 to	1860 to	1910 to	1950 to	1975 to	3/1985 to	Total Count	Earthquake
m_b(a)	155	80	50	40	25	10.16	21.84	(years)	1780	1860	1910	1950	1975	3/1985	1/2007	for TE	Catalog
Completeness Region 2																	
3.3 to 3.9			0.102	0.507	0.625	1	0	51.31			0	0	1	0	0	1	EPRI-SOG
			0.102	0.507	0.625	1	1	73.15			0	0	1	0	0	1	Update
			0.102	0.507	0.625	1	1	73.15			0	0	1	0	0	1	Update-EPRI AS
3.9 to 4.5			0.148	0.899	1	1		78.66			0	0	0	0	0	0	EPRI-SOG
			0.148	0.899	1	1	1	100.50			0	0	0	0	0	0	Update
			0.148	0.899	1	1	1	100.50			0	0	0	0	0	0	Update-EPRI AS
4.5 to 5.1			0.236	0.980	1	1		86.36			0	0	0	0	0	0	EPRI-SOG
			0.236	0.980	1	1	1	108.20			0	0	0	0	0	0	Update
			0.236	0.980	1	1	1	108.20			0	0	0	0	0	0	Update-EPRI AS
5.1 to 5.7			0.236	0.980	1	1		86.36			0	0	0	0	0	0	EPRI-SOG
			0.236	0.980	1	1	1	108.20			0	0	0	0	0	0	Update
			0.236	0.980	1	1	1	108.20			0	0	0	0	0	0	Update-EPRI AS
5.7 to 6.3			0.702	1	1	1		110.16			0	0	0	0	0	0	EPRI-SOG
			0.702	1	1	1	1	132.00			0	0	0	0	0	0	Update
			0.702	1	1	1	1	132.00			0	0	0	0	0	0	Update-EPRI AS
6.3 to 6.9		0.015	1	1	1	1		125.96		0	0	0	0	0	0	0	EPRI-SOG
		0.015	1	1	1	1	1	147.80		0	0	0	0	0	0	0	Update
		0.015	1	1	1	1	1	147.80		0	0	0	0	0	0	0	Update-EPRI AS
Completeness Region 13																	
3.3 to 3.9			0.242	0.714	0.882	1		72.87			7	3	2	1	0	13	EPRI-SOG
			0.277	0.714	0.882	1	1	96.44			8	3	2	1	1	15	Update
			0.242	0.714	0.882	1	1	94.71			3	2	2	1	1	9	Update-EPRI AS
3.9 to 4.5			0.242	0.774	0.954	1		77.07			1	0	0	0	0	1	EPRI-SOG
			0.277	0.774	0.954	1	1	100.64			1	0	0	0	0	1	Update
			0.277	0.774	0.954	1	1	100.64			1	0	0	0	0	1	Update-EPRI AS
4.5 to 5.1			0.297	0.920	0.989	1		86.54			1	0	0	0	0	1	EPRI-SOG
			0.297	0.920	0.989	1	1	108.38			1	0	0	0	0	1	Update
			0.297	0.920	0.989	1	1	108.38			1	0	0	0	0	1	Update-EPRI AS
5.1 to 5.7			0.692	0.989	1	1		109.32			0	0	0	0	0	0	EPRI-SOG
			0.692	0.989	1	1	1	131.16			0	0	0	0	0	0	Update
			0.692	0.989	1	1	1	131.16			0	0	0	0	0	0	Update-EPRI AS
5.7 to 6.3			0.981	1	1	1		124.21			0	0	0	0	0	0	EPRI-SOG
			0.981	1	1	1	1	146.05			0	0	0	0	0	0	Update
			0.981	1	1	1	1	146.05			0	0	0	0	0	0	Update-EPRI AS
6.3 to 6.9			1	1	1	1		125.16			0	0	0	0	0	0	EPRI-SOG
			1	1	1	1	1	147.00			0	0	0	0	0	0	Update
			1	1	1	1	1	147.00			0	0	0	0	0	0	Update-EPRI AS

m_b = body-wave magnitude
TE = equivalent period of completeness

Notes:

a) Blank cells for time periods before 1985 denote time periods for which EPRI-SOG considered catalog to be unusable.

LNP COL 2.5-2

Notes:

Table 2.5.2-211
Assessed Probabilities of Detection for the Gulf of Mexico Completeness Region

	Assessed Probability of Detection for Time Period[a]								Earthquake Counts for Time Period								
	1625 to 1780	1780 to 1860	1860 to 1910	1910 to 1950	1950 to 1975	1975 to 1984	1984 to 1/2007										
	Corresponding Time Length (years)																
m_b[a]	155	80	50	40	25	9	23	TE (years)	1625 to 1780	1780 to 1860	1860 to 1910	1910 to 1950	1950 to 1975	1975 to 3/1985	3/1985 to 1/2007	Total Count for TE	Earthquake Catalogue
Gulf of Mexico Completeness Region																	
3.3 to 3.9						0.06	0.18	4.68						1	9	10	Updated Catalog
3.9 to 4.5					0.18	0.18	0.18	10.26					0	0	1	1	Updated Catalog
4.5 to 5.1					0.52	0.57	0.57	31.24					1	1	1	3	Updated Catalog
5.1 to 5.7					0.90	0.94	0.98	53.5					0	0	1	1	Updated Catalog
5.7 to 6.3					0.99	0.99	1	56.66					0	0	1	1	Updated Catalog
6.3 to 6.9					1	1	1	57					0	0	0	0	Updated Catalog

a) Blank cells for time periods before 1985 denote time periods for which EPRI-SOG considered catalog to be unusable.

m_b = body-wave magnitude
TE = equivalent period of completeness

LNP COL 2.5-2

Table 2.5.2-212
Frequencies for Repeated Large-Magnitude Charleston Earthquakes

Recurrence Model	Weight	Recurrence Interval (years)	Equivalent Annual Frequency
Charleston ≈2000-year record (weight 0.8)	0.10108	337	2.96E-03
	0.24429	435	2.30E-03
	0.30926	531	1.88E-03
	0.24429	649	1.54E-03
	0.10108	836	1.20E-03
Charleston ≈5000-year record (weight 0.2)	0.10108	334	3.00E-03
	0.24429	559	1.79E-03
	0.30926	841	1.19E-03
	0.24429	1265	7.90E-04
	0.10108	2120	4.72E-04

LNP COL 2.5-2

Table 2.5.2-213
PSHA Results for 0.5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

0.5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	**5th%**	**16th%**	**50th%**	**84th%**	**95th%**
1.00E-05	4.68E-02	1.05E-02	2.29E-02	4.37E-02	7.08E-02	8.71E-02
1.00E-04	1.58E-02	3.63E-03	5.75E-03	1.20E-02	2.69E-02	3.98E-02
1.00E-03	2.71E-03	9.33E-04	1.41E-03	2.34E-03	3.89E-03	5.75E-03
2.00E-03	1.68E-03	4.17E-04	7.76E-04	1.51E-03	2.51E-03	3.47E-03
5.00E-03	7.75E-04	7.41E-05	1.91E-04	5.89E-04	1.41E-03	2.04E-03
1.00E-02	3.45E-04	1.18E-05	3.72E-05	1.74E-04	6.76E-04	1.26E-03
2.00E-02	1.20E-04	1.15E-06	4.68E-06	3.16E-05	2.19E-04	5.37E-04
3.00E-02	5.70E-05	2.40E-07	1.18E-06	9.33E-06	8.51E-05	2.63E-04
5.00E-02	1.96E-05	2.57E-08	1.55E-07	1.74E-06	2.04E-05	8.13E-05
1.00E-01	3.55E-06	1.78E-09	6.76E-09	1.51E-07	2.24E-06	9.77E-06
3.00E-01	1.13E-07	1.00E-10	4.07E-10	2.19E-09	5.89E-08	3.24E-07
1.00E+00	1.52E-09	1.00E-10	1.00E-10	1.00E-10	1.70E-09	8.71E-09

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-214
PSHA Results for 1-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

1-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-04	2.74E-02	6.31E-03	1.10E-02	2.40E-02	4.47E-02	6.03E-02
1.00E-03	4.87E-03	1.66E-03	2.34E-03	3.80E-03	7.41E-03	1.15E-02
3.00E-03	2.12E-03	6.76E-04	1.10E-03	1.86E-03	3.09E-03	4.37E-03
1.00E-02	7.01E-04	8.71E-05	2.04E-04	5.50E-04	1.23E-03	1.82E-03
2.00E-02	2.69E-04	1.51E-05	4.17E-05	1.55E-04	4.90E-04	9.33E-04
3.00E-02	1.33E-04	4.57E-06	1.38E-05	5.89E-05	2.34E-04	5.13E-04
5.00E-02	4.63E-05	8.32E-07	2.88E-06	1.48E-05	6.92E-05	1.95E-04
1.00E-01	8.41E-06	6.17E-08	2.29E-07	1.91E-06	1.05E-05	3.09E-05
2.00E-01	1.18E-06	3.72E-09	1.32E-08	1.86E-07	1.59E-06	4.07E-06
3.00E-01	3.57E-07	1.35E-09	2.95E-09	4.27E-08	5.37E-07	1.59E-06
5.00E-01	8.33E-08	6.17E-10	1.12E-09	6.03E-09	1.32E-07	4.27E-07
1.00E+00	1.22E-08	1.00E-10	1.00E-10	1.32E-09	1.62E-08	6.92E-08

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-215
PSHA Results for 2.5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

2.5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-04	4.25E-02	1.10E-02	2.24E-02	4.07E-02	6.31E-02	7.76E-02
1.00E-03	1.05E-02	3.02E-03	4.37E-03	8.13E-03	1.74E-02	2.51E-02
3.00E-03	4.03E-03	1.55E-03	2.09E-03	3.31E-03	5.89E-03	8.71E-03
1.00E-02	1.47E-03	4.57E-04	7.24E-04	1.32E-03	2.19E-03	3.02E-03
2.00E-02	6.79E-04	1.29E-04	2.40E-04	5.50E-04	1.12E-03	1.66E-03
5.00E-02	1.51E-04	1.32E-05	2.88E-05	8.71E-05	2.57E-04	4.90E-04
1.00E-01	3.23E-05	1.82E-06	4.37E-06	1.59E-05	5.01E-05	1.12E-04
2.00E-01	5.58E-06	2.14E-07	5.75E-07	2.69E-06	8.71E-06	1.74E-05
3.00E-01	1.99E-06	5.13E-08	1.55E-07	9.12E-07	3.63E-06	6.92E-06
5.00E-01	5.75E-07	7.08E-09	2.75E-08	1.95E-07	1.18E-06	2.29E-06
1.00E+00	1.05E-07	1.15E-09	2.63E-09	1.86E-08	2.19E-07	4.68E-07
3.00E+00	4.22E-09	1.00E-10	6.17E-10	1.32E-09	8.32E-09	2.19E-08

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-216
PSHA Results for 5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	1.43E-02	4.27E-03	6.61E-03	1.18E-02	2.24E-02	3.24E-02
3.00E-03	5.48E-03	2.04E-03	2.75E-03	4.47E-03	8.13E-03	1.26E-02
1.00E-02	1.92E-03	6.76E-04	1.00E-03	1.70E-03	2.82E-03	3.89E-03
2.00E-02	9.27E-04	2.24E-04	3.80E-04	7.76E-04	1.48E-03	2.09E-03
3.00E-02	5.39E-04	9.77E-05	1.78E-04	4.17E-04	8.91E-04	1.38E-03
5.00E-02	2.35E-04	2.95E-05	5.89E-05	1.55E-04	3.89E-04	6.92E-04
1.00E-01	6.03E-05	5.13E-06	1.15E-05	3.47E-05	9.12E-05	1.82E-04
2.00E-01	1.37E-05	8.91E-07	2.24E-06	8.13E-06	2.09E-05	3.89E-05
3.00E-01	5.85E-06	2.69E-07	8.71E-07	3.39E-06	9.55E-06	1.74E-05
5.00E-01	2.05E-06	5.13E-08	2.40E-07	1.12E-06	3.72E-06	6.61E-06
1.00E+00	4.56E-07	5.01E-09	3.16E-08	2.04E-07	9.12E-07	1.70E-06
3.00E+00	2.44E-08	8.91E-10	1.32E-09	6.03E-09	5.01E-08	1.15E-07

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-217
PSHA Results for 10-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

10-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	1.41E-02	4.47E-03	6.76E-03	1.18E-02	2.14E-02	3.24E-02
3.00E-03	5.71E-03	2.09E-03	2.82E-03	4.47E-03	8.13E-03	1.35E-02
1.00E-02	2.05E-03	6.46E-04	1.00E-03	1.78E-03	2.88E-03	4.27E-03
2.00E-02	9.91E-04	2.04E-04	3.80E-04	8.32E-04	1.51E-03	2.29E-03
5.00E-02	2.69E-04	2.95E-05	6.61E-05	1.78E-04	4.27E-04	7.94E-04
1.00E-01	8.02E-05	6.92E-06	1.62E-05	4.68E-05	1.15E-04	2.40E-04
2.00E-01	2.25E-05	1.38E-06	3.98E-06	1.35E-05	3.31E-05	6.46E-05
3.00E-01	1.08E-05	5.50E-07	1.86E-06	6.76E-06	1.78E-05	3.09E-05
5.00E-01	4.35E-06	1.00E-07	6.92E-07	2.69E-06	7.94E-06	1.35E-05
1.00E+00	1.19E-06	1.62E-08	1.55E-07	6.61E-07	2.29E-06	3.89E-06
2.00E+00	2.56E-07	2.75E-09	1.70E-08	1.12E-07	4.90E-07	9.55E-07
5.00E+00	1.90E-08	8.51E-10	1.29E-09	5.50E-09	3.47E-08	8.71E-08

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-218
PSHA Results for 25-Hz Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

25-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	1.16E-02	3.09E-03	4.68E-03	8.51E-03	1.86E-02	2.95E-02
3.00E-03	5.21E-03	1.48E-03	2.19E-03	3.63E-03	7.24E-03	1.29E-02
1.00E-02	2.00E-03	3.80E-04	7.41E-04	1.51E-03	2.82E-03	4.47E-03
3.00E-02	6.01E-04	5.50E-05	1.29E-04	3.80E-04	9.77E-04	1.70E-03
1.00E-01	1.09E-04	6.46E-06	1.55E-05	5.13E-05	1.38E-04	3.02E-04
2.00E-01	3.75E-05	1.82E-06	5.13E-06	1.91E-05	4.68E-05	1.00E-04
3.00E-01	1.95E-05	7.08E-07	2.75E-06	1.07E-05	2.69E-05	5.62E-05
5.00E-01	8.36E-06	1.78E-07	1.38E-06	4.90E-06	1.38E-05	2.57E-05
1.00E+00	2.64E-06	3.89E-08	3.02E-07	1.48E-06	4.90E-06	9.12E-06
2.00E+00	7.75E-07	8.51E-09	4.07E-08	3.16E-07	1.41E-06	2.95E-06
5.00E+00	1.13E-07	1.02E-09	2.04E-09	2.09E-08	1.66E-07	5.13E-07
7.00E+00	4.91E-08	8.32E-10	1.12E-09	7.41E-09	6.03E-08	2.34E-07

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-219
PSHA Results for 100-Hz (PGA) Spectral Acceleration on CEUS Generic Hard Rock for the LNP Site

100-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	8.07E-03	2.51E-03	3.47E-03	5.62E-03	1.29E-02	2.29E-02
3.00E-03	3.27E-03	1.15E-03	1.62E-03	2.57E-03	4.57E-03	7.94E-03
1.00E-02	1.00E-03	1.91E-04	3.47E-04	7.76E-04	1.66E-03	2.51E-03
2.00E-02	3.79E-04	3.98E-05	8.32E-05	2.29E-04	6.61E-04	1.20E-03
3.00E-02	1.93E-04	1.62E-05	3.63E-05	1.02E-04	3.16E-04	6.46E-04
5.00E-02	7.70E-05	5.50E-06	1.29E-05	3.98E-05	1.10E-04	2.34E-04
1.00E-01	2.19E-05	1.18E-06	3.80E-06	1.35E-05	3.24E-05	6.03E-05
2.00E-01	6.88E-06	1.78E-07	1.23E-06	4.57E-06	1.20E-05	2.00E-05
3.00E-01	3.62E-06	6.76E-08	5.37E-07	2.46E-06	6.76E-06	1.10E-05
5.00E-01	1.55E-06	2.19E-08	1.55E-07	8.91E-07	2.95E-06	5.50E-06
1.00E+00	4.12E-07	3.24E-09	1.55E-08	1.59E-07	7.59E-07	1.62E-06
3.00E+00	2.39E-08	6.61E-10	9.55E-10	3.72E-09	3.31E-08	1.10E-07

Notes:

Hz = hertz

LNP COL 2.5-2

Table 2.5.2-220
Uniform Hazard Response Spectra for the LNP Site for Generic Hard Rock Conditions

Period (sec)	Frequency (Hz)	Spectral Acceleration (g) for Annual Exceedance Frequency of:			
		Mean 10^{-3}	Mean 10^{-4}	Mean 10^{-5}	Mean 10^{-6}
0.01	100	0.0100	0.0433	0.1599	0.6291
0.04	25	0.0188	0.1056	0.4489	1.7314
0.1	10	0.0199	0.0885	0.3132	1.0870
0.2	5	0.0188	0.0782	0.2322	0.7024
0.4	2.5	0.0145	0.0610	0.1594	0.3973
1	1	0.0072	0.0347	0.0937	0.2115
2	0.5	0.0037	0.0221	0.0657	0.1498

Notes:

Hz = hertz
sec = second

LNP COL 2.5-2

Table 2.5.2-221
Rock Hazard Reference and Deaggregation Earthquakes

	Reference (Controlling) Earthquake		Deaggregation Earthquakes		
Hazard	Magnitude (m_b)	Distance (km)	Magnitude (m_b)	Distance (km)	Weight
Mean 10^{-3} 5 and 10 Hz	6.6	302	5.3	63.7	0.180
			6.0	139	0.060
			6.9	464	0.760
Mean 10^{-3} 1 and 2.5 Hz	6.8 6.9[(a)]	368 442[(a)]	5.4	49.7	0.079
			6.1	141	0.047
			6.9	469	0.874
Mean 10^{-4} 5 and 10 Hz	6.5	161	5.4	27.7	0.320
			6.2	70	0.077
			7.1	455	0.603
Mean 10^{-4} 1 and 2.5 Hz	6.9 7.1[(a)]	299 447[(a)]	5.5	20.2	0.105
			6.3	72	0.052
			7.1	459	0.843
Mean 10^{-5} 5 and 10 Hz	6.0	34	5.4	13.6	0.615
			6.3	29	0.156
			7.2	453	0.229
Mean 10^{-5} 1 and 2.5 Hz	6.7 7.1[(a)]	159 446[(a)]	5.6	12.2	0.218
			6.4	45	0.112
			7.2	456	0.670
Mean 10^{-6} 5 and 10 Hz	5.8	11	5.4	8.9	0.681
			6.4	15	0.297
			7.2	450	0.022
Mean 10^{-6} 1 and 2.5 Hz	6.4 7.2[(a)]	50 443[(a)]	5.7	8.9	0.400
			6.5	32	0.240
			7.2	455	0.360

Notes:

a) computed using earthquakes with distances > 100 km

Hz = Hertz
km = kilometer
m_b = body-wave magnitude

LNP COL 2.5-2

Table 2.5.2-222
GMRS Profile LNP 1 for LNP 1
Surface Elevation 36 ft. NAVD88

Layer Number	Thickness (ft.)	Shear-Wave Velocity (ft/sec)	Unit Weight (kips/ft^3)	Material Curves
1	7	1500	0.120	PR 0-50 ft. or SR 0-20 ft.
2	7	1500	0.120	PR 0-50 ft. or SR 0-20 ft.
3	13	1500	0.120	PR 0-50 ft. or SR 21-50 ft.
4	17	2300	0.130	PR 0-50 ft. or SR 21-50 ft.
5	15	2300	0.130	PR >50 ft. or SR 51-120 ft.
6	26	2850	0.138	Linear, κ layer 1
7	46	2700	0.138	Linear, κ layer 1
8	62	3450	0.138	Linear, κ layer 1
9	18	3400	0.138	Linear, κ layer 1
10	24	3300	0.120	PR >50 ft. or SR 121-250 ft.
11	24	3300	0.120	PR >50 ft. or SR 121-250 ft.
12	40	3550	0.120	Linear, κ layer 2
13	43	3350	0.120	Linear, κ layer 2
14	38	4200	0.140	Linear, κ layer 3
15	60	3350	0.140	Linear, κ layer 3
16	60	3800	0.140	Linear, κ layer 3
17	240	4600	0.140	Linear, κ layer 3
18	360	5900	0.140	Linear, κ layer 3
19	250	7400	0.150	Linear, κ layer 4
20	250	5100	0.150	Linear, κ layer 4
21	150	7200	0.150	Linear, κ layer 4
22	100	6150	0.150	Linear, κ layer 4
23	200	7250	0.150	Linear, κ layer 4
24	600	5400	0.150	Linear, κ layer 5
25	150	5900	0.150	Linear, κ layer 5
26	200	6200	0.150	Linear, κ layer 5
27	650	5200	0.150	Linear, κ layer 5
28	600	5600	0.150	Linear, κ layer 5
29	100	4800	0.150	Linear, κ layer 5
Halfspace		9300	0.169	0.1% Damping

Notes:
ft. = feet; ft/sec = feet per second; kips/ft^3 = kips per cubic foot

LNP COL 2.5-2

Table 2.5.2-223
GMRS Profile LNP 2 for LNP 2
Surface Elevation 36 ft. NAVD88

Layer Number	Thickness (ft.)	Shear-Wave Velocity (ft/sec)	Unit Weight (kips/ft^3)	Material Curves
1	7	1500	0.120	PR 0-50 ft. or SR 0-20 ft.
2	7	1500	0.120	PR 0-50 ft. or SR 0-20 ft.
3	14	1500	0.120	PR 0-50 ft. or SR 21-50 ft.
4	18	2500	0.130	PR 0-50 ft. or SR 21-50 ft.
5	19	2500	0.130	PR >50 ft. or SR 51-120 ft.
6	20	3950	0.138	Linear, κ layer 1
7	49	3400	0.138	Linear, κ layer 1
8	51	4300	0.138	Linear, κ layer 1
9	12	3625	0.138	Linear, κ layer 1
10	27	2650	0.120	PR >50 ft. or SR 121-250 ft.
11	26	2650	0.120	PR >50 ft. or SR 121-250 ft.
12	35	3350	0.120	Linear, κ layer 2
13	45	3350	0.120	Linear, κ layer 2
14	45	4300	0.140	Linear, κ layer 3
15	50	3400	0.140	Linear, κ layer 3
16	75	4100	0.140	Linear, κ layer 3
17	240	4600	0.140	Linear, κ layer 3
18	360	5900	0.140	Linear, κ layer 3
19	250	7400	0.150	Linear, κ layer 4
20	250	5100	0.150	Linear, κ layer 4
21	150	7200	0.150	Linear, κ layer 4
22	100	6150	0.150	Linear, κ layer 4
23	200	7250	0.150	Linear, κ layer 4
24	600	5400	0.150	Linear, κ layer 5
25	150	5900	0.150	Linear, κ layer 5
26	200	6200	0.150	Linear, κ layer 5
27	650	5200	0.150	Linear, κ layer 5
28	600	5600	0.150	Linear, κ layer 5
29	100	4800	0.150	Linear, κ layer 5
Halfspace		9300	0.169	0.1% Damping

Notes:
ft. = feet; ft/sec = feet per second; kips/ft^3 = kips per cubic foot

LNP COL 2.5-2

Table 2.5.2-224
Statistics of Damping Ratios for Sedimentary Rock

Kappa Layer	Average Equivalent Damping Ratio (%) for:		
	κ = 0.0059 sec	κ = 0.0132 sec	κ = 0.0243 sec
Profile LNP 1			
1	0.62%	1.39%	2.56%
2	0.55%	1.23%	2.26%
3	0.38%	0.85%	1.57%
4	0.29%	0.65%	1.19%
5	0.34%	0.76%	1.40%
Profile LNP 2			
1	0.59%	1.31%	2.41%
2	0.66%	1.48%	2.72%
3	0.45%	1.00%	1.85%
4	0.34%	0.77%	1.41%
5	0.40%	0.90%	1.66%

LNP COL 2.5-2

Table 2.5.2-225
Time History Data Sets Used For Each Deaggregation Earthquake

Hazard Level	Deaggregation Earthquakes (DE)				
	Designation	Magnitude (m_b)	Distance (km)	Weight	NUREG/CR-6728 CEUS Data Set
Mean 10^{-3} 5 and 10 Hz	HF DEL	5.3	63.7	0.180	**M** 4.5 – 6, D 0 – 50 km
	HF DEM	6.0	139	0.060	**M** 6 – 7, D 100 – 200 km
	HF DEH	6.9	464	0.760	**M** >7, D 100 – 200 km
Mean 10^{-3} 1 and 2.5 Hz	LF DEL	5.4	49.7	0.079	**M** 4.5 – 6, D 0 – 50 km
	LF DEM	6.1	141	0.047	**M** 6 – 7, D 100 – 200 km
	LF DEH	6.9	469	0.874	**M** >7, D 100 – 200 km
Mean 10^{-4} 5 and 10 Hz	HF DEL	5.4	27.7	0.320	**M** 4.5 – 6, D 0 – 50 km
	HF DEM	6.2	70	0.077	**M** 6 – 7, D 50 – 100 km
	HF DEH	7.1	455	0.603	**M** >7, D 100 – 200 km
Mean 10^{-4} 1 and 2.5 Hz	LF DEL	5.5	20.2	0.105	**M** 4.5 – 6, D 0 – 50 km
	LF DEM	6.3	72	0.052	**M** 6 – 7, D 50 – 100 km
	LF DEH	7.1	459	0.843	**M** >7, D 100 – 200 km
Mean 10^{-5} 5 and 10 Hz	HF DEL	5.4	13.6	0.615	**M** 4.5 – 6, D 0 – 50 km
	HF DEM	6.3	29	0.156	**M** 6 – 7, D 10 – 50 km
	HF DEH	7.2	453	0.229	**M** >7, D 100 – 200 km
Mean 10^{-5} 1 and 2.5 Hz	LF DEL	5.6	12.2	0.218	**M** 4.5 – 6, D 0 – 50 km
	LF DEM	6.4	45	0.112	**M** 6 – 7, D 10 – 50 km
	LF DEH	7.2	456	0.670	**M** >7, D 100 – 200 km
Mean 10^{-6} 5 and 10 Hz	HF DEL	5.4	8.9	0.681	**M** 4.5 – 6, D 0 – 50 km
	HF DEM	6.4	15	0.297	**M** 6 – 7, D 10 – 50 km
	HF DEH	7.2	450	0.022	**M** >7, D 100 – 200 km
Mean 10^{-6} 1 and 2.5 Hz	LF DEL	5.7	8.9	0.400	**M** 4.5 – 6, D 0 – 50 km
	LF DEM	6.5	32	0.240	**M** 6 – 7, D 10 – 50 km
	LF DEH	7.2	455	0.360	**M** >7, D 100 – 200 km

Notes:

DEH, DEL, DEM = Deaggregation Earthquakes (high, low, medium magnitudes)
HF = high frequency
km = kilometer
LF = low frequency
m_b = body-wave magnitude
M = moment magnitude

LNP COL 2.5-3

Table 2.5.2-226
LNP Site GMRS (Sheet 1 of 2)

Spectral Frequency (Hz)	5 Percent Damped Spectral Acceleration (g)			
	10^{-5} UHRS	Horizontal GMRS	Vertical/Horizontal	Vertical GMRS
100.000	0.1537	0.0691	0.744	0.0514
60.241	0.1889	0.0850	0.762	0.0648
50.000	0.2144	0.0965	0.799	0.0771
40.000	0.2396	0.1078	0.874	0.0942
33.333	0.2621	0.1180	0.900	0.1061
30.303	0.2717	0.1223	0.894	0.1093
25.000	0.3163	0.1423	0.869	0.1237
23.810	0.3250	0.1463	0.860	0.1259
22.727	0.3335	0.1501	0.850	0.1276
21.739	0.3419	0.1538	0.840	0.1292
20.833	0.3500	0.1575	0.827	0.1303
20.000	0.3580	0.1611	0.815	0.1313
18.182	0.3755	0.1690	0.784	0.1324
16.667	0.3922	0.1765	0.755	0.1332
15.385	0.4081	0.1837	0.732	0.1345
14.286	0.4235	0.1906	0.714	0.1360
13.333	0.4383	0.1973	0.697	0.1375
12.500	0.4527	0.2037	0.682	0.1389
11.765	0.4610	0.2075	0.668	0.1386
11.111	0.4634	0.2085	0.655	0.1365
10.526	0.4657	0.2096	0.642	0.1345
10.000	0.4679	0.2105	0.630	0.1326
9.091	0.4725	0.2126	0.619	0.1317
8.333	0.4767	0.2145	0.614	0.1318
7.692	0.4635	0.2086	0.610	0.1272
7.143	0.4515	0.2032	0.606	0.1231
6.667	0.4407	0.1983	0.602	0.1194
6.250	0.4317	0.1943	0.600	0.1166
5.882	0.4233	0.1905	0.600	0.1143
5.556	0.4156	0.1870	0.600	0.1122
5.263	0.4085	0.1838	0.600	0.1103
5.000	0.4018	0.1808	0.600	0.1085
4.545	0.3870	0.1741	0.600	0.1045
4.167	0.3740	0.1683	0.600	0.1010
3.846	0.3624	0.1631	0.600	0.0978
3.571	0.3519	0.1584	0.600	0.0950
3.333	0.3425	0.1541	0.600	0.0925
3.125	0.3339	0.1503	0.600	0.0902
2.941	0.3260	0.1467	0.600	0.0880
2.778	0.3188	0.1435	0.600	0.0861
2.632	0.3121	0.1404	0.600	0.0843
2.500	0.3058	0.1376	0.600	0.0826
2.381	0.2993	0.1347	0.600	0.0808
2.273	0.2933	0.1320	0.600	0.0792
2.174	0.2876	0.1294	0.600	0.0776
2.083	0.2822	0.1270	0.600	0.0762
2.000	0.2772	0.1247	0.600	0.0748
1.818	0.2658	0.1196	0.600	0.0718
1.667	0.2558	0.1151	0.600	0.0691
1.538	0.2451	0.1103	0.600	0.0662

LNP COL 2.5-3

Table 2.5.2-226
LNP Site GMRS (Sheet 2 of 2)

Spectral Frequency (Hz)	5 Percent Damped Spectral Acceleration (g)			
	10^{-5} UHRS	Horizontal GMRS	Vertical/Horizontal	Vertical GMRS
1.429	0.2317	0.1043	0.600	0.0626
1.333	0.2199	0.0989	0.600	0.0594
1.250	0.2062	0.0928	0.600	0.0557
1.176	0.1941	0.0874	0.600	0.0524
1.111	0.1834	0.0825	0.600	0.0495
1.053	0.1738	0.0782	0.600	0.0469
1.000	0.1652	0.0743	0.600	0.0446
0.909	0.1548	0.0697	0.600	0.0418
0.833	0.1460	0.0657	0.600	0.0394
0.769	0.1383	0.0622	0.600	0.0373
0.714	0.1315	0.0592	0.600	0.0355
0.667	0.1255	0.0565	0.600	0.0339
0.625	0.1201	0.0541	0.600	0.0324
0.588	0.1153	0.0519	0.600	0.0311
0.556	0.1109	0.0499	0.600	0.0299
0.526	0.1069	0.0481	0.600	0.0289
0.500	0.1033	0.0465	0.600	0.0279
0.455	0.0893	0.0402	0.600	0.0241
0.417	0.0781	0.0352	0.600	0.0211
0.385	0.0691	0.0311	0.600	0.0187
0.357	0.0617	0.0278	0.600	0.0167
0.333	0.0555	0.0250	0.600	0.0150
0.313	0.0505	0.0227	0.600	0.0136
0.294	0.0461	0.0207	0.600	0.0124
0.278	0.0425	0.0191	0.600	0.0115
0.263	0.0393	0.0177	0.600	0.0106
0.250	0.0365	0.0164	0.600	0.0099
0.238	0.0341	0.0153	0.600	0.0092
0.227	0.0319	0.0144	0.600	0.0086
0.217	0.0299	0.0135	0.600	0.0081
0.208	0.0282	0.0127	0.600	0.0076
0.200	0.0266	0.0120	0.600	0.0072
0.182	0.0232	0.0105	0.600	0.0063
0.167	0.0206	0.0092	0.600	0.0055
0.154	0.0183	0.0082	0.600	0.0049
0.143	0.0165	0.0074	0.600	0.0044
0.133	0.0149	0.0067	0.600	0.0040
0.125	0.0136	0.0061	0.600	0.0037
0.118	0.0124	0.0056	0.600	0.0033
0.111	0.0114	0.0051	0.600	0.0031
0.100	0.0097	0.0044	0.600	0.0026

Notes:

Hz = hertz

LNP COL 2.5-3

**Table 2.5.2-227 (Sheet 1 of 3)
LNP Site PBSRS at Elevation 51 ft. Scaled by 1.2121 consistent with Reactor Foundation Elevation SCOR FIRS Scaled to 0.1g Horizontal Peak Ground Acceleration**

Spectral Frequency (Hz)	5-percent Damped Spectral Acceleration (g)		
	Horizontal PBSRS	Vertical/Horizontal	Vertical PBSRS
100.000	0.1293	0.6950	0.0899
60.241	0.1535	0.7126	0.1094
50.000	0.1704	0.7920	0.1350
40.000	0.1829	0.8788	0.1607
33.333	0.1973	0.9363	0.1847
30.303	0.2030	0.9435	0.1915
25.000	0.2348	0.9005	0.2114
23.810	0.2427	0.8881	0.2155
22.727	0.2504	0.8697	0.2178
21.739	0.2580	0.8524	0.2200
20.833	0.2656	0.8362	0.2221
20.000	0.2730	0.8210	0.2241
18.182	0.2863	0.7838	0.2244
16.667	0.3025	0.7443	0.2251
15.385	0.3220	0.7130	0.2296
14.286	0.3308	0.6912	0.2286
13.333	0.3391	0.6721	0.2279
12.500	0.3452	0.6567	0.2267
11.765	0.3510	0.6425	0.2255
11.111	0.3566	0.6301	0.2247
10.526	0.3620	0.6230	0.2255
10.000	0.3644	0.6164	0.2246
9.091	0.3696	0.6029	0.2228
8.333	0.3745	0.5899	0.2209
7.692	0.3792	0.5809	0.2203
7.143	0.3836	0.5780	0.2217
6.667	0.3828	0.5752	0.2202
6.250	0.3811	0.5727	0.2182
5.882	0.3777	0.5703	0.2154
5.556	0.3746	0.5681	0.2128
5.263	0.3669	0.5660	0.2077
5.000	0.3597	0.5660	0.2036
4.545	0.3272	0.5660	0.1852
4.167	0.2982	0.5660	0.1688
3.846	0.2746	0.5660	0.1554
3.571	0.2552	0.5660	0.1445
3.333	0.2390	0.5678	0.1357
3.125	0.2256	0.5698	0.1285
2.941	0.2132	0.5717	0.1219

LNP COL 2.5-3

Table 2.5.2-227 (Sheet 2 of 3)
LNP Site PBSRS at Elevation 51 ft. Scaled by 1.2121 consistent with Reactor Foundation Elevation SCOR FIRS Scaled to 0.1g Horizontal Peak Ground Acceleration

Spectral Frequency (Hz)	5-percent Damped Spectral Acceleration (g)		
	Horizontal PBSRS	Vertical/Horizontal	Vertical PBSRS
2.778	0.2084	0.5736	0.1195
2.632	0.2040	0.5754	0.1173
2.500	0.1998	0.5776	0.1154
2.381	0.1959	0.5800	0.1136
2.273	0.1922	0.5823	0.1119
2.174	0.1888	0.5846	0.1104
2.083	0.1856	0.5870	0.1089
2.000	0.1825	0.5894	0.1076
1.818	0.1756	0.5951	0.1045
1.667	0.1695	0.5978	0.1013
1.538	0.1628	0.6022	0.0981
1.429	0.1548	0.6082	0.0941
1.333	0.1477	0.6122	0.0904
1.250	0.1381	0.6157	0.0850
1.176	0.1296	0.6190	0.0802
1.111	0.1221	0.6222	0.0760
1.053	0.1154	0.6252	0.0722
1.000	0.1094	0.6280	0.0687
0.909	0.1019	0.6292	0.0641
0.833	0.0955	0.6304	0.0602
0.769	0.0899	0.6314	0.0568
0.714	0.0851	0.6324	0.0538
0.667	0.0808	0.6332	0.0512
0.625	0.0770	0.6341	0.0488
0.588	0.0736	0.6349	0.0467
0.556	0.0705	0.6356	0.0448
0.526	0.0677	0.6363	0.0431
0.500	0.0652	0.6370	0.0415
0.455	0.0562	0.6370	0.0358
0.417	0.0490	0.6370	0.0312
0.385	0.0432	0.6370	0.0275
0.357	0.0385	0.6370	0.0245
0.333	0.0346	0.6370	0.0220
0.313	0.0313	0.6370	0.0200
0.294	0.0286	0.6370	0.0182
0.278	0.0263	0.6370	0.0167
0.263	0.0243	0.6370	0.0155
0.250	0.0225	0.6370	0.0144
0.238	0.0210	0.6370	0.0134

LNP COL 2.5-3

Table 2.5.2-227 (Sheet 3 of 3)
LNP Site PBSRS at Elevation 51 ft. Scaled by 1.2121 consistent with Reactor Foundation Elevation SCOR FIRS Scaled to 0.1g Horizontal Peak Ground Acceleration

Spectral Frequency (Hz)	5-percent Damped Spectral Acceleration (g)		
	Horizontal PBSRS	Vertical/Horizontal	Vertical PBSRS
0.227	0.0197	0.6370	0.0125
0.217	0.0184	0.6370	0.0117
0.208	0.0173	0.6370	0.0110
0.200	0.0164	0.6370	0.0104
0.182	0.0143	0.6370	0.0091
0.167	0.0126	0.6370	0.0080
0.154	0.0112	0.6370	0.0071
0.143	0.0100	0.6370	0.0064
0.133	0.0091	0.6370	0.0058
0.125	0.0082	0.6370	0.0053
0.118	0.0075	0.6370	0.0048
0.111	0.0069	0.6370	0.0044
0.100	0.0059	0.6370	0.0037

Notes:

FIRS = foundation input response spectra
g = unit of measure of acceleration of gravity
Hz = hertz
PBSRS = performance-based surface response spectra
SCOR = soil column outcropping response

LNP COL 2.5-3

Table 2.5.2-228
Best Estimate Properties for SSI Analyses of the LNP Site

Layer	Thickness (ft.)	Total Depth (ft.)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Damping Ratio (%)	Compression Wave Velocity (ft/sec)	Elevation of Layer Base (ft.)
1	2.5	2.5	110	836	1.3	1590	48.5
2	2.5	5	110	824	1.6	1590	46
3	2.5	7.5	110	796	2.0	1590	43.5
4	3.5	11	110	788	2.3	1590	40
5	2	13	110	796	2.4	5000	38
6	2	15	110	786	2.6	5000	36
7	3.5	18.5	120	1503	2.4	5600	32.5
8	2.5	21	120	1500	2.5	5600	30
9	1	22	120	1500	2.5	5600	29
10	3.5	25.5	120	1501	2.0	5600	25.5
11	3.5	29	120	1496	2.1	5600	22
12	6.9	35.9	120	1482	2.1	5600	15.1
13	4.1	40	120	1476	2.1	5600	11
14	2.8	42.8	120	1476	2.1	5600	8.2
15	8.4	51.2	130	2267	2.1	7550	-0.2
16	8.4	59.6	130	2266	2.1	7550	-8.6
17	7.1	66.7	130	2254	2.2	7550	-15.7
18	7.1	73.8	130	2251	2.2	7550	-22.8
19	1.2	75	138	2772	1.4	8700	-24
20	24.6	99.6	138	2772	1.4	8700	-48.6
21	47.4	147	138	2694	1.4	8550	-96
22	61.3	208.3	138	3374	1.4	10600	-157.3
23	17.9	226.2	138	3315	1.4	9450	-175.2
24	24.1	250.3	120	3243	1.9	7250	-199.3
25	24.6	274.9	120	3210	1.9	7250	-223.9
26	40	314.9	120	3539	1.3	7900	-263.9
27	42	356.9	120	3358	1.3	7900	-305.9
28	38.4	395.3	140	4144	0.9	8900	-344.3
29	59.4	454.7	140	3369	0.9	8100	-403.7
30	59.4	514.1	140	3721	0.9	9000	-463.1
31	242.7	756.8	140	4541	0.9	11000	-705.8
32	355.8	1112.6	140	5934	0.9	14400	-1061.6
33	249.4	1362	150	7294	0.7	17850	-1311
34	252.9	1614.9	150	5101	0.7	12350	-1563.9
35	148.3	1763.2	150	7279	0.7	17400	-1712.2
36	106.1	1869.3	150	6259	0.7	14900	-1818.3
37	199	2068.3	150	7168	0.7	17500	-2017.3
38	601.2	2669.5	150	5429	0.8	13000	-2618.5
39	149.2	2818.7	150	5955	0.8	14200	-2767.7
40	192.7	3011.4	150	6200	0.8	14950	-2960.4
41	652.3	3663.7	150	5168	0.8	12600	-3612.7
42	603.7	4267.4	150	5555	0.8	13450	-4216.4
43	96.6	4364	150	4800	0.8	11500	-4313
44	Half Space		169	9396	0.1	16100	

Notes:

% = percent; ft. = feet; ft/sec = feet per second; pcf = pound per cubic foot; SSI = soil structure interaction

LNP COL 2.5-3

Table 2.5.2-229
Lower Bound Properties for SSI Analyses of the LNP Site

Layer	Thickness (ft.)	Total Depth (ft.)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Damping Ratio (%)	Compression Wave Velocity (ft/sec)	Elevation of Layer Base (ft.)
1	2.5	2.5	110	373	2.6	935	48.5
2	2.5	5	110	342	4.4	935	46
3	2.5	7.5	110	315	5.8	935	43.5
4	3.5	11	110	300	6.8	935	40
5	2	13	110	301	7.3	5000	38
6	2	15	110	294	7.9	5000	36
7	3.5	18.5	120	1123	5.4	5000	32.5
8	2.5	21	120	1115	5.5	5000	30
9	1	22	120	1115	5.5	5000	29
10	3.5	25.5	120	1074	5.3	5000	25.5
11	3.5	29	120	1070	5.5	5000	22
12	6.7	35.7	120	1111	5.6	5000	15.3
13	4.3	40	120	1100	5.9	5000	11
14	2.4	42.4	120	1100	4.8	5000	8.6
15	8.3	50.7	130	1851	4.9	6165	0.3
16	8.3	59	130	1850	5.0	6165	-8
17	7.2	66.2	130	1841	5.1	6165	-15.2
18	7.2	73.4	130	1838	2.4	6165	-22.4
19	1.6	75	138	2264	2.4	7022	-24
20	24.2	99.2	138	2264	2.4	7022	-48.2
21	46.8	146	138	2199	2.4	6532	-95
22	61.5	207.5	138	2755	2.4	7634	-156.5
23	17.9	225.4	138	2707	2.4	6654	-174.4
24	23.9	249.3	120	2145	4.7	5920	-198.3
25	24.6	273.9	120	2148	4.7	5920	-222.9
26	40	313.9	120	2890	1.9	6450	-262.9
27	42.1	356	120	2742	1.9	6450	-305
28	38.3	394.3	140	3384	1.3	7267	-343.3
29	59.8	454.1	140	2750	1.3	6614	-403.1
30	61.1	515.2	140	3038	1.3	7348	-464.2
31	242.7	757.9	140	3708	1.3	8981	-706.9
32	354.8	1112.7	140	4845	1.3	11758	-1061.7
33	246.6	1359.3	150	5956	1.0	14574	-1308.3
34	255.7	1615	150	4165	1.0	10084	-1564
35	150.7	1765.7	150	5943	1.0	14207	-1714.7
36	100.8	1866.5	150	5111	1.0	12166	-1815.5
37	199.6	2066.1	150	5853	1.0	14289	-2015.1
38	600.3	2666.4	150	4432	1.2	10614	-2615.4
39	149.6	2816	150	4863	1.2	11594	-2765
40	199.2	3015.2	150	5062	1.2	12207	-2964.2
41	650.5	3665.7	150	4220	1.2	10288	-3614.7
42	597	4262.7	150	4535	1.2	10982	-4211.7
43	104.1	4366.8	150	3919	1.2	9390	-4315.8
44	Half Space		169	7672	0.1	13146	

Notes:

% = percent; ft. = feet; ft/sec = feet per second; pcf = pound per cubic foot; SSI = soil structure interaction

LNP COL 2.5-3

Table 2.5.2-230
Upper Bound Properties for SSI Analyses of the LNP Site

Layer	Thickness (ft.)	Total Depth (ft.)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Damping Ratio (%)	Compression Wave Velocity (ft/sec)	Elevation of Layer Base (ft.)
1	2.5	2.5	110	1280	0.9	1948	48.5
2	2.5	5	110	1275	1.1	1948	46
3	2.5	7.5	110	1291	1.2	1948	43.5
4	3.5	11	110	1287	1.3	1948	40
5	2	13	110	1273	1.4	5000	38
6	2	15	110	1266	1.5	5000	36
7	3.5	18.5	120	1982	1.1	7226	32.5
8	2.5	21	120	1980	1.2	7226	30
9	1	22	120	1980	1.2	7226	29
10	3.5	25.5	120	1931	0.6	7226	25.5
11	3.5	29	120	1931	0.6	7226	22
12	7.1	36.1	120	1906	0.6	7226	14.9
13	3.9	40	120	1902	5.9	7226	11
14	3.2	43.2	120	1902	0.6	7226	7.8
15	9	52.2	130	2993	0.5	9737	-1.2
16	9	61.2	130	2991	0.5	9737	-10.2
17	9.2	70.4	130	2887	0.5	9737	-19.4
18	4.6	75	130	2887	0.5	9737	-24
19	4.6	79.6	130	2887	0.5	9737	-28.6
20	20	99.6	138	4731	0.6	10655	-48.6
21	48.8	148.4	138	3984	0.6	10472	-97.4
22	51.9	200.3	138	5157	0.6	12982	-149.3
23	11.9	212.2	138	4356	0.6	11574	-161.2
24	27	239.2	120	3972	0.5	9308	-188.2
25	26.2	265.4	120	3975	0.6	9308	-214.4
26	35.1	300.5	120	4335	0.7	9798	-249.5
27	44.5	345	120	4112	0.7	9798	-294
28	44.7	389.7	140	5075	0.5	11329	-338.7
29	49.8	439.5	140	4126	0.5	10043	-388.5
30	72.6	512.1	140	4620	0.5	11023	-461.1
31	244.3	756.4	140	5562	0.5	13472	-705.4
32	356	1112.4	140	7267	0.5	17636	-1061.4
33	254.7	1367.1	150	8934	0.4	21862	-1316.1
34	244	1611.1	150	6247	0.4	15126	-1560.1
35	153.4	1764.5	150	8915	0.4	21311	-1713.5
36	96.4	1860.9	150	7666	0.4	18249	-1809.9
37	205.2	2066.1	150	8779	0.4	21433	-2015.1
38	601.3	2667.4	150	6649	0.4	15922	-2616.4
39	148	2815.4	150	7294	0.4	17391	-2764.4
40	198.5	3013.9	150	7593	0.4	18310	-2962.9
41	647.8	3661.7	150	6330	0.4	15432	-3610.7
42	602.2	4263.9	150	6803	0.4	16473	-4212.9
43	95.2	4359.1	150	5879	0.4	14085	-4308.1
44	Half Space		169	11507	0.1	19718	

Notes:

% = percent; ft. = feet; ft/sec = feet per second; pcf = pound per cubic foot; SSI = soil structure interaction

Table 2.5.2-231
Lower Lower Bound Properties for SSI Analyses of the LNP Site

LNP COL 2.5-3

Layer	Thickness (ft.)	Total Depth (ft.)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Damping Ratio (%)	Compression Wave Velocity (ft/sec)	Elevation of Layer Base (ft.)
1	2.5	2.5	110	373	2.6	935	48.5
2	2.5	5	110	342	4.4	935	46
3	2.5	7.5	110	315	5.8	935	43.5
4	3.5	11	110	300	6.8	935	40
5	2	13	110	301	7.3	5000	38
6	2	15	110	294	7.9	5000	36
7	3.5	18.5	120	1066	5.4	5000	32.5
8	*2.5*	*21*	*120*	1058	*5.5*	5000	30
9	1	22	120	1058	5.5	5000	29
10	3.5	25.5	120	1019	5.3	5000	25.5
11	3.5	29	120	1015	5.5	5000	22
12	6.7	35.7	120	1054	5.6	5000	15.3
13	*4.3*	*40*	*120*	1043	*5.9*	5000	11
14	2.4	42.4	120	1043	4.8	5000	8.6
15	8.3	50.7	130	1756	4.9	5848	0.3
16	8.3	59	130	1755	5.0	5848	-8
17	7.2	66.2	130	1746	5.1	5848	-15.2
18	7.2	73.4	130	1744	2.4	5848	-22.4
19	*1.6*	*75*	*138*	2147	*2.4*	6661	-24
20	24.2	99.2	138	2264	2.4	7022	-48.2
21	46.8	146	138	2199	2.4	6532	-95
22	61.5	207.5	138	2755	2.4	7634	-156.5
23	17.9	225.4	138	2707	2.4	6654	-174.4
24	23.9	249.3	120	2145	4.7	5920	-198.3
25	24.6	273.9	120	2148	4.7	5920	-222.9
26	40	313.9	120	2890	1.9	6450	-262.9
27	42.1	356	120	2742	1.9	6450	-305
28	38.3	394.3	140	3384	1.3	7267	-343.3
29	59.8	454.1	140	2750	1.3	6614	-403.1
30	61.1	515.2	140	3038	1.3	7348	-464.2
31	242.7	757.9	140	3708	1.3	8981	-706.9
32	354.8	1112.7	140	4845	1.3	11758	-1061.7
33	246.6	1359.3	150	5956	1.0	14574	-1308.3
34	255.7	1615	150	4165	1.0	10084	-1564
35	150.7	1765.7	150	5943	1.0	14207	-1714.7
36	100.8	1866.5	150	5111	1.0	12166	-1815.5
37	199.6	2066.1	150	5853	1.0	14289	-2015.1
38	600.3	2666.4	150	4432	1.2	10614	-2615.4
39	149.6	2816	150	4863	1.2	11594	-2765
40	199.2	3015.2	150	5062	1.2	12207	-2964.2
41	650.5	3665.7	150	4220	1.2	10288	-3614.7
42	597	4262.7	150	4535	1.2	10982	-4211.7
43	104.1	4366.8	150	3919	1.2	9390	-4315.8
44	Halfspace		169	7672	0.1	13146	

Notes:

% = percent; ft. = feet; ft/sec = feet per second; pcf = pound per cubic foot; SSI = soil structure interaction

2.5.3 SURFACE FAULTING

LNP COL 2.5-4

FSAR Subsection 2.5.3 contains an evaluation of the potential for tectonic and nontectonic surface deformation at the LNP site. Information contained in this subsection developed in accordance with the NRC's Regulatory Guide 1.208 is intended to demonstrate compliance with 10 CFR 100.23, Geologic and Seismic Siting Criteria.

This subsection describes the evidence used to evaluate the potential for tectonic surface deformation at the LNP site and surrounding site area. Information and site characterization activities conducted to evaluate tectonic deformation also pertain to the evaluation of nontectonic deformation, including subsidence and collapse due to karst development.

The conclusions regarding the potential for surface deformation are summarized as follows:

- There are no capable tectonic fault sources within the site area (8 km [5 mi.] radius) or vicinity (40 km [25 mi.] radius). There is no evidence of Quaternary tectonic surface faulting or fold deformation within the LNP site location (1 km [0.6 mi.] radius).
- The potential for nontectonic deformation at the site from phenomenon other than karst-related collapse or subsidence is negligible.
- The LNP site lies within a region susceptible to dissolution and karst development. The potential for surface deformation related to dissolution and karst formation at the LNP site will be mitigated through appropriate ground remediation and foundation design measures.

2.5.3.1 Geological, Seismological, and Geophysical Investigations

Published information and other available data for the site area that provide a framework for evaluating tectonic features and karst development in the site vicinity are summarized in FSAR Subsections 2.5.1.2.4 and 2.5.1.2.1.3, respectively.

Based on a review of available geologic data, there are no documented Quaternary tectonic faults in the site region (within a 320 km [200 mi.] radius) (Figure 2.5.1-224). Refer to FSAR Subsection 2.5.1.1 for additional details. The USGS completed a compilation of all Quaternary faults, liquefaction features, and possible tectonic features in the central and eastern United States, including the Florida peninsula region. (References 2.5.3-201 and 2.5.3-202) These compilations did not show any Quaternary tectonic faults or tectonic features within the site vicinity or Class C features (i.e., those for which geologic evidence is insufficient to demonstrate the existence of a tectonic fault, Quaternary slip, or deformation associated with the feature) (FSAR Subsection 2.5.1.1.4.3.5).

Postulated faults within the site vicinity and site area identified by Vernon (Reference 2.5.3-203) are not classified in the USGS compilation. As noted below, there is no well-documented evidence that these faults exist or that they are capable tectonic sources.

The faults mapped by Vernon (Reference 2.5.3-203) are referred to in the FSAR for the CR3 site (Reference 2.5.3-204); the westernmost extension of the mapped faulting lies 4.8 km (3 mi.) east of the CREC. The CR3 FSAR provides no documentation of studies to evaluate the existence or capability of these faults. Based on photogeologic studies and subsurface explorations (test borings and a seismic refraction survey), it was concluded in the CR3 FSAR that no faults are present beneath the CR3 site. (Reference 2.5.3-204) The results of engineering geology investigations of the foundation rock system, confirmed by construction observations, revealed that the entire foundation system of the CR3 plant contains near-vertically oriented fracture zones, but did not identify any faults. (Reference 2.5.3-204)

Investigations that have been performed to evaluate the existence of the postulated faults within the LNP site area and the potential for surface fault rupture at the LNP site, as well as the surrounding LNP site area, include the following:

- Compilation and review of existing data and literature.

- Lineament analyses based on interpretation of aerial photography and remote sensing imagery. Investigations involved interpretation of aerial photographs (1949 black and white, 1:20,000 scale; 2007 color, 1:7920 scale); Landsat imagery; and LIDAR data (Reference 2.5.3-205) collected for the LNP COLA study.

- Discussions with current researchers in the area. Researchers were contacted who were familiar with the structural and tectonic framework of the region, carbonate stratigraphy, and post-Cretaceous faulting in the carbonate platform. Thomas M. Scott, Ph.D., P.G., Assistant State Geologist, reviewed geologic cores collected at the LNP site.

- Field reconnaissance. Field investigations focused on (1) a review of the geology of the site location (within approximately 1 km [0.6 mi.] of the LNP site) and site area (within a radius of approximately 8 km [5 mi.]); and (2) reconnaissance of localities of reported Cenozoic faulting and postulated features suggestive of possible neotectonic activity in the site area (e.g., the Inverness fault) and the surrounding site vicinity (e.g., the Long Pond fault).

- Review of seismicity data (FSAR Subsection 2.5.2.1).

In addition to these investigations, boring logs, core photos, surface geophysical testing, downhole geophysical logging, and downhole seismic testing information collected as part of the site investigation program, as described in FSAR Subsection 2.5.4.2, were evaluated to identify subsurface evidence for dissolution features. Dr. Anthony F. Randazzo, Ph.D., P.G. of GEOHAZARDS, Inc., (Emeritus Professor of Geology at the University of Florida) and Dr. Thomas M. Scott (Ph.D, P.G., former Assistant State Geologist, currently Senior Principal Geologist, SDII Global Corporation, Tampa Bay, Florida) assisted in the interpretation and review of these data.

2.5.3.2 Geological Evidence, or Absence of Evidence, for Surface Deformation

Recent geologic maps and evaluations of subsurface data for hydrostratigraphic analysis of the Floridan aquifer system do not show any structural features within the LNP site area (8 km [5 mi.] radius) (FSAR Subsection 2.5.1.2.4). (References 2.5.3-206 and 2.5.3-217) In an older publication (Reference 2.5.3-203), seven faults were identified within the Citrus and Levy Counties area; three of these were identified within the LNP site area (Figure 2.5.3-201). The three postulated faults located in the site area are the Inverness fault and two unnamed faults (designated as A and B on Figure 2.5.3-201).

The northern end of the postulated Inverness fault is located 2 km (1.2 mi.) east of the LNP site. Vernon's (Reference 2.5.3-203) field evidence for the Inverness fault is based in part on outcrops of the Inglis member of the Moodys Branch formation located east of the fault, along Tsala Apopka Lake (approximately 27 km [17 mi.]) southeast of the LNP site. These exposures lie at elevations of +8 to +15 m (+28 to +50 ft.) amsl, whereas five wells, W-874, W-1767, W-1791, W-1847 and W-1848, located approximately 3 km (2 mi.) to the southwest of the fault indicate that the Inglis member lies at elevations ranging from –0.3 m (–1 ft.) in the south to +11 m (+37 ft.) amsl in the north. Vernon (Reference 2.5.3-203) stated that numerous exposures of the Williston member of the Moodys Branch Formation, the Ocala Limestone (restricted), and the Suwannee Limestone on the hills southwest of the fault indicate comparable displacements. Based on this field evidence, Vernon (Reference 2.5.3-203) concluded that the northeast block had been tilted in faulting, the southeastern portion being upthrown with a displacement as much as 15 m (50 ft.), whereas the northwest portion is downthrown with displacements of as much as 6 m (20 ft.).

Unnamed postulated faults (designated A and B) are located approximately 4 km (2.5 mi.) and 7 km (4.3 mi.) southwest and northeast of the LNP site, respectively. These postulated faults were not specifically described or discussed by Vernon. (Reference 2.5.3-203) The only information regarding the sense of displacement is that shown on the Vernon geologic map of the Citrus and Levy Counties area. (Reference 2.5.3-203)

More recent studies and information reviewed for this study do not provide any evidence of these postulated faults (References 2.5.3-217, 2.5.3-218, 2.5.3-219, 2.5.3-220, and 2.5.3-221) and the postulated faults could not be identified on any aerial photographs, Landsat imagery, or LIDAR data sets (see discussion below).

In personal communications Thomas Scott states that he and other geologists from the FGS interpret the structural features (slickensides and tilted bedding) that Vernon (Reference 2.5.3-203) cites as evidence of surface faulting for postulated faults outside the site area to be probable nontectonic surface deformation related to karst collapse.

Vernon's (Reference 2.5.3-203) interpretation of the faults is further questioned when current stratigraphic interpretation is used to evaluate his apparent vertical displacements inferred from lithologic correlation across the faults. The stratigraphic correlation of Vernon (Reference 2.5.3-203), which suggested vertical offset across the postulated faults, follows the nomenclature originally established by Applin and Applin (Reference 2.5.3-207), who subdivided the Ocala Limestone into two different rock types. (Reference 2.5.3-206) This interpretation was later revised by Puri (Reference 2.5.3-208), who interpreted the Ocala Limestone as a group consisting of, in ascending order, the Inglis, Williston, and Crystal River formations (Reference 2.5.3-206). Miller (Reference 2.5.3-206), however, states that Puri's three formations cannot be recognized lithologically even at their type sections and cannot be differentiated in the subsurface. Therefore, Miller (Reference 2.5.3-206) does not consider the Inglis, Williston, and Crystal River formations to be either readily recognizable or mappable. Scott (Reference 2.5.3-222) reduced the Ocala Group to a formation to meet the requirement of the North American Code.

Today these units are considered to be part of the Upper Eocene Ocala Limestone, which is shown as a single unit in the geologic column for the Florida platform, although the limestone consists of two undifferentiated units (References 2.5.3-209 and 2.5.3-207) (Figure 2.5.1-214). Therefore, based on limited outcrop exposures, limited core data, and recent stratigraphic interpretation that the Inglis and Williston units cannot be differentiated in the subsurface, the displacement of stratigraphic units proposed by Vernon (Reference 2.5.3-203) to identify subsurface faults in the site area is highly speculative.

Scott (Reference 2.5.3-210) notes that many of the postulated faults in the state have been identified as offsets in the top of the Ocala Limestone, a karstified, unconformable surface that may have 50 m (164 ft.) or more of relief. Based on this lithology, Scott (Reference 2.5.3-210) surmised that it is very difficult to identify faulting in the extremely heterogeneous Neogene sediments, especially with incomplete cores, rock cuttings, and surface outcrops.

The existence of the postulated faults is based primarily on inferred correlation and offsets of stratigraphic units between widely spaced bedrock exposures and well data. A comprehensive study of the hydrostratigraphy of the SWFWMD by Arthur et al (Reference 2.5.3-217) using the most up-to-date database of lithologic information available at the FGS does not recognize evidence for the faults postulated by Vernon. Maps of the Ocala Limestone and Avon Park Formation surfaces that were constructed using what data were available to Vernon (Reference 2.5.3-203), and a significant amount of data unavailable in the late 1940s and early 1950s do not show the postulated Vernon faults (Reference 2.5.3-217).

There are no known localities where the faults can be observed. Very limited exposures of bedrock are present in the site area; the most continuous bedrock identified during field reconnaissance was in exposures along the Cross Florida Barge Canal (CFBC) and along the Withlacoochee River (Figure 2.5.1-201). The postulated Inverness fault crosses the Withlacoochee River upstream of the dam, where the gentle slopes adjacent to the reservoir do not provide any exposures of bedrock or overlying marine terrace deposits. The postulated faults in the site area project across marine terraces that are estimated to range from late Pleistocene to middle to early Pleistocene or possibly Pliocene age (FSAR Subsection 2.5.1.2.1.2). Based on interpretation of aerial photographs and LIDAR data described below, there is no geomorphic expression of the postulated faults across these marine terrace surfaces. There is no evidence to suggest that the postulated structures are faults, or if they are faults, that they have been active in the Quaternary.

Surface morphology and subsurface data indicate that there has been a long period of erosion and karst development in the site location (FSAR Subsection 2.5.1.2.1.3). The LNP site surface morphology is consistent with that of an eroded, older (paleo) karst landscape mantled by several feet to tens of feet of sand (i.e., a mantled epikarst subsurface formed over a denuded karst). The presence of thick Quaternary sand and locally mixed sand and organic sediment identified in some site borings suggests that paleosinks or buried paleochannels may be present in the LNP site location (1 km [0.6 mi.]). Site borings, however, encountered very few voids in the upper 150+ m (500 ft.) of the Avon Park Formation (see FSAR Subsection 2.5.4). With the exception of a small surface sinkhole that formed in response to drilling at one borehole, no sinkholes at the land surface were observed during site investigations and reconnaissance within the LNP site.

2.5.3.2.1 Results of Lineament Analysis

A lineament analysis was undertaken as part of the LNP study to identify and characterize lineaments in the site area that might intersect the LNP site. The lineament analysis involved a review of observations and conclusions of previous lineament analyses, and interpretation of aerial photographs and other remote sensing techniques, including Landsat imagery and LIDAR data.

Photolineaments in an aerial photograph or satellite image are helpful in defining potential fracture systems (faults and joints) that may lead to the mapping or understanding of groundwater flow, sinkhole development, and mineral deposits. The expression of a fracture as a lineament in unconsolidated sediments overlying competent rock may result from a number of nontectonic factors as outlined by Upchurch (Reference 2.5.3-216):

- Settlement of unconsolidated sediments into solution-enlarged fractures in the underlying, consolidated strata.

- Differential weathering, illuviation, or erosion caused by groundwater movement across karst surfaces.
- Differential consolidation of sediments into relict erosional features preserved in underlying unconformity surfaces.
- Growth of vegetation in clay- or silt-rich, moisture-holding soils located over somewhat deeper bedrock features associated with fractures.

Mapping features typically used to delineate lineaments include: (1) alignments and elongations of three or more depressions (potential sinkholes; sinkhole lakes); (2) alignments of soil tones; (3) alignments of vegetation patterns; and (4) alignments of stream segments and stream valleys. It should be noted, however, that the term "photolineaments," describes an unconfirmed alignment of features that may, or may not, reflect an underlying fracture. Photolineaments also can be caused by spurious alignments of naturally occurring features, livestock trails, human activities, and image processing. Until an underlying fracture has been confirmed, the term photolineament should be used in lieu of a fracture trace (Reference 2.5.3-216).

2.5.3.2.1.1 Previous Lineament Analyses

Table 2.5.3-201 presents a summary of key observations and conclusions regarding the orientations of fractures interpreted from previous lineament analyses conducted in the LNP site region and from site characterization studies conducted for the LNP COLA.

Vernon (Reference 2.5.3-203) mapped lineaments from physiographic expressions as shown on mosaics and contact prints of aerial photographs and interpreted them as faults and fractures. Figure 2.5.3-202 shows the distribution of lineaments in the northern portion of the Florida Peninsula that Vernon (Reference 2.5.3-203) interpreted as fractures that formed under a combination of tensional stresses related to anticlinal flexure during the development of the postulated Ocala Uplift. Vernon states that "Judging from their distribution and alignment with the Ocala uplift the fractures are associated with land movements of a regional nature and the stresses forming the folds of the Ocala uplift created these joints and faults. The compressive stresses effective all along the arch would tend to concentrate shearing parallel to the axis of the fold (Reference 2.5.3-203)."

Vernon (Reference 2.5.3-203) concludes that a (primary) system of fractures (joints) trend generally northwest-southeast, a "secondary system" trends northeast-southwest and intersects the primary system at large angles and are irregularly spaced. Further, he defines two "well-developed patterns" of fractures in localized areas within the region that deviate from the two systems and are more irregular. Vernon's definition of prominent fractures, while not clearly stated, implies that the "primary" fracture set is the most abundant lineament in a given orientation relative to the "secondary" fracture set. Dip orientation was not discussed for the fractures, but Vernon did state that the postulated faults were steeply dipping.

The CR3 FSAR (Reference 2.5.3-224) states that fracturing of the rock in response to the Ocala Uplift and consolidation of the thick sequence of Cretaceous and Tertiary sediments over a stable, competent Paleozoic basement, has produced conspicuous systematic conjugate joint sets, composed of nearly vertical and 45-degree dipping joints. The "primary conjugate" joint set is oriented both parallel and perpendicular to the Ocala Uplift (i.e., northwest and northeast trending), whereas the second primary conjugate joint set is oriented in approximately north-south and east-west trends. Two "secondary conjugate" fracture systems also were noted and assumed to be the result of stress adjustment to the principal fracture systems. The use of "conjugate joint sets" implies that the fractures have formed contemporaneously, but the CR3 FSAR does not provide timing constraints or detailed information with which to evaluate the structural relationships among the various fracture sets. Therefore, the joint orientations should be referred to as "orthogonal" rather than "conjugate" fracture sets. The CR3 FSAR provides no detailed maps, rose diagrams, or plots to support the general conclusions cited regarding the orientations of fractures observed in the foundation excavations during construction. No information on fracture spacing or the criteria used to distinguish between major (primary) and minor (secondary) fractures is provided in the CR3 FSAR or supporting documents.

Other site-specific studies have attempted to characterize fracture systems based on photolineament analysis. Figure 2.5.3-221 presents a rose diagram showing the frequency and orientations of photolineaments at a wellfield in Pasco County, Florida (approximately 85 km (53 mi.), from the LNP site. Upchurch (Reference 2.5.3-216) attributes the bimodal pattern to effects of tidal stresses on joint development in rocks of Florida. The two sets of fractures exhibited in this diagram (approximately N45°W and N50°E; north-south and east-west) are similar to the two primary joint sets identified at the CR3 site.

Upchurch (Reference 2.5.3-216) also discusses the results of a study completed within the Tampa Bay Regional Reservoir (approximately 130 km [82 mi.] south of the LNP site) in which photolineaments were 'ground truthed' using ground penetrating radar, refraction and reflection seismic profiling, and standard penetration test drilling. The results showed that almost 60 percent of the photolineaments could not be confirmed and that about 42 percent were either shortened or lengthened. Only about 6 percent of the photolineaments were confirmed as fracture traces without modification. Upchurch notes that since Florida limestones and dolostones are geologically immature, photolinears are not regularly spaced relative to the Paleozoic and Mesozoic limestones. The textural and lithification heterogeneity in the limestone/dolostone is too great for fractures to develop and propagate uniformly over large areas (100+ acres). The false fractures were predominantly influenced by a paleosol and relict drainage pattern buried under the marine terrace sand (Reference 2.5.3-225).

Vernon (Reference 2.5.3-203) observed that the regional fracture pattern is consistent with some stream patterns and sinkhole alignments. Particularly well-developed joints or faults (as interpreted by Vernon [Reference 2.5.3-203]) are shown along portions of the Ocklawaha, Withlacoochee, and Kissimmee Rivers, all of which show strongly developed rectangular trends northeast-southwest and northwest-southeast with large angle turns. (Reference 2.5.3-203)

The system of fractures that Vernon (Reference 2.5.3-203) originally mapped in Citrus and Levy Counties is observed throughout the state, as shown by a figure created in 1973 by the Florida Department of Transportation (DOT) (Reference 2.5.3-211) (Figure 2.5.3-203). The distribution of lineaments throughout the Florida Peninsula is relatively consistent and extensive, making it difficult to identify a single structure, such as the Ocala platform, that is responsible for regional lineament orientations. General regional trends (statewide) are relatively consistent in northeast-southwest, northwest-southeast, north northeast-south southwest, north-northwest-south-southeast orientations with some variability. Upchurch indicates that a bimodal pattern of photolinears (recorded in northwest and northeast orientations) is ubiquitous throughout the State and likely is caused by tidal stresses (Reference 2.5.3-216). Differential subsidence or compaction of sediments on the flanks of the Ocala platform or broad epeiorgenic warping related to dissolution of carbonate, however, cannot be precluded as mechanisms for formation of fractures within the site vicinity. Arthur et al. (Reference 2.5.3-217) citing previous studies suggests regional lineaments (drainage patterns) in the southern part of the SWFWMD may have been influenced by fractures formed in response to peripheral Miocene-Pliocene stress fields associated with Caribbean tectonics.

Mapped lineaments by Vernon (Reference 2.5.3-203) and the DOT (Reference 2.5.3-211) and rose diagrams showing the frequency and orientation of lineaments observed in these studies for the site vicinity (40 km [25 mi.]) are shown on Figure 2.5.3-222 and Figure 2.5.3-223, respectively. Vernon (Figure 2.5.3-202) mapped 39 lineaments in the site vicinity, whereas the more recent map from the DOT (Figure 2.5.3-203) includes 48 lineaments within the site vicinity (40 km [25 mi.]). Major populations of lineaments (most frequent) mapped by Vernon occur at N42°W and N38°W, with very minor occurrences at N56E. Major (most frequent) populations of lineaments mapped by the DOT strike N47°E, with slightly less abundant populations at N52°E, N72°E, and N57°W. Minor (less frequent) populations have orientations that strike N20°W, N25°E, and N72°W. Lineaments within the LNP site vicinity (40 km [25 mi.]) from the DOT statewide map tend to include similar (N20°W, N57°W), but not exact (N47°W) orientations mapped by Vernon. Vernon states that in mapping the (postulated) joint pattern an attempt was made to include all irregular trends whether poorly marked or well-developed, and to include only the well-developed trends running northwest-southeast or at right angles to the set (Reference 2.5.3-203). The inconsistencies in the results of the two studies highlight the subjective nature and uncertainties inherent in identifying structural trends solely from photolineament analysis.

Culbreth (Reference 2.5.3-212) completed a series of gravity profiles across selected lineaments in south Florida to determine if lineaments represent surface manifestations of basement structures. The profiles led Culbreth to make the following observations and conclusions:

- Some lineaments are associated with a gravity anomaly (density gradient) that can be modeled as a geologic feature in the sub-Zuni basement, indicating that some lineaments may be surface manifestations of basement features.
- Other nearby lineaments do not show a gravity signature, indicating that not all lineaments represent basement structures that can be detected by density gradients.
- Structures that are not characterized by density contrasts, such as fractures within crustal blocks and "strike-slip" faults between blocks of the same density, may underlie some lineaments.
- Gravity anomalies were observed in some profiles that do not correspond to any mapped lineaments.
- Lineaments may also reflect changes in stratigraphy or be the result of other surface or near-surface processes.
- Surface characteristics, such as geomorphology and cultural intensity, may influence the recognition of lineaments at the surface.

Culbreth suggests that a possible explanation of the manifestation of basement structures overlain by 2 to 5 km of sedimentary rock as lineaments at the present ground is that lineaments are propagated upward through overlying material through stresses induced by Earth tides. This hypothesis assumes that a localized, increased response to Earth-tide forces at discontinuities or zones of decreased rigidity, such as along faults or other vertical discontinuities in the basement, creates stresses in the overlying rock that leads to fracturing. Zones of fractures are created, and then enhanced by diagenetic processes, resulting in increased weathering and possible faulting. This process results in the manifestation of a subsurface feature through overlying sediments.

Culbreth (Reference 2.5.3-212) identified four factors that may affect lineament distribution and density. The four factors include the (1) type, scale, and resolution of the imagery; (2) techniques used for mapping; (3) prevalence of cultural features; and (4) geomorphology of the study area. The impact of the first two features on lineament identification is predictable, whereas the impact of cultural features and geomorphology can complicate the interpretation of the lineament analysis. In areas where there is a low lineament density, there is commonly high urban development. The urban development alters the landscape and obscures features used to identify lineaments. (Reference 2.5.3-212)

The effect of geomorphology on lineament density is also directly related to the amount of topographic relief for the area. (Reference 2.5.3-212) In areas characterized by multiple marine terraces and well-developed drainage patterns, headward erosion across the marine terraces and channel development is enhanced in zones of weakness. Evidence of lineament control on erosion and channel development is supported by the nearly rectilinear drainage patterns observed in many streams through the area. (Reference 2.5.3-212) In areas characterized by parallel, shallow, swampy depressions between beach ridges, linear features observed on satellite images and air photos are a result of the beach ridges rather than manifestations of fractures in bedrock that have propagated upward through the sediments. The beach ridges have imparted a fabric to the area that hinders the identification of lineaments.

2.5.3.2.1.2 Evaluation of Previously Mapped Structures and Lineaments in the Site Area (8 km [5 mi.] Radius)

Faults and fractures inferred from previous studies and lineament analyses by Vernon (Reference 2.5.3-203) and the DOT (Reference 2.5.3-211) were evaluated using:

- 2000 Landsat data (Figure 2.5.3-204, uninterpreted; Figure 2.5.3-205 and Figure 2.5.3-206, Vernon and DOT interpretations, respectively).
- 1949 aerial photograph mosaic (Figure 2.5.3-207, uninterpreted; Figure 2.5.3-208 and Figure 2.5.3-209, Vernon and DOT interpretations, respectively).
- 10 m (32.8 ft.) USGS National Elevation Dataset-Digital Elevation Model (NED DEM) data; (Figure 2.5.3-210, uninterpreted; Figure 2.5.3-211 and Figure 2.5.3-212, Vernon and DOT interpretations, respectively).
- High-resolution DEM developed from LIDAR data acquired in 2007 (Figure 2.5.3-213 and Figure 2.5.3-215, uninterpreted; Figure 2.5.3-214 Vernon and DOT interpretations).

The postulated faults of Vernon (Reference 2.5.3-203) that intersect the site area (FSAR Subsection 2.5.1.2.4) are not apparent in any of these data sets. There are no strong tonal lineaments, alignments of water bodies, or continuous topographic anomalies along any of the postulated faults. Unnamed fault B, which intersects the northeastern part of the site area, coincides with a DOT regional lineament, but this lineament is not more strongly expressed in the Landsat data, 1949 mosaic, or hillshade relief map derived from the LIDAR data than other lineaments that are interpreted to be fractures. A broad, northwest-trending topographic low area marked by greater stream incision is present in the eastern part of the study area north of the Withlacoochee River; linear features within this broad zone coincide in part with fracture trends/lineaments identified by both Vernon (Reference 2.5.3-203) and the DOT

(Reference 2.5.3-211) (Figure 2.5.3-211 and Figure 2.5.3-212). The general elevation of the geomorphic surface on either side of the depression is similar, and there are no systematic steps across individual linear features within this zone to suggest surface fault displacement. It is likely that this zone represents a zone of greater dissolution localized along fracture trends that has been enhanced by fluvial incision and channel erosion during paleo sea level highstands.

2.5.3.2.1.3 Site Location (1 km [0.6 mi.] Radius) Lineament Analysis

None of the previously mapped regional lineaments intersect the site location (Figure 2.5.3-213 and Figure 2.5.3-214). There is no topographic expression of the Inverness or unnamed fault B in the hillshade relief map derived from the LIDAR data that covers parts of these postulated structures.

The detailed topographic DEM provided by the LIDAR data was used to map small-scale linear topographic breaks and features. To further identify and evaluate lineaments that may be present within the LNP site location, 1949 1:20,000-scale aerial photograph stereo pairs covering the site location were reviewed. The 1949 aerial photography was used because this photography predates much of the logging activity that appears to have modified the natural surface morphology of the site.

Examination of detailed topographic maps derived from the LIDAR data (Figure 2.5.3-215 and Figure 2.5.3-216) shows smaller-scale topographic lineaments in the site location that form a rectilinear pattern consistent with major and minor bedrock orthogonal joint trends mapped in the site vicinity. Diagrams showing the orientation of lineaments identified in the LNP site location (1-km [0.6-mi.] radius) and orientations of fracture sets reported from mapping of the CR3 excavation are shown on Figure 2.5.3-224. Comparison of these data sets shows that the orientation of the lineaments observed in the site location are consistent with the mapped bedrock fracture (joint) trends at CR3. The most prominent trends observed in the lineament data also are consistent with observations of fracture sets mapped in exposures of the Avon Park Formation at the Gulf Hammock quarry and Wacasassa River localities discussed in FSAR Subsection 2.5.1.2.4.1 (Figures 2.5.3-222 and 2.5.3-223). The prominent northwest-trending alignment of shallow depressions/wetlands identified approximately 300 m (1000 ft.) west of the LNP unit footprints (Figures 2.5.3-216, 2.5.3-218, and 2.5.3-220) is consistent with the predominant fracture set mapped by Vernon (Reference 2.5.3-203) based on analysis of regional lineaments and with one of the predominant orthogonal fracture sets mapped in exposures of the Avon Park Formation at the Gulf Hammock quarry and Wacasassa River localities (Figures 2.5.3-222 and 2.5.3-223).

The lineaments appear to be better expressed in the eastern part of the site location where there is slightly greater relief (up to about 1.5 m [5 ft.]) between the higher areas and topographic lows than in the western part of the site location, where maximum relief is about 0.6 to 1 m (2 to 3 ft.). Alignments of circular shallow depressions that are associated with wetlands and cypress domes (Figure 2.5.3-217, Figure 2.5.3-218, Figure 2.5.3-219, and Figure 2.5.3-220) also appear to follow the trends of major orthogonal joint sets. The small-scale linear topographic features identified from the LIDAR topographic data set generally are not apparent in the aerial photograph except where they define the margins of lower wetland areas characterized by different vegetation.

There are no mapped lineaments that cross the LNP 1 and LNP 2 footprint areas. The sites are located in a relatively low-lineament area between zones of more prominent northwest-trending lineaments. A zone of northeast-trending lineaments lies between the two units. The rectilinear margins of the slight topographic high and low areas within the site location have a similar appearance to the patterns of the tidal zone marsh islands observed in the Landsat image of the coastline in the site area (Figure 2.5.3-204). The linear features mapped in the site location are interpreted to be due to differential carbonate dissolution localized along joints and enhanced by marine erosion during previous sea level highstands.

2.5.3.3 Correlation of Earthquakes with Capable Tectonic Sources

A discussion of the updated earthquake catalog developed for the LNP COL application study is presented in FSAR Subsection 2.5.2.1. There are no recorded earthquakes larger than m_b = 3.0 within the LNP site vicinity (40 km [25 mi.]). There are no historically reported earthquakes or alignments of earthquakes in the surrounding site region (320 km [200 mi.] radius) that can be associated with a mapped bedrock fault (FSAR Subsection 2.5.1.1.4.4).

2.5.3.4 Ages of Most Recent Deformations

The most recent period of bedrock deformation in the site vicinity probably occurred during the Mesozoic and is related to rifting that led to development of the Gulf of Mexico basin and Atlantic Ocean (FSAR Subsection 2.5.1.1.2). Basement faults in the site area as mapped by Barnett (Reference 2.5.3-226) are not shown to displace the pre-Zuni surface and thus are pre- Middle Jurassic in age. Basement rock underlying the Florida platform was subsequently buried by Mesozoic and Cenozoic marine deposits and younger siliceous deposits, and it is presently at a depth of approximately 1330 m (4377 ft.) beneath the LNP site. There is no well-documented evidence of faulting in the late Cretaceous and Cenozoic section overlying the basement in the site vicinity (FSAR Subsections 2.5.1.1.4.3.4, 2.5.1.1.4.3.5, and 2.5.1.2.4). Postulated faults of Vernon (Reference 2.5.3-203) are inferred to displace the Avon Park Formation and Ocala Limestone units of Eocene age, but this has not been confirmed by more recent studies. Maps recently developed for the FGS by Arthur et al. (Reference 2.5.3-217) are based on the most current lithologic information available. The Arthur et al. study incorporated information from mapping of surface geology with interpretation of subsurface information, primarily water well and petroleum exploration well data, to develop structure contour maps on various datums. Discontinuities or anomalies that would suggest displacements of these surfaces by faulting were not identified in the LNP site vicinity. Maps of the top of the Ocala Limestone and the Avon Park Formation show no faults.

The LNP site is located on a marine terrace that is estimated to be older than 340,000 years, possibly of early Pleistocene to late Pliocene age (see discussion in FSAR Subsection 2.5.1.2.1.2). There is no geomorphic evidence to suggest that the bedrock surface (marine plantation surface) underlying the Quaternary terrace cover deposits in the site location has been displaced or deformed by tectonic faulting. The nearly horizontal terrace surface generally exhibits only minor relief that may be the result of differential erosion along tidal channels during the development of the marine terrace platform or from subsequent dissolution and surface karst development. There are no pronounced lineaments across the site location that suggest the presence of a through going fault.

2.5.3.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures

There are no documented bedrock faults of Cenozoic or younger age within the site vicinity (40 km [25 mi.] radius) (FSAR Subsection 2.5.1.2.4). Postulated faults and fracture trends identified by Vernon (Reference 2.5.3-203) are subparallel to regional joint and fracture trends that are observed throughout the State of Florida. Joint trends inferred from small-scale topographic lineaments and alignments of wetlands and cypress heads in the site location are consistent with joint trends inferred from regional lineament analyses and major and minor orthogonal joint sets observed in the excavation for the CR3 plant (FSAR Subsection 2.5.3.2.1.3). Postulated pre-Middle Jurassic basement faults inferred from sparse well control that are mapped in the site area by Barnett (Reference 2.5.3-226) are very speculative and not well constrained by the available data. These faults, if they exist, likely formed during the Mesozoic and do not appear to have displaced the sub-Zuni erosional surface or younger Eocene deposits. (see discussion in FSAR Subsection 2.5.1.2.4.2).

2.5.3.6 Characterization of Capable Tectonic Sources

A “capable tectonic source,” as defined by Regulatory Guide 1.208, is described by at least one of the following characteristics:

- 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.

- A reasonable association with one or more large earthquakes or sustained earthquake activity that usually is accompanied by significant surface deformation.

- Structural association with a capable tectonic source having characteristics of either of the two items above, such that movement on one could be reasonably expected to be accompanied by movement on the other.

There are no capable tectonic sources within a 40 km (25 mi.) radius of the LNP site. The existence of the postulated faults of Vernon (Reference 2.5.3-203) is not supported by available data, and there is no evidence of Quaternary activity associated with the features. Additional observations that indicate the absence of capable tectonic sources in the LNP site location are: (1) discontinuous expression of the lineaments in the site location, which appear to be joints along which there may have been preferential dissolution and weathering: (2) the low relief exhibited by the marine terrace surface at the LNP site and lack of apparent vertical displacement of the general terrace surface across mapped lineaments in the site area: and (3) the absence of evidence of surface faulting observed in borings or from interpretation of boring data at the LNP site.

2.5.3.7 Designation of Zones of Quaternary Deformation in the Site Region

Based on the above data and information summarized in FSAR Subsection 2.5.1.1.4, no zones of Quaternary tectonic deformation that would require additional investigation are identified within the LNP site region (320-km [200 mi.] radius). Review of available data and subsurface investigations conducted for this study identified no evidence for tectonic surface deformation at either LNP 1 or LNP 2, or elsewhere in the site area (8 km [5 mi.] radius). Refer to FSAR Subsection 2.5.1.2 for additional information on site geology.

2.5.3.8 Potential for Surface Deformation at the Site

2.5.3.8.1 Potential for Tectonic Surface Deformation at the Site

Based on the above data, the potential for tectonic deformation at the LNP site is negligible as there are no capable tectonic faults or geomorphic features indicative of Quaternary deformation within the LNP site area (8 km [5 mi.] radius).

Excavations for all safety-related structures for LNP 1 and LNP 2 will be mapped in detail, and the NRC will be notified immediately if previously unknown geologic features are identified that could represent a hazard to the facilities. Following Regulatory Guide 1.208, any potential deformation feature identified in the excavations will be characterized to assess surface deformation or ground motion generating potential.

2.5.3.8.2 Potential for Nontectonic Surface Deformation at the Site

2.5.3.8.2.1 Potential for Nontectonic Surface Deformation (Non-Karst Related)

The potential for nontectonic deformation at the site from phenomenon other than karst-related collapse or subsidence is negligible. There is no evidence of nontectonic deformation at the LNP site in the form of glacially induced faulting, post-Mesozoic volcanic intrusion, salt migration, or growth faulting. Based on a review of geologic literature, the site region (320 km [200 mi.] radius) has not experienced glacial or periglacial conditions. There is no documented intrusive or extrusive volcanic activity of Tertiary age within the site region (320 km [200 mi.] radius). Diabase observed in a well (Robinson No. 1 well) approximately 500 m (1640 ft.) north of the site, which underlies a sequence of Cretaceous and Cenozoic sediment, is inferred to be Triassic in age (Figure 2.5.1-243). Within the site region, the Apalachicola basin and Tampa embayment were two main depocenters for thick evaporite sedimentation during the Jurassic and Cretaceous. There are no thick evaporite deposits beneath the LNP site and surrounding site area (8 km [5 mi.] radius), and there are no reported salt migration features (salt domes) or growth faults in the site vicinity.

As discussed in FSAR Subsection 2.5.1.2.6, there are no mining activities or oil and gas extraction activities within the site area (8 km [5 mi.] radius) that may produce man-induced surface subsidence or collapse.

2.5.3.8.2.2 Potential for Nontectonic Surface Deformation Related to Karst Features

The LNP site area (8 km [5 mi.] radius) is situated in an area known to have potential for karst feature development (Figure 2.5.1-237). This is because of the unique geologic environment of carbonate bedrock covered by a thin mantle of surficial Quaternary deposits composed mainly of sands. Different mechanisms related to carbonate dissolution and karst formation in such environments may result in surface deformation, by slow subsidence related to solution sinkhole development (see FSAR Subsection 2.5.1.2.1.3.2.1). (Reference 2.5.1-317)

Collapse and subsidence are related to the presence of cavities in the subsurface (Figures 2.5.1-240, 2.5.1-241, and 2.5.1-242). Water circulating through primary (original spaces between individual particles in a deposit) or secondary porosity (joints or fractures formed subsequent to deposition and induration) in carbonate rocks leads to dissolution of carbonate material and the formation of enlarged pores and cavities. Increased quantities of water from rainfall and greater flow velocities due to higher energy gradients, induced by drainage, contribute to solution resulting in increased porosity. The increased porosity enhances water circulation and aggravates further solution, leading to an increase in stress within the remaining rock framework. This directly reduces the strength of the mass and aggravates stress corrosion. (Reference 2.5.3-213)

Although the formation of cavities and voids in limestone rock are due to long-term geologic conditions, collapse of overhead rock and soil may be accelerated by loading which may result from the static weight of the overburden, man-caused changes to the environment, rainfall, or a combination of factors, all of which represent increased static or dynamic loads to the overhead structure. Surface subsidence or collapse generally manifests itself within a limited area over or near a ruptured cavity and may take the form of a single, centralized collapse or a large collapse with numerous satellite sinkholes and fractures around the perimeter. (Reference 2.5.3-214)

Changes in surface water runoff and groundwater levels as a result of variations in rainfall are major factors in developing and triggering collapse. Lowering of groundwater causes a loss of buoyancy that leads to general soil stress, and ultimately collapse. An abundance of rainfall can accelerate vertical seepage, increase piping activity, and trigger collapse. (Reference 2.5.3-214)

Man-caused changes on the natural environment are an important factor in developing and triggering collapse. Two of the most common collapse-precipitating activities are the withdrawal of groundwater for residential and industrial use (groundwater pumping) and the concentration of surface runoff or change in surface runoff patterns resulting from the construction and development activities. (Reference 2.5.3-214) Modified drainage and diverted surface water commonly accompany construction activities and can lead to focused infiltration of surface runoff, flooding, and erosion of sinkhole-prone earth materials. (Reference 2.5.3-215) Though many variables contribute to the ultimate cause of collapse, a singular event usually acts as the final triggering mechanism. (Reference 2.5.3-214)

Evaluation of subsurface karst features in the vicinity of safety-related facilities at LNP 1 and LNP 2 is provided in FSAR Subsection 2.5.4.2. Construction activities (dewatering the foundation excavation, road construction, etc) will be monitored and designed to minimize changes to the hydrogeologic and surface water conditions that could in turn trigger formation of sinkholes near the LNP facilities.

LNP COL 2.5-4

Table 2.5.3-201 (Sheet 1 of 3)
Summary of Key Observations and Conclusions Regarding Fracture Orientations from Previous Studies and LNP COLA Mapping Investigations in the LNP Site Vicinity (40-km [25-mi.] Radius)

Source	Region Mapped (Type of Study)	Nomenclature-Direction	Comments
Vernon (1951) (Reference 2.5.1-262)	Northern and Central Florida (Regional Lineament Analysis Using Aerial Photographs and Photo Mosaics)	**Primary system**-generally northwest-southeast **Secondary system**-trending northeast-southeast that intersects primary system at large angles and are irregularly spaced along the flanks of the Ocala Uplift. Two well-developed patterns of fractures deviate from the two systems. One pattern radiates in all directions from Union County. The other trend more westerly and more irregular in pattern.	In Citrus and Levy Counties Vernon makes the following observations. Regional fractures parallel the axis of the Ocala Uplift (northwest trending) and allocate the crest of the fold exactly. Geologic sections crossing the Ocala uplift show that many of the fractures paralleling the axis of the fold are faults, but insufficient data are available on geologic sections crossing transverse joints to determine the displacement along these. A rose diagram based on the orientations of fractures mapped by Vernon in the LNP site vicinity (40 km [25 mi.]) is provided in Figure 2.5.3-223. Major (most frequent) populations of lineaments occur at N42W and N38W, with very minor occurrences at N56E.
Lineament map by Florida Department of Transportation (DOT) (1973) (Reference 2.5.3-211)	State of Florida (Regional Lineament Analysis Using LANDSAT images)	**Lineaments mapped based on interpretation of aerial photographs and satellite imagery**	FSAR Figure 2.5.3-203 shows the lineament map from the DOT Lineament Study. A consistent distribution of lineaments is apparent throughout the state. Trends appear consistent in NE-SW, NW-SE, NNE-SSW, NNW-SSE orientations with some variability. A rose diagram based on the orientations of lineaments mapped by the DOT in the LNP site vicinity (40 km [25 mi.] radius) (Figure 2.5.3-223) shows that major (most frequent) populations of lineaments occur at N47E strike, with slightly less abundant populations at N52E, N72E, and N57W. Minor (less frequent) populations occur at N20W, N25E, and N72W.

LNP COL 2.5-4

Table 2.5.3-201 (Sheet 2 of 3)
Summary of Key Observations and Conclusions Regarding Fracture Orientations from Previous Studies and LNP COLA Mapping Investigations in the LNP Site Vicinity (40-km [25-mi.] Radius)

Source	Region Mapped (Type of Study)	Nomenclature-Direction	Comments
Crystal River Unit No. 3 Nuclear Generating Plant (CR3) FSAR (1975) (Reference 2.5.1-201)	CR3 site-northwestern portion of Citrus County, approximately 13.7 km (8.5 mi.) southwest of the LNP site (Mapping of Fractures in Bedrock Exposed in the Foundation Excavation)	**Two major fracture systems** **Primary "conjugate" joint set** - oriented both parallel (NW) and perpendicular (NE) to the Ocala Uplift. **Second major "conjugate" joint set** – Approximately north-south and east-west trends **Two minor (secondary) "conjugate" fracture sets** N60ºW – N30ºE and N30ºW – N60ºE	General conclusions: Fracturing of the rock in response to the Ocala Uplift and consolidation of the thick sequence of Cretaceous and Tertiary sediments over a stable, competent Paleozoic basement, has produced conspicuous systematic conjugate joint sets, composed of nearly vertical and 45-degree dipping joints. Results of the engineering geology investigation of the CR3 foundation rock system confirmed by constructions observations: • Entire foundation system contains nearly vertical oriented fracture zones • Two major and two minor conjugate fracture systems were traceable at the site and measurable in the excavations • The primary conjugate set (regional joint system) consists of fractures oriented N 45ºW with cross fractures perpendicular to this regional trend. • The second major conjugate fracture set consists of a north-south trend with cross fractures of the set trending east west. • Two secondary conjugate fracture sets that were observed during excavation are oriented N60ºW – N30ºE and N30ºW – N60ºE. These are oblique cross fracture sets to the principal fracture systems and are considered to be the result of stress adjustment to the principal fracture system.

LNP COL 2.5-4

Table 2.5.3-201 (Sheet 3 of 3) Summary of Key Observations and Conclusions Regarding Fracture Orientations from Previous Studies and LNP COLA Mapping Investigations in the LNP Site Vicinity (40-km [25-mi.] Radius)

Source	Region Mapped (Type of Study)	Nomenclature-Direction	Comments
Upchurch (2008)	(Cross Bar Ranch, Pasco County, Florida) (Site-specific Lineament Analysis)	**Bimodal pattern of photolinears**-northeast-southwest and northwest-southeast noted as prominent conjugate sets of photolineaments along with a smaller number of features exhibiting other orientations (small north-south group).	General conclusions: Bimodal pattern of photolinears may be tidally induced fractures. Upchurch cautions that it should not be assumed that photolineaments and/or fracture traces are associated with faulting in Florida. While some may reflect faulting, most reflect tidal flexing or are spurious.
LNP COLA Studies (FSAR Subsection 2.5.4.1.2.1.1; and Subsection 2.5.3.2.1.3)	Wacasassa River and Gulf Hammock Quarry Exposures	**Regional fracture trends** Terminology used in reference to previously defined fracture trends mapped by Vernon (Reference 2.5.1-262) **Local fracture set** Terminology applied to observations of fractures in bedrock exposures.	Orthogonal fracture sets observed at the Wacasassa River and Gulf Hammock Quarry exposures exhibited general orientations of N39W and N51E. Less prominent fractures with orientations of N-S and E-W also were noted. Fracture spacing of approximately 6 to 8 m (20 to 25 ft.) was observed at the Wacasassa River and Gulf Hammock Quarry exposures.
	LNP Site Location (1-km [0.6-mi.] radius) (Lineament Analysis based on interpretation of aerial photographs and hillshade relief map derived from LIDAR data)	**Generally consistent with fracture sets described at the Crystal River Unit No. 3 site.**	Examination of detailed topographic maps derived from the LIDAR data (Figure 2.5.3-215 and Figure 2.5.3-216) shows smaller scale topographic lineaments in the site location that form a rectilinear pattern. A rose diagram shows the relative frequency a major abundance of lineaments in a N37W, N42E, and N57E with slightly less abundance at N03E. Minor abundances occur at orientations of N52W, N62W and N88W (Figure 2.5.3-224).

Notes:
km = kilometer; mi. = miles

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

LNP COL 2.5-1
LNP COL 2.5-2
LNP COL 2.5-3
LNP COL 2.5-5
LNP COL 2.5-6
LNP COL 2.5-7
LNP COL 2.5-8
LNP COL 2.5-9
LNP COL 2.5-10
LNP COL 2.5-11
LNP COL 2.5-12
LNP COL 2.5-13
LNP COL 2.5-16

This subsection presents the information on the properties and stability of soils and rocks that may affect the nuclear power plant facilities, under both static and dynamic conditions including the vibratory ground motions associated with the ground motion response spectrum. The discussion focuses on the stability of the materials, as they influence the safety of seismic Category I facilities (nuclear islands). The discussion also 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 the following subsections:

- Geologic Features (FSAR Subsection 2.5.4.1)
- Properties of Subsurface Materials (FSAR Subsection 2.5.4.2)
- Foundation Interfaces (FSAR Subsection 2.5.4.3)
- Geophysical Surveys (FSAR Subsection 2.5.4.4)
- Excavations and Backfill (FSAR Subsection 2.5.4.5)
- Groundwater Conditions (FSAR Subsection 2.5.4.6)
- Response of Soil and Rock to Dynamic Loading (FSAR Subsection 2.5.4.7)
- Liquefaction Potential (FSAR Subsection 2.5.4.8)
- Earthquake Site Characteristics (FSAR Subsection 2.5.4.9)
- Static Stability (FSAR Subsection 2.5.4.10)
- Design Criteria (FSAR Subsection 2.5.4.11)
- Techniques to Improve Subsurface Conditions (FSAR Subsection 2.5.4.12)

Subsection headings and heading numbers follow Regulatory Guide 1.206 for FSAR Subsection 2.5.4, instead of the DCD headings. The combined license information section is included as FSAR Subsection 2.5.6.

2.5.4.1 Geologic Features

LNP COL 2.5-1
LNP COL 2.5-5

This subsection presents a summary of the non-tectonic processes and geologic features that could relate, if present, to permanent ground deformations or foundation instability at the LNP 1 and LNP 2 safety-related facilities. A summary of the subsurface conditions at LNP 1 and LNP 2 is first presented, based on the subsurface investigation results, and followed by the discussions on the foundation soil and rock properties and stability of these materials. Processes and features evaluated include areas of actual or potential subsurface subsidence, solution activity, uplift, or collapse; zones of alteration, irregular weathering, or structural weakness; unrelieved stresses in bedrock; rocks or soils that may become unstable; and a history of deposition and erosion.

The discussion is based on the site geology summarized in FSAR Subsection 2.5.1, surface faulting described in FSAR Subsection 2.5.3, and results of the site-specific subsurface investigation activities presented in FSAR Subsection 2.5.4.2.

2.5.4.1.1 Summary of Subsurface Conditions

The LNP 1 and LNP 2 nuclear islands will be founded at subgrade elevation 3.4 m (11 ft.) NAVD88 at locations shown on Figure 2.5.4.2-201A. Soil boring and rock coring logs are presented in Appendix 2BB. Figures 2.5.4.2-202A and 2.5.4.2-202B present geologic cross sections through LNP 1 based on these boreholes, and Figure 2.5.4.2-203A and Figure 2.5.4.2-203B present geologic cross sections through LNP 2. The cross-section locations for each of these figures are provided in the legend on each figure.

The depth of undifferentiated sediments (including Quaternary and Tertiary sediments) and the estimated top of rock are indicated on the cross sections (Figure 2.5.4.2-202A, Figure 2.5.4.2-202B, Figure 2.5.4.2-203A, and Figure 2.5.4.2-203B) and listed in Table 2.5.4.2-207. The definition to characterize top of rock is presented in FSAR Subsection 2.5.4.2.2.7.

2.5.4.1.1.1 Description of Soil and Rock

Surface geologic deposits observed at the site consist of undifferentiated Quaternary age fluvial and terrace sediments, primarily silty fine sands. The sands overlie the Avon Park Formation, a shallow marine carbonate rock unit of mid-Eocene age, characterized as cream to brown or tan, poorly indurated to well-indurated, variably fossiliferous limestone, interbedded in places with tan to brown, very poorly to well-indurated, fossiliferous, vuggy dolostones. Carbonized plant remains are common in the rock sequence in the form of thin, poorly indurated laminae and cyclic interbeds.

A review of the boring logs and geophysical data indicates that the base of undifferentiated Quaternary and Tertiary sediments occurs at approximately -7.3 m (-24 ft.) NAVD88 at the site, varying due to depositional history and/or weathering.

2.5.4.1.1.1.1 Correlation with Site Geologic Setting

As discussed in FSAR Subsection 2.5.1, the LNP site is within the Limestone Shelf and Hammocks subzone of the Gulf Coastal Lowlands geomorphic province, which is characterized by broad, flat marine erosional plains, underlain by Eocene limestone, and covered by thin Pleistocene sands deposited by the regressing Gulf of Mexico, as described in FSAR Subsection 2.5.1.2.1.1.

The boring logs presented in Appendix 2BB indicate that the LNP site subsurface is consistent with the Gulf Coastal Lowlands geomorphic province. The results of petrographic examinations of rock core samples, as described in FSAR Subsection 2.5.4.2.3.2.2, are also consistent with rock of this geomorphic province. Of the 20 petrographic samples that were examined, 18 were identified as completely dolomitized limestone.

2.5.4.1.1.2 Dip of Rock Strata

As discussed in FSAR Subsection 2.5.4.4.2.2, the weighted global mean dip of bedding planes is approximately horizontal at LNP 1 and LNP 2. The data from this analysis are presented in Table 2.5.4.4-202.

2.5.4.1.2 Subsidence, Dissolution Activity, Uplift, or Collapse

As described in FSAR Subsection 2.5.1.2.5, there is no record that human activities, such as mining, have been performed in soil or rock near the vicinity of the LNP site, and hence there is no risk associated with mine subsidence or collapse.

The potential for subsidence or collapse pertaining to future solution activity of the LNP subsurface is described in this section, as well as the potential for uplift.

2.5.4.1.2.1 Dissolution Activity

Movement of water through a carbonate rock formation is a catalyst for dissolution activity. The ability for water to move within a rock mass requires either jointing, fractures, or porous characteristics to facilitate movement. As these features become hydraulically interconnected by chemical dissolution of the rock, they enhance the movement of groundwater. At greater depths within the aquifer, the regional aquifer gradients drive the groundwater movement in a predominantly horizontal flow path towards the aquifer's ultimate discharge into the Gulf of Mexico.

Based on results of field investigations, the Floridan aquifer system in the Avon Park Formation at the LNP site consists of interbedded carbonate rock units such as fossiliferous limestone, dolomitized limestone and dolomite. The permeability of these interbedded units is generally high but does vary due to differences in rock texture (primary porosity), secondary fossiliferous porosity, amount of fracturing and degree of dolomitization of the limestone, among other factors. Dolomitization may either increase or decrease the porosity of the rock, but the recrystallization that occurs during dolomitization can result in increased density of the rock. Once limestone has been converted to dolomite, there is less potential for future dissolution of the rock by groundwater.

Specific Avon Park Formation aquifer zones at the LNP site were evaluated using downhole geophysical logging, exploratory borehole rock coring logs, acoustic televiewer logs, and seismic geophysical testing results. Summaries of these testing results are presented in FSAR Subsections 2.5.4.2.1.1, 2.5.4.4.2.1, 2.5.4.4.2.2, and 2.5.4.4.2.3. Karst features at the LNP site, in a manner consistent with Florida geology, typically exhibit a "plus-sign" morphology, whereby chemical dissolution activity in the limestone occurs along both vertical and horizontal planes (vertical fractures and horizontal bedding planes). As described in detail in FSAR Subsection 2.5.4.4.2.5, the gamma-gamma geophysical logs identified randomly distributed lower-density zones that have no spatial significance (i.e., two such low-density zones do not occur at the same depth in adjacent borings). The low-density zones generally have material present and were not voids. The thickness of these possible karst features is typically limited to less than 1.5 m (5 ft.). These features are likely associated with vertical/near-vertical fractures.

The results of the neutron-neutron (porosity) logging, as described in FSAR Subsection 2.5.4.4.2.6, consistently identified the presence of a lower-porosity zone relative to surrounding rock at depths of approximately 42.7 m to 57.9 m (140 ft. to 190 ft.) bgs at LNP 2, and to a lesser extent, also at LNP 1. The downhole seismic studies of formation shear-wave velocities also indicate a slightly higher velocity in the 42.7-m to 57.9-m (140- to 190-ft.) depth zone. Rock coring results in this interval also indicate generally better recoveries and higher rock quality designations (RQD) than zones immediately above or below this depth interval.

These findings indicate that the carbonate rocks of the Avon Park Formation in the 42.7-m to 57.9-m (140-ft. to 190-ft.) depth interval are less susceptible to rapidly developing karst activity associated with vertical infiltration of surface water than the rock units above this interval. This zone within the aquifer is also more dolomitized, displays relatively lower porosity characteristics based on geophysical logging, is more competent structurally, and dissolution activity in this zone will be primarily limited to the gradual processes caused by groundwater movement throughout the aquifer.

There is some indication of dissolution activity within horizons at deeper depths at the LNP site, based on petrographic thin section analysis of rock core samples. The deepest paleosinks observed at or near the LNP site lie beyond the footprints of the nuclear islands and extend to depths of approximately 73 m (240 ft.) bgs (approximately -61 m [-200 ft.] NAVD88), thereby indicating the possible base of the epikarstic horizon at the LNP site. This 73-m (240-ft.) depth coincides with a low seismic velocity zone (site response sublayer Av3b, Figure 2.5.1-250) observed in the shear (V_s) and compression (V_p) wave velocity data obtained at the LNP site. Some of the decreased density and lower velocities observed in this horizon indicate dissolution, but it is noted that the shear-wave velocities in this zone are still in the range of 2100 to 3500 ft/sec, and the material properties may reflect instead the original depositional properties of the rock (e.g., the presence of small interbeds of silt within this unit suggest a different depositional environment than horizons above or below this unit). This is consistent with the findings of the offset boring program that the soft beds within the Avon Park Formation are zones of severely weathered to degraded dolomite, and not cohesive soils. Fewer karst features were encountered in this zone than in the horizons above a depth of approximately 45.7 m (150 ft.) bgs.

Weaker zones of weathered or organic material below the elevation of -61 m (-200 ft.) NAVD88 also were observed as described in FSAR Subsection 2.5.4.4.2.5, but given the depth (approximately 132 m [433 ft.] bgs) and size (about 18 cm [7 in.]) of these features, they are judged to be insignificant to the design of the facilities.

The following subsections describes the nature, frequency, thickness, and lateral extent of the postulated features of dissolution at the LNP site.

2.5.4.1.2.1.1 Nature of Features

As described in FSAR Subsection 2.5.1, Vernon describes a regional fracture set trending NW-SE and NE-SW. (Reference 2.5.1-261) Others have observed a second ENE-WSW and WNW-ESE fracture set, as described in FSAR Subsection 2.5.3.2.1.

A subset of these regional fractures has been identified during subsequent field investigation, with primary and orthogonal fracture spacing on the order of 5.8 to 7.2 m (19 to 23.5 ft.). This fracture set has been observed in local outcrops near the LNP site during field reconnaissance at the Gulf Hammock quarry (located approximately 19 km [11.8 mi.] NNW from the LNP site), and along the banks of the Waccasassa River (located approximately 25 km [15.7 mi.] NNW from the LNP site). In addition, exposed orthogonal vertical fractures of approximately N-S and E-W strikes have been observed as the local dominating joints at the Gulf Hammock quarry and as the less prominent subset along the banks of the Waccasassa River. High-angled joints were also observed during the site investigation. The vertical joints were observed to be on the order of 0.6 to 1 m (2 to 3 ft.) wide at the surface and diminishing in width with depth. The linear orientations of the land features in the area appear to be controlled by the two above-mentioned orthogonal joint sets. For example, the Waccasassa River flows in a north 6 degrees west orientation where the aforementioned joints were observed, and the Withlacoochee River flows west-northwest. Sections of both the Waccasassa and Withlacoochee Rivers appear to be controlled by aforementioned rock joints, as the bends are abrupt, and the sections are linear and distinct.

The Upper Floridan aquifer is a layered aquifer system, which produces water along zones near lithological contacts. These lithological contacts are where horizontal zones of weakness tend to occur. The variable Avon Park Formation contains layers or zones that are pervasively dolomitized in places and undolomitized in others. The types of secondary porosity (vugs and cavities, or fractures) differ depending on the degree of dolomitization of the rock. Limestone is more ductile and the apparent secondary porosity tends to be associated with vugs and cavities, whereas dolomite is harder and more brittle, and the secondary porosity tends to be associated with fractures. As described in FSAR Subsection 2.5.4.2.3.2.2, 18 out of 20 samples of rock that were petrographically analyzed have been completely dolomitized.

As discussed in FSAR Subsection 2.5.4.1.2.1, the carbonate rocks of the Avon Park Formation are generally less susceptible to solution activity compared to the Ocala Formation that underlies much of Florida, including the Crystal River Plant (Reference 2.5.1-322). Furthermore, the Avon Park, at the LNP, in the 42.7-m to 57.9-m (140-ft. to 190-ft.) depth interval is less susceptible to karst activity associated with infiltration of surface water than the rock units above this interval. This depth interval within the aquifer is more dolomitized and displays relatively lower porosity characteristics based on geophysical logging. The dissolution process is described in FSAR Section 2.5.4.1.2.1.1.1.

2.5.4.1.2.1.1.1 Dissolution Process

The Crystal River 3 FSAR indicates that the Ocala Limestone present at the Crystal River site is dissolving at a rate of 1 x 10^{-4} percent per year, or 6 x 10^{-3} percent over 60 years. Due to high levels of dolomitization with recrystallization and the less soluble nature of dolomite than limestone, the Avon Park Formation is less susceptible to dissolution activity and consequential development of karst features than the Ocala Limestone. Given the insignificant rate of annual dissolution activity of the Ocala Limestone at the Crystal River Plant and recognizing that the LNP is founded on the Avon Park, the rate of dissolution activity at LNP is less than 1 x 10^{-4} percent per year.

2.5.4.1.2.1.2 Frequency and Thickness of Features

A review of the subsurface investigations data was conducted to evaluate the potential for karst features development within the limestone bedrock strata. The features were evaluated based on field observations during the rock coring, such as rod drop and circulation loss, as well as the recovered core and RQD data.

The results of this evaluation are presented in Tables 2.5.4.2-205A and 2.5.4.2-205B, “Summary of Karst Features Encountered in Boreholes.” Depth and thickness of each feature are listed and summarized at each plant location and arranged by boring number.

The information is presented graphically on Figure 2.5.4.1-201A and Figure 2.5.4.1-201B, showing histograms of the thicknesses of the observed features. The thickness of an individual feature is typically less than 1.5 m (5 ft.).

It is noted that the histograms presented in Figure 2.5.4.1-201A and Figure 2.5.4.1-201B are based on the site characterization boreholes, which were largely concentrated in the upper 180 ft.The histogram is based on all of the available data and reflects the fact that there are more data available for the higher portion of the geologic profile simply because, as with all sites, there are more shallow borings and samples than deep borings and samples. Of course, there is a need for more data at shallow depths because the stresses induced by foundations have to be accommodated by the shallower formations, whereas at deeper depths, the induced stresses diminish eventually to nil with depths on the order of 1.5 to 2 times the dimensions of the foundation being supported; hence less data are required at deeper depths. FSAR Subsection 2.5.4.10 demonstrates that potential features located below 180 ft. are of considerably less significance given the depth and the robustness of the foundation design, specifically the 35-ft. thick RCC Bridging Mat and the 6-ft. thick AP1000 basemat. The impacts of the subsurface below the upper 180 ft. are discussed in regard to bearing capacity, settlement, and other geotechnical parameters in FSAR Subsection 2.5.4.10.

2.5.4.1.2.1.3 Lateral Extent of Features

As described in FSAR Subsection 2.5.4.4.2.5, the gamma-gamma logs indicate randomly distributed low-density zones that have no spatial significance (i.e., two such low-density zones do not occur at the same depth in adjacent borings). The low-density zones generally have material present and are not voids. The thickness of these low-density zones is typically limited to less than 1.5 m (5 ft.).

The Avon Park Formation typically exhibits higher degrees of dolomitization than the late Eocene Ocala Limestone, and consequentially, less susceptibility to dissolution activity (Reference 2.5.1-322). Eighteen out of twenty (20) samples of rock that were petrographically analyzed have been completely dolomitized. This is significant because the more dolomitized Avon Park Formation layers have a higher percentage of recrystallized magnesium carbonate, and is therefore less susceptible to the types of karst activity known to occur within the pure calcium carbonate limestone zones typically present within the Ocala Limestone.

Vernon (Reference 2.5.3-203) describes a regional fracture set trending NW-SE and NE-SW. In March 2008, a subset of these regional fractures was identified during field investigations. This fracture set was observed in local outcrops near the LNP Site during field reconnaissance at the Gulf Hammock Quarry and along the banks of the Waccasassa River.

At the Gulf Hammock Quarry, along an Avon Park Formation outcrop striking due North, primary and orthogonal vertical fractures were observed. Fractures were evident at 30-ft. spacing along this outcrop, and had iron staining consistent with water infiltration along the fracture.

Along a portion of the Waccasassa River where the Avon Park Formation outcrops, striking at North 6 degrees West, primary and orthogonal vertical fractures were evident at 35-ft. spacing.

Given the strikes of these Avon Park Formation outcrops, and given the observed vertical fractures and spacing, a subset to the regional fracture set was postulated. This local fracture set, with primary fractures consistent with the North 39 degrees West strike associated with Vernon's regional fracture set, features a primary fracture spacing of approximately 19 ft. and an orthogonal fracture spacing of approximately 23.5 ft. The Avon Park Formation outcrop strikes and the postulated fracture pattern associated with the local observed outcrops are shown on Figure 2.5.4.1-202.

In order to quantify the vertical dimension (thickness) of postulated karst features associated with these fracture sets, field observation data, gathered during rock coring, were evaluated. The results of this evaluation are presented in Table 2.5.4.2-205A and Table 2.5.4.2-205B and presented graphically on Figure 2.5.4.1-201A and Figure 2.5.4.1-201B. The thicknesses of these features are typically limited to less than 1.5 m (5 ft.).

Three methods were used to estimate the lateral dimension of the karst features on the LNP Site: field observations, geophysical testing, and excess grout takes from the subsurface investigation. Based on this analysis, the average width-to-height ratio of features associated with vertical fractures is 1H:5V, limiting the lateral extent of these features to approximately 20 percent of the vertical extent, as supported by geophysical testing and field observations. Dr. Anthony Randazzo, a subject matter expert, is supportive of the approach that the horizontal dimension is a fraction of the vertical dimension of the feature.

The largest single potential karst feature identified in Table 2.5.4.2-205A and Table 2.5.4.2-205B (19.5 ft.) would correspond to a vertical feature that is 20 percent of 19.5 ft., or 3.9 ft. wide.

Given the conservative estimations made in determining the lateral extent of the postulated karst features at the LNP Site, the RCC Bridging Mat was designed to span a 10-ft. diameter void beneath the Bridging Mat (elevation -24 ft. NAVD) at any plan location, at any depth. Additionally, following the offset boring program, it was determined that the Avon Park Formation contains fewer karst features than were conservatively postulated based on the original site characterization, as evidenced by the high recovery during the offset boring program. The soft beds within the Avon Park Formation have been identified and described as zones of severely weathered to degraded dolomite, and not the cohesive soils that were postulated following site characterization. Thus the design karst feature (10-ft. diameter void) is regarded as sufficiently conservative.

2.5.4.1.2.1.4 Mitigation of Potential Surface Deformation Related to Karst Features

Beneath the LNP 1 and LNP 2 nuclear islands, the uppermost approximately 20.4 m (67 ft.) of undifferentiated sediments and rock (to elevation -7.3 m [-24 ft.] NAVD88) will be excavated and backfilled with roller compacted concrete prior to construction, as described in FSAR Subsection 2.5.4.5.

As described in FSAR Subsection 2.5.4.5, the roller compacted bridging mat has been designed to span a 3-m (10-ft.) diameter void immediately beneath the bridging mat at any location within the footprint of the nuclear island, as well as a larger-diameter void immediately beneath the grouted zone.

The potential for unacceptable deformation of the AP1000 basemat, related to karst features immediately beneath the plant structures is eliminated with these measures in place.

2.5.4.1.2.2 Uplift or Collapse

The LNP site is located on the west coast of the Florida platform. As described in FSAR Subsection 2.5.1.1.2.3, the Florida platform represents long-term carbonate sedimentation on a passive margin, and late Quaternary deposits have not experienced significant uplift, subsidence, or tectonic deformation. (Reference 2.5.1-236)

2.5.4.1.3 Zones of Alteration, Irregular Weathering, or Structural Weakness

The bedrock, which underlies the undifferentiated Quaternary sediments, is the middle Eocene-aged Avon Park Formation. The upper portion of this formation, which consists of calcareous silts (units S2 and S3, also referred to as undifferentiated Tertiary sediment) appears to have been altered by weathering and greater degrees of dissolutioning (FSAR Subsection 2.5.1.2). This zone occurs near elevation -7.3 m (-24 ft.) NAVD88, although it is highly undulatory by nature. A review of the boring logs indicates that the base of the zone varies by up to approximately 2.1 m (7 ft.) below elevation -7.3 m (-24 ft.) NAVD88 within the extents of the nuclear islands. These undifferentiated sediments will be excavated, and the excavation surface will be cleaned and prepared as described in FSAR Subsection 2.5.4.5.3.

Zones of structural weaknesses, such as extensive fractured or faulted zones, are not present; however, vertical joints, sometimes connected by horizontal bedding planes, are present and may act as zones of weakness. These discontinuities, likely a factor in the localization and development of dissolution activity at the site, were considered in the rock mass properties used in design.

2.5.4.1.4 Unrelieved Stresses in Bedrock

There is no evidence of unrelieved stresses in bedrock, as noted in FSAR Subsection 2.5.1.2.6.

2.5.4.1.5 Rocks or Soils that may Become Unstable

The potential hazard from rocks or soils that may become unstable was determined to be low. While evidence of historic solution activity is present, FSAR Subsections 2.5.4.1.2.1, 2.5.4.5, and 2.5.4.10 present background information, construction techniques, and engineering analyses, which indicate the mitigation of any significant future solution activity immediately beneath the nuclear island. The potential for liquefaction is presented in FSAR Subsection 2.5.4.8.

2.5.4.1.6 History of Deposition and Erosion

As described in FSAR Subsection 2.5.1.2, the geomorphic province of the LNP site is characterized by broad, flat marine erosional plains, underlain by Eocene limestones, and covered by thin sands deposited by the regressing Gulf of Mexico. The limestone plain is erosional, and overlain by sand dunes, ridges, and coast-parallel paleoshore sand belts associated with marine terrace(s) (FSAR Subsection 2.5.1.2.1.2).

This erosion was aerial, and resulted in an unconformable surface at the top of the undifferentiated Tertiary sediments (calcareous silts).

2.5.4.2 Properties of Subsurface Materials

LNP COL 2.5-6 The investigation activities at and near the LNP site were conducted to develop a comprehensive characterization of subsurface conditions that will influence foundation performance of safety-related structures, including the static and dynamic engineering properties of soil and rock in the site area. The LNP site is approximately 1257 hectares (ha) (3105 acres [ac.]), with the primary location for the two reactors and ancillary power production support facilities comprising approximately 121 ha (300 ac.) near the center of the site. This subsection presents the detailed discussions of the type, quantity, extent, purpose, and results of the investigation activities at LNP 1, the southernmost reactor, and LNP 2, the northernmost reactor. Type, quantity, and depth of boreholes and in situ tests were selected to follow the guidance in the NRC Regulatory Guide 1.132, and laboratory tests were performed to follow the guidance in NRC Regulatory Guide 1.138. Plan and profile plots of information from site explorations are also provided. Properties of soils and rocks used in evaluations are summarized.

2.5.4.2.1 Description of Investigation Activities

Field subsurface investigation activities were performed at the LNP site from January 2007 through December 2007 under the overall direction of CH2M HILL in accordance with the Site Investigation Work Plan (SIWP). (Reference 2.5.4.2-201) Laboratory tests were conducted on samples recovered during the field investigations following the retrieval of soil and rock samples.

The following subsections provide a summary of the field activities and laboratory tests that were conducted for the COLA. Detailed discussions of the criteria used to develop the scope of the subsurface investigation activities are presented throughout this subsection. Changes to the planned activities made to address observations during the investigation, and their rationale, are also described.

2.5.4.2.1.1 Soil Boring and Rock Coring

The subsurface investigation program of soil boring and rock coring was completed in four phases: initial, main, supplemental investigation, and offset boring program phases. They included the following field activities:

- Initial investigation phase: Ten boreholes (I-series boreholes) were advanced using sonic drilling techniques within the vicinity of the plant layout to determine the subsurface conditions and conduct geophysical logging.

- Main investigation phase: Ninety boreholes at the site for the two reactor units (LNP 1 and LNP 2) were advanced during the main phase to obtain soil and rock samples for geologic characterization and for laboratory tests. This phase program included 68 boreholes drilled at or near the planned AP1000 structures (A-, B-, D-, and E-series boreholes), 12 general characterization boreholes drilled around the LNP site area (GSC-series), 8 boreholes drilled at the planned cooling tower locations (CT-series), and 2 boreholes drilled at the intake structure locations (IT-series).

- Supplemental investigation phase: This phase program included 18 additional boreholes.

- Offset Boring Program phase: This phase included 8 additional boreholes.

The initial and main investigation phases were developed to address the safety-related structure performance for a “uniform” site as specified in DCD Subsection 2.5.4.5 and in accordance with the guidance in NRC Regulatory Guide 1.132.

Review of the shear-wave velocity measurements during the initial and main phases, however, indicated that the subsurface conditions are potentially non-uniform because the shear-wave velocity varied by about 20 percent within individual subsurface layers across the site. Additionally, the shear-wave velocity at approximately 61 m (200 ft.) bgs dropped from approximately 1370 m/sec (4500 ft/sec) to approximately 760 m/sec (2500 ft/sec) and was trending down to the termination depth of the boreholes. Therefore, supplemental boreholes were planned to better define the subsurface condition, including the top of rock, and obtain additional shear-wave velocity measurements at depth below 76.2 m (250 ft.). The site will be improved as discussed in FSAR Subsection 2.5.4.12.

The supplemental investigation phase consisted of the following field activities:

- Nine additional boreholes were advanced to depths between 25.9 and 45.7 m (85.0 and 150 ft.) at locations adjacent to the A-, B- and GSC-series boreholes (these include Boreholes A-14, A-18, A-21, A-22, A-24, B-04, B-07, B-30, and GSC-08). The addition of these boreholes was based on conclusions reached during a review of the available data. In 5 of A-series boreholes (such as A-14, A-18, A-21, A-22, and A-24), rock was indicated at significantly deeper depth due to continued drilling using tri-cone bit. Two B & GSC series boreholes (B-30 and GSC-08) had rod drops, which is an indication of possible karst features. Additionally, Boreholes B-04 and B-07 drilled within the Turbine Building footprint had rock encountered at significantly deeper depths than adjacent boreholes. The purpose of these additional boreholes was to either establish top of rock, which is defined in FSAR Subsection 2.5.4.2.2.7, to support the Turbine Building foundation design, or to evaluate the potential existence of karst features within the rock formations.

- Four additional deep boreholes (i.e., AD-01 and AD-02 at LNP 2 and AD-03 and AD-04 at LNP 1) were advanced to 152.4 m [500 ft.] bgs. The purpose of these deep boreholes was to obtain the dynamic properties (mainly V_S) of rock at depth up to 152.4 m (500 ft.) bgs. Note that steel isolation casings were used during drilling of these boreholes to a depth of approximately 61 m (200 ft.) bgs.

- GSC-01A, GSC-01B, and GSC-07A were drilled to trace deep soil conditions.

- Two additional boreholes (B-23A and B-25A) were drilled as offset Boreholes of B-23 and B-25, respectively, because the rig used for drilling B-23 and B-25 was not tested for standard penetration test energy transfer efficiency.

The offset boring program consisted of the following field activities:

- Six (6) PQ-size, triple tube type offset core borings were drilled within a 1.5 m (5 ft.) radius of a previously drilled borehole.

- Two (2) borings were drilled to better establish the top of rock surface underlying the Turbine Building in LNP 2.

Figures 2.5.4.2-201A, 2.5.4.2-201B, and 2.5.4.2-201C show the plan view of the borehole locations for the four phases of field investigations, and Table 2.5.4.2-201 summarizes the borehole information for boreholes drilled within the nuclear island and adjacent structures.

2.5.4.2.1.1.1 Criteria for Selection of Borehole Locations

Appendix D of NRC Regulatory Guide 1.132 provides specific criteria on the spacing of principal boreholes for safety-related structures for favorable, uniform geologic conditions, as follows:

- At least one borehole beneath every safety-related structure.

- For larger, heavier structures, such as the Containment Building, at least one borehole per 929 square meters (m^2) (10,000 square feet [$ft.^2$]) and approximately 30.5 m (100 ft.) spacing.

- In addition, a number of boreholes along the perimeter, at corners, and other selected locations.

- One borehole per 30.5 m (100 ft.) for essentially linear structures.

The initial and main subsurface investigation phases were developed to satisfy these criteria. The initial issue of SIWP specified 100 boreholes for both AP1000 structures at the LNP site, which provided coverage for the nuclear islands (seismic Category I structures), as well as the adjacent structures. (Reference 2.5.4.2-201)

Five supplemental A-series holes were drilled adjacent to Boreholes A-14, A-18, A-21, A-22, and A-24 at approximately 1.5 m (5 ft.) away to accurately establish the depth to the top of rock at these locations. These supplemental boreholes are denoted as A-14A, A-18A, A-21A, A-22A, and A-24A. For these particular boreholes (i.e., Boreholes A-14, A-18, A-21, A-22, and A-24), drillers advanced the mud rotary drilling beyond the high blow count materials to avoid wall impingement and stuck rods, which resulted in rock coring commencing at depths below top of rock.

Boreholes B-30 and GSC-08 encountered zones with no sample recovery, a possible indication of karst features. In Borehole B-30, karst features in a zone between elevations -14.4 and -18.5 m [-47.3 and -60.8 ft.]) NAVD88 were encountered. In Borehole GSC-08, a 0.9 m (3.0 ft.), a thick karst feature from elevation -20.4 to -21.3 m (-66.8 to -69.8 ft.) NAVD88 and another 1.1 m (3.5 ft.) thick karst feature from elevation -23.9 to -24.9 m (-78.3 to -81.8 ft.) NAVD88 were recorded during drilling. The core run at an elevation -21.9 to -23.4 m (-71.8 to -76.8 ft.) NAVD88 in Borehole GSC-08 had 20 percent rock recovery, with 10 percent RQD.

To further investigate the presence of karst features where zones of no recovery or rod drop occurred, two supplemental boreholes (Boreholes B-30A and GSC-08A) were drilled approximately 1.5 m (5 ft.) from B-30 and GSC-08 boreholes using the HQ-size coring tools. Up to about 0.6 m (2.0 ft.) of karst feature with a total thickness of 1.0 m (3.4 ft.) were encountered in Borehole B-30A at elevations between -12.8 and -15.2 m (-42 and -50 ft.) NAVD88. In Borehole GSC-08A, a karst feature of 1.99 m (6.5 ft.) in thickness was encountered at a depth between -21.46 and -23.45 m (-70.4 and -76.9 ft.) NAVD88.

Two supplemental boreholes (Boreholes B-04A and B-07A) were also added adjacent to B-04 and B-07 borehole locations to provide supplement information about the rock profile under the LNP 2 Turbine Building. Also, because the hammer used to obtain the standard penetration test (SPT) blow counts in Boreholes B-23 and B-25 was not tested for energy transfer efficiency, two supplemental boreholes (Boreholes B-23A and B-25A) were drilled in proximity to B-23 and B-25 borehole locations and SPTs conducted with a calibrated hammer. Consequently, the SPT data obtained from the original B-23 and B-25 boreholes were not used.

Preliminary review of the shear-wave velocity information, collected as part of the main phase of the field activities, found that the shear-wave velocity below a depth of approximately 61 m (200 ft.) dropped from approximately 1370 m/sec

(4500 ft/sec) to approximately 760 m/sec (2500 ft/sec) and this lower shear-wave velocity extended to the maximum depth of drilling and sampling (91.4 m [300 ft.]) carried out during the initial phase and main phase explorations. Because of the thickness of the zone of low shear-wave velocity in the Avon Park Formation could have an impact on site responses under the design earthquake ground motions, four additional deep boreholes were drilled during the supplemental investigation to depths of about 152.4 m (500 ft.) (two additional boreholes at each reactor area) to provide site-specific parameters and to determine the extent of the lower-velocity rock zone. These deep boreholes are denoted as AD-01, AD-02, AD-03, and AD-04.

As part of the offset boring program phase, six boreholes were drilled within a 1.5 m (5 ft.) radius of an existing A-series borehole in order to evaluate the properties of materials previously not recovered during core drilling, including the verification of the existence, thickness, and location of postulated beds of soft, soil-like material. Two additional borings were also drilled to evaluate the top-of-rock surface beneath the LNP 2 turbine building.

In total, the boreholes drilled during the initial, main, supplemental, and offset boring program subsurface investigation phases provide coverage for the safety-related structures that satisfies and exceeds the criteria listed in Regulatory Guide 1.132. The boreholes have average spacing of less than 30.5 m (100 ft.) on center (more than one borehole per 929 m^2 [10,000 $ft.^2$] under safety-related structures). The coverage is considered sufficient for characterizing foundation performance of safety-related structures.

2.5.4.2.1.1.2 Criteria for Selection of Borehole Depths

Appendix D of NRC Regulatory Guide 1.132 provides specific criteria for depths of principal boreholes for safety-related structures, as follows:

- Where soils are thick, the maximum required depth for engineering purposes (d_{max}), may be taken as the depth at which the change in vertical stress for the combined foundation loading is less than 10 percent of the effective in situ overburden stress.

- Boreholes should extend at least 10 m (33 ft.) below the lowest part of the foundation.

- If rock is encountered at lesser depths than those given, boreholes should penetrate to the greatest depth where discontinuities or zones of weakness or alteration can affect foundations and should penetrate at least 6 m (20 ft.) into sound rock.

- At least one-fourth of the principal boreholes and a minimum of one borehole per structure should penetrate into sound rock or to a depth equal to d_{max}.

- Other boreholes should penetrate to a depth below the foundation elevation equal to the width of the structure.
- Other boreholes for soil-structure interaction studies should penetrate to depths greater than those required for general engineering properties.
- For weathered shale or soft rock, depths should be as for soils.

Top of rock was encountered between 8 and 42 m (26 and 137.5 ft.) bgs in the boreholes over the LNP site. The top of rock under the nuclear islands was encountered no deeper than 23.2 m (76 ft.) bgs. Boreholes were generally advanced to depths that satisfy the criteria for minimum depth below the foundation or into sound rock.

As specified in the initial SIWP, principal boreholes were planned to satisfy the following criteria (Reference 2.5.4.2-201):

- Each of the principal boreholes (i.e., boreholes drilled within footprint of nuclear islands but excluding the boreholes drilled during the supplemental phase as shown in Table 2.5.4.2-201) under the middle of the nuclear islands was advanced at least 48.8 m (160 ft.) bgs, which is 39.5 m (129.5 ft.) below the nuclear island basemat elevation of +3.5 m (+11.5 ft.) NAVD88. The existing ground surface is at approximately elevation +12.8 m (+42 ft.) NAVD88.
- Some of the principal boreholes beneath the safety-related structures sites were advanced to greater depths of 76.2 to 91.4 m (250 to 300 ft.), and the four supplemental deep boreholes were advanced to depths of 152.4 m (500 ft.). The depths are equivalent to or greater than the maximum dimension of the nuclear island of 78 m (256 ft.).
- In the initial and main investigation phases, 14 boreholes were advanced to depths necessary to characterize soil/rock properties for dynamic properties (including V_S) to be used for ground motion calculations and soil/rock-structure interaction studies (7 per plant site; 5 in I-series and 2 in A-series boreholes, I-01 to I-10 and A-07, A-08, A-19, and A-20). The majority of these boreholes were advanced to at least 80.8 m (265 ft.) bgs (approximate elevation -67.9 m [-223 ft.] NAVD88). Two of the I-series boreholes (one at each plant site, I-02 and I-07) were advanced to depths greater than 91.4 m (300 ft.) bgs (approximate elevations lower than -78.6 m [-258 ft.] NAVD88), which is about 3.05 m (10 ft.) below the depth equivalent to the maximum dimension of the nuclear island of 78 m (256 ft.) below the subgrade elevation.
- After the initial and main investigation phases, 12 additional boreholes were drilled during the supplemental phase of the investigation. The depths of these boreholes are between 26.4 to 45.7 m (86.5 to 150 ft.). These

boreholes were drilled mostly for the confirmation of the top of rock or to investigate potential karst feature locations within the rock.

- Four additional deep boreholes were drilled during the supplemental phase (two for each nuclear island) to a depth of 152.4 m (500 ft.). The main objective of these deep boreholes was to obtain soil/rock properties for earthquake ground motion characterization (including V_S) at depths below 91.4 m (300 ft.) bgs.

- Two additional boreholes (B-23A and B-25A) were drilled as offset boreholes of B-23 and B-25, respectively, because the rig used for drilling B-23 and B-25 was not tested for standard penetration test energy transfer efficiency.

- Six additional boreholes drilled during the offset boring program were drilled to reach the depth of their corresponding adjacent nuclear island boreholes. Two boreholes drilled in the LNP 2 turbine building during the offset boring program were drilled to help establish the top of rock surface.

In total, these boreholes (as described in SIWP [Reference 2.5.4.2-201]) satisfy the depth requirements in NRC Regulatory Guide 1.132 for principal boreholes, as further described in FSAR Subsection 2.5.4.10.1.1.

2.5.4.2.1.1.3 Drilling and Sampling Methods

Drilling activities were performed by Universal Engineering Sciences of Gainesville, Florida, Boart Longyear of Ocala, Florida, and Huss Drilling of Dade City, Florida. These drilling companies used sonic, mud rotary, and rock coring methods, as summarized below.

- Rotosonic (sonic) drilling was used in the initial investigation phase for soil drilling and rock coring in the I-series boreholes (Boreholes I-01 through I-10). The sonic method was also used to advance the drilling in the top 61 to 64.6 m (200 to 212 ft.) of the AD-series boreholes (Boreholes AD-01 through AD-04). The sonic method uses a hydraulically activated drill head unit that imparts high frequency sinusoidal wave vibrations into a drill string to effectuate cutting action at the bit face. The casing used in the sonic drilling method always provides full support to the drill hole as drilling occurs. Continuous sampling of soil and rock was performed using a 10 cm (4 in.) diameter core barrel during the sonic drilling. This method is suitable for obtaining a nearly continuous sample for visual inspection in accordance with the requirements in Regulatory Guide 1.132 for continuous sampling.

- In the main investigation phase, soil drilling was advanced using the mud rotary drilling method. In the mud rotary drilling, a positive hydrostatic fluid head is used in the borehole to stabilize the borehole sidewall while drilling through loose or soft soils. Mud rotary is the recommended drilling method for soils in Regulatory Guide 1.132. This method, described below, also allowed collection of representative soil samples as well as reliable SPT blow counts in the materials.

- The drilling fluid consisted of a mixture of bentonite and water.
- 7.9 to 10 cm ($3^{1}/_{8}$ to 4 in.) diameter tri-cone bits were used during drilling.
- The SPT was conducted with either an automatic or cathead type hammer and the blow counts were recorded on the field log for each sample. Verification of energy transfer efficiency of the SPT hammer was performed during the filed investigation in accordance with American Society for Testing and Materials (ASTM) D4633-05 (Reference 2.5.4.2-202).

- In the main investigation phase, rock coring beneath the soil was performed using NQWL and HQWL double-tube, diamond-tipped rock core tools in accordance with ASTM D2113 (Reference 2.5.4.2-203). The stationary inner core barrel used in the double-tube coring reduced rock core damage, as compared to the single-tube coring, and it is a recommended coring method for rock in Regulatory Guide 1.132. NQWL was the predominant size (nominal sample diameter of 4.76 cm [1.875 in.]) used for most of the boreholes, while HQWL-size coring tools (nominal sample diameter of 6.35 cm [2.5 in.]) was used primarily for boreholes in which the Suspension P- and S-wave (P-S) Logging surveys were performed to obtain shear-wave velocity (such as Boreholes A-07, A-08, A-19, A-20, AD-01 through AD-04).

- In the offset boring program phase, drilling efforts consisted of advancing through the overburden down to rock using mud rotary methods, and included sampling in some instances. After casing was set through the overburden and at least 5 ft. into rock, the rock coring was advanced using PQ sized triple tube type coring methods. Rock drilling was monitored, at a minimum, for drilling pressure and rotational speed. In addition, time of drilling, RQD, recovery, blow count, circulation data, and soil / rock visual classifications were recorded on the boring logs. Thin Wall Shelby Tubes and/or split-spoons were used to obtain samples of the postulated soft beds while rock coring. In-situ Vane Shear Testing was also attempted.

When the mud rotary drilling method was used, disturbed soil samples were collected using an SPT sampler in accordance with Regulatory Guide 1.132 and ASTM D1586-99 requirements (Reference 2.5.4.2-204). In the main phase boreholes where seismic wave velocities were measured, continuous disturbed SPT soil samples and rock coring samples were collected.

No “undisturbed” soil samples were recovered above the top of rock during the drilling, because the soils at the LNP site generally consist of sandy materials not suitable for good quality “undisturbed” samples. Two relatively “undisturbed” soil-like samples, however, were recovered through rock coring from depths of 132.3 to 132.7 m (434 to 435.4 ft.) in the AD-series boreholes for laboratory tests. These soil-like materials were observed at all four deep boreholes (AD-01 through AD-04) at depths from 118.9 to 143 m (390 to 470 ft.). The thickness of individual intervals of encountered soil-like materials ranged from 0.03 to 0.5 m (0.1 to 1.6 ft.), and total thickness varied from 0.6 to 1.6 m (1.9 to 5.3 ft.) within this depth range at each AD-series borehole. Rock core samples were collected directly from the NQ-, HQ-, or PQ size rock core, and they were managed as either “routine care” or “special care” cores. Methods for management of soil and rock samples are described in FSAR Subsection 2.5.4.2.1.5.

The drilling, coring, and sampling methods selected for the project are standard procedures recommended in NRC Regulatory Guide 1.132. They are considered appropriate for the subsurface materials encountered at the LNP site, and they provide reliable data for characterizing foundation conditions for safety-related structures.

2.5.4.2.1.1.4 Field Observations, Logs, and Field Tests

Field investigation activities were performed to characterize the soil and rock types, soil consistency, and rock soundness. The following procedures were followed during the field investigation:

- Field observations, including visual descriptions of each soil sample and rock core, were recorded on soil boring and rock coring logs in accordance with the SIWP. (Reference 2.5.4.2-201) Appendix 2BB presents the soil boring and rock core logs. As-built survey coordinates and elevations were also included on the borehole logs. Surveying was performed by Florida-registered surveyors from CH2M HILL.

- SPTs were performed at regular intervals in soil, in accordance with ASTM D1586-99. (Reference 2.5.4.2-204)

- Field indicators of rock soundness and strength were established for representative sections of each core run based on the RQD (Reference 2.5.4.2-205), the R-value indicator of strength (Reference 2.5.4.2-206), and field point-load test (PLT) (Reference 2.5.4.2-207).

- Rock PMTs were performed in two boreholes to obtain information about the in situ modulus of the rock. This information was used to estimate the compressibility of the rock. These tests were conducted in accordance with a procedure, presented in the SIWP. (Reference 2.5.4.2-201)

2.5.4.2.1.1.5 Basis for Selection of Field Rock Hardness and Strength Tests

Rock consistency at the LNP site was characterized primarily using the laboratory UCS test results (see FSAR Subsection 2.5.4.2.1.5). A semi-quantitative “R-scale” measure of rock core strength was also obtained in the field in accordance with the International Society of Rock Mechanics (ISRM procedure) (Reference 2.5.4.2-206). The R-scale strength value is determined based on the observed response of rock to impact with a geologic hammer. The resulting R-scale values provide a semi-quantitative field indication of rock strength, and a basis to identify sound rock. Although tables that correlate R-scale measurement to UCS are available, these correlations are not considered accurate enough to assign specific UCS values for design.

In addition, PLTs were performed in accordance with ASTM D5731 (Reference 2.5.4.2-207) on rock core samples collected from the boreholes at various depths. The PLT is a simple quantitative test in which a core segment is compressed in a test device between two conical platens, and the resulting pressure to fail the rock is recorded. The point load index (I_{50}) of the rock is then determined based on the failure pressure and specimen geometry. The I_{50} value is proportional to the rock strength (UCS). (Reference 2.5.4.2-207)

In-situ Soil Testing, LC, of Lancaster, Virginia performed rock PMTs in two boreholes (B-19 at LNP 1 and B-11 at LNP 2). These tests were performed at numerous depths to provide information on the in situ modulus of the rock mass. The results of these tests are discussed in FSAR Subsection 2.5.4.2.2.5.

The UCS strength data, combined with the field R-scale results, were reviewed and compared for each general rock type to establish design rock strengths. The results of these interpretations provide adequate strength data for evaluation of rock bearing characteristics and stability at the LNP site.

2.5.4.2.1.2 Geophysical Surveys

A program of borehole geophysical surveys was implemented at LNP 1 and LNP 2 for the COLA. These methods included Suspension P-S velocity logging, downhole velocity logging, and acoustic televiewer surveys. Results of the Suspension P-S and downhole surveys were used to estimate the shear and compression wave velocities of the geologic formations; the acoustic televiewer surveys were used to evaluate the potential for fractures and other characteristics of the rock formations.

Non-seismic geophysical loggings were also conducted on the I-series and AD-series boreholes to assist in establishing the interface zone between the upper soils and soil-like material and the rock (that is, depth of the top of rock) and to obtain geologic information on lithology, degree of consolidation, porosity and permeability, and pore-fluid amount and characteristics. The non-seismic geophysical surveys consisted of natural gamma (clay) measurements, gamma-gamma (density) measurements, neutron-neutron (porosity) measurements, and induction (conductivity) measurements.

Detailed descriptions of the seismic and non-seismic geophysical survey activities are presented in FSAR Subsection 2.5.4.4.1.

2.5.4.2.1.3 Hydraulic Conductivity Tests

Sixteen monitoring wells and seven observation wells were installed at the LNP site to monitor seasonal fluctuations in groundwater elevations and to evaluate hydraulic conductivity of soil and rock. Observation wells were installed primarily to measure drawdown effects on the aquifer during the aquifer performance test on the pumped well. The monitoring wells were installed around the site to collect quarterly data on the seasonal variation in the site-wide potentiometric surface, and were also sampled quarterly for groundwater quality data. Monitoring well locations were installed in accordance with the SIWP (Reference 2.5.4.2-201). Figure 2.4.12-214 shows the locations of the monitoring wells. An aquifer pumping test was performed in one well (PW 1), and the locations of the pumping and observation wells are shown on Figure 2.4.12-225. In addition, slug tests were performed in each of the 23 wells. Table 2.4.12-207 provides a summary of well construction details.

2.5.4.2.1.4 Management of Soil and Rock Core Samples

Soil and rock core samples were handled with levels of care appropriate for their intended uses. Sample management details are described in the SIWP and summarized as follows (Reference 2.5.4.2-201):

- Soil samples recovered by SPT methods were stored in jars with a watertight lid. These samples were retained for visual-manual field classification and for index testing, as summarized in FSAR Subsection 2.5.4.2.1.5.

- Rock cores were managed as either “routine-care” or “special-care” samples, depending on the intended use of the samples, as recommended in ASTM D5079 (Reference 2.5.4.2-208).

    – Routine care was used for most rock cores intended for long-term storage, but not for laboratory testing of engineering properties. Routine care included placement in wooden rock core boxes for long-term storage in access-controlled storage areas. These samples will be available for inspection by designers, contractors, or regulatory staff.

    – Special-care rock core samples were protected from shock and variations in moisture and temperature. Immediately after field collection, the samples were tightly wrapped in plastic film and a layer of aluminum foil. A coat of wax was then applied to entirely cover the sample. Special-care rock core samples were stored in a dedicated temperature- and humidity-monitored and controlled storage area prior to shipment to the testing laboratory. These samples were used for laboratory strength testing.

Chain-of-custody of soil and rock samples was maintained. Each day, samples collected during the day were transferred to access-controlled sample storage areas, which were accessible only to designated field team staff and designated client personnel. Samples were logged in upon placement in storage, and logged out upon removal for transportation to the testing laboratory. Chain-of-custody forms were completed for each sample to document transfer of custody to the geotechnical laboratory subcontractor staff.

Routine-care rock samples were stored on-site in a designated storage facility. Special-care rock samples and glass jar samples were transferred to a geotechnical testing laboratory. Foam cushioning material was used to protect special-care, rock core specimens from shock when they were transported to the testing laboratory by car. Upon receipt at the testing laboratory, special-care rock samples and glass jar samples were stored in designated temperature- and humidity-controlled storage areas in accordance with the laboratory quality plan. Upon completion of laboratory testing activities, samples (tested and untested) were transferred back to Progress Energy.

2.5.4.2.1.5 Laboratory Testing of Soil and Rock

Laboratory tests were performed on special-care rock and SPT soil samples recovered during the drilling and sampling program. S&ME, Inc., performed the geotechnical laboratory tests at their laboratory in Louisville, Tennessee, except as stated below:

- GeoTesting Express of Boxborough, Massachusetts, performed rock triaxial compressive strength tests.

- CTL Group of Skokie, Illinois, performed the X-ray examinations of rock core samples.

- GeoSystems, LLP of Kingwood, Texas, performed the petrographic examinations of rock core samples.

The S&ME laboratory performed work under their own quality program, which CH2M HILL audited and approved for work on this project. GeoTesting Express, GeoSystem, and CTL Group laboratories performed work under their own quality programs, which were audited and approved by S&ME. Laboratory tests were performed in accordance with the testing methods recommended in NRC Regulatory Guide 1.138 and the following appropriate ASTM standards.

2.5.4.2.1.5.1 Summary of Laboratory Tests Performed

The following information summarizes the types and numbers of laboratory tests performed on soil and rock samples:

- UCS tests were performed on 213 special-care rock core samples in accordance with ASTM D7012-04, Method C (Reference 2.5.4.2-209) to define the unconfined compressive strength of rock cores. Note that ASTM D7012-04 is a new standard that includes ASTM D2664, D5407, D2938, and D3148 (standards referenced in NRC Regulatory Guide 1.138). The bulk densities and moisture contents were reported with the UCS results.

- Seventy-six of the UCS tests were performed with axial and radial strain measurements. These tests allowed characterization of the elastic modulus and Poisson's ratio of the rock, as well as the unconfined compressive strength in accordance with ASTM D7012-04, Method D (Reference 2.5.4.2-209).

- Triaxial compressive tests were performed on nine rock samples to obtain rock strength parameters in accordance with ASTM D7012-04, Method A (Reference 2.5.4.2-209). These tests differed from UCS tests by application of a confining pressure surrounding the rock samples. The level of confinement was selected to represent typical in situ confining stresses. The bulk densities were reported with the results of the tests.

- Split-tensile strength tests were performed on 42 intact rock core samples in accordance with ASTM D3967-05 (Reference 2.5.4.2-210). The tensile strength test provides a measurement of the unconfined tensile strength of rock. The bulk densities and moisture contents were reported with the tensile strength test results.

- Twenty rock samples were submitted for petrographic examination to provide a detailed assessment of the lithology and mineralogy of the rocks. The examination was performed in accordance with Rock Testing Handbook (RTH) Method 102-93 (Reference 2.5.4.2-211).

- Twenty rock samples were submitted for X-ray Fluorescence examination in accordance with ASTM C1271-99 (Reference 2.5.4.2-212), to provide a detailed assessment of the chemical composition of the rocks.

- Index tests were performed on more than 100 SPT soil samples collected above the top of rock. Index tests included moisture contents (Reference 2.5.4.2-213); Atterberg limits (Reference 2.5.4.2-214); gradation, including hydrometer and wash #200 (References 2.5.4.2-215 and 2.5.4.2-216); and specific gravity (Reference 2.5.4.2-217).

- Sixteen SPT soil samples were tested for resistivity (Reference 2.5.4.2-218), 23 SPT soil samples were tested for pH (Reference 2.5.4.2-219), and 10 SPT soil samples were tested for organic content (Reference 2.5.4.2-220).

- Index and engineering tests were performed on two soil-like samples recovered from depths of 132.3 to 132.7 m (434 to 435.4 ft.) bgs in Boreholes AD-03 and AD-04. These samples were obtained by rock coring methods and stored as special-care samples. The tests were performed to provide data on the strength, compressibility, and consolidation state of a thin, non-lithified geologic layer encountered below rock. This layer was characterized by high organic content. The tests performed on the two samples included the following:

- Consolidation test in accordance with ASTM D2435-04, Method A (Reference 2.5.4.2-221).

- Consolidated-undrained triaxial compression test in accordance with ASTM D4767-04 (Reference 2.5.4.2-222).

- Atterberg limits test in accordance with ASTM D4318-05 (Reference 2.5.4.2-214).

- Gradation with hydrometer test in accordance with ASTM D422-63 (Reference 2.5.4.2-215).

- Unit weight and in situ moisture contents, including specific gravity, in accordance with ASTM D2216-06 (Reference 2.5.4.2-213) and ASTM D854-06 (References 2.5.4.2-217).

- Organic content in accordance with ASTM D2974-00, Method C (Reference 2.5.4.2-220).

Rock cores were prepared by sawing them to the required test sample length and by grinding and trimming their ends. The straightness and end flatness of the samples were checked; any conditions that did not meet the dimensional tolerances were noted in the test data sheets. Strain gages were mounted on the samples tested for modulus and Poisson's ratio. No special modifications to the testing procedures were required or applied for the laboratory tests.

Results of the laboratory tests are presented in FSAR Subsection 2.5.4.2.3.

2.5.4.2.1.5.2 Criteria for Selection of Soil Samples for Laboratory Testing

The near-surface geology at the LNP site consists of undifferentiated Quaternary and Tertiary sediments, which overlies the Avon Park Formation. Where the material could be drilled and sampled using SPT methods, it was considered to be soil or soil-like material, regardless of geologic origin.

The existing soils beneath the extents of the nuclear islands above the top of rock will be excavated for construction of the nuclear islands, while the adjacent facilities will be supported on reinforced concrete drilled piers (shafts). Depths between the base of the nuclear island and the top of rock will be backfilled with RCC and dental concrete, as discussed in FSAR Subsection 2.5.4.5. Laboratory samples of soils were collected primarily to provide data on soils that may be left in-place under nonsafety-related structures.

A soil-like deposit was encountered in the deep boreholes (AD-series boreholes) at depths greater than 121.9 m (400 ft.) bgs. Laboratory samples of the soil-like material were collected from depths of 132.3 to 132.7 m (434 to 435.4 ft.) bgs, and used primarily to provide data for consolidation settlement analysis.

2.5.4.2.1.5.3 Criteria for Selection of Rock Core Samples for Laboratory Testing

Rock core specimens were selected as “special-care” samples for laboratory tests based on the following criteria:

- Samples were collected at targeted elevation ranges, including near and below the nuclear islands basemat elevation of +3.5 m (+11.5 ft.) NAVD88.
- Samples were collected to characterize different rock types within or between boreholes, as determined by CH2M HILL field representatives.
- Samples were collected to span the range of apparent rock core soundness based on the field observations of hardness, strength, or PLT value, as determined by CH2M HILL field representatives.
- Samples were targeted for the various rock layers identified or developed based on field classifications and observations (primarily RQD and sample recovery). Identification of rock layers is presented in FSAR Subsection 2.5.4.2.2.2.

The number of UCS tests was selected to provide sufficient coverage for the determination of critical rock parameters for the various rock layers identified at the sites. In total, UCS tests were performed on 213 special-care rock core samples, 145 of which were collected within the footprints of the nuclear islands (seismic Category I structures).

2.5.4.2.2 Results of Soil and Rock Tests Obtained from Field Investigations

During the soil drilling and rock coring, various field observations and tests were performed to characterize the soil and rock engineering properties. Both quantitative and qualitative information were obtained from these tests. These field observations and tests included the following:

- SPT blow counts (N-values) and visual classifications in soil.
- RQD, R-scale strength values, PLT indices, and pressuremeter modulus in rock.

This subsection summarizes the results of these field observations and tests, as well as the criteria used for defining the transition from soil to the top of rock.

2.5.4.2.2.1 Standard Penetration Test Blow Counts (N)

This indicator of soil consistency was recorded at 0.76 to 1.5 m (2.5 to 5 ft.) depth intervals in each borehole from the ground surface to the depth of the top of rock. Tests were performed in accordance with ASTM D1586-99 (Reference 2.5.4.2-204) using automatic hammers or cathead-type hammers, for which the energy transfer efficiency tests were performed in accordance with ASTM D4633 (Reference 2.5.4.2-202).

The field-recorded N-values (uncorrected for the hammer energy transfer efficiency) were used for determining the soil layer boundaries for LNP 1 and LNP 2 using the following criteria:

- Top soil layer (S-1). This layer is defined by N-values less than 30 blows per foot (bpf). An N-value greater than 30 bpf could be included in this layer, provided that an N-value less than 30 bpf was recorded below it.
- Intermediate soil layer (S-2). This layer represents soils or soil-like material with N-values generally between 30 bpf and 50 bpf. An N-value greater than 50 bpf could be included in this layer, provided that an N-value less than 50 bpf was recorded below it. Additionally, a sample with an SPT N-value less than 30 bpf could be included in layer S-2, provided that an SPT N-value of the soil or soil-like material sample above this sample is greater than 30 bpf.
- Bottom soil layer (S-3). This layer represents the soils or soil-like material between the bottom of S-2 and the top of rock, and it generally includes soils or soil-like material with N-values greater than 50 bpf.

The recorded N-values are shown on the soil borehole logs in Appendix 2BB and on the cross-sectional plots shown in Figures 2.5.4.2-202A, 2.5.4.2-202B, 2.5.4.2-203A, and 2.5.4.2-203B. The summary statistics of the field-recorded N-values for the above soil layers are presented in Table 2.5.4.2-202.

The field-recorded N-values were also corrected for the hammer energy transfer efficiency using the following Equation 2.5.4.2-201, according to ASTM D-6066-96 (Reference 2.5.4.2-223):

$$N_{60} = \frac{Hammer\ Enery\ Transfer\ Efficiency}{60} * N_{field} \qquad \text{Equation 2.5.4.2-201}$$

where N_{60} is the SPT N-value corrected for a hammer with 60 percent energy transfer efficiency. The summary statistics of the corrected N-values for the above soil layers are presented in Table 2.5.4.2-203.

2.5.4.2.2.2 Rock Quality Designation, Rock Mass Quality, and Karst Features

RQD was recorded for each rock core run in accordance with ASTM D6032 (Reference 2.5.4.2-205) to characterize the rock soundness. RQD values are shown on the rock coring logs in Appendix 2BB and on the cross-sectional plots shown in Figures 2.5.4.2-202A, 2.5.4.2-202B, 2.5.4.2-203A, and 2.5.4.2-203B.

The recorded RQD values were grouped by elevation ranges and the summary statistics were calculated for each elevation range. The elevation ranges for rock correspond to rock layers based primarily on the shear-wave velocity values recorded during the geophysical surveys (see FSAR Subsections 2.5.4.4.2.1.1 and 2.5.4.4.2.1.2).

The rock mass quality was estimated for each core run using the criteria proposed by Sabatini et al. (Reference 2.5.4.2-224), which are based on RQD values. Table 2.5.4.2-204 summarizes the statistics of RQD and rock mass quality for the various elevation ranges (rock layers). In Table 2.5.4.2-204, the rock layer designations “SAV” and “NAV” stand for Avon Park Formation at the south and north reactor sites, respectively.

Karst features were identified by reviewing rock core logs included in Appendix 2BB for drilling observations indicative of karst features. The following criteria were used to identify the presence of karst features that include voids and soil infill:

Void was identified when the driller or geologist noted a void (i.e., void, cavity, no resistance, rod drop, or possible void) based on the drilling characteristics.

Where a karst feature did not meet above criterion for "Void", it was identified as soil infill provided it met one of the following criteria:

Soil infill was identified when soils were logged within rock cores. Additionally, the unrecovered zone logged as continuation of the soil infill in the rock core logs was considered as soil infill.

Soil infill was identified when driller commented typical drilling response of soil (i.e., soil like, soft drilling, clay, silt, sand, etc.) during rock coring and a corresponding "no recovery" zone was recorded in the rock core run (i.e., recovery is less than 100 percent).

Soil infill was identified when a "no recovery" zone was recorded in a rock core run (i.e., recovery is less than 100 percent) and a rapid drilling rate (i.e., less than or equal to 4 minutes/5-foot core run) was recorded.

Soil infill was identified when a "no recovery" zone was recorded in a rock core run (i.e., recovery is less than 100 percent) and loss of fluid circulation within the rock core run equal to or greater than 50 percent was recorded on the rock core log.

Neither voids nor soil infill were concluded when the driller indicated "no rig behavior like voids or infill" or “equipment malfunction” or slow drilling rate at the specific depths to be considered as voids or soil infill.

The above criteria were followed except for Boreholes A-14, A-18, A-21, A-22, and A-24, where the rock above the first rock core run was believed to be grinded using mud rotary drilling. The top of rock at these locations were determined based on the corresponding offset Boreholes A-14A, A-18A, A-21A, A-22A, and A-24A. Soil infill was identified when the split spoon sampler recovered soil materials during standard penetration testing within the zone above the first rock core run of Boreholes A-14, A-18, A-21, A-22, and A-24 and below the top of rock at these boreholes identified from the offset boreholes.

The results of this evaluation for LNP 1 and LNP 2 are presented in Table 2.5.4.2-205A and Table 2.5.4.2-205B, respectively. These tables list karst features encountered in boreholes at LNP 1 and LNP 2. Depth and thickness of each feature are listed and summarized at each plant location and arranged by borehole number.

The following observations are evident from the data in Table 2.5.4.2-205A and Table 2.5.4.2-205B:

- The maximum karst feature thickness encountered at the LNP 2 site was 5.9 m (19.5 ft.) in Borehole A-11 between elevations of -56.4 to -63.4 m (-188.5 to -208 ft.) NAVD88. The recovery and RQD of this karst feature was less than 10 percent and equal to 0 percent, respectively. The drilling time was between 2 and 4 minutes for each 1.5 m (5 ft.) core run.

- The thickest karst feature at LNP 1 occurred in Borehole B-30 as a zone of features between elevations -14.4 to -18.5 m (-47.3 to -60.8 ft.) NAVD88. This 4.1 m (13.5 ft.) karst feature, which likely included infilling, was conservatively considered to be a void. An additional suspect zone or a potential karst feature was encountered with 0 to 3 percent recovery and zero RQD between elevations of -48.1 to -55.2 m (-158 to -181 ft.) in Borehole A-14 that is 7 m (23 ft.) in thickness. This feature was not included in the karst features table because the drilling time did not meet the rapid drilling criterion.

Based on the results of the Offset Boring Program, it is concluded that postulated infilled features are severely weathered or degraded dolomite with properties consistent with the Avon Park Formation.

2.5.4.2.2.3 R-Scale Strength Values

This qualitative indicator of rock strength involves observation of rock core response to blows with a geologic hammer and assignment of an “R-scale” strength value based on the response, as described in FSAR Subsection 2.5.4.2.1.1.5. R-scale strength values are shown on the rock coring logs in Appendix 2BB.

The R-scale rating system (Reference 2.5.4.2-206) is summarized as follows:

- R0 (extremely weak rock). Core can be indented by thumbnail; soil-like.

- R1 (very weak rock). Core crumbles under firm blow from geologic hammer.

- R2 (weak rock). Shallow indentations made by blow from geologic hammer; core fractures under firm hammer blow but does not crumble.
- R3 (medium weak rock). Core fractures by single blow of geologic hammer.
- R4 (strong rock). Core fractures only after more than one blow from a geologic hammer.

In the ISRM guidance, these R-scale measurements are correlated with approximate ranges in compressive strength. For example, R2 rock corresponds to a range in UCS from 5 to 24 MPa (725 to 3500 psi).

Rock at the LNP site generally corresponds to R2 or better rock. However, due to the inherent variability in how these tests are performed (for example, the force of the hammer blow and interpretation of the response), the R-scale tests are only considered a semi-quantitative indication of rock strength.

2.5.4.2.2.4 Point-Load Strength Index

This quantitative field measurement of rock hardness was recorded on samples recovered from numerous depths in the boreholes. PLTs were performed in accordance with ASTM D5731 (Reference 2.5.4.2-207).

ASTM D-5731 (Reference 2.5.4.2-207) specifies that the PLT method applies to rock with compressive strength over 15 MPa (2200 psi). More than half of the UCS tests (114 of the total tests of 213), however, yielded compressive strength results of less than 15 MPa (2200 psi). Since a majority of the UCS tests did not meet the ASTM criterion, a decision was made following the field investigation that the results of PLTs would not be used for the evaluations and analyses of rock strengths.

2.5.4.2.2.5 Rock Pressuremeter Test (PMT) Modulus (Epmt)

This quantitative indicator of rock compressibility was measured at numerous depths within two boreholes (Borehole B-19 at LNP 1 and Borehole B-11 at LNP 2). In-situ Soil Testing, LC, performed the tests in accordance with the procedure described in the SIWP (Reference 2.5.4.2-201). The test involved obtaining volumetric strain versus pressure information over a 76.2 cm (30 in.) long section of the borehole wall. By conducting these tests within the borehole, a measure of the in situ compressibility of the rock was obtained. This measure of compressibility implicitly accounted for some of the internal structure of the rock and the in situ stress state of the rock.

The rock Young's modulus values estimated from the results of these tests range from 6.9 to 1689 MPa (1 to 245 kilopounds [kips] per square inch [ksi]) in Borehole B-11 and from 213 to 2171 MPa (31 to 315 ksi) in Borehole B-19, as presented in Table 2.5.4.2-206.

Note that in Borehole B-11, some of the tests were performed with only three test pressures, due to widening of the borehole and/or the presence of soft rock, weak rock, clay seams, or infill zones. This size and consistency of the borehole prevented the entire volumetric strain versus pressure range from being developed. In these tests, the Young's moduli were estimated using only limited data between the second and third test pressure results. This approach would tend to overestimate the compressibility of the material. Therefore, these modulus values should be considered as "worst case" estimates for the geotechnical analyses and evaluations.

Also in Borehole B-11, voids were reported from depths of 21.6 to 21.9 m (71 to 72 ft.) and from 22.6 to 22.7 m (74 to 74.5 ft.) bgs during the rock coring. Hence, the test result at depth of 22.5 m (74.1 ft.) bgs could not be used for establishing rock engineering properties, as the voids prevented an accurate volumetric strain versus pressure relationship from being established.

The field PMT test depth was limited by the stability of the open holes and the limits of testing equipment. The deepest rock pressuremeter testing was performed at a depth of 39 and 39.3 m (128 and 129 ft.) bgs in Boreholes B-11 and B-19, respectively, because the total drilling/coring depth of the two boreholes was 46 m (151.5 ft.) bgs and no deeper intervals were suitable for rock pressuremeter testing in these two boreholes.

2.5.4.2.2.6 Hydraulic Conductivity Tests

Sixteen monitoring wells and seven observation wells were installed at the LNP site to monitor seasonal fluctuations in groundwater elevations and to evaluate hydraulic conductivity of soil and rock. Monitoring wells were installed in accordance with the SIWP (Reference 2.5.4.2-201). Figure 2.4.12-214 shows the locations of the monitoring wells. An aquifer pumping test was performed in one well (PW-1), and the locations of the pumping and observation wells are shown on Figure 2.4.12-223. In addition, slug tests were performed in each of the 23 wells. Table 2.4.12-207 provides a summary of well construction details. Table 2.4.12-208 and Table 2.4.12-209 summarize groundwater elevations and gradients in on-site groundwater monitoring wells. Table 2.4.12-210 provides a summary of in-well slug test results, and Table 2.4.12-211 presents the pump test results. The resulting groundwater elevations, gradients, and hydraulic conductivity results are discussed in FSAR Subsection 2.4.12.2.

2.5.4.2.2.7 Criteria for Soil Depth and Top of Rock

The "soil depth" or "top of rock" was defined as the first rock core run with the following exceptions:

- The first rock core run was not classified as the top of rock if the recovery was less than 50 percent or the RQD was less than 25 percent.

- The first rock core run was not classified as the top of rock if the subsequent rock core run had recovery less than 50 percent or RQD less than 25 percent.

- When the first rock core run was not classified as the top of rock, the subsequent rock core runs were not classified as the top of rock until the rock core run with recovery of 50 percent and RQD of 25 percent was encountered.

It should be noted that engineering judgment was exercised to determine the top of rock at Boreholes A-14A, A-16, B-07A, B-17, B-22, CT-02, and CT-06 based on boring logs in Appendix 2BB, and considering the top of rock of adjacent boreholes and potential presence of cap rock in addition to above criteria.

The “top of rock” identified using above criteria is listed in Table 2.5.4.2-207. It should be noted that for Boreholes A-14, A-18, A-21, A-22, and A-24, the rock could have been grinded using mud rotary drilling to a depth below top of rock. Therefore, top of rock based on above criterion may be deeper than the actual top of rock at these Borehole locations. Offset Boreholes A-14A, A-18A, A-21A, A-22A, and A-24A were drilled a few feet away from the original Boreholes A-14, A-18, A-21, A-22, and A-24, respectively, to determine the top of rock at these boreholes. Therefore, the elevations of the top of rock of the offset boreholes were used to represent the top of rock of the original boreholes.

Different criteria were used for the determination of top of rock and karst features during the Offset Boring Program. When performing continuous SPT sampling in the overburden, top of rock was generally defined as split-spoon refusal. In one instance top of rock was determined based the amount of rig chatter and by input from the driller.

2.5.4.2.3 Results of Soil and Rock Laboratory Tests

Laboratory tests for soil and rock index and engineering properties were performed as described previously. This subsection summarizes the results of these tests.

2.5.4.2.3.1 Soil Laboratory Test Results

Index tests, consisting of Atterberg limits, gradation, in situ moisture content, specific gravity, pH and resistivity, and organic content, were performed on samples collected from the soils above the top of rock. Table 2.5.4.2-208 lists the summary statistics of each of these index properties for the soil layers defined in FSAR Subsection 2.5.4.2.2.1. The individual test results of each tested sample grouped by soil layers are provided in Appendix 2CC (Reference 2.5.4.2-225).

Index, strength, and consolidation properties were characterized for the soil-like samples collected at approximately 132.3 m (434 ft.) bgs. The summary statistics of these test results are given in Tables 2.5.4.2-209 and 2.5.4.2-210.

As described in FSAR Subsection 2.5.4.5, safety-related structures will be supported on RCC overlying Avon Park Formation. Soil will remain in place outside the nuclear island footprint between the ground surface and top of rock. The soil engineering property data can be appropriate for evaluating soil-structure interaction of the nuclear island, as needed. The soil information will also be used for the analysis of nonsafety-related structure foundations that do not affect the performance of safety-related structures.

2.5.4.2.3.2 Rock Laboratory Test Results

Results of the rock laboratory tests are presented in this subsection. These tests included UCS, secant and tangent moduli of intact rock core samples, Poisson's ratio, index properties (bulk density and moisture contents), split-tensile strengths, triaxial compressive strengths, petrographic examinations, and X-ray examinations. Individual test results of each tested rock core sample are provided in Appendix 2CC.

2.5.4.2.3.2.1 Rock Strength, Elastic Modulus, Poisson's Ratio and Index Test Results

Table 2.5.4.2-211 show the summary statistics UCS, secant and tangent moduli Poisson's ratio, and index properties for rock samples grouped by their LNP units and elevation ranges. Table 2.5.4.2-212 shows similar information for tensile strengths.

2.5.4.2.3.2.2 Petrographic Examination and X-Ray Results

Petrographic analyses were conduced on 20 samples, which were classified using Dunham's rock classification scheme (Reference 2.5.4.2-226). Based on this method, four rock types were identified which differ in composition, texture, matrix, and porosity.

- Rock Type 1 — This corresponds to sample SC-4 of Borehole A-07. The petrographic analysis indicates that the rock is a skeletal packstone with abundant carbonate grains in grain support. Moldic and vuggy porosity represent former carbonate grains removed through dissolution. Porosity in this rock is dominated by a mixture of interparticle (7.2 percent) and moldic (7.2 percent) porosity types.

- Rock Type 2 — This corresponds to sample SC-9 of Borehole A-07. The results of the petrographic analysis indicate that this sample was deposited as a skeletal packstone, but has been partially dolomitized during shallow burial. Small dolomite crystals have replaced much of the rock fabric. Carbonate fragments are in grain support and inter-particle voids are filled largely by lime mud. Foraminifera are the most abundant skeletal component. Dolomitization has not been fabric selective, and has partially obliterated some carbonate grains. Moldic and vuggy pores were formed by the dissolution of former grains, and are largely the result of foram dissolution providing for the 23.2 percent porosity of the thin section sample.

- Rock Type 3 — Six samples were used to characterize this rock type: SC-1 and SC-15 of Borehole A-08, SC-1 and SC-11 of Borehole A-19, and SC-2 and SC-8 of Borehole A-20. The results of the petrographic analysis indicate that these rocks were originally deposited as limestones with packstone and packstone to wackestone textures, but they have been completely dolomitized. Moldic pores and solution-enhanced moldic pores are common and represent former grains, which contribute to the 21.2 to 33.6 percent total thin section porosity.

- Rock Type 4 — Twelve samples were used to characterize this rock type. These samples tend to be deeper than the other rock types, and were obtained from the following locations: SC-17 of Borehole A-07, SC-25 of Borehole A-08, SC-26 and SC-19 of Borehole A-20, SC-1 and SC-7 of Borehole AD-01, SC-1 and SC-7 of Borehole AD-02, SC-3 and SC-6 of Borehole AD-03, and SC-1 and SC-3 of Borehole AD-04. The results of the petrographic analysis indicate that these samples have been completely dolomitized, but dolomitization has been replaced on a very fine scale and has preserved much of the original limestone texture. These rocks exhibit packstone, packstone to wackestone and wackestone textures. Moldic and vuggy pores represent the 20.4 to 49.6 percent porosity of the thin section.

It is noted that these rock types (rock type 1 through 4) are not related to the rock layers presented in FSAR Subsection 2.5.4.4.2.1.2.

The results of the laboratory tests indicate that the intact limestone has undergone dolomitization, as well as a diagenetic process where the calcite cement has been re-dissolved as the chemical conditions changed over time. This process lead to the development of secondary porosity that was not fabric selective, contributing to the formation of moldic and vuggy pores.

The petrographic analyses were conducted on intact rock samples. By the nature of this type of testing, interpretations regarding porosity and permeability represent behavior of the intact rock and not the secondary features that have developed over time. As noted in previous discussions, these secondary features consist of fractures and joints, and have been the cause of rock dissolution in limestone rock found at the site.

Results of petrographic examination and X-ray are provided in Appendix 2CC.

2.5.4.2.4 Rock and Soil Properties for Use in Engineering Analyses

This subsection provides a summary of rock and soil properties interpreted from the results of field and laboratory investigations and testing programs, as described previously. These rock and soil properties were used in the seismic site response analyses discussed in FSAR Subsection 2.5.2, and in the engineering analyses and evaluations presented in FSAR Subsection 2.5.4.10.

2.5.4.2.4.1 Rock Engineering Properties

Table 2.5.4.2-211 presents the rock UCS laboratory test summary statistics for intact rock samples grouped by LNP 1 and LNP 2 and by elevation range. These elevation ranges correspond to the rock layers defined for LNP 1 and LNP 2, as described in FSAR Subsection 2.5.4.4.2.1.1 and 2.5.4.4.2.1.2 respectively. The mean, standard deviation, minimum, and maximum values were calculated for each property and for each sample group. Also included in these tables are elastic modulus, Poisson's ratio, and properties of the rock samples derived as part of the in situ and laboratory strength tests.

Rock mass shear strength properties were estimated using the Hoek-Brown criteria (Reference 2.5.4.2-228). This method is dependent on both the UCS of intact rock and on the spacing, orientation, and condition of discontinuities in the rock mass as represented by the geologic strength index (GSI). The computer program RocLab (Reference 2.5.4.2-229) was used for this purpose. The following model parameters were used in RocLab:

- UCS of intact rock core samples.
- GSI.
- Material constant (m_i).
- Disturbance factor (D).

Representative rock UCS values based on average UCS values in Table 2.5.4.2-211 and average GSI values for each rock layer were used in the RocLab analysis. The Hoek-Brown criterion parameter m_i, which depends on the type of rock, is required in the RocLab analysis. Hoek and Brown (Reference 2.5.4.2-228) provided values of m_i for many types of rock. For the limestone in LNP site, an m_i value of 8, which is the lower-bound value of various types of limestone provided in Hoek and Brown (Reference 2.5.4.2-228), was used. Lower values of m_i correspond to lower estimated rock mass strength. Therefore, parameter m_i value of 8 is considered reasonable and conservative for the rock encountered in the LNP site.

A rock disturbance factor (D), which depends upon the degree of disturbance was estimated and provided in Table 2.5.4.2-213. The value of D varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses according to Hoek et al. (Reference 2.5.4.2-227). Excavation will primarily occur in soil and blasting is not likely to be necessary at the LNP site because any rock to be removed will be excavated in a manner (such as mechanical ripping) to protect the remaining rock mass from excessive damage as discussed in FSAR Subsection 2.5.4.5. However, the stress relaxation due to excavation may still cause disturbance to the in situ rock mass, such as opening of joints due to decrease of overburden stress. Therefore, a conservative disturbance factor of 0.7 was assigned to the top rock layer. Smaller disturbance factors for the remaining rock layers were assigned because the farther the rock is from the excavation, the smaller the disturbance will be. The disturbance factors are shown in Table 2.5.4.2-213.

The calculated Hoek-Brown rock mass shear strength parameters (cohesion and friction angle) are summarized in Table 2.5.4.2-213.

2.5.4.2.4.2 Rock Dynamic Properties

Table 2.5.4.2-214 presents the statistics values of V_S, compressional wave velocity (V_P), and Poisson's ratio from the Suspension P-S velocity logging conducted at LNP 1 and LNP 2. Detailed discussion of these results is included in FSAR Subsection 2.5.4.4.1.1. Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B present orthogonal fence diagram profiles showing the V_S at individual boreholes at LNP 1 and LNP 2. These data characterize low strain shear modulus of the rock mass in boreholes advanced as deep as elevation -135.3 m (-444 ft.) NAVD88. The shearing strains during the Suspension P-S velocity logging and downhole tests are usually less than 0.001 percent (Reference 2.5.4.2-230).

These dynamic rock property data were used directly in the following engineering analyses:

- Calculation of elastic modulus for evaluating the settlement of the rock subjected to loads from the nuclear island (FSAR Subsection 2.5.4.10.3.1).

- Characterization of the soil and rock model to support site-specific seismic ground response analyses (FSAR Subsections 2.5.4.7 and 2.5.2.5).

2.5.4.2.4.3 Rock Elastic Modulus Properties

The rock mass modulus (E_{rm}) is a primary design parameter for analysis of foundation deformation. This parameter has been calculated for each of the LNP 1 and LNP 2 rock layers using three independent data sources, as follows:

- V_S and Poisson ratio results from suspension logging tests (see FSAR Subsection 2.5.4.4) were used to calculate the low-strain elastic modulus for each measured depth. The low-strain elastic modulus values were reduced by 50 percent to account for strain-related effects under static loading per Mayne et al (Reference 2.5.4.2-231). Depth-weighted average values of E_{rm} were then calculated for each rock layer.

- Rock pressuremeter test results for E_{rock} (see Table 2.5.4.2-206) provided direct estimates of E_{rm}. These results were used to calculate the average E_{rm} for rock layers where test results are available.

- Rock UCS tests with strain measurements were performed on numerous samples. The secant modulus values from these tests were calculated at half of the failure stress (E_{50}). The average E_{50} for each rock layer from these tests (see Table 2.5.4.2-211), along with the average GSI for the layer (see Table 2.5.4.2-213), and a conservative disturbance factor (see Table 2.5.4.2-213), were used to calculate the modulus of the rock mass (E_{rm}) based on two different relationships. These relationships are shown in Equation 2.5.4.2-202 (Reference 2.5.4.2-232) and Equation 2.5.4.2-203 (Reference 2.5.4.2-233):

$$E_{rm} = E_{50} \times \left[0.02 + \frac{1 - D/2}{1 + e^{[(60+15*D-GSI)/11]}} \right] \qquad \text{Equation 2.5.4.2-202}$$

$$E_{rm} = \frac{E_{50}}{100} \times e^{\frac{GSI}{21.7}} \qquad \text{Equation 2.5.4.2-203}$$

Table 2.5.4.2-215 presents the resulting E_{rm} for each layer at LNP 1 and LNP 2 for each of the three data sources. As shown, the suspension logging test results represent the highest E_{rm}, whereas E_{rm} calculated from UCS tests are approximately 40 to 90 percent lower than those estimated using suspension logging data. Pressuremeter test results are not available for each layer; but where available, they represent the lowest estimates of E_{50}.

The ranges of E_{rm} for each rock layer presented in Table 2.5.4.2-215 are used as design inputs in FSAR Subsection 2.5.4.10.

2.5.4.2.4.4 Soil Index, Strength and Consolidation Properties

The engineering properties of the soils and weathered rock recovered above the top of rock were estimated using empirical relationships that relate these properties to the N-values and the index parameters obtained from laboratory tests. The following soil engineering properties were estimated:

Correlations to N-values:

- Relative density (D_r).
- Effective friction angle (ϕ').
- Poisson's ratio (v).
- Elastic Young's modulus (E) and shear modulus (G).

Correlations to index parameters:

- Critical void ratio friction angle (ϕ_{cv}').
- Over-consolidation ratio (OCR).
- Undrained shear strength (S_u).

- Compression index (C_c).
- Unloading-reloading index (C_r).
- Coefficient of secondary compression ($C_{\varepsilon\alpha}$).

The summary statistics of the estimated engineering properties are presented in the following tables:

- Table 2.5.4.2-216: Soil properties estimated from the index test results. These properties are for the fine-grained component of the soils, in that index parameters are based on correlations that are related to the plasticity of the soil. Only limited deposits of fine-grained soil are found above the top of rock, and the predominant characteristic of the soil is cohesionless with the possibility of some cementation.
- Table 2.5.4.2-217: Soil properties estimated from the N-values. These properties, such as effective friction angle, relative density, elastic modulus, etc., are for the cohesionless component of the soil above the top of rock. This is the predominant soil type at the LNP site. Note, some of these properties were estimated directly from N-value, and others were indirectly obtained through the parameters estimated from N-value.

As noted above, the soil engineering properties estimated from the index test results (see Table 2.5.4.2-216) will not be applied for the entire soil layer. This is because the soil samples used for laboratory index test were generally retrieved from isolated intervals. In general, the soil parameters presented in Table 2.5.4.2-217 for cohesionless soils should be used in geotechnical analyses. However, the soil parameters presented in Table 2.5.4.2-216 may be used if a specific borehole log is to be used for geotechnical analysis and it has cohesive soil layers shown in that specific borehole log.

There is also some uncertainty regarding the applicability of the N-value correlations for soils that had combinations of high N-values and V_S values greater than 610 m/sec (2000 ft/sec). The high shear-wave velocity suggests some degree of cementation within these soil layers, since the V_S values do not appear to be confining pressure (depth) sensitive and the magnitude of the velocity is higher than would be measured in an equivalent sand deposit at the same depth. Use of conventional N-value correlations, which were developed based on soils without cementation, may or may not correctly account for the response at low shearing strains where the cementation within soil layers has not been broken. However, once the soils are deformed or subjected to higher stresses, they would be expected to behave more consistent with a normal granular material. The response at larger strains or stresses is of primary interest for stability, bearing capacity, and settlement perspective, and therefore the normal relationships are expected to provide a reasonable and likely conservative determination of material response. Under seismic loading conditions, low-strain response as represented by the results of downhole and Suspension P-S velocity logging surveys is the primary interest.

Soils above the top of rock will not be left in place under safety-related structures, though they will remain in place around the nuclear island. Therefore, LNP site soils will not directly affect bearing capacity and settlement characteristics of safety-related structures.

2.5.4.2.4.5 Backfill Engineering Properties

Engineering properties of backfill to be placed adjacent to safety-related structures are discussed in FSAR Subsection 2.5.4.5.4.

LNP COL 2.5-6

Table 2.5.4.2-201 (Sheet 1 of 3) Summary of Boreholes Drilled within Nuclear Island and Adjacent Structures

			Borehole Information				
Location	**Nuclear Plant Site**	**Investigation Phase**	**ID**	**Easting (ft.)**	**Northing (ft.)**	**Ground Surface Elevation (ft. NAVD88)**	**Maximum Depth Explored (ft.)**
Middle of nuclear island	LNP 1	Initial	I-07	458026.5	1723097.83	42.4	307
		Main	A-16	457958.14	1723075.88	42.7	176
			A-17	458007.32	1723025.65	42.3	251
			A-19	457976.41	1723149.85	43.1	266
			A-20	458060.91	1723068.13	42.3	265
			A-21	458055.57	1723168.46	42.4	200
		Supplement	AD-03	458040.43	1723083.80	42.4	500
			AD-04	458030.47	1723034.55	42.6	500
			A-21A	458054.06	1723171.13	42.8	150
		Offset	O-1	458057.4	1723173.4	42.7	205
	LNP 2	Initial	I-02	457700.57	1724046.46	42.3	317
		Main	A-04	457634.24	1724023.64	41.3	161.5
			A-05	457680.2	1723975.26	42	161.5
			A-07	457649.39	1724100.76	42.3	266
			A-08	457734.11	1724017.18	42.1	266
			A-09	457731.35	1724113.79	41.9	201
		Supplement	AD-01	457716.32	1724033.54	42	500
			AD-02	457716.61	1723982.46	42.3	500

LNP COL 2.5-6

Table 2.5.4.2-201 (Sheet 2 of 3)
Summary of Boreholes Drilled within Nuclear Island and Adjacent Structures

Location	Nuclear Plant Site	Investigation Phase	Borehole Information: ID	Easting (ft.)	Northing (ft.)	Ground Surface Elevation (ft. NAVD88)	Maximum Depth Explored (ft.)
Immediately outside the middle of nuclear island but within footprint of nuclear island	LNP 1	Initial	I-08	458076.81	1723054.99	42.5	266
		Main	A-14	457929.76	1722999.73	42.4	223.4
			A-15	457994.33	1722937.17	42.5	202
			A-18	458047.86	1722986.91	42.3	200.5
			A-22	458088.04	1723199.82	42.6	201.5
			A-23	458146.47	1723141.37	40.8	250
			A-24	458174.25	1723114.5	40.6	160
		Supplement	A-14A	457934.54	1722992.56	42.2	111
			A-18A	458049.26	1722992.24	42.1	100.5
			A-22A	458083.35	1723191.23	42.9	121
			A-24A	458176.73	1723110.02	40.3	86.5
		Offset	O-2	457937.7	1722994.8	42.7	225
			O-3	458086.9	1723189.3	42.5	205
			O-4	458053.5	1722990.9	42.3	205
	LNP 2	Initial	I-03	457771.7	1723978.77	42.1	266
		Main	A-02	457608.03	1723946.22	41.6	251.5
			A-03	457671.79	1723884.35	42.1	201
			A-06	457719.1	1723934.54	42.5	161.5
			A-10	457766.24	1724149.29	42.2	202
			A-11	457813.34	1724091.71	42.5	285.5
			A-12	457848.86	1724065.26	42.1	165
		Offset	O-5	457769.9	1724150.2	42.6	240
			O-6	457853.4	1724065.3	42.2	205

LNP COL 2.5-6

Table 2.5.4.2-201 (Sheet 3 of 3)
Summary of Boreholes Drilled within Nuclear Island and Adjacent Structures

Location	Nuclear Plant Site	Investigation Phase	Borehole Information: ID	Easting (ft.)	Northing (ft.)	Ground Surface Elevation (ft. NAVD88)	Maximum Depth Explored (ft.)
Along the sides and within the footprint of structures adjacent to the nuclear island	LNP 1	Initial	I-09	457888.42	1722958.55	42.4	267
			I-10	458130.66	1723172.15	42	266
		Main	A-13	457933.48	1722927.1	40.6	200
			B-21	458119.39	1723224.85	41.8	152
			B-22	458287.4	1723410.27	40.5	150
			B-23	458210.12	1723150.69	40.7	150.5
			B-24	458351.54	1723356.33	40.9	150
			B-26	458111.7	1723010.2	42.4	151.5
			B-27	458154.74	1722971.12	42.4	150
			B-28	458242.56	1723060.05	41.5	150
			E-07	458250.48	1723243.73	41.7	186.5
		Supplement	B-23A	458207.72	1723147.49	42.4	70.1
	LNP 2	Initial	I-04	457585.56	1723902.28	41.6	266
			I-05	457804.46	1724148.79	42.2	266
		Main	A-01	457603.76	1723879.21	41.6	161.5
			B-06	457791.61	1724172.55	42.5	151.5
			B-07	457955.45	1724369.74	43.1	151
			B-08	457874.54	1724091.94	42.4	151
			B-09	458022.24	1724303.19	42.9	151
			B-11	457786.69	1723966.34	42.7	151.5
			B-12	457828.46	1723919.8	43.3	150
			B-13	457903.45	1723995.13	42.2	150
			E-03	457932.62	1724208.16	42	186
		Supplement	B-7A	457965.47	1724358.85	43.2	150
		Offset	B-31	457978	1724391.9	43.4	150
			B-33	457955.2	1724328.8	43	100

Notes:

ft. = foot

LNP COL 2.5-1
LNP COL 2.5-5
LNP COL 2.5-6

Table 2.5.4.2-202
Summary Statistics of Recorded SPT Blow Counts (N-Values)

Layer	Minimum Blows/ft.	Maximum Blows/ft.	Average Blows/ft.	Standard Deviation Blows/ft.
South Reactor Site (LNP 1)				
S-1	0	37	9	7
S-2	0	100	43	28
S-3	3	100	82	27
North Reactor Site (LNP 2)				
S-1	1	29	10	7
S-2	3	100	43	31
S-3	4	100	86	24

Notes:

Zero blow count or N-value was recorded when the weight of the hammer alone pushed the split-spoon sampler through the sampling interval.

For blow counts greater than 100, N-value was truncated at 100.

ft. = foot

LNP COL 2.5-6

Table 2.5.4.2-203
Summary Statistics of Energy Corrected SPT Blow Counts (N_{60}-Values)

Layer	Minimum Blows/ft.	Maximum Blows/ft.	Average Blows/ft.	Standard Deviation Blows/ft.
South Reactor Site (LNP 1)				
S-1	0	51	11	9
S-2	0	100	52	31
S-3	4	100	86	25
North Reactor Site (LNP 2)				
S-1	1	44	11	8
S-2	5	100	45	30
S-3	3	100	86	22

Notes:

Corrected SPT blow counts based on a hammer with 60% energy efficiency.

ft. = foot

LNP COL 2.5-6

Table 2.5.4.2-204
Summary of RQD Values and Rock Mass Quality

Layer	Elevation Range (NAVD88) ft.	RQD (%) Min	Max	Average	Standard Deviation	Rock Mass Quality
South Reactor (LNP 1)						
SAV-1	Top of rock to -180	0	100	36	26	Very Poor to Excellent
SAV-2	-180 to -309	0	75	13	19	Very Poor to Good
SAV-3	-309 to -458	0	86	33	22	Very Poor to Good
North Reactor (LNP 2)						
NAV-1	Top of rock to -97	0	100	56	30	Very Poor to Excellent
NAV-2	-97 to -148	0	100	54	26	Very Poor to Excellent
NAV-3	-148 to -303	0	77	18	21	Very Poor to Good
NAV-4	-303 to -458	0	95	38	23	Very Poor to Excellent

Notes:

Rock layers (elevation ranges) were determined mainly based on the recorded V_S profiles (see FSAR Subsections 2.5.4.4.2.1.1 and 2.5.4.4.2.1.2).

ft. = foot

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 1 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-16	42.7	20.3	21.0	22.4	21.7	22.1	0.70	Infill	75%	0%		Soil Seam
A-16	42.7	23.2	26.0	19.5	16.7	18.1	2.80	Infill	44%	0%		Fast drilling (2 min/5' run)
A-16	42.7	30.0	31.0	12.7	11.7	12.2	1.00	Infill	80%	25%		Fast drilling (2 min/5' run)
A-18A	42.1	30.5	35.5	11.6	6.6	9.1	5.00	Infill	0%	0%		Fast drilling (4 min/5' run), Sand layer
A-16	42.7	34.5	36.0	8.2	6.7	7.5	1.50	Infill	70%	50%	100%	Loss of circulation
A-22A	42.9	37.6	41.0	5.3	1.9	3.6	3.40	Infill	32%	9%		Fast drilling (4 min/5' run)
A-19	43.1	40.0	41.0	3.1	2.1	2.6	1.00	Infill	20%	0%		Soft at 40.0-41.0'
A-16	42.7	40.2	41.0	2.5	1.7	2.1	0.80	Infill	84%	47%		Fast drilling (4 min/5' run)
A-20	42.3	40.0	45.0	2.3	-2.7	-0.2	5.00	Infill	64%	0%		Silt (ML)
A-22A	42.9	41.0	43.0	1.9	-0.1	0.9	2.00	Infill	70%	20%		Silty Sand (SM)
B-17	42.2	40.5	46.5	1.7	-4.3	-1.3	6.00	Infill	0%	0%		Fast drilling (2 min/5' run)
B-30A	42.5	40.8	42.5	1.7	0.0	0.9	1.70	Infill	66%	57%		Fast drilling (2 min/5' run)
A-23	40.8	40.0	45.0	0.8	-4.2	-1.7	5.00	Infill	68%	0%		Silt (ML)
A-16	42.7	42.1	46.0	0.6	-3.3	-1.4	3.90	Infill	22%	0%		Fast drilling (2 min/5' run)
A-24A	40.3	40.1	41.5	0.2	-1.2	-0.5	1.40	Infill	72%	63%		Fast drilling (2 min/5' run)
A-17	42.3	42.3	42.5	0.0	-0.2	-0.1	0.20	Infill	68%	0%		Seam of Sandy Lean Clay
A-21A	42.8	43.0	45.0	-0.2	-2.2	-1.2	2.00	Infill	60%	23%		Fast drilling (3 min/5' run)
GSC-07A	43.1	43.7	46.0	-0.6	-2.9	-1.8	2.30	Infill	54%	0%		Fast drilling (2 min/5' run)
A-18	42.3	43.5	44.1	-1.2	-1.8	-1.5	0.60	Infill	NA	NA		Silt with Sand (ML), Split spoon sample
A-22A	42.9	44.5	46.0	-1.6	-3.1	-2.4	1.50	Infill	70%	20%		Fast drilling (3 min/5' run)
A-19	43.1	44.9	46.0	-1.8	-2.9	-2.4	1.10	Infill	78%	35%		Fast drilling (2 min.s/5' run)
B-27	42.4	45.1	46.0	-2.7	-3.6	-3.1	0.90	Infill	85%	45%		Silt (ML)
A-24A	40.3	43.0	46.5	-2.7	-6.2	-4.5	3.50	Infill	30%	8%		Fast drilling (1 min/5' run)
GSC-07A	43.1	46.0	48.3	-2.9	-5.2	-4.0	2.25	infill	94%	24%		Sandy Silt (ML)
GSC-09	41.3	45.0	49.0	-3.7	-7.7	-5.7	4.00	Infill	20%	0%		Fast drilling (≤4 min/5' run)
D-04	41.9	45.8	52.0	-3.9	-10.1	-7.0	6.20	Infill	16%	0%		Fast drilling (3 min/5' run)
B-27	42.4	46.5	51.0	-4.1	-8.6	-6.3	4.50	infill	66%	0%		Silt (ML)
D-05	41.8	46.1	46.7	-4.3	-4.9	-4.6	0.60	Infill	88%	35%		Fast drilling (2 min/5' run)
A-15	42.5	47.0	52.0	-4.5	-9.5	-7.0	5.00	Infill	62%	0%		Sandy Silt (SM)
A-23	40.8	46.0	50.0	-5.2	-9.2	-7.2	4.00	Infill	64%	13%		Silt with Sand (ML)
A-22	42.6	48.5	49.5	-5.9	-6.9	-6.4	1.00	Infill	NA	NA		Silty Sand with Limestone (SM), Split spoon sample
A-21A	42.8	48.9	50.0	-6.1	-7.2	-6.7	1.10	Infill	78%	15%		Fast drilling (2 min/5' run)
A-18	42.3	48.5	49.8	-6.2	-7.5	-6.9	1.30	Infill	NA	NA		Silt with Sand (ML) and Sandy Silt (ML), Split spoon sample
A-22A	42.9	49.1	51.0	-6.2	-8.1	-7.2	1.90	Infill	62%	34%		Fast drilling (3 min/5' run)
B-30	42.2	48.5	50.0	-6.3	-7.8	-7.1	1.50	Infill	70%	33%		Fast drilling (4 min/5' run)
A-20	42.3	49.2	50.0	-6.9	-7.7	-7.3	0.80	Infill	84%	22%		Fast drilling (4 min/5' run)
B-17	42.2	49.4	51.5	-7.2	-9.3	-8.2	2.15	Infill	57%	0%		Fast drilling (2 min/5' run)
A-13	40.6	47.9	50.0	-7.3	-9.4	-8.4	2.10	Infill	58%	0%		Fast drilling (4 min/5' run)
B-25	42.5	50.0	55.0	-7.5	-12.5	-10.0	5.00	Infill	0%	0%		Possible Sand layer

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 2 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
GSC-07A	43.1	50.7	51.0	-7.6	-7.9	-7.8	0.30	Infill	94%	24%		Fast drilling (3 min/5' run)
A-24A	40.3	48.9	51.5	-8.6	-11.2	-9.9	2.60	Infill	48%	15%		Fast drilling (1 min/5' run)
D-05	41.8	50.5	51.7	-8.7	-9.9	-9.3	1.20	Infill	70%	45%		Fast drilling (1 min/5' run)
D-06	41.6	50.5	51.0	-8.9	-9.4	-9.2	0.50	Infill	90%	43%		Fast drilling (3 min/5' run)
A-16	42.7	51.9	58.7	-9.2	-16.0	-12.6	6.80	Infill	50%	18%		Silt (ML)
A-13	40.6	50.0	53.3	-9.4	-12.7	-11.1	3.30	Infill	88%	13%		Silty Sand with Limestone fragments (SM)
D-04	41.9	52.0	53.0	-10.1	-11.1	-10.6	1.00	infill	60%	0%		Poorly Graded Sand (SP)
A-19	43.1	53.3	56.0	-10.2	-12.9	-11.6	2.70	Infill	46%	0%		Fast drilling (4 min/5' run)
GSC-09	41.3	51.8	54.6	-10.5	-13.3	-11.9	2.85	infill	43%	35%	100%	Very soft from 52.0-55.0'
A-22	42.6	53.5	55.0	-10.9	-12.4	-11.7	1.50	Infill	NA	NA		Sandy Silt (ML), Split spoon sample
A-21	42.4	53.5	55.0	-11.1	-12.6	-11.9	1.50	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
A-14A	42.2	53.4	61.0	-11.2	-18.8	-15.0	7.65	Infill	47%	0%		Fast drilling, sand/silt sized particles washed out
A-18	42.3	53.5	53.9	-11.2	-11.6	-11.4	0.35	Infill	NA	NA		Sandy Silt (ML), Split spoon sample
B-16	42.6	54.2	56.0	-11.6	-13.4	-12.5	1.80	Infill	64%	0%		Fast drilling (3 min/5' run)
B-27	42.4	54.7	56.0	-12.3	-13.6	-12.9	1.30	Infill	97%	43%		Silt (ML)
GSC-07A	43.1	56.3	57.7	-13.2	-14.6	-13.9	1.40	infill	98%	34%		Silt (ML)
B-30A	42.5	56.5	57.5	-14.0	-15.0	-14.5	1.00	Infill	80%	50%		Softer drilling at 52.5-57.5', fast drilling (3 min/5')
B-17	42.2	56.3	56.5	-14.1	-14.3	-14.2	0.25	Infill	95%	85%		Fast drilling (2 min/5' run)
D-05	41.8	56.0	56.7	-14.2	-14.9	-14.6	0.70	Infill	86%	75%		Fast drilling (2 min/5' run)
A-24A	40.3	55.1	56.5	-14.8	-16.2	-15.5	1.40	Infill	72%	14%		Fast drilling (3 min/5' run)
A-22	42.6	58.5	58.6	-15.9	-16.0	-16.0	0.10	Infill	NA	NA		Sandy Silt (ML), Split spoon sample
B-24	40.9	57.0	57.2	-16.1	-16.3	-16.2	0.20	infill	82%	45%		Fat Clay (CH)
A-18	42.3	58.5	58.7	-16.2	-16.4	-16.3	0.20	Infill	NA	NA		Silt with Sand (ML), Split spoon sample
A-22A	42.9	59.5	61.0	-16.6	-18.1	-17.4	1.50	Infill	70%	25%		Fast drilling (4 min/5' run)
GSC-07A	43.1	59.7	60.3	-16.6	-17.2	-16.9	0.60	infill	98%	34%		Silt (ML)
A-20	42.3	59.4	60.0	-17.1	-17.7	-17.4	0.60	Infill	94%	60%		Silty Sand (SM)
B-24	40.9	58.0	58.4	-17.1	-17.5	-17.3	0.40	infill	82%	45%		Fat Clay (CH)
A-16	42.7	59.9	61.0	-17.2	-18.3	-17.8	1.10	Infill	78%	10%		Fast drilling (3 min/5' run)
A-19	43.1	60.9	61.0	-17.8	-17.9	-17.9	0.10	Infill	98%	87%		Fast drilling (4 min/5' run)
A-16	42.7	61.0	63.0	-18.3	-20.3	-19.3	2.00	Infill	96%	43%		Silt (ML)
GSC-07A	43.1	61.4	62.6	-18.3	-19.5	-18.9	1.20	infill	98%	84%		Silt (ML)
A-24A	40.3	58.6	61.5	-18.3	-21.2	-19.8	2.90	Infill	42%	27%		Fast drilling (2 min/5' run)
GSC-09	41.3	60.4	64.0	-19.1	-22.7	-20.9	3.60	infill	30%	20%	100%	Very soft from 61.0-64.0'
D-05	41.8	61.3	61.7	-19.5	-19.9	-19.7	0.40	Infill	92%	68%		Fast drilling (3 min/5' run)
B-30A	42.5	62.1	62.5	-19.6	-20.0	-19.8	0.40	Infill	92%	47%	100%	No Recovery and Loss of Circulation
B-29	41.7	61.5	63.0	-19.8	-21.3	-20.6	1.50	Infill	52%	42%		Possible sand lense
GSC-10	42.3	62.4	63.8	-20.1	-21.5	-20.8	1.40	infill	70%	13%		Sandy Silt (ML)
A-14A	42.2	62.6	66.0	-20.4	-23.8	-22.1	3.45	Infill	31%	0%		Fast drilling (2 min/5' run)
B-19	41.3	62.0	63.0	-20.7	-21.7	-21.2	1.00	Infill	91%	88%		Cored fast (soft) at 62-63.0'
A-22	42.6	63.5	64.3	-20.9	-21.7	-21.3	0.80	Infill	NA	NA		Silt with Sand (ML), Split spoon sample

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 3 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-21	42.4	63.5	63.9	-21.1	-21.5	-21.3	0.40	Infill	NA	NA		Silt with Sand (ML), Split spoon sample
A-18	42.3	63.5	63.8	-21.2	-21.5	-21.4	0.30	Infill	NA	NA		Silt with Sand (ML), Split spoon sample
A-20	42.3	63.9	65.0	-21.6	-22.7	-22.2	1.10	infill	84%	57%		Sand with Silt (SP-SM)
GSC-07A	43.1	64.8	66.0	-21.7	-22.9	-22.3	1.20	infill	98%	84%		Silt (ML)
A-16	42.7	65.8	66.0	-23.1	-23.3	-23.2	0.20	Infill	96%	43%		Fast drilling (3 min/5' run)
GSC-09	41.3	65.0	69.2	-23.7	-27.9	-25.8	4.20	infill	100%	16%	100%	Carbonate sand (SP)
B-17	42.2	66.4	66.5	-24.2	-24.3	-24.3	0.10	Infill	98%	87%		Fast drilling (3 min/5' run)
A-13	40.6	65.0	65.8	-24.4	-25.2	-24.8	0.80	Infill	36%	10%		Silty Sand (SM)
A-24	40.6	65.0	66.1	-24.4	-25.5	-25.0	1.10	Infill	NA	NA		Silty Sand (SM) over Silt with Sand (SM), Split spoon sample
B-28	41.5	66.2	68.2	-24.7	-26.7	-25.7	2.00	void	54%	32%		Rod drop at 66.0-68.0
A-24A	40.3	66.4	66.5	-26.1	-26.2	-26.1	0.15	Infill	97%	63%		Fast drilling (4 min/5' run)
A-14	42.4	68.5	69.3	-26.1	-26.9	-26.5	0.80	Infill	NA	NA		Silty Sand with Limestone Lenses (SM), Split spoon sample
A-18	42.3	68.5	68.7	-26.2	-26.4	-26.3	0.15	Infill	NA	NA		Silt (ML), Split spoon sample
B-25	42.5	68.9	70.0	-26.4	-27.5	-27.0	1.10	Infill	78%	42%		Fast drilling (4 min/5' run)
A-16	42.7	70.4	71.0	-27.7	-28.3	-28.0	0.60	Infill	88%	50%		Fast drilling (3 min/5' run)
A-22A	42.9	70.8	71.0	-27.9	-28.1	-28.0	0.20	Infill	96%	74%		Fast drilling (3 min/5' run)
GSC-08A	43.1	72.0	72.8	-28.9	-29.7	-29.3	0.80	infill	100%	15%		Poorly Graded Sand with Silt (SP-SM)
B-19	41.3	70.8	71.5	-29.5	-30.2	-29.8	0.75	Infill	85%	83%		Very soft at 69-70.5'
A-23	40.8	70.4	70.7	-29.6	-29.9	-29.7	0.27	Infill	96%	26%		Silt (ML)
GSC-09	41.3	71.6	74.8	-30.3	-33.5	-31.9	3.20	Infill	36%	22%	100%	No Recovery and Loss of Circulation
A-22	42.6	73.5	73.8	-30.9	-31.2	-31.1	0.30	Infill	NA	NA		Elastic Silt (MH) over Silty Sand with Limestone (SM), Split spoon sample
B-20	40.4	71.5	72.3	-31.1	-31.9	-31.5	0.80	Infill	88%	88%		Silt (ML)
B-19	41.3	72.8	74.2	-31.5	-32.9	-32.2	1.40	Infill	74%	70%	25%	Extremely soft (silt), possible silt filled cavity
B-24	40.9	72.4	72.6	-31.5	-31.7	-31.6	0.20	Infill	76%	0%		Fat Clay (CH)
A-14A	42.2	74.1	76.0	-31.9	-33.8	-32.9	1.90	Infill	62%	22%		Fast drilling (4 min/5' run)
E-07	41.7	74.5	76.5	-32.8	-34.8	-33.8	2.00	Infill	60%	28%		Fast drilling (4 min/5' run)
A-22A	42.9	75.8	76.0	-32.9	-33.1	-33.0	0.20	Infill	96%	62%		Fast drilling (3 min/5' run)
A-16	42.7	75.9	76.0	-33.2	-33.3	-33.3	0.10	Infill	98%	67%		Fast drilling (4 min/5' run)
B-26	42.4	76.5	81.5	-34.1	-39.1	-36.6	5.00	Infill	0%	0%		Soft drilling, possible unconsolidated material
B-19	41.3	75.6	75.8	-34.3	-34.5	-34.4	0.20	Infill	74%	70%	25%	Silt with Limestone Fragments (ML)
A-19	43.1	77.5	78.5	-34.4	-35.4	-34.9	1.00	Infill	67%	30%		Fast drilling (3 min/5' run)
A-24A	40.3	75.8	76.5	-35.5	-36.2	-35.9	0.70	Infill	86%	51%		Fast drilling (2 min/5' run)
E-06	42.8	78.3	81.0	-35.5	-38.2	-36.9	2.70	Infill	46%	33%		Fast drilling (4 min/5' run)
A-18	42.3	78.5	79.7	-36.2	-37.4	-36.8	1.20	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
A-16	42.7	79.0	81.0	-36.3	-38.3	-37.3	2.00	Infill	60%	47%		Fast drilling (2 min/5' run)
A-22A	42.9	80.0	81.0	-37.1	-38.1	-37.6	1.00	Infill	80%	26%		Fast drilling (2 min/5' run)
A-19	43.1	80.4	81.0	-37.3	-37.9	-37.6	0.65	Infill	67%	30%		Fast drilling (3 min/5' run)
B-28	41.5	78.8	80.0	-37.3	-38.5	-37.9	1.20	Infill	76%	0%		Fast drilling (4 min/5' run)
B-17	42.2	79.6	81.5	-37.4	-39.3	-38.4	1.90	Infill	62%	10%		Fast drilling (3 min/5' run)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 4 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
GSC-08A	43.1	81.2	82.0	-38.1	-38.9	-38.5	0.80	Infill	84%	83%		Drilling soft intermittently at about 78'
D-04	41.9	80.9	81.1	-39.0	-39.2	-39.1	0.20	Infill	46%	0%		Limey Clay (CL)
A-24A	40.3	80.3	81.5	-40.0	-41.2	-40.6	1.25	Infill	75%	46%		Fast drilling (2 min/5' run)
GSC-12	41.0	81.3	85.0	-40.3	-44.0	-42.1	3.75	Infill	25%	10%		Fast drilling (3 min/5' run)
E-08	42.4	82.8	85.0	-40.4	-42.6	-41.5	2.20	void	56%	38%	80%	No Recovery and Loss of Circulation
A-14	42.4	83.5	83.6	-41.1	-41.2	-41.1	0.05	Infill	NA	NA		Silty Sand with Limestone (SM), Split spoon sample
A-21	42.4	83.5	84.3	-41.1	-41.9	-41.5	0.80	Infill	NA	NA		Silty Gravelly Sand (SM), Split spoon sample
A-18	42.3	83.5	84.8	-41.2	-42.5	-41.9	1.30	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
E-06	42.8	84.0	86.0	-41.2	-43.2	-42.2	2.00	Infill	60%	37%		Fast drilling (4 min/5' run)
B-30A	42.5	84.5	85.5	-42.0	-43.0	-42.5	1.00	void	82%	53%	100%	Rod drop at 84.5-85.5'
GSC-09	41.3	83.4	90.0	-42.1	-48.7	-45.4	6.60	infill	0%	0%	100%	85.0-90.0' is Sand
A-21A	42.8	85.9	88.6	-43.1	-45.8	-44.5	2.70	Infill	46%	15%	100%	No Recovery and Loss of Circulation
E-06	42.8	86.0	89.7	-43.2	-46.9	-45.1	3.70	void	26%	7%	20%	86-89.5' very soft, possible void
E-05	42.6	85.9	90.5	-43.3	-47.9	-45.6	4.60	Infill	8%	0%		Fast drilling (2 min/5' run)
GSC-08A	43.1	86.8	87.0	-43.7	-43.9	-43.8	0.20	Infill	96%	67%		Fast drilling (4 min/5' run)
B-22	40.5	85.0	87.5	-44.5	-47.0	-45.8	2.50	Infill	32%	20%		Soft 85.0-87.5'
B-30	42.2	87.0	88.0	-44.8	-45.8	-45.3	1.00	void	88%	37%	100%	Void 87-88'
B-30A	42.5	87.5	87.9	-45.0	-45.4	-45.2	0.40	void	24%	13%	100%	Rod drop at 87.5-87.9'
B-20	40.4	85.6	86.5	-45.2	-46.1	-45.7	0.90	Infill	82%	50%		Fast drilling (4 min/5' run)
A-22A	42.9	88.7	91.0	-45.8	-48.1	-47.0	2.30	Infill	54%	12%	50%	No Recovery and Loss of Circulation
A-22	42.6	88.5	89.8	-45.9	-47.2	-46.6	1.30	Infill	NA	NA		Silty Sand with Limestone (SM), Split spoon sample
A-21	42.4	88.5	89.5	-46.1	-47.1	-46.6	1.00	Infill	NA	NA	100%	Silt (ML) over Silty Gravelly Sand (SM), Split spoon sample
A-24A	40.3	86.5	86.5	-46.2	-46.2	-46.2	0.05	Infill	99%	44%		Fast drilling (2 min/5' run)
A-18	42.3	88.5	89.1	-46.2	-46.8	-46.5	0.60	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
B-30A	42.5	89.1	90.5	-46.6	-48.0	-47.3	1.40	Infill	24%	13%	100%	Fast drilling (2 min/5' run)
B-24	40.9	87.9	90.0	-47.0	-49.1	-48.1	2.10	Infill	58%	35%	90%	No Recovery and Loss of Circulation
B-19	41.3	88.5	90.0	-47.2	-48.7	-48.0	1.50	Infill	70%	70%	100%	Very soft at 88.5-90.0'
B-30	42.2	89.5	103.0	-47.3	-60.8	-54.1	13.50	void	0%	0%	100%	Void 89.5-91', 91-95.0', open, 95-103.0' rod drop
A-23	40.8	88.2	89.5	-47.4	-48.7	-48.1	1.30	Infill	74%	64%		Soft at 88.2-90.0'
B-22	40.5	88.0	90.0	-47.5	-49.5	-48.5	2.00	void	32%	20%		No resistance at 88.0-90.0'
A-18	42.3	90.0	91.0	-47.7	-48.7	-48.2	1.00	Infill	60%	47%		Fast drilling (4 min/5' run)
GSC-08A	43.1	91.0	92.0	-47.9	-48.9	-48.4	1.00	Infill	80%	58%	100%	Loss of circulation
B-30A	42.5	90.5	92.5	-48.0	-50.0	-49.0	2.00	void	24%	13%	100%	Rod drop at 90.5-92.5'
D-04	41.9	89.9	90.0	-48.0	-48.1	-48.1	0.10	Infill	98%	59%	100%	No Recovery and Loss of Circulation
GSC-11	42.9	91.0	95.0	-48.1	-52.1	-50.1	4.00	Infill	20%	0%		Fast drilling (3 min/5' run)
B-21	41.8	89.9	92.0	-48.1	-50.2	-49.2	2.10	Infill	58%	25%	100%	No Recovery and Loss of Circulation
E-06	42.8	91.7	92.2	-48.9	-49.4	-49.2	0.50	infill	70%	15%		Clay (CL) at 91.7-92.2
GSC-08	43.2	92.3	92.7	-49.1	-49.5	-49.3	0.40	Infill	66%	38%		Silt (ML)
GSC-09	41.3	90.5	93.0	-49.2	-51.7	-50.5	2.50	Infill	20%	0%		90.5-93.0' very soft
B-22	40.5	90.2	95.0	-49.7	-54.5	-52.1	4.80	void	4%	0%		No resistance at 90.0-95.0'

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 5 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-30A	42.5	92.5	93.5	-50.0	-51.0	-50.5	1.00	Infill	80%	45%		Fast drilling (1 min/5' run)
B-19	41.3	91.5	91.9	-50.2	-50.6	-50.4	0.40	infill	56%	9%		Silt (ML)
E-06	42.8	93.5	93.9	-50.7	-51.1	-50.9	0.40	infill	70%	15%		Silt (ML) 93.5 - 93.9
A-14	42.4	93.5	93.6	-51.1	-51.2	-51.2	0.10	Infill	NA	NA		Sandy Silt With Limestone (ML), Split spoon sample
A-22A	42.9	94.4	96.0	-51.5	-53.1	-52.3	1.60	Infill	68%	18%		Fast drilling (4 min/5' run)
GSC-08A	43.1	94.7	97.0	-51.6	-53.9	-52.8	2.30	Infill	54%	13%	100%	No Recovery and Loss of Circulation
GSC-09	41.3	93.5	95.0	-52.2	-53.7	-53.0	1.50	Infill	20%	0%		94.0-95.0' very soft
A-24	40.6	93.0	95.0	-52.4	-54.4	-53.4	2.00	Infill	60%	35%	100%	No Recovery
B-21	41.8	94.2	97.0	-52.4	-55.2	-53.8	2.80	Infill	44%	15%	100%	No Recovery and Loss of Circulation
A-14A	42.2	95.1	96.0	-52.9	-53.8	-53.4	0.90	Infill	82%	7%	100%	No Recovery and Loss of Circulation
B-19	41.3	94.3	96.5	-53.0	-55.2	-54.1	2.20	Infill	56%	9%		Soft at 96-96.5'
D-04	41.9	95.0	96.0	-53.1	-54.1	-53.6	1.00	void	30%	6%		Possible void 95.0-96.0'
B-20	40.4	93.9	94.0	-53.5	-53.6	-53.6	0.10	Infill	68%	40%		Calcareous Silt (ML)
E-06	42.8	96.7	102.0	-53.9	-59.2	-56.6	5.30	void	14%	11%	100%	Possible void, core barrel drop to 102
D-04	41.9	96.0	98.5	-54.1	-56.6	-55.4	2.50	Infill	30%	6%		Very soft drilling 96.0-98.5'
B-29	41.7	96.0	96.4	-54.3	-54.7	-54.5	0.40	Infill	98%	38%		Silty Sand (SM)
A-22A	42.9	97.4	101.0	-54.5	-58.1	-56.3	3.60	Infill	28%	0%		Fast drilling (3 min/5' run)
E-08	42.4	97.0	100.0	-54.6	-57.6	-56.1	3.00	Infill	40%	20%	100%	No Recovery and Loss of Circulation
A-21A	42.8	97.6	100.0	-54.8	-57.2	-56.0	2.40	Infill	52%	15%		Fast drilling (4 min/5' run)
GSC-08	43.2	98.4	100.0	-55.2	-56.8	-56.0	1.60	Infill	68%	47%	100%	No Recovery and Loss of Circulation
B-22	40.5	96.3	100.0	-55.8	-59.5	-57.7	3.70	Infill	26%	0%		Fast drilling (3 min/5' run)
GSC-12	41.0	97.0	98.5	-56.0	-57.5	-56.8	1.50	Infill	82%	60%		Soft Drilling
A-21	42.4	98.5	99.3	-56.1	-56.9	-56.5	0.80	Infill	NA	NA		Silty Gravelly Sand (SM), Split spoon sample
B-19	41.3	98.0	101.5	-56.7	-60.2	-58.5	3.50	void	30%	8%		No resistance
GSC-08	43.2	100.0	101.0	-56.8	-57.8	-57.3	1.00	Infill	94%	60%		Fat Clay (CH) over Silt (ML)
B-30A	42.5	100.2	102.5	-57.7	-60.0	-58.9	2.30	Infill	54%	28%		Fast drilling (1 min/5' run)
B-17	42.2	100.0	101.5	-57.8	-59.3	-58.6	1.50	Infill	70%	42%	100%	No Recovery and Loss of Circulation
B-20	40.4	98.7	101.5	-58.3	-61.1	-59.7	2.80	Infill	46%	0%		Fast drilling (3 min/5' run)
B-21	41.8	100.5	104.0	-58.7	-62.2	-60.5	3.50	void	2%	0%		Cavity at 100.5-104.0'
GSC-08A	43.1	102.0	104.0	-58.9	-60.9	-59.9	2.00	Infill	88%	60%		Fast drilling (4 min/5' run), Silt (ML)
B-22	40.5	100.0	100.5	-59.5	-60.0	-59.8	0.50	infill	30%	0%		Silty Clay (CL)
B-30A	42.5	102.5	105.1	-60.0	-62.6	-61.3	2.60	Infill	48%	22%		Fast drilling (2 min/5' run)
E-06	42.8	103.0	106.0	-60.2	-63.2	-61.7	3.00	Infill	20%	0%		Fast drilling (4 min/5' run)
A-16	42.7	103.1	104.2	-60.4	-61.5	-61.0	1.10	Infill	94%	62%		Silt (ML)
A-21A	42.8	103.2	105.0	-60.4	-62.2	-61.3	1.80	Infill	64%	18%		Fast drilling (3 min/5' run)
B-24	40.9	101.4	105.0	-60.5	-64.1	-62.3	3.60	Infill	28%	0%		Fast drilling (4 min/5' run)
GSC-09	41.3	101.9	102.2	-60.6	-60.9	-60.8	0.30	infill	58%	20%		Clay with Limestone (CL)
A-22	42.6	103.5	105.0	-60.9	-62.4	-61.7	1.50	Infill	NA	NA		Silty Sand with Limestone (SM), Split spoon sample
B-22	40.5	101.5	105.0	-61.0	-64.5	-62.8	3.50	Infill	30%	0%		Fast drilling (3 min/5' run)
E-07	41.7	102.7	106.5	-61.0	-64.8	-62.9	3.80	Infill	24%	15%		Fast drilling (3 min/5' run)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 6 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-21	42.4	103.5	103.9	-61.1	-61.5	-61.3	0.40	Infill	NA	NA		Silt with Sand and Gravel (ML), Split spoon sample
A-22A	42.9	104.2	106.0	-61.3	-63.1	-62.2	1.80	Infill	64%	40%		Fast drilling (3 min/5' run)
B-29	41.7	103.2	106.0	-61.5	-64.3	-62.9	2.80	infill	44%	31%	100%	Fast drilling (3 min/5' run)
B-30	42.2	103.7	105.0	-61.5	-62.8	-62.2	1.30	Infill	14%	0%		Fast drilling (≤ 4 min/5' run)
GSC-08	43.2	104.7	110.0	-61.5	-66.8	-64.2	5.30	Infill	0%	0%		Fast drilling (2 min/5' run)
B-17	42.2	103.8	106.5	-61.6	-64.3	-63.0	2.70	Infill	46%	20%	100%	Fast drilling (4 min/5' run)
B-20	40.4	102.3	106.5	-61.9	-66.1	-64.0	4.20	Infill	16%	0%		Fast drilling (4 min/5' run)
B-21	41.8	104.1	107.0	-62.3	-65.2	-63.8	2.90	infill	2%	0%		Fast drilling (≤ 4 min/5' run)
GSC-11	42.9	105.5	108.3	-62.6	-65.4	-64.0	2.80	infill	40%	28%		Soft zone at 107.0' for 1.0 to 1.5'
B-25	42.5	106.0	107.0	-63.5	-64.5	-64.0	1.00	infill	90%	43%		106-107' soft drilling
GSC-12	41.0	105.5	106.5	-64.5	-65.5	-65.0	1.00	Infill	75%	26%		Soft drilling
B-30	42.2	108.2	115.0	-66.0	-72.8	-69.4	6.80	Infill	0%	0%		Fast drilling (4 min/5' run)
GSC-12	41.0	107.0	108.0	-66.0	-67.0	-66.5	1.00	Infill	75%	26%		Soft at 107.0-108.0'
B-20	40.4	106.5	108.0	-66.1	-67.6	-66.9	1.50	void	24%	10%		Void space 106.5-108.0'
GSC-07A	43.1	109.3	111.0	-66.2	-67.9	-67.1	1.70	Infill	66%	0%		Fast drilling (4 min/5' run)
GSC-08	43.2	110.0	113.0	-66.8	-69.8	-68.3	3.00	void	38%	37%		Rod drop 3 ft. at 110'
E-07	41.7	109.3	111.5	-67.6	-69.8	-68.7	2.20	Infill	56%	40%		Fast drilling (2 min/5' run)
B-29	41.7	109.4	111.0	-67.7	-69.3	-68.5	1.60	Infill	68%	46%	100%	Fast drilling (3 min/5' run)
B-17	42.2	111.0	111.5	-68.8	-69.3	-69.1	0.50	Infill	90%	40%		Fast drilling (4 min/5' run)
B-20	40.4	109.2	111.5	-68.8	-71.1	-70.0	2.30	Infill	24%	10%		Fast drilling (2 min/5' run)
GSC-08A	43.1	112.1	113.5	-69.0	-70.4	-69.7	1.40	infill	6%	0%		Silt (ML)
E-06	42.8	112.0	112.6	-69.2	-69.8	-69.5	0.60	infill	42%	0%		Silt (ML)
B-19	41.3	110.8	111.5	-69.5	-70.2	-69.8	0.75	Infill	85%	66%		Soft at 111.0-111.5'
B-29	41.7	111.7	116.0	-70.0	-74.3	-72.2	4.30	infill	14%	0%	100%	Fast drilling (3 min/5' run)
GSC-11	42.9	113.1	114.0	-70.2	-71.1	-70.7	0.90	Infill	82%	56%		Loose drilling at 111.0'
E-06	42.8	113.1	116.0	-70.3	-73.2	-71.8	2.90	void	42%	0%		Possible void
GSC-08A	43.1	113.5	120.0	-70.4	-76.9	-73.7	6.50	void	6%	0%		3.5' of Void at 113.5-117', Rod lowered to 120'
B-19	41.3	112.0	116.5	-70.7	-75.2	-73.0	4.50	Infill	98%	98%		Soft at 112.0-116.5'
A-22A	42.9	113.7	116.0	-70.8	-73.1	-72.0	2.30	Infill	54%	0%		Fast drilling (3 min/5' run)
B-22	40.5	111.4	115.0	-70.9	-74.5	-72.7	3.60	Infill	28%	7%		Fast drilling (2 min/5' run)
A-21	42.4	113.5	115.0	-71.1	-72.6	-71.9	1.50	Infill	NA	NA		Silt with Sand (SM), Split spoon Sample
GSC-09	41.3	112.5	115.0	-71.2	-73.7	-72.4	2.55	Infill	49%	23%		Fast drilling (4 min/5' run)
E-05	42.6	114.3	115.5	-71.7	-72.9	-72.3	1.20	Infill	76%	38%	100%	No Recovery and Loss of Circulation
GSC-08	43.2	115.0	115.3	-71.8	-72.1	-72.0	0.30	Infill	20%	10%		Sand (SW)
A-16	42.7	115.1	116.0	-72.4	-73.3	-72.9	0.90	Infill	82%	33%		Fast drilling (3 min/5' run)
A-14	42.4	115.0	115.5	-72.6	-73.1	-72.9	0.50	void	96%	82%	100%	Void 115-115.5'
GSC-07A	43.1	115.7	116.0	-72.6	-72.9	-72.8	0.30	Infill	94%	46%		Fast drilling (3 min/5' run)
A-20	42.3	115.0	115.5	-72.7	-73.2	-73.0	0.50	Infill	78%	37%		Silty Sand (SM)
GSC-08	43.2	116.0	120.0	-72.8	-76.8	-74.8	4.00	Infill	20%	10%		Fast drilling (3 min/5' run)
B-21	41.8	114.7	117.0	-72.9	-75.2	-74.1	2.30	infill	55%	22%	100%	drilling very soft but not like void

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 7 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-24	40.9	113.9	115.0	-73.0	-74.1	-73.6	1.10	Infill	78%	23%		Fast drilling (4 min/5' run)
E-07	41.7	115.0	116.5	-73.3	-74.8	-74.1	1.50	Infill	70%	43%		Fast drilling (3 min/5' run)
B-20	40.4	114.6	116.5	-74.2	-76.1	-75.2	1.90	Infill	62%	25%		Fast drilling (2 min/5' run)
A-19	43.1	117.5	120.0	-74.4	-76.9	-75.7	2.50	void	50%	17%	100%	Possible void from 117.5 to 120.0'
GSC-09	41.3	116.0	125.0	-74.7	-83.7	-79.2	9.00	Infill	20%	7%		Fast drilling (3 min/5' run)
A-21A	42.8	118.0	120.0	-75.2	-77.2	-76.2	2.00	Infill	60%	18%		Fast drilling (4 min/5' run)
B-22	40.5	116.0	120.4	-75.5	-79.9	-77.7	4.40	Infill	30%	17%		Silt and Sand with Clay
E-07	41.7	117.5	121.5	-75.8	-79.8	-77.8	4.00	Infill	20%	13%		Fast drilling (3 min/5' run)
A-20	42.3	118.2	120.0	-75.9	-77.7	-76.8	1.80	infill	78%	37%		Silty Sand (SM)
A-22A	42.9	118.9	121.0	-76.0	-78.1	-77.1	2.10	Infill	58%	34%		Fast drilling (4 min/5' run)
A-21	42.4	118.5	119.6	-76.1	-77.2	-76.7	1.10	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
B-21	41.8	118.6	122.0	-76.8	-80.2	-78.5	3.45	Infill	31%	8%		Soft zones
GSC-08	43.2	120.0	121.5	-76.8	-78.3	-77.6	1.50	Infill	30%	0%		Sandy Silt (ML)
GSC-08A	43.1	120.0	123.5	-76.9	-80.4	-78.7	3.50	infill	0%	0%		Felt like drilling sediment
B-29	41.7	118.8	121.0	-77.1	-79.3	-78.2	2.20	Infill	56%	27%	100%	Fast drilling (4 min/5' run)
GSC-10	42.3	119.6	121.0	-77.3	-78.7	-78.0	1.40	Infill	72%	13%	100%	No Recovery, Loss of circulation
A-23	40.8	118.3	120.0	-77.5	-79.2	-78.4	1.70	Infill	66%	8%		Fast drilling (4 min/5' run)
E-06	42.8	120.9	121.0	-78.1	-78.2	-78.2	0.10	Infill	98%	70%		Fast drilling (4 min/5' run)
GSC-08	43.2	121.5	125.0	-78.3	-81.8	-80.1	3.50	void	30%	0%		120-125.0' Rod dropped
GSC-11	42.9	121.4	123.5	-78.5	-80.6	-79.6	2.10	infill	48%	20%		121.0-122.5' Soft
B-30	42.2	121.0	125.0	-78.8	-82.8	-80.8	4.00	Infill	20%	0%		Fast drilling (4 min/5' run)
A-14	42.4	121.4	121.6	-79.0	-79.2	-79.1	0.20	void	72%	20%	100%	Small void 121.4-121.6'
B-17	42.2	121.3	121.5	-79.1	-79.3	-79.2	0.20	Infill	96%	85%		Fast drilling (3 min/5' run)
B-19	41.3	120.5	121.5	-79.2	-80.2	-79.7	1.00	infill	99%	78%		Soft at 120.5-121.5'
A-18	42.3	122.0	125.0	-79.7	-82.7	-81.2	3.00	void	40%	18%	50-80%	Possible void (rod drop to 125') 122.0-125.0'
B-25	42.5	122.4	125.0	-79.9	-82.5	-81.2	2.60	Infill	48%	11%		Soft at 123.5-124.0'
GSC-08A	43.1	123.5	126.0	-80.4	-82.9	-81.7	2.50	infill	58%	0%		Elastic Silt (MH)
GSC-11	42.9	123.5	124.0	-80.6	-81.1	-80.9	0.50	void	48%	20%		123.5' slipped down
B-20	40.4	121.3	121.5	-80.9	-81.1	-81.0	0.20	Infill	96%	60%		Fast drilling (1 min/5' run)
A-21	42.4	123.5	123.8	-81.1	-81.4	-81.3	0.30	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
B-21	41.8	123.1	125.5	-81.3	-83.7	-82.5	2.40	Infill	22%	14%		Numerous soft zones
B-26	42.4	123.7	124.2	-81.3	-81.8	-81.5	0.45	infill	58%	19%		Silty Sand (SM)
B-22	40.5	122.1	125.0	-81.6	-84.5	-83.1	2.90	Infill	42%	20%		Fast drilling (3 min/5' run)
A-21A	42.8	124.5	125.0	-81.7	-82.2	-82.0	0.50	Infill	90%	57%		Fast drilling (4 min/5' run)
A-14	42.4	124.7	125.8	-82.3	-83.4	-82.9	1.10	void	50%	16%	100%	Void at 124.7-125.8'
GSC-12	41.0	123.4	125.0	-82.4	-84.0	-83.2	1.60	infill	68%	40%	100%	No Recovery and Loss of Circulation
E-07	41.7	125.0	126.5	-83.3	-84.8	-84.1	1.50	Infill	70%	48%		Fast drilling (3 min/5' run)
GSC-08A	43.1	126.4	127.0	-83.3	-83.9	-83.6	0.60	void	58%	0%		Last foot had slow and fast sections (likely 6" void)
GSC-11	42.9	126.4	129.0	-83.5	-86.1	-84.8	2.60	infill	48%	31%		Soft at 124.0-127.0'
B-21	41.8	125.5	128.0	-83.7	-86.2	-85.0	2.50	void	22%	14%		Cavity at 125.5-128.0'

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 8 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
GSC-09	41.3	125.0	125.5	-83.7	-84.2	-84.0	0.50	infill	34%	0%		Carbonate Sand with Silt (SP-SM)
B-29	41.7	125.6	126.0	-83.9	-84.3	-84.1	0.40	Infill	92%	36%	100%	No Recovery and Loss of Circulation
B-29	41.7	126.1	131.0	-84.4	-89.3	-86.9	4.90	infill	2%	0%	100%	Fast drilling (3 min/5' run)
GSC-08	43.2	128.5	130.0	-85.3	-86.8	-86.1	1.50	Infill	70%	52%		Fast drilling (4 min/5' run)
B-22	40.5	125.9	130.0	-85.4	-89.5	-87.5	4.10	Infill	18%	12%		Fast drilling (2 min/5' run)
GSC-09	41.3	126.7	130.0	-85.4	-88.7	-87.1	3.30	Infill	34%	0%		Fast drilling (3 min/5' run)
B-28	41.5	127.0	127.8	-85.5	-86.3	-85.9	0.80	void	84%	22%		Rod drop at 127-127.5'
B-20	40.4	126.4	126.5	-86.0	-86.1	-86.1	0.10	Infill	98%	43%		Fast drilling (4 min/5' run)
A-21	42.4	128.5	129.6	-86.1	-87.2	-86.7	1.10	Infill	NA	NA		Silty Gravelly Sand (SM), Split spoon sample
A-18	42.3	128.5	129.1	-86.2	-86.8	-86.5	0.60	Infill	66%	40%		Soft Drilling
E-07	41.7	127.9	131.5	-86.2	-89.8	-88.0	3.60	Infill	28%	13%		Fast drilling (2 min/5' run)
B-21	41.8	128.3	132.0	-86.5	-90.2	-88.4	3.70	Infill	27%	7%		Several soft zones, probably not cavities
GSC-08	43.2	130.0	131.3	-86.8	-88.1	-87.5	1.30	Infill	92%	8%		Sandy Silt (ML)
A-19	43.1	130.2	130.8	-87.1	-87.7	-87.4	0.60	Infill	88%	82%		Softer at 130.0' and below
A-18	42.3	129.8	131.0	-87.5	-88.7	-88.1	1.20	Infill	66%	40%		Soft drilling 128.5', and 130.0' below
B-20	40.4	128.0	131.5	-87.6	-91.1	-89.4	3.50	Infill	30%	8%		Fast drilling (4 min/5' run)
A-22	42.6	130.5	131.5	-87.9	-88.9	-88.4	1.00	Infill	80%	48%	100%	No Recovery and Loss of Circulation
E-05	42.6	131.0	135.5	-88.4	-92.9	-90.7	4.50	infill	8%	7%		Soft material throughout run, fast drilling
GSC-08A	43.1	131.8	132.0	-88.7	-88.9	-88.8	0.20	Infill	96%	45%		Fast drilling (4 min/5' run)
A-14	42.4	131.2	133.4	-88.8	-91.0	-89.9	2.20	Infill	56%	9%	100%	No Recovery and Loss of Circulation
A-18	42.3	131.2	131.5	-88.9	-89.2	-89.1	0.30	Infill	53%	40%		Soft drilling 131.2-131.5'
B-19	41.3	130.4	131.5	-89.1	-90.2	-89.6	1.15	Infill	77%	45%		Soft at 129.5-130.0'
A-24	40.6	130.0	130.7	-89.4	-90.1	-89.8	0.70	Infill	40%	0%		Carbonate Derived Silty Sand (SM)
B-22	40.5	130.0	131.6	-89.5	-91.1	-90.3	1.60	Infill	60%	0%		Silty Clay (CL) over Poorly Graded Sand (SP)
GSC-12	41.0	130.5	135.0	-89.5	-94.0	-91.8	4.50	infill	10%	0%	100%	Very soft 131.5-134.0'
GSC-11	42.9	132.7	134.0	-89.8	-91.1	-90.5	1.28	Infill	74%	26%		Fast drilling (4 min/5' run)
A-18	42.3	132.7	133.6	-90.4	-91.3	-90.9	0.90	Infill	53%	40%		Soft drilling 132.7-133.6'
B-21	41.8	132.6	137.0	-90.8	-95.2	-93.0	4.40	void	12%	8%		Cavity from 133.5-135.0', and from 135.5'-136.0
A-14	42.4	133.4	135.4	-91.0	-93.0	-92.0	2.00	Infill	28%	0%	100%	Very soft drilling 133.4-135.4'
A-21	42.4	133.5	133.8	-91.1	-91.4	-91.2	0.25	Infill	NA	NA		Silty Sand with Gravel (SM), Split spoon sample
B-20	40.4	131.5	132.0	-91.1	-91.6	-91.4	0.50	Infill	60%	22%		Silt (ML)
A-13	40.6	132.1	135.0	-91.5	-94.4	-93.0	2.90	void	42%	0%	100%	Soft drilling, possible void
B-29	41.7	133.9	136.0	-92.2	-94.3	-93.3	2.10	Infill	58%	7%	100%	Fast drilling (4 min/5' run)
E-06	42.8	135.0	138.0	-92.2	-95.2	-93.7	3.00	void	40%	0%		Core barrel drop to 138
B-28	41.5	134.0	135.0	-92.5	-93.5	-93.0	1.00	infill	26%	7%		Soft drilling 134.0-135.0'
A-22	42.6	135.5	136.5	-92.9	-93.9	-93.4	1.00	Infill	80%	53%	100%	No Recovery and Loss of Circulation
B-27	42.4	136.0	136.5	-93.6	-94.1	-93.8	0.50	Infill	50%	16%		Silty Sand
B-17	42.2	136.0	136.5	-93.8	-94.3	-94.1	0.50	Infill	90%	48%		Fast drilling (4 min/5' run)
E-05	42.6	136.6	140.5	-94.0	-97.9	-96.0	3.90	Infill	22%	0%		Fast drilling (4 min/5' run)
B-20	40.4	134.5	136.5	-94.1	-96.1	-95.1	2.00	Infill	60%	22%		Fast drilling (3 min/5' run)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 9 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
GSC-11	42.9	137.3	139.0	-94.5	-96.2	-95.3	1.70	Infill	66%	30%		Soft at 137.0-138.0'
GSC-10	42.3	137.1	141.0	-94.8	-98.7	-96.8	3.90	Infill	22%	7%		Fast drilling (4 min/5' run)
A-14	42.4	137.4	138.4	-95.0	-96.0	-95.5	1.00	Infill	28%	0%	100%	Soft drilling 137.4-138.4'
B-27	42.4	138.5	141.0	-96.1	-98.6	-97.3	2.50	Infill	50%	16%		Soft material at 138.0'
GSC-11	42.9	140.5	144.0	-97.6	-101.1	-99.4	3.50	infill	30%	0%		Very soft from 141.5-143.5'
A-20	42.3	140.0	140.5	-97.7	-98.2	-98.0	0.50	Infill	80%	25%	100%	Silt (ML)
B-29	41.7	140.1	141.0	-98.4	-99.3	-98.9	0.90	Infill	82%	28%	100%	No Recovery and Loss of Circulation
A-18	42.3	142.5	146.0	-100.2	-103.7	-102.0	3.50	Infill	30%	13%		Fast drilling (3 min/5' run)
B-29	41.7	143.0	146.0	-101.3	-104.3	-102.8	3.00	Infill	40%	33%		Very soft at 143.3-145.0'
B-21	41.8	143.7	145.5	-101.9	-103.7	-102.8	1.80	void	66%	48%		Possible cavity from 143.5-145.5
A-22	42.6	145.2	146.5	-102.6	-103.9	-103.3	1.30	Infill	74%	35%	100%	No Recovery and Loss of Circulation
B-19	41.3	145.2	146.5	-103.9	-105.2	-104.6	1.30	Infill	74%	54%		Soft at 144.5-145.0'
B-28	41.5	147.0	150.0	-105.5	-108.5	-107.0	3.00	infill	58%	0%		Soft drilling 147.0-150.0'
B-19	41.3	150.5	151.5	-109.2	-110.2	-109.7	1.00	Infill	80%	35%		Soft at 149.5-151.5'
GSC-12	41.0	153.9	155.0	-112.9	-114.0	-113.5	1.10	Infill	78%	48%	75%	No Recovery and Loss of Circulation
E-08	42.4	158.6	160.0	-116.2	-117.6	-116.9	1.40	Infill	72%	23%	100%	No Recovery and Loss of Circulation
GSC-12	41.0	158.5	159.0	-117.5	-118.0	-117.8	0.50	Infill	95%	34%		Soft drilling at 158.5-159.0'
GSC-08	43.2	160.8	165.0	-117.6	-121.8	-119.7	4.20	Infill	16%	0%		Fast drilling (2 min/5' run)
E-07	41.7	159.5	161.5	-117.8	-119.8	-118.8	2.00	Infill	60%	20%		Fast drilling (4 min/5' run)
A-22	42.6	161.5	166.5	-118.9	-123.9	-121.4	5.00	Infill	0%	0%		Fast drilling (2 min/5' run)
E-05	42.6	162.1	165.5	-119.5	-122.9	-121.2	3.40	Infill	32%	28%		Fast drilling (4 min/5' run)
GSC-12	41.0	161.0	162.0	-120.0	-121.0	-120.5	1.00	Infill	74%	52%		Very soft at 161.0-162.0'
A-16	42.7	165.3	166.0	-122.6	-123.3	-123.0	0.70	Infill	86%	37%		Fast drilling (4 min/5' run)
A-21	42.4	169.0	171.5	-126.6	-129.1	-127.9	2.50	Infill	50%	22%		Significant circulation loss
A-20	42.3	178.6	180.0	-136.3	-137.7	-137.0	1.40	Infill	72%	23%		Fast drilling (4 min/5' run)
A-20	42.3	180.0	180.4	-137.7	-138.1	-137.9	0.40	Infill	90%	13%		Silty Sand (SM)
A-19	43.1	183.0	184.0	-139.9	-140.9	-140.4	1.00	Infill	96%	14%		Soft at 183.0-184.0'
A-21	42.4	189.9	191.5	-147.5	-149.1	-148.3	1.60	Infill	68%	0%	100%	Loss of circulation
A-21	42.4	195.1	196.5	-152.7	-154.1	-153.4	1.35	void	72%	7%	100%	Void at 195.5'
A-13	40.6	196.8	200.0	-156.2	-159.4	-157.8	3.20	Infill	36%	16%	100%	No Recovery and Loss of Circulation
A-21	42.4	198.8	200.3	-156.4	-157.9	-157.2	1.50	Infill	61%	9%		Fast drilling (3 min/5' run)
A-17	42.3	200.3	201.0	-158.0	-158.7	-158.4	0.70	Infill	86%	67%		Fast drilling (4 min/5' run)
AD-04	42.6	220.5	221.0	-177.9	-178.4	-178.2	0.50	Infill	90%	56%		Loss of circulation
GSC-07A	43.1	229.2	229.9	-186.1	-186.8	-186.5	0.70	infill	92%	35%		Clay with Silt (CL-ML)
AD-04	42.6	233.0	236.0	-190.4	-193.4	-191.9	3.00	void	40%	0%		Possible void space and fast drilling (3 min/5')
A-20	42.3	236.0	240.0	-193.7	-197.7	-195.7	4.00	Infill	20%	0%		Fast drilling (4 min/5' run)
AD-04	42.6	239.2	241.0	-196.6	-198.4	-197.5	1.80	Infill	64%	32%		Fast drilling (3 min/5' run)
AD-04	42.6	242.2	246.0	-199.6	-203.4	-201.5	3.80	void	24%	0%		Potential cavity or silt infill
A-20	42.3	245.8	250.0	-203.5	-207.7	-205.6	4.20	Infill	16%	0%		Fast drilling (2 min/5' run)
AD-04	42.6	247.0	251.0	-204.4	-208.4	-206.4	4.00	void	20%	0%	50%	Potential cavity or silt zone

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205A (Sheet 10 of 10)
Summary of Karst Features Encountered in Boreholes at South Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (8ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-23	40.8	248.0	250.0	-207.2	-209.2	-208.2	2.00	Infill	60%	0%		Fast drilling (4 min/5' run)
A-20	42.3	250.9	255.0	-208.6	-212.7	-210.7	4.10	Infill	18%	0%		Fast drilling (4 min/5' run)
A-20	42.3	255.0	257.2	-212.7	-214.9	-213.8	2.20	Infill	54%	0%		Poorly Graded Sand (SP) and Silt (ML)
AD-04	42.6	259.8	261.0	-217.2	-218.4	-217.8	1.25	Infill	75%	15%		Fast drilling (4 min/5' run)
AD-04	42.6	261.0	266.0	-218.4	-223.4	-220.9	5.00	void	0%	0%	50%	Rapid advancement (possible void or silt)
AD-04	42.6	266.0	271.0	-223.4	-228.4	-225.9	5.00	Infill	0%	0%		Rapid advancement (possible void or silt)
AD-04	42.6	271.3	274.0	-228.7	-231.4	-230.0	2.75	Infill	25%	0%		Unconsolidated nature
AD-03	42.4	324.0	329.0	-281.6	-286.6	-284.1	5.00	Infill	42%	0%		Silty Sand (SM)
AD-03	42.4	329.0	331.3	-286.6	-288.9	-287.8	2.30	Infill	88%	40%		Sandy Silt (ML)
AD-03	42.4	334.0	339.0	-291.6	-296.6	-294.1	5.00	void	0%	0%	100%	Possible void space, very soft material
AD-03	42.4	342.0	344.3	-299.6	-301.9	-300.8	2.30	infill	34%	0%	100%	Sandy Silt (ML)
AD-03	42.4	347.4	349.0	-305.0	-306.6	-305.8	1.60	infill	74%	32%		Silt (ML)
AD-03	42.4	354.0	354.4	-311.6	-312.0	-311.8	0.40	infill	100%	24%		Sandy Silt (ML)
AD-03	42.4	354.8	356.0	-312.4	-313.6	-313.0	1.20	infill	100%	24%		Silt (ML)
AD-04	42.6	390.0	390.3	-347.4	-347.7	-347.6	0.30	infill	100%	0%		Clay (CL)
AD-04	42.6	411.3	411.7	-368.7	-369.1	-368.9	0.40	infill	80%	26%		Clay (CL)
AD-04	42.6	418.7	419.2	-376.1	-376.6	-376.4	0.50	infill	96%	50%		Clayey Silt (ML)
AD-04	42.6	424.5	425.2	-381.9	-382.6	-382.3	0.70	void	58%	12%		Bit drops at 424.4' (0.7'), Clay
AD-04	42.6	434.0	435.4	-391.4	-392.8	-392.1	1.40	infill	100%	0%		Clayey Silt (ML)
AD-04	42.6	446.0	447.8	-403.4	-405.2	-404.3	1.80	void	56%	20%		Bit drop 2.0'
AD-04	42.6	449.0	451.0	-406.4	-408.4	-407.4	2.00	void	32%	0%		Void at top of run
AD-04	42.6	452.0	454.0	-409.4	-411.4	-410.4	2.00	void	32%	0%		1.0' of drilling in middle of void near bottom
AD-03	42.4	459.0	459.2	-416.6	-416.8	-416.7	0.20	Infill	84%	0%		Silt (ML)
AD-03	42.4	461.8	462.0	-419.4	-419.6	-419.5	0.20	Infill	84%	0%		Fine Sand (SP)
AD-03	42.4	462.8	463.0	-420.4	-420.6	-420.5	0.25	infill	84%	0%		Silty Sand (SM)
AD-03	42.4	477.0	479.0	-434.6	-436.6	-435.6	2.00	Infill	94%	24%		Breccia
AD-03	42.4	480.7	484.0	-438.3	-441.6	-440.0	3.30	Infill	74%	26%		Loss of circulation
AD-03	42.4	488.8	489.0	-446.4	-446.6	-446.5	0.25	infill	95%	43%	100%	Lost circulation in large cavity

Note:

The karst feature intervals estimated by driller during drilling may be adjusted based on the no recovery intervals in the rock core logs.

ft. = foot
NAVD88 = North American Vertical Datum 1988

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 1 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	MINIMUM	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-11	42.5	38.0	38.2	4.5	4.3	4.4	0.20	Infill	96%	93%		Soft at 38.0'
B-08	42.4	41.0	43.7	1.4	-1.3	0.0	2.70	Infill	54%	33%		Fast drilling (4 minimum/5' run)
D-03	42.0	40.7	42.9	1.3	-0.9	0.2	2.20	Infill	74%	50%		Fast drilling (4 minimum/5' run)
B-13	42.2	41.7	45.0	0.5	-2.8	-1.2	3.30	Infill	34%	20%		Fast drilling (3 minimum/5' run)
A-11	42.5	43.4	45.5	-0.9	-3.0	-2.0	2.10	Infill	58%	35%		Fast drilling (3 minimum/5' run)
D-03	42.0	42.9	46.4	-0.9	-4.4	-2.7	3.50	infill	80%	0%		Sand with Silt (SM)
A-12	42.1	43.2	45.0	-1.1	-2.9	-2.0	1.80	Infill	64%	8%		Very soft drilling at 43.5'
D-02	41.3	44.5	45.0	-3.2	-3.7	-3.5	0.50	Infill	90%	86%		Fast drilling (4 minimum/5' run)
B-08	42.4	46.0	48.3	-3.6	-5.9	-4.8	2.30	Infill	46%	24%		Fast drilling (2 minimum/5' run)
A-09	41.9	46.2	49.0	-4.3	-7.1	-5.7	2.80	Infill	50%	27%	100%	Very easy drilling and sandy interval
A-11	42.5	48.5	50.5	-6.0	-8.0	-7.0	2.00	Infill	60%	23%		Fast drilling (3 minimum/5' run)
A-12	42.1	48.4	50.0	-6.3	-7.9	-7.1	1.60	Infill	68%	47%		Fast drilling (3 minimum/5' run)
B-13	42.2	49.4	50.0	-7.2	-7.8	-7.5	0.60	Infill	88%	69%		46-48.0' very soft
E-01	40.9	48.5	51.0	-7.6	-10.1	-8.9	2.50	Infill	100%	57%		Silty Sand (SM)
A-03	42.1	50.3	51.0	-8.2	-8.9	-8.6	0.70	Infill	86%	77%		Fast drilling (4 minimum/5' run)
B-14	41.7	49.9	51.0	-8.2	-9.3	-8.8	1.10	Infill	78%	62%		Fast drilling (2 minimum/5' run)
GSC-05	41.3	50.9	51.0	-9.6	-9.7	-9.7	0.10	Infill	98%	38%		Fast drilling (3 minimum/5' run)
B-11	42.7	54.5	56.5	-11.8	-13.8	-12.8	2.00	Infill	60%	8%		Soft and fast drilling (3 minimum/5' run)
B-13	42.2	54.1	55.0	-11.9	-12.8	-12.4	0.90	Infill	82%	72%		52.5-53.0' and 53.5-54.5' soft
A-12	42.1	54.2	55.0	-12.1	-12.9	-12.5	0.80	Infill	84%	68%		Fast drilling (4 minimum/5' run)
B-14	41.7	55.3	56.0	-13.6	-14.3	-14.0	0.70	Infill	86%	57%		Fast drilling (2 minimum/5' run)
GSC-05	41.3	55.6	56.0	-14.3	-14.7	-14.5	0.45	Infill	91%	66%		Fast drilling (3 minimum/5' run)
GSC-02	40.4	55.5	56.0	-15.1	-15.6	-15.3	0.55	Infill	89%	58%		Fast drilling (2 minimum/5' run)
E-01	40.9	56.0	56.6	-15.1	-15.7	-15.4	0.60	Infill	80%	42%		Silty Sand (SM)
E-02	39.8	55.8	56.0	-16.0	-16.2	-16.1	0.20	Infill	96%	69%		Fast drilling (4 minimum/5' run)
B-08	42.4	60.3	61.0	-17.9	-18.6	-18.3	0.70	Infill	86%	45%		Fast drilling (3 minimum/5' run)
B-14	41.7	60.0	61.0	-18.3	-19.3	-18.8	1.00	Infill	80%	58%		Fast drilling (2 minimum/5' run)
GSC-05	41.3	59.9	61.0	-18.6	-19.7	-19.2	1.10	Infill	78%	30%		Fast drilling (4 minimum/5' run)
B-13	42.2	62.0	64.0	-19.8	-21.8	-20.8	2.00	Infill	69%	55%		62.0-64.0' very soft
E-02	39.8	60.5	61.0	-20.7	-21.2	-21.0	0.50	Infill	90%	50%		Fast drilling (3 minimum/5' run)
B-09	42.9	64.4	66.0	-21.5	-23.1	-22.3	1.60	Infill	66%	8%		Fast drilling (3 minimum/5' run)
B-03	43.9	65.9	66.0	-22.0	-22.1	-22.1	0.10	Infill	98%	65%	100%	No Recovery and Loss of Circulation
E-04	43.1	65.5	66.0	-22.4	-22.9	-22.7	0.50	Infill	89%	63%		Fast drilling (2 minimum/5' run)
B-12	43.3	66.0	68.0	-22.7	-24.7	-23.7	2.00	Infill	64%	40%		Very soft at 66-68.0'
A-10	42.2	65.0	67.0	-22.8	-24.8	-23.8	2.00	Infill	68%	36%		Soft 65-66.0', Fast drilling (3 minimum/5' run)
B-02	41.8	65.0	65.5	-23.2	-23.7	-23.5	0.50	Infill	90%	75%		Fast drilling (3 minimum/5' run)
B-11	42.7	66.0	67.5	-23.3	-24.8	-24.1	1.50	Infill	66%	37%		Soft at 66.0-67.0' and 68.0-68.5'
E-03	42.0	65.3	66.0	-23.3	-24.0	-23.7	0.70	Infill	86%	25%		Fast drilling (3 minimum/5' run)
GSC-06	42.5	65.8	66.5	-23.3	-24.0	-23.7	0.70	Infill	86%	32%		Fast drilling (2 minimum/5' run)
B-04A	42.0	65.5	68.6	-23.5	-26.6	-25.1	3.10	Infill	38%	19%		65.5-67.0', 68.0-68.5' very soft (silt lenses)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 2 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-14	41.7	65.6	66.0	-23.9	-24.3	-24.1	0.40	Infill	92%	75%		Fast drilling (3 min/5' run)
A-06	42.5	66.5	68.2	-24.0	-25.7	-24.8	1.65	void	76%	40%		Many cavities or lost material from coring
GSC-02	40.4	64.5	66.0	-24.1	-25.6	-24.9	1.50	Infill	70%	44%		Fast drilling (3 min/5' run)
B-10	42.0	66.3	66.5	-24.3	-24.5	-24.4	0.20	Infill	100%	68%	50%	Silt and Limestone Fragments (ML)
GSC-05	41.3	65.9	66.0	-24.6	-24.7	-24.7	0.10	Infill	98%	9%		Fast drilling (4 min/5' run)
D-01	40.8	67.7	71.0	-26.9	-30.2	-28.6	3.30	infill	67%	25%	100%	Silt and Limestone Interbeds (ML)
B-13	42.2	69.4	70.0	-27.2	-27.8	-27.5	0.60	Infill	88%	26%		69.5-70.0' very soft, fast drilling
B-15	42.3	69.7	71.0	-27.4	-28.7	-28.1	1.30	Infill	74%	53%		Very soft
E-04	43.1	70.9	71.0	-27.8	-27.9	-27.9	0.10	Infill	98%	90%		Fast drilling (4 min/5' run)
B-11	42.7	71.0	72.0	-28.3	-29.3	-28.8	1.00	void	70%	37%		71-72.0' void
B-14	41.7	70.1	71.0	-28.4	-29.3	-28.9	0.90	Infill	82%	48%		Fast drilling (3 min/5' run)
B-10	42.0	70.4	71.5	-28.4	-29.5	-29.0	1.10	Infill	78%	66%		Fast drilling (3 min/5' run)
GSC-06	42.5	71.1	71.5	-28.6	-29.0	-28.8	0.45	Infill	91%	52%		Fast drilling (3 min/5' run)
E-01	40.9	69.7	71.0	-28.8	-30.1	-29.4	1.35	Infill	73%	52%		Fast drilling (4 min/5' run)
D-01	40.8	71.0	73.6	-30.2	-32.8	-31.5	2.60	Infill	48%	34%		Carbonate Silt
A-09	41.9	72.5	75.0	-30.6	-33.1	-31.9	2.50	Infill	66%	38%	100%	Soft drilling from 72.5' to 75.0'
A-02	41.6	72.3	72.8	-30.7	-31.2	-31.0	0.50	Infill	95%	84%		Silt (ML)
A-03	42.1	72.9	73.9	-30.8	-31.8	-31.3	1.00	Infill	74%	53%	30%	Silt seam from 72.9' to 73.9'
B-13	42.2	73.1	75.0	-30.9	-32.8	-31.9	1.90	Infill	62%	16%		Soft at 71.5-72.0' and very soft at 73.0-74.5'
E-02	39.8	70.8	71.0	-31.0	-31.2	-31.1	0.25	Infill	95%	85%		Fast drilling (3 min/5' run)
A-11	42.5	73.5	75.0	-31.0	-32.5	-31.8	1.50	Infill	72%	40%		Got soft at 73.5, hard again at 75.0'
B-01	40.8	72.0	75.0	-31.2	-34.2	-32.7	3.05	Infill	39%	17%		Soft from 72-74.0', 3 min/5' run
B-11	42.7	74.0	74.5	-31.3	-31.8	-31.6	0.50	void	70%	37%		74-74.5' void
A-10	42.2	74.4	76.3	-32.2	-34.1	-33.2	1.90	Infill	94%	52%		Silt (ML)
A-12	42.1	74.9	77.0	-32.8	-34.9	-33.9	2.10	Infill	70%	26%	80%	Soft drilling from 75' to 77', Silt (ML), Loss of Circulation
E-04	43.1	75.9	76.0	-32.8	-32.9	-32.9	0.10	Infill	98%	78%		Fast drilling (4 min/5' run)
GSC-03	40.5	73.6	75.0	-33.1	-34.5	-33.8	1.40	Infill	72%	67%		72.0-72.5' and 73.0-74.5' soft
B-14	41.7	75.0	76.0	-33.3	-34.3	-33.8	1.00	void	80%	38%		Cavities
A-05	42.0	75.6	76.9	-33.6	-34.9	-34.2	1.25	Infill	88%	55%		Calcareous Silty Fat Clay (CH)
B-12	43.3	77.0	77.5	-33.7	-34.2	-34.0	0.50	void	42%	22%		No resistance felt - 77.0-77.5
B-09	42.9	76.7	78.6	-33.8	-35.7	-34.7	1.95	Infill	62%	43%	100%	No Recovery and Loss of Circulation
A-06	42.5	76.9	78.3	-34.5	-35.8	-35.1	1.35	Infill	86%	20%		Fat Clay (CH)
GSC-05	41.3	75.8	76.0	-34.5	-34.7	-34.6	0.20	Infill	96%	50%		Fast drilling (2 min/5' run)
B-11	42.7	77.4	77.7	-34.7	-35.0	-34.9	0.30	Infill	54%	43%		Silt (ML)
B-12	43.3	78.0	78.2	-34.7	-34.9	-34.8	0.20	void	42%	22%		No resistance felt - 78.0-78.2'
A-10	42.2	77.0	77.5	-34.8	-35.3	-35.1	0.50	Infill	94%	15%		Sand (SW)
A-03	42.1	77.3	77.5	-35.2	-35.4	-35.3	0.20	Infill	49%	27%		Fat Clay to Highly Plastic Silt (CH)
A-02	41.6	77.0	77.3	-35.4	-35.7	-35.6	0.30	Infill	100%	60%		Silt (ML)
GSC-03	40.5	76.0	77.0	-35.5	-36.5	-36.0	1.00	void	44%	14%		76.0-77.0' void

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 3 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
E-02	39.8	75.5	76.0	-35.7	-36.2	-36.0	0.50	Infill	90%	48%		Fast drilling (3 min/5' run)
A-02	41.6	78.2	78.3	-36.6	-36.7	-36.6	0.10	Infill	100%	60%		Silt (ML)
E-04	43.1	79.7	81.0	-36.6	-37.9	-37.2	1.35	Infill	73%	50%	100%	No Recovery and Loss of Circulation
B-12	43.3	80.0	80.2	-36.7	-36.9	-36.8	0.20	Infill	42%	22%		Silt (ML)
A-08	42.1	78.9	81.0	-36.8	-38.9	-37.9	2.10	Infill	64%	23%		Clay (CL)
D-01	40.8	78.0	80.0	-37.2	-39.2	-38.2	2.00	Infill	60%	28%		Fast drilling (4 min/5' run)
GSC-06	42.5	79.7	81.5	-37.2	-39.0	-38.1	1.80	Infill	64%	25%	50%	No Recovery and Loss of Circulation
B-01	40.8	78.2	80.0	-37.4	-39.2	-38.3	1.80	Infill	64%	10%		Soft from 76.5-77.0'
A-01	41.6	79.5	79.9	-37.9	-38.3	-38.1	0.35	Infill	77%	28%		Lean Clay - Elastic Silt (CL-ML)
E-03	42.0	80.0	81.0	-38.0	-39.0	-38.5	1.00	Infill	80%	53%		Fast drilling (3 min/5' run)
GSC-05	41.3	79.5	81.0	-38.2	-39.7	-39.0	1.50	Infill	70%	23%		Fast drilling (4 min/5' run)
E-01	40.9	79.5	79.7	-38.6	-38.8	-38.7	0.20	Infill	98%	83%		Silt (ML)
B-14	41.7	80.4	81.0	-38.7	-39.3	-39.0	0.60	Infill	88%	45%		Fast drilling (3 min/5' run)
A-10	42.2	81.1	81.4	-38.9	-39.2	-39.1	0.30	Infill	94%	15%		Silt (ML)
GSC-06	42.5	81.5	82.0	-39.0	-39.5	-39.3	0.50	void	76%	28%	100%	Void at 81.5-82.0'
D-01	40.8	80.0	80.3	-39.2	-39.5	-39.4	0.30	infill	60%	28%		Fat Clay (CH)
B-13	42.2	81.5	82.5	-39.3	-40.3	-39.8	1.00	Infill	96%	82%		81.5-82.5' soft, Clay (CL) at 81.2-81.3'
A-09	41.9	81.2	81.5	-39.3	-39.6	-39.5	0.30	Infill	94%	63%	100%	Soft near bottom of run at 81'
GSC-03	40.5	80.0	82.0	-39.5	-41.5	-40.5	2.00	void	25%	0%		80.0-82.0' void
E-02	39.8	79.9	81.0	-40.1	-41.2	-40.7	1.10	Infill	78%	63%		Fast drilling (4 min/5' run)
B-11	42.7	83.8	86.5	-41.1	-43.8	-42.5	2.70	Infill	46%	10%		Soft at 82-82.5', 83-83.5', 84.5-85'
GSC-03	40.5	82.0	83.0	-41.5	-42.5	-42.0	1.00	Infill	25%	0%		82.0-83.0' soft
B-03	43.9	85.8	86.0	-41.9	-42.1	-42.0	0.20	Infill	96%	90%	100%	No Recovery and Loss of Circulation
E-04	43.1	85.2	86.0	-42.1	-42.9	-42.5	0.80	Infill	84%	27%		Fast drilling (4 min/5' run)
GSC-04	40.0	82.3	85.0	-42.3	-45.0	-43.7	2.70	Infill	46%	8%	100%	No circulation below 80.0'
D-01	40.8	83.3	83.4	-42.5	-42.6	-42.5	0.10	Infill	94%	70%		Carbonate Silt (ML)
A-10	42.2	85.0	87.0	-42.8	-44.8	-43.8	2.00	Infill	60%	58%		Fast drilling (4 min/5' run)
B-02	41.8	84.9	85.5	-43.1	-43.7	-43.4	0.60	Infill	88%	82%	5-10%	Fast drilling (3 min/5' run)
B-01	40.8	84.2	85.0	-43.4	-44.2	-43.8	0.80	Infill	84%	38%	100%	No Recovery and Loss of Circulation
B-14	41.7	85.2	86.0	-43.5	-44.3	-43.9	0.80	void	84%	78%		Fast drilling (3 min/5' run), Cavities
GSC-03	40.5	84.0	85.0	-43.5	-44.5	-44.0	1.00	void	25%	0%		84.0-85.0' void
E-03	42.0	85.7	86.0	-43.7	-44.0	-43.9	0.30	Infill	94%	63%		Fast drilling (3 min/5' run)
A-09	41.9	86.1	86.5	-44.2	-44.6	-44.4	0.40	Infill	92%	67%	100%	No Recovery and Loss of Circulation
B-12	43.3	87.5	88.5	-44.2	-45.2	-44.7	1.00	Infill	62%	20%		Soft zones
B-04A	42.0	87.0	88.1	-45.0	-46.1	-45.6	1.10	Infill	78%	41%		87-87.5' soft and Fat Clay (CH)
B-13	42.2	88.5	89.5	-46.3	-47.3	-46.8	1.00	Infill	80%	71%	100%	88.5-89.5' soft
A-09	41.9	88.3	91.0	-46.4	-49.1	-47.8	2.70	Infill	40%	0%	100%	No Recovery and Loss of Circulation
B-01	40.8	87.2	90.0	-46.4	-49.2	-47.8	2.80	Infill	44%	18%	100%	86-87' silty clay, still no circulation
B-04A	42.0	89.0	90.0	-47.0	-48.0	-47.5	1.00	Infill	78%	41%		Soft 89-90.0'
GSC-03	40.5	88.2	90.0	-47.7	-49.5	-48.6	1.80	Infill	64%	22%		Various soft spots

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 4 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-11	42.7	90.5	91.5	-47.8	-48.8	-48.3	1.00	Infill	80%	58%		Fast drilling (4 min/5' run)
B-14	41.7	90.5	91.0	-48.8	-49.3	-49.1	0.50	Infill	90%	20%		Fast drilling (4 min/5' run)
E-01	40.9	90.3	91.0	-49.4	-50.1	-49.8	0.70	Infill	86%	80%	100%	Loss of circulation
B-13	42.2	91.7	91.9	-49.5	-49.7	-49.6	0.20	Infill	77%	53%	95%	Silty Clay (CL-ML)
D-01	40.8	90.6	91.0	-49.8	-50.2	-50.0	0.40	Infill	92%	50%		Fast drilling (4 min/5' run)
E-02	39.8	89.6	91.0	-49.8	-51.2	-50.5	1.40	Infill	72%	6%		Fast drilling (4 min/5' run)
B-04A	42.0	91.9	95.0	-49.9	-53.0	-51.5	3.10	Infill	38%	25%		Fast drilling (3 min/5' run), no recovery
A-05	42.0	92.3	92.6	-50.3	-50.7	-50.5	0.35	Infill	86%	48%		Calcareous Fat Clay (CH)
A-10	42.2	93.2	95.2	-51.0	-53.0	-52.0	2.00	Infill	80%	48%		Silt (ML)
B-01	40.8	92.0	93.0	-51.2	-52.2	-51.7	1.00	Infill	46%	29%		92-93' silty clay
B-13	42.2	93.8	95.0	-51.6	-52.8	-52.2	1.20	Infill	77%	53%	95%	Clay (CL)
A-04	41.3	93.1	93.3	-51.8	-52.0	-51.9	0.20	Infill	99%	50%		Fat Clay to Elastic Silt (CH)
A-06	42.5	94.3	94.4	-51.8	-51.9	-51.9	0.10	Infill	90%	58%		Fat Clay at 94.25' - 94.35'
B-11	42.7	95.2	95.4	-52.5	-52.7	-52.6	0.20	Infill	88%	29%		Clayey seam at 95.2-95.4 (CL)
B-05	42.9	95.9	96.0	-53.0	-53.1	-53.1	0.10	Infill	98%	98%		Fast drilling (4 min/5' run)
B-01	40.8	94.0	94.5	-53.2	-53.7	-53.5	0.50	void	46%	29%		94-94.5 possible void
GSC-03	40.5	94.0	95.0	-53.5	-54.5	-54.0	1.05	void	79%	46%	100%	94.5-94.8' void
E-01	40.9	94.4	94.8	-53.5	-53.9	-53.7	0.40	void	96%	83%		Cavities at 94.4-94.6' and 94.6-94.8'
E-02	39.8	93.4	93.7	-53.6	-53.9	-53.7	0.25	infill	90%	59%		Elastic Silt (MH)
B-14	41.7	96.2	101.0	-54.5	-59.3	-56.9	4.80	Infill	4%	0%		Fast drilling (3 min/5' run)
A-07	42.3	97.0	100.0	-54.7	-57.7	-56.2	3.00	Infill	40%	0%		Sand lense 97.0' - 100.0'
B-01	40.8	96.5	100.0	-55.7	-59.2	-57.5	3.50	Infill	38%	0%		Silt (ML)
GSC-03	40.5	96.3	100.0	-55.8	-59.5	-57.7	3.70	Infill	26%	0%	100%	95.0-95.5' soft
B-08	42.4	99.8	101.0	-57.4	-58.6	-58.0	1.20	Infill	76%	69%		Fast drilling (4 min/5' run)
GSC-04	40.0	98.4	98.5	-58.4	-58.5	-58.4	0.05	Infill	90%	74%	100%	Clay (CL)
B-02	41.8	100.3	100.5	-58.5	-58.7	-58.6	0.20	Infill	96%	93%		Fast drilling (4 min/5' run)
A-08	42.1	101.0	101.4	-58.9	-59.3	-59.1	0.40	Infill	100%	46%		Poorly Graded Sand (SP)
GSC-04	40.0	99.5	100.0	-59.5	-60.0	-59.8	0.50	Infill	90%	74%	100%	No Recovery and Loss of Circulation
GSC-03	40.5	100.5	105.0	-60.0	-64.5	-62.3	4.50	Infill	10%	0%	100%	Fast drilling (3 min/5' run)
B-12	43.3	103.4	105.0	-60.1	-61.7	-60.9	1.60	Infill	68%	26%		Fast drilling (4 min/5' run)
B-04A	42.0	103.4	105.0	-61.4	-63.0	-62.2	1.60	Infill	68%	26%		Fast drilling (4 min/5' run)
E-03	42.0	104.4	106.0	-62.4	-64.0	-63.2	1.60	Infill	68%	19%		Fast drilling (4 min/5' run)
B-13	42.2	105.0	107.5	-62.8	-65.3	-64.1	2.50	Infill	50%	16%		Suspect sand 105-107.5'
B-09	42.9	105.8	106.0	-62.9	-63.1	-63.0	0.20	Infill	96%	42%	90-95%	Fast drilling (4 min/5' run)
A-10	42.2	105.3	107.0	-63.1	-64.8	-63.9	1.75	Infill	65%	38%		Fast drilling (4 min/5' run)
GSC-05	41.3	105.3	106.0	-64.0	-64.7	-64.4	0.70	Infill	86%	18%		Fast drilling (2 min/5' run)
B-14	41.7	105.8	106.0	-64.1	-64.3	-64.2	0.20	Infill	96%	80%		Fast drilling (3 min/5' run)
GSC-04	40.0	104.4	105.0	-64.4	-65.0	-64.7	0.60	Infill	88%	37%	100%	No Recovery and Loss of Circulation
A-01	41.6	106.3	106.5	-64.7	-64.9	-64.8	0.20	Infill	96%	86%		Fast drilling (3 min/5' run)
B-13	42.2	107.5	108.4	-65.3	-66.2	-65.8	0.90	Infill	50%	16%		Poorly Graded Sand (SP)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 5 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-04A	42.0	107.5	110.0	-65.5	-68.0	-66.8	2.50	Infill	50%	0%		Fast drilling (3 min/5' run)
GSC-03	40.5	106.5	110.0	-66.0	-69.5	-67.8	3.50	Infill	30%	0%	100%	Fast drilling (2 min/5' run)
E-02	39.8	106.0	106.0	-66.2	-66.2	-66.2	0.05	Infill	99%	16%		Fast drilling (4 min/5' run)
B-01	40.8	107.6	110.0	-66.8	-69.2	-68.0	2.45	Infill	51%	20%		106-107.5 soft, probably sand, 4 min/5' run
A-12	42.1	110.0	112.8	-67.9	-70.7	-69.3	2.75	Infill	45%	45%		Upper 2.75' was soft and no recovery
E-03	42.0	109.9	111.0	-67.9	-69.0	-68.5	1.10	Infill	78%	43%		Fast drilling (3 min/5' run)
B-04A	42.0	110.0	111.4	-68.0	-69.4	-68.7	1.40	Infill	74%	12%		Carbonate Silt and Sand (SP-SM)
GSC-04	40.0	108.7	110.0	-68.7	-70.0	-69.4	1.30	Infill	74%	33%	100%	No Recovery and Loss of Circulation
GSC-05	41.3	110.95	111.0	-69.7	-69.7	-69.7	0.05	Infill	99%	95%		Fast drilling (3 min/5' run)
E-01	40.9	110.6	111.0	-69.7	-70.1	-69.9	0.40	Infill	92%	53%		Fast drilling (4 min/5' run)
B-13	42.2	113.7	115.0	-71.5	-72.8	-72.2	1.30	Infill	74%	0%		Fast drilling (4 min/5' run)
B-04A	42.0	113.7	115.0	-71.7	-73.0	-72.4	1.30	Infill	74%	12%		Fast drilling (3 min/5' run), no recovery
GSC-03	40.5	112.3	120.0	-71.8	-79.5	-75.6	7.75	Infill	45% / 0%	23% / 0%		Fast drilling (3 min/5' run), two core runs
B-05	42.9	115.4	116.0	-72.5	-73.1	-72.8	0.60	Infill	88%	79%		Fast drilling (4 min/5' run)
B-01	40.8	114.2	115.0	-73.4	-74.2	-73.8	0.80	Infill	84%	20%		Fast drilling (4 min/5' run)
B-14	41.7	115.7	116.0	-74.0	-74.3	-74.2	0.30	Infill	94%	85%		Fast drilling (2 min/5' run)
B-13	42.2	116.5	118.5	-74.3	-76.3	-75.3	2.00	Infill	57%	10%	100%	Suspect sand bed
A-09	41.9	116.4	116.5	-74.5	-74.6	-74.6	0.10	Infill	98%	97%	100%	No Recovery and Soft Drilling
B-04A	42.0	116.5	120.0	-74.5	-78.0	-76.3	3.50	Infill	36%	11%		116.5-120' very soft
A-10	42.2	117.0	117.3	-74.8	-75.1	-75.0	0.30	Infill	100%	70%		Sand (SP)
GSC-04	40.0	114.9	115.0	-74.9	-75.0	-75.0	0.10	Infill	98%	78%	100%	No Recovery and Loss of Circulation
E-02	39.8	115.0	116.0	-75.2	-76.2	-75.7	1.05	Infill	79%	22%		Fast drilling (4 min/5' run)
B-09	42.9	119.5	121.0	-76.6	-78.1	-77.4	1.50	Infill	70%	68%		Fast drilling (2 min/5' run)
A-05	42.0	118.7	121.5	-76.7	-79.5	-78.1	2.80	Infill	44%	12%		Fast drilling (4 min/5' run)
B-12	43.3	120.0	130.0	-76.7	-86.7	-81.7	10.00	Infill	0%	0%		May be sand, not rock, fast drilling
B-01	40.8	117.8	120.0	-77.0	-79.2	-78.1	2.20	Infill	56%	13%		Fast drilling (3 min/5' run)
B-11	42.7	120.1	121.5	-77.4	-78.8	-78.1	1.40	Infill	72%	13%		Fast drilling (3 min/5' run)
E-04	43.1	121.0	121.0	-77.9	-77.9	-77.9	0.05	Infill	99%	80%		Fast drilling (3 min/5' run)
A-11	42.5	120.5	121.2	-78.0	-78.7	-78.4	0.70	Infill	90%	70%		Carbonate Silt with Silica Sand (ML)
B-04A	42.0	120.0	125.0	-78.0	-83.0	-80.5	5.00	Infill	0%	0%		Fast drilling (2 min/5' run)
B-14	41.7	120.7	121.0	-79.0	-79.3	-79.2	0.30	void	94%	67%		Cavities
GSC-04	40.0	119.3	120.0	-79.3	-80.0	-79.7	0.70	Infill	86%	74%	100%	No Recovery and Loss of Circulation
E-01	40.9	120.8	121.0	-79.9	-80.1	-80.0	0.20	Infill	96%	55%		Fast drilling (4 min/5' run)
GSC-02	40.4	120.95	121.0	-80.6	-80.6	-80.6	0.05	Infill	99%	70%		Fast drilling (3 min/5' run)
B-03	43.9	124.5	125.0	-80.6	-81.1	-80.9	0.50	Infill	80%	75%		Soft drilling 124.5-125
B-11	42.7	123.9	126.5	-81.2	-83.8	-82.5	2.65	Infill	47%	12%		Fast drilling (4 min/5' run), feel like gravel
E-01	40.9	123.0	126.0	-82.1	-85.1	-83.6	3.00	Infill	40%	0%		Fast drilling (4 min/5' run)
GSC-03	40.5	123.0	125.0	-82.5	-84.5	-83.5	2.00	Infill	60%	0%		Fast drilling (4 min/5' run)
E-04	43.1	125.7	126.0	-82.6	-82.9	-82.8	0.30	Infill	94%	65%		Fast drilling (2 min/5' run)
B-04A	42.0	125.0	128.5	-83.0	-86.5	-84.8	3.50	Infill	18%	0%		Very soft to 128.5'

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 6 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
B-09	42.9	125.9	126.0	-83.0	-83.1	-83.1	0.10	Infill	98%	98%		Fast drilling (2 min/5' run)
B-03	43.9	127.0	128.0	-83.1	-84.1	-83.6	1.00	Infill	72%	52%		Soft lenses 127-128'
E-03	42.0	126.0	126.0	-84.0	-84.0	-84.0	0.05	Infill	99%	40%		Fast drilling (2 min/5' run)
B-01	40.8	124.9	125.0	-84.1	-84.2	-84.1	0.15	Infill	97%	16%		Fast drilling (3 min/5' run)
A-09	41.9	126.3	126.5	-84.4	-84.6	-84.5	0.20	Infill	96%	43%	100%	No Recovery and Loss of Circulation
GSC-04	40.0	124.4	125.0	-84.4	-85.0	-84.7	0.60	Infill	88%	24%	100%	No Recovery and Loss of Circulation
A-02	41.6	126.3	126.5	-84.7	-84.9	-84.8	0.20	Infill	96%	82%	100%	No Recovery and Loss of Circulation
GSC-03	40.5	125.5	126.0	-85.0	-85.5	-85.3	0.50	void	66%	16%	100%	125.5-126.0' void
E-02	39.8	125.1	126.0	-85.3	-86.2	-85.7	0.95	Infill	81%	70%		Fast drilling (4 min/5' run)
E-01	40.9	126.2	131.0	-85.3	-90.1	-87.7	4.80	Infill	4%	0%		Fast drilling (3 min/5' run)
GSC-02	40.4	125.9	126.0	-85.5	-85.6	-85.5	0.15	Infill	97%	65%		Fast drilling (3 min/5' run)
B-09	42.9	128.6	131.0	-85.7	-88.1	-86.9	2.40	Infill	52%	22%		Fast drilling (3 min/5' run)
B-07A	43.2	129.4	130.0	-86.2	-86.8	-86.5	0.60	Infill	88%	13%		Fast drilling (3 min/5' run)
E-04	43.1	129.9	131.0	-86.8	-87.9	-87.4	1.10	Infill	98%	70%		Fast drilling (4 min/5' run)
GSC-03	40.5	127.5	128.7	-87.0	-88.2	-87.6	1.20	Infill	66%	16%	100%	127.5-128.0' soft
B-11	42.7	130.0	131.5	-87.3	-88.8	-88.1	1.50	Infill	70%	8%		Fast drilling (3 min/5' run)
A-10	42.2	130.2	132.0	-88.0	-89.8	-88.9	1.80	Infill	64%	0%	50%	No Recovery and Loss of Circulation
B-04A	42.0	130.0	131.6	-88.0	-89.6	-88.8	1.60	Infill	56%	0%		Carbonate Silts and Sands (SM)
GSC-06	42.5	131.0	131.5	-88.5	-89.0	-88.8	0.50	Infill	90%	26%		Fast drilling (4 min/5' run)
E-03	42.0	130.9	131.0	-88.9	-89.0	-88.9	0.15	Infill	97%	63%		Fast drilling (3 min/5' run)
B-12	43.3	132.3	135.0	-89.0	-91.7	-90.4	2.70	Infill	46%	0%		Fast drilling (4 min/5' run)
A-05	42.0	131.4	131.6	-89.4	-89.6	-89.5	0.15	Infill	98%	82%		Fat Calcareous Clay (CH)
A-09	41.9	131.3	131.5	-89.4	-89.6	-89.5	0.20	Infill	96%	53%	100%	No Recovery and Loss of Circulation
GSC-04	40.0	129.5	130.0	-89.5	-90.0	-89.8	0.50	Infill	90%	25%	100%	No Recovery and Loss of Circulation
E-02	39.8	130.9	131.0	-91.1	-91.2	-91.1	0.15	Infill	97%	93%		Fast drilling (3 min/5' run)
A-08	42.1	133.8	136.0	-91.7	-93.9	-92.8	2.20	Infill	56%	15%		Fast drilling (4 min/5' run)
B-13	42.2	134.2	135.0	-92.0	-92.8	-92.4	0.80	Infill	84%	24%		Fast drilling (4 min/5' run)
B-09	42.9	135.5	136.0	-92.6	-93.1	-92.9	0.50	Infill	90%	80%		Fast drilling (3 min/5' run)
E-04	43.1	135.8	136.0	-92.7	-92.9	-92.8	0.25	Infill	95%	13%		Fast drilling (3 min/5' run)
E-03	42.0	134.9	136.0	-92.9	-94.0	-93.5	1.10	Infill	78%	0%		Fast drilling (2 min/5' run)
B-04A	42.0	135.0	136.4	-93.0	-94.4	-93.7	1.35	Infill	52%	10%		Carbonate Silts and Sands (SM)
GSC-04	40.0	133.5	135.0	-93.5	-95.0	-94.3	1.50	Infill	70%	8%	100%	No Recovery and Loss of Circulation
B-14	41.7	135.7	136.0	-94.0	-94.3	-94.2	0.30	Infill	94%	47%		Fast drilling (3 min/5' run)
B-13	42.2	137.2	140.0	-95.0	-97.8	-96.4	2.80	Infill	44%	7%		Fast drilling (4 min/5' run)
A-07	42.3	137.5	137.8	-95.2	-95.5	-95.4	0.30	Infill	82%	62%		Silt (ML)
E-02	39.8	135.6	136.0	-95.8	-96.2	-96.0	0.40	Infill	92%	22%		Fast drilling (4 min/5' run)
B-07A	43.2	139.4	140.0	-96.2	-96.8	-96.5	0.60	Infill	88%	18%		Fast drilling (4 min/5' run)
GSC-03	40.5	137.7	140.0	-97.2	-99.5	-98.4	2.30	Infill	54%	0%		Fast drilling (4 min/5' run)
E-04	43.1	140.7	141.0	-97.6	-97.9	-97.8	0.30	Infill	94%	76%		Fast drilling (3 min/5' run)
A-07	42.3	140.1	141.0	-97.8	-98.7	-98.3	0.90	Infill	82%	62%		Loss of circulation

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 7 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
A-12	42.1	139.9	140.0	-97.8	-97.9	-97.9	0.10	Infill	98%	86%	80%	No Recovery and Loss of Circulation
B-04A	42.0	140.0	141.5	-98.0	-99.5	-98.8	1.50	Infill	58%	0%		Carbonate Silts and Sands (SM)
E-01	40.9	139.3	141.0	-98.4	-100.1	-99.3	1.70	Infill	66%	0%		Fast drilling (4 min/5' run)
B-14	41.7	140.6	141.0	-98.9	-99.3	-99.1	0.40	void	92%	47%		Cavities
A-09	41.9	141.0	141.5	-99.1	-99.6	-99.4	0.50	Infill	90%	57%	100%	No Recovery and Loss of Circulation
GSC-04	40.0	139.5	140.0	-99.5	-100.0	-99.8	0.50	Infill	90%	0%	100%	No Recovery and Loss of Circulation
A-04	41.3	141.4	141.5	-100.1	-100.2	-100.2	0.10	Infill	98%	86%	100%	No Recovery and Loss of Circulation
GSC-02	40.4	140.7	141.0	-100.3	-100.6	-100.4	0.35	Infill	93%	44%		Fast drilling (4 min/5' run)
B-04A	42.0	143.0	145.0	-101.0	-103.0	-102.0	2.00	Infill	58%	0%		Fast drilling (4 min/5' run), no recovery
B-01	40.8	142.0	143.0	-101.2	-102.2	-101.7	1.00	void	80%	12%		142-143' void
B-02	41.8	144.0	144.1	-102.2	-102.3	-102.3	0.10	void	98%	82%	50-75%	144-144.5' loss of circulation in a void
A-12	42.1	144.4	145.0	-102.3	-102.9	-102.6	0.65	Infill	87%	76%	80%	No Recovery and Loss of Circulation
E-03	42.0	144.7	146.0	-102.7	-104.0	-103.4	1.30	Infill	74%	15%		Fast drilling (3 min/5' run)
B-01	40.8	143.5	144.0	-102.7	-103.2	-103.0	0.50	Infill	80%	12%		143.5-144 soft
B-14	41.7	144.9	146.0	-103.2	-104.3	-103.8	1.10	void	78%	48%		Cavities
A-09	41.9	146.1	146.5	-104.2	-104.6	-104.4	0.40	Infill	92%	82%	100%	No Recovery and Loss of Circulation
GSC-04	40.0	144.2	145.0	-104.2	-105.0	-104.6	0.80	Infill	84%	10%	100%	No Recovery and Loss of Circulation
B-04	42.8	149.6	151.0	-106.8	-108.2	-107.5	1.40	Infill	72%	19%	100%	No Recovery and Loss of Circulation
E-04	43.1	150.0	151.0	-106.9	-107.9	-107.4	1.00	Infill	80%	53%		Fast drilling (3 min/5' run)
A-10	42.2	150.7	152.0	-108.5	-109.8	-109.2	1.26	Infill	75%	58%	100%	Continued Loss of Circulation
E-03	42.0	150.9	151.0	-108.9	-109.0	-109.0	0.10	Infill	98%	58%		Fast drilling (4 min/5' run)
A-09	41.9	150.9	151.5	-109.0	-109.6	-109.3	0.60	Infill	88%	68%	75-100%	No Recovery and Loss of Circulation
GSC-04	40.0	149.2	150.0	-109.2	-110.0	-109.6	0.80	Infill	84%	10%	100%	No Recovery and Loss of Circulation
B-14	41.7	150.9	151.0	-109.2	-109.3	-109.3	0.10	Infill	98%	75%		Fast drilling (4 min/5' run)
A-01	41.6	153.5	153.6	-111.9	-112.0	-111.9	0.10	Infill	100%	92%		Silty Sand (SM)
E-04	43.1	155.0	156.0	-111.9	-112.9	-112.4	1.00	Infill	80%	23%		Fast drilling (4 min/5' run)
GSC-03	40.5	153.0	153.5	-112.5	-113.0	-112.8	0.50	void	96%	60%		153.0-153.5' void
A-12	42.1	154.8	155.0	-112.7	-112.9	-112.8	0.25	Infill	95%	70%	80%	No Recovery and Loss of Circulation
A-11	42.5	155.5	159.9	-113.0	-117.4	-115.2	4.40	Infill	64%	0%	100%	Poorly Graded Silica Sand (SP)
A-09	41.9	155.5	156.5	-113.6	-114.6	-114.1	1.00	Infill	80%	73%	100%	No Recovery and Loss of Circulation
E-03	42.0	155.9	156.0	-113.9	-114.0	-113.9	0.15	Infill	97%	37%		Fast drilling (2 min/5' run)
A-10	42.2	156.3	157.0	-114.1	-114.8	-114.5	0.70	Infill	86%	39%	100%	No Recovery and Loss of Circulation
E-01	40.9	155.8	156.0	-114.9	-115.1	-115.0	0.20	void	96%	80%		Cavities at 154.85', at 155.2', and at 155.8'
A-09	41.9	157.9	162.7	-116.0	-120.8	-118.4	4.80	void	28%	10%	100%	Expects to be in void space from ~158'
A-11	42.5	160.1	160.5	-117.6	-118.0	-117.8	0.40	Infill	64%	0%	100%	Limestone and Carbonate Silt (ML)
E-03	42.0	160.4	161.0	-118.4	-119.0	-118.7	0.60	Infill	88%	23%	100%	Fast drilling (3 min/5' run)
E-01	40.9	160.0	161.0	-119.1	-120.1	-119.6	1.00	Infill	80%	55%		Fast drilling (4 min/5' run)
A-10	42.2	161.6	162.0	-119.4	-119.8	-119.6	0.40	Infill	92%	42%	100%	No Recovery and Loss of Circulation
GSC-04	40.0	159.9	160.0	-119.9	-120.0	-120.0	0.10	Infill	98%	76%	100%	No Recovery and Loss of Circulation
A-09	41.9	162.7	163.4	-120.8	-121.5	-121.2	0.70	Infill	73%	7%	100%	Carbonate Silty Sand with Gravel (SM)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 8 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
E-03	42.0	165.3	166.0	-123.3	-124.0	-123.6	0.75	Infill	85%	28%		Fast drilling (3 min/5' run)
A-10	42.2	166.4	167.0	-124.2	-124.8	-124.5	0.60	Infill	88%	18%	100%	No Recovery and Loss of Circulation
E-01	40.9	165.8	166.0	-124.9	-125.1	-125.0	0.20	Infill	96%	75%		Fast drilling (4 min/5' run)
E-01	40.9	169.4	171.0	-128.5	-130.1	-129.3	1.60	void	68%	18%		Cavities (>3/4") at 168.0', and 169.2'
E-03	42.0	170.6	171.0	-128.6	-129.0	-128.8	0.40	Infill	92%	29%		Fast drilling (3 min/5' run)
A-10	42.2	171.2	172.0	-129.0	-129.8	-129.4	0.80	Infill	84%	31%	100%	No Recovery and Loss of Circulation
E-03	42.0	174.3	176.0	-132.3	-134.0	-133.2	1.70	Infill	66%	33%		Fast drilling (4 min/5' run)
A-09	41.9	175.9	176.0	-134.0	-134.1	-134.1	0.10	Infill	98%	42%	100%	No Recovery and Loss of Circulation
A-10	42.2	176.2	177.0	-134.0	-134.8	-134.4	0.80	Infill	84%	35%	100%	No Recovery and Loss of Circulation
E-01	40.9	175.8	176.0	-134.9	-135.1	-135.0	0.20	Infill	96%	57%		Fast drilling (4 min/5' run)
E-01	40.9	176.8	181.0	-135.9	-140.1	-138.0	4.20	Infill	16%	0%		Fast drilling (4 min/5' run)
A-09	41.9	180.2	181.0	-138.3	-139.1	-138.7	0.80	Infill	84%	40%	100%	No Recovery and Loss of Circulation
A-10	42.2	180.9	182.0	-138.7	-139.8	-139.3	1.10	Infill	78%	40%	100%	No Recovery and Loss of Circulation
E-03	42.0	180.8	181.0	-138.8	-139.0	-138.9	0.25	Infill	95%	18%		Fast drilling (4 min/5' run)
E-03	42.0	185.3	186.0	-143.3	-144.0	-143.6	0.75	Infill	85%	20%		Fast drilling (3 min/5' run)
A-10	42.2	186.6	187.0	-144.4	-144.8	-144.6	0.40	Infill	92%	40%	100%	Circulation regained below 187 ft
A-09	41.9	192.8	196.0	-150.9	-154.1	-152.5	3.20	Infill	36%	0%		Fast drilling (4 min/5' run)
A-03	42.1	196.0	196.5	-153.9	-154.4	-154.2	0.50	Infill	84%	40%		Elastic Silt (MH) from 196.0' to 196.5'
A-02	41.6	196.3	196.5	-154.7	-154.9	-154.8	0.20	Infill	96%	56%		Core barrel sand-locked at 196.5'
A-11	42.5	203.5	205.5	-161.0	-163.0	-162.0	2.00	Infill	60%	0%	50%	Fast drilling (4 min/5' run)
AD-02	42.3	205.6	206.4	-163.3	-164.1	-163.7	0.80	Infill	80%	15%		Silty Sand (SM)
A-11	42.5	207.0	210.5	-164.5	-168.0	-166.3	3.50	Infill	30%	0%	50%	Fast drilling (4 min/5' run)
AD-01	42.0	209.4	214.0	-167.4	-172.0	-169.7	4.60	Infill	32%	0%		Sandy Silt (ML)
A-11	42.5	213.7	215.5	-171.2	-173.0	-172.1	1.80	Infill	64%	9%	50%	Fast drilling (4 min/5' run)
A-07	42.3	216.4	219.7	-174.1	-177.4	-175.8	3.30	Infill	34%	0%		Soft and rapid drilling at 216.5-220.0'
A-11	42.5	218.1	220.5	-175.6	-178.0	-176.8	2.40	Infill	52%	8%	100%	Fast drilling (4 min/5' run)
AD-02	42.3	219.0	219.3	-176.7	-177.0	-176.9	0.30	Infill	82%	22%		Silty Limestone Fragments
A-11	42.5	221.3	225.5	-178.8	-183.0	-180.9	4.25	Infill	15%	0%	100%	Fast drilling (4 min/5' run)
A-07	42.3	222.5	226.0	-180.2	-183.7	-182.0	3.50	Infill	30%	0%		Fast drilling (4 min/5' run)
A-08	42.1	223.5	226.0	-181.4	-183.9	-182.7	2.50	Infill	50%	8%		Fast drilling (4 min/5' run)
A-11	42.5	228.0	230.5	-185.5	-188.0	-186.8	2.50	Infill	50%	0%	100%	Fast drilling (3 min/5' run)
AD-01	42.0	229.9	234.0	-187.9	-192.0	-190.0	4.10	Infill	48%	0%		Silty Sand Sized Material (SM)
A-11	42.5	231.0	250.5	-188.5	-208.0	-198.3	19.50	Infill	<10%	0%	80 to 100%	Fast drilling (2 min/5' run)
AD-01	42.0	234.7	239.0	-192.7	-197.0	-194.9	4.30	Infill	32%	0%		Silt (ML)
AD-02	42.3	236.7	237.1	-194.4	-194.8	-194.6	0.40	Infill	96%	0%		Silt (ML)
A-07	42.3	238.0	241.0	-195.7	-198.7	-197.2	3.00	Infill	40%	22%		Fast drilling (4 min/5' run)
A-02	41.6	242.6	246.5	-201.0	-204.9	-202.9	3.95	Infill	36%	0%		Sandy Silt (ML)
AD-01	42.0	251.9	254.0	-209.9	-212.0	-211.0	2.10	Infill	58%	24%	100%	No Recovery and Nor re-circulating mud
A-07	42.3	260.5	261.0	-218.2	-218.7	-218.5	0.50	Infill	90%	25%		Fast drilling (4 min/5' run)
A-11	42.5	265.9	270.5	-223.4	-228.0	-225.7	4.60	Infill	8%	0%	100%	Fast drilling (3 min/5' run)

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.2-205B (Sheet 9 of 9)
Summary of Karst Features Encountered in Boreholes at North Reactor Site

Borehole Number	Top of Boring Elevation (ft. NAVD88)	Top Depth of Feature (ft.)	Bottom Depth of Feature (ft.)	Elevation of Feature Top (ft. NAVD88)	Elevation of Feature Bottom (ft. NAVD88)	Elevation of Feature Midpoint (ft. NAVD88)	Feature Thickness (ft.)	Feature Type	REC	RQD	Percentage of Circulation Loss	Driller's Comments/ Geologist's Notes/Comments
AD-01	42.0	269.8	270.0	-227.8	-228.0	-227.9	0.20	Infill	32%	0%		Clayey Silt (ML)
A-11	42.5	276.8	280.5	-234.3	-238.0	-236.2	3.70	Infill	26%	0%	100%	Fast drilling (4 min/5' run)
AD-02	42.3	279.0	280.0	-236.7	-237.7	-237.2	1.00	Infill	82%	0%		Sandy Silt (ML)
A-11	42.5	283.7	285.5	-241.2	-243.0	-242.1	1.80	Infill	64%	0%	100%	Fast drilling (3 min/5' run)
AD-01	42.0	300.3	300.5	-258.3	-258.5	-258.4	0.20	Infill	78%	29%		Clay (CL)
AD-02	42.3	300.7	304.6	-258.4	-262.3	-260.4	3.90	Infill	74%	11%		Sandy Silt (ML), Silt (ML)
AD-02	42.3	305.1	305.4	-262.8	-263.1	-263.0	0.30	Infill	74%	11%		Silt (ML)
AD-01	42.0	324.7	329.0	-282.7	-287.0	-284.9	4.30	Infill	14%	0%		Wash out fine soft material
AD-02	42.3	326.7	329.0	-284.4	-286.7	-285.6	2.30	Infill	100%	42%		Sandy Silt with Limestone (ML)
AD-01	42.0	329.0	329.4	-287.0	-287.4	-287.2	0.40	Infill	70%	19%		Sandy Silt to Gravelly Silt (ML)
AD-01	42.0	330.4	331.2	-288.4	-289.2	-288.8	0.75	Infill	70%	19%		Sandy Silt to Gravelly Silt (ML)
AD-01	42.0	331.7	332.0	-289.7	-290.0	-289.8	0.25	Infill	70%	19%		Sandy Silt to Gravelly Silt (ML)
AD-02	42.3	339.9	340.0	-297.6	-297.7	-297.6	0.15	Infill	94%	62%		Organic Material (OH)
AD-01	42.0	368.9	369.0	-326.9	-327.0	-327.0	0.10	Infill	98%	35%	100%	No Recovery and Loss of Circulation
AD-01	42.0	415.3	415.9	-373.3	-373.9	-373.6	0.60	Infill	88%	18%		Silty Clay (CL-ML)
AD-02	42.3	416.6	417.4	-374.3	-375.1	-374.7	0.80	Infill	94%	26%		Organic Elastic Silt to Fat Clay (MH-CH)
AD-01	42.0	424.7	425.6	-382.7	-383.6	-383.1	0.85	Infill	88%	30%		Clayey Limestone Fragments (GC)
AD-01	42.0	426.2	426.4	-384.2	-384.4	-384.3	0.20	Infill	88%	30%		Clayey Limestone Fragments (GC)
AD-01	42.0	430.7	431.0	-388.7	-389.0	-388.9	0.30	Infill	100%	46%		Clay (CL)
AD-01	42.0	431.9	433.0	-389.9	-391.0	-390.5	1.10	Infill	100%	46%		Clayey Limestone Fragments (GC)
AD-02	42.3	439.7	440.2	-397.4	-397.9	-397.7	0.50	Infill	64%	11%		Silt (ML)
AD-02	42.3	464.9	465.9	-422.6	-423.6	-423.1	1.00	Infill	78%	0%		Silt with Limestone Fragments (ML)
AD-01	42.0	465.7	466.8	-423.7	-424.8	-424.3	1.10	Infill	82%	17%		Silty Limestone Fragments (GM)
AD-02	42.3	466.5	469.0	-424.2	-426.7	-425.5	2.50	Infill	78%	0%		Silt with Limestone Fragments (ML)

Note:

The karst feature intervals estimated by driller during drilling may be adjusted based on the no recovery intervals in the rock core logs.

LNP COL 2.5-6

Table 2.5.4.2-206
Rock Young's Modulus Values Estimated from PMT Results

Boring ID	Depth of Test (ft.)	Young Modulus (E_{pmt}), ksi	# of Test Pressures Used
B-11	63.0	116	7
B-11	74.1	4 [(a)]	3
B-11	87.6	245	10
B-11	95.1	4	3
B-11	107.9	2	3
B-11	118.1	2	3
B-11	127.9	1	3
B-19	67.9	173	8
B-19	78.1	31	6
B-19	87.9	315	8
B-19	107.9	58	6
B-19	118.1	78	8
B-19	128.9	73	7

Notes:

a) This modulus was not used for establishing rock properties, due to void encountered at this depth in Borehole B-11.

ft. = foot
E_{pmt} = rock pressuremeter test modulus
ksi = kips per square inch

LNP COL 2.5-6

Table 2.5.4.2-207 (Sheet 1 of 4)
Estimated "Top of Rock"

Borehole Name	Northing	Easting	Top of Borehole Elevation (ft. NAVD88)	Depth to Top of Rock (ft.)	Elevation at Top of Rock (ft. NAVD88)
A-01	1723879.2	457603.8	41.6	49	-7.4
A-02	1723946.2	457608	41.6	56.5	-14.9
A-03	1723884.4	457671.8	42.1	43.5	-1.4
A-04	1724023.6	457634.2	41.3	55	-13.7
A-05	1723975.3	457680.2	42.0	58	-16.0
A-06	1723934.5	457719.1	42.5	58.5	-16.0
A-07	1724100.8	457649.4	42.3	76	-33.7
A-08	1724017.2	457734.1	42.1	66	-23.9
A-09	1724113.8	457731.4	41.9	38.5	3.4
A-10	1724149.3	457766.2	42.2	62	-19.8
A-11	1724091.7	457813.3	42.5	35.5	7.0
A-12	1724065.3	457848.9	42.1	45	-2.9
A-13	1722927.1	457933.5	40.6	55	-14.4
A-14*	1722999.7	457929.8	42.4	66.2	-23.8
A-14A	1722992.6	457934.5	42.2	66	-23.8
A-15	1722937.2	457994.3	42.5	40	2.5
A-16	1723075.9	457958.1	42.7	46	-3.3
A-17	1723025.7	458007.3	42.3	51	-8.7
A-18*	1722986.9	458047.9	42.3	35.7	6.6
A-18A	1722992.2	458049.3	42.1	35.5	6.6
A-19	1723149.9	457976.4	43.1	56	-12.9
A-20	1723068.1	458060.9	42.3	55	-12.7
A-21*	1723168.5	458055.6	42.4	49.6	-7.2
A-21A	1723171.1	458054.1	42.8	50	-7.2
A-22*	1723199.8	458088	42.6	45.7	-3.1
A-22A	1723191.2	458083.4	42.9	46	-3.1
A-23	1723141.4	458146.5	40.8	55	-14.2
A-24*	1723114.5	458174.3	40.6	61.8	-21.2
A-24A	1723110	458176.7	40.3	61.5	-21.2
B-01	1723999.6	457491.6	40.8	80	-39.2
B-02	1724128.3	457619.4	41.8	55.5	-13.7
B-03	1724210.1	457702.3	43.9	60	-16.1
B-04	1724317.2	457809.8	42.8	136	-93.2
B-04A	1724269.5	457868.6	42.0	75	-33
B-05	1724427.7	457904.5	42.9	91	-48.1
B-06	1724172.6	457791.6	42.5	66.5	-24.0
B-07	1724369.7	457955.5	43.1	137.5	-94.4
B-07A	1724358.9	457965.5	43.2	125	-81.8
B-08	1724091.9	457874.5	42.4	51	-8.6
B-09	1724303.2	458022.2	42.9	66	-23.1

LNP COL 2.5-6

Table 2.5.4.2-207 (Sheet 2 of 4)
Estimated "Top of Rock"

Borehole Name	Northing	Easting	Top of Borehole Elevation (ft. NAVD88)	Depth to Top of Rock (ft.)	Elevation at Top of Rock (ft. NAVD88)
B-10	1723789.1	457699.9	42.0	61.5	-19.5
B-11	1723966.3	457786.7	42.7	66.5	-23.8
B-12	1723919.8	457828.5	43.3	60	-16.7
B-13	1723995.1	457903.5	42.2	45	-2.8
B-14	1724122.8	458024.1	41.7	26	15.7
B-15	1724222.8	458094.3	42.3	61	-18.7
B-16	1723050.3	457812.4	42.6	56	-13.4
B-17	1723177.4	457948	42.2	51.5	-9.3
B-18	1723259.1	458027.2	42.0	46	-4.0
B-19	1723369.6	458134	41.3	61.5	-20.2
B-20	1723468.6	458221.5	40.4	81.5	-41.1
B-21	1723224.9	458119.4	41.8	82	-40.2
B-22	1723410.3	458287.4	40.5	80	-39.5
B-23	1723150.7	458210.1	40.7	70.5	-29.8
B-23A	1723147.5	458207.7	42.4	NE	NE
B-24	1723356.3	458351.5	40.9	55	-14.1
B-25	1722845.8	458024.3	42.5	65	-22.5
B-25A	1722853	458017.4	42.2	NE	NE
B-26	1723010.2	458111.7	42.4	94.5	-52.1
B-27	1722971.1	458154.7	42.4	51	-8.6
B-28	1723060.1	458242.6	41.5	65	-23.5
B-29	1723157.5	458338.8	41.7	61	-19.3
B-30	1723272.1	458444.8	42.2	70	-27.8
B-30A	1723272.4	458440.3	42.5	34	8.5
B-31	1724391.9	457978.0	43.4	132.4	-89.0
B-33	1724328.8	457955.2	43.0	72.3	-29.3
D-01	1724095.5	457510.2	40.8	76	-35.2
D-02	1724164.5	457585	41.3	35	6.3
D-03	1724234.5	457645.5	42.0	32	10.0
D-04	1723150.5	457831.9	41.9	60	-18.1
D-05	1723221.4	457903.2	41.8	40.7	1.1
D-06	1723292.3	457976.6	41.6	36	5.6
E-01	1723795	457523.7	40.9	41	-0.1
E-02	1724255.5	457486.3	39.8	51	-11.2
E-03	1724208.2	457932.6	42.0	60	-18.0
E-04	1723843.2	457904.3	43.1	61.5	-18.4
E-05	1722853.5	457850.2	42.6	95.5	-52.9
E-06	1723312.3	457814.1	42.8	81	-38.2
E-07	1723243.7	458250.5	41.7	70	-28.3
E-08	1722897.6	458228	42.4	70	-27.6
GSC-01	1724390.3	457810.6	43.1	NE	NE

LNP COL 2.5-6

Table 2.5.4.2-207 (Sheet 3 of 4)
Estimated "Top of Rock"

Borehole Name	Northing	Easting	Top of Borehole Elevation (ft. NAVD88)	Depth to Top of Rock (ft.)	Elevation at Top of Rock (ft. NAVD88)
GSC-01A	1724368.5	457807.6	42.9	NE	NE
GSC-01B	1724347.3	457805.5	42.8	NE	NE
GSC-02	1724300.1	457447.2	40.4	51	-10.6
GSC-03	1724068	457449.3	40.5	90	-49.5
GSC-04	1723824.3	457384	40.0	85	-45.0
GSC-05	1723689.4	457584.8	41.3	41	0.3
GSC-06	1723774.1	457972	42.5	61.5	-19.0
GSC-07	1723499.5	458024.9	42.7	NE	NE
GSC-07A	1723463.8	458028.1	43.1	56	-12.9
GSC-08	1723365.5	457759	43.2	85	-41.8
GSC-08A	1723362.2	457763.1	43.1	77	-33.9
GSC-09	1723154	457653.4	41.3	80	-38.7
GSC-10	1722899.7	457706.1	42.3	66	-23.7
GSC-11	1722723.7	457915.1	42.9	109	-66.1
GSC-12	1722835.8	458289.6	41.0	85	-44.0
I-01	1724110.3	457635.3	42.5	NA	NA
I-02	1724046.5	457700.6	42.3	NA	NA
I-03	1723978.8	457771.7	42.1	NA	NA
I-04	1723902.3	457585.6	41.6	NA	NA
I-05	1724148.8	457804.5	42.2	NA	NA
I-06	1723163	457960.6	42.3	NA	NA
I-07	1723097.8	458026.5	42.4	NA	NA
I-08	1723055	458076.8	42.5	NA	NA
I-09	1722958.6	457888.4	42.4	NA	NA
I-10	1723172.2	458130.7	42.0	NA	NA
AD-01	1724033.5	457716.3	42.0	NA	NA
AD-02	1723984.3	457716.4	42.3	NA	NA
AD-03	1723086.2	458040.8	42.4	NA	NA
AD-04	1723035	458032	42.6	NA	NA
CT-01	1724860.4	455975.6	43.4	86	-42.6
CT-02	1724719.3	456342.2	42.3	31.5	10.8
CT-03	1724626.2	456581.9	40.8	35.5	5.3
CT-04	1724456.6	456923.6	40.8	35	5.8
CT-05	1723052.6	456340.9	41.5	45	-3.5
CT-06	1722977.3	456619.7	41.4	71.5	-30.1
CT-07	1722823.9	456814.3	42.0	45	-3.0

LNP COL 2.5-6

Table 2.5.4.2-207 (Sheet 4 of 4)
Estimated "Top of Rock"

Borehole Name	Northing	Easting	Top of Borehole Elevation (ft. NAVD88)	Depth to Top of Rock (ft.)	Elevation at Top of Rock (ft. NAVD88)
CT-08	1722706.4	457111.8	42.2	41.5	0.7
IT-01	1705495.9	457735.8	20.9	75	-54.1
IT-02	1705642.1	457838.7	29.6	91	-61.4
O-1	1723173.4	458057.4	42.7	20	22.7
O-2	1722994.8	457937.7	42.7	15.5	27.2
O-3	1723189.3	458086.9	42.5	15.0	27.5
O-4	1722990.9	458053.5	42.3	31.5	10.8
O-5	1724150.2	457769.9	42.6	22.0	20.6
O-6	1724065.3	457853.4	42.2	14	28.2

Notes:

*: For Boreholes A-14, A-18, A-21, A-22, and A-24, the top of rock may be higher than where the rock coring started because the rock could have been grinded using mud rotary drilling to a depth below top of rock. Offset boreholes, i.e., Boreholes A-14A, A-18A, A-21A, A-22A, and A-24A were drilled a few feet away from the original Boreholes A-14, A-18, A-21, A-22, and A-24, respectively, to determine top of rock. Therefore, the elevations of the top of the rock of the offset boreholes is used to represent that of the original boreholes and no information from the original boreholes was considered in terms of top of rock.

NA = Top of rock cannot be determined because the borehole or the top portion of the borehole was drilled using sonic drilling, which pulverizes soil and rock during drilling and did not allow the determination of RQD value.

NE = Rock was not encountered during drilling. Only soil boring was performed.
ft. = foot

LNP COL 2.5-6

Table 2.5.4.2-208 (Sheet 1 of 2)
Summary of Laboratory Soil Index Test Results

Layer/Statistics		Liquid Limit [a]	Plasticity Index [a]	Percent Passing #200 Sieve	Specific Gravity	In Situ Moisture (percent)	pH	Resistivity Ohm-cm	Organic Content [b] (percent)
North Reactor (LNP 2)									
LAYER S-1	Minimum	18	4	2	2.65	11.8	7.0	4499	0.3
	Maximum	88	65	98	2.84	70.9	7.0	4499	2.3
	Average	35	21	22	2.72	27.0	7.0	4499	1.6
	Standard Deviation	17	15	25	0.08	13.0	-	-	0.9
	Count [c]	18	18	68	7	43	1	1	4
LAYER S-2	Minimum	22	4	24	2.70	18.9	7.3	-	-
	Maximum	40	29	94	2.83	36.0	7.3	-	-
	Average	32	18	53	2.77	27.1	7.3	-	-
	Standard Deviation	9	13	23	-	5.4	-	-	-
	Count [c]	3	3	20	2	13	1	0	0
LAYER S-3	Minimum	18	3	6	2.71	8.7	6.3	1361	5.1
	Maximum	80	41	93	2.85	46.7	8.4	5751	5.1
	Average	38	16	48	2.80	24.4	7.6	3514	5.1
	Standard Deviation	22	15	19	0.07	7.5	0.7	1961	-
	Count [c]	6	6	65	3	42	6	5	1
South Reactor (LNP 1)									
LAYER S-1	Minimum	19	2	3	2.67	11.2	5.3	3542	2.6
	Maximum	60	44	98	2.68	64.4	9.2	19790	31.2
	Average	32	20	27	2.68	25.9	7.5	11666	13.8
	Standard Deviation	13	13	27	0.01	12.4	1.6	11489	12.3
	Count [c]	11	11	54	2	35	4	2	4

LNP COL 2.5-6

Table 2.5.4.2-208 (Sheet 2 of 2) Summary of Laboratory Soil Index Test Results

Layer/Statistics		Liquid Limit [a]	Plasticity Index [a]	Percent Passing #200 Sieve	Specific Gravity	In Situ Moisture (percent)	pH	Resistivity Ohm-cm	Organic Content [b] (percent)
LAYER S-2	Minimum	-	-	17	2.80	15.5	7.9	4027	-
	Maximum	-	-	97	2.88	28.1	8.1	5609	-
	Average	-	-	64	2.84	22.0	8.0	4818	-
	Standard Deviation	-	-	21	0.04	3.5	0.1	1119	-
	Count [c]	0	0	28	3	15	2	2	0
LAYER S-3	Minimum	31	5	19	2.80	18.5	7.3	3267	-
	Maximum	31	5	99	2.84	27.7	8.7	3267	-
	Average	31	5	55	2.82	23.0	8.0	3267	-
	Standard Deviation	-	-	23	0.03	2.7	1.0	-	-
	Count [c]	1	1	55	2	29	2	1	0

Notes:

a) The liquid limit and plasticity index were performed on limited quantity of samples, which were considered potentially plastic. However, most of the soil samples in the project site are non-plastic materials based on field description. Therefore, the statistics should only be considered valid for the tested samples and are not representative of the soil layers.

b) Organic content was only performed where it was suspicious. Most of the samples at the project site are not organic type of soils. Therefore, the statistics are not representative for the soil layers.

c) Count of the liquid limit and plasticity index columns refers to the number of tests performed on samples that were not determined to be nonplastic. For all other columns, count refers to all tests.

Ohm-cm = Ohm-centimeter

LNP COL 2.5-6

Table 2.5.4.2-209
Summary of Index and Triaxial Strength Test Results for the Soil-Like Materials within Rock Mass

Borehole ID	Sample ID	Depth (ft.) Top of Sample	Depth (ft.) Bottom of Sample	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	% Passing #200 sieve (%)	Dry Unit Weight (pcf) (triaxial sample)	Specific Gravity	Moisture Content (%) (triaxial sample)	Compressive Strength (ksf)	Confining Pressure (ksf)	Strain (%)
AD-3	SC-7S	434	434.25	Nonplastic		40	-	1.77	-	-		-
AD-4	SC-8S	434	435.4	125	89	90	51.2	1.72	114	9.93	1.07	1.00

Notes:

ft. = foot
ksf = kips per square foot
pcf = pound per cubic foot

LNP COL 2.5-6

Table 2.5.4.2-210
Summary of Index and Consolidation Test Results for the Soil-Like Materials within Rock Mass

		Depth (ft.)			Moisture Content (%) (Consolidation Sample)						
Borehole ID	Sample ID	Top of Sample	Bottom of Sample	Dry Unit Weight (pcf) (consolidation sample)	Saturated	Natural	Organic Content (%)	Preconsolidation Pressure, P_c (ksf)	Virgin Compression Index, C_c	Recompression Index, C_r	Initial Void Ratio, e_0
AD-3	SC-7S	434	434.25	41.8	61	57	42.6	4.00	0.23	0.05	1.64
AD-4	SC-8S	434	435.4	57.4	114	58	25.9	13.96	0.22	0.05	0.88

Notes:

Although the above samples were obtained from similar depths, there is a significant difference in the preconsolidation pressure, Pc, obtained from the performed consolidation tests. This difference could be due to the disturbance effect on the two samples, since these samples were obtained via a rock coring technique and are therefore disturbed samples of the material.

ft. = foot
ksf = kips per square foot
pcf = pound per cubic foot

LNP COL 2.5-6

Table 2.5.4.2-211 (Sheet 1 of 3)
Summary Statistics of UCS, Elastic Moduli, Poisson’s Ratio, and Index Test Results of Intact Rock Samples

Elevation Range (ft. NAVD88)	Statistics	Unconfined Compressive Strength (psi)	Secant Modulus (at 50% failure stress) (x10^6 psi)	Poisson's Ratio - Secant (at 50% failure stress)	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (axial) (x10^6 psi)	Tangent Modulus (Radial) (x10^6 psi)	Poisson's Ratio - Tangent
North Reactor Site (LNP 2)									
Top of rock to -97 (NAV-1)	Minimum	384	0.67	0.22	108	2	0.60	0.90	0.25
	Maximum	9717	8.29	0.51	165	25	12.94	31.05	0.66
	Average	2414	2.57	0.34	134	14	2.78	7.11	0.44
	Standard Deviation	2634	2.29	0.08	12	6	3.05	7.91	0.10
	No. of Samples	67	18	18	67	66	18	18	18
-97 to -148 (NAV-2)	Minimum	433	0.70	0.18	116	1	1.07	1.67	0.20
	Maximum	8536	7.01	0.46	155	23	6.68	26.30	0.64
	Average	2938	3.74	0.30	136	11	3.75	12.66	0.37
	Standard Deviation	2279	2.17	0.09	10	5	1.99	8.41	0.14
	No. of Samples	27	13	13	27	27	13	13	13
-148 to -303 (NAV-3)	Minimum	227	1.24	0.36	111	12	1.65	3.14	0.53
	Maximum	2455	1.24	0.36	124	32	1.65	3.14	0.53
	Average	711	1.24	0.36	118	23	1.65	3.14	0.53
	Standard Deviation	867	NA	NA	5	6	NA	NA	NA
	No. of Samples	6	1	1	6	6	1	1	1

LNP COL 2.5-6

Table 2.5.4.2-211 (Sheet 2 of 3)
Summary Statistics of UCS, Elastic Moduli, Poisson's Ratio, and Index Test Results of Intact Rock Samples

Elevation Range (ft. NAVD88)	Statistics	Unconfined Compressive Strength (psi)	Secant Modulus (at 50% failure stress) (x10^6 psi)	Poisson's Ratio - Secant (at 50% failure stress)	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (axial) (x10^6 psi)	Tangent Modulus (Radial) (x10^6 psi)	Poisson's Ratio - Tangent
-303 to -458 (NAV-4)	Minimum	465	6.25	0.16	120	9	5.93	37.09	0.16
	Maximum	6938	6.25	0.16	155	32	5.93	37.09	0.16
	Average	2526	6.25	0.16	135	20	5.93	37.09	0.16
	Standard Deviation	2991	NA	NA	16	10	NA	NA	NA
	No. of Samples	4	1	1	4	4	1	1	1
South Reactor Site (LNP 1)									
Top of rock to -180 (SAV-1)	Minimum	131	0.34	0.16	111	1	0.68	1.20	0.13
	Maximum	18458	10.78	0.58	167	34	10.18	75.98	0.71
	Average	3760	4.06	0.29	138	10	4.11	14.62	0.36
	Standard Deviation	3335	2.82	0.10	12	7	2.53	13.79	0.15
	No. of Samples	97	36	36	97	97	36	36	36
-180 to -309 (SAV-2)	Minimum	236	1.00	0.50	116	17	0.79	1.54	0.51
	Maximum	1038	1.00	0.50	132	28	0.79	1.54	0.51
	Average	736	1.00	0.50	125	23	0.79	1.54	0.51
	Standard Deviation	436	NA	NA	8	5	NA	NA	NA
	No. of Samples	3	1	1	3	3	1	1	1

LNP COL 2.5-6

Table 2.5.4.2-211 (Sheet 3 of 3)
Summary Statistics of UCS, Elastic Moduli, Poisson's Ratio, and Index Test Results of Intact Rock Samples

Elevation Range (ft. NAVD88)	Statistics	Unconfined Compressive Strength (psi)	Secant Modulus (at 50% failure stress) (x10^6 psi)	Poisson's Ratio - Secant (at 50% failure stress)	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (axial) (x10^6 psi)	Tangent Modulus (Radial) (x10^6 psi)	Poisson's Ratio - Tangent
-309 to -458 (SAV-3)	Minimum	1925	1.96	0.05	129	8	1.72	7.10	0.09
	Maximum	5143	6.86	0.42	156	22	6.81	33.82	0.71
	Average	3690	4.50	0.22	144	13	4.27	18.04	0.32
	Standard Deviation	1447	2.09	0.15	11	6	2.18	11.56	0.27
	No. of Samples	5	4	4	5	5	4	4	4

Notes:

Secant modulus was defined by the slope of stress versus strain relationship between the zero stress condition and the stress condition at 50% of the failure stress; the tangent modulus was defined by the slope of the stress versus strain relationship at the 50% failure stress state.

NAVD = North American Vertical Datum
ft. = foot
NA = not applicable
pcf = pound per cubic foot
psi = pound per square inch

LNP COL 2.5-6

Table 2.5.4.2-212
Summary Statistics of Tensile Strength Test Results on Intact Rock Samples

Elevation Range (ft. NAVD88)	Statistics	Tensile Strength (psi)	Bulk Density (pcf)	Moisture Content (%)
North Reactor Site (LNP 2)				
Top of rock to -97 (NAV-1)	Minimum	43	114	6
	Maximum	887	148	33
	Average	238	131	17
	Standard Deviation	301	11	7
	No. of Samples	11	11	11
-97 to -148 (NAV-2)	Minimum	78	109	7
	Maximum	1095	150	22
	Average	562	137	12
	Standard Deviation	428	14	5
	No. of Samples	7	7	7
-148 to -303 (NAV-3)	Minimum	23	122	27
	Maximum	23	122	27
	Average	23	122	27
	Standard Deviation	-	-	-
	No. of Samples	1	1	1
-303 to -458 (NAV-3)	Minimum	164	121	21
	Maximum	164	121	21
	Average	164	121	21
	Standard Deviation	-	-	-
	No. of Samples	1	1	1
South Reactor Site (LNP 1)				
Top of rock to -180 (SAV-1)	Minimum	24	119	1
	Maximum	2759	169	22
	Average	702	143	9
	Standard Deviation	648	11	5
	No. of Samples	20	20	20
-180 to -309 (SAV-2)	Minimum	No Tensile Strength Test Data available for this layer.		
	Maximum			
	Average			
	Standard Deviation			
	No. of Samples			
-309 to -458 (SAV-3)	Minimum	584	149	7
	Maximum	732	153	12
	Average	658	151	10
	Standard Deviation	105	3	4
	No. of Samples	2	2	2

Notes:
NAVD = North American Vertical Datum
ft. = foot
pcf = pound per cubic foot
psi = pound per square inch

LNP COL 2.5-6

Table 2.5.4.2-213
Summary of Hoek-Brown Rock Mass Strength Parameters

Layer	Unit Weight (pcf)	Representative Unconfined Compressive Strength of Intact Rock (psi)	Geological Strength Index (GSI)	Hoek-Brown Criterion Parameter, m_i	Disturbance Factor, D	Rock Mass Cohesion, c (psi)	Rock Mass Friction Angle, φ (°)
South Reactor Site (LNP 1)							
SAV-1	138	3700	31	8	0.7	27	24
SAV-2	125	700	21	8	0.2	21	15
SAV-3	144	3600	27	8	0.2	82	22
North Reactor Site (LNP 2)							
NAV-1	134	2400	37	8	0.7	26	24
NAV-2	136	2900	38	8	0.5	53	25
NAV-3	118	700	22	8	0.2	20	16
NAV-4	135	2500	31	8	0.2	72	21

Notes:

pcf = pound per cubic foot
psi = pound per square inch

LNP COL 2.5-6

Table 2.5.4.2-214
Summary of Rock Dynamic Properties from Suspension Logging

Layer	Statistic	V_S (ft/sec)	V_P (ft/sec)	Poisson's Ratio, v	Young's Modulus, E (ksi)
South Reactor Site (LNP 1)					
SAV-1	Minimum	1610	5210	0.15	223
	Maximum	8230	17090	0.48	5028
	Average	3932	9601	0.39	1379
	Standard Deviation	1227	2072	0.06	851
	No. of Values	435	442	434	434
SAV-2	Minimum	1580	5700	0.34	197
	Maximum	4730	11900	0.47	1621
	Average	2932	7763	0.41	676
	Standard Deviation	588	836	0.03	252
	No. of Values	290	292	290	290
SAV-3	Minimum	1980	6600	0.19	354
	Maximum	6060	12120	0.46	2893
	Average	3839	9045	0.38	1304
	Standard Deviation	739	1151	0.04	474
	No. of Values	162	162	162	162
North Reactor Site (LNP 2)					
NAV-1	Minimum	2320	5850	0.18	452
	Maximum	6410	13330	0.45	3208
	Average	3660	8365	0.38	1093
	Standard Deviation	670	1015	0.04	402
	No. of Values	261	261	261	261
NAV-2	Minimum	2570	6870	0.17	550
	Maximum	7250	13610	0.46	3611
	Average	4614	9916	0.35	1733
	Standard Deviation	830	1242	0.05	569
	No. of Values	219	219	219	219
NAV-3	Minimum	1470	5800	0.18	163
	Maximum	5800	10930	0.48	2168
	Average	3097	8008	0.41	708
	Standard Deviation	593	680	0.03	252
	No. of Values	455	455	455	455
NAV-4	Minimum	2590	7170	0.26	557
	Maximum	6170	12120	0.46	2900
	Average	3963	9105	0.38	1295
	Standard Deviation	751	1207	0.04	472
	No. of Values	171	171	171	171

Notes:

ft/sec = foot per second
ksi = kips per square inch

LNP COL 2.5-6

Table 2.5.4.2-215
Summary of Values for E_{rm} (ksi)

Rock Layer	Statistic	Method Used to Obtain E_{rm} (ksi): Shear Wave Velocity[a]	Rock Pressuremeter	UCS Testing[b]	UCS Testing[c]
North Reactor Site (LNP 2)					
NAV-1	Minimum	226	1	33	37
	Maximum	1604	245	411	456
	Average	547	62	127	142
NAV-2	Minimum	275	-	48	40
	Maximum	1806	-	477	404
	Average	867	-	255	216
NAV-3	Minimum	82	-	51	34
	Maximum	1084	-	51	34
	Average	354	-	51	34
NAV-4	Minimum	278	-	416	261
	Maximum	1450	-	416	261
	Average	647	-	416	261
South Reactor Site (LNP 1)					
SAV-1	Minimum	112	31	13	14
	Maximum	2514	315	404	450
	Average	690	121	152	170
SAV-2	Minimum	99	-	39	26
	Maximum	810	-	39	26
	Average	338	-	39	26
SAV-3	Minimum	177	-	104	68
	Maximum	1446	-	363	238
	Average	652	-	238	156

Notes:

a) Taken as 50% of E_o calculated using shear-wave velocity measurements per Mayne et al (Reference 2.5.4.2-231).

b) Hoek and Diederichs (Reference 2.5.4.2-232).

c) Yang (Reference 2.5.4.2-233).

ksi = kips per square inch

LNP COL 2.5-6

Table 2.5.4.2-216
Estimated Properties of Fine-Grained Component of Soils above the Top of Rock Based on Laboratory Index Properties

Layer	Statistics	f'_{cv} (deg.)	OCR	s_u (psf)	C_c	C_r	C_{ea}
North Reactor Site (LNP 2)							
LAYER S-1	Minimum	24	1.0	76	0.09	0.02	0.001
	Maximum	38	4.9	904	0.88	0.18	0.007
	Average	31	1.7	449	0.31	0.06	0.003
	Standard Deviation	4	1.3	258	0.20	0.04	0.002
	No. of Samples	16	14	14	16	14	14
LAYER S-2	Minimum	29	1.0	136	0.30	0.06	0.003
	Maximum	31	1.0	1136	0.39	0.08	0.004
	Average	30	1.0	636	0.34	0.07	0.004
	Standard Deviation	1	0.0	707	0.07	0.01	0.000
	No. of Samples	2	2	2	2	2	2
LAYER S-3	Minimum	27	1.0	986	0.27	0.05	0.004
	Maximum	31	4.0	2426	0.55	0.11	0.005
	Average	29	2.0	1471	0.38	0.08	0.004
	Standard Deviation	2	1.7	828	0.15	0.03	0.001
	No. of Samples	3	3	3	3	3	3
South Reactor Site (LNP 1)							
LAYER S-1	Minimum	26	1.0	76	0.09	0.02	0.001
	Maximum	38	13.0	2175	0.59	0.12	0.005
	Average	31	4.4	769	0.30	0.06	0.002
	Standard Deviation	3	4.5	834	0.16	0.03	0.001
	No. of Samples	10	9	9	10	10	9
LAYER S-3	Minimum	—	—	—	—	—	—
	Maximum	—	—	—	—	—	—
	Average	—	—	—	—	—	—
	Standard Deviation	—	—	—	—	—	—
	No. of Samples	0	0	0	0	0	0

Notes:

No cohesive soil samples collected from South Reactor S-2 Layer;
South Reactor Layer S-3 has no summary statistics as index testing indicated results were outliers.

psf = pound per square foot

LNP COL 2.5-6

Table 2.5.4.2-217
Estimates of Properties of Cohesionless Soil Layers above the Top of Rock Based on SPT N-Values

Soil Property	Symbol	Units	North Reactor S-1	North Reactor S-2	North Reactor S-3	South Reactor S-1	South Reactor S-2	South Reactor S-3
Mean SPT N-Value	N	blows/ft.	10	43	85	9	43	82
Mean SPT N-Value adjust to a 60% drill rod energy ratio	N_{60}	blows/ft.	11	45	86	11	52	86
Moist Unit Weight	γ_m[(a)]	pcf	110	120	130	110	120	130
Relative Density	D_r[(a)]	percent	25	50	90	25	50	90
Effective Friction Angle	ϕ'[(b)]	degree	28	31	36	28	31	36
Effective Cohesion	c'[(c)]	psi	0	0	0	0	0	0
Poisson's Ratio	v[(d)]	—	0.15	0.19	0.27	0.15	0.19	0.27
Elastic Modulus	E[(e)]	psi	808	3307	6319	808	3821	6319
	E[(f)]	psi	1736	4028	6944	1667	4028	6736
	E[(g)]	psi	1020	4317	8232	918	4317	7942
	E[(h)]	psi	683	2148	3940	640	2148	3812
Shear Modulus	G[(i)]	psi	353	1389	2498	353	1605	2498

Notes:

a) These properties were estimated using the guidelines proposed by Teng (Reference 2.5.4.2-234).

b) Effective friction angle (ϕ') was estimated using the guideline of NAVFAC (Reference 2.5.4.2-235).

c) Effective cohesion (c') was set to zero.

d) Poisson's ratio (*v*) was estimated using correlation by Trautmann and Kulhawy, as a function of effective friction angle (Reference 2.5.4.2-236).

e) Young's modulus (E) was estimated using correlation by Kulhawy and Mayne (Reference 2.5.4.2-236), as a function of N_{60}.

f) Young's modulus (E) was estimated using the relationship of Webb (Reference 2.5.4.2-237), as a function of N.

g) Young's modulus (E) was estimated using correlation by Farrent (Reference 2.5.4.2-238), as a function of Poisson's ratio and N.

h) Young's modulus (E) was estimated using correlation by Begemann (Reference 2.5.4.2-239), as a function of N.

i) Shear Modulus (G) was estimated using correlation by Lambe and Whitman (Reference 2.5.4.2-240), based on E from Kulhawy and Mayne (Reference 2.5.4.10-202) correlation.

ft. = foot; pcf = pound per cubic foot; psi = pound per square foot

2.5.4.3 Foundation Interfaces

LNP COL 2.5-5
LNP COL 2.5-6

"Plant North" for LNP 1 and LNP 2 is rotated 45 degrees clockwise from State Plane North (i.e., N45E), as shown on Figure 2.5.4.2-201A (and other figures). Where "Plant" is listed before a direction, the direction listed is relative to Plant North directed N45E. Where "Plant" is not indicated before a direction, the direction listed is relative to State Plane North.

The current surface conditions at LNP 1 and LNP 2 are as follows:

- LNP 1 is located in a previously undeveloped area with existing ground elevation ranging from approximately 12.3 to 13.2 m (40 to 43 ft.) NAVD88, as shown on Figure 2.5.4.2-201A. The ground surface at LNP 1 is covered with trees, brush, wetlands, and dirt/gravel access roads as shown on Figure 2.5.4.2-201B.

- LNP 2 is located in a previously undeveloped area north of LNP 1, with existing ground surface elevation ranging from approximately 12.1 to 13.4 m (40 to 44 ft.) NAVD88, as shown on Figure 2.5.4.2-201A. The ground surface at LNP 2 is covered with trees, brush, wetlands, and dirt/gravel access as shown on Figure 2.5.4.2-201C.

- Approximate ground surface topography of LNP 1 and LNP 2 is shown on Figure 2.5.1-249.

The nominal final site grade for LNP 1 and LNP 2 is elevation 15.5 m (51 ft.) NAVD88. The surrounding grade will be lower to accommodate site grading, drainage, and local site flooding requirements. The LNP 1 and LNP 2 nuclear islands will be founded at basemat elevation 3.5 m (11.5 ft.) NAVD88, with a mudmat and waterproofing geomembrane extending below the basemat to elevation 3.4 m (11 ft.) NAVD88. Additional excavation below this elevation will be performed to remove the undifferentiated, unconsolidated sediments below the nuclear island foundation, such that the typical subgrade elevation will be -7.3 m (-24.0 ft.) NAVD88. The nuclear island subgrade will be backfilled from -7.3 m (-24.0 ft.) NAVD88 to 3.4 m (11 ft.) NAVD88 with RCC, as discussed in FSAR Subsection 2.5.4.5.3. The planned excavation extents at LNP 1 and LNP 2 safety-related structures are shown in relation to the geologic profiles on Figures 2.5.4.5-201B and 2.5.4.5-202B, as discussed in FSAR Subsection 2.5.4.5.

Locations of boreholes are shown in relation to safety-related structures on Figure 2.5.4.2-201A. Borehole logs are included in Appendix 2BB. Geologic mapping of the nuclear island excavations will be performed prior to fill placement and construction of the nuclear island.

2.5.4.4 Geophysical Surveys

LNP COL 2.5-6 Geophysical surveys were performed to characterize the properties of soil and rock at the LNP site. The borehole geophysical survey methods, scope, objectives, and results of the surveys are presented in this subsection.

During the pre-COLA site selection investigations, surface refraction and microgravity surface geophysical surveys were performed in addition to a series of preliminary boreholes. It was found that these geophysical survey investigation methods did not produce reliable results at the LNP site due to numerous subsurface heterogeneities including the presence of soft or weathered zones below the top of rock and variable soil depth. As a result, the COLA investigation instead included a large number of borehole geophysical logs and surveys. The pre-COLA surface geophysical survey results are not considered part of the COLA data set.

2.5.4.4.1 Descriptions of Borehole Geophysical Surveys

Table 2.5.4.4-201 summarizes the borehole geophysical surveys and logs that were used to characterize the properties of soil and rock at the LNP site. Descriptions of the methods used for each set of tests are provided in the following subsections, followed by the results of these tests.

2.5.4.4.1.1 Suspension P-S Velocity Logging Surveys

Suspension P-S velocity logging surveys were conducted in 18 boreholes by GeoVision of Corona, California (References 2.5.4.4-201 and 2.5.4.4-202) at LNP 1 and LNP 2. These surveys were conducted to characterize V_S and V_P profiles with depth. The surveys were performed in two stages. Stage 1 included measurements in four uncased boreholes (Boreholes A-07, A-08, A-19, and A-20) and 10 PVC-cased boreholes (Boreholes I-01 through I-10), and were performed between February 25, 2007, and May 4, 2007 down to depths between 70 m (230 ft.) and 88 m (289 ft.). Where used, PVC casing was grouted into the borehole using a stiffness compatible grout, which is approximately compatible to stiffness of surrounding rock, comprised of Portland cement, bentonite, and water. Each borehole using PVC casing was grouted by tremie methods. Stage 2 involved measurements in four uncased deep boreholes (Borehole AD-01 through AD-04) between 61.6 (202 ft.) and 148 m (486 ft.) (below the casings), and were performed between August 29, 2007, and September 25, 2007.

Suspension logging is the primary geophysical method used for characterizing the dynamic properties of soil and rock at the LNP site. These surveys were conducted using an OYO Suspension P-S Logging System. In this method, a seismic source and two receivers are mounted as a single unit and suspended in the borehole fluid by cable. The source energy generates a compression wave within the borehole fluid, which is propagated as compression and shear waves along the borehole wall. The time difference for the waves to arrive at the two geophones, located at higher elevations than the source within the borehole and separated by a known distance of 1 m (3.3 ft.), is used to calculate V_S and V_P of the rock or soil near the borehole wall between the geophones.

The probe was lowered to the bottom of borehole, and moved upward a vertical distance of 0.49 m (1.6 ft.) between measurements. At each measurement, two opposite horizontal and one vertical wave records were generated for the development of semi-continuous profiles of V_P and V_S within each tested borehole. Results of these surveys are summarized in Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B.

2.5.4.4.1.2 Acoustic Televiewer Surveys

GeoVision of Corona, California, also performed acoustic televiewer, caliper, and deviation surveys in the boreholes that were used for Suspension P-S velocity logging. These surveys were conducted to establish the verticality of the boreholes and for uncased boreholes to collect acoustic images of the borehole walls. The acoustic image information was used to evaluate areas where low rock RQD and rapid drilling or rod drops occurred. Such conditions were possibly indicative of dissolution features that could affect rock stability. The acoustic televiewer surveys were also used to characterize the dip and orientation of planar features in these boreholes (including bedding planes and fractures).

As for the suspension logging, the acoustic surveys were performed in two stages. Stage 1 included verticality (or deviation) and acoustic imaging surveys in 4 uncased boreholes (Boreholes A-07, A-08, A-19, and A-20) and verticality surveys in 10 PVC-cased boreholes (Boreholes I-1 through I-10), and were performed between February 25, 2007, and May 4, 2007. Stage 2 included verticality and acoustic imaging surveys in four uncased deep boreholes (Boreholes AD-01 through AD-04) below depths of 61 m (200 ft.) (below the casings), and were performed between August 29, 2007, and September 25, 2007. Because acoustic images cannot be taken in the PVC-cased boreholes, only the deviation data were collected in the I-series boreholes.

These surveys were performed using a Robertson High-Resolution Acoustic Televiewer probe. This device provides oriented high-resolution images of the borehole walls in “pseudo-color.” The probe scans the borehole wall using an ultrasound beam, and the amplitude and travel time of the reflected signal are recorded simultaneously by the probe. The dip and orientation of the probe are also recorded for result calibration. Features, such as fractures and bedding planes, appear as sinusoidal traces on the oriented images produced by these surveys. The traces are then used to calculate the strikes and dips of such features.

The probe was lowered to the bottom of borehole, and moved upward in vertical increments of 0.002 m (0.1 in.) to provide a continuous acoustic image of the borehole wall.

2.5.4.4.1.3 Downhole Surveys

GeoVision of Corona, California, performed downhole surveys in 14 boreholes. These surveys were performed in two stages: stage 1 included measurements in four PVC-cased boreholes (Boreholes I-02, I-05, I-07, and I-10) that were performed between May 1, 2007, and May 3, 2007; and stage 2 involved measurements in four uncased deep boreholes (Boreholes AD-1 through AD-04) and six PVC-cased boreholes (Boreholes I-01, I-03, I-04, I-06, I-08, and I-09) that were performed between November 30, 2007, and December 6, 2007. The downhole logging was performed as a secondary method to the Suspension P-S velocity logging surveys for the characterization of the V_S and V_P profiles.

PVC casing was used in some boreholes to maintain borehole wall stability. The casing was grouted into the borehole using a stiffness compatible grout comprised of Portland cement, bentonite, and water. Each borehole using PVC casing was grouted by tremie methods.

In this method, an oriented geophone probe was secured against the borehole wall at a known depth, and a sequence of waves, two opposite horizontal and one vertical, were generated at the ground surface by striking a horizontal board with a sledgehammer. The time for wave arrival at the downhole probe was used to calculate wave velocity between the probe and the ground surface. The probe was moved a vertical distance of 0.76 m (2.5 ft.) in the top 6.1 m (20 ft.), 1.5 m (5 ft.) between depths of 6.1 and 45.7 m (20 and 150 ft.), and 3 m (10 ft.) for depths below 45.7 m (150 ft.) between measurements. The V_P data were obtained as the probe was lowered down, and the V_S data were acquired as the probe was raised to the ground surface. The data were then processed to generate the profiles of V_S and V_P with depth for each tested borehole. Results of these surveys are presented in Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B.

2.5.4.4.1.4 Non-Seismic Geophysical Surveys

A series of geophysical surveys were conducted using non-seismic methods. The purpose of these surveys was to identify and correlate where possible the primary geologic units based on similarity in natural gamma, gamma-gamma, neutron-neutron, and conductivity properties. These tests were conducted using Mount Sopris MGXII digital logging unit and winch with 457 m (1500 ft.) of single conductor cable. The non-seismic geophysical surveys were conducted by Technos Inc., of Miami, Florida (Reference 2.5.4.4-203).

- Natural Gamma Log: Technos performed the natural gamma loggings in the 10 PVC-cased I-series boreholes (I-1 through I-10) and 4 deep boreholes (AD-1 through AD-4) from October 24 through December 2, 2007. This survey involved the measurements of the amount of natural gamma radiation emitted from the borehole wall. The primary use of the log was for identification of lithology and stratigraphy of subsurface soils and rocks, especially those of clays and shales. The radius of measurements for this logging method was 15 to 30 cm (6 to 12 in.). The testing data were collected at approximately 0.03 m (0.1 ft.) vertical intervals. (Reference 2.5.4.4-203)

- Gamma-Gamma (Density) Log: Technos conducted the gamma-gamma (density) logging in the same boreholes where the natural gamma logging were performed (Boreholes I-01 through I-10 and AD-01 through AD-04). This method used both a radiation source and a detector (receiver). The gamma rays emitted from the source were scattered by the surrounding soils and rocks, and reflected back to the detector as a function of the bulk density of the soil media. The gamma-gamma log provides a response that is averaged over a vertical distance of 47 cm (18.5 in.) within the borehole because of the distance between the source and the detector. The radius of measurements for this logging method was only about 15 cm (6 in.). Therefore, borehole diameter variations and well construction factors will affect gamma-gamma log more than other logs presented in this section. (Reference 2.5.4.4-203)

- Neutron-Neutron (Porosity) Log: Neutron-neutron (porosity) loggings were also performed by Technos in the same boreholes where the above gamma-gamma surveys were performed (Boreholes I-01 through I-10 and AD-01 through AD-04). The primary use of this logging was to identify relative moisture contents and porosity of the surrounding soils and rocks. Since water is bounded by clay minerals, the results of the logging may also be used to indicate the presence of clays in the soils. The radius of measurements for this logging method was approximately 15 to 30 cm (6 to 12 in.). Therefore, borehole diameter variations and well construction factors can affect this log, but not as severely as the density log. The testing data were collected at approximately 0.03 m (0.1 ft.) vertical intervals. (Reference 2.5.4.4-203)

- Induction (Conductivity) Log: Induction (conductivity) loggings were performed by Technos in the same boreholes where the above geophysical loggings were performed (Boreholes I-01 through I-10 and AD-01 through AD-04). This logging measured the electrical conductivity of the borehole wall to a radius of about 0.76 m (2.5 ft.) from the probe. The testing data were collected at approximately 0.03 m (0.1 ft.) vertical interval. The electrical conductivity is a function of soil/rock type, porosity, permeability, and pore fluid composition. (Reference 2.5.4.4-203)

2.5.4.4.2 Geophysical Survey Investigation Results

The results of the borehole geophysical surveys establish the V_S and V_P profiles, lithology, and other material properties within the soil and rock underlying and adjacent to the nuclear islands for the LNP site. The following subsections summarize these results.

2.5.4.4.2.1 Suspension P-S Logging Surveys

The results of suspension logging surveys at LNP 1 and LNP 2 are superimposed over the plant north-south and east-west geologic fence diagrams in Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B, respectively. These figures show V_S as a function of depth for each of the boreholes to the maximum depth of the survey. The results of suspension logging surveys at LNP 1 and LNP 2 are summarized below.

2.5.4.4.2.1.1 LNP 1 (South Reactor)

The following trends are indicated by the suspension logging V_S data in Boreholes A-19, A-20, AD-03, and AD-04 (mud rotary boreholes) and Boreholes I-06 through I-10 (sonic boreholes) at LNP 1:

- The transition from low V_S in soil to higher V_S in rock occurs gradually from depth of about 16.7 to 24.4 m (55 ft. to 80 ft.) bgs. The V_S measured above the top of rock ranges from about 380 to 1410 m/sec (1250 ft/sec to 4630 ft/sec). The lower V_S value of about 380 m/sec (1250 ft/sec) was measured in Borehole I-07 (a sonic-cased borehole) at a depth of 12.5 m (41.0 ft.) bgs, while the higher V_S value of 1410 m/sec (4630 ft/sec) was obtained in Borehole I-07at a depth of 11 m (36.1 ft.) bgs.

- Rock layers have been identified at LNP 1 by grouping rock elevation intervals with similar V_s magnitude and variability. The V_S measurements indicate the following rock layers (elevation ranges):

  – SAV-1: This layer ranges from top of rock, which is presented in Table 2.5.4.2-207, to elevation -54.9 m (-180 ft.) NAVD88 (approximate depth of 67.7 m [222 ft.] bgs), and is characterized by average V_S value of 1199 m/sec (3932 ft/sec).

  – SAV-2: This layer ranges from elevation -54.9 to -94.2 m (-180 ft. to -309 ft.) NAVD88 (approximate depth of 67.7 to 106.9 m (222 ft. to 351 ft.] bgs), and is characterized by average V_S value of 894 m/sec (2932 ft/sec).

  – SAV-3: This layer ranges from elevation -94.1 to -139.6 m (-309 to -458 ft.) NAVD88 (approximate depth of 106.9 to 152.4 m [351 ft. to 500 ft.] bgs), and is characterized by average V_S value of 1170 m/sec (3839 ft/sec).

- The V_S profiles in rock measured below 61 m (200 ft.) bgs in the deep boreholes (Boreholes AD-03 and AD-04) show slight increases in V_S with depth (up to the maximum depth surveyed at 148 m [486 ft.] bgs).

- Figure 2.5.4.4-201 shows the superimposed V_S profiles at their measured elevations from all the suspension logging borehole surveys at LNP 1. Although the results show a large variation of V_S profile with elevation at LNP 1, V_S in rock is generally above 600 m/sec (2000 ft/sec), with some exceptions near the top of the SAV-1 layer and near the interface between the SAV-1 and SAV-2 layers.

- Figure 2.5.4.4-202 compares the V_S profiles obtained in a mud rotary borehole (Borehole A-19) to that measured in an adjacent sonic borehole (Borehole I-06). The comparison indicates a slightly higher V_S profile obtained in the mud rotary borehole than in the sonic borehole.

The results of the Suspension P-S surveys in the uncased boreholes indicate that the borehole conditions below the depth of about 15.2 m (50 ft.) were well-suited for suspension velocity logging, resulting in good quality velocity profiles. However, logging in the top 15.2 m (50 ft.) of the boreholes generated more erratic results due to collapse and erosion of the borehole wall and the presence of casing. The velocity logging in the five PVC-cased I-series boreholes generally resulted in fair to poor or uninterpretable data, due mainly to signal degradation caused by the sonic drilling technique and poor coupling of the PVC casing. Measurements at several depths could not be processed sufficiently to calculate velocity values, and results were, therefore, not reported for these depths.

2.5.4.4.2.1.2 LNP 2 (North Reactor)

The following trends are indicated by the suspension logging V_S data at Boreholes A-07, A-08, AD-01, and AD-02 (mud rotary boreholes) and Boreholes I-01 through I-05 (sonic boreholes) at LNP 2:

- The transition from low V_S in soil to higher V_S in rock occurs at depths of about 21.3 m (70 ft.) bgs. The V_S measured above the top of rock varied from about 190 to 1311 m/sec (620 ft/sec to 4300 ft/sec). The lower V_S value of about 190 m/sec (620 ft/sec) was measured in Borehole I-05 (a sonic cased borehole) at a depth of approximately 14.5 m (47.6 ft.) bgs; while the higher V_S value of 1311 m/sec (4300 ft/sec) was obtained in Borehole A-08 (a mud rotary uncased borehole) at a depth of 12 m (39.4 ft.) bgs.

- Rock layers have been identified at LNP 2 by grouping rock elevation intervals with similar V_s magnitude and variability. The V_S measurements indicate the following rock layers (elevation ranges):

  – NAV-1: This layer ranges from top of the rock, which is presented in Table 2.5.4.2-207, to -29.6 m (-97 ft.) NAVD88 (approximate 42.3 m [139 ft.] bgs), and is characterized by average V_S value of1116 m/sec (3660 ft/sec).

  – NAV-2: This layer ranges from elevation -29.6 to -45.1 m (-97 to -148 ft.) NAVD88 (approximate depth of 42.3 to 57.9 m [139 to 190 ft.] bgs), and is characterized by average V_S value of 1407 m/sec (4614 ft/sec).

- NAV-3: This layer ranges from elevation -45.1 to -92.3 m (-148 to -303 ft.) NAVD88 (approximate depth of 57.9 to 105.1 m [190 to 345 ft.] bgs), and is characterized by average V_S value of 944 m/sec (3097 ft/sec).

- NAV-4: This layer ranges from elevation -92.3 to -139.6 m (-303 ft. to -458 ft.) NAVD88 (approximate depth of 105.1 to 152.4 m [345 ft. to 500 ft.] bgs), and is characterized by average V_S value of1208 m/sec (3963 ft/sec).

- The V_S profiles in rock measured below 61 m (200 ft.) bgs in the deep boreholes (Boreholes AD-01 and AD-02) show slight increase in V_S with depth (up to the maximum depth surveyed at 148 m [486 ft.] bgs).

- Figure 2.5.4.4-203 shows the superimposed V_S profiles at their measured elevations from all the suspension logging borehole surveys at LNP 2. Although the results show a large variation of V_S profile with depth at LNP 2, V_S in rock is generally above 600 m/sec (2000 ft/sec) except for an approximately 12 m (40 ft.) depth interval at Borehole I-02 within the NAV-3 layer.

- Figure 2.5.4.4-204 compares the V_S profiles obtained in a mud rotary borehole (Borehole A-07) to that measured in an adjacent sonic borehole (Borehole I-01). The comparison indicates consistent V_S values obtained in these boreholes drilled using the two drilling methods. The overall V_S profile shows that the values measured in the sonic boreholes are generally comparable to those obtained in the mud rotary boreholes.

Similar to the observations for LNP 1, the results of the Suspension P-S surveys in the uncased boreholes indicate that the borehole conditions below the depth of about 15.2 m (50 ft.) were well-suited for suspension velocity logging. However, logging in the top 15.2 m (50 ft.) of the boreholes generated more erratic results due to collapse and erosion of the borehole wall and presence of casing. The logging in the five PVC-cased I-series boreholes generally resulted in fair to poor or uninterpretable data, due mainly to signal degradation caused by the sonic drilling technique and poor coupling of the PVC casing.

As summarized above, V_S and V_P measured in rock at LNP 1 are marginally lower than at LNP 2. As shown on Figures 2.5.4.4-201 and 2.5.4.4-203, results for LNP 1 layer SAV-1 exhibits more variation than the corresponding shallow rock results for LNP 2 layers NAV-1 and NAV-2. Below these layers, results at LNP 1 (layers SAV-2 and SAV-3) and at LNP 2 (layers NAV-3 and NAV-4) exhibit similar results and variation at corresponding elevations. The suspension logging measurements produced average wave velocities over a 1 m (3.3 ft.) interval. The precisions of the measured V_S and V_P are estimated to be about 5 percent and 10 percent, respectively. (Reference 2.5.4.4-201)

2.5.4.4.2.2 Acoustic Televiewer Surveys

The acoustic images generated by the televiewer surveys provide clear images of the walls in the uncased boreholes. These images show fractures, bedding planes, and eroded areas (voids). These voids are limited to about 30.5 cm (12 in.) in maximum height in surveyed boreholes based on surveyed results.

The maximum borehole deviation from the true vertical was measured at 2.6 degrees. This deviation results in a maximum depth error in the boreholes of 8 cm (0.27 ft.) over a depth of 79.3 m (260.2 ft.) (or 0.1 percent), which is less than the 0.4 percent limit after survey depth error allowed by ASTM D5753-05 (Reference 2.5.4.4-204). No adjustments on the borehole log depths, hence, are required.

The computer program Dips was used to calculate mean dip magnitude and dip direction based on the bedding and joint/fracture data from the acoustic televiewer by first plotting the dip magnitudes and dip directions of several data sets on a stereonet, and selecting the subgroups of features to include in the mean calculation (Reference 2.5.4.4-205). The mean of the dip magnitude and dip direction values were then calculated using geometric addition of three-dimensional vectors. The global mean plane and pole were then plotted on the stereonet. The global mean is reported as unweighted and weighted dip magnitude and dip direction as provided in Table 2.5.4.4-202. The weighted global mean dip magnitude and dip direction were calculated using Terzaghi weighting methods (Reference 2.5.4.4-206).

Following is a summary of key observations from the acoustic televiewer surveys and corresponding stereonet analyses:

- As indicated in Table 2.5.4.4-202, bedding features exhibit good correlation and allowed calculation of mean bedding plane dip magnitude and direction. Based on these results, the means of the bedding planes are essentially horizontal at both LNP 1 and LNP 2.

- Numerous non-bedding planar fractures (interpreted as joints) were also identified in the acoustic televiewer logs. The stereonet analysis indicated that these features exhibit little correlation and have essentially random orientation at the LNP site.

- Two essentially vertical open fractures were observed in the acoustic logs, one each at borehole A-08 (at LNP 2) and A-19 (at LNP 1). Observation of even a few essentially vertical features in vertical boreholes is considered significant (i.e., not commonly expected due to the parallel orientation of the features and the boreholes), and they likely indicate the presence of vertical joints at the LNP site.

Acoustic televiewer surveys were conducted in Boreholes A-07, A-08, A-19, A-20, and AD-01 through AD-04, located within the footprints of the nuclear islands.

2.5.4.4.2.3 Downhole Velocity Surveys

The interpreted V_S profiles from the downhole surveys at LNP 1 (Boreholes I-6 through I-10, AD-03, and AD-04) and LNP 2 site (Boreholes I-01 through I-05, AD-01 and AD-02) are presented in Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B, along with the V_S profiles obtained from the suspension logging surveys.

The measured downhole velocity data are generally fair in quality, due mainly to the poor transmission of the surface generated signals into the underlying hard rock and conversion of the shear waves to other modes at the shallow fracture zones (Reference 2.5.4.4-201). In some of the cased boreholes, poor coupling of the PVC casing appeared to affect the ability to interpret wave arrival times.

The downhole survey method evaluates the wave velocities based on arrival times from the ground surface to the depth of the receiver, compared with a much shorter receiver-to-receiver distance of 1 m (3.3 ft.) for the suspension logging method. Therefore, the downhole method does not provide the same high-resolution of thin wave velocity variations with depth as is provided by the suspension logging method. For this reason, velocity variations within intervals detected by suspension logging are not as apparent in the downhole results.

2.5.4.4.2.4 Natural Gamma Log

The natural gamma log was included in the downhole logging suite at the LNP site because it is a passive log that can be run through cased boreholes to supplement the subsurface lithologic and stratigraphic unit interpretation for the site. The natural gamma log provides measurement of the gamma radiation emitted by rocks and unconsolidated materials surrounding the borehole, correlating to clay content in this application. The depth, thickness, variability, and lateral extent of any clay layers present at the LNP site is an important element for consideration in foundation design and interpretation of site geologic conditions.

Representative logging results are presented for two boreholes each at LNP 2 (Boreholes I-01 and AD-01) and LNP 1 (Boreholes I-06 and AD-03) in Figures 2.5.4.4-205A, 2.5.4.4-205B, 2.5.4.4-205C, and 2.5.4.4-205D, respectively. For Boreholes AD-01 and AD-03, which were advanced by rotary methods, the rock core column is also presented along with the geophysical results. The logging results indicate very low natural gamma values of less than 30 counts per second (cps) in the natural sands, silts and limestone, and the natural gamma values increase above 40 cps, to as much as 100 cps, where clay content increases. Typically, there is an indication of increased clay content in the shallow portions of the boreholes, within the soil deposit above the top of rock (Reference 2.5.4.4-203). A clear high natural gamma response is observed at approximately 10.7 to 12.2 m (35 to 40 ft.) bgs in several of the boreholes at LNP 1 and LNP 2, including each of the four boreholes shown on Figures 2.5.4.4-205A, 2.5.4.4-205B, 2.5.4.4-205C, and 2.5.4.4-205D. The top of rock typically lies within the high natural gamma responses, likely indicating a weathered rock zone. The following observations were made from the natural gamma logs at each LNP 1 and LNP 2:

- South Reactor Site (LNP 1): There is a shallow higher natural gamma response. However, the high gamma count extends from the ground surface downward, unlike at the north reactor site. This pattern may indicate a more weathered, clayey, shallow environment at LNP 1. Between the approximate depths of 42.6 to 57.9 m (140 and 190 ft.) bgs, there is a subtle signature seen in most of the natural gamma logs. This signature is characterized by a thin and subtle increase in response around 42.6 m (140 ft.) deep, a general decrease in response, and a second thin and subtle increase in response at around 57.9 m (190 ft.) bgs. This feature is identified in two boreholes logged at LNP 1. The only other notable features in the natural gamma logs are unusually low responses by about 10 to 20 cps, seen in Borehole I-06, and a small zone in Borehole AD-04. It may indicate sandier or cleaner limestone conditions considering no construction differences in the I-series boreholes.

- North Reactor Site (LNP 2 site): The natural gamma logs indicate average background values of about 30 to 40 cps, both above and below the higher natural gamma response seen in several boreholes at the LNP sites at approximately 10.7 to 12.2 m (35 to 40 ft.) bgs. The depth or thickness of the higher natural gamma response is fairly uniform at the LNP 2 site. The magnitude of the natural gamma response in this zone is also fairly uniform, with the exception of Borehole I-05. In Borehole I-05, the response is at least 1.5 times that in other boreholes, indicating more clay. Borehole I-05 is located to the northeast of the LNP 2 nuclear island, and may indicate a change in shallow subsurface conditions toward that area. The subtle signature between the approximate depths of 42.6 to 57.9 m (140 and 190 ft.) bgs seen in boreholes at LNP 1 was also observed in most of boreholes logged at LNP 2. The only other notable features in the natural gamma logs are unusually low responses by about 10 to 20 cps, seen in Borehole I-02.

2.5.4.4.2.5 Gamma-Gamma (Density) Log

Of the geophysical methods, the gamma-gamma (density) log measures the smallest volume of soil or rock surrounding the borehole used on this project (that is, the measurement samples the soil or rock closest to the borehole wall). Representative logging results for two boreholes each at LNP 2 (Boreholes I-01 and AD-01) and LNP 1 (Boreholes I-06 and AD-03) are presented in Figures 2.5.4.4-205A, 2.5.4.4-205B, 2.5.4.4-205C, and 2.5.4.4-205D, respectively.

High gamma-gamma measurements within rock formations provide an indication of whether soil in-fill is present, and this information could be related to poor rock quality. However, the gamma-gamma log could be affected by drilling or construction issues. For example, the average values for the AD-series boreholes (advanced by rotary methods) are about 4000 to 4500 cps, but the average values in the I-series boreholes (advanced by sonic method) are about 5000 cps. This difference is likely due to the difference in drilling and casing diameter. Localized low-density zones (< 1.5 m [5 ft.] thick) are commonly and randomly seen throughout the logged boreholes. Significant low-density zones are those that are thicker or have a greater magnitude response (about 1.5 times background values or more, > 7500 to 9000 cps), indicating low-density. However, there are also zones of thinner low-density zones or zones of relative lower-density, which are seen as more subtle changes in response (greater than background but less than 7500 cps). The following observations were made from the gamma-gamma logs at each LNP 1 and LNP 2:

- South Reactor Site (LNP 1): At LNP 1, the thickest low-density zone occurs in Borehole I-06, and it extends from the depths of 26.5 to 42 m (87 to 138 ft.) bgs. Boreholes I-09, AD-03, and AD-04 also show low-density zones that are all on the order of only 6 m (20 ft.) thick. These zones also appear to be randomly distributed. However, the low-density zones in Boreholes AD-03 and AD-04 are at similar depths, and these boreholes are only approximately 15.2 m (50 ft.) from each other. It is possible that they represent a similar low-density zone. In Boreholes AD-03 and AD-04, there are two deeper subtle low-density zones that occur at the same depths. One low-density zone is from depths of about 103.6 to 106.7 m (340 to 350 ft.), and the other is a much thinner response centered at about the depth of 131.9 m (433 ft.). The deeper feature is associated with a peat layer recorded at approximately this same depth on the borehole logs for boreholes AD-03 and AD-04.

- North Reactor Site (LNP 2 site): At LNP 2, the single thickest low-density zone occurs in Borehole I-02, and it extends from 22.9 to 50.3 m (75 to 165 ft.) bgs. Borehole I-05 shows the greatest magnitude response within two low-density zones. One low-density zone is located between depths of 15.2 to 24.4 m (50 and 80 ft.), and another zone is between depths of 42.7 to 48.8 m (140 and 160 ft.). Other thinner, but still significant low-density, zones occur in Boreholes I-01, I-05, and AD-01. These appear to be randomly distributed and to have no spatial correlation.

The gamma-gamma logs were compared to a summary of indicators for possible karst features taken from the drilling logs. The comparison did not include any of the I-series boreholes or the upper 61 m (200 ft.) of the AD-series boreholes, because they were drilled using the sonic drilling method. The data from the closest boreholes, therefore, were used for indications of karst features. The features identified from the drilling data that may indicate karst features do not consistently correlate with low-density zones at either the north or south reactor sites. For the I-series boreholes and shallow portions of the AD-series boreholes, this could be due to extrapolating the data from one borehole to another. The lack of correlation may indicate that both the low-density zones identified in the geophysical logs and the features summarized from the drilling data have limited spatial extent.

The significant low-density zones indicated in the gamma-gamma logs all occur shallower than 61 m (200 ft.) deep. Below 61 m (200 ft.), the gamma-gamma logs show a few subtle low-density zones (thinner or lower magnitude response). For example, in most of the deep boreholes a relatively subtle low-density response is observed corresponding to the relatively low V_s interval near the top of the SAV-2 (LNP 1) and middle of the NAV-3 (LNP 2) rock layers. The gamma-gamma log measures a very small radius from the probe, and therefore in the upper 61 m (200 ft.) it may be responding to areas of less consolidated materials that have been disturbed during drilling.

2.5.4.4.2.6 Neutron-Neutron (Porosity) Log

This logging measures a slightly larger volume of soil or rock around the borehole wall than the gamma-gamma (density) log. Higher cps values indicate a decrease in porosity and lower cps values indicate an increase in porosity. When comparing the neutron-neutron log with the gamma-gamma log, they generally should move opposite from one another. Representative logging results are presented for two boreholes each at LNP 2 (Boreholes I-01 and AD-01) and LNP 1 (Boreholes I-06 and AD-03) in Figures 2.5.4.4-205A, 2.5.4.4-205B, 2.5.4.4-205C, and 2.5.4.4-205D, respectively.

Background conditions in the neutron-neutron logs are relative in nature, both within a single borehole, as well as between boreholes. The background values measured with the neutron-neutron (porosity) log are about 80 cps in the AD-series boreholes, and about 100 cps in the I-series boreholes. The AD-series and I-series boreholes were drilled using different techniques; they were constructed and cased using different casing diameters that would account for the slight differences in background values.

In general, the neutron-neutron logs for the south reactor site (LNP 1) show slightly higher values of about 120 cps than the north reactor site (LNP 2), which has a background of about 100 cps. This indicates an overall lower porosity (higher density) at LNP 1. One of the I-series boreholes (Borehole I-9) at LNP 1 had higher cps values (by 20 to 40 cps) than the other I-series boreholes, indicating an overall lower porosity at this location.

One pattern that is seen in most boreholes is a relatively lower porosity zone that generally lies between the depths of 42.6 and 57.9 m (140 and 190 ft.) bgs. This is more distinctly seen in the boreholes logged at LNP 2. At LNP 1, this lower porosity zone is broader, and has less distinct upper and lower boundaries. This correlates with the subtle, but repeatable signature, seen in the natural gamma log.

2.5.4.4.2.7 Induction (Conductivity) Log

This logging measures the largest volume of all the geophysical logs performed, at about 0.76 m (2.5 ft.) radius from the probe. The natural sands, silts, and limestone at the site are characterized by induction conductivity values of 30 to 40 Millisiemens per meter (mS/m). The induction log conductivity values may increase in the presence of clays due to their electrical conductivity or where pore fluids have an increase in specific conductance. The induction log values increase above 40 to 100 mS/m or more under these conditions. Representative logging results for two boreholes each at LNP 2 (Boreholes I-01 and AD-01) and

LNP 1 (Boreholes I-06 and AD-03) are presented in Figures 2.5.4.4-205A, 2.5.4.4-205B, 2.5.4.4-205C, and 2.5.4.4-205D, respectively.

The following observations were made from the induction logs at each LNP 1 and LNP 2:

- South Reactor Site (LNP 1): Results at LNP 1 indicate high conductivities in the middle portion of the boreholes, generally from the depths of 27.4 to 56.3 m (90 to 185 ft.). These appear to be thin, localized high conductivities, and randomly distributed throughout this depth. Borehole AD-03 has a greater concentration of these thin, high conductivity layers within this zone. Borehole I-09 shows the lowest conductivities throughout the log, indicating lower levels of conductivity materials (silts/clays) at this location.

- North Reactor Site (LNP 2 site): The induction log generally shows little variations throughout the log. At LNP 2, the logs show more uniform conductivities in the upper portions of the boreholes (generally less than 30.4 m [100 ft.] deep) than at LNP 1. A thin, high conductivity, response occurs at a depth of about 27.4 to 28.9 m (90 to 95 ft.) in all induction logs at LNP 2. Because this feature is seen in all logs, and has a large spatial extent, it is likely associated with geologic conditions at the site. This feature is not clearly seen at LNP 1.

The induction logs from the four deep AD-series show a thin, very high conductivity, response at depths of 126.4 and 131 m (415 and 430 ft.). This very distinct response is seen at both LNP 1 and LNP 2, which indicates that it has a large spatial extent and is likely associated with a geologic feature at the site. This feature is also seen in the natural gamma logs with an increase in clays indicated, more strongly at LNP 1 than at LNP 2. A review of the drilling logs for the AD-series boreholes reveals that a silty clay layer (at LNP 2) and organic peat layers (at LNP 1) were encountered at these depths.

2.5.4.4.2.8 Criteria for Use of Geophysical Survey Results as Design Parameters

Multiple geophysical survey methods were implemented at the LNP site. The following information summarizes how the data from each of these methods have been used for engineering analyses, and the basis for the use:

- **Suspension P-S Velocity Logging Survey Data.** Suspension P-S velocity logging profiles provide high-resolution measurements of V_S and V_P at discrete depths. The results are confirmed by comparison with results from other methods. For these reasons, the suspension logging results are used as the primary source of V_S and V_P data for engineering analyses.

- **Downhole Velocity Survey Data.** Downhole logging profiles permit interpretations of V_S with depth, but with resolution over significantly longer depth intervals than the suspension logging results, as indicated on

Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B. For this reason, the downhole velocity results do not provide the same level of resolution with depth as do the suspension logging results. The downhole results are similar to the depth-averaged V_S results from the suspension logging profiles. Therefore, they serve as a confirmation of the suspension logging results.

- **Acoustic Televiewer, Caliper, and Deviation Survey Data.** Acoustic televiewer images provide confirmation that bedding is essentially horizontal at the site, that non-bedding discontinuities appear randomly distributed in orientation, and that approximately vertical open fractures or joints are present at the site. The caliper data provide additional data to confirm the characterization of karst features, and the deviation survey data confirm that the boreholes were advanced in practically vertical orientations. Aside from these confirmatory uses, these results have not been further used for engineering analyses.

- **Non-Seismic Geophysical Surveys.** The various non-seismic geophysical survey methods (natural gamma, gamma-gamma, neutron-neutron, and induction logs) provide information to supplement the site geologic characterization and to confirm borehole observations. These results have not been explicitly used for engineering analyses.

LNP COL 2.5-6

Table 2.5.4.4-201
Borehole Geophysical Surveys and Logging Performed at the LNP Site

Survey Method	Boreholes	Main Objective
Suspension P-S velocity logging	I-01 to I-10 A-07, A-08, A-19, A-20 AD-01 to AD-04	To obtain the V_P and V_S profiles
Downhole velocity logging	I-01 to I-10 AD-01 to AD-04	To obtain an independent estimate of V_P and V_S measurements
Acoustic televiewer:		To collect acoustic images of borehole walls
Image and verticality	A-07, A-08, A-19, A-20 AD-01 to AD-04	
Verticality only	I-01 to I-10	
Natural gamma log	I-01 to I-10 AD-01 to AD-04	To identify lithology and stratigraphy
Induction log	I-01 to I-10 AD-01 to AD-04	To determine soil/rock type, porosity, permeability, and composition of pore fluids
Gamma-gamma (density) log	I-01 to I-10 AD-01 to AD-04	To determine relative bulk density and identify lithology
Neutron-neutron (porosity) log	I-01 to I-10 AD-01 to AD-04	To indicate relative changes in moisture content and porosity

LNP COL 2.5-1
LNP COL 2.5-6

Table 2.5.4.4-202
Discontinuity Data from Dips Analysis

Site	Mean Dip (°)	Mean Dip Direction (°)	Weighted Dip (°)	Weighted Dip Direction (°)	Description
LNP 1	1	118	2	118	Bedding Plane
LNP 2	2	253	2	254	Bedding Plane

Notes:

Numerous non-bedding planar fractures (interpreted as joints) were also identified in the acoustic televiewer logs. The stereonet analysis indicated that these features exhibit little correlation and have essentially random orientation at the LNP site.

2.5.4.5 Excavations and Backfill

LNP COL 2.5-7 Soil and rock excavations will be required to construct the LNP nuclear islands on rock at a subgrade elevation of approximately -7.3 m (-24 ft.) NAVD88. This subsection describes the anticipated excavation and backfill plans for the nuclear islands, including the planned diaphragm walls, excavation extents and methods, and the properties of backfill beneath and adjacent to safety-related structures.

Construction sequencing for these activities is described in FSAR Subsections 2.5.4.12 and 3.8.5.10.

2.5.4.5.1 Diaphragm Walls and Grouting

In order to support the excavation of the nuclear islands, reinforced concrete diaphragm walls will be constructed as the boundary of the excavation limits. These excavation limits are discussed in FSAR Subsection 2.5.4.5.2 and are shown on Figures 2.5.4.5-201A, 2.5.4.5-201B, 2.5.4.5-202A, and 2.5.4.5-202B.

These diaphragm walls will be installed, prior to excavation, from the existing ground surface ranging from an approximate elevation of 12.8 to 13.1 m (42 to 43 ft.) NAVD88 at LNP 1, and approximately 12.5 to 13.1 m (41 to 43 ft.) NAVD88 at LNP 2. The diaphragm walls will serve as a temporary excavation support system to facilitate excavation to elevation -7.3 m (-24 ft.) NAVD88, and will extend in depth to elevation -16.5 m (-54 ft.) NAVD88 to support construction dewatering, as discussed in FSAR Subsection 2.5.4.6.2. Constructed approximately 9.1 m (30 ft.) into rock, the diaphragm walls will be advanced using a kelly-mounted Hydrofraise excavator, standard practice for the installation of such walls. For the portion of the diaphragm wall under the Turbine Building, Annex Building, and the Radwaste Building foundation mat, the top of the diaphragm wall will be at least 1.5 m (5 ft.) below the bottom of the respective buildings' foundation mat as shown in Figure 3.7-226.

The diaphragm walls will include seven rows of prestressed anchors, spaced as shown on Figure 2.5.4.5-203. The anchors will be inclined at 45 degrees and bonded into the limestone of the Avon Park Formation. The prestressed anchors will be placed at 3 m (10 ft.) spacing around the entire perimeter of each diaphragm wall.

For design purposes of the diaphragm walls, the concrete compressive strength is to be 4000 psi, with 1 percent reinforcement on both sides of the wall. The minimum required wall thickness is 1.1 m (3.5 ft.).

Concurrent with the installation of the diaphragm walls, a grouting program will be undertaken to form the bottom of the "bathtub" as described in FSAR Subsection 2.5.4.6.2. The grouting operation will be conducted from, at or near, the existing ground surface by drilling boreholes from the surface down to the approximate elevation of -30.2 m (-99 ft.) NAVD88. The top elevation of the grouted zone will be at elevation -7.3 m (-24 ft.) NAVD88, resulting in a 22.9 m (75 ft.) thick grouted zone.

Grouting will generally be performed by the upstage method with pneumatic packers and a suite of grout mixes that range in viscosities from 35 seconds to over 80 seconds. Primary grout holes will be spaced on a 4.8 m (16 ft.) hexagonal pattern, and split-spaced with secondary grout holes to achieve “no take” conditions. Provisions will be in place to perform additional split-spacing to tertiary grout holes, as dictated by the performance of the production grouting. Effective grouting pressures will be limited to approximately 0.5 psi/ft of depth, monitored using a GIN curve and penetrability curve developed during the Grout Test Program. Hole spacing, grouting pressures, and acceptable grout takes will be established with the grout program. The target residual conductivity of the production grouting will be 15 Lugeons. Grouting is nonsafety-related, however it will be performed under a quality program.

2.5.4.5.1.1 Diaphragm Wall with Anchors

A diaphragm wall with prestressed tiebacks will be used as a groundwater cutoff and excavation support system to facilitate the 67-ft.-deep excavation. The analysis includes an assessment of the required diaphragm wall thickness and reinforcement, the arrangement and required number of anchors, the maximum expected anchor load for each construction stage, and the required bonding length of each anchor.

A diaphragm wall system with prestressed tiebacks is planned to enable the excavation and dewatering of the nuclear island. This continuous wall is designed as an excavation support system to facilitate the 67-ft.-deep excavation and prevent excessive groundwater from entering the excavation area.

The diaphragm wall with tiebacks was considered to be a stiff wall system, and construction-sequencing analysis using classical soil pressures was employed for the design.

An earth pressure diagram for a rigid wall (with a fixed base) consists of an apparent earth pressure on the upper section of the wall and a triangular distribution on the lower section of the wall. The earth pressures are based on the at-rest lateral earth pressure condition.

SAP2000 was used to analyze moment and shear force distribution of the continuous beam. For the reinforced concrete component design, the American Concrete Institute (ACI) 318 Ultimate Strength Design (USD) method was used.

As a design input, the groundwater level is assumed to be at ground surface behind the diaphragm wall, and 5 ft. below the excavation in front of the wall; i.e., the excavation is dewatered and there is no water pressure in front of the diaphragm wall during each stage of construction. Full hydrostatic pressure was considered behind the wall.

For each stage of construction, 2 ft. of over-excavation is considered below an anchor location. The bonding strength between grout and limestone rock is interpreted to be 200 psi (1.4 Mpa) based on published data.

The inclination of the anchors is 45 degrees, and all anchors will be keyed into competent rock. The drilled anchor holes are 6 inches in diameter.

The compressive strength of concrete is 4000 psi, and the elastic modulus of concrete 3000 ksi, calculated based on the concrete compressive strength.

The diaphragm wall includes 7 rows of prestressed anchors. To reduce the shear force and moment imposed on the wall by the earth pressure, anchors are closely spaced at the lower section of the wall, and relatively widely spaced at the upper section. The construction sequencing analysis involved eight stages of analysis, each stage considering an over-excavation of 2 ft. below the anchor location.

For the structure component design, ACI 318 USD methodology was used and a load factor of 1.2 was used for the design; allowable strength design (ASD) methodology was used for the anchor design, and a factor safety of 2.0 is used to determine the bonding length.

The concrete compressive strength is to be 4000 psi. The minimum required wall thickness is 3.5 ft., and the reinforcement ratio is to be 1 percent, reinforced on both sides (2 percent total). The embedment into rock is to elevation -54 ft. NAVD.

2.5.4.5.1.2 Permeation Grouting

Due to the high groundwater table and the documented permeability of the Avon Park Formation beneath the site, the upper 75 ft. of the Avon Park Formation will be grouted to diminish its porosity and permeability. The grouting will allow the excavation to be made in a safe and predictable manner by minimizing the upward flow of groundwater into the excavation and to aid in the resistance to uplift pressures on the excavation bottom. An uplift analysis indicated sufficient reduction of shear stresses in the grouted rock, and the computed factor of safety exceeded 1.5.

The grouting is non safety related. However, diminishing the porosity and reducing the permeability will have the beneficial effect of impeding flow through the uppermost Avon Park Formation and, therefore, minimize the potential for the initiation and/or growth of solution activity.

Although this will be an added benefit, the increase in compressive and shear strength of the Avon Park Formation has not been considered in other analyses. Bearing capacity, settlement, and site response were assessed on the basis of properties of the Avon Park Formation as measured during the site characterization program without grouting. The success of the Grout Program will be determined by the lack of groundwater intrusion during the excavation dewatering and not the increase in density, stiffness, or strength of the Avon Park Formation.

As a design input for the determination of the grouted zone, the groundwater is conservatively considered to be at the existing ground surface (between elevation 42 ft. and elevation 43 ft. NAVD88).

As part of the construction dewatering effort, a zone beneath each proposed nuclear island will be grouted in order to achieve the following three goals:

1) Form a “bottom of the bathtub” to prevent the flow of groundwater up through the bottom of the excavation.

2) Protect the excavation base from heaving.

3) Inhibit the flow of water through porous zones in this zone beneath each nuclear island, thereby reducing the future potential for solution activity.

The top elevation of the grouted zone (elevation -24 ft. NAVD88) was based on the top of rock and defines the elevation which the RCC Bridging Mat will be founded on. The proposed thickness of this grouted zone (75 ft., to elevation -99 ft. NAVD88) was determined based on the review of site data and discussions with site geologists. For example, shear wave velocity measurements from Borings A7, I2, AD1, A8, and I3, indicate a shelf within the Avon Park Formation at approximate elevation -97 ft. NAVD88 under the North Reactor LNP 2, where shear wave velocity increases from approximately 3500 ft/sec to approximately 5000 ft/sec. Boring Logs from Borings A7, A8, A9, and A10 indicate that the Avon Park Formation, in general, becomes less weathered, has a higher recovery, and higher RQD below elevation -97 NAVD88.

A similar shelf exists under the South Reactor LNP 1 at approximately -180 feet. However, Boring Logs from Borings A14, A17, A19, and A20 indicate that the Avon Park limestone, in general, has a higher recovery and higher RQD below elevation -97 ft. Additionally, geophysical logs from A-19 and A-20 indicate a higher shear wave velocity below elevation -97 ft. NAVD88. Based on the above information, elevation -99 ft. NAVD88 has been designated as the bottom of the grouted zone resulting in a relatively large, 75-ft.-thick zone. As discussed in FSAR Subsection 2.5.4.1.2.1.1, this shelf extends at least 50 ft. in depth and is characterized as a lower-porosity zone.

Grouting 75 ft. of the Avon Park Formation beneath the RCC Bridging Mat will accomplish goals one (1), two (2), and three (3) listed above. As previously noted, no credit was taken for this grout increasing the strength or stiffness of the grouted zone.

The grout will be bounded horizontally by the diaphragm wall between the bottom of the RCC Bridging Mat (elevation -24 ft. NAVD88) and bottom of the diaphragm wall (elevation -54 ft. NAVD88). From this elevation to the bottom of the grouted zone (elevation -99 ft. NAVD88), the grouted zone will be bounded by a grout curtain.

The Grout Program will be accomplished in two phases. Prior to the excavation of the nuclear island foundations, grout holes will be drilled from the existing ground surface to the proposed bottom of the target grouted zone (approximately 150 ft bgs). The first phase will consist of drilling and grouting on 8-ft. center-to-center spacing with a relatively low mobility grout (LMG). This LMG helps to form a perimeter to contain the second phase of grouting. The LMG grouting includes the installation of the grout curtain below the diaphragm wall. The purpose of the grout curtain is to “extend” the diaphragm wall and form a border around the grouted zone. A high mobility grout (HMG) will be drilled and grouted on split-spacing between the LMG holes. The HMG will fill in the area defined by the LMG. This is considered the second phase of the Grout Program.

State-of-the-practice computerized monitoring of all grouting will take place, including the measurement of grout take in terms of pressure and volume. A field test will be conducted prior to construction of this grouted zone to establish appropriate mixes for both the LMG and HMG and to confirm that the grout hole spacing is adequate. The 8-ft. grout hole spacing is currently based on experience in the industry. It is noted as a good starting point to be refined with a field test prior to and during construction.

2.5.4.5.1.2.1 Permeation Grouting Operation

The grouting operation will be conducted from, at or near, the existing ground surface by drilling boreholes from the surface down to the approximate elevation of -30.2 m (-99 ft.) NAVD88, and setting casing (either perforated or “tube-a-manchette” – a rubber sleeve between two packers). While uncased holes would be preferred, the existing site characterization data suggest that the holes may cave before they can be grouted; therefore, casing will be specified. The top elevation of the grouted zone will be at elevation -7.3 m (-24 ft.) NAVD88, resulting in a 22.9 m (75 ft.) thick grouted zone.

Grouting will generally be performed by the upstage method with pneumatic packers and a combination of lower mobility grout (LMG) and high mobility grout (HMG) to be established with a Grout Test Program prior to the commencement of the grouting program, as discussed in FSAR Subsection 2.5.4.5.1.2.2. Grout holes are initially spaced to achieve “no take” conditions. Hole spacing, grouting pressures, and acceptable grout takes will be established with the Grout Test Program. Grouting is non safety-related, however it will be performed under a quality program.

A grout intensity number (GIN) curve and target permeability (in Lugeons) will be used to dictate target grout pressures/volumes. The grout holes will be installed using an automated real-time monitoring system for water pressure testing and grouting, capable of computing a suite of engineering data allowing side-by-side evaluation of geology, grout mixes, Lugeon values and apparent Lugeon values, and plotting data into reports and CADD drawings.

2.5.4.5.1.2.2 Permeation Grout Testing Program

A Grout Test Program has been performed and the results have been finalized. Though grouting is not safety-related, mix design, material control, laboratory testing, grout placement, and field testing were conducted to meet NQA-1 quality requirements.

Mix designs were established for the various grout types, indicating the proportions of material constituents, as well as the target design parameters. All grout mixes were a combination of water, cement, flyash, bentonite, and superplasticizer. Mortar grout mixes or “low mobility sand grout mixes” were not used.

A Grout Test was implemented to specifications consistent with the design parameters set forth in this FSAR. The Grout Test Program consisted of nineteen grout holes arranged in a hexagonal pattern, including seven “Primary” grout holes of a higher viscosity grout, and twelve “Secondary” grout holes of a lower viscosity grout. These 19 holes were upstage and downstage grouted from a depth of 141 ft. bgs to a depth of 66 ft. bgs, as prescribed for the large-scale foundation grouting effort.

The purpose of the Grout Test Program was to validate the grout design and grouting techniques, to measure the change in the shear wave velocity and permeability of the grouted zone, and to determine the grout take in the Avon Park Formation.

The grout holes were installed using an automated real-time monitoring system for the water pressure testing and grouting, capable of computing a suite of engineering data allowing side-by-side evaluation of geology, grout mixes, Lugeon values and apparent Lugeon values, and plotting data into reports and CADD drawings.

Six initial and final verification core holes were drilled and water tested to verify pre- and post-test conditions. Prior to the commencement of and upon completion of the Grout Test Program, P-S suspension logging was performed to determine the effect of the grouting on the stiffness of the grouted mass. Because no appreciable change in shear wave velocity was observed post-grouting, the increased stiffness of the grouted zone will still be bounded by the randomization used in the site response analysis, as discussed in FSAR Subsection 2.5.2.5.1.

2.5.4.5.2 Excavation Extents

After the installation of the diaphragm walls and grouting operation described in FSAR Subsection 2.5.4.5.1, LNP 1 and LNP 2 will be vertically excavated to the approximate location of the Avon Park Formation at elevation -7.3 m (-24 ft.) NAVD88. The diaphragm walls serve as the excavation limits for the nuclear island.

Figures 2.5.4.5-201A and 2.5.4.5-201B show the planned nuclear island excavation limits at LNP 1 in plan view, and along southwest to northeast ("Plant South" to "Plant North") cross sections, respectively. Figures 2.5.4.5-202A and 2.5.4.5-202B indicate this same information for LNP 2.

Since the top of the Avon Park Formation is an erosional surface, its elevation is expected to be undulatory. As discussed in FSAR Subsection 2.5.4.5.3, elevation -7.3 m (-24 ft.) NAVD88 has been selected as a target elevation for nuclear island subgrade improvement.

Seismic Category II and nonsafety-related structures adjacent to the nuclear island will be supported on drilled shaft foundations. Considering the soil conditions at the site and the anticipated structural loads, shallow foundations will not provide adequate bearing capacity within permissible settlement and differential settlement requirements, and soil improvement techniques are not recommended due to the high water table and wetland conditions at the site. The conceptual design of the drilled shafts and installation is summarized in FSAR Subsection 3.8.5.9. Foundation design concepts under Seismic Category II and nonsafety-related structures adjacent to the nuclear island are shown on Figures 2.5.4.5-201A, 2.5.4.5-201B, 2.5.4.5-202A, 2.5.4.5-202B, 3.7-209, and 3.7-226.

2.5.4.5.3 Excavation Methods and Subgrade Improvement

As previously discussed in FSAR Subsection 2.5.1, the Suwannee and Ocala limestone formations are absent from the site, creating a geologic unconformity between the Avon Park Formation and the overlying undifferentiated Quaternary and Tertiary sediments. The Suwannee and the Ocala formations were eroded away, creating an erosion surface at the top of the older Avon Park Formation. This erosion surface is undulatory and the zone of the unconformity is of variable thickness. Careful reviews of the FSAR geotechnical and geophysical investigations were conducted, with consideration given to RQD, core recovery, SPT blow counts, shear-wave velocity, and overall condition of the core samples. A geologic and engineering interpretation was made that subsurface materials below elevation -7.3 m (-24 ft.) NAVD88 exhibit more desirable properties for foundation suitability than the materials above this elevation.

At both LNP 1 and LNP 2, rock at the nuclear island subgrade elevation -7.3 m (-24 ft.) NAVD88 will need to satisfy the following criteria:

- Rock will be moderately to highly cemented (naturally).
- Subgrade will not have solution features, loose rock, or open or soil-filled joints or fractures.

Foundation rock at elevation -7.3 m (-24 ft.) NAVD88 that does not satisfy these criteria will be removed and replaced or improved. A detailed excavation, subgrade improvement, and verification program will be developed prior to construction. The program will include the following general items:

- Specification of excavation methods. It is anticipated that excavation methods will include mass excavation of soils and highly weathered rock, and ripping of moderately weathered rock.

- Quality control and quality assurance programs.

- Methods for dewatering and protection of the subgrade from degradation during excavation and dewatering. Anticipated construction dewatering requirements are discussed in FSAR Subsection 2.5.4.6.2. It is not expected that the sound rock at the subgrade elevations will significantly degrade due to excavation, dewatering, or exposure to the elements during construction. Any degraded rock at subgrade elevation will be removed or improved prior to placement of dental concrete or the mudmat.

- Specification of methods for construction dewatering, disposal of water, and management of seepage and piping.

- Complete geologic mapping of the excavation, where exposed, will be undertaken prior to and during subgrade improvement activities. This mapping will occur in stages as the excavation is advanced. Due to the presence of the diaphragm wall and the inherent instability of the undifferentiated Quaternary sediments (if vertically excavated), the excavation will occur in 3 m (10 ft.) vertical increments at a 2 to 1 (horizontal to vertical) slope, inside the boundaries of the each diaphragm wall. The face of the slope created will be mapped, and then the slope will be removed, exposing the diaphragm wall. This process will be repeated in 3 m (10 ft.) increments down to the final excavation level. Complete geologic mapping of the excavation bottom will occur at the appropriate time.

- Excessively fractured or weathered rock will be over-excavated to the bottom of the weathered or fractured zone and filled with dental concrete.

- Soil-filled joints or fractures will be washed free of soil infilling to at least 1.5 m (5 ft.) below subgrade and filled with dental concrete.

- The inspection and mapping of the completed excavations will be performed by appropriately qualified and trained project inspection personnel. Soundings, test holes, and similar measures will be used to augment visual identification of areas needing repairs and to document acceptance of corrective measures, as appropriate.

Milestones for the excavation, subgrade improvement, and verification program are not identified at this time, but will be developed in conjunction with detailed design and construction planning. Additional description of foundation design is provided in FSAR Subsection 2.5.4.12.

2.5.4.5.4 Properties of Backfill Beneath and Adjacent to Nuclear Island

Based on a design grade elevation of 15.5 m (51 ft.) NAVD88, the elevation of each nuclear island basemat will be 3.4 m (11 ft.) NAVD88. A 15.2 cm (6 in.) mudmat will be located beneath each nuclear island basemat at elevation 3.4 m (11 ft.) NAVD88. Structural fill between the excavation bottom (elevation -7.3 m [-24 ft.] NAVD88) and the nuclear island mudmat (elevation 3.4 m [11 ft.] NAVD88) will consist of an RCC bridging mat, as shown on Figures 2.5.4.5-201B and 2.5.4.5 202B. A waterproofing membrane will be located between the RCC and the mudmat, meeting AP1000 DCD requirements of 0.55 static coefficient of friction between horizontal membrane and concrete. For buildings adjacent to the nuclear islands, the design grade will be raised to elevation 15.5 m (51 ft.) NAVD88 using engineered fill.

The following is the Design Description of the RCC. This RCC fill will serve two purposes: 1) replace the weakly cemented, undifferentiated Tertiary sediments that are present above elevation -7.3 m (-24 ft.) NAVD88, thereby, creating a uniform subsurface with increased bearing capacity; and 2) bridge conservatively postulated karst features.

The RCC bridging mat has been designed to bridge a 3-m (10-ft.) air-filled cavity located immediately beneath the RCC (elevation -7.3 m [-24 ft.] NAVD88) at any plan location for loading conditions identified in DCD Tier 1 Table 5.0-1 and Tier 2 Table 2-1. The 1-year specified compressive strength (f'_c) of the RCC is 2500 psi. The design of the RCC bridging mat has considered a nominal tensile strength of 250 psi.

A theoretical rock profile for the North and South Plant Units was developed using LNP site-specific rock properties and layering information. A SAP2000 Finite Element Model (FEM – linearly elastic) of the RCC, nuclear island basemat, and the subsurface rock was created using the design geometry, the rock profile beneath the RCC Bridging Mat, and the total loads applied by the nuclear island.

Also included in the FEM was the presence of theoretical cavities of different sizes and configurations. Three different cases, with cavities located at different depths, were considered:

- Case A: Cavities were located immediately below the grouted limestone, at elevation -99 ft. NAVD88 (75 ft. under the RCC).
- Case B: Cavities were located immediately below the RCC, at elevation -24 ft. NAVD88.
- Case C: Cavities were located at the top of rock layer NAV-3, which is the layer with lower Elastic Modulus for the North Reactor profile, below elevation -149 ft. NAVD88 (125 ft. under the RCC). This case was analyzed only in the North Reactor, where the lower Elastic Modulus layer is somewhat thicker than in the South Reactor profile.

Examples of the locations of these cavities are shown on Figure 2.5.4.5-204.

Sample stress plots and result tables were generated for the maximum stresses derived from the different cases of this analysis.

RCC bridging mat will be constructed using unreinforced RCC. Neither the AP1000 DCD nor ACI 349-01 addresses requirements for unreinforced (plain) concrete. ACI 349-01 specifies load factors and strength reduction factors for nuclear safety-related concrete structures. ACI 318-99 (Chapter 22) provides design methodology for unreinforced (plain) concrete. Thus, for the RCC bridging mat design, load factors, and strength reduction factors from ACI 349-01 and methodology from ACI 318-99 Chapter 22 are used for compressive and tensile capacity. For shear stress across lift joints, the strength is represented by a Mohr envelop relationship as described in USACE EM 1110-2-2006. A safety factor of 2.17 was then applied to ensure adequate performance. The 2.17 factor of safety incorporates both the load factor and the strength reduction factor for plain concrete.

The Pre-COL RCC testing performed and the Post-COL RCC Testing planned is described in FSAR Subsection 3.8.5.11. The RCC testing is to verify that the specified 2500 psi RCC compressive strength, ACI 318-99 (Chapter 22) specified tensile strength, and USACE EM 1110-2-2006 specified shear strengths across lift joints can be achieved.

The nuclear island vertical load considered in the analysis is 287,000 kips. The total vertical load of 287,000 kips corresponds to an average uniform load of 8.93 ksf, which exceeds the DCD Tier 1 requirements for bearing capacity. For the RCC bridging mat analysis, 70 percent of the total vertical load was considered dead load, and 30 percent was considered live load.

In the 3-D FEM, the shear forces were fully transmitted between the basemat and the RCC and between the RCC and the subsurface rock.

In the 3-D FEM, the subsurface material (limestone) that was included in the model below the RCC was sufficiently extended in both lateral direction and depth so that at the borders of the model, the stresses and deformations, due to the external loads applied to the NI basemat, are relatively small.

Any additional strength provided by grouting the upper 75 ft. of limestone was conservatively not included in this analysis. The rock mass properties (ungrouted) for that layer were used.

The LNP 2 profile presented lower values of rock mass elastic modulus; therefore, in most cases, the resulting tensile stresses were higher in LNP 2 than in LNP 1.

Controlled low strength material fill will be placed adjacent to the sidewalls of the nuclear islands to an elevation at least 1.5 m (5 ft.) below the bottom of the adjacent buildings' foundation mat. Engineered fill will be placed from the top of the controlled low strength material fill to the bottom of the foundation mats of the adjacent Turbine Building, Annex Building, and the Radwaste Building. Figure 3.7-226 shows the approximate planned limits of controlled low strength material fill adjacent to nuclear island structures at LNP 1 and LNP 2.

Table 2.5.4.5-201 is a summary of the anticipated engineering properties for each backfill type. The characteristics and use of the materials described in Table 2.5.4.5-201 are as follows:

- RCC fill. This will consist of a roller compacted concrete bridging mat to be used to replace undifferentiated Tertiary sediments and to bridge conservatively postulated karst features.
- Controlled low strength material (CLSM) fill will be placed adjacent to the sidewalls of the nuclear islands to an elevation at least 1.5 m (5 ft.) below the bottom of the adjacent buildings' foundation mat as shown in Figure 3.7-226.
- Engineered fill will be used under the footprint of the TB, AB, and RB and to a lateral extent of ~30 ft. beyond the building footprint as discussed in FSAR Subsection 3.7.1.1.1 and shown in Figure 3.7-208. Engineered fill will also be placed from the top of the controlled low strength material fill to the bottom of the foundation mats of the adjacent Turbine Building, Annex Building, and the Radwaste Building as shown in Figure 3.7-226.

The engineering properties listed in Table 2.5.4.5-201 will be included in the construction specifications. Engineered fill material sources, once identified, will be mix-designed and tested to demonstrate that they are consistent with the properties in Table 2.5.4.5-201. The development of the CLSM fill specification and associated testing will occur prior to construction.

LNP COL 2.5-7

Table 2.5.4.5-201
Engineering Properties of Structural Fill and Backfill

	As-Placed Engineering Properties[a]	
Backfill Type	**Strength Parameters**	**V_s (ft/sec)**
Roller Compacted Concrete Bridging Mat	1-Year Compressive Strength: ≥2500 psi	>3500 ft/sec
Controlled Low Strength Material Backfill	Not Applicable	1000[b] ft/sec
Engineered fill[c]	Drained friction angle of 34 degrees (or equivalent shear strength); SM-SC USCS Classification	850[d] ft/sec

Notes:

a) These engineering properties are considered representative values of the backfill type.
b) Value is typical for controlled low strength material fill, conservatively based on engineering judgment.
c) Engineered fill will be compacted to 95 percent of its maximum dry density as determined by ASTM D 1557, Modified Proctor method, with a dry unit weight of 110 pcf. The moisture content of the fill will be controlled to within +/- 2 percent of its optimum moisture.
d) Expected range of the average shear wave velocity in the engineered fill is 500 ft/sec to 1000 ft/sec.

V_s = Shear Wave Velocity
psi = pound per square inch
ft/sec = foot per second

2.5.4.6 Groundwater Conditions

LNP COL 2.5-6
LNP COL 2.5-8

Groundwater conditions for the LNP site were established by periodic measurements of groundwater levels since well installation in 2007, as summarized in FSAR Subsection 2.4.12. Data from these measurements provide a basis for engineering design and for preliminary construction dewatering plans.

2.5.4.6.1 Groundwater Elevations

Groundwater elevations at site monitoring wells are presented in FSAR Subsection 2.4.12. Water-table data collected in 2007 indicates that the water table ranges in depth at LNP 1 and LNP 2 areas from less than 0.3 m (1 ft.) bgs during rainy periods to approximately 1.5 m (5 ft.) bgs during drier periods.

As described in FSAR Subsection 2.4.12.5, post-construction groundwater elevations are not anticipated to exceed elevation +14.6 m (+48 ft.) NAVD88. The nuclear islands will be founded on RCC over rock; groundwater conditions are not expected to adversely affect foundation performance.

2.5.4.6.2 Construction Dewatering

Dewatering will be required to maintain groundwater levels beneath the nuclear island to an elevation of -7.3 m (-24.0 ft.) NAVD88 or lower during excavation and construction of the nuclear islands. Expected construction dewatering flow rates and anticipated dewatering methods are summarized in this subsection.

Due to the size of the excavation, as well as the expected quantity of groundwater that could potentially be encountered, a diaphragm wall will be constructed around each entire nuclear island to minimize lateral groundwater inflow into the excavation. In addition, the Avon Park Formation will be drilled and pressure grouted to elevation -30.2 m (-99 ft.) NAVD88 before excavation begins to minimize seepage from the rock upward into the excavation, and to resist possible uplift pressure. The diaphragm wall will be keyed approximately 9.1 m (30 ft.) into the grouted bedrock. Thus, two different engineered barriers will form a “bathtub” with the diaphragm wall being the sides and the grouted Avon Park Formation being the bottom of the “bathtub.” With this design, one has to dewater the excavation area with relatively shallow wells and sumps within the area.

Expected construction dewatering pumping rates were calculated using the Visual MODFLOW software package, which includes the U.S. Geological Survey’s three-dimensional finite-difference modeling code, MODFLOW 2000. A three-dimensional model was constructed that has six horizontal layers, each of uniform thickness. The top layer is 20.4 m (67 ft.) thick and represents the undifferentiated Quaternary and Tertiary sediments (fine sand and silty sand). The second and third layers represent the uppermost 22.9 m (75 ft.) of Avon Park Formation, which is grouted beneath the nuclear island. The lowermost four layers of the model represent the middle portion of the Avon Park Formation, which is ungrouted. Together, Model Layers 2 through 6 represent the permeable portion of the Avon Park Formation, which in this area is also referred to as the Upper Floridan aquifer. The base of the model was set at -137 m (-450 ft.) NAVD88.

The diaphragm wall penetrates through the undifferentiated surficial sand (Model Layer 1) and the uppermost 9.1 m (30 ft.) of Avon Park Formation (Layer 2 in the model). Figure 2.5.4.6-201 shows a cross section of the model and displays how the diaphragm wall and the grouted Avon Park Formation are represented in the model.

The entire model area (one unit) is 1174 m by 1174 m (3850 ft. by 3850 ft.) in area. The excavation area and the diaphragm wall occupy an area of 82.3 m (270 ft.) long by 51.8 m (170 ft.) wide for each unit and are located in the center of the model area. Only one unit is modeled because the units will be dewatered at different times.

Table 2.5.4.6-201 lists the hydraulic characteristics of each geological material represented in the model. The hydraulic conductivity values used in the model are presented in FSAR Subsection 2.4.12. Average values were obtained from hydraulic conductivity field tests (FSAR Subsection 2.4.12).

The model accounts for the results of a sensitivity analysis conducted to determine the susceptibility for increased flow to the shallow interior wells based on postulated leakage “windows” that develop in the diaphragm wall. While state-of-the-art diaphragm wall construction has reduced vulnerability to such defects (such as panels rotating, assuming a degree of curvature, or otherwise not aligning adequately), sump pumps located at the bottom of the excavation will be pumped to address any potential “window leakage,” as well as rainfall and surface runoff during the excavation process.

The gross permeability of the diaphragm wall is taken as 10^{-6} cm/sec (0.002835 ft/day). Potential leakage through “windows,” as mentioned above, may necessitate greater than expected pumping rates in order to maintain dry working conditions within the excavation. The permeability of the grouted Avon Park Formation has been conservatively considered to be 10^{-4} cm/sec (0.2835 ft/day), but this parameter has been varied to account for the possible variation in the effectiveness of the grouting operation.

The hydraulic conductivity of the ungrouted Avon Park Formation at the LNP site ranges from 8.47×10^{-4} to 1.92×10^{-2} cm/sec (2.4 to 54.4 ft/day), and averages approximately 4.9×10^{-3} cm/sec (13.9 ft/day).

The total flow that must be accommodated with sumps and shallow wells is conservatively determined to be in the range of 1136 to 1893 lpm (300 to 500 gpm) at steady-state conditions during construction, based on the site hydraulic conductivity characteristics summarized in Table 2.5.4.6-201 and the hydrogeological conditions at the site, as described in FSAR Subsection 2.4.12.

The groundwater pumping rate during excavation can be managed by six submersible pumps (each with 379 lpm [100 gpm] capacity) installed in wells located around the inside perimeter of the diaphragm wall and the grouted zone with pumps placed in sumps within the excavation.

When the excavation has reached its target depth, the exposed rock and the diaphragm wall will be inspected and evaluated for leakage. In the event that significant leakage is observed (e.g., greater than 379 lpm [100 gpm]), a second round of drilling and pressure grouting at specific locations will be implemented to seal areas where groundwater is seeping through the engineered barriers.

During construction, a groundwater monitoring program will be implemented to monitor the head differential between the inside and the outside of the diaphragm wall, as well as the uplift pressure on the bottom of the excavation, as described in FSAR Subsection 2.5.4.10.3.5.

LNP COL 2.5-8

Table 2.5.4.6-201
Hydraulic Conductivities and Calculated Dewatering Rates

Model Simulation Run No.	Hydraulic Conductivity [a]										Dewatering Rate [b]		Hydraulic Head in Grouted Limestone
	Layer 1 Sand		Layers 2 & 6 Limestone		Layers 3-5 Limestone		Grouted Limestone		Diaphragm Wall				
	ft/day	cm/s	ft/day	cm/s	ft/day	cm/s	ft/day	cm/s	ft/day	cm/s	gpm	ft^3/day	
1 (Base Run)	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835	1.0E-06	67	12,844	-9.27
2	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.02835	1.0E-05	147	28,369	-8.85
3	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.0002835	1.0E-07	56	10,818	-9.37
4	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	2.835	1.0E-03	0.002835	1.0E-06	452	87,046	-5.77
5	9.2	3.2E-03	13.9	4.9E-03	179.0	6.3E-02	0.2835	1.0E-04	0.002835	1.0E-06	67	12,920	-9.16
6	35.0	1.2E-02	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835	1.0E-06	68	13,023	-9.79
7	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835 [c]	1.0E-06 [c]	94	18,141	-9.28

Notes:

a) 1.0 cm/s = 2835 ft/day

b) All dewatering accomplished by wells and sumps installed within the flow barriers.

c) In Run No. 7, the diaphragm wall had a typical hydraulic conductivity of 1.0E-06 cm/s, except for three vertical windows in the wall where the diaphragm panels separate and a 3.5-ft. gap occurs. This simulation was run to determine how seepage rate would be affected by multiple defects in the wall. The cells representing each "gap" were assigned hydraulic conductivities of the surrounding sand (3.2E-03 cm/s).

ft/day = foot per day; cm/s = centimeter per second; gpm = gallon per minute; ft^3/day = cubic foot per day

2.5.4.7 Response of Soil and Rock to Dynamic Loading

LNP COL 2.5-2
LNP COL 2.5-6

This subsection presents a summary of information regarding the response of soil and rock to dynamic loading. Cross references to other subsections in this FSAR are provided herein.

Descriptions of investigations performed to identify surface faulting features in the LNP site and vicinity are presented in FSAR Subsection 2.5.3. As stated therein, there are no capable tectonic fault sources within the site area or vicinity. There is no evidence of Quaternary tectonic surface faulting or fold deformation within the LNP site location. The potential for nontectonic deformation at the site from phenomenon other than karst-related collapse or subsidence is negligible. The LNP site lies within a region susceptible to dissolution and karst development. The materials below the bottom of the nuclear island to an elevation of -30 m (-99 ft.) NAVD88 will be improved as described in FSAR Subsection 2.5.4.12.

Results of V_S and V_P surveys at the LNP site are presented in FSAR Subsection 2.5.4.4. Results of V_S from Suspension P-S velocity logging and downhole logging within boreholes at LNP 1 and LNP 2 are presented on Figures 2.5.4.2-204A, 2.5.4.2-204B, 2.5.4.2-205A, and 2.5.4.2-205B. Interpretations of these data relative to the site geologic conditions are presented in FSAR Subsection 2.5.4.4.2. These data were used to develop site-specific dynamic velocity profiles for site response analyses as presented in FSAR Subsection 2.5.2.5.

Dynamic triaxial shear tests and resonant column tests were not performed as part of the investigation because of the following:

- The basemats for the nuclear islands for LNP 1 and LNP 2 bear on RCC which in turn bears on rock. Considering the low seismic environment and the foundation configuration, no site specific soil structure interaction analysis for safety class structures is required and, therefore, no Modulus Degradation Curves or Damping Curves as typically measured by these types of tests were required.

- During the site investigation, it was extremely difficult to obtain quality undisturbed samples of the Quaternary and Tertiary sediments at the site and reconstituted samples from SPT samples would not be representative as the cementation effects would be lost. The uncertainty in the modulus reduction and damping relationship was incorporated in the site response analysis by modeling a range of behavior (relatively linear to relatively nonlinear) for the softer layers of weathered limestone/calcareous silts. The range in dynamic properties had only a small effect on the computed GMRS and an even smaller effect on the FIRS computed ground motion at the base of the excavation. Hence, it was judged that the EPRI curves would be suitable for site response analysis and nonsafety-related drilled shaft design. (Reference 2.5.2-260)

Structures adjacent to the nuclear island are founded on drilled shafts embedded in the rock. Both beneficial and adverse effects of soil will be considered in the design of drilled shafts to ensure no building interaction at the foundation level.

2.5.4.8 Liquefaction Potential

LNP COL 2.5-9 The potential for liquefaction of existing soils at LNP 1 and LNP 2 was evaluated by conducting liquefaction analyses using the following relationship stated in Regulatory Guide 1.198.

$$FS_{\text{against liquefaction}} = FS = CRR/CSR$$

where CRR (cyclic resistance ratio) is the available soil resistance and CSR (cyclic stress ratio) is the cyclic stress generated by the earthquake.

The CSR was determined using the empirical methods as cited in Regulatory Position 3.5. The SPT blow count method as cited in Regulatory Position 1.2 of Regulatory Guide 1.198 with corrections recommended in Youd et al (Reference 2.5.4.8-201) was used to determine the CRR.

The CSR was determined from Seismic Input Motions consistent with Regulatory Position 3.3.2 together with the empirical methods cited in Regulatory Position 3.5.

The following subsections identify the location of soils and groundwater at the LNP sites that were considered in the liquefaction evaluation, the procedures that were followed to assess liquefaction potential, and the results of the liquefaction evaluations.

2.5.4.8.1 Soil and Groundwater Conditions

Soil conditions at LNP 1 and LNP 2 generally consist of undifferentiated Quaternary and Tertiary sediments, which generally consist of sands, silts, and clays as described in FSAR Subsection 2.5.4.2.1.1.2. These sediments overly the Avon Park Formation. The density of the Quaternary and Tertiary granular soils ranges from relatively loose to very dense, based on SPT blow count measurements. Generally low SPT blow counts are recorded in the Quaternary Sands (e.g., N-values less than 10 blows per foot). Blow counts in the Tertiary sediments are generally above 20 blows per foot, except in isolated zones. These isolated zones are typically of limited thickness (e.g., less than 1.5 m [5 ft.]), and surrounding blow counts are usually greater than 20 blows per foot. High shear-wave velocity values plus very high blow counts at some elevations indicate that cementation exists in some of the Tertiary sediments at the site. Groundwater is typically located within 1 m (3 ft.) of the existing ground surface.

Construction of the LNP facilities will result in the following soil cases relative to liquefaction analysis:

- Soil beneath the foundation for the nuclear islands will be excavated and replaced with RCC as discussed in FSAR Subsection 2.5.4.3. Therefore, all SPT data from borings drilled within the nuclear island footprints were excluded from the liquefaction analysis. It is noted, however, that SPT data from borings drilled along the perimeter or just outside the nuclear islands were not excluded from the liquefaction analysis.

- Soil beyond the nuclear island perimeter, which will be left in place, was subject to liquefaction analysis except for soil within approximately 2.1 m (7 ft.) of existing grade which will be removed or improved to prevent liquefaction.

- Soil beyond the nuclear island perimeter that will be excavated as part of the overall plant construction (e.g., the Turbine Building Condenser Pit) was excluded from the analysis.

- Seismic Category II and nonsafety-related structures adjacent to the nuclear island will be supported on drilled shafts socketed into rock. Soil left in place that surrounds the shafts was addressed in the liquefaction analysis.

2.5.4.8.2 Liquefaction Analysis Procedure

As stated above, liquefaction analysis was conducted in accordance with the Regulatory Positions stated in Regulatory Guide 1.198 with SPT blow counts corrected as recommended in Youd et al (Reference 2.5.4.8-201).

The determination of CSR and CRR involved the following steps:

- CSR was determined from the seismic ground motions estimated for the site in terms of acceleration versus time. (Regulatory Position 3.3.2).

- CRR is estimated as a function of soil characteristics and field stress conditions. The soil characteristics include fine contents, SPT blow counts, soil type, and overburden pressure. The field stress conditions are determined by the groundwater locations and soil density. Various methods of evaluating CRR are available, including the SPT, the cone penetrometer test, the Becker penetration test, and shear-wave velocity procedures. The most common method involves the use of the SPT blow count. The blow counts used in the liquefaction analysis are adjusted for drilling and sampling equipment and method to obtain corrected N-values. The adjustments include borehole diameter, hammer transfer energy, sample liner characteristics, and length of rods.

Cohesive soils, such as fat clay (CH), lean clay (CL), and elastic silt (MH) are not considered to be liquefiable, following the guidance provided in Youd et al (Reference 2.5.4.8-201) and Regulatory Guide 1.198.

Cohesionless soils with low factors of safety against liquefaction (FS ≤ 1.1) are considered to be liquefiable under the design earthquake. Soils with intermediate factors of safety (FS ≈ 1.1 to 1.4) are considered to be non-liquefiable, but increased dynamic pore pressures should be taken into account. Soils with high factors of safety (FS ≥ 1.4) are generally not considered to be liquefiable under the design earthquake, but under certain circumstances would suffer relatively minor cyclic pore pressure generation that could result in some reduction in shear strength.

2.5.4.8.3 Cyclic Resistance of Soils

The Youd et al analysis procedure uses empirical relationships that correlate CRR of soils to the corrected SPT blow counts to evaluate liquefaction potential (Reference 2.5.4.8-201). The corrected SPT blow counts, or $(N_1)_{60}$, at the LNP sites, were obtained by applying correction factors to the field measured N-value, N_{field} as shown in Equation 2.5.4.8-201:

$$(N_1)_{60} = N_{field} * C_N * C_E * C_B * C_R * C_S \qquad \text{Equation 2.5.4.8-201}$$

Where C_N, C_E, C_B, C_R, and C_S are correction factors for overburden pressure, hammer transfer energy, borehole diameter, rod length, and sampler type (with and without liner). Additional correction factors were made for confining pressure (K_σ) and for earthquake magnitude. The ground surface at both of the LNP sites is relatively flat and therefore no adjustments were made for ground surface slope (K_α). The background for these correction factors is discussed in detail in Youd et al. (Reference 2.5.4.8-201)

A fines content correction was also applied to define a $(N_1)_{60\text{-}CS}$ value for use in the liquefaction evaluation. The fines content correction was based on the methods discussed in Youd et al (Reference 2.5.4.8-201) where grain-size information was available. In cases where grain-size information was not available, the fines content was based on visual descriptions and on lower-bound estimates from field logs.

2.5.4.8.4 Earthquake Induced Cyclic Stress

Earthquake-induced cyclic stresses within soils considered for liquefaction analysis were estimated based on the seismic ground motions, specifically horizontal ground accelerations versus time as identified on Table 2.5.4.8-201. These ground motions are based on the SHAKE analyses used to develop the GMRS, including the soil profile randomization procedure. The ground motions were scaled up to 0.10 g.

2.5.4.8.5 Results of Liquefaction Analysis

Soil characteristics obtained at various depths in applicable A-series and B-series boreholes were used to evaluate the liquefaction potential at the LNP sites. The analyses involved estimating CSR and CRR for cohesionless soil layers and then determining the FS from the following equation:

$$FS = (CRR_{7.5}/CSR) * MSF * K_{\sigma} * K_{\alpha} \qquad \text{Equation 2.5.4.8-202}$$

In Equation 2.5.4.8-202, $CRR_{7.5}$ is the empirical correlation between corrected blow count and CRR from the Youd et al paper (Reference 2.5.4.8-201), CSR is determined as described above, and MSF is the magnitude scaling factor. The MSF was determined using the MSF equation in Youd et al. This equation uses the moment magnitude for the site. A moment magnitude of 7.1 was used in the analysis based on the deaggregation results of the PSHA reported in Tables 2.5.2-221 and 2.5.2-225.

For borings where the liquefaction analysis shows potential for liquefaction, the borehole identification, bottom depth of the SPT sample, soil type, and the field SPT N-Value used in the liquefaction analysis are summarized in revised Tables 2.5.4.8-202A and 2.5.4.8-202B. The revised Tables 2.5.4.8-202A and 2.5.4.8-202B also present the results of the liquefaction analysis including the factors of safety against liquefaction and the depth of the postulated liquefiable zone. Figures 2.5.4.8-201A and 2.5.4.8-201B show, in plan and elevation respectively, the location of the liquefaction zones identified in revised Table 2.5.4.8-202A for LNP 1. Figure 2.5.4.8-202A and Figure 2.5.4.8-202B show, in plan and elevation view respectively, the liquefaction zones identified in revised Table 2.5.4.8-202B for LNP 2. In these figures, the liquefaction zones with a factor of safety of less than or equal to 1.1 are shown by circles with yellow infill. For LNP 1, liquefiable zones were postulated in boreholes O-2, A-15, A-18/O-4, and B-28. Boreholes O-2, A-15 and A-18/O-4 are in the nuclear island excavation zone. Borehole B-28 is under the Annex Building. For LNP 2, liquefiable zones were postulated for boreholes B-01, B-07, B-07A, B-31, and B-33. Borehole B-01 with liquefiable zones is well away from the AP1000 footprint. Boreholes B-07, B-07A, B-31, and B-33 are under the Turbine Building. Based on these figures, it was concluded that liquefiable zones under the LNP 1 and 2 footprints are confined to the northwest corner of the Unit 2 Turbine Building and in isolated random pockets under the remaining LNP 1 and 2 footprints.

Soil beneath the nuclear island foundation will be removed and replaced with Roller Compacted Concrete (RCC). Thus, the bearing stability of the nuclear island foundation is not affected by the postulated liquefaction. The random isolated pockets of liquefiable soils also do not affect the nuclear island sliding and overturning stability based on Westinghouse analysis. The Westinghouse analysis concludes that the nuclear island is stable against sliding, and there is no quality requirement for backfill adjacent to the nuclear island to maintain stability against sliding. The Westinghouse analysis also concludes that there is no passive pressure required to maintain stability against overturning.

For the area under the Annex, Turbine, and Radwaste building footprint, in-situ soil will be replaced or improved to a depth of approximately 2.1 m (7 ft.) below existing grade (elevation 12.8 m [42 ft.] NAVD88). The plant design grade will be established at elevation 15.5 m (51 ft.) NAVD88 by placing engineered fill above the improved / replaced in-situ material. In addition, the earthwork design incorporates vertical and horizontal drains to prevent buildup of excess pore pressures that cause liquefaction as shown in Figures 2.5.4.8-205 and 2.5.4.8-206 for LNP 1 and 2 respectively.

2.5.4.8.6 Median Centered Liquefaction Evaluations for 10^{-5} UHRS

As a sensitivity analysis, the median centered liquefaction potential (factor of safety <1.0) for 10^{-5} UHRS was evaluated. The methodology and design parameters used for 10^{-5} UHRS liquefaction analysis were the same as that used for design basis liquefaction analysis described in FSAR Subsection 2.5.4.8 except liquefaction was postulated when the computed factor of safety was <1.0 and the soil cyclic shear stress were computed for the 10^{-5} UHRS ground motions and the median shear wave velocity soil profile derived from the randomized soil profiles used to compute the 10^{-5} UHRS. In addition, the equivalent number of stress cycles was computed for the weighted average moment magnitude of 5.74 for the site. Tables 2.5.4.8-203A and 2.5.4.8-203B present liquefaction analysis results for 10^{-5} UHRS for LNP 1 and 2 respectively. The results include the computed factors of safety against liquefaction and the depth below the Annex, Radwaste, or Turbine Building foundation mat where liquefaction is postulated. Figures 2.5.4.8-207 and 2.5.4.8-208 show, in plan and elevation respectively, the location of the liquefaction zones identified in Table 2.5.4.8-203A for LNP 1. Figure 2.5.4.8-209 and Figure 2.5.4.8-210 show, in plan and elevation view respectively, the liquefaction zones identified in Table 2.5.4.8-203B for LNP 2. In these figures, the liquefaction zones with a factor of safety of less than or equal to 1.0 are shown by circles with yellow infill. For Unit 1, liquefiable zones were postulated in boreholes O-2, A-15, A-18/O-4, A-13, and B-28. Boreholes O-2, A-15 and A-18/O-4 are in the nuclear island excavation zone. Borehole A-13 (factor of safety = 1.0) is under the Radwaste Building, and B-28 is under the Annex Building. For Unit 2, liquefiable zones were postulated for boreholes B-01, B-07, B-07A, B-31, and B-33. Borehole B-01 is well away from the AP1000 footprint. Boreholes B-07, B-07A, B-31, and B-33 are under the Turbine Building. Based on these figures, it can be concluded that liquefiable zones under the LNP 1 and 2 footprints are confined to the northwest corner of the LNP 2 Turbine Building and in isolated random pockets under the remaining LNP 1 and 2 footprints. These conclusions for median centered liquefaction potential for 10^{-5} UHRS are the same as the conclusions for the design basis liquefaction analysis described in FSAR Subsection 2.5.4.8.

LNP COL 2.5-9

Table 2.5.4.8-201
Summary of Peak Ground Acceleration Used for Liquefaction Analysis

Structure	Rock Peak Ground Acceleration (g)	Site Class	F_a	a_{max} (g)
North Reactor	0.07	C	1.2	0.1
South Reactor	0.07	C	1.2	0.1

Notes:

Site Class and F_a were estimated based on International Building Code (IBC) (2006).

a_{max} = Horizontal peak acceleration at ground surface. A minimum value of 0.1 g was used, per 10 CFR 50.
g = gravity acceleration

LNP COL 2.5-9

Table 2.5.4.8-202A
Summary of Soil Layers Susceptible to Liquefaction in LNP 1 Site

Borehole	Bottom Depth of SPT Sample (ft.)[(a)]	Soil Type	Field SPT N-Value (bpf)	Factor of Safety (FS)
A-15	16	SP	5	1.0
A-15	21	SP	1	0.8
A-15	26	SC	2	1.1
A-18	20	NR	0	0.7
B-28	36.5	ML	0	0.9
O-2	9	SP-SC	2	0.9
O-2	10.5	SP-SC	2	0.9
O-2	12.0	SP-SC	1	0.8
O-4	24.0	ML	0	0.9

Notes:

a) Depth of SPT sample is relative to original site grade at approximately El 41-43 ft. NAVD88

BPF = Blows per Foot
SC = Clayey Sand
SM = Silty Sand
SP = Poorly Graded Sand
NR = Not Recorded
ML = Silt with Sand

LNP COL 2.5-9

Table 2.5.4.8-202B (Sheet 1 of 2)
Summary of Soil Layers Susceptible to Liquefaction in LNP 2 Site

Borehole	Bottom Depth of SPT Sample (ft.)[(a)]	Soil Type	Field SPT N-Value (bpf)	Factor of Safety (FS)
B-01	26.5	SM	2	0.8
B-01	31.5	SM	2	0.8
B-07	31.5	SP-SM	3	1.0
B-07	36.5	SP-SM	2	0.8
B-07	51.5	SP-SM	2	0.8
B-07	56.5	SP-SM	2	0.8
B-07	61.5	SP-SM	3	0.9
B-07	76.5	SP-SM	3	1.0
B-07A	26.5	SP-SM	5	1.0
B-07A	31.5	SM	4	1.1
B-07A	36.5	SP-SM	3	0.8
B-07A	41.5	SM	3	0.8
B-07A	51.5	SM	2	1.1
B-07A	76.5	SP-SM	6	0.9
B-31	40.5	SP	4	1.0
B-31	69.0	SP	5	1.0
B-31	70.5	SP	6	1.1
B-31	73.5	SP	5	1.0
B-31	76.5	SP	2	0.7
B-31	78.0	SP	6	1.1
B-31	79.5	SP	4	0.9
B-31	81.0	SP	2	0.7
B-31	82.5	SP	3	0.8
B-31	84.0	SP	3	0.8
B-31	85.5	SP	3	0.8
B-31	87.0	SP	2	0.7
B-31	88.5	SP	1	0.7
B-31	90.0	SP	0	0.7
B-31	91.5	SP	4	0.9
B-31	93.0	SP	3	0.8
B-31	94.5	SP	7	1.1
B-31	96.0	SP	0	0.6
B-31	97.5	SP	0	0.6
B-31	99.0	SP	1	0.6
B-31	103.5	SP-SM	7	1.1
B-31	109.5	SP-SC	5	0.9
B-31	118.5	SP-SM	0	0.7
B-31	120.0	SP-SM	0	0.7
B-31	121.5	SP-SM	0	0.7
B-31	123.0	SP-SM	0	0.7
B-31	124.5	SP-SM	0	0.7

LNP COL 2.5-9

Table 2.5.4.8-202B (Sheet 2 of 2)
Summary of Soil Layers Susceptible to Liquefaction in LNP 2 Site

Borehole	Bottom Depth of SPT Sample (ft.) [(a)]	Soil Type	Field SPT N-Value (bpf)	Factor of Safety (FS)
B-31	126.0	SP-SM	0	0.7
B-31	127.5	SP-SM, ML	0	1.0
B-31	129.0	SP-SM	0	0.7
B-31	130.5	SP-SM	0	0.7
B-33	28.5	SP	4	1.0
B-33	30.0	SP	5	1.2
B-33	31.5	SP	3	0.9
B-33	33.0	SP	2	0.8
B-33	34.5	SP	2	0.8
B-33	36.0	SP	1	0.7
B-33	37.5	SP	2	0.8
B-33	39.0	SP	2	0.8
B-33	40.5	SP	2	0.8
B-33	42.0	SP	1	0.7
B-33	43.5	SP	0	0.7
B-33	45.0	SP	0	0.7
B-33	46.5	SP	0	0.7
B-33	58.5	SP	5	1.1
B-33	66.0	SP	7	1.1

Notes:

a) Depth of SPT sample is relative to original site grade at approximately El 41-43 ft. NAVD88

BPF = Blows per Foot
SC = Clayey Sand
SM = Silty Sand
SP = Poorly Graded Sand
NR = Not Recorded
ML = Silt with Sand

LNP COL 2.5-9

Table 2.5.4.8-203A
Summary of Soil Layers Susceptible to Liquefaction in LNP 1 Site For 10^{-5} UHRS

Borehole	Bottom Depth of SPT Sample (ft.) [a]	Soil Type [c], [d], [e] [f], [g]	Field SPT N-Value (BPF) [b]	Factor of Safety (FS)
A-15	16.0	SP	5	0.8
A-15	21.0	SP	1	0.7
A-15	26.0	SC	2	1.0
A-18	20.0	NR	0	0.5
B-28	36.5	ML	0	0.8
O-2	9.0	SP-SC	2	0.8
O-2	10.5	SP-SC	2	0.8
O-2	12.0	SP-SC	1	0.6
O-4	24.0	ML	0	0.8
A-13	16.5	SM	3	1.0

Notes:

a) Depth of SPT sample is relative to original site grade at approximately El 41-43 ft. NAVD88
b) BPF = Blows per Foot
c) SC = Clayey Sand
d) SM = Silty Sand
e) SP = Poorly Graded Sand
f) NR = Not Recorded
g) ML = Silt with Sand

LNP COL 2.5-9

Table 2.5.4.8-203B (Sheet 1 of 3)
Summary of Soil Layers Susceptible to Liquefaction in LNP 2 Site
For 10^{-5} UHRS

Borehole	Bottom Depth of SPT Sample (ft.) [a]	Soil Type, [c], [d], [e] [f], [g]	Field SPT N-Value (BPF) [b]	Factor of Safety (FS)
B-01	26.5	SM	2	0.7
B-01	31.5	SM	2	0.7
B-07	31.5	SP-SM	3	0.9
B-07	36.5	SP-SM	2	0.7
B-07	51.5	SP-SM	2	0.7
B-07	56.5	SP-SM	2	0.7
B-07	61.5	SP-SM	3	0.8
B-07	76.5	SP-SM	3	0.9
B-07A	26.5	SP-SM	5	0.9
B-07A	31.5	SM	4	1.0
B-07A	36.5	SP-SM	3	0.7
B-07A	41.5	SM	3	0.7
B-07A	51.5	SM	2	1.0
B-07A	76.5	SP-SM	6	0.8
B-31	40.5	SP	4	0.9
B-31	69.0	SP	5	0.9
B-31	70.5	SP	6	1.0
B-31	73.5	SP	5	0.9
B-31	76.5	SP	2	0.7
B-31	78.0	SP	6	1.0
B-31	79.5	SP	4	0.8

LNP COL 2.5-9

Table 2.5.4.8-203B (Sheet 2 of 3)
Summary of Soil Layers Susceptible to Liquefaction in LNP 2 Site For 10^{-5} UHRS

Borehole	Bottom Depth of SPT Sample (ft.) [(a)]	Soil Type, [(c), (d), (e) (f), (g)]	Field SPT N-Value (BPF) [(b)]	Factor of Safety (FS)
B-31	81.0	SP	2	0.7
B-31	82.5	SP	3	0.7
B-31	84.0	SP	3	0.7
B-31	85.5	SP	3	0.7
B-31	87.0	SP	2	0.7
B-31	88.5	SP	1	0.6
B-31	90.0	SP	0	0.6
B-31	91.5	SP	4	0.8
B-31	93.0	SP	3	0.7
B-31	94.5	SP	7	1.0
B-31	96.0	SP	0	0.6
B-31	97.5	SP	0	0.6
B-31	99.0	SP	1	0.6
B-31	103.5	SP-SM	7	1.0
B-31	109.5	SP-SC	5	0.8
B-31	118.5	SP-SM	0	0.6
B-31	120.0	SP-SM	0	0.6
B-31	121.5	SP-SM	0	0.6
B-31	123.0	SP-SM	0	0.6
B-31	124.5	SP-SM	0	0.6
B-31	126.0	SP-SM	0	0.6

LNP COL 2.5-9

Table 2.5.4.8-203B (Sheet 3 of 3)
Summary of Soil Layers Susceptible to Liquefaction in LNP 2 Site For 10^{-5} UHRS

Borehole	Bottom Depth of SPT Sample (ft.) [(a)]	Soil Type, [(c), (d), (e) (f), (g)]	Field SPT N-Value (BPF) [(b)]	Factor of Safety (FS)
B-31	127.5	SP-SM, ML	0	0.9
B-31	129.0	SP-SM	0	0.6
B-31	130.5	SP-SM	0	0.6
B-33	28.5	SP	4	0.9
B-33	30.0	SP	5	1.0
B-33	31.5	SP	3	0.8
B-33	33.0	SP	2	0.7
B-33	34.5	SP	2	0.7
B-33	36.0	SP	1	0.6
B-33	37.5	SP	2	0.7
B-33	39.0	SP	2	0.7
B-33	40.5	SP	2	0.7
B-33	42.0	SP	1	0.6
B-33	43.5	SP	0	0.6
B-33	45.0	SP	0	0.6
B-33	46.5	SP	0	0.6
B-33	58.5	SP	5	1.0
B-33	66.0	SP	7	1.0

Notes:

a) Depth of SPT sample is relative to original site grade at approximately El 41-43 ft. NAVD88
b) BPF = Blows per Foot
c) SC = Clayey Sand
d) SM = Silty Sand
e) SP = Poorly Graded Sand
f) NR = Not Recorded
g) ML = Silt with Sand

2.5.4.9 Earthquake Site Characteristics

LNP COL 2.5-2 The methods used to calculate site amplification at the GMRS elevation (top of competent layer) are presented in FSAR Subsection 2.5.2.5. Methods for calculation of the LNP site GMRS and FIRS are presented in FSAR Subsection 2.5.2.6. The site amplification functions for LNP 1 and LNP 2 were enveloped to calculate the LNP site GMRS.

The horizontal and vertical LNP site GMRS are presented on Figure 2.5.2-296.

2.5.4.10 Static Stability

LNP COL 2.5-10 The static stability of the LNP 1 and LNP 2 nuclear islands was evaluated for foundation bearing capacity, sliding, foundation settlement, and lateral pressures against below-grade walls. These evaluations are presented in FSAR Subsections 2.5.4.10.1, 2.5.4.10.2, 2.5.4.10.3, and 2.5.4.10.4, respectively. As described in FSAR Subsection 2.5.4.5.3, suitable foundation material is present at LNP 1 and LNP 2 nuclear islands subgrade elevation of -7.3 m (-24 ft.) NAVD88. Infilling and voids associated with joints, fractures, and bedding planes have been conservatively modeled in these evaluations. The source and derivation of the subsurface materials engineering properties used in these evaluations are described in FSAR Subsection 2.5.4.2.

2.5.4.10.1 Bearing Capacity

The bearing capacities at the LNP nuclear island subgrades under static and dynamic loading conditions have been evaluated as presented in this subsection. The resulting bearing capacities exceed the demand for the AP1000 nuclear islands, as listed in the DCD, and therefore satisfy safety requirements. A conservative method was used in this analysis, and appropriate FS values for static and dynamic loading conditions were considered, as summarized in FSAR Subsection 2.5.4.10.1.3.

2.5.4.10.1.1 Bearing Capacity Analysis Methodology

Rock mass properties and compressive strength values from the North and South Reactor Avon Park Formation Profiles were used to calculate the bearing capacity of the RCC and subsurface limestone formation. These rock profiles included the lower-strength zones located below elevation -180 ft. NAVD88 for LNP 1 and below elevation -150 ft. NAVD88 for LNP 2. Bearing capacity results were compared with the static and dynamic allowable load bearing pressures.

The subsurface at LNP consists of limestone formations that extend to a depth of more than 450 ft. below plant grade, beneath about 67 ft. of undifferentiated Quaternary and Tertiary sediments. Beneath the nuclear island basemat, the undifferentiated sediments will be replaced by a

35-ft.-thick RCC Bridging Mat. Beneath the RCC, 75 ft. of limestone will be grouted for dewatering purposes.

A nominal rock profile was developed which considered plant site-specific rock properties.

The bearing capacity of the RCC Bridging Mat was calculated using the ACI 318-89 (Reference 2.5.4.10-201) permissible service load stresses on concrete. The bearing capacity of the subsurface limestone formation was calculated using two different methods: a simplified American Association of State Highway and Transportation Officials (AASHTO) formulation for footings on broken or jointed rock; and the U.S. Army Corps of Engineers formulation for two different failure modes of rock subsurface, considering both static and dynamic loads.

The shear strength of the subsurface limestone formation, based on the rock mass strength parameters (cohesion and friction angle) was compared to the shear stresses calculated with a Finite Element Model.

The factors of safety comparing the bearing capacity of the RCC with the subsurface limestone formation were calculated using static and dynamic allowable bearing pressures.

The gross bearing pressures to be imposed on the RCC are 0.43 MPa (8.9 kips per square foot [ksf]) for static loading and 1.15 MPa (24.0 ksf) for dynamic loading. The dynamic allowable bearing pressure corresponds to the maximum subgrade pressure at the basemat that results from a time-history analysis on soft rock. For the subsurface rock bearing capacity calculations, the RCC self weight was included as an additional bearing pressure load of 5.16 ksf. The buoyancy effects due to the hydrostatic pressure acting at the bottom of the RCC were considered in this analysis. For conservative buoyancy effects, the water table was considered to be at elevation 38 ft. NAVD88.

The compressive strength of the RCC was considered to be 2500 psi, which is considered to occur after one year of the concrete placement.

The dynamic forces and moments at the basemat that were used in this analysis to estimate the dynamic eccentricities of the North and South Reactors correspond to the maximum seismic reactions at the center line of the Containment Building that result from a time-history analysis.

The factors of safety for static and dynamic loading of the RCC are above the minimum requirements, and the RCC bearing capacity is adequate to accommodate the static and dynamic pressures that were considered in this analysis. The estimated factors of safety resulted in 12.1 for static loading and 4.5 for dynamic loading. The calculated factors of safety are significantly larger than the acceptable factors of safety of 3.0 for static loading and 2.0 for dynamic loading.

The incremental shear stresses induced at or below elevation -150 ft. NAVD88 (where a lower-strength zone exists) were found to be less than 2 psi (less than 25 percent of the incremental shear stress induced at the nuclear island basemat). For this reason, characterization of the subsurface conditions below elevation -150 ft. NAVD88 was determined to be adequate.

2.5.4.10.1.1.1 Allowable Bearing Stresses

The allowable bearing stresses in concrete on a loaded area shall not exceed the following value under both static and dynamic loading conditions, as shown in Equation 2.5.4.10-201 (Reference 2.5.4.10-201):

$$B_c \leq 0.3 f_c^{'} \qquad \text{Equation 2.5.4.10-201}$$

In Equation 2.5.4.10-201, B_c is the allowable bearing capacity and f'_c is the concrete compressive strength.

The corresponding static and dynamic factors of safety were determined by dividing the ultimate bearing capacity by the bearing pressures used in this analysis,

$$FS = B_c/q \qquad \text{Equation 2.5.4.10-202}$$

where q is the AP1000 bearing demand.

Appropriate FS under static and dynamic loading are discussed in FSAR Subsection 2.5.4.10.1.2.

2.5.4.10.1.2 Bearing Capacity Results and Design Criteria

Table 2.5.4.10-201 presents the bearing capacities calculated using the ACI 318-89 criteria for allowable bearing stresses in concrete described in FSAR Subsection 2.5.4.10.1.1.1. The resulting FS based on the design static load of 0.43 MPa (8.9 ksf) and design dynamic load of 1.15 MPa (24 ksf) are also presented for each result.

Minimum FS of 3.0 for static loads (dead plus live loads) and 2.0 for dynamic or seismic loads are commonly considered acceptable (Reference 2.5.4.10-202). As shown in Table 2.5.4.10-201, these minimum FS are satisfied by each of the presented cases for LNP 1 and LNP 2.

2.5.4.10.1.3 Bearing Capacity of Adjacent Buildings

The LNP 1 and LNP 2 Annex Buildings (seismic Category II structures) will be founded on deep foundations (4000-psi concrete drilled shafts) that are socketed into the Avon Park Formation limestone. The Turbine Buildings, Radwaste

Buildings, and Diesel Generator Buildings will be founded on similar deep foundations (4000-psi concrete drilled shafts). Socket design and shaft spacing will be finalized with formal AP1000 building foundation bearing loads and pressures, including appropriate provisions for resistance to liquefaction. Prior to the construction of each drilled shaft, a pilot hole will be drilled to verify the capacity of the rock to resist the imposed loads.

2.5.4.10.2 Resistance to Sliding

LNP COL 2.5-6

The LNP 1 and LNP 2 nuclear islands will each be founded on a roller compacted concrete bridging mat, which will be founded on suitable rock. During excavation, loose material at the subgrade elevation will be removed, resulting in a relatively clean, exposed layer of rock, as discussed in FSAR Subsection 2.5.4.5.3. The RCC will interlock with the rock subgrade, and the concrete mudmat and nuclear island foundation will be placed over the RCC fill. While the RCC will adhere to the rock subgrade, the adhesion of the RCC to the rock subgrade is conservatively ignored when addressing sliding stability. Friction alone, between the rock and the RCC, will be capable of resisting sliding, as concrete on rock generally has a friction angle in the range of 48 to 60 degrees.

The weakest interface beneath the nuclear island foundation will be the lift joints within the RCC, when no bedding mix is used. Direct shear testing will be conducted prior to construction of the RCC bridging mat to ensure an adequate friction angle. On large-scale RCC projects, 42-degree friction angles are typically achieved, which would exceed the 35-degree requirement set forth by the DCD.

As described in FSAR Subsection 2.5.4.5.4, the space between the diaphragm wall and the nuclear island sidewall will be filled with concrete fill.

2.5.4.10.3 Settlement

LNP COL 2.5-12
LNP COL 2.5-16

The LNP nuclear islands will be founded on a roller compacted concrete bridging mat, which will be founded on suitable rock. As described in this subsection, elastic settlement of the rock under foundation loads is proportional to the elastic modulus of the rock mass, and the total settlements and differential settlements computed for the LNP 1 and LNP 2 nuclear islands are small and within tolerable limits. In light of the small total settlements calculated (less than 0.8 cm [0.3 in.]), any recompression settlement or heave is regarded as negligible.

2.5.4.10.3.1 Elastic (Total) Settlement under Foundation Loads

The elastic settlements of the subsurface, due to the weight of the RCC and the total construction loads applied to the nuclear island, were calculated.

The subsurface at LNP consists of limestone formations that extend to a depth of more than 450 ft. below plant grade, beneath about 67 ft. of undifferentiated Quaternary and Tertiary sediments. Beneath the nuclear island basemat, the undifferentiated sediments will be replaced by a 35-ft.-thick RCC Bridging Mat. The upper 75 ft. of limestone will be grouted for dewatering purposes.

Nominal rock profiles were developed for both the North and South Plant Units using LNP site-specific rock properties and layering information. These rock profiles included the lower-strength zones located below elevation -180 ft. NAVD88 for LNP 1 and below elevation -150 ft. NAVD88 for LNP 2. A SAP2000 elastic Finite Element Model of the RCC, nuclear island basemat, and the subsurface rock was developed using the design geometry, the rock profile configuration beneath the RCC, and the total loads applied on the nuclear island. The method that was used to determine the rock mass elastic modulus was based on shear-wave velocity measurements (Reference 2.5.4.10-203).

Three different methods were used to calculate the elastic settlements under static loading beneath the nuclear island basemat and beneath the RCC:

- Finite Element Model
- AASHTO 2002
- Elastic Theory

For the first method, a 3-D Elastic Finite Element Model (FEM) using solid elements was developed using SAP2000 verified and validated software. Settlements of the RCC Bridging Mat were calculated using the FEM. Two cases were analyzed: Case A: Settlements correspond to elevation -24 ft. NAVD88 (bottom of RCC); and Case B: Settlements correspond to elevation 11 ft. NAVD88 (top of RCC). This model included the in-place rock mass properties beneath the RCC bridging mat down to elevation -139.6 m (-458 ft.) NAVD88. The average settlements at the nuclear island basemat and the bottom of the RCC are presented in Table 2.5.4.10-202.

The elastic settlement results of the FEM Case A were compared with the results from two analytical procedures.

- Elastic settlement calculation using the subgrade modulus at three different locations: center, border midpoint, and corner of the RCC Bridging Mat.
- The elasticity deformation theory, considering a constrained rock mass elastic modulus and the Boussinesq solution for vertical stress distribution.

Subgrade Modulus is the ratio of bearing pressure (psf) over the settlement (ft.) (Reference 2.5.4.10-204). Subgrade modulus values for LNP 1 and LNP 2 are reported in FSAR Subsection 2.5.4.10.3.4. The elastic settlements can be calculated by using the following expression:

$$\delta = q / K_s \qquad \text{Equation 2.5.4.10-203}$$

In Equation 2.5.4.10-203, δ is the elastic settlement, q is the bearing pressure and considered as $q = q_{NI} + q_{RCC}$, and K_s is the subgrade modulus. q_{NI} is the nuclear island construction loads and q_{RCC} is the load due to the RCC selfweight. The RCC area is considered to be an equivalent rectangle. Using the Equation 2.5.4.10-203, elastic settlements were calculated at three points (center [internal], midpoint [south] and corner [north]) of the RCC.

In the third method, the relationship between the settlement of a rock interval, the stress increase, and the elastic modulus is based on simple elastic theory (Reference 2.5.4.10-205), as presented in Equation 2.5.4.10-204:

$$\Delta S = \sum_i H_i \Delta\sigma_i \big/ M_i \qquad \text{Equation 2.5.4.10-204}$$

In Equation 2.5.4.10-204, ΔS is the total elastic settlement for all rock layers below the foundation, H_i is the thickness of the i^{th} layer, M_i is the constrained modulus (related to the elastic modulus) of the i^{th} layer, and $\Delta\sigma_i$ is the change in vertical stress at the i^{th} layer due to foundation loading. The total elastic settlement of all layers within the depth of influence below the foundation is summed to calculate the overall foundation settlement.

The resulting elastic foundation settlements under static loading using the three methods presented above are small, as listed in Table 2.5.4.10-202. These settlements would occur as the nuclear island facilities are constructed. No additional elastic settlements would occur after construction, when foundation loading is constant.

The average settlements predicted by the FEM analysis were in agreement with the results of the two alternative analytical procedures. For the FEM analysis, the average settlement at elevation -24 ft. NAVD88 (bottom of RCC) resulted in approximately 0.2 inches at both the North and South Reactors.

The differences in settlements predicted by the FEM and by the analytical methods are negligible. The analytical equations consistently lead to slightly lower settlement values.

In Case B of the FEM analysis, settlement results at elevation 11 ft. NAVD88 (top of RCC) are reported in order to assess RCC deformation due to the applied loads. The average difference between values at this elevation and at elevation -24 ft. NAVD88 is approximately 0.01 inches.

Given the small incremental shear stresses being induced below elevation -150 ft. NAVD88, as well as the small predicted settlement values, the characterization of the subsurface below elevation -150 ft. NAVD88 (approximately 200 ft. below final plant grade) performed was determined to be adequate.

Elastic settlements calculated by the first method (Finite Element Model) are considered the "best estimates" of settlement, as the Finite Element Model best accounts for the distribution of the stresses, and all of the stresses are relatively low (in the elastic range).

The total settlements listed in Table 2.5.4.10-202 are within the range of acceptable settlement limits for the AP1000.

2.5.4.10.3.2 Differential Settlement

The potential differential settlement across the nuclear island basemats is calculated using the 3-D finite element model. The maximum settlement is shown to occur in the middle of the nuclear island. Based on conservative estimates of total settlements, the slope associated with this settlement is expected to be less than 0.00083 (or 1:1200), which is within the acceptable range for the AP1000 under both LNP 1 and LNP 2 as defined in FSAR Subsection 2.5.4.10.3.3.

Adjacent nonsafety-related structures will be founded on deep foundations (4000-psi concrete drilled shafts) that are socketed into the Avon Park Formation. While foundation bearing loads and pressures for AP1000 structures are not yet finalized, conservative settlement analyses indicate that these structures will exhibit very little total settlement (less than 25 mm [1 in.]), and therefore, any potential for differential settlement is negligible. The results of the differential settlement analysis are presented in Table 2.5.4.10-203. Once AP1000 foundation bearing loads and pressures for structures adjacent to the nuclear island are finalized, a detailed analysis of differential settlements between the nuclear islands and adjacent structures will be performed, which will account for differential settlement of the nuclear island.

2.5.4.10.3.3 Design Criteria for Foundation Settlement

The following design criteria are tolerable values for the AP1000 nuclear island, as listed in Table 2.5-1 of the DCD and Revision 1 of TR85 (Reference 2.5.4.10-209):

- Total settlement of the nuclear island foundation mat: up to 76 mm (3 in.).
- Differential settlement across the nuclear island foundation mat: up to 13 mm (0.5 in.) per 15.2 m (50 ft.) (slope of 1:1200).
- Differential settlement between nuclear island and adjacent structures: up to 76 mm (3 in.).

As discussed in FSAR Subsections 2.5.4.10.1, 2.5.4.10.2, and 2.5.4.10.3, the engineering analyses indicate that these design criteria will be satisfied at LNP 1 and LNP 2. Conservative methods of settlement analyses and design parameters were used, as described in those subsections.

2.5.4.10.3.4 Subgrade Modulus

The subgrade modulus (K_s) for the LNP nuclear islands is given as:

$$K_s = q / \delta \qquad \text{Equation 2.5.4.10-205}$$

where q is the bearing pressure (psf) and δ is the elastic settlement of the mat (ft.).

The relationship between bearing pressure and the elastic settlement of mat foundation is defined by Equation 2.5.4.10-206 (Reference 2.5.4.10-205):

$$K_s = q / \delta = 1 / (B' (1- \mu_{av}^2) I_s I_f / E_{rm\ av}) \qquad \text{Equation 2.5.4.10-206}$$

where B' is the least lateral dimension contributing basemat area, and I_s and I_f are the influence factors from chart solutions of Steinbrenner equations for deformation under a rectangular elastic half space. $E_{rm\ av}$ is the weighted average rock mass modulus of the subsurface and μ_{av} is the weighted average Poisson's ratio. In order to calculate K_s, terms in Equation 2.5.4.10-206 are determined as follows:

1. The subgrade modulus is a function of the soil parameters and foundation dimensions. These geometrical parameters are used to calculate the influence factors. K_s values are determined at the center, at the corner, and the edge midpoints of the RCC mat equivalent rectangle. A compressible rock thickness of 3B was considered to determine K_s.

2. The influence factors I_s and I_f were determined. The influence factor, I_s, was given with the following expression from Bowles (Reference 2.5.4.10-205).

   $$I_s = I_1 + ((1-2\mu) / (1-\mu)) I_2 \qquad \text{Equation 2.5.4.10-207}$$

   All I_1 and I_2 values are shown in Bowles (Reference 2.5.4.10-205).

3. Young's modulus (E_{max}) and rock mass modulus (E_{rm}) are used in the evaluation of the subgrade modulus. Young's modulus values were determined from shear-wave velocity measurements from suspension loggings. The E_{rm} for each rock layer was calculated by reducing E_{max} by 50 percent. This reduction reflects the strain degradation effects recommended by Mayne et al. (Reference 2.5.4.10-203) and is appropriate for these subgrade modulus calculations.

4. The effect of horizontal layering beneath the RCC mat is assessed in principle by taking a weighted average of the elastic modulus of each layer, and taking into account the influence of the distribution of the stresses beneath the foundation. The stress distribution for the layered system is considered to be the same as that for a homogeneous half space. It is further considered that the contribution to the stiffness of the composite system made by an individual layer is directly proportional to the strain energy contained in that layer. Based on this principle, the equivalent elastic modulus for the layered system is evaluated as the weighted average of the elastic modulus of each layer in accordance with the strain energy in the layer.

   The weighted averages of the E_{rm} and μ_{av} values were computed in order to include the variations in the soil profile along the influence depth of 3B. The weighted averages of E_{rm} were calculated by using the following expression:

$$E_{rm\ av} = \frac{E_{rm1}L_1 + E_{rm2}L_2 + \cdots\cdots + E_{rmn}L_n}{H} \qquad \text{Equation 2.5.4.10-208}$$

   where E_{rmn} and L_n represent rock mass modulus value and depth of each rock layer. H is the depth of influence. $E_{rm\ av}$ values were computed for both reactors.

   Similar to E_{rm} weighted average calculation, weighted average Poisson's ratios (μ_{av}) were calculated with the same approach. The results of the weighted average rock mass modulus and Poisson's ratio computations are shown in Table 2.5.4.10-204.

5. Terzaghi and Peck (Reference 2.5.4.10-208) suggested to determine the distribution of the K_S (i.e., at the center and at the corner of basemat) if the load distribution is not uniform on the basemat. With this in mind, the nuclear island loads are higher under the Containment Building and lower around the edges (i.e., not uniform). Therefore, K_S values at four locations under the basemat are calculated by using Equation 2.5.4.10-206. In order to determine the K_S for the center of the RCC mat, by following the principle of super-position, the area is divided into four sections and 4q is used in Equation 2.5.4.10-206 to account for four contributing corners. Similarly, K_S values at Point B and Point D (midpoints of the edges) were determined by following the principle of super-position, where the area is divided into two sections and 2q is used in Equation 2.5.4.10-206 to account for two contributing corners. The K_S value at the corner was calculated by considering the equivalent rectangle as one contributing area.

6. The average subgrade moduli for each unit were calculated by including effect (weight) of subgrade modulus under each location, as explained by

Bowles (Reference 2.5.4.10-205). The weighted average subgrade modulus was calculated as follows:

$$K_{s\ av} = (4x\ K_{s\ center} + K_{s\ corner})\ /\ 5 \qquad \text{Equation 2.5.4.10-209}$$

The subgrade modulus for each reactor is also presented in Table 2.5.4.10-204.

2.5.4.10.3.5 Subsurface Instrumentation

LNP COL 2.5-13 Settlement of the nuclear island will be monitored throughout construction. A detailed settlement monitoring program will be developed prior to construction.

As presented in FSAR Subsection 2.5.4.10.3.3, nuclear island foundation settlements on the sound rock subgrade are expected to be small. The settlement monitoring program will be implemented to monitor settlement and heave with two primary elements: water pressure monitoring and settlement (heave) monitoring.

With respect to water pressures, the following activities are planned:

- Monitoring the head outside the perimeter of the diaphragm wall with 10 piezometers (open standpipes) installed to elevation -24ft. NAVD88.
- Monitoring the head with piezometers (a) within the excavation at elevation 0 ft. NAVD88 (~2/3 depth of excavation) with 6 piezometers (b) at elevation -29 ft. NAVD88 (5 ft. below the bottom of the excavation) with 6 piezometers and (c) at elevation -99 ft. NAVD88 (immediately below the grouted zone) with 3 piezometers.
- Settlement monuments, currently expected to be telltales at elevation -24 ft. NAVD88 to monitor heave and settlement as the excavation proceeds.

Settlement monuments will likely be installed and monitored throughout the construction process as follows:

- Settlement bench marks will be installed within the subgrade mudmat (at approximate elevation 3.4 m [11 ft.] NAVD88) at the four corners of each nuclear island and at the (plant) northernmost point of each Containment Building. These will be monitored before and periodically during construction of the nuclear island basemats and sidewalls prior to placement of backfill materials.
- Additional bench marks will be installed approximately 1 m (3 ft.) above site grade (at approximate elevation 16.5 m [54 ft.] NAVD88) and connected to the sidewalls of the nuclear island, directly above the deeper bench mark locations described previously. These bench marks will be monitored during

backfilling operations and, periodically, during and after construction of the nuclear island structures.

Settlement bench marks will be installed approximately 1 m (3 ft.) above site grade (at approximate elevation 16.5 m (54 ft.] NAVD88) on the turbine buildings, annex buildings, and radwaste buildings, at the corners of these buildings that abut the nuclear islands. These bench marks, used to measure the differential settlement between the nuclear islands and the adjacent buildings, will be monitored during and after the construction of the nuclear island and adjacent structures.

Monitoring will be continued until at least 90 percent of expected settlement has occurred or the rate of settlement has virtually stopped. This will be evaluated by review of the settlement versus time curves at the bench mark locations.

A monitoring program will be implemented after construction to monitor any long-term settlement. While long-term settlement is expected to be minimal, the settlement bench marks installed during the construction phase (connected to the sidewalls of the nuclear islands) will be used post-construction to monitor settlement of the nuclear island structures.

2.5.4.10.4 Lateral Earth Pressures

LNP COL 2.5-7
LNP COL 2.5-11

Lateral earth pressures will develop against below-grade nuclear island sidewalls due to placement of concrete fill in the annular space between the diaphragm wall and the nuclear island sidewall, in addition to the soil backfill materials above the diaphragm wall. The earth pressure calculation considers the pressure imposed during construction and the long-term condition when construction has been fully completed. The pressure on the nuclear island wall is calculated as the maximum value at any elevation either during construction or operation. For the case during construction, the pressure on the nuclear island wall is calculated, including hydrostatic pressure, crane loads, a 3 m (10 ft.) lift of wet concrete fill at any elevation, and the compaction equipment (tamper) used for construction.

The following subsections describe the basic design input and calculation methodology for the lateral earth pressure calculation.

2.5.4.10.4.1 Design Input

The following loads were applied in the lateral pressure determination:

- Live Load on the ground surface is 250 psf.
- Crane Surcharge Load at a distance of 15 ft. from the wall.
- Water Table at elevation 43 ft. NAVD88.

- Pseudo static earthquake load coefficient is 0.1g.
- Density of concrete fill is 150 pcf.
- Moist/Saturated density of natural soil/compacted granular backfill is 125 pcf.
- Shear strength of natural soil/compacted granular backfill (ϕ') is 34 degrees, c is 0 psf.

The backfill adjacent to the nuclear island sidewalls will be placed as described in FSAR Subsection 2.5.4.5.4. In addition, light, hand-operated compaction equipment will be used to compact the soil adjacent to the nuclear island sidewalls. This will render compaction-induced soil stresses against the sidewalls to be small at the ground surface, decreasing to insignificant with depth.

2.5.4.10.4.2 Methodology

The relationship between each material and the corresponding lateral pressure is defined as follows (Reference 2.5.4.10-206):

- Lateral hydrostatic pressure coefficient for plastic concrete fill (K) = 1.
- Lateral pressure coefficient for hardened concrete fill (k) =$\mu/(1- \mu)$.
- The at-rest earth pressure coefficient (Ko) for natural soil/compacted granular backfill.

$$K_o = 1\text{-}\sin(\phi'); = 0.44 \qquad \text{Equation 2.5.4.10-210}$$

The lateral pressure, P against the nuclear island sidewalls at any depth is calculated as follows:

$$P = \sigma'_v{}^*K_L + P_h + P_c + P_s{}^*K_o + P_{Eq} \qquad \text{Equation 2.5.4.10-211}$$

Where σ'_v is the effective overburden pressure at the depth z, P_h is the groundwater pressure, P_c is pressure due to crane loading, P_s is the earth pressure due to the surface surcharge, and P_{Eq} is due to earthquake loading, and other terms are as defined previously.

The lateral earth pressure coefficient (K_L) could be due to either plastic concrete lift (K), hardened concrete (k), or earth pressure at rest.

The lateral earthquake load includes seismic lateral earth pressure for at-rest conditions and hydrodynamic water thrust. The seismic at-rest pressure is calculated from the Woods' method and hydrodynamic pressure is calculated from the Westergaard method (Reference 2.5.4.10-207).

- Hydrostatic pressures and hydrodynamic water thrust will act against the sidewalls during seismic loading conditions.

- Structures adjacent to the nuclear islands can potentially increase the at-rest pressures that develop against the nuclear island sidewalls. However, these adjacent structures will be founded on drilled piers socketed into sound rock, which is much stiffer than the soil adjacent to nuclear islands. Due to this difference in rock and soil stiffness, it is anticipated that these adjacent structure foundation loads will not be transferred to the soil. Therefore, loads from structures adjacent to nuclear islands were considered insignificant in the calculation of the at-rest pressure distributions.

- Surface surcharges from live loads, and lateral loads for the crane, can potentially increase the at-rest pressures that develop against the nuclear island sidewalls; these are added to the static and earthquake lateral loads.

The resulting at-rest lateral pressure profiles for the soil backfill, concrete fill, and natural soil are presented for representative sidewall elevations in Table 2.5.4.10-205.

LNP COL 2.5-10

Table 2.5.4.10-201
Summary of Bearing Capacity Analyses at Nuclear Islands – Static and Dynamic Loading

Bearing Capacity Analysis Results

Loading Conditions for Analyses		Concrete Allowable Stresses Method (ACI-318-89)	
Unit	**Load Condition**	**Bearing Capacity (ksf)**	**Factor of Safety[a]**
LNP 1	Static	108	12.1
LNP 1	Dynamic	108	4.5
LNP 2	Static	108	12.1
LNP 2	Dynamic	108	4.5

Notes:

a) Factor of safety for static and dynamic load conditions are calculated as ultimate bearing capacity divided by 8.9 ksf and 24 ksf, respectively.

ksf = kips per square foot

LNP COL 2.5-16

Table 2.5.4.10-202
Elastic Settlement under Nuclear Islands

Location	Elastic Settlements Based on (in.)		
	FEM SAP2000	Subgrade Modulus	Elastic Theory
LNP 1 - West Side	0.2	-	-
LNP 1 - Internal	0.2	0.3	-
LNP 1 - North Side	0.1	0.1	-
LNP 1 - South Side	0.1	0.1	-
LNP 1 - East Side	0.2	-	-
LNP 1 Average	0.2	0.2	0.2
LNP 2 - West Side	0.2	-	-
LNP 2 - Internal	0.3	0.3	-
LNP 2 - North Side	0.1	0.1	-
LNP 2 - South Side	0.1	0.1	-
LNP 2 - East Side	0.2	-	-
LNP 2 Average	0.2	0.2	0.2

LNP COL 2.5-16

Table 2.5.4.10-203
Differential Settlement under Nuclear Islands

Differential Settlement Location			Total Settlement at 1st Point (in.)	Total Settlement at 2nd Point (in.)	Distance Between Boreholes	Range in Differential Settlement (Slope) [(a)]
First Point	Second Point	Description	Best Estimate	Best Estimate	(ft.)	Best Estimate
Based on Settlement Results at Specific Points:						
4	8	LNP 2, West-East	0.2	0.2	174	0.0000045
2	6	LNP 2, North-South	0.1	0.1	268	0.0000031
4	8	LNP 1, West-East	0.2	0.2	174	0.0000023
2	6	LNP 1, North-South	0.1	0.1	268	0.0000026
Based on Maximum Differential Settlements:						
6	9	LNP 2	0.1	0.3	130	0.000085
6	9	LNP 1	0.1	0.2	130	0.000074

Notes:

The results correspond to the FEM analysis.

a) The differential settlement (slope) is defined as the difference in total settlement at two locations divided by the horizontal distance between those two locations (based on estimated settlements to third decimal place).

in. = inch, ft. = foot,

LNP COL 2.5-7

Table 2.5.4.10-204
Subgrade Modulus Based on Seismic Wave Velocity

Reactor	Weighted Average Rock Mass Modulus (ksf)	Weighted Average Poisson's Ratio	Location[a]	Subgrade Modulus, K_S (kcf) [b]
LNP-1	7.94E+05	0.39	Center	610
			Corner	1630
			Midpoint B	1220
			Midpoint D	850
			Average	814
LNP-2	8.41E+05	0.39	Center	587
			Corner	1568
			Midpoint B	1174
			Midpoint D	818
			Average	783

Notes:

a) Subgrade Modulus is calculated for center and corners of the basemat.
b) A compressible rock thickness of 3B (where B is width of basemat) was considered to determine subgrade modulus.

kcf = kilopound per cubic foot
ksf = kilopound per square foot

LNP COL 2.5-11

Table 2.5.4.10-205
Lateral Earth Pressures on Nuclear Island Sidewalls

Elevation (ft. NAVD88)	Lateral Earth Pressure (ksf)			
	Case 1[(a)]	Case 2[(b)]	Case 3[(c)]	Case 4[(d)]
51	0.61	0.61	0	0.61
43	1.26	1.37	1.17	0.95
33	2.38	2.20	2.67	1.70
11	3.74	3.56	1.99	3.52

Notes:

a) In Case 1, the lateral earth pressures due to 8 ft. of live load (250 psf), crane load, hydrostatic load, and earthquake load are evaluated.
b) In Case 2, the lateral earth pressures due to failure of the two rows of anchors supporting the diaphragm wall are evaluated.
c) In Case 3, the lateral earth pressures during the concrete fill placement are evaluated.
d) In Case 4, the lateral earth pressures induced by post construction loads (8 ft. backfill, 32 ft. concrete fill, hydrostatic, live load, and earthquake loads) are evaluated.

ft. NAVD88 = feet North American Vertical Datum 1988
ksf = kilopound per square foot

2.5.4.11 Design Criteria

LNP COL 2.5-3

This subsection summarizes the design criteria and methods used in the stability evaluations for safety-related structures, including factors of safety, assumptions, and conservatism used in the analyses. Cross references to subsections where these items are described are provided.

FSAR Table 2.0-201 compares the DCD site geotechnical parameter criteria with the corresponding site characteristics at LNP 1 and LNP 2, including the following items:

- Average Allowable Static Bearing Capacity.
- Maximum Allowable Dynamic Bearing Capacity for Normal plus SSE.
- Shear-Wave Velocity.
- Lateral Variability.
- Liquefaction Potential.

LNP COL 2.5-11

Design criteria and methods used in the evaluations of safety-related structures are found in the following subsections:

- Criteria for selection of borehole locations and depths are presented in FSAR Subsections 2.5.4.2.1.1.1 and 2.5.4.2.1.1.2, respectively.
- Criteria for selection of soil samples and rock core for laboratory testing are presented in FSAR Subsections 2.5.4.2.1.5.2 and 2.5.4.2.1.5.3, respectively.
- Criteria for selection of rock and soil properties used in the engineering analyses are presented in FSAR Subsection 2.5.4.2.4.
- Criteria for selection of geophysical survey results as design parameters are presented in FSAR Subsection 2.5.4.4.2.8.
- Criteria for evaluation of nuclear island subgrade conditions and identification of the need for subgrade improvement are presented in FSAR Subsection 2.5.4.5.3.
- Criteria for groundwater elevations are presented in FSAR Subsection 2.5.4.6.1. Selection of construction dewatering methods is presented in FSAR Subsection 2.5.4.6.2.

- Criteria for determination of nuclear island allowable bearing pressures, including analysis methods and selection of conservative rock strength parameters, are presented in FSAR Subsection 2.5.4.10.1. Selection of static and dynamic factors of safety is presented in FSAR Subsection 2.5.4.10.1.

- Criteria for determination of nuclear island settlement and subgrade rebound, including analysis methods and selection of conservative rock and soil parameters, are presented in FSAR Subsection 2.5.4.10.3. Tolerable settlement limits are presented in FSAR Subsection 2.5.4.10.3.3.

- Criteria for estimation of nuclear island sidewall lateral earth pressures are presented in FSAR Subsection 2.5.4.10.4.

For engineering analyses supporting the design and evaluation of safety-related structures, each software package used was validated and verified to operate properly on the computers used for the analyses in accordance with the Paul C. Rizzo Associates, Inc., Quality Assurance program. Specific software packages used for these analyses are described in the above-referenced design criteria subsections.

2.5.4.12 Techniques to Improve Subsurface Conditions

LNP COL 2.5-7 Major structures will derive support from the Avon Park Formation, at elevation -7.3 m (-24 ft.) NAVD88. Prior to excavation, grouting will be performed between this foundation elevation and elevation -30.2 m (-99 ft.) NAVD88 to create a relatively impervious zone of limestone to facilitate dewatering during construction.

Prior to the excavation of the nuclear island foundations, grout holes will be drilled from the existing ground surface to the proposed bottom of the targeted grout zone (elevation -32 m [-99 ft.] NAVD88). Grouting will be performed using a suite of mixes developed in the Grout Test Program. Primary grout holes will be spaced on a 4.8 m (16 ft.) hexagonal pattern, and split-spaced with secondary grout holes to achieve “no take” conditions. Provisions will be in place to perform additional split-spacing to tertiary grout holes, as dictated by the performance of the production grouting. This hole spacing was developed based on the results of the Grout Test Program conducted in early 2009. State-of-the-practice computerized monitoring of all grouting will take place, including the measurement of grout take in terms of pressure and volume.

Grouting will reduce the gross porosity and the gross permeability of the Avon Park Formation in this grouted zone. An additional benefit of this grouting is the long-term reduction of groundwater flow through the formation and the consequential reduction in the potential for renewed solution activity. This grouting program is not intended to strengthen the formation. However, the improved strength of the Avon Park Formation will add conservatism to the design. Grouting is nonsafety-related; however, it will be performed under a quality program.

Upon completion of the grout program and dewatering effort, the nuclear island foundations will be excavated to the interpreted top of the Avon Park Formation at elevation -7.3 m (-24 ft.) NAVD88. Sound rock is present at this elevation, which is capable of supporting the structures with surface repairs and dental concrete as necessary to level this erosional surface. Criteria for acceptable subgrade conditions are presented in FSAR Subsection 2.5.4.5.3. Rock that does not satisfy the criteria will be removed and replaced with concrete or grout.

Subsequent to the excavation described in FSAR Subsection 2.5.4.5.3, a RCC Bridging Mat will be constructed at elevation -24 ft. The mat will be installed in 1-ft. lifts to elevation 11 ft. The extent that the RCC placement is shown on Figure 2.5.4.5-201A and Figure 2.5.4.5-201B for LNP 1, and Figure 2.5.4.5-202A and Figure 2.5.4.5-202B for LNP 2.

The RCC will be placed in lift thicknesses of approximately 1 ft. Bedding Mix will be used over each entire lift surface for the RCC bridging mat construction. The Pre-COL RCC testing performed and the Post-COL RCC Testing planned is described in FSAR Subsection 3.8.5.11.

The specified density of RCC is in the range 143 to 153 pcf. During the construction of the RCC Bridging Mat, field measurements of RCC density will be performed using a “single-probe nuclear densometer” for each 1-ft. lift during placement of the RCC.

Verification laboratory tests will be performed to confirm that the compressive strength of the RCC is satisfactory. The tests will be conducted using six-inch cylindrical test specimens molded during construction, in accordance with ASTM C 1435/C 1434M-05: “Standard Practice for Molding Roller-Compacted Concrete in Cylinder Molds Using a Vibrating Hammer”. Concrete to make the test specimens will be taken from six different locations for each 1-ft. lift of the RCC. Three samples will be taken at each of the six locations. The compressive strength tests will be conducted within 1 year of placement of the RCC. Compressive strength testing will be performed in accordance with ASTM C 39 “Test Method for Compressive Strength of Cylindrical Concrete Specimens.” All laboratory testing will conform to NQA-1 quality requirements. The strength level of RCC, adjusted for aging, will be considered satisfactory if either conditions 1 and 2 or conditions 1 and 3 are satisfied:

1) The average of compressive strength from three cylinders molded at a location equals or exceeds f’c.

2) No individual strength test (average of two cylinders) falls below f’c by more than 500 psi.

3) If individual strength tests (average of two cylinders), adjusted for aging, fall below f’c by more than 500 psi, a minimum of three cores drilled from the area in question shall be tested. The cores shall be drilled in accordance with ASTM C42: “Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.” RCC in areas represented by core tests shall be considered adequate if the average of compressive strength from three cores is equal to at least 85 percent of f’c and if no individual core compressive strength is less than 75 percent of f’c.

If these acceptance criteria are not met, an evaluation of the acceptability of the RCC for its intended function shall be performed before acceptance.

A detailed excavation, subgrade improvement, and verification program will be developed prior to and during construction. Subgrade improvement and verification methods summarized in FSAR Subsections 2.5.4.5.3 and 3.8.5.11, or equivalent, will be included in this program. The operational monitoring program for LNP 1 and LNP 2 is described in FSAR Subsection 2.4.12.4.

2.5.4.12.1 Impact of Dissolution Rate

As discussed in FSAR Subsection 2.5.4.1.2.1.1.1, the current dissolution rate of the Avon Park Formation is insignificant with regards to the foundation design. The operation of LNP’s production wells, after full installation of the AP1000 basemat, RCC Bridging Mat, and grouted zone, was shown to have little significant impact on the groundwater regime of the site. Compared to the natural regime at the site, the LNP construction was shown to impact the hydrology approximately the same as the seasonal fluctuations. Given this and the very low expected dissolution rates described in FSAR Subsection 2.5.4.1.2.1.1.1, the potential for increased dissolution as a result of construction is also insignificant.

2.5.5 STABILITY OF SLOPES

LNP COL 2.5-14 The nominal plant grade floor elevation at the LNP site will be at 15.5 m (51 ft.) NAVD88, with minor variations to allow drainage for an area of about 370 m by 390 m (1210 ft. by 1280 ft.) around the nuclear island. No permanent slopes will be present at the site that could adversely affect safety-related structures.

LNP COL 2.5-15 The AP1000 does not utilize safety-related dams or embankments, and there are no existing upstream or downstream dams that could affect the LNP site safety-related facilities.

STD DEP 1.1-1

2.5.6 COMBINED LICENSE INFORMATION

2.5.6.1 Basic Geologic and Seismic Information

LNP COL 2.5-1 This COL item is addressed in FSAR Subsections 2.5.1 and 2.5.4.1.

2.5.6.2 Site Seismic and Tectonic Characteristics Information

LNP COL 2.5-2 This COL item is addressed in FSAR Subsections 2.5.2, 2.5.4.7, and 2.5.4.9.

2.5.6.3 Geoscience Parameters

LNP COL 2.5-3 This COL item is addressed in FSAR Subsections 2.5.2.6 and 2.5.4.11.

2.5.6.4 Surface Faulting

LNP COL 2.5-4 This COL item is addressed in FSAR Subsection 2.5.3.

2.5.6.5 Site and Structures

LNP COL 2.5-5 This COL item is addressed in FSAR Subsections 2.5.4.1 and 2.5.4.3.

2.5.6.6 Properties of Underlying Materials

LNP COL 2.5-6 This COL item is addressed in FSAR Subsections 2.5.4.2, 2.5.4.3, 2.5.4.4, 2.5.4.6, 2.5.4.7, and 2.5.4.10.2.

2.5.6.7 Excavation and Backfill

LNP COL 2.5-7 This COL item is addressed in FSAR Subsections 2.5.4.5 and 2.5.4.6.2.

2.5.6.8 Groundwater Conditions

LNP COL 2.5-8 This COL item is addressed in FSAR Subsection 2.5.4.6.

2.5.6.9 Liquefaction Potential

LNP COL 2.5-9 This COL item is addressed in FSAR Subsection 2.5.4.8.

2.5.6.10 Bearing Capacity

LNP COL 2.5-10 This COL item is addressed in FSAR Subsection 2.5.4.10.

2.5.6.11 Earth Pressures

LNP COL 2.5-11 This COL item is addressed in FSAR Subsections 2.5.4.10.4 and 2.5.4.11.

2.5.6.12 Static and Dynamic Stability of Facilities

LNP COL 2.5-12 This COL item is addressed in FSAR Subsection 2.5.4.10.3.

2.5.6.13 Subsurface Instrumentation

LNP COL 2.5-13 This COL item is addressed in FSAR Subsection 2.5.4.10.3.5.

2.5.6.14 Stability of Slopes

LNP COL 2.5-14 This COL item is addressed in FSAR Subsection 2.5.5.

2.5.6.15 Embankments and Dams

LNP COL 2.5-15 This COL item is addressed in FSAR Subsections 2.4.4 and 2.5.5.

2.5.6.16 Settlement of Nuclear Island

LNP COL 2.5-16 This COL item is addressed in FSAR Subsection 2.5.4.10.3.

2.5.6.17 Waterproofing System

LNP COL 2.5-17 This COL item is addressed in FSAR Subsection 14.3.3.2.

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APPENDIX 2AA
EARTHQUAKE CATALOG

LNP COL 2.5-1
LNP COL 2.5-2

The updated earthquake catalog prepared for the project constitutes this appendix. The development of this catalog is described in FSAR Subsection 2.5.2.1.1. This catalog was used to select the final catalog of earthquakes occurring within 320 km (200 mi.) of the LNP site.

The headings for the data in the table are described below:

Year – Year in Coordinated Universal Time (UTC)

Month – Month in Coordinated Universal Time (UTC)

Day – Day in Coordinated Universal Time (UTC)

Hour – Hour in Coordinated Universal Time (UTC)

Minute – Minute in Coordinated Universal Time (UTC)

Second – Second in Coordinated Universal Time (UTC)

Latitude – Latitude (North)

Longitude – Longitude (West negative)

Depth – hypocentral depth in km

m_b* – m_b adjusted to remove bias from computed earthquake recurrence rates

m_b – best estimate of earthquake body-wave magnitude

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 December, 2006).

EPRI Flag – earthquake dependency:

- MAIN, mainshock with dependent events.
- blank, mainshock with no associated dependent events.
- [number], EPRI UNID of mainshock.

R (km) – distance from LNP site in km

Event No. – Project assigned identification number

APPENDIX 2BB
GEOTECHNICAL BORING LOGS

LNP COL 2.5-1
LNP COL 2.5-5
LNP COL 2.5-6

This appendix contains geotechnical boring logs that are the basis for discussion in relevant sections of FSAR Section 2.5. The logs are of soil and rock borings and represent a record of subsurface conditions as performed as part of the LNP COLA field investigations.

The appendix contains the logs of 126 bore holes.

APPENDIX 2CC
SOIL AND ROCK LABORATORY TEST RESULTS

LNP COL 2.5-6 This appendix contains the results of the soil and rock laboratory tests as described in FSAR Subsection 2.5.4.2.3.